THE EFFECTS OF PARTIAL SHEAR CONNECTION ON COMPOSITE STEEL-CONCRETE BEAMS SUBJECTED TO COMBINED FLEXURE AND TORSION

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College of Health and Science
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Australia

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DEDICATIONS

To my parents and my partner
ACKNOWLEDGEMENTS

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Finally yet importantly, I would like to dedicate this thesis to Rachel Huang and my parents, Han Fing Tan and Ah Dian Gwee for their endless support, love and patience, which gave me the motivation to continue and complete this thesis.
STATEMENT OF AUTHENTICATION

I hereby declare that this submission is my own work and to the best of my knowledge it contains no material previously published or written neither by another person, nor material which to a substantial extent has been accepted for any other degree or diploma at University of Western Sydney or any other educational institution, except where due acknowledgement is made in this thesis. Any contribution made to the research by others, with whom I have worked at University of Western Sydney or elsewhere, is explicitly acknowledged in the thesis.

I also declare that the intellectual content of this thesis is the product of my own work, except to the extent that assistance from others in the project’s design and conception or in style, presentation and linguistic expression is acknowledged.

Signature: ........................................

Date: ........................................
PREFACE

This thesis is submitted in partial fulfillment of the requirements for the degree of Doctor of Philosophy at University of Western Sydney, Sydney, Australia. The work described herein was performed by the candidate in the College of Health and Science, School of Engineering, University of Western Sydney. The candidate was supervised by Professor Brian Uy during the period from April 2004 to August 2010. This thesis has been supported by papers that have been published, accepted or submitted for the consideration in internationally renowned journals and conferences, which are listed in the following:

JOURNAL PAPERS


# TABLE OF CONTENTS

## LIST OF CONTENTS

1. **INTRODUCTION**  
   1.1 Introduction  
   1.2 Background and motivation for research  
   1.3 Composite steel-concrete beams in sagging moment regions  
     - 1.3.1 Plastic neutral axis in the concrete slab, (FSC, $\beta = 1.0$)  
     - 1.3.2 Plastic neutral axis in the steel beam, (FSC, $\beta = 1.0$)  
     - 1.3.3 Partial shear connection, (PSC, $\beta < 1.0$)  
   1.4 Composite steel-concrete beams in hogging moment regions  
     - 1.4.1 Plastic neutral axis in the concrete slab, (FSC, $\beta = 1.0$)  
     - 1.4.2 Plastic neutral axis in the steel beam, (FSC, $\beta = 1.0$)  
     - 1.4.3 Partial shear connection, (PSC, $\beta < 1.0$)
1.5 Previous research 11
1.6 Research methodology 14
1.7 Thesis layout 16
1.8 Summary 19

Figures 20

2 LITERATURE REVIEW 27

2.1 Introduction 27
2.2 History of composite steel-concrete beams 28
2.3 Shear connector design rules 30
2.4 Partial shear connection 31
2.5 Push-out tests 35
2.6 Composite steel-concrete beams subjected to combined flexure and shear 38
2.7 Composite steel-concrete beams subjected to combined flexure and torsion 40
2.8 Curved in plan composite steel-concrete beams 47
2.9 Flexure-torsion interaction relationship 49
2.10 Analytical studies on composite steel-concrete beams 51
2.11 Summary 54

Figures 56

3 EXPERIMENTAL SERIES I - STRAIGHT COMPOSITE STEEL-CONCRETE BEAMS 60
3.1 Introduction 60
3.2 Details of test specimen 60
  3.2.1 Beam test specimens 60
  3.2.2 Push-out test specimens 61
3.3 Test specimen fabrication 62
  3.3.1 Beam test specimens 62
  3.3.2 Push-out test specimens 63
3.4 Instrumentation 64
  3.4.1 Beam test specimens 64
  3.4.2 Push-out test specimens 65
3.5 Test frame and machine 65
  3.5.1 Beam test specimens 65
  3.5.2 Push-out test specimens 66
3.6 Loading procedure 67
  3.6.1 Beam test specimens 67
  3.6.2 Push-out test specimens 68
3.7 Material tests 68
  3.7.1 Concrete 68
    3.7.1.1 Cylinder compression tests 68
    3.7.1.2 Splitting tests 69
  3.7.2 Structural steel 70
  3.7.3 Steel reinforcing bar 71
  3.7.4 Shear studs 71
3.8 Push-out tests 72
  3.8.1 Push-out test observations 72
  3.8.2 Push-out test results 73
3.9 Beam tests 74
  3.9.1 Beam test observations 75
    3.9.1.1 Composite steel-concrete beam CBF-1 75
4 EXPERIMENTAL SERIES II - CURVED IN PLAN COMPOSITE STEEL-CONCRETE BEAMS

4.1 Introduction

4.2 Details of test specimen
   4.2.1 Beam test specimens
   4.2.2 Push-out test specimens

4.3 Test specimen fabrication

4.4 Instrumentation

4.5 Test frame and machine

4.6 Loading procedure

4.7 Material tests
4.7.1 Concrete
   4.7.1.1 Cylinder compression tests 162
   4.7.1.2 Splitting tests 163
4.7.2 Structural steel 163
4.7.3 Steel reinforcing bar 164
4.7.4 Shear studs 164

4.8 Push-out tests 164
   4.8.1 Push-out observations 165
   4.8.2 Push-out test results 165

4.9 Beam tests 167
   4.9.1 Beam observations 167
      4.9.1.1 Composite steel-concrete beam CCBF-1 168
      4.9.1.2 Composite steel-concrete beam CCBF-2 168
      4.9.1.3 Composite steel-concrete beam CCBF-3 169
      4.9.1.4 Composite steel-concrete beam CCBF-4 169
      4.9.1.5 Composite steel-concrete beam CCBP-1 170
      4.9.1.6 Composite steel-concrete beam CCBP-2 170
      4.9.1.7 Composite steel-concrete beam CCBP-3 170
      4.9.1.8 Composite steel-concrete beam CCBP-4 171
   4.9.2 Beam test results 171
      4.9.2.1 Strength response for composite steel-concrete beams 172
      4.9.2.2 Neutral axis for composite steel-concrete beams 175
      4.9.2.3 Torsional interface slip for composite steel-concrete beams 177
      4.9.2.4 Longitudinal interface slip for composite steel-concrete beams 177

4.10 Summary 178

Tables 181

Figures 195

5 FLEXURE-TORSION INTERACTION RELATIONSHIP 234

5.1 Introduction 234
5.2 Previous research

5.3 Experimental series I
  5.3.1 Ultimate torsion equation
  5.3.2 Flexure-torsion interaction relationship
  5.3.3 Modified design models

5.4 Experimental series II
  5.4.1 Flexure-torsion interaction relationship
  5.4.2 Modified design models

5.5 Summary

Tables

Figures

6 FINITE ELEMENT MODELLING

6.1 Introduction

6.2 Description of ABAQUS

6.3 Modelling procedure in ABAQUS

6.4 Material constitutive relationships
  6.4.1 Concrete
    6.4.1.1 Concrete property
    6.4.1.2 Concrete smeared cracking
    6.4.1.3 Concrete damaged plasticity
  6.4.2 Structural steel
  6.4.3 Shear studs

6.5 Contact interactions and boundary conditions
  6.5.1 Concrete slab and steel beam interface
  6.5.2 Concrete slab and reinforcing steel interface
  6.5.3 Concrete slab and shear studs interface
  6.5.4 Steel beam and shear studs interface
6.5.5 Simply supported and twisting restraint conditions 264

6.6 Load applications 264
6.6.1 Applied load method 264
6.6.2 Modified Riks analysis method 265

6.7 Finite element type and mesh 265
6.7.1 Solid elements 266
6.7.2 Truss elements 266

6.8 Sensitivity analysis 267
6.8.1 Sensitivity to concrete compressive strength 267
6.8.2 Sensitivity to mesh size 269
   6.8.2.1 Shear connectors 269
   6.8.2.2 Concrete slab 269
   6.8.2.3 Steel beam 270

6.9 Summary 270

Figures 272

7 COMPARISONS BETWEEN FEM AND EXPERIMENTAL TESTS 290

7.1 Introduction 290

7.2 Comparisons between models and experimental series I 291
   7.2.1 Validation of the finite element model 291
      7.2.1.1 Composite steel-concrete beam CBF-1 291
      7.2.1.2 Composite steel-concrete beam CBF-2 292
      7.2.1.3 Composite steel-concrete beam CBF-3 292
      7.2.1.4 Composite steel-concrete beam CBP-1 293
      7.2.1.5 Composite steel-concrete beam CBP-2 293
      7.2.1.6 Composite steel-concrete beam CBP-3 293
      7.2.1.7 Comparisons of the finite element model 294
      7.2.1.8 Concrete slabs CS-1, CS-2 and CS-3 295
      7.2.1.9 Steel beams SB-1, SB-2 and SB-3 295
7.2.2 Discussion 296

7.3 Comparisons between models and experimental series II 297

7.3.1 Validation of the finite element model 298

7.3.1.1 Composite steel-concrete beam CCBF-1 298
7.3.1.2 Composite steel-concrete beam CCBF-2 298
7.3.1.3 Composite steel-concrete beam CCBF-3 299
7.3.1.4 Composite steel-concrete beam CCBF-4 299
7.3.1.5 Composite steel-concrete beam CCBP-1 299
7.3.1.6 Composite steel-concrete beam CCBP-2 300
7.3.1.7 Composite steel-concrete beam CCBP-3 300
7.3.1.8 Composite steel-concrete beam CCBP-4 301
7.3.1.9 Comparisons of the finite element model 301

7.3.2 Discussion 302

8 PARAMETRIC STUDY 342

8.1 Introduction 342

8.2 Beam selection for parametric study 343

8.3 Parametric study for straight composite steel-concrete beams 344

8.3.1 6 m straight composite steel-concrete beams 344
8.3.2 8 m straight composite steel-concrete beams 345
8.3.3 10 m straight composite steel-concrete beams 345
8.3.4 12 m straight composite steel-concrete beams 345
8.3.5 14 m straight composite steel-concrete beams 346

8.3.6 Discussion 346

8.3.6.1 Effects of the beam span length 346
8.3.6.2 Effects of the degree of shear connection 347
LIST OF TABLES

Table 3.1 Straight beam specimen details 90
Table 3.2 Push-out specimen details 91
Table 3.3 Cylinder compression test results 92
Table 3.4 Splitting test results 93
Table 3.5 Structural steel tensile test results 94
Table 3.6 Reinforcing longitudinal bar tensile test results 95
Table 3.7 Stirrup bar tensile test results 96
Table 3.8 Headed shear stud tensile test results 97
Table 3.9 Push-out test results 98
Table 3.10 Straight beam test results 99
Table 3.11 Interface slips for straight composite steel-concrete beams 100
Table 4.1 Curved in plan composite steel-concrete beam specimen details 181
Table 4.2 Push-out specimen details 182
Table 4.3 Cylinder compression test results in 28 days 183
Table 4.4 Cylinder compression test results for CCBF beam tests 184
Table 4.5 Cylinder compression test results for CCBP beam tests 185
Table 4.6 Cylinder compression test results for push-out tests 186
LIST OF FIGURES

Figure 1.1 Cross-sectional view of a typical composite steel-concrete beam 20
Figure 1.2 Construction methods (a) and (b), (Uy and Liew, 2003) 21
Figure 1.3 Construction methods (c) and (d), (Uy and Liew, 2003) 22
Figure 1.4 Grosvenor Place, Sydney, Australia, (Uy and Liew, 2003) 23
Figure 1.5 Curved in plan highway girder beam 23
Figure 1.6 Edge beams, (Uy and Liew, 2003) 24
Figure 1.7 Western Distributor Highway, Darling Harbour, Australia 24
Figure 1.8 Ultimate sagging moment, plastic neutral axis in concrete slab 25
Figure 1.9 Ultimate sagging moment, plastic neutral axis in steel beam 25
Figure 1.10 Ultimate sagging moment, PSC 25
Figure 1.11 Ultimate hogging moment, plastic neutral axis in concrete slab 26
Figure 1.12 Ultimate hogging moment, plastic neutral axis in steel beam 26
Figure 1.13 Ultimate hogging moment, PSC 26
Figure 2.1 Mechanical shear connectors 56
Figure 2.2 Headed shear studs 57
Figure 2.3 Flexure-torsion interaction relationship curves 58
Figure 2.4 Flexure-torsion interaction relationship for straight composite beams 59
Figure 3.1 Cross-sectional view for CBF-1, CBF-2 and CBF-3 101
Figure 3.2 Plan view for CBF-1, CBF-2 and CBF-3 102
Figure 3.3 Elevation view for CBF-1, CBF-2 and CBF-3 103
Figure 3.4 Cross-sectional view for CBP-1, CBP-2 and CBP-3 104
Figure 3.5 Plan view for CBP-1, CBP-2 and CBP-3 105
Figure 3.6 Elevation view for CBP-1, CBP-2 and CBP-3 106
Figure 3.7 Cross-sectional view for CS-1, CS-2 and CS-3 107
Figure 3.8 Plan view for CS-1, CS-2 and CS-3 108
Figure 3.9 Elevation view for SB-1, SB-2 and SB-3 109
Figure 3.10 Cross-sectional views for SB-1, SB-2 and SB-3 110
Figure 3.11 Push-out test specimens for PT-F1 and PT-F2 111
Figure 3.12 Push-out test specimens for PT-P1 and PT-P2 112
Figure 3.13 Beam test specimens ready for concrete casting 113
Figure 3.14 Concrete cylinders 113
Figure 3.15 Push-out test specimens ready for concrete casting 114
Figure 3.16 Push-out test specimens welded together 114
Figure 3.17 LVDT for measuring the longitudinal interface slip 115
Figure 3.18 LVDT locations in push-out tests 115
Figure 3.19 Composite steel-concrete beam set-up 116
Figure 3.20 Restraining system at the support 116
Figure 3.21 Concrete slab set-up 117
Figure 3.64 Strain distribution at right point load for CBP-3 154
Figure 3.65 End support interface slips for straight composite beams 155
Figure 4.1 Cross-sectional view for CCBF-1, CCBF-2, CCBF-3 and CCBF-4 195
Figure 4.2 Plan view for CCBF-1, CCBF-2, CCBF-3 and CCBF-4 196
Figure 4.3 Elevation view for CCBF-1, CCBF-2, CCBF-3 and CCBF-4 197
Figure 4.4 Cross-sectional view for CCBP-1, CCBP-2, CCBP-3 and CCBP-4 198
Figure 4.5 Plan view for CCBP-1, CCBP-2, CCBP-3 and CCBP-4 199
Figure 4.6 Elevation view for CCBP-1, CCBP-2, CCBP-3 and CCBP-4 200
Figure 4.7 Push-out test specimens for CBPT-F1 and CBPT-F2 201
Figure 4.8 Push-out test specimens for CBPT-P1 and CBPT-P2 202
Figure 4.9 Reinforcement cage 203
Figure 4.10 Curved composite beam test specimens after concrete pouring 203
Figure 4.11 Curved composite steel-concrete beam set-up 204
Figure 4.12 Restraining system at the support 204
Figure 4.13 Steel web stress-strain curves 205
Figure 4.14 Steel flange stress-strain curves 206
Figure 4.15 Longitudinal reinforcing bar stress-strain curves 207
Figure 4.16 Stirrup bar stress-strain curves 208
Figure 4.17 Shear stud tensile test specimen 209
Figure 4.18 Shear stud tensile test 209
Figure 4.19 Shear stud stress-strain curves 210
Figure 4.20 Push-out test summarised results 211
Figure 4.21 Load-horizontal displacements for push-out tests 212
Figure 4.22 Diagonal torsional cracks for CCBF-1 213
Figure 4.23 Separation between the concrete slab and the steel beam 213
Figure 4.24 Twisting at mid span for CCBF-2 214
Figure 4.25 Few flexural concrete cracks for CCBF-3 214
Figure 4.26 Diagonal torsional cracks for CCBF-4 215
Figure 4.27 Uplift of the steel beam for CCBF-4 215
Figure 4.28 Diagonal torsional cracks for CCBP-2 216
Figure 4.29 Longitudinal interface slip for CCBP-3 216
Figure 4.30 Load-mid span deflection for curved composite beams 217
Figure 4.31 Moment-mid span curvature for curved composite beams 218
Figure 4.32 Moment-mid span curvature for CCBF 219
Figure 4.33 Moment-mid span curvature for CCBP 220
Figure 4.34 Torque-twist for curved composite steel-concrete beams 221
Figure 4.35 Torque-twist for CCBF 222
Figure 4.36 Torque-twist for CCBP 223
Figure 4.37 Strain distribution at mid span for CCBF-1 224
Figure 4.38 Strain distribution at mid span for CCBF-2 225
Figure 4.39 Strain distribution at mid span for CCBF-3 226
Figure 4.40 Strain distribution at mid span for CCBF-4 227
Figure 7.12 Comparison between FEM and CS-2 322
Figure 7.13 Comparison between FEM and CS-3 323
Figure 7.14 Cross-sectional views of FEM at point load for CS 324
Figure 7.15 Isotropic views of FEM for CS 325
Figure 7.16 Comparison between FEM and SB-1 326
Figure 7.17 Comparison between FEM and SB-2 327
Figure 7.18 Comparison between FEM and SB-3 328
Figure 7.19 Cross-sectional and isotropic views of FEM at point load for SB 329
Figure 7.20 Comparison between FEM and CCBF-1 330
Figure 7.21 Comparison between FEM and CCBF-2 331
Figure 7.22 Comparison between FEM and CCBF-3 332
Figure 7.23 Comparison between FEM and CCBF-4 333
Figure 7.24 Comparison between FEM and CCBP-1 334
Figure 7.25 Comparison between FEM and CCBP-2 335
Figure 7.26 Comparison between FEM and CCBP-3 336
Figure 7.27 Comparison between FEM and CCBP-4 337
Figure 7.28 Cross-sectional views of FEM at point load for CCBF 338
Figure 7.29 Cross-sectional views of FEM at point load for CCBP 339
Figure 7.30 Isotropic views of FEM for CCBF 340
Figure 7.31 Isotropic views of FEM for CCBP 341
Figure 8.1 Plan view of a typical composite floor system 360
Figure 8.2 Strength interaction curves for 6 m straight composite beams 361
Figure 8.3 Strength interaction curves for 8 m straight composite beams 362
Figure 8.4 Strength interaction curves for 10 m straight composite beams 363
Figure 8.5 Strength interaction curves for 12 m straight composite beams 364
Figure 8.6 Strength interaction curves for 14 m straight composite beams 365
Figure 8.7 Strength interaction ratio curves for 6 m straight composite beams 366
Figure 8.8 Strength interaction ratio curves for 8 m straight composite beams 367
Figure 8.9 Strength interaction ratio curves for 10 m straight composite beams 368
Figure 8.10 Strength interaction ratio curves for 12 m straight composite beams 369
Figure 8.11 Strength interaction ratio curves for 14 m straight composite beams 370
Figure 8.12 Strength interaction curves for FSC straight composite beams 371
Figure 8.13 Strength interaction curves for PSC straight composite beams 372
Figure 8.14 Strength interaction ratio curves for FSC straight composite beams 373
Figure 8.15 Strength interaction ratio curves for PSC straight composite beams 374
Figure 8.16 Design model for 6 and 8 m straight composite beams 375
Figure 8.17 Design model for 10, 12 and 14 m straight composite beams 376
Figure 8.18 Strength interaction curves for 6 m curved composite beams 377
Figure 8.19 Strength interaction curves for 8 m curved composite beams 378
Figure 8.20 Strength interaction curves for 10 m curved composite beams 379
Figure 8.21 Lateral buckling of the steel web for curved in plan composite beams 380
Figure 8.22 Strength interaction curves for 12 m curved composite beams 381
Figure 8.23 Strength interaction curves for 14 m curved composite beams 382
Figure 8.24 Strength interaction ratio curves for 6 m curved composite beams 383
Figure 8.25 Strength interaction ratio curves for 8 m curved composite beams 384
Figure 8.26 Strength interaction ratio curves for 10 m curved composite beams 385
Figure 8.27 Strength interaction ratio curves for 12 m curved composite beams 386
Figure 8.28 Strength interaction ratio curves for 14 m curved composite beams 387
Figure 8.29 Strength interaction curves for FSC curved composite beams 388
Figure 8.30 Strength interaction curves for PSC curved composite beams 389
Figure 8.31 Strength interaction ratio curves for FSC curved composite beams 390
Figure 8.32 Strength interaction ratio curves for PSC curved composite beams 391
Figure 8.33 Design model for 6, 8, 10 and 12 m curved in plan composite beams 392
# LIST OF NOTATION

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
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<tbody>
<tr>
<td>$7.5f'_c$</td>
<td>modulus of rupture of the concrete</td>
</tr>
<tr>
<td>$A_I$</td>
<td>cross sectional area of the longitudinal reinforcement</td>
</tr>
<tr>
<td>$A_s$</td>
<td>cross sectional area of the one leg of the closed hoops</td>
</tr>
<tr>
<td>$A_{sw}$</td>
<td>cross-sectional area of the bar forming a closed tie</td>
</tr>
<tr>
<td>$b_f$</td>
<td>flange width</td>
</tr>
<tr>
<td>$b_w$</td>
<td>web height</td>
</tr>
<tr>
<td>$C$</td>
<td>total compression force</td>
</tr>
<tr>
<td>$C_c$</td>
<td>compression force for the concrete</td>
</tr>
<tr>
<td>$C_s$</td>
<td>compression force for the steel beam</td>
</tr>
<tr>
<td>$d$</td>
<td>diameter of the specimen</td>
</tr>
<tr>
<td>$f'$</td>
<td>cylinder compressive strength of the concrete</td>
</tr>
<tr>
<td>$f'_c$</td>
<td>characteristic compressive strength of the concrete</td>
</tr>
<tr>
<td>$f_{cm}$</td>
<td>principal tensile strength of the concrete</td>
</tr>
<tr>
<td>$f_{cy}$</td>
<td>compressive strength of the concrete</td>
</tr>
<tr>
<td>$f_{sy}$</td>
<td>yield strength of the reinforcing steel</td>
</tr>
<tr>
<td>$f_{sy,f}$</td>
<td>yield strength of the fitments</td>
</tr>
<tr>
<td>$K_c$</td>
<td>torsional rigidity of the concrete slab</td>
</tr>
</tbody>
</table>
\( L \)  
lever arm

\( L_s \)  
length of the specimen

\( L_c \)  
lever arm for the concrete in compression

\( L_r \)  
lever arm for the reinforcing steel in tension

\( L_s \)  
lever arm for the steel beam in tension or compression

\( m = \frac{A_1 s}{[A_s (x_1 + y_1)]} \)

\( M_{\text{applied}} \)  
applied bending moment

\( M \)  
flexural capacity of the composite steel-concrete section

\( M_F \)  
flexural moments at failure in test

\( M_U \)  
theoretical values of the ultimate flexural moments

\( M_u \)  
flexural capacity of the composite steel-concrete section under pure bending

\( M_{uo} \)  
ultimate moment capacity of the composite beam in pure bending

\( P \)  
applied load at failure

\( s \)  
centre-to-centre spacing of the shear or torsional reinforcement

\( S \)  
plastic shear modulus

\( T \)  
torsional capacity of the composite steel-concrete section

\( T_C \)  
contribution of the concrete towards ultimate torque

\( t_c \)  
thickness of the concrete slab

\( T_{cu} \)  
torsional capacity of the unreinforced concrete slab

\( t_f \)  
flange thickness

\( T_F \)  
torsional moments at failure in test
$T_f$ contribution of the steel beam towards ultimate torque  
$T_j$ torsional capacity of the steel beam  
$T_p$ ultimate torsional capacity of the composite steel-concrete section  
$T_r$ tension force for the reinforcing steel  
$T_s$ contribution of the torsional reinforcement towards ultimate torque  
$T_s$ tension force for the steel beam  
$T_{tr}$ torsional capacity of the steel reinforcement  
$T_U$ theoretical values of the ultimate torsional moments  
$T_u$ torsional capacity of the composite steel-concrete section under pure torque  
$t_w$ web thickness  
$V_u$ ultimate shear capacity of the composite beam in combined shear and bending  
$V_{uo}$ ultimate shear capacity of the composite beam in pure bending  
$x$ thickness of the concrete slab  
$x_0$ smaller distance between corner bars in the rectangular cross-section  
$x_1$ smaller centre to centre dimensions of the closed hoops  
$y$ width of the concrete slab  
$y_0$ longer distance between corner bars in the rectangular cross-section  
$y_1$ larger centre to centre dimensions of the closed hoops  
$\alpha = 0.66m + 0.33(y_1/x_1)$  
$\beta$ level of shear connection
\[ \varepsilon_0 = 0.002, \text{ strain corresponding to the maximum compressive stress} \]

\[ \varepsilon_c = 0.0038, \text{ strain at which the concrete crushes} \]

\[ \tau_y = 0.6 \times \text{steel beam yield stress} \]
# LIST OF ABBREVIATIONS

<table>
<thead>
<tr>
<th>2-D</th>
<th>2-Dimensional</th>
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<tbody>
<tr>
<td>3-D</td>
<td>3-Dimensional</td>
</tr>
<tr>
<td>AISC</td>
<td>American Standard</td>
</tr>
<tr>
<td>AS</td>
<td>Australian Standard</td>
</tr>
<tr>
<td>Avg</td>
<td>average</td>
</tr>
<tr>
<td>BHP</td>
<td>Broken Hill Proprietary Company</td>
</tr>
<tr>
<td>BS</td>
<td>British Standard</td>
</tr>
<tr>
<td>CBF</td>
<td>composite steel-concrete beam with full shear connection</td>
</tr>
<tr>
<td>CBF</td>
<td>composite steel-concrete beam with full shear connection</td>
</tr>
<tr>
<td>CBP</td>
<td>composite steel-concrete beam with partial shear connection</td>
</tr>
<tr>
<td>CCBF</td>
<td>curved in plan composite steel-concrete beam with full shear connection</td>
</tr>
<tr>
<td>CCBP</td>
<td>curved in plan composite steel-concrete beam with partial shear connection</td>
</tr>
<tr>
<td>CS</td>
<td>concrete slab</td>
</tr>
<tr>
<td>EC</td>
<td>British Standard (Eurocode)</td>
</tr>
<tr>
<td>et al.</td>
<td>and others</td>
</tr>
<tr>
<td>FEA</td>
<td>finite element analysis</td>
</tr>
<tr>
<td>Abbreviation</td>
<td>Description</td>
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<tr>
<td>--------------</td>
<td>------------------------------------</td>
</tr>
<tr>
<td>FEM</td>
<td>finite element method</td>
</tr>
<tr>
<td>FSC</td>
<td>full shear connection</td>
</tr>
<tr>
<td>LVDT</td>
<td>linear variable differential transducer</td>
</tr>
<tr>
<td>Max</td>
<td>maximum</td>
</tr>
<tr>
<td>mRad</td>
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<td>not applicable, not available</td>
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<td>NA</td>
<td>neutral axis</td>
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<tr>
<td>PSC</td>
<td>partial shear connection</td>
</tr>
<tr>
<td>PT</td>
<td>push-out test</td>
</tr>
<tr>
<td>SB</td>
<td>steel beam</td>
</tr>
<tr>
<td>Std. Dev.</td>
<td>Standard deviation</td>
</tr>
<tr>
<td>UB</td>
<td>universal beam</td>
</tr>
<tr>
<td>UNF</td>
<td>unified fine threads</td>
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</tbody>
</table>
Composite steel-concrete construction has been widely used around the world. Techniques for the ultimate load analysis and design of composite steel-concrete beams are also well-established and solutions can be obtained with relative ease. However, beams under combined actions such as edge or curved in plan beams subjected to torsion and flexure can be difficult to ascertain their strengths and behaviour due to their complex stress state. These effects of combined actions are not currently addressed in the Australian Standard AS2327.1 (Standards Australia, 2003) or any their international codes.

Therefore, this thesis will present two full-scale experimental series to investigate the behaviour of straight and curved in plan composite steel-concrete beams subjected to torsion and flexure with the influence of partial shear connection. Push-out and material tests were carried out to determine the material properties of each component of the composite steel-concrete beams.

A new development of a 3-D finite element models using ABAQUS capable of simulating composite steel-concrete beams subjected to combined flexure and torsion with the influence of partial shear connection. From the comparisons, the models were validated with the experimental results and showed good agreement in terms of load-deflection response and ultimate strengths.
A parametric study was then conducted to investigate the behaviour of the beams under the effects of different beam span lengths and levels of shear connection by extending the finite element model. From the study, composite steel-concrete beams have shown an increase in the torsional and flexural capacities with the increase in span length, whilst the partial shear connection has no significant effect to its strengths. Additionally, the rate of increase in the torsional strength also increases with span length. Simplified design models have been proposed to represent the flexure-torsion interaction relationship for both straight and curved in plan composite steel-concrete beams.
CHAPTER 1

1 INTRODUCTION

1.1 INTRODUCTION

Nowadays, composite steel-concrete structures are used in civil engineering projects worldwide such as applications including building, bridges, foundations and special structures, (Bridges et. al, 1992). Composite steel-concrete systems combine both the advantages of a ductile steel frame with the stiffness of then concrete slabs to carry the loads. Additionally, provided that the steel frame can support the construction loads, composite construction can reduce the time of construction.

Composite construction has been commonly used in modern buildings and highways due to their advantages over traditional reinforced concrete construction. However, due to the complexity of the designs for modern structures, some of the supporting beams such as edge or curved in plan member are subjected to combined loading. One of such combinations is the application of combined flexure and torsion. The effects of combined loading are not currently addressed in the Australian Standards AS2327.1 (Standards Australia, 2003) or any other international codes. Therefore, this thesis has presented a study to investigate the behaviour of composite steel-concrete beams subjected to combined flexure and torsion.

Chapter 1 has presented a summary of the research work carried out in this thesis and also outlines its contributions and significance to the field of composite
steel-concrete construction. Deficiencies have been identified through intensive background reading of research papers, which also highlighted the need for future research in composite steel-concrete construction especially when partial shear connection (PSC) design is used. This thesis endeavours to investigate those identified deficiencies, particularly related to curved in plan composite steel-concrete beams subjected to combined flexure and torsion. This area is then further explained in the next chapter as part of the literature review chapter for this thesis. The objectives and scope of this thesis will also be presented in this chapter. A brief thesis chapter layout will be explained to provide an overview of its content with a summarised description.

1.2 BACKGROUND AND MOTIVATION FOR RESEARCH

The design of structures for buildings and bridges is mainly concerned with the provision and support of load-bearing horizontal surfaces. Except in long span bridges, no other material has a combination of low cost, high strength and resistance to corrosion, abrasion and fire than the reinforced concrete floors or decks. The economical span for a reinforced concrete slab is achieved when its thickness becomes just sufficient to resist the wet concrete loading to which it may be subjected to. For more than a few metres span, it is cheaper to support on concrete beams than to thicken the concrete slab. When the beams are also of concrete, the monolithic nature of the construction makes it possible to act as the top flange of the concrete beam that supports it with a substantial breadth of the concrete slab. For spans of more than 10 m and where fire resistance is not a major issue, such as bridges and multi-storey car parks, steel beams become cheaper than concrete beams. It is considered the norm to design steel beams to carry the whole weight of the
concrete slab and its loading. However, when shear connectors were developed, it became practical to connect the concrete slab to the steel beam and so to obtain a T-beam section that had long been used in building construction. The term “composite steel-concrete beams” refers in this thesis to the type of structure illustrated in Figure 1.1. In such a composite steel-concrete beam, the comparatively high strength of the concrete in compression complements the high strength of the steel in tension. The fact that each material is used to take advantage of its best attribute makes composite steel-concrete construction very efficient and economical. The floor slab above the steel beam maybe constructed by several different methods as presented in Figures 1.2 and 1.3.

- (a) in-situ concrete slab
- (b) precast reinforced concrete planks with in-situ concrete topping slab
- (c) precast reinforced concrete planks with in-situ grouting at the joint
- (d) in-situ concrete on trapezoidal metal decking

Composite steel-concrete construction is also extensively used in modern buildings and highways. Some of the famous buildings using composite steel-concrete construction include the Forrest Place in Perth, Australia, Casseldon Place in Melbourne, Australia, Grosvenor Place in Sydney, Australia as shown in Figure 1.4, Myer Centre in Adelaide, Australia, Petronas Tower in Kuala Lumpur, Malaysia, Republic Plaza and One Raffles Link in Singapore and Taipei 101 in Taipei, Taiwan. The main advantages of using composite steel-concrete construction are higher span to depth ratio, significant reduction in mid span deflection and higher stiffness ratio than traditional steel or reinforced concrete beam structures.
In certain types of structures such as highway bridges, highway interchanges and balconies, the supporting beams are usually horizontally curved in plan as shown in Figure 1.5. These supporting beams can be subjected to combined flexure and torsion due to their curved in plan profile and the vertical uniform applied loads. Other common structures that experience combined loadings include edge beams of a building as shown in Figure 1.6. Currently, there is no rational method in the Australian Standard AS2327.1 (Standards Australia, 2003) or other international standards such as the Eurocode 4 (British Standards Institution, 2005), the British Standards BS5950-1 (British Standards Institution, 2004) and the American Institute of Steel Construction AISC (American Institute of Steel Construction, 2006) for designing these structural forms. Without proper design guidelines for the curved in plan composite steel-concrete beams, straight composite steel-concrete beams are commonly used instead. A good example is the Western Distributor Highway at Darling Harbour in Sydney, Australia as illustrated in Figure 1.7. Moreover, this problem can become more complicated when PSC is used in the design, as PSC is commonly used to reduce the construction cost and improve the ductility without a great reduction in the flexural moment capacity compared to full shear connection (FSC). Therefore, there is a need for improvement of all standards to include the calculation for composite steel-concrete beams subjected to torsion and flexure.

1.3 COMPOSITE STEEL-CONCRETE BEAMS IN SAGGING MOMENT REGIONS

For composite steel-concrete beams, the level or degree of shear connection is of great importance to the design of the structures and can greatly affect the structural behaviour of the composite steel-concrete beams. It is defined as the
percentage of the total strength of the shear connectors in a shear span length against the weakest component of the composite steel-concrete cross-section in consideration. The strength of shear connection in a shear span length is defined as the sum of the shear strength of each shear connector in a shear span length. In the sagging moment regions of composite steel-concrete beams, the weakest component constitutes either the compressive strength of the concrete slab or the axial tensile strength of the steel beam. The concrete slab is assumed to be fully crushed in compression and the steel beam is assumed to be fully yielded in tension and compression which depends on the neutral axis location and the degree of the shear connection provided. The ratio of the strength of the shear connection provided to the strength of the weakest element (concrete slab or steel beam) is defined as the level of shear connection ($\beta$). The three cases that may exist in the sagging moment regions are listed in the following:

- Plastic neutral axis in the concrete slab, (FSC, $\beta = 1.0$)
- Plastic neutral axis in the steel beam, (FSC, $\beta = 1.0$)
- Partial shear connection, (PSC, $\beta < 1.0$)

### 1.3.1 Plastic neutral axis in the concrete slab, (FSC, $\beta = 1.0$)

When the steel component is the weakest element compared with the concrete and the shear connector components, the plastic neutral axis will lie within the concrete component. The ultimate flexural moment can be determined from a single couple as shown in Figure 1.8 and presented in Equation 1.1.

$$M_u = TL = CL$$  \hspace{1cm} (1.1)

where,
When the concrete component is the weakest component compared with the steel and the shear connector components, the plastic neutral axis will lie within the steel component. The ultimate flexural moment can be determined by summing the moments about the centroid of the tension force as shown in Figure 1.9 and presented in Equation 1.2.

\[ M_u = C_c L_c + C_s L_s \]  \hspace{1cm} (1.2)

where,

- \( M_u \) = ultimate flexural moment
- \( C_c \) = compression force for the concrete
- \( C_s \) = compression force for the steel beam
- \( L_c \) = lever arm for the concrete in compression
- \( L_s \) = lever arm for the steel beam in compression

### 1.3.3 Partial shear connection, (PSC, \( \beta < 1.0 \))

When the shear connector component is the weakest compared with the concrete and the steel components, the plastic neutral axis may lie within the concrete or steel components or both. The ultimate flexural moment can be
determined by summing the moments about the centroid of the tension force as shown in Figure 1.10 and presented in Equation 1.3.

\[ M_u = C_c L_c + C_s L_s \]  \hspace{1cm} (1.3)

where,

\[ M_u \] = ultimate flexural moment
\[ C_c \] = compression force for the concrete
\[ C_s \] = compression force for the steel beam
\[ L_c \] = lever arm for the concrete in compression
\[ L_s \] = lever arm for the steel beam in compression

### 1.4 COMPOSITE STEEL-CONCRETE BEAMS IN HOGGING MOMENT REGIONS

In the hogging moment regions of composite steel-concrete beams, the axial tensile strength of the primary longitudinal reinforcement is usually less than the axial strength of the steel beam. Therefore, the primary longitudinal reinforcement usually represents the weakest component in the hogging moment regions. The tensile resistance of the longitudinal reinforcement and the compression of the steel beam determine the flexural moment resistance of the composite steel-concrete beam section. When PSC is considered, the flexural moment resistance is dependent on the longitudinal reinforcement strength in tension rather than the concrete slab in compression. Again, the ratio of the strength of the shear connection provided to the strength of the weakest element (concrete slab or steel beam) is defined as the level of shear connection \( (\beta) \). The three cases that may exist in the hogging moment regions are listed in the following:
• Plastic neutral axis in the concrete slab, (FSC, $\beta = 1.0$)
• Plastic neutral axis in the steel beam, (FSC, $\beta = 1.0$)
• Partial shear connection, (PSC, $\beta < 1.0$)

1.4.1 Plastic neutral axis in the concrete slab, (FSC, $\beta = 1.0$)

When the steel beam component is the weakest component compared with the longitudinal reinforcing steel and the shear connector components, the plastic neutral axis will lie within the concrete component. The ultimate flexural moment can be determined from a single couple as shown in Figure 1.11 and presented in Equation 1.4.

$$M_u = TL = CL$$

(1.4)

where,

$M_u$ = ultimate flexural moment

$T$ = total tension force

$C$ = total compression force

$L$ = lever arm

1.4.2 Plastic neutral axis in the steel beam, (FSC, $\beta = 1.0$)

When the longitudinal reinforcing steel component is the weakest component compared with the steel beam and the shear connector components, the plastic neutral axis will lies within the steel beam component. The ultimate flexural moment can be determined by summing the moments about the centroid of the compression force as shown in Figure 1.12 and presented in Equation 1.5.
\[ M_u = T_s L_s + T_r L_r \]  \hspace{1cm} (1.5)

where,

\[ M_u = \text{ultimate flexural moment} \]
\[ T_r = \text{tension force for the longitudinal reinforcing steel} \]
\[ T_s = \text{tension force for the steel beam} \]
\[ L_r = \text{lever arm for the longitudinal reinforcing steel in tension} \]
\[ L_s = \text{lever arm for the steel beam in tension} \]

### 1.4.3 Partial shear connection, (PSC, \( \beta < 1.0 \))

For special cases, PSC in the hogging moment regions may need to be considered in the design even though it is not allowed by international codes of practice. When the shear connector component is the weakest component compared with the longitudinal reinforcing steel and the steel beam components, the plastic neutral axis may lie within the concrete or steel beam components or both. Again, the ultimate flexural moment can be determined by summing the moments about the centroid of the compression force as shown in Figure 1.13 and presented in Equation 1.6.

\[ M_u = T_s L_s + T_r L_r \]  \hspace{1cm} (1.6)

where,

\[ M_u = \text{ultimate flexural moment} \]
\[ T_r = \text{tension force for the longitudinal reinforcing steel} \]
\[ T_s = \text{tension force for the steel beam} \]
\[ L_r = \text{lever arm for the longitudinal reinforcing steel in tension} \]
\[ L_s = \text{lever arm for the steel beam in tension} \]

For a FSC design, the strength of the shear connectors in a shear span length is considered to be greater than or similar to either the strength of the concrete slab or the steel beam, whichever is the weaker component. Another definition of FSC is the number of shear connectors required in which the bending resistance of the composite steel-concrete beams will not increase even with additional shear connectors, (Johnson and Molenstra, 1991). Therefore, the shear connectors along the composite steel-concrete beam span are able to transfer the total longitudinal axial force between the concrete slab and the steel beam to develop the maximum flexural moment capacity. It is also assumed that there is zero longitudinal slip at the concrete slab and steel beam interface for FSC.

For a PSC design, the number of shear connectors is always less than those provided in a FSC design. The flexural moment capacity of the composite steel-concrete beams is governed by the strength and number of the shear connectors. Any increase or decrease in the number of shear connectors provided to a PSC design will in term resulted in an increase or a decrease in the flexural moment capacity respectively. Nevertheless, recent studies have suggested that PSC design can improve the available rotation capacity of the composite steel-concrete beams which is required for developing plastic hinge. This development of plastic hinge allows the usage of plastic analysis in the design of composite steel-concrete beams. Therefore, the level of shear connection has been always a vital parameter in composite steel-concrete research.
1.5 PREVIOUS RESEARCH

The word “torsion” has been avoided by most designers in the field of reinforced concrete, steel and composite steel-concrete construction. The torsional moment capacity calculation for reinforced concrete and steel box sections has been widely recognised in all standards and research papers. However, due to the nature of open steel sections being commonly used in composite steel-concrete beams in floor systems, the torsional moment capacity of composite steel-concrete beams has been largely ignored in the design. The reason is that composite steel-concrete beams consisting of open steel sections are generally prone to large warping stresses and excessive angles of twist when they are subjected to torsion. Therefore, the common practice is to avoid torsional moment calculations of composite steel-concrete beams consisting of open steel sections. However, simplified methods are available to calculate the torsional moment capacity for composite steel-concrete beams. This method involves taking the summation of the torsional capacities from the concrete slab, the torsional reinforcement and the steel beam. This method proved to be accurate when composite steel-concrete beams are under pure torsion.

However, there are times when combined loading arises, as wide applications of horizontally curved highway bridges have been constructed. This form of construction has stimulated research on the torsional behaviour of composite steel-concrete beams subjected to combined flexure and torsion. The earliest research was conducted by Colville (1973) with four single curved in plan composite steel-concrete beams to develop a simplified method for the design of headed shear studs.

Singh and Mallick (1977) later conducted tests on eight straight composite steel-concrete beams subjected to combined flexure and torsion to investigate their
flexure-torsion interaction relationships. They concluded that there was an increase in torsional moment capacity in the presence of flexure for the composite steel-concrete beams and also an increase in the flexural moment capacity in the presence of torsion.

Ghosh and Mallick (1979) continued with six more straight composite steel-concrete beams with different ratios of combined flexure and torsion. They concluded that it was possible that in the presence of flexure, closely spaced torsional and transverse reinforcement and local compression caused by shear studs at the base of the concrete slab in resisting slip might combine to give rise to some complicated stress state. This stress state has increased the tensile resistance of the concrete component in the composite steel-concrete sections.

Ray and Mallick (1980) also conducted tests on five straight composite steel-concrete beams with different ratios between the flexural and torsional moments. They concluded that the loading history of the test specimens did not have any appreciable influence on the behaviour of the composite steel-concrete sections under combined flexure and torsion.

Nie et al. (2000) conducted experiments on four straight composite steel-concrete beams to show that the current method of flexure-shear-torsion interaction equations for reinforced concrete beams could not be directly applied to the design of composite steel-concrete beams. Suggested formulae have been proposed in their paper to predict the resistance of composite steel-concrete beams under flexure, shear and torsion.

Thevendran et al. (2000) conducted experimental studies on curved in plan composite steel-concrete beams. Five composite steel-concrete beams with different
horizontal curvature were considered and tested to failure. Their test results indicated that the load carrying capacity decreased with an increase in the span to radius of curvature ratio.

However, all the experimental tests mentioned before were carried out using FSC design and inconsistent conclusions have been given regarding the interaction relationship between the flexural and torsional moment capacities of the composite steel-concrete beams.

Other than experimental tests, numerical and finite element analyses have been uncovered during the past few years for composite steel-concrete beams under combined loading. Pi et al. (2006) formulated a total Lagrangian finite element model using nonlinear inelastic analysis for both composite steel-concrete beams and columns. A rotation matrix was used in the position vector analysis and the nonlinear strain derivations. The concrete-steel interface was considered as an independent displacement in the formulation to take into account the slip conditions at the connections between the concrete slab and steel beam.

Using this formulation, Erkmen and Bradford (2009) have developed a 3-Dimensional (3-D) elastic total Lagrangian formulation for curved in plan composite steel-concrete beams. This formulation has included the effects of nonlinear geometric and PSC in tangential and radial directions. The numerical formulation results have provided good agreement with experimental results and a simplified finite element model using commercial software package ABAQUS.

Finite element analysis was also used by Thevendran et al. (1999) to model the composite steel-concrete beams under combined loading. Their nonlinear finite element analysis model was found to be in good agreement with the experimental
results in terms of deformations, stress distribution and ultimate strength from Thevendran et al. (2000).

Most of the numerical and finite element analyses have simplified the components of the composite steel-concrete beams such as employing shell elements for the concrete slab, shell elements for the steel beam and beam elements for the shear studs. Those models have proved to work for situations when composite steel-concrete beams were subjected to shear, compression, tension and flexure. But it is impossible to model composite steel-concrete beams or even concrete slabs subjected to torsion using shell elements.

Therefore, this thesis will provide two full-scale experimental test series and 3-D solid finite element models to investigate the behaviour of composite steel-concrete beams subjected to combined flexure and torsion. More importantly, the interaction relationships between the flexural and torsional moment capacities for both straight and horizontally curved in plan composite steel-concrete beams will be provided for future design or reference. The ability to involve PSC in composite steel-concrete designs will also be reinforced in this thesis.

1.6 RESEARCH METHODOLOGY

The methodology used in this thesis can be divided into both experimental and analytical studies to provide a better understanding of those issues mentioned in Sections 1.2 and 1.5. The methodology and scope are summarised below:

(1) To carry out an experimental investigation on straight concrete slabs subjected to combined flexure and torsion.
(2) To carry out an experimental investigation on straight steel beams subjected to combined flexure and torsion.

(3) To carry out an experimental investigation on straight composite steel-concrete beams subjected to combined flexure and torsion with the influence of PSC.

(4) To carry out an experimental investigation on curved in plan composite steel-concrete beams subjected to combined flexure and torsion with the influence of PSC.

(5) To develop a 3-D finite element model to simulate the behaviour of straight concrete slabs subjected to combined flexure and torsion.

(6) To develop a 3-D finite element model to simulate the behaviour of straight steel beams subjected to combined flexure and torsion.

(7) To develop a 3-D finite element model to simulate the behaviour of straight composite steel-concrete beams subjected to combined flexure and torsion with the influence of PSC.

(8) To develop a 3-D finite element model to simulate the behaviour of curved in plan composite steel-concrete beams subjected to combined flexure and torsion with the influence of PSC.

(9) To extend the 3-D finite element model to complete a parametric study for straight composite steel-concrete beams subjected to combined flexure and torsion with the influence of PSC.

(10) To extend the 3-D finite element model to complete a parametric study for curved in plan composite steel-concrete beams subjected to combined flexure and torsion with the influence of PSC.
To develop flexure-torsion interaction relationship curves for both straight and curved in plan composite steel-concrete beams subjected to combined flexure and torsion with the influence of PSC and span length.

1.7 THESIS LAYOUT

The layout of the thesis is divided into nine main chapters. Each chapter has an introduction to outline the contents within the chapter and a summary to conclude the findings from the chapter.

Chapter 1 presents a brief introduction of composite steel-concrete beam construction. It provides the reasoning and motivation behind the thesis topic and explains the present situation regarding the progress of the research for composite steel-concrete beams being subjected to combined flexure and torsion. It also identifies the grey areas in the research field and the deficiencies of the past and present research work. The scope of the thesis has been outlined in this chapter including the methodology used.

Chapter 2 presents a detailed literature review on the past and present research work which has been carried out by the other researchers relating to the thesis topic. This chapter has presented several issues regarding the previous research. This chapter includes the history of composite steel-concrete beams, shear connection design rules, PSC, push-out tests, composite steel-concrete beams subjected to combined loading and analytical studies on composite steel-concrete beams.

Chapter 3 presents the first set of experimental test series of the thesis. These experimental tests include straight concrete slabs, steel beams and composite steel-
concrete beams tested under combined flexure and torsion with the influence of PSC. This chapter provides the test specimen details, fabrication process, instrumentation used, test set-up and loading procedure of the experimental tests. Material tests were also conducted to determine the material properties of the beam test specimens using compression, tensile and push-out tests. Test observations and results are presented in this chapter for discussion.

Chapter 4 presents the second set of experimental test series of the thesis. The main difference between the first and second series is the structural design of the composite steel-concrete beams. This experimental series considers curved in plan composite steel-concrete beams rather than straight beams. The same parameters such as the different degree of shear connection and load combination are provided in this series. The horizontal curvature of the composite steel-concrete beams was varied to enable different load combinations of flexure and torsion to be considered. Material tests were carried out again since different batches of concrete and steel beams were used for the experimental series I and II. Test results and test observations are also presented in this chapter. Most importantly, in Chapter 4, a new phenomenon has been identified from the test results. This phenomenon is the discovery of the new term “torsional slip”, which it is associated with the difference between the curvatures of the concrete slab and the steel beam as a result of applied torque to the composite steel-concrete sections.

Chapter 5 presents the flexure-torsion interaction relationships obtained from both the experimental test series I and II. Discussion between those interaction relationship curves from previous research papers, standards and the experimental
tests is presented in this chapter. Initial recommended design models for composite steel-concrete beams are also provided for future reference.

**Chapter 6** presents the development of a finite element method to simulate the behaviour of the concrete slabs, steel beams and composite steel-concrete beams subjected to combined flexure and torsion. PSC has been considered in the analysis which allowed for the inclusion of longitudinal interface slip between the concrete slabs and the steel beams. The software package ABAQUS was used to enable the input of nonlinear material stress-strain properties of all composite steel-concrete beam components.

**Chapter 7** presents the comparisons between the experimental results from the experimental series I and II in **Chapters 3 and 4** respectively with the results from the finite element analysis in **Chapter 6**. The results from the finite element models have been validated by the experimental test results in terms of load-deflection response. From the good correlation of the test results, it provided confidence that the 3-D finite element model can be used for the modelling of composite steel-concrete beams subjected to combined loading with the influence of PSC.

**Chapter 8** presents the parametric study carried out by extending the finite element analysis. This study improved the flexure-torsion interaction relationship diagrams for both experimental series I and II by including more load combinations. Other parameters such as the span length of the composite steel-concrete beams, the horizontal curvature of the curved in plan composite steel-concrete beams and the different levels of shear connection have been introduced to this parametric study as well.
Chapter 9 will conclude all the experimental and analytical results in this thesis. Findings are summarised in this chapter with recommendations and suggestions being provided for future studies.

1.8 SUMMARY

This chapter has presented an overview of the research work carried out in this thesis. The reasoning behind this thesis topic has been explained in the introduction of this chapter. The methodology and the scope of this thesis are clearly defined and the general layout of the content in each chapter of this thesis has also been briefly described.
Figure 1.1 Cross-sectional view of a typical composite steel-concrete beam
Figure 1.2 Construction methods (a) and (b), (Uy and Liew, 2003)
Figure 1.3 Construction methods (c) and (d), (Uy and Liew, 2003)
Figure 1.4 Grosvenor Place, Sydney, Australia, (Uy and Liew, 2003)

Figure 1.5 Curved in plan highway girder beam
Edge beams

Figure 1.6 Edge beams, (Uy and Liew, 2003)

Straight beams

Figure 1.7 Western Distributor Highway, Darling Harbour, Australia
Figure 1.8 Ultimate sagging moment, plastic neutral axis in concrete slab

Figure 1.9 Ultimate sagging moment, plastic neutral axis in steel beam

Figure 1.10 Ultimate sagging moment, PSC
Figure 1.11 Ultimate hogging moment, plastic neutral axis in concrete slab

Figure 1.12 Ultimate hogging moment, plastic neutral axis in steel beam

Figure 1.13 Ultimate hogging moment, PSC
CHAPTER 2

2 LITERATURE REVIEW

2.1 INTRODUCTION

This chapter presents a detailed literature review, which is relevant to the content of this thesis. A summary of important findings and conclusions from past and present research studies is presented in this chapter. Deficiencies from the previous research studies are also identified and highlighted.

This literature review has looked into the history of composite steel-concrete beams to understand the need for composite steel-concrete construction in building designs. Issues such as the level of shear connection and the loading combinations of the composite steel-concrete beams are investigated in detail. The shear connection is provided by mechanical shear connectors which allows the transfer of longitudinal shear forces at the concrete slab and steel beam interface and also resists the vertical uplift force in the event of torsion loading. These connectors ensure the two different materials act as a single unit where the compressive strength of the concrete slab and the tensile strength of the steel beam are utilised. The different load combinations especially the combination of torsion and flexure acts on the composite steel-concrete beams have also significantly affected the behaviour of the composite steel-concrete beams and their strength capacities.
2.2 HISTORY OF COMPOSITE STEEL-CONCRETE BEAMS

Composite steel-concrete beam construction was first developed as a means of fireproofing the steel support beams in suspended concrete slabs. The concrete encasement was not designed as a contributing component to the load bearing capacity of the steel beams. This was because while the reinforced concrete had been patented in 1854, it was not until the early 1900s that the use of its tensile strength was widely recognised. It was then that the contribution of concrete encasement to the load bearing capacity was appreciated. Even though, reinforced concrete design formulas in use during that time did not take into account for reinforcement as large as a steel beam, there was a general understanding that these composite steel-concrete beams were stronger than their non-composite steel-concrete beam counterparts.

Scott (1925) began with experimental tests on concrete encased steel beams at the National Physical Laboratory in London to quantify the widely held belief that concrete encasement contributed to the load bearing capacity of the steel beams. He concluded that the concrete encasement provided did increase the flexural strength of the steel beams. He also proposed an empirical formula for the calculation of the first composite steel-concrete beam for its ultimate flexural moment capacity.

Caughy and Scott (1929) later conducted further testing on these composite steel-concrete beams. They rearranged the order of the concrete and the steel components. Instead of having the steel beam being fully encased in concrete, they rested a concrete slab on the top flange of the steel beam. At the end of the experimental tests, they observed differential longitudinal displacements developed between the top flange of the steel beam and the concrete slab. They suggested that
there should be some form of mechanical shear connectors provided to prevent these interface longitudinal displacements.

By 1930, many of these composite steel-concrete structures had been built, (Knowles, 1973). The use of prestressing in the design of composite steel-concrete bridges to induce upward cambering of the steel beam to reduce the deflection of the bridges was also introduced by Knight (1934). After twenty years since composite steel-concrete beams were introduced, the structural behaviour was well understood. Many countries developed their own design codes based on either the American or German codes. Researchers now turned their attention to the design of the shear connectors. Viest et al. (1952) started a series of tests on composite steel-concrete beams focusing on the design of the shear connectors. He focused on the use of steel channel section shear connectors first before he focused on steel headed shear studs later in Viest (1956).

With a greater understanding of the behaviour of shear connectors and the composite steel-concrete beams, research studies started to intensify. Investigation of nonlinear design methods for composite steel-concrete beam systems began. The introduction of computers drew researchers towards various convergence methods that until this stage had involved calculations too laborious to be considered practical, (Knowles, 1973). Researchers then began to focus on the torsional behaviour of composite steel-concrete beams and attempted to further optimise the design methods to maximise the economic benefits of the concrete steel-concrete construction method that was rapidly becoming the norm in multi-storey building and bridge construction, (Knowles, 1973).
By 1990, composite steel-concrete beam construction was commonly used in all forms of structures. Designers started to use finite element packages to analyse and design complex composite steel-concrete structural systems. Nevertheless, more refinements and extensions are needed in the studies of composite steel-concrete construction to enable designers to deal with increasing complex structures in the future.

2.3 SHEAR CONNECTOR DESIGN RULES

The connection between the steel beam and concrete slab is in the form of mechanical shear connectors, which allow the transfer of the longitudinal shear forces in the concrete slab to the steel beam and vice versa, and which also prevent vertical separation of the concrete slab and the steel beam. There are several forms of mechanical shear connectors available in the market as shown in Figure 2.1. The most commonly used shear connector is the headed shear stud, which consists of a head and a plain shank connected to the top flange of the steel beam by a weld collar as illustrated in Figure 2.2.

The design of shear connection on composite steel-concrete beams is governed by the individual standards all over the world. Since there two different regions such as the hogging and sagging moment regions in continuous beams as stated in Chapter 1, there are different methods in calculating the number of shear connectors required for different levels of shear connection.

Earlier versions of the Eurocode 4 (British Standards Institution, 1985) permitted the use of PSC over the entire length of the continuous composite steel-concrete beams. However, changes were made to the Eurocode 4 (British Standards Institution, 1990a) and the British code BS5950 (British Standards Institution,
1990b) stated that PSC was only permitted to apply in the sagging moment regions of the composite steel-concrete beams. The next revision of the Eurocode 4 (British Standards Institution, 1992) has introduced a limit range. For composite steel-concrete beams with a span not greater than 5 m, the minimum level of shear connection was limited to 40 % in the sagging moment regions ($\beta > 0.4$). For beams with a span greater than 25 m, they have to be designed with FSC. The current Eurocode 4 (British Standards Institution, 2005) has changed the criteria for the composite steel-concrete beam span to the distance in the sagging moment regions and provided different limitations depending on the steel flange and concrete slab dimensions. Nevertheless, composite steel-concrete beams with a sagging length less than or equal to 25 m must have greater than 40 % of overall shear connection ($\beta > 0.4$). Composite steel-concrete beams with a sagging length of more than 25 m must be designed with FSC ($\beta = 1.0$). Johnson (1994) identified the reason behind this conservative approach by the Eurocode 4, which was due to the simplification of neglecting the tensile strength of the concrete slab and the strain-hardening capacity of the reinforcement.

The current Australian Standard AS2327.1 (Standard Australia, 2003) specifies that the degree of shear connection not being less than 50 % for simply-supported beams ($\beta > 0.5$). Moreover, there is no available standard for designing continuous composite steel-concrete beams in Australia at the moment.

### 2.4 PARTIAL SHEAR CONNECTION

The level of shear connection is defined as the ratio of shear connection provided to the strength of the weakest element. The weakest element can be different for continuous composite steel-concrete beams where sagging and hogging
regions existed. Using the standard plastic theory for the determination of the ultimate flexural strength, for the sagging regions, the concrete slab is assumed to be fully crushed in compression and the steel beam is assumed to be fully yielded in tension or both tension and compression. Therefore, the weakest element in the sagging regions is either the concrete slab or the steel beam.

However for the hogging regions, the strength of the composite steel-concrete beams depends primarily on the strength of the steel reinforcement in tension and the steel beam in compression, as well as the level of shear connection. Therefore, partial shear connection can exist and it is depends on the steel reinforcing strength in tension rather than the concrete slab in compression as in the sagging regions. The weakest element is either the steel reinforcement or the steel beam. For most practical case, the steel reinforcement will be the weakest element in the hogging regions.

PSC is more economical than FSC with benefits include higher ductility, lower construction cost and higher rotation capacity for the composite steel-concrete beams. This statement has been proved by several researchers in the past and present. The following research papers have reinforced the statement with many experimental tests and analytical studies.

Crisinel (1990) tested three 6 m composite steel-concrete beams under sagging moments with different degrees of shear connection of 20, 40 and 56 %. From the test results, he concluded that despite a small loss of rigidity due to PSC, there was a huge increase in ductility for the PSC composite steel-concrete beams. He also suggested that for spans less than 20 m, the degree of shear connection
should be greater than 50%. For spans less than 15 m, 25% should be allowed to be used as a limit in the Eurocode 4 (British Standards Institution, 1990a).

Wright and Francis (1990) conducted tests with four 8 m composite steel-concrete beams with different levels of shear connection from 20 to 47%. They reconfirmed the potential benefits of using PSC in terms of ductility and strength when the experimental results showed better ductility for beams with lower level of shear connection without significant reduction in their flexural moment capacities.

PSC seems to be also recommended by researchers for composite steel-concrete joint structures. Aribert and Lachal (1992) conducted experimental tests using PSC in composite steel-concrete end plate joints. However, the ductility was limited due to the premature failure of the non-ductile cold-formed angles used as shear studs. Aribert (1996) used previous test results and input into his numerical simulation to show that there was an increase in interface slip for PSC design. This increase in longitudinal interface slip has caused an increase in the available joint rotation capacity of the composite end plate joints.

Bode et al. (1997) reported similar conclusions in their paper where a series of composite joint tests constructed with fin plate connections was tested. They concluded that PSC caused a reduction in stiffness and flexural moment capacities of the joints but achieved higher ultimate rotation capacities. Bode et al. (1997) also carried out tests on beam-to-beam joints with various levels of shear connection and spacing arrangements of shear connectors. They concluded that the longer the distance between the first shear connector and the joint connection, the greater the joint available rotation capacity would be.
Diedricks et al. (1999) supported the theory that PSC could lead to an increase in the joint rotational capacity. Two full-scale side plate composite steel-concrete joints were tested using the standard cruciform testing procedure to represent FSC and PSC. The experimental results indicated a reduction in the stiffness and strength of the joint but an increase in rotational capacity of the joint when PSC was adopted. Additionally, it was found that the rigid plastic model used in the paper gave a good prediction of the joint moment capacity.

Fabbrocino and Pecce (2000) conducted three inverted simply-supported composite steel-concrete beam tests under negative bending. Two beams had higher shear connection than the FSC design with different shear connection arrangements and one beam had 90% of FSC. However, the composite steel-concrete beam with 90% of shear connection achieved similar results in terms of strength, ductility and stiffness with the other two composite steel-concrete beams. The only noticeable difference was the high longitudinal interface slip observed for the composite steel-concrete beam with PSC.

Loh et al. (2004a) conducted extensive experimental tests to investigate the behaviour of eight composite steel-concrete beams subjected to hogging moment. Three of the beams were subjected to unidirectional, static loading and the remainder were loaded under constant range, quasi static cycles. The level of shear connection and the percentage of reinforcement were being varied. Loh et al. (2004a) concluded that the experimental results showed that it was beneficial to use PSC within the hogging regions of continuous and semi-continuous composite steel-concrete structures. Enhanced ductility was obtained through the increase in rotational capacity which is necessarily for plastic design. Another paper from Loh et al.
(2004b) developed an iterative based analytical model to consider the nonlinear material response of these composite steel-concrete beams and also provided a modified rigid plastic analysis approach to consider the effects of PSC.

Tan and Uy (2005) conducted a set of experiments on semi-continuous composite steel-concrete beams. The main objective was to investigate the effects of PSC on this form of structures. Two full-scale experiments were carried out with one designed for FSC and the other designed with 50 % of FSC. They concluded that the use of PSC resulted in higher ductility of the system when compared to the FSC beam without significant reduction in flexural capacity.

PSC has been accepted in both composite steel-concrete beam and joint designs. The importance of the serviceability limit state in some cases further highlighted the need for PSC, as it can increase the ductility and rotation capacities of the composite steel-concrete beams and connection joints. The need for providing sufficient rotational capacity to allow for plastic hinge to develop is important for plastic analysis as plastic analysis incorporates the assumption that any portion of a member containing a plastic hinge needs to possess sufficient rotation capacity to maintain the moment of resistance during the deformations necessary to develop the collapse mechanism of the structure, (Kemp, 1985).

2.5 PUSH-OUT TESTS

The behaviour of the shear connectors in terms of static and fatigue strengths is obtained by carrying out experimental tests such as push-out tests as stated in the Eurocode 4 (British Standards Institution, 2005). Push-out tests are a form of standard material test, which determine the load-slip relationship of the shear connectors used in the composite steel-concrete beam tests.
Carlsson and Hajjar (2000) concluded that the results from the composite steel-concrete beam tests were more difficult to analyse as the shear connectors were loaded by various forces and affected by residual stresses. The nonlinearity of the concrete slab and the steel beam also contribute to the complexity of the equation. Therefore, push-out test results often are more conservative than the results of composite steel-concrete beam tests. This is because the shear connectors in the push-out tests are subjected only to direct shear. Therefore, the push-out tests are only used as material tests for determining the structural characteristic of the shear studs.

Oehlers (1990) pointed out that the results from the push-out tests could also varied widely due to the use of different sizes, shapes, material properties, arrangement of specimens, number of shear connectors and support restraints. The different failure modes further increased the scatter of results. Therefore, for each experimental test, there should be a new set of push-out tests with the same type of shear connectors used.

Fabbrocino et al. (1998) and Rex and Easterling (2002) concluded that the push-out tests described in Eurocode 4 (British Standards Institution, 1990b) were found suitable to model shear connectors in composite steel-concrete beams.

Lam and El-Lobody (2005) developed a numerical model using ABAQUS to simulate the push-out tests to investigate variations in concrete strength and headed shear stud diameter. The model has been validated against previous push-out test results and compared with data given from the current standards. They concluded that the finite element model provided a better understanding to the different failure
modes observed during the tests. The shear capacity of the headed shear studs in the concrete slabs could be determined using the finite element model.

More recently, trapezoidal profiled slabs are becoming increasingly more popular for high rise buildings when compared with solid slabs because they can achieve large spans with little or no propping and they require less concrete and plywood formwork. However, the profiles used to achieve these savings can have a detrimental effect on the shear stud behaviour. Mirza and Uy (2009a) developed a non-linear finite element model using ABAQUS to study the behaviour of the shear connectors on the structural performance, steel fibres are introduced to further augment the ductility and strength of the shear connector region in the concrete slab. The results obtained from the finite element analyses were verified against experimental results and indicated that the strength and load-slip behaviour were significantly influenced by the inclusion of steel fibres.

Mirza and Uy (2009b) also investigated the behaviour of headed shear studs for composite steel-concrete beams at elevated temperatures with the finite element program ABAQUS. As a result of elevated temperatures, the material properties changed with temperatures. It was concluded that finite element analysis showed that the shear stud strength under fire exposure was very sensitive and the profiled steel sheeting slabs exhibit greater fire resistance when compared with that of a solid slab as a function of their ambient temperature strength.

Even though there is a change in the loading procedure for the push-out tests in Eurocode 4 (British Standards Institution, 2005) where cycle loading is required, the push-out tests carried out in this thesis have followed the standard push-out tests.
from Eurocode 4 (British Standards Institution, 1992) where static loading was used instead.

The reasoning for the change of the loading procedure from static to cyclic loading for the push-out tests is that composite steel-concrete beams can be subjected to repeated loads or seismic loads that may force the shear connectors to a reverse shear load during unloading. In order to investigate the behaviour of the shear connectors, there is a need for a test that can simulate the actual behaviour of the shear connectors under reverse cyclic loading. Therefore, twenty-five cycles between 5 to 40% of the expected failure load is carried out to determine the shear connector’s behaviour under cyclic loading in the new Eurocode 4 (British Standards Institution, 2005). The shear capacity of the shear connector subjected to cyclic loading tends to be lower than those under static loading due to the degradation in the shear connector combined with concrete crushing.

2.6 COMPOSITE STEEL-CONCRETE BEAMS SUBJECTED TO COMBINED FLEXURE AND SHEAR

The contributions of the concrete slab and its composite action to the vertical shear strength of the composite steel-concrete beams are ignored in the current design codes. The general assumption for composite steel-concrete beams is that the web of the steel beam resists the entire vertical shear forces which can lead to conservative designs.

Baskar and Shanmugam (2003) investigated the effects of combined flexure and shear on composite steel-concrete plate girders. Six composite steel-concrete plate girders were tested to failure in order to study their ultimate load behaviour. Experimental tests were conducted with different depth to thickness ratios, flange
dimensions, span lengths, moment to span ratios and loading types. The results showed that the ultimate load carrying capacity and the tension field width of the composite steel-concrete plate girders were found to increase significantly compared that of bare steel girders. This effect was more pronounced in composite steel-concrete beams subjected to positive bending. The axial tension introduced due to the flexural moment prevented shear buckling of the web. They have also concluded that the composite action was more effective for girders with slender webs.

Liang et al. (2005) developed a 3-D finite element model to investigate the behaviour of composite steel-concrete beams subjected to combined flexure and shear. The finite element model using ABAQUS was validated with the experimental results from Chapman and Balakrishnan (1964). Doubly curved thin shell elements were used to model the concrete slab and the steel beam. The shear studs were modelled using truss elements. They concluded that with an increase in the flexure to shear ratio, the ultimate loading capacity decreased. The maximum shear capacity was increased by 85 % through composite action which was quite significant considering that the design were based on the assumption that the steel web alone resisted the entire vertical shear force. The proposed design model for moment-shear interaction relationship was presented in Equation 2.1.

\[
\left(\frac{M_{us}}{M_{uo}}\right)^6 + \left(\frac{V_{us}}{V_{uo}}\right)^6 = 1
\]  

(2.1)

where,

\[ M_{us} = \text{ultimate moment capacity of the composite steel-concrete beam in combined shear and bending} \]

\[ M_{uo} = \text{ultimate moment capacity of the composite steel-concrete beam in } \]
pure bending

\[ V_{us} = \text{ultimate shear capacity of the composite steel-concrete beam in combined bending shear and bending} \]

\[ V_{uo} = \text{ultimate shear capacity of the composite steel-concrete beam in pure bending} \]

2.7 COMPOSITE STEEL-CONCRETE BEAMS SUBJECTED TO COMBINED FLEXURE AND TORSION

In 1960, the number of research into the behaviour of reinforced concrete under torsion started to increase. Now with a clear understanding of the behaviour of reinforced concrete beams in torsion, researchers realised the importance of the similar understanding of the behaviour for composite steel-concrete beams subjected to torsion as well.

Kuo and Heins (1970) performed a series of tests in which composite steel-concrete beams were subjected to torsion. Based on the experimental results, an empirical formula was presented in Equation 2.2 for the determination of the torsional moment capacity for composite steel-concrete sections.

\[ T_p = \left(7.5\sqrt{f'_c}\right)1.5(K_e/t_e) \quad (2.2) \]

where,

\[ T_p = \text{ultimate torsional moment capacity of the composite steel-concrete section} \]

\[ f'_c = \text{characteristic compressive strength of the concrete} \]

\[ 7.5\sqrt{f'_c} = \text{modulus of rupture of the concrete} \]
\[ K_c = \text{torsional rigidity of the concrete slab} \]
\[ t_c = \text{thickness of the concrete slab} \]

Colville (1973) performed a series of tests on composite steel-concrete beams that were curved in plan to investigate the behaviour of headed shear studs. He hoped that it would provide design procedures for the use of this type of shear connectors on composite steel-concrete beams subjected to torsion. He modified the formula proposed by Kuo and Heins (1970) to account for the presence of flexure. He proposed the following Equation 2.3 for the calculation of composite steel-concrete sections subjected to combined torsion and flexure.

\[
T_p = \left( \frac{K_c}{t_c} \right) \sqrt{56.2 f_{c'} + \sqrt{f_{c'}(M/S)}} \tag{2.3}
\]

where,

\[ T_p = \text{ultimate torsional moment capacity of the composite steel-concrete section} \]
\[ f_{c'} = \text{characteristic compressive strength of the concrete} \]
\[ M = \text{applied bending moment} \]
\[ S = \text{plastic shear modulus} \]
\[ K_c = \text{torsional rigidity of the concrete slab} \]
\[ t_c = \text{thickness of the concrete slab} \]

Colville (1973) demonstrated a reasonably close correlation between the calculated theoretical values for ultimate member capacity and element stresses at service and measured loads. However, the measured deflections in three of the four beams exceeded the predicted values by at least 50% indicating that the application
of torsion to composite steel-concrete beams caused an increase in vertical deflection as yet to be accounted for in the theory.

The next significant set of tests on composite steel-concrete beams subjected to torsional loading was undertaken by Singh and Mallick (1977) at the Indian Institute of Technology, India. Singh and Mallick (1977) decided to further investigate the effects of torsion on composite steel-concrete beams carried out by Colville (1973). However, rather than testing curved in plan beams and allowing the curvature of the beams to induce the torque, Singh and Mallick (1977) applied the torque directly to the straight composite steel-concrete beams.

The formula from Hsu and Mo (1985) was used for the calculation of the torsional moment capacity of reinforced concrete beams. This formula proposed that the torsional reinforcement was provided in the form of closed hoops and longitudinal bars. Singh and Mallick (1977) suggested that the torsional moment capacity of a composite steel-concrete section could be expressed as the concrete torsional moment capacity with the addition of the torsional capacity of the steel beam as shown in Equation 2.4.

\[
T_u = T_{cu} + T_{tr} + T_j
\]  

(2.4)

where,

\( T_u \) = ultimate torsional moment capacity of the composite steel-concrete sections

\( T_{cu} \) = torsional moment capacity of the unreinforced concrete slab

\( T_{tr} \) = torsional moment capacity of the steel reinforcement

\( T_j \) = torsional moment capacity of the steel beam
\[ T_{cw} = \frac{3}{\sqrt{14.3x^5yf_{cy}}} \]  

(2.5)

where,

\[ x, y = \text{thickness and width of the concrete slab respectively} \]

\[ f_{cy} = \text{compressive strength of the concrete} \]

\[ T_{\nu} = 1.2x_1y_1(A_s f_{sy} / s) \]  

(2.6)

where,

\[ x_1, y_1 = \text{smaller and larger centre to centre dimensions of the closed hoops} \]

\[ A_s = \text{cross sectional area of the one leg of the closed hoops} \]

\[ f_{sy} = \text{yield strength of the reinforcing steel} \]

\[ s = \text{spacing of the closed hoops} \]

Singh and Mallick (1977) tested eight composite steel-concrete beams of identical cross sectional dimensions, four under pure torsion (T1 to T4) and four (FT1 to FT4) under varying ratios of torsion and bending. Both series of composite steel-concrete beams were provided with shear studs according to the design formula proposed by Colville (1973). In each case, the ratio of applied torque at failure to the ultimate torsional capacity was plotted against the ratio of applied moment at failure to the ultimate flexural moment capacity. The flexure-torsion interaction relationship diagram suggested an increase in the flexural moment capacity beyond the ultimate flexural moment capacity in the presence of torsion and an increase in the torsional moment capacity beyond the ultimate torsional moment capacity in the presence of flexure. Singh and Mallick (1977) also noted that longitudinal interface slip and vertical uplift of the concrete slab in relation to the steel beam was almost negligible. This would indicate close to full interaction between the steel and concrete elements.
when the shear studs were provided in accordance with the procedures outlined by Colville (1973).

Ghosh and Mallick (1979) further investigated the positive interaction relationship between flexure and torsion observed by Singh and Mallick (1977). Ghosh and Mallick (1979) chose to neglect the contribution of the steel beam to the torsional moment capacity of the composite steel-concrete section in their predictions of the ultimate torsional moment capacities of the specimens. Using the Hsu and Mo (1985) formula for reinforced concrete, they adopted a modified expression for the contribution of the reinforcing steel to the torsional moment capacity of the sections in Equation 2.7.

\[ T_{se} = \alpha x_1 y_1 \left( A_s f_{sy} / s \right) \]  

(2.7)

where,

\[ T_{se} = \] modified torsional moment capacity of the steel reinforcement
\[ \alpha = 0.66m + 0.33(y_1/x_1) \]
\[ m = A_{ls} / [A_s(x_1 + y_1)] \]
\[ A_s = \] cross sectional area of the one leg of the closed hoops
\[ A_{ls} = \] cross sectional area of the longitudinal reinforcement
\[ x_1, y_1 = \] smaller and larger centre to centre dimensions of the closed hoops
\[ f_{sy} = \] yield strength of the reinforcing steel

Six straight composite steel-concrete beams of identical cross sectional dimensions were tested, one (GT1) in pure torsion and the remaining beams (GFT1 to GFT5) under varying ratios of flexure and torsion. Once again the ratio of flexure to torsion was arbitrary. However, they noted that most of Singh and Mallick (1977)
were concentrated at $M/M_u$ values greater than 0.8, so they concentrated their results on the region of the interaction relationship curve where $M/M_u$ was less than 0.8. Ghosh and Mallick (1979) noted the difference between the two different failure modes, combined flexural torsional and pure torsional failures. The combined flexural torsional failure was characterised by the yielding of the steel beam, the concrete crushing at the top surface of the concrete slab and the development of diagonal cracks at the composite steel-concrete beam ends. The pure torsional failure was characterised by the development of diagonal cracks over the full length of the composite steel-concrete beam and a non-ductile failure. Ghosh and Mallick (1979) also noticed that the concrete flexural yield line on the top surface of the concrete was inclined at an angle similar to the angle of the diagonal torsional cracks.

Ghosh and Mallick (1979) observed a conspicuous increase in the torsional moment capacity in the presence of $M/M_u$ values of between 0.4 and 0.5. They concluded that load history and sequence of loading could be behind the variation in experimental results. They also concluded that there might be a complex stress distribution in a composite steel-concrete section in the presence of flexure that increased the tensile capacity of the concrete. The calculation of the tensile strength of concrete was an approximation at best and the true value was known to be highly variable throughout the sections, yet this value had a significant impact on the torsional moment capacity of the composite steel-concrete sections.

Ray and Mallick (1980) investigated the contribution of the steel beam to the torsional moment capacity of the composite steel-concrete sections and if the sequence of loading affected the interaction relationship of flexure and torsion as suggested by Ghosh and Mallick (1979). A series of six straight composite steel-
concrete beams of identical cross-sectional dimensions were tested. From the experimental results, Ray and Mallick (1980) found that whilst the contribution of the steel beam to the ultimate torsional moment capacity of the composite steel-concrete sections was small, it did however make a significant contribution to the stiffness of the composite steel-concrete sections in torsion. Another conclusion was that the failure mode of the composite steel-concrete beams was predominantly torsional for low $M/T$ ratios and flexural for higher $M/T$ ratios. An increase in torsional moment capacity as $M/M_u$ approached 0.4 was noticed, whilst there was a reduction in torsional capacity as $M/M_u$ increased from 0.6 to 0.8. Ray and Mallick (1980) concluded that the presence of flexure was beneficial when applied up to a certain level because it added compression to the top layer of the concrete slab. The authors also concluded that the load history and sequence have little impact on the behaviour of composite steel-concrete beams subjected to combined flexure and torsion.

Nie et al. (2000) summarised the previous papers regarding straight composite steel-concrete beams subjected to combined flexure and torsion with their own set of experimental tests. Their objective was to investigate the flexure-torsion interaction relationship diagrams. Four identical composite steel-concrete beams were tested under different ratios of flexural to torsional loading. During the tests, the ratio of $M/T$ loading was kept constant throughout the loading phase. Nie et al. (2000) also investigated the effects of having different amount of torsional reinforcements. They noticed similar trends with their failure modes of the composite steel-concrete beams as Ghosh and Mallick (1979). They concluded that in the presence of flexure, there was a 10 to 50% increase in torsional moment capacity of a section. However in the presence of torsion, there was no increase or decrease in
the flexural moment capacity of the composite steel-concrete beams. Nie et al. (2000) also proposed formulae for the interaction relationship curve for straight composite steel-concrete beams subjected to combined flexure and torsion as shown in Equations 2.8 to 2.10 and illustrated in Figure 2.3 where it was compared with other interaction relationship curves.

\[
(T/T_u)^2 = 1 + 3.17(M/M_u) \quad (2.8)
\]

when \(M/M_u \leq 0.65\) and \(T/T_u \geq 1.0\)

\[
(T/T_u) = 3.42 - 2.55(M/M_u) \quad (2.9)
\]

when \(M/M_u > 0.65\) and \(T/T_u > 0.6\)

\[
(M/M_u) - (T/0.6T_u) = 1.0 \quad (2.10)
\]

when \(M/M_u \leq 0.65\) and \(T/T_u \leq 0.6\)

where,

\[T_u\] = torsional moment capacity of the composite steel-concrete sections under pure torque

\[T\] = torsional moment capacity of the composite steel-concrete sections

\[M_u\] = flexural moment capacity of the composite steel-concrete sections under pure bending

\[M\] = flexural moment capacity of the composite steel-concrete sections

### 2.8 CURVED IN PLAN COMPOSITE STEEL-CONCRETE BEAMS

Composite steel-concrete beams that are curved in plan are likely to be found in the construction of modern highway bridges, interchanges and balconies in
building. As mentioned previously in Chapter 1, curved in plan composite steel-concrete beams are subjected to combined torsion and flexure. With adequate bracing, the composite steel-concrete beams can be restrained to prevent lateral-torsional buckling. However, there is a limit to the bracing that can be provided for long span curved in plan composite steel-concrete beams. Therefore, it is essential that the real behaviour of such curved in plan composite steel-concrete beams can be investigated with detailed experimental and analytical studies.

Shanmugam et al. (1995) first considered the behaviour of curved in plan steel I-beams. Two sets of I-beams with one comprised of rolled sections and the other of built-up sections were tested. Each beam was subjected to a concentrated applied load at an intermediate point when the beam was laterally restrained. The experimental results for both deformation and ultimate strength were found to be in good agreement with the corresponding values predicted from the models using ABAQUS. They concluded that the horizontally curved in plan steel beams showed a reduction in the ultimate flexural moment capacity with a decrease in the radius of curvature to span-length ratio. This paper provided the beginning for research into curved in plan composite steel-concrete beams.

Thevendran et al. (2000) published a study regarding the behaviour of curved in plan composite steel-concrete beams. The objectives of the study were to investigate failure modes, stress distributions and ultimate flexural moment capacity of composite steel-concrete beams. Five composite steel-concrete beams with different radius of curvature from zero (SP1) to 120 mm (SP5) were constructed and tested to failure. The study concluded that there was a drop in the vertical load carrying capacity with an increase in the radius of curvature. The failure mode
changed from a flexural failure mode for straight beams or small radius of curvature to a combined flexural and torsional failure mode for composite steel-concrete beams with large radius of curvature.

Due to the lack of experimental tests for curved in plan composite steel-concrete beams, a justification for more detailed testing to be carried out in this area to investigate the behaviour of curved in plan composite steel-concrete beams is evident. Different parameters such as the span length, the level of shear connection and torsional reinforcement have not been studied in great depth. To really understand the complexity of the stress distribution and the lateral-torsional buckling of the composite steel-concrete beam, more full-scale tests are required.

2.9 FLEXURE-TORSION INTERACTION RELATIONSHIP

Flexure-torsion interaction curve provides relationship between the flexural and torsional moment capacities of the composite steel-concrete beams subjected to combined flexure and torsion. From this interaction relationship, design model can be proposed for Standards or other calculation manuals.

From previous research, several flexure-torsion interaction relationships had been identified especially in the case of composite steel-concrete beams as illustrated in Figure 2.3. The earliest interaction curve was presented by Colville (1973), even though his focus in his research was to investigate the behaviour of the shear connectors in the curved in plan composite steel-concrete beams. He provided the earliest form of interaction relationship for composite steel-concrete beams subjected to flexure and torsion as shown in Equation 2.1.
\[
\left(\frac{M}{M_u}\right)^2 + \left(\frac{T}{T_u}\right)^2 = 1
\]  

(2.11)

where,

\[\begin{align*}
T_u &= \text{torsional moment capacity of the composite steel-concrete sections under pure torque} \\
T &= \text{torsional moment capacity of the composite steel-concrete sections} \\
M_u &= \text{flexural moment capacity of the composite steel-concrete sections under pure bending} \\
M &= \text{flexural moment capacity of the composite steel-concrete sections}
\end{align*}\]

However, since his focus was on the shear connectors, there were a limited number of beam specimens with different load combinations. The next set of interaction curve was proposed by Ray and Mallick (1980) with straight composite steel-concrete beams. Ray and Mallick (1980) refined the interaction curves from two previous studies done by Singh and Mallick (1977) and Ghosh and Mallick (1979) as shown in Figure 2.4. They noticed that there was a huge difference between each curves in the region where torque was applied to the beam specimens. More tests was required to identify the lack of understanding in this region and also that there was a difference between straight and curved in plan composite steel-concrete beams from the interaction curves in Figure 2.3.

Nie et al. (2000) also provided an interaction relationship curve based on straight composite steel-concrete beams. More load combinations of flexure and torsion were applied to the test specimens to provide a much more detailed interaction relationship for flexural and torsional moment capacities as illustrated in Figure 2.3. The equations of the curve were given in Equations 2.8 to 2.10.
With all the available flexural-torsion interaction relationships presented in this thesis, a conclusion can be made from Figures 2.3 and 2.4 that there are differences between each curves proposed from different researchers. The area of their studies was also limited to straight composite steel-concrete beams instead of curved in plan members. Therefore, this thesis provided the required tests and - parametric study for both straight and curved in plan composite steel-concrete beams subjected to combined flexure and torsion. Design models were obtained from the provided flexure-torsion interaction relationships to allow better understanding of the behaviour for composite steel-concrete beams.

2.10 ANALYTICAL STUDIES ON COMPOSITE STEEL-CONCRETE BEAMS

To avoid the huge cost and significant time requirements of conducting full-scale experiments, researchers have approached to finite element or numerical models to simulate the experimental tests. The availability of high speed computers and commercial finite element packages facilitate the development of these tools through refined 3-D finite element analysis. Several papers have already introduced modelling of composite steel-concrete beams, columns, connection joints and push-out tests under different loading scenarios.

Yam and Chapman (1968) conducted a series of numerical analyses to investigate the inelastic behaviour of composite steel-concrete beams. They produced a predictor-corrector method of step by step numerical integration. Several assumptions were needed for the analysis to run. These assumptions were the strain distribution has to be linear over the depth of the composite steel-concrete beams, the stress-strain curves for steel were the same for both tensile and compressive regions,
the concrete and steel have equal curvature at all points along the composite steel-concrete beams and there was no separation between the concrete and the steel. Using the numerical analysis, the results from the numerical analyses were in good agreement with their experimental results.

Using the elemental formulation from the empirical equation of Yam and Chapman (1968), Razaqpur and Nofal (1989) modelled the shear connectors. Shear connectors were commonly represented as rigid or elastic springs, a smeared layer or just being ignored. The proposed element was a 2-D element with two end nodes and three translational degrees of freedom at both ends. After comparison with the experimental results of Yam and Chapman (1968), Razaqpur and Nofal (1989) concluded that the 2-D bar element was suitable for composite steel-concrete beams to consider torsional and shear loading.

Thevendran et al. (1999) used the software package ABAQUS to develop a 3-D finite element model to predict the load-deflection behaviour of the curved in plan composite steel-concrete beams. The model was then validated with the experimental results from Thevendran et al. (2000). The concrete slab was modelled using four-node isoparametric thick shell elements with the coupling of bending and membrane stiffness. In compression, the concrete was modelled as an elastic-plastic material with strain hardening. The uniaxial stress-strain curve was expressed in Equation 2.12.

\[ f_c = f' \left( 2 \varepsilon_c / \varepsilon_0 - (\varepsilon_c / \varepsilon_0)^2 \right) \] (2.12)

where,

\( f' \) = cylinder compressive strength of the concrete
\[ \varepsilon_0 = \text{strain corresponding to the maximum compressive stress} \]
\[ = 0.002 \]
\[ \varepsilon_c = \text{strain at which the concrete crushes} \]
\[ = 0.0038 \]

In tension, the concrete was expressed by a bilinear curve where stress increased linearly with strain up to the maximum tensile stress and then descended linearly to zero with an increasing strain. This constituted a strain softening model for the concrete. As for the rest of the steel components, strain hardening elasto-plastic materials were modelled. They concluded that the model provided a good agreement with the experimental results.

Sebastian and McConnel (2000) developed a finite element program with the ability to model composite steel-concrete beams with profiled steel sheeting. Concrete was modelled nonlinearly as an elastic isotropic material, whilst post-cracked concrete was modelled as an orthotropic material. Steel was modelled as an elasto-plastic material with strain hardening. For verification, the finite element program was used to model a reinforced concrete slab, a continuous composite steel-concrete beam, a bridge and a space truss bridge. The program provided good agreement in predicting the internal deformations, shear stud actions and crack patterns for the concrete slab.

Pi et al. (2006) developed a total Lagrangian finite element model to model the nonlinear inelastic analysis for both composite steel-concrete beams and columns. The interface slip between the steel and concrete components due to PSC was considered as an independent displacement in the formulation. This model provided excellent numerical performance for nonlinear inelastic analysis of
composite steel-concrete members when comparisons were made with other experimental and numerical results.

Erkmen and Bradford (2009) further extended the 3-D elastic total Lagrangian formulation from Pi et al. (2006) for the numerical analysis of composite steel-concrete beams which were curved in plan. Geometric nonlinearities were considered in the derivation of the strain expressions. The partial interaction at the concrete-steel interface in the tangential and radial directions due to the flexible shear connectors was incorporated in the proposed formulation. Comparing with the experimental and ABAQUS results, the developed formulation has captured only the fundamental behaviour of the composite steel-concrete beams curved in plan.

2.11 SUMMARY

This chapter has presented a detailed review of the research literature which is important to the objectives of this thesis. Limited experimental work and the lack of research of the issue of PSC for composite steel-concrete beams subjected to combined torsion and flexure is evident. Moreover, curved in plan composite steel-concrete beams are often replaced by straight beams in construction to avoid the check for the torsional moment capacity.

Findings from the research carried out for composite steel-concrete beams subjected to combined loading have shown that the force interaction relationships for reinforced concrete or steel beams are not suitable for composite steel-concrete beams. Independent force interaction relationships need to be developed for the use of composite steel-concrete members. Proposed interaction relationship diagrams have been provided in this chapter. However, due to the lack of test specimens, insufficient co-ordinates were able to be plotted onto the graphs. There are also large
differences between each of the proposed interaction relationship diagrams. More test specimens are needed and PSC should also be considered since it has been widely used in most construction designs. This thesis has considered both straight and curved in plan composite steel-concrete beams subjected to combined torsion and flexure. PSC has been also considered into the testing.

As for the existing analytical work done on composite steel-concrete beams, there have been several methods used previously to model the composite steel-concrete beams such as the finite element and numerical analyses. Nevertheless, the analytical studies on composite steel-concrete beams under torsion are less established. This thesis has presented a nonlinear finite element model using an ABAQUS model capable of considering composite steel-concrete beams subjected to combined torsion and flexure and incorporating the effects of PSC.
Figure 2.1 Mechanical shear connectors
Figure 2.2 Headed shear studs
Figure 2.3 Flexure-torsion interaction relationship curves
Figure 2.4 Flexure-torsion interaction relationship for straight composite beams

Ray and Mallick (1980)
Singh and Mallick (1977)
Ghosh and Mallick (1979)
CHAPTER 3

3 EXPERIMENTAL SERIES I - STRAIGHT COMPOSITE STEEL-CONCRETE BEAMS

3.1 INTRODUCTION

This chapter presents the first experimental stage of a wider investigation into the behaviour of composite steel-concrete beams subjected to combined flexure and torsion, in particular the initial development of a rational design method for curved in plan members. The experimental series I of this thesis required the construction of a series of six composite steel-concrete beams, three concrete slabs and three steel I-beams. All test specimens were tested to failure under varying combinations of flexural and torsional loading. This chapter outlines the design and construction of the test specimens, the test setup, the instrumentation employed to measure the strength responses, test observations and results. This experimental series I also provides the benchmark for experimental series II in terms of the design of the composite steel-concrete sections and the level of shear connection.

3.2 DETAILS OF TEST SPECIMEN

3.2.1 Beam test specimens

Six simply supported composite steel-concrete beams CBF-1, 2, 3 and CBP-1, 2, 3 were tested under combined flexure and torsion. CBF-1, 2 and 3 were
designed with FSC, whilst CBP-1, 2 and 3 have been designed with 50 % degree of shear connection. Each of the test specimens was 4.6 m in length and simply supported at a span of 4 m. The steel beams adopted were universal beam sections of a 200UB29.8 cross-section. The concrete slab have a thickness was 120 mm with a width of 500 mm. The steel beam was connected to the concrete slab by 19 mm nominal diameter headed shear studs. The detailed geometries of all the composite steel-concrete test specimens are showed in Figures 3.1 to 3.6 with their cross-sectional, plan and elevation views.

Three concrete slabs CS-1, 2 and 3 with similar dimensions as the composite steel-concrete beams without the steel beams were tested. The same reinforcement arrangement has been used in the concrete slab as shown in Figures 3.7 and 3.8. Three steel beams SB-1, 2 and 3 were also tested for combined flexure and torsion loading. Their detailed geometries are as shown in Figures 3.9 and 3.10. The material, geometric and design details of all composite steel-concrete beams, concrete slabs and steel beams are summarised in Table 3.1.

3.2.2 Push-out test specimens

Push-out tests were conducted to determine the load-slip characteristics and the ultimate shear capacity of the headed shear studs used in the composite steel-concrete beam experimental tests. Four push-out tests were carried out in this experimental series I. Two push-out tests (PT-F1 and PT-F2) were carried out to determine the load-slip characteristics for the FSC composite steel-concrete beams as shown in Figure 3.11, whilst the other two (PT-P1 and PT-P2) were carried out for the PSC composite steel-concrete beams as shown in Figure 3.12. They were all
designed according to the Eurocode 4 (British Standards Institution, 1992). The details for the push-out test specimens were summarised in Table 3.2.

3.3 TEST SPECIMEN FABRICATION

3.3.1 Beam test specimens

All beam test specimens were fabricated and assembled in the Structures Laboratory of the University of Wollongong. The steel beams were supplied with the web stiffeners installed at the end supports and mid span of the steel beams. Plywood formwork and shuttering were constructed to provide stability during the pouring of the concrete. Formwork supports were fabricated from the steel angles hinged and bent such that a horizontal surface and diagonal brace was formed when they were tack welded at regular intervals along both sides of the steel beams.

The concrete slab soffit forms were constructed using plywood cut to size and placed onto the steel form supports. Screws and right-angled steel brackets were used to connect the plywood side forms together. Three beams were then aligned next to each other to provide better stability. Qualified stud fabricators were employed to install the shear studs onto the top flange of the steel beam. Silicone sealant was then applied to any gaps in the formwork before the formwork was swept out and a coating of hydraulic oil was applied to the inside of the formwork as a release agent when the formwork was removed.

The reinforcing bars were prefabricated into steel cages using steel wire ties before placing into the formwork as shown in Figure 3.13. Reinforcing bar chairs of 25 mm height were tied to the underside of the transverse reinforcing bars of the steel cages. U-shaped deformed bars of 12 mm were fabricated as lifting lugs. Each lifting
lug was tack welded onto the transverse reinforcing bar about 300 mm from the end of the formwork.

Standard cylinder molds were prepared for the concrete compressive and indirect tensile tests as shown in Figure 3.14. Concrete casting was carried out on the materials, beams and push-out specimens. Concrete was placed into the formwork using shovels and wheelbarrows. The concrete was then compacted using pneumatic vibrators. Exposed concrete surfaces were then hand finished before covering up with polythene sheets and cured under wet condition for 7 days. The formworks were then be stripped after 14 days from the concrete pour.

3.3.2 Push-out test specimens

The push-out specimens were also prefabricated. The universal 200UB29.8 steel beams were flame cut in two T-sections while the concrete slabs were cast horizontally as shown in Figure 3.15. This procedure was in accordance with the guidelines provided from the Eurocode 4 (British Standards Institution, 1992). Oehler (1990) stated that if the concrete slab was cast vertically, there will be a high risk of bad compaction at the base of the shear connectors. This poor compaction resulted in air voids appearing in the concrete slab which subsequently caused a reduction in the shear capacity of the shear connectors.

After 28 days from the time when the concrete was poured, the two T-sections were welded back together as shown in Figure 3.16.
3.4 INSTRUMENTATION

3.4.1 Beam test specimens

Strain gauges were used to measure the strains in the steel flanges and webs in the mid span and point load region. Those measurements were used to determine the neutral axis and the value of curvature at the locations of the composite steel-concrete beams. Most of the strain gauges have been located in the mid span of the composite steel-concrete beam. The strain gauges were also placed on the reinforcing bars to measure the strains of the reinforcement during the tests. They were placed around the mid span and immediate quarter span of the beam test specimens.

Three linear variable displacement transducers (LVDT) were used to measure the deflections at the 1/4, 1/2 and 3/4 of the composite steel-concrete beam span. The LVDT measured the vertical displacement of the composite steel-concrete beams. Another two LVDT were located at each end support of the composite steel-concrete beams to measure the longitudinal interface slip between the concrete slab and the steel beam during testing as shown in Figure 3.17.

Electronic inclinometers were also used to measure the end rotations of the composite steel-concrete beams at the end supports and the twist rotations at the applied point load regions. The end rotational inclinometers were placed at a distance to the support location due to the web stiffeners. The twist rotational inclinometers were placed at each end of the width of the concrete slab at the mid span of the composite steel-concrete beams.

Load cells were used in the tests to measure all the external forces acting onto the test specimens. A 900 kN capacity load cell was placed between the hydraulic
jack and the transfer beam to measure the total applied vertical load. Vertical support reactions were measured using two 450 kN capacity load cells placed between the supports and the bottom flange of the steel beams. Vertical torsional restrained reactions were measured using two 225 kN capacity load cells between the steel packers being used to restrain the concrete from twisting and the concrete slab.

### 3.4.2 Push-out test specimens

Four LVDT were aligned to each location of the shear studs in the concrete slab. They were used to measure the vertical concrete-steel vertical interface slip between the concrete slab and the steel beam at the shear stud locations as illustrated in Figure 3.18. The fifth LVDT was placed at one side of the concrete slab to determine the horizontal displacement of the concrete slab on the roller support. This LVDT checked for any large horizontal displacement of the concrete slab due to uneven vertical loading from the hydraulic jack.

### 3.5 TEST FRAME AND MACHINE

#### 3.5.1 Beam test specimens

The test set-up for the composite steel-concrete beam tests is illustrated in Figure 3.19. Each beam test specimen was supported by a roller system at one end and a pinned system at the other end at an actual span length of 4.1 m. A 1000 kN hydraulic jack was used to apply the vertical load onto a transfer beam. The transfer beam would then distribute the applied load onto the beam test specimens with two point loads. For beams subjected to combined flexure and torsion, their supports condition has to be different from those just under flexure. Since there would be reaction torque at the support ends due to the induced applied torque from the
transfer beams, the supports have to be restrained from twisting as illustrated in Figure 3.20. Steel packers were placed under one side of the concrete slab to prevent the downward displacement of the concrete slab. Load cell was placed under the placed to determine the reaction force used to restrain the supports from twisting.

The concrete slab tests have a similar set-up as the composite steel-concrete beam tests as shown in Figure 3.21. The loading rate was slower at a rate of 20 sec/mm since the concrete slab has a lower flexural moment capacity than the composite steel-concrete beam. As for the steel beam tests, special arrangements were made before carrying out the tests to simulate the steel beam component of the composite steel-concrete beam sections. In the composite steel-concrete beam sections, top steel flanges were restrained from any lateral and side-sway movements since they were restrained by the shear studs and the concrete slab. A lateral bracing frame system was constructed for this purpose as illustrated in Figure 3.22. Two lever arms were also constructed as well to induce torsion in the applied loading frame as shown in Figure 3.23.

3.5.2 Push-out test specimens

The set-up for the push-out tests was chosen according to the Eurocode 4 (British Standards Institution, 1992). The test specimens were rested on a pre-fabricated support frame. Since there were two parts of the concrete slab attached to a steel beam, one concrete slab was rested on a fixed support, whilst the other concrete slab was rested on a roller support as shown in Figure 3.24.
3.6 LOADING PROCEDURE

3.6.1 Beam test specimens

Displacement control was used in all the composite steel-concrete beam and steel beam tests with a loading rate of 80 sec/mm for the first hour. As for the concrete slab, a loading rate of 20 sec/mm was used initially since their strength capacities were lower than steel beams and composite steel-concrete beams. The loading rate was then increased progressively until the peak load of the composite steel-concrete beam test specimens was reached. The loading would continue to determine the ductility of the test specimens. It would be terminated when either the maximum stroke of the jack was reached or when the load capacity of the beam test specimens dropped significantly.

Point loads were applied at different eccentric locations at the 1/3 and 2/3 of the beam span to induce different levels of combined torsion and flexure. Flexure was induced to the beams by the application of two equal vertical force acting downwards on the top surface of the beam test specimens. These loading points were located 675 mm either side of the mid span for all beam test specimens. As for applying torque to the beam test specimens, the vertical point loads were shifted to location either side of the longitudinal centroid of the beams. Vertical reaction forces were provided at each end support to restrain the transverse rotation due to the eccentric loading.

Test specimens CBF-1, CBP-1, CS-1 and SB-1 were tested under pure flexure. The point loads were applied at the mid span as line loads extending over the full width of the test specimens. Test specimens CBF-2, CBP-2, CS-2 and SB-2 were loaded 100 mm either side of the longitudinal centreline of the concrete slabs. Test
specimens CBF-3, CBP-3, CS-3 and SB-3 were loaded at an eccentricity of 175 mm from the longitudinal centreline of the concrete slabs.

3.6.2 Push-out test specimens

Displacement control was also used for the push-out test specimens. Vertical load was applied to the steel beam at a rate of 50 kN/min. The applied load induced shear force to the shear studs in both of the concrete slabs. The push-out test specimens were loaded until failure or when the specimens were unable to resist any additional loads. The test would also be terminated once one side of the concrete slab was detached or when the horizontal displacement became excessive.

3.7 MATERIAL TESTS

3.7.1 Concrete

Concrete tests comprised of standard cylinder compression and indirect tensile splitting tests. The tests were carried at the 7, 14, 21, 28 and the testing days of the beam and push-out test specimens. An 1800 kN AVERY compression testing machine was used to apply a compressive force at a loading rate of 900 kN/min onto the concrete cylinders.

3.7.1.1 Cylinder compression tests

The cylinder compression tests were carried out according to AS1012.9 (Standards Australia, 1999) to determine the compressive strength of the concrete used in the experimental series I. The prepared cylinders would be measured to determine their dimensions of diameter and height. Rubber cap was used for the
tests. Two samples were tested each day to provide an average compressive strength of the concrete. The test results are summarised in Table 3.3.

From Table 3.3, sample 5 has been ignored in the calculation of average strength due to premature fracture observed during the tests. Since the beam test specimens were tested on and after 50 days, whilst the push-out test specimens were tested on and after 78 days. Therefore, the average concrete compressive strength after 28, 50 and 78 days were 26.2, 26.7 and 27.8 N/mm$^2$ respectively.

3.7.1.2 Splitting tests

The splitting tests were carried out according to AS1012.10 (Standards Australia, 2000) to determine the indirect tensile strength of the concrete used in experimental series I. The concrete cylinders were placed in a testing jig with their axes horizontal and central between the platens of the testing machine. The compressive load from the testing machine would apply an indirect tensile force to split the concrete cylinder along the longitudinal direction. The failure mean principal tensile strength $f_{ctm}$, in MPa was calculated from Equation 3.1.

\[
f_{ctm} = \frac{2P}{\pi L_s d}
\]  

(3.1)

where,

- $f_{ctm}$ = principal tensile strength of the concrete
- $P$ = applied load at failure
- $L_s$ = length of the specimen
- $d$ = diameter of the specimen
Two concrete cylinders were tested each day to provide an average tensile strength of the concrete except for 78 days where only one concrete cylinder was left for testing. The test results are summarised in Table 3.4. The average concrete tensile strength after 28, 50 and 78 days were 2.47, 3.23 and 3.21 N/mm² respectively.

3.7.2 Structural steel

Universal steel beams of 200UB29.8 cross-section with the manufacturer nominal yield strength of 300 N/mm² were used in the experimental series I. Nevertheless, standard tensile steel coupon tests were carried out to determine the yield stress for the steel flanges and webs of the steel beams used. The tensile tests were prepared and carried out according to AS1391 (Standards Australia, 1991) under a 500 kN INSTRON testing machine.

Four steel beam coupons were cut out from the flange and four from the web with a nominal thickness of 9.6 and 6.4 mm respectively. The dimensions of the prefabricated steel coupons were presented in Figure 3.25. During the tests, an extensometer of 50 mm gauge length was used to measure the elongation and also to control the strain rate. The steel coupons were measured to determine their actual dimensions of width and thickness. The test results are presented in Table 3.5 with the stress-strain curves plotted in Figures 3.26 and 3.27 for steel webs and flanges respectively. The yield stresses for the flange and web coupons were 341 and 355 N/mm² respectively. The ultimate stresses for the flange and web coupons were 495 and 510 N/mm² respectively.
3.7.3 Steel reinforcing bar

N12 reinforcing bars with a nominal diameter of 12 mm were used as longitudinal reinforcement in the concrete slabs, whilst R10 stirrup reinforcing bars were used as torsional reinforcement in the concrete slabs with a nominal diameter of 10 mm. The longitudinal reinforcing bars were Grade 500 \( (f_{sy} = 500 \text{ N/mm}^2) \) respectively and were standard deformed bars with longitudinal ribs. The stirrup reinforcing bars were Grade 400 \( (f_{sy} = 400 \text{ N/mm}^2) \).

Five 400 mm in length longitudinal reinforcing bars and five 400 mm stirrup reinforcing bars were cut out for tensile tests to determine their yield stress according to AS1391 (Standards Australia, 1991). The results are summarised in Tables 3.6 and 3.7 for longitudinal and stirrup reinforcing bars respectively. Stress-strain curves were plotted as illustrated in Figures 3.28 and 3.29 for longitudinal and stirrup reinforcing bars respectively too. As there was not clear defined yield plateau for the longitudinal reinforcing bars, the yield stress was taken as the 0.2 % proof stress of the stress-strain curves. The yield stresses for the longitudinal and stirrup reinforcing bars were 520 and 399 N/mm\(^2\) respectively. The ultimate stresses for the longitudinal and stirrup reinforcing bars were 663 and 501 N/mm\(^2\) respectively.

3.7.4 Shear studs

Shear studs in the composite steel-concrete beam tests were subjected to vertical axial force due to the uplifting of the concrete slabs from the steel beams when torque was induced to the composite steel-concrete sections. Therefore, it is logical to determine the tensile strength of the shear studs. The tensile tests adopted were based on AS1391 (Standards Australia, 1991). The shear studs were fabricated to the required dimension as illustrated in Figure 3.30 for the tensile tests. Five shear
studs were tested to provide a mean tensile strength for the shear studs used in the composite steel-concrete beam tests. Their dimension and the test results are summarised in Table 3.8. Stress-strain curves are plotted in Figure 3.31. The yield and ultimate stresses for the shear studs were 421 and 540 N/mm$^2$ respectively.

### 3.8 PUSH-OUT TESTS

Push-out tests provided the load–slip characteristics and the ultimate shear capacity of the shear studs that were used in the composite steel-concrete beam tests. For push-out tests 1 and 2 (PT-F1 and PT-F2), there were a total of eight shear studs, whilst for push-out tests 3 and 4 (PT-P1 and PT-P2), there were a total of four shear studs welded onto the steel beams. Load cell was used to determine the total load applied to the push-out test specimens. LVDT were used to measure the interface slip between the concrete slabs and the steel flanges at each shear stud location. The horizontal displacement was measured at the roller support using LVDT as well.

#### 3.8.1 Push-out test observations

The behaviour for each push-out test specimens was fairly similar among one another. Before each test, an initial load of between 10 to 20 kN at a loading rate of 0.4 mm/min was applied to the test specimens to check the test set-up and instrumentation. Only then, vertical load was reapplied to the test specimens at a loading rate of 1 mm/min. As higher load was applied, there was a separation between the concrete slab and the steel beam interface at the lower half of the specimens. This separation widened as the applied load increased. The applied load would increase until a loud sound was heard, indicating either the fracture of a shear stud shank or the weld around the bottom of the shear stud as illustrated in Figure 3.32. The load reading would start to reduce from that point onwards. The tests were
continued until further failure of the subsequent shear studs or the detachment of one of the concrete slabs as shown in Figure 3.33. The push-out test specimens were then removed from the test frame.

### 3.8.2 Push-out test results

For push-out test 1 (PT-F1), the total maximum load capacity of the shear studs reached a peak of 848 kN at an average interface slip of 2.5 mm. Therefore, the maximum load per shear stud was determined to be 106 kN. The load capacity remained similar until the vertical slip reached approximate 5.3 mm when failure occurred. Premature failure was observed and found to be caused by poor welding undertaken on one of the shear studs.

For push-out test 2 (PT-F2), the maximum load capacity of the shear studs has reached a peak of 870 kN at an average interface slip of 3 mm. Therefore, the maximum load per shear stud was determined to be 109 kN. The applied load remained until the vertical slip reached approximate 10 mm when stud shank fracture failure occurred.

For the push-out test 3 and 4 (PT-P1 and PT-P2), which represented the shear stud arrangement for PSC, their peak load capacities were 422 and 454 kN respectively. Therefore, the maximum loads per shear stud were 106 and 113 kN for PT-P1 and PT-P2 respectively.

The horizontal displacement of the roller support of the push-out tests was plotted as shown in Figure 3.34. All the push-out test specimens did not have excessive horizontal displacement during the tests, which provided an indication of
the success of the push-out tests. The maximum applied load for all push-out tests had been reached before the horizontal displacement reached 5 mm.

A summary of test results for the four push-out test specimens is presented in Table 3.9. The average maximum load had been calculated by dividing the maximum applied load with the number of shear studs in each test specimen. The average slip was determined by averaging the values recorded from the LVDT. A summary of results for all push-out tests is plotted in Figure 3.35. From Table 3.9 and Figure 3.35, we could conclude that there was not a great difference in the shear capacity of the shear studs for both the FSC and PSC. However, PSC push-out test specimens had provided significant ductility compared to FSC push-out test specimens as illustrated from Figure 3.35, where the interface slips at fracture were higher for PSC push-out test specimens.

The phenomenon where the ductility of the partial shear connection specimens is much lower than those with full shear connection has been well known and reported by Loh et al. (2004a & b). The results from the four push-out tests have reinforced this phenomenon as the average slip capacity of 10.4 mm for PT-P1 and PT-P2 were higher than those for PT-F1 and PT-F2 with a value of 5.5 mm. PT-P1 and PT-P2 were push-out test specimens designed based on the composite steel-concrete beams with PSC, whilst PT-F1 and PT-F2 were based on beams with FSC.

### 3.9 BEAM TESTS

This section provides the behaviour of straight composite steel-concrete beams, steel beams and concrete slabs subjected to combined torsion and flexure. Six composite steel-concrete beams, three steel beams and three concrete slabs were tested. Three composite steel-concrete beams (CBF-1, CBF-2 and CBF-3) were
designed with FSC and three composite steel-concrete beams (CBP-1, CBP-2 and CBP-3) were designed with PSC. Three steel beams (SB-1, SB-2 and SB-3) and three concrete slabs (CS-1, CS-2 and CS-3) were tested under different combinations of flexure and torsion. All test observations were recorded during the tests. In additionally, the behaviour of the beam test specimens was monitored by instrumentation such as strain gauges, LVDT, load cells and inclinometers.

3.9.1 Beam test observations

All test observations are referenced to the test specimens as viewed from the data logger. The left near side is the extent of the side face of the test specimens from the mid span to the left hand end of the test specimens when viewed from the data logger. Significant incidents such as first crack and separation between concrete slab and steel beam were recorded.

3.9.1.1 Composite steel-concrete beam CBF-1

In order to monitor the structural behaviour of the composite steel-concrete beams, large numbers of instrumentation sensors were used in these tests. The amount of electronic data collected exceeded that of any past tests carried out in the laboratory. Due to that reason, the internal memory of the data logger was filled prior to the completion of the beam test for CBF-1. No warning was given by the data logging program and it continued to sample data into the memory. Only 1/5 of the test results were able to be retained. The remaining data was lost and unable to be recovered. To prevent similar incident from happening, an external memory card was used for the remaining tests.
For CBF-1, the first few flexural concrete cracks were noticed on the side faces of the concrete slab at an applied load of 283 kN. As the applied load reached 305 kN, these cracks extended to an average length of 50 mm with an interval spacing of 50 to 100 mm. Most of the concrete cracking occurred right underneath the applied load plates. Compressive crushing of the concrete was observed on the side faces of the concrete slab at an applied load of 310 kN. A major longitudinal crack opened on both side faces at a depth of 50 mm from the top surface of the concrete slab on both side faces at applied load of 317 kN. The applied load fell to 285 kN as this longitudinal crack propagated rapidly towards the mid span where it returned to the top surface. Later, a large chunk of concrete fell off due to the buckling of the steel reinforcing bars as shown in Figure 3.36. The test was then concluded as the longitudinal end rotation at the supports became severe. Local buckling of the top flange of the steel beam was observed beneath the left side point load.

3.9.1.2 Composite steel-concrete beam CBF-2

For CBF-2, the first few flexural concrete cracks were noticed at an applied load of 246 kN in the vicinity of the point loads. Compressive crushing of concrete slab also began beneath the left side circular load bearing plate. Concrete cracks extended outwards from either sided of the plate at an inclined angle and reached the sides at an applied load of 306 kN. At the ultimate applied load of 312 kN, the concrete cracks extended radically in both directions along the side face. A large plate of concrete fell over from the top layer of reinforcement at an applied load of 300 kN due to torsional shear-tension stress. The applied load continued to drop as the diagonal crack on the near side opened up and allowed the steel reinforcing bars to buckle. The test was concluded at the applied load of 200 kN and a mid span
deflection of 130 mm as the transfer beam had rotated excessively. The plane of compressive crushing through the concrete slab was at an inclined angle as illustrated in Figure 3.37.

3.9.1.3 Composite steel-concrete beam CBF-3

For CBF-3, the first few concrete vertical cracking of the side faces were observed at applied load of 110 kN. The concrete cracks extended up to a length of 60 mm with an interval spacing of 50 to 100 mm underneath the point loads. At the applied load of 215 kN, inclined punching shear concrete cracks were noticed and radial concrete cracks were formed on the top surface of the concrete slab around both the left and right circular load bearing plates. Localised punching shear failure of the concrete beneath the right side point load occurred at an applied load of 260 kN. The concrete cover over a length of 300 mm along the right far side fell off explosively due to punching failure of the concrete slab. The test could not be continued as the transfer beam toppled from the spherical seats at the point of failure. The beam was observed to have undergone 5 to 10 mm of plastic mid span deflection.

The composite steel-concrete beam was reused for retest with the different arrangement for the transfer beam. Larger load bearing plates measuring 150 by 300 mm were used to avoid the same premature punching shear failure of the concrete slab. Vertical concrete cracking of the side faces beneath the point loads commenced at an applied load of 204 kN. At the applied load of 258 kN, inclined punching shear concrete cracks appeared and extended from the bottom edge of the side faces towards the inside edges of the load bearing plates. Large radial concrete cracks formed on the top surface of the concrete slab around both the load bearing plates.
The right near side face concrete fell off in a plate extending from the inside edge of the right side point load to the mid span as the applied load reached 280 kN. At this point, inclined torsional concrete cracks were observed all over the surfaces of the test specimen in the region from the left side support reaction to the left side point load. The test was concluded as the twist rotation at the left side point load became severe. The web of the steel beam was also observed to have buckled beneath the left side point load.

3.9.1.4 Composite steel-concrete beam CBP-1

For CBP-1, the first few flexural concrete cracks were noticed on the side faces of the concrete slab at an applied load of 190 kN. Similar to CBF-1, the concrete cracking occurred right beneath the applied load plates. As the applied load reached 218 kN, the concrete cracks with interval spacing of 50 to 100 mm were more visible on the side faces of the concrete slab. At the applied load of 267 kN, concrete crushing was observed on the top surface of the concrete slab next to the point load plates. At the ultimate applied load of 273 kN, the mid span deflection reached 96 mm. The applied load fell to 241 kN with a mid span deflection of 191 mm before the test was concluded due to severe twist rotation. Longitudinal interface slip was observed at both the test specimen ends as shown in Figure 3.38.

3.9.1.5 Composite steel-concrete beam CBP-2

For CBP-2, the first few flexural concrete cracks were noticed at an applied load of 142 kN in the vicinity of the point loads. Concrete cracks extended from the sides to the point load plates at an inclined angle. Concrete crushing of the concrete slabs began beneath the left side load bearing plate when the applied load reached 277 kN. The ultimate applied load of 283 kN when the mid span deflection reached
Concrete cracks then extended radically in both directions along the applied load plates until a large plate of concrete fell off from the top and bottom layers of the reinforcement at the applied load of 279 kN due to the buckling of the steel reinforcing bars. The test was concluded due to the excessive concrete crushing at the left applied load at the applied load of 194 kN. Longitudinal interface slip was also observed at both the test specimen ends of CBP-2 but the slip was smaller than CBP-1.

3.9.1.6 Composite steel-concrete beam CBP-3

For CBP-3, the first few vertical flexural concrete cracks were noticed at an applied load of 102 kN in the vicinity of the point loads. Cracks with an interval spacing of 50 to 100 mm formed along the side faces of the concrete slab. These cracks extended at an inclined angle toward the load bearing plates of the test specimens. When the applied load reached 225 kN, the inclined crack reached the load bearing plates. Large radial concrete cracks formed at the top surface of the concrete slab around both the load bearing plates. The ultimate applied load of 255 kN was reached with a mid span deflection of 45 mm. The applied load remained the same as the inclined torsional concrete cracks were observed and the twisting of the concrete slab increased. The test was concluded when the twist rotation of the concrete slab became severe as illustrated in Figure 3.39. Small longitudinal interface slip was observed at both the test specimen ends of CBP-3.

3.9.1.7 Steel beams SB-1, SB-2 and SB-3

For SB-1, the steel beam was only subjected to pure flexure in the tests. The steel beam reached its maximum flexural moment capacity at an applied load of 175 kN. As the steel beam started to yield, the top flange of the steel beam started to
buckle due to the compression forces. Even though, the top flange of SB-1 had been restrained from any lateral displacement. There was still some regions of the steel beam that were not restrained by the restraining system employed as shown in Figure 3.40. Lateral buckling failure mode had limited the ductility of the steel beam under pure flexure as illustrated in Figure 3.41.

For SB-2 and SB-3, both the steel beams were subjected to combined flexure and torsion. However, the torsional moment capacity of open section steel beam is very low. Therefore, SB-2 and SB-3 reached a maximum applied load of 64 and 36 kN respectively. The top flange of the steel beam for both SB-2 and SB-3 started to twist when there was an applied load as shown in Figure 3.42. The test was concluded due to excessive twisting of the spherical loading plate.

3.9.1.8 Concrete slabs CS-1, CS-2 and CS-3

For CS-1, CS-2 and CS-3, their failure modes were similar. The concrete slabs failed due to concrete crushing at the mid span region for CS-1 and at the point load regions for CS-2 and CS-3 as shown in Figure 3.43. For all the concrete slab tests, they had reached similar maximum applied load of 26 kN. CS-2 and CS-3 have a slightly higher peak of 29 and 30 kN respectively. This increase could be due to the presence of torsion during the tests. Nevertheless, concrete cracking for all concrete slabs were around the mid span region indicating a flexural failure mode.

3.9.2 Beam test results

The detailed experimental results obtained for the six composite steel-concrete beams, three steel beams and three concrete slabs are presented in this
section. The following main aspects of the behaviour will be appraised. In each aspect, figures and tables are presented to justify and illustrate the findings.

1. Strength and deformation response in terms of load-deflection, moment-curvature and torque-twist of the beams.

2. Flexural strain distribution based on the mid span and the right point load strain gauge measurements at reinforcement and steel beams provided the neutral axes of the composite steel-concrete beam sections.

3. Longitudinal interface slip between the concrete slab and the top steel flange at both end supports of the composite steel-concrete beams.

3.9.2.1 Strength response for composite steel-concrete beams

Figure 3.44 shows the load-deflection curves for all the composite steel-concrete beams measured at the mid span. In terms of stiffness, the initial stiffness of the composite steel-concrete beams was similar, even though their levels of shear connection and the load combinations were different.

In terms of load-carrying capacity, the composite steel-concrete beams with different levels of shear connection were compared with each other such as CBF-1 with CBP-1, CBF-2 with CBP-2 and CBF-3 and CBP-3. From load-mid span deflection curves, CBF-1, CBF-2 and CBF-3 achieved load carrying capacities of 312, 315 and 286 kN respectively. A slight increase of 1 % or 3 kN was presented when CBF-2 was subjected to torsion compared to CBF-1. This trend was also observed even for composite steel-concrete beams with PSC. The load-carrying capacities for CBP-1, CBP-2 and CBP-3 were 273, 283 and 255 kN respectively. CBP-2 has an increase of 4 % or 10 kN when there was a presence of torsion compared to CBF-1.
When the same load combination was applied, composite steel-concrete beams with FSC had higher load-carrying capacities than those with PSC. However, the differences were not so large. For example, CBP-1 achieved 88 % of the load-carrying capacity of CBF-1, CBP-2 achieved 90 % of the load-carrying capacity of CBF-2 and CBP-3 achieved 89 % of the load-carrying capacity of CBF-3.

In terms of ductility, the governing criterions for the composite steel-concrete beams were the level of applied torque and their torsional moment capacities. Composite steel-concrete beams with no applied torque such as CBF-1 and CBP-1 have higher ductility compared to composite steel-concrete beams subjected to combined flexure and torsion. In addition, the composite steel-concrete beams that were subjected to more torque in their load combinations would fail at an earlier stage of the tests such as CBF-3 and CBP-3.

In terms of flexural moment capacity, similar behaviour is also indicated in the moment-mid span curvature curves for composite steel-concrete beams as shown in Figure 3.45. CBF-1, CBF-2 and CBF-3 achieved flexural moment capacities of 220, 214 and 197 kNm respectively, while CBP-1 and CBP-2 and CBP-3 achieved flexural moment capacities of 188, 194 and 177 kNm respectively. Therefore, when comparing composite steel-concrete beams with same load combination, composite steel-concrete beams with FSC had slightly higher flexural moment capacities than those with PSC. CBP-1 achieved 85 % of the flexural moment capacity of CBF-1, CBP-2 achieved 90 % of the flexural moment capacity of CBF-2 and CBP-3 also achieved 90 % of the flexural moment capacity of CBF-3. Therefore, there was just a reduction of less than 10 % of the flexural moment capacity even with a reduction of 50 % of the degree of shear connection provided.
Using the rigid plastic analysis method, initial estimates of the flexural moment capacities of 218 kNm for FSC and 182 kNm for 50% of the shear connection under pure flexure were calculated. Those calculated values proved to be similar to the test results of 220 and 188 kNm for CBF-1 and CBP-1 respectively. The rigid plastic analysis method has further reinforced the test results obtained from the experimental series I.

Figure 3.46 shows the torque-twist curves for CBF-2, CBF-3, CBP-2 and CBP-3 which had combined flexure and torsion applied during the tests. As mentioned before, composite steel-concrete beams with a higher torque will fail at an earlier stage of the tests when CBF-3 and CBP-3 were compared with CBF-2 and CBP-2. In terms of the torsional twist, there were higher torsional twists for composite steel-concrete beams with FSC than those with PSC. This could be explained by the excessive concrete cracking for composite steel-concrete beams which disturbed the measurement from the inclinometers on the concrete surface.

Table 3.10 summaries all the maximum strengths such as the load-carrying, flexural and torsional moment capacities of the beam test specimens.

3.9.2.2 Strength response for concrete slabs

From Figures 3.47 and 3.48, the load-carrying and flexural moment capacities for all three concrete slabs are similar even though they were loaded differently. The load-carrying capacities for CS-1, CS-2 and CS-3 were 26, 29 and 30 kN respectively. In addition, the flexural moment capacities for CS-1, CS-2 and CS-3 being 19, 21 and 21 kNm respectively. This indicates that all three slabs achieved their maximum flexural moment capacities before they failed. It is obvious for CS-1 because it was subjected to pure flexure only. But interestingly, CS-2 and CS-3
achieved a slightly higher flexural moment capacity than CS-1. The reason behind this could be that with the presence of torsion, the flexural moment capacities increased slightly for both CS-2 and CS-3.

From Figure 3.48, the curvature of CS-2 and CS-3 is going to the opposite face of the diagram, this indicating the torsional stress of the concrete slabs affecting the longitudinal stress of the concrete slabs. Since the strain measurements were taken from the strains of the longitudinal reinforcement in the concrete slabs, the negative strains indicated that the reinforcement has changed from compression to tension to resist the applied torque. The tensile stress from the reinforcement had counter the compressive section of the concrete slabs. This was more obvious when more torque was applied to CS-3. Therefore, it could be the twisting of the concrete slab induced reinforcement tensile stress to counter the compressive stress in the concrete slab sections. This counter stress increased the flexural moment capacity for CS-2 and CS-3.

From Figure 3.49, both concrete slabs CS-2 and CS-3 have reached a maximum of 1.6 and 2.9 kNm of torque respectively. However from basic calculations, the upper bound and the lower bound torsional capacities of the concrete slabs were 10.9 and 6.3 kNm respectively. Therefore, there is still a reserve of torsional capacity available for CS-2 and CS-3. This also indicated that the failure criterions for CS-2 and CS-3 were their flexural moment capacities of the concrete slabs.

3.9.2.3 Strength response for steel beams

In the case of the steel beam tests, both SB-2 and SB-3 could not utilise the maximum section flexural moment capacity of 108 kNm from SB-1 which was
subjected to pure flexure. The reason is that both SB-2 and SB-3 have to resist torsion loading. Since open steel sections such as I-beams are weak in torsional moment capacity, they are prone to failure due to applied torque. From Figures 3.50 and 3.51, SB-2 and SB-3 only achieved 64 and 36 kN for their load-carrying capacities and 38 and 22 kNm for their flexural moment capacities respectively. But from Figure 3.52, both SB-2 and SB-3 have reached their maximum torsional capacities of 3.3 and 3 kNm respectively, which is similar to the theoretical calculated torsional capacity value of 3 kNm. Therefore under combined flexure and torsion, the concrete slabs tend to fail in bending, whilst the steel beams tend to fail in torque.

3.9.2.4 Neutral axis for composite steel-concrete beams

The strain distributions for all composite steel-concrete beams are plots of the measured internal strains at several arbitrary applied loads against the structural depth of the respective strain gauges. The concrete and steel strains were plotted as separate series and a linear trendline was fitted to each in order to detect any difference in curvature between the two elements. When the strain distribution is continuous across the steel-concrete interface, the section is assumed to have full composite action. The neutral axis is the point at which the strains are zero. The depth to the neutral axis can be approximated from the strain distribution as the depth to the point of the intersection of the strain profiles for all applied loads.

From Figure 3.53, the mid span strain distribution for CBF-1 is continuous across the depth of the section, indicating full composite action. The neutral axis occurred at 217 mm from the base of the section and it was located in the concrete slab. However from Figure 3.54, the right point load strain distribution is
discontinuous at the concrete-steel interface, indicating partial composite action. The strain profiles of the concrete and steel elements have the same slope for each load increment, indicating that the discontinuity is due to longitudinal slip between the two elements. Due to the longitudinal slip, there are two neutral axes for steel and concrete at the depth of 187 and 238 mm from the base of the section respectively. This indicating that even with FSC design, there are longitudinal slips at some sections of the composite steel-concrete beams.

From Figures 3.55 and 3.56, the strain distribution for CBF-2 is similar in form to the strain distribution of CBF-1. The mid span strain distribution indicates full composite action and the neutral axis locates at the 213 mm from the base of the section. The right point load strain distribution indicating partial composite action with the neutral axes of 222 and 197 mm from the base of the section for concrete and steel elements respectively. However, the slip strain is less than CBF-1 at the same point.

From Figure 3.57 and 3.58, the mid span strain distribution for CBF-3 is continuous indicating full composite action with a neutral axis of 250 mm from the base of the section. The right point load strain distribution has discontinuity indicating partial composite action. There are two neutral axes of 222 and 197 mm from the base of the section for concrete and steel elements respectively. The similarity of the neutral axes between CBF-2 and CBF-3 indicating similar failure mode for both test specimens. Both of the composite steel-concrete beams were subjected to combined flexure and torsion. Therefore, the conclusion is that both CBF-2 and CBF-3 have reached their ultimate torsional capacities which reduced their ductility and resulted lesser longitudinal interface slip than CBF-1
As for CBP-1, 2 and 3, from Figures 3.59 to 3.64, the strain distributions for both the mid span and right point load are discontinuous indicating partial composite action. Since, CBP-1, 2 and 3 were designed to have PSC, it is a norm to have longitudinal interface slip at the concrete-steel interface.

3.9.2.5 Longitudinal interface slip for composite steel-concrete beams

Longitudinal interface slip between the concrete slab and the top steel flange was measured by the use of LVDT and summarised in Table 3.11. Relative interface slip was measured when the maximum applied load was reached. As expected, the relative longitudinal interface slip for FSC registered an average of 1.3 and 4.3 mm for PSC. This reinforced the previous finding that even with FSC design composite steel-concrete beams, there is partial composite action at some sections of the composite steel-concrete beams. However, greater longitudinal interface slip was required when PSC was designed for composite steel-concrete beams. Another finding was that when the applied torque increased, the relative longitudinal interface slip reduced. This illustrated that composite steel-concrete beams reached their maximum load capacities earlier when the applied torque increased as shown in Figure 3.65.

3.10 SUMMARY

The chapter has presented the experimental results of a series of straight composite steel-concrete beams, concrete slabs and steel beams designed with varying levels of shear connection. The behaviours of the beams under the influence of varying shear connection levels and different levels of load combination have been evaluated. The following outlines the important findings obtained from the beam tests.
(1) The average ultimate shear capacities and the load-slip characteristics for both FSC and PSC push-out tests were similar even though different number of shear studs was used. Therefore, the number of shear studs did not affect the average ultimate load capacity of shear studs in the push-out tests based on Eurocode 4 (British Standards Institution, 1992).

(2) Even in the presence of applied torque to the composite steel-concrete beams, their load-carrying and flexural moment capacities did not decrease. There were only a slight increase of 1 to 4 % for the capacities in some cases of the composite steel-concrete beams subjected to combined flexure and torsion. However, in the presence of flexure, there was an increase in the torsional moment capacities for all composite steel-concrete beams.

(3) For composite steel-concrete beams using only 50% of shear connection, the reduction in their flexural moment capacities was less than 10 % compared to FSC composite steel-concrete beams. Therefore, this finding reinforced the use of PSC in composite steel-concrete beam designs.

(4) When composite steel-concrete beams and steel beams were subjected to combined flexure and torsion, their ductility was depended on the level of applied torque. The greater the applied torque, the lower the ductility would be. For example, CBP-3 and CBF-3 has failed at an earlier stage of the tests compared to CBP-2 and CBF-2 respectively. The reason is due to the higher applied torque for CBP-3 and CBF-3. Under combined flexure and torsion, the composite steel-concrete beams and steel beams tend to fail in torsion, whilst the concrete slabs tend to fail in bending.

(5) All three concrete slabs achieved their maximum flexural moment capacities before they failed. But interestingly, CS-2 and CS-3 achieved a
slightly higher moment capacity than CS-1. The reason could be that with
the presence of torsion, the flexural moment capacities increased slightly
for both CS-2 and CS-3.

(6) Even with FSC in composite steel-concrete beam designs, there was partial
composite action at some sections of the composite steel-concrete beams as
longitudinal interface slips had been found from their strain distributions.
However, greater longitudinal interface slip was required when PSC was
designed for composite steel-concrete beams.
### Table 3.1 Straight beam specimen details

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Beam</th>
<th>Degree of shear connection (%)</th>
<th>Stud section (19 mm studs)</th>
<th>Total number of studs</th>
<th>Stud spacing (mm)</th>
<th>Load eccentricity from centreline (mm)</th>
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<td>175</td>
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Number of specimens: 12
Concrete slab size: 4600 x 500 x 120 mm
Concrete strength: 32 N/mm²
Concrete cover: 25 mm
Steel beam: 200 UB 29.8 (BHP-300PLUS)
Longitudinal reinforcement: N12 – Grade 500 N/mm²
Transverse reinforcement: R10 - Grade 500 N/mm²
Shear studs: 19 mm headed studs
Table 3.2 Push-out specimen details

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Number of specimens 4

Concrete slab size 600 x 600 x 120 mm

Concrete strength 32 N/mm²

Concrete cover 25 mm

Steel beam 200 UB 29.8 (BHP-300PLUS)

Reinforcement N12 – Grade 500 N/mm²

Shear studs 19 mm headed studs
Table 3.3 Cylinder compression test results

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<th>Type of cap</th>
<th>Max. load (kN)</th>
<th>Compressive strength (N/mm$^2$)</th>
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Table 3.5 Structural steel tensile test results

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Table 3.7 Stirrup bar tensile test results

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### Table 3.8 Headed shear stud tensile test results

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Table 3.9 Push-out test results

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Table 3.11 Interface slips for straight composite steel-concrete beams

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Figure 3.1 Cross-sectional view for CBF-1, CBF-2 and CBF-3
Figure 3.2 Plan view for CBF-1, CBF-2 and CBF-3
Figure 3.3 Elevation view for CBF-1, CBF-2 and CBF-3
Figure 3.4 Cross-sectional view for CBP-1, CBP-2 and CBP-3
Figure 3.5 Plan view for CBP-1, CBP-2 and CBP-3
Figure 3.6 Elevation view for CBP-1, CBP-2 and CBP-3
Figure 3.7 Cross-sectional view for CS-1, CS-2 and CS-3
Concrete slab
Stirrups R10 @ 200 mm
N12 @ 150 mm

Figure 3.8 Plan view for CS-1, CS-2 and CS-3
Figure 3.9 Elevation view for SB-1, SB-2 and SB-3
Figure 3.10 Cross-sectional views for SB-1, SB-2 and SB-3
Figure 3.11 Push-out test specimens for PT-F1 and PT-F2
Figure 3.12 Push-out test specimens for PT-P1 and PT-P2
Figure 3.13 Beam test specimens ready for concrete casting

Figure 3.14 Concrete cylinders
Figure 3.15 Push-out test specimens ready for concrete casting

Figure 3.16 Push-out test specimens welded together
Figure 3.17 LVDT for measuring the longitudinal interface slip

Figure 3.18 LVDT locations in push-out tests
Figure 3.19 Composite steel-concrete beam set-up

Figure 3.20 Restraining system at the support
Figure 3.21 Concrete slab set-up

Figure 3.22 Steel beam bracing system
Figure 3.23 Lever arm to induce torque for steel beams

Figure 3.24 Support conditions for push-out tests
Figure 3.25 Steel coupon cut-out
Figure 3.26 Steel web stress-strain curves
Figure 3.27 Steel flange stress-strain curves
Figure 3.28 Longitudinal reinforcing bar stress-strain curves
Figure 3.29 Stirrup bar stress-strain curves
Figure 3.30 Shear stud cut-out
Figure 3.31 Shear stud stress-strain curves
Figure 3.32 Various failure modes of the push-out tests

Stud shank failure

Weld failure
Figure 3.33 Detachment of the concrete slab in a push-out test
Figure 3.34 Load-horizontal displacements for push-out tests
Figure 3.35 Push-out test summarised results
Figure 3.36 Steel reinforcing bars buckling for CBF-1

Figure 3.37 Inclined concrete crushing for CBF-2
Figure 3.38 Interface end slip for CBP-1

Figure 3.39 Excessive twisting for CBP-3
Figure 3.40 Unrestrained steel top flange region for SB-1

Figure 3.41 Lateral buckling of top steel flange for SB-1
Figure 3.42 Twisting of the lever arm for SB-2

Figure 3.43 Concrete crushing of concrete slab for CS-2
Figure 3.44 Load-mid span deflection for straight composite beams
Figure 3.45 Moment-mid span curvature for straight composite beams
Figure 3.46 Torque-twist for straight composite steel-concrete beams
Figure 3.47 Load-mid span deflection for straight concrete slabs
Figure 3.48 Moment-mid span curvature for concrete slabs
Figure 3.49 Torque-twist for concrete slabs
Figure 3.50 Load-mid span deflection for steel beams
Figure 3.51 Moment-mid span curvature for steel beams
Figure 3.52 Torque-twist for steel beams
Figure 3.53 Strain distribution at mid span for CBF-1
Figure 3.54 Strain distribution at right point load for CBF-1
Figure 3.55 Strain distribution at mid span for CBF-2
Figure 3.56 Strain distribution at right point load for CBF-2
Figure 3.57 Strain distribution at mid span for CBF-3
Figure 3.58 Strain distribution at right point load for CBF-3
Figure 3.59 Strain distribution at mid span for CBP-1
Figure 3.60 Strain distribution at right point load for CBP-1
Figure 3.61 Strain distribution at mid span for CBP-2
Figure 3.62 Strain distribution at right point load for CBP-2
Figure 3.63 Strain distribution at mid span for CBP-3
Figure 3.64 Strain distribution at right point load for CBP-3
Figure 3.65 End support interface slips for straight composite beams
CHAPTER 4

4 EXPERIMENTAL SERIES II - CURVED IN PLAN
COMPOSITE STEEL-CONCRETE BEAMS

4.1 INTRODUCTION

This chapter presents the second series of tests in the experimental programme. Experimental series II was designed as an extension to complement the set of composite steel-concrete beams described in Chapter 3. The difference is that instead of having straight members, series II has looked into curved in plan members. The cross-sectional detail of the composite steel-concrete beams reminded the same. However, the span length of the composite steel-concrete beam test specimens has increase from 4 m to 6 m. One point load instead of two has been applied to the composite steel-concrete beams. In Chapter 3, the experimental results illustrated that PSC and the different levels of flexure and torsion loadings had significant effects on the performance of the composite steel-concrete beams. Therefore, the experimental series II of this thesis required the constructed of a series of eight curved in plan composite steel-concrete beams. All test specimens were tested to failure under varying combinations of flexural and torsional loadings even though a single point load was applied to the mid span. This chapter outlines the design and construction of the test specimens, the test setup, the instrumentation employed to measure the response during the tests and the test observations and results. This
experimental series II provides the final investigation into curved in plan composite steel-concrete beams and the different levels of shear connection provided.

4.2 DETAILS OF TEST SPECIMEN

4.2.1 Beam test specimens

Eight simply supported composite steel-concrete beams CCBF-1, 2, 3, 4 and CCBP-1, 2, 3, 4 were tested under combined flexure and torsion as summarised in Table 4.1. CCBF-1, 2, 3 and 4 were designed with FSC, whilst CCBP-1, 2, 3 and 4 have been designed with 50% degree of shear connection. Each of the test specimens was 6.2 m in length and was simply supported at a span of 6 m. The steel beams adopted were also the universal beam sections of a 200UB29.8 cross-section. The concrete slabs have a slab thickness of 120 mm with a width of 500 mm. The steel beam was connected to the concrete slab by 19 mm nominal diameter headed shear studs. However, the total number of shear connectors was different from the experimental series I due to the difference in the span length. Point loads were applied at mid span of the composite steel-concrete beams. Different span to the radius of curvature ratios were used to induce differing levels of combined torsion and flexure onto the composite steel-concrete beam. The typical geometry of all the test specimens is shown in Figures 4.1 to 4.6.

4.2.2 Push-out test specimens

Push-out tests were conducted to determine the load-slip characteristics and the ultimate shear capacity of the headed shear connectors used in the composite steel-concrete beam experimental tests. Four push-out tests were carried out. Two push-out tests (CBPT-F1 and CBPT-F2) were carried out to determine the load-slip
characteristics for the FSC composite steel-concrete beams as shown in Figure 4.7, whilst the other two (CBPT-P1 and CBPT-P2) were carried out for the PSC composite steel-concrete beams as shown in Figure 4.8. They were all designed according to the Eurocode 4 (British Standards Institution, 1992). The spacing of the shear connectors at each push-out test specimen was different compared to the push-out test specimens from the experimental series I. The spacing was 460 mm instead of 285 mm. The details for the push-out test specimens were summarised in Table 4.2.

4.3 TEST SPECIMEN FABRICATION

All test specimens were prefabricated by an external steel fabricator and assembled in the Structures Laboratory of the University of Wollongong. The I-section steel beams were cold-rolled to the required curvatures as indicated in the Table 4.1. The plywood formwork was then constructed around the profile of the steel beams’ curvatures. The reinforcing bars were prefabricated since it was hard to make steel cages to fix into the curved formwork as shown in Figure 4.9. 25 mm height chairs were tied to the underside of the transverse reinforcing bars of the steel cages. U-shaped 12 mm deformed steel bars were fabricated as lifting lugs. Each lifting lug was tack welded to the transverse reinforcing bar about 300 mm from the end of the formwork. Qualified stud fabricators were then employed to install the shear connectors onto the top flange of the steel beams. Silicone sealant was then applied to any gaps in the formwork before the formwork was swept out and a coating of hydraulic oil was applied to the inside of the formwork as a release agent when the formwork was removed.
Prefabricated formworks for the push-out specimens were placed onto the precut T-section of the steel beam. Silicone sealant was also applied to any gaps in the formwork before the coating of hydraulic oil. Standard cylinder and beam molds were prepared for the concrete compressive and indirect tensile tests.

Concrete casting was carried out on the materials, beams and push-out specimens. Concrete was placed into the formworks using a shovel and wheelbarrows. The concrete was then compacted using a pneumatic vibrator. The exposed concrete surfaces were hand finished before covering up with polythene sheets and cured under wet condition for 7 days as shown in Figure 4.10. The formworks were stripped 14 days after the concrete pour.

### 4.4 INSTRUMENTATION

Strain gauges were used to measure the strain in the steel flanges and webs in the region of the point load. Those measurements were used to determine the neutral axis of the steel beams. All the strain gauges have been located in the mid span of the beams. Strain gauges were even placed in the reinforcing bars to determine the strain distributions from the longitudinal reinforcing bars.

Three linear variable displacement transducers (LVDT) were used to measure the deflection at the 1/4, 1/2 and 3/4 of the composite steel-concrete beam spans. Another two LVDT were located at each end support of the composite steel-concrete beams to measure the maximum longitudinal interface slip between the concrete slab and the steel beam during testing.
Electronic inclinometers were used to measure the end rotations of the composite steel-concrete beams at the end supports and the twist rotations at the point loads of the composite steel-concrete beam test specimens.

4.5 TEST FRAME AND MACHINE

The set-up for the composite steel-concrete beam tests is illustrated in Figure 4.11. The test specimen was supported by a roller system at one end and a pinned system at the other end with a span length of 6 m. A 1000 kN hydraulic jack was used to apply the mid span vertical load. Displacement control was used in all the tests with a loading rate of 80 sec/mm for the first hour. The loading rate was then increased progressively until the peak load capacity of the test specimens was reached. The loading was terminated when either the maximum stroke of the jack was reached or when the load capacity of the specimens dropped drastically. The end support arrangement is shown in Figure 4.12. This arrangement induced a counter resistance to the twisting of the composite steel-concrete beam. This arrangement also prevents the composite steel-concrete beam from falling off the supports during the tests. Load cells were used in all the tests to record all the reactions and the vertical applied load.

For the push-out tests, the test specimens were rested on the prefabricated support frame that was used in experimental series I. One side of the supports was fixed and the other was simply supported. Same loading test machine with 1000 kN capacity was used for push-out tests as well as the composite steel-concrete beam tests.
4.6 LOADING PROCEDURE

For the composite steel-concrete beam test specimens, flexure and torsion were induced to the beams by the application of one vertical forces acting downwards on the top surface of the concrete slab at the mid span. Vertical reaction forces were also provided at each end support to restrain the transverse rotation due to the eccentric loading by the restraining packers. Displacement control was used at a loading rate of 80 sec/mm for the first hour. The rate was then increased progressively until the peak load capacity of the test specimens was reached or excessive twist rotation of the concrete slab was observed at the mid span region.

The push-out test specimens were loaded monotonically until failure or when the test specimens were unable to resist any additional loads. The tests would also be terminated once one side of the concrete slab was detached or when the deformation became excessive. The loading rate was maintained at approximately 50 kN/min.

4.7 MATERIAL TESTS

All materials were ordered from the same batch for the tests. The material properties of the concrete, the steel beam components, reinforcing bar and the shear studs are outlined in the following sections. The tensile tests for all materials except the concrete cylinder specimens were conducted by AS1391 (Standard Australia, 1991). Properties for concrete were be determined by the cylinder compression tests from AS1012.9 (Standard Australia, 1999) and the splitting tests from AS1012.10 (Standard Australia, 2000).
4.7.1 Concrete

Concrete tests comprised of standard cylinder compression and indirect tensile splitting tests. The tests were carried at the 7, 14, 21, 28 and the testing days of the composite steel-concrete beam and push-out test specimens. An 1800 kN AVERY compression testing machine was used to apply a compressive force at a loading rate of 900 kN/min onto the concrete cylinders.

4.7.1.1 Cylinder compression tests

The cylinder compression tests were carried out according to AS1012.9 (Standards Australia, 1999) to determine the compressive strength of the concrete used in the experimental series II. The prepared cylinders would be measured to determine their dimensions of diameter and height. Rubber cap was used for the tests. Two samples were tested each day to provide an average compressive strength of the concrete.

In Table 4.3, the cylinder compression test results in 28 days were summarised. The average concrete compressive strength after 28 days was 38.8 N/mm$^2$. The compressive strength was higher than the ordered strength of 32 N/mm$^2$ from the purchase orders.

In Tables 4.4 and 4.5, the cylinder compression test results for the composite steel-concrete beam testing days for FSC and PSC has been summarised respectively. During each testing day, two concrete cylinders were tested and recorded in the test results. The average concrete compressive strength for each test specimen was constant after the 28 days of curing with a range from 37 to 42 N/mm$^2$. 
Even after 113 days, the average concrete compressive strength was 38 N/mm$^2$ from Table 4.6 for the cylinder compression test results for push-out tests.

### 4.7.1.2 Splitting tests

The splitting tests were carried out according to AS1012.10 (Standards Australia, 2000) to determine the indirect tensile strength of the concrete used in experimental series II. One test specimen was used for each testing day. From Table 4.7, the concrete tensile strength for each testing day was constant with a range from 2.85 to 3.71 N/mm$^2$.

### 4.7.2 Structural steel

Same universal steel beams of 200UB29.8 cross-section with the manufacturer nominal yield strength of 300 N/mm$^2$ were used in the experimental series II. However, another set of tensile coupon tests was needed seem a new batch of structural steel was used. Four steel beam coupons were cut out from the flange and four from the web with a nominal thickness of 9.6 and 6.4 mm respectively.

The method of preparation and test procedures followed the guidelines presented in AS 1391 (Standards Australia, 1991). A 500 kN INSTRON universal testing machine was used to carry out the tensile tests. The experimental results have been plotted as shown in Figures 4.13 and 4.14 for the steel webs and flanges respectively. The mean yield and ultimate tensile strengths for the steel flanges were 347 and 495 N/mm$^2$ respectively. The mean yield and ultimate tensile strengths for the steel web were 374 and 512 N/mm$^2$ respectively. Both the test results have been summarized in Table 4.8.
4.7.3 Steel reinforcing bar

Five reinforcing bars of N12 nominal diameter were used as longitudinal transverse reinforcement in the concrete slabs, whilst R10 stirrup bars were used as torsional reinforcement in the concrete slab with a nominal diameter of 10 mm. Tensile testing was carried out according to AS1391 (Standards Australia, 1991) and the stress-strain curves were plotted as shown in Figures 4.15 and 4.16 for the N12 reinforcing bar and R10 stirrup bar respectively. The mean yield and ultimate tensile strengths for N12 reinforcing bar were 586 and 683 N/mm$^2$ respectively. The mean yield and ultimate tensile strengths for R10 stirrup bar were 399 and 498 N/mm$^2$ respectively. The results for N12 reinforcing bar and R10 stirrup bar have been summarized in Tables 4.9 and 4.10 respectively.

4.7.4 Shear studs

Each shear stud was first machined such that the head was removed and the cross-section of the middle region was reduced as illustrated in Figure 4.17. The existing threaded ends were rigidly fixed to a pair of couplers before tested in direct tension as shown in Figure 4.18. Five shear studs were tested to determine the stress-strain relationship for the shear studs from the tensile tests as shown in Figure 4.19. The mean 0.2 % proof stress and ultimate tensile strengths for the shear studs were 395 and 499 N/mm$^2$ respectively. The test results have been summarized in Table 4.11.

4.8 PUSH-OUT TESTS

Four push-out tests (CBPT-F1, CBPT-F2, CBPT-P1 and CBPT-P2) were conducted to determine the load-slip characteristics and the ultimate shear capacity
of the shear studs used in this experimental series II. For push-out tests 1 and 2 (CBPT-F1 and CBPT-F2), there were a total of four shear studs to represent FSC, whilst for push-out tests 3 and 4 (CBPT-P1 and CBPT-P2), there were a total of two shear studs to represent PSC. The ultimate load and the vertical displacements for each shear stud location were continuously recorded in all the tests. The horizontal displacement at the roller support was measured as well.

4.8.1 Push-out observations

Before the actual push-out tests began, a load of between 10 to 20 kN was applied at a loading rate of 0.4 mm/min to the test specimens to check the test set-up and instrumentation. Once the check was completed, the hydraulic jack would unload to reset the calibration of the instrumentation. The tests would start with a loading rate of 1 mm/min. Separation of the concrete slab from the steel beam was observed as the applied load was constant for all push-out tests. The applied load then suddenly dropped about 100 kN when a loud sound was heard, indicating the failure of a shear stud. Further sounds were heard later with a drop of about 100 kN of applied load each time for CBPT-F1 and CBPT-F2. Horizontal displacement of the roller support started to display when the first sound was heard. The tests were continued until the detachment of one of the concrete slabs.

4.8.2 Push-out test results

The summary of results for the four push-out tests is presented in Table 4.12 with the average load-slip characteristics shown in Figure 4.20. The average load had been calculated by dividing the total load with the number of shear studs and the average slip was determined by averaging the values recorded from the LVDTs.
For push-out test 1 (CBPT-F1), the total maximum load capacity of the shear studs reached a peak of 536 kN at an average interface slip of 13.4 mm. Therefore, the maximum load per shear stud was determined to be 134 kN. The load capacity remained similar until the vertical slip reached approximate 16.6 mm when stud shank failure occurred.

For push-out test 2 (CBPT-F2), the maximum load capacity of the shear studs has reached a peak of 536 kN at an average interface slip of 11.6 mm. Therefore, the maximum load per shear stud was determined to be 134 kN. The applied load remained until the vertical slip reached approximate 15.4 mm when stud shank fracture failure occurred.

For the push-out test 3 and 4 (CBPT-P1 and CBPT-P2), which represented the shear stud arrangement for PSC, their peak load capacities were 290 kN at 12.8 m and 278 kN at 10.9 mm respectively. Therefore, the maximum loads per shear stud were 145 and 139 kN for CBPT-P1 and CBPT-P2 respectively. Their average slips at fracture were 14.7 and 12.2 mm for CBPT-P1 and CBPT-P2 respectively.

The horizontal displacement of the roller support of the push-out tests was plotted as shown in Figure 4.21. All the push-out test specimens did not have excessive horizontal displacement during the push-out tests, which provided an indication of the success of the push-out tests. The horizontal displacements for all the push-out tests were slightly higher than the previous push-out tests from experimental series I. The reason was due to the higher ductility achieved from the push-out tests.

In conclusion, the push-out test results have shown that there is no difference in the ultimate shear capacity of the shear studs used for FSC and PSC. Their load-
slip characteristics were similar in as illustrated in Figure 4.20. In terms of ductility, all push-out tests achieved similar average slip at fracture from 13.5 to 16 mm.

4.9 BEAM TESTS

This section provides the behaviour of curved in plan composite steel-concrete beams subjected to combined flexure and torsion. A total of eight composite steel-concrete beams were tested in this section. The details of the composite steel-concrete beams were presented earlier in Section 4.2. Four composite steel-concrete beams (CCBF-1, CCBF-2, CCBF-3 and CCBF-4) were designed with FSC and four composite steel-concrete beams (CCBP-1, CCBP-2, CCBP-3 and CCBP-4) were designed with PSC. All the composite steel-concrete beams were loaded statically up to failure with different combinations of flexure and torsion. The evaluation of the test results covers the observed behaviour of the test specimens and also the behaviour of various components in terms of strength and ductility. The various parameters that had influenced the modes of failure of the composite steel-concrete beams are also discussed in addition to detailed comparison and appraisal of the test results.

4.9.1 Beam observations

The following presents an outline of the test observations which was viewed from the data logger and observed during the tests. The left near side is the extent of the side face of the test specimens from the mid span to the left hand end of the test specimens when viewed from the data logger.
4.9.1.1 Composite steel-concrete beam CCBF-1

For CCBF-1, the first few flexural concrete cracks were noticed in the vicinity of the mid span. However, when the applied load increased slightly to 20 kN, diagonal torsional concrete cracks were observed at both the supports of CCBF-1. The diagonal cracks increased in length and started to open up as shown in Figure 4.22. A large noise was heard at applied load of 80 kN. The maximum load was reached later at 104 kN with a mid span deflection of 86 mm. The failure mode was due to the diagonal torsional concrete cracks at the supports as the torsional moment capacity of CCBF-1 had been achieved. The test was concluded as the twist rotation at the mid span became severe. The maximum twist rotation achieved by CCBF-1 was 171 mRad. Uplift separation of the concrete slab from the steel beam was observed at both end supports as shown in Figure 4.23.

4.9.1.2 Composite steel-concrete beam CCBF-2

For CCBF-2, due to the load combination of flexure and torsion, CCBF-2 behaved nonlinearly from the start to the end of the tests. The cracking pattern was similar to CCBF-1. The twisting of the composite steel-concrete beam was noticed in the tests as shown in Figure 4.24. The maximum load of 138 kN with a mid span deflection of 95 mm was achieved by CCBF-2. Higher ductility has been observed for CCBF-2 with a mid span deflection of 99 mm at failure. The test was concluded due to severe twisting of the composite steel-concrete beams. Uplift separation of the concrete slab was observed.
4.9.1.3 Composite steel-concrete beam CCBF-3

For CCBF-3, more torque was applied to the composite steel-concrete beam due to the higher span to curvature ratio. Fewer flexural concrete cracks had been observed for CCBF-3 as shown in Figure 4.25 compared to CCBF-1 and CCBF-2. CCBF-3 started to twist immediately as the load was applied. At the applied load of 30 kN, a loud sound was heard which could be due to the adjustment of the reinforcement or the shear studs. Another sound was heard when the applied load was 37 kN. The twist rotation began to increase significant with only a slight increase of the applied load. The maximum load was reached at 72 kN with a mid span deflection of 45 mm. Diagonal torsional concrete cracks were observed throughout the test with excessive twisting of the composite steel-concrete beam. The test was concluded due to excessive twisting of CCBF-3. Uplift separation of the concrete slab was observed.

4.9.1.4 Composite steel-concrete beam CCBF-4

For CCBF-4, it has the highest span to curvature ratio among the test specimens. As the load was applied, a loud sound was heard when the load was 2 kN. Diagonal torsional concrete cracks were developed from the start of the tests and began to open and widen at the end supports as shown in Figure 4.26. More cracks were observed than CCBF-1, CCBF-2 and CCBF-3. The duration of this test was short and the test was concluded due to excessive twisting of CCBF-4. The maximum load was only 49 kN with a mid span deflection of 42 mm. Uplift separation of the concrete slab was observed. There was an uplift of the steel beam at both end supports observed at the end of the test as shown in Figure 4.27.
4.9.1.5 Composite steel-concrete beam CCBP-1

For CCBP-1, the configuration of this composite steel-concrete beam was similar to CCBF-1 except with 50 % of FSC. The behaviour was also similar to CCBF-1. There were flexural concrete cracks at the vicinity of the mid span region as well torsional concrete cracks at the end supports. Several loud sounds were heard at the applied loads of 30, 41, 49, 55 and 102 kN. The maximum load was 122 kN with a mid span deflection of 115 mm. High ductility was observed for CCBP-1. Longitudinal interface slip of 7 mm was obtained at the end of the test. Uplift separation of the concrete slab from the steel beam was observed at both end supports. The test was concluded due to the failure in the torsional moment capacity.

4.9.1.6 Composite steel-concrete beam CCBP-2

For CCBP-2, the configuration of this composite steel-concrete beam was similar to CCBF-2 except with 50 % of FSC. More loud sounds were heard during the tests at the applied loads of 41, 56, 91, 95, 98, 101 and 111 kN. This was due to flexibility of the composite steel-concrete beam with lesser shear connectors. More torsional concrete cracks were observed compared to CCBF-2 as shown in Figure 4.28 as well as more flexural concrete cracks. The maximum load was 114 kN with a mid span deflection of 68 mm. Uplift separation of the concrete slab was observed. The test was concluded due to the excessive twisting of CCBP-2.

4.9.1.7 Composite steel-concrete beam CCBP-3

For CCBP-3, the configuration of this composite steel-concrete beam was similar to CCBF-3 except with 50 % of FSC. Due to the increase of torque, CCBP-3 tends to twist when the load was applied. More torsional concrete cracks were
formed at the end supports as the applied load increased. Even though, PSC was used in the design, there was less longitudinal interface slip needed for CCBP-3 as the failure criteria was the torsional moment capacity as shown in Figure 4.29. The maximum load was 78 kN with a mid span deflection of 47 mm. The test was concluded due to severe twisting of the composite steel-concrete beam. Uplift separation of the concrete slab was observed.

4.9.1.8 Composite steel-concrete beam CCBP-4

For CCBP-4, the configuration of this composite steel-concrete beam was similar to CCBF-4 except with 50 % of FSC. Only torsional concrete cracks were observed in the test. The maximum load was only 56 kN with a mid span deflection of 42 mm. Uplift separation of the concrete slab was observed as well. Loud sounds were heard at the early stage of the loading when the applied loads were 15, 18 and 20 kN. The torsional concrete cracks were lengthened and widened as the applied load increased. The test was then concluded when the twist rotation was too high at 99 mRad.

4.9.2 Beam test results

The detailed experimental results obtained for the eight curved in plan composite steel-concrete beams are presented in this section. The following main aspects of the behaviour will be appraised. In each aspect, figures and tables are presented to justify and illustrate the findings.

(1) Strength and deformation response in terms of load-deflection, moment-curvature and torque-twist of the beams.
(2) Flexural strain distribution based on the mid span strain gauge measurements at reinforcement and steel beams provided the neutral axes of the composite steel-concrete beam sections.

(3) Longitudinal interface slip between the concrete slab and the top steel flange at both end supports of the composite steel-concrete beams.

(4) Rotation at the end supports of the composite steel-concrete beams based on the inclinometer readings.

(5) Torsional interface slip between the concrete slab and the top steel flange at the mid span of the composite steel-concrete beams.

4.9.2.1 Strength response for composite steel-concrete beams

Figure 4.30 shows the load-deflection curves for all the composite steel-concrete beams measured at the mid span. In terms of stiffness, the initial stiffness of all the composite steel-concrete beams was very different, since different span to curvature ratios have been designed for each test specimen. As the span to radius of curvature ratio increased, the initial stiffness decreased.

In terms of load-carrying capacity, the composite steel-concrete beams with higher span to radius of curvature ratio have achieved a lower load capacity except for CCBF-1. The reason as that the higher torque was induced from having higher span to radius of curvature ratio, therefore less applied load was needed to reach the maximum torsional moment capacities of the test specimens. When the levels of shear connection were compared, the load capacities for PSC composite steel-concrete beams seem to achieve slightly higher load capacity than FSC composite steel-concrete beams. For example, CCBP-1 achieved 117 % of the load-carrying capacity of CCBF-1, CCBP-3 achieved 108 % of the load-carrying capacity of
CCBF-3 and CCBP-4 achieved 114% of the load-carrying capacity of CCBF-4, whilst only CCBP-2 achieved 83% of the load-carrying capacity of CCBF-2.

In terms of ductility, CCBF-1, CCBF-2, CCBP-1 and CCBP-2 were more ductile than the other test specimens. Less torque had been applied to those test specimens due to their lower span to radius of curvature ratios, therefore they only reached their maximum torsional moment capacities at a later stage of the tests. This allowed more ductility to develop over the course of the tests.

Figure 4.31 shows the moment-curvature curves for all the eight composite steel-concrete beams measured at the mid span. Composite steel-concrete beams with lower span to radius of curvature ratios had higher flexural moment capacities than those with higher span to radius of curvature ratios. For example, CCBF-4 achieved 46% of the flexural moment capacity of CCBF-1 and CCBP-4 achieved 45% of the flexural moment capacity of CCBP-1. These test results showed that the ultimate flexural moment capacity decreased with an increase in the span to radius of curvature ratio. With the increase of the span to radius of curvature ratio, the main failure criteria changed from bending to the combined action of bending and twisting.

Figure 4.32 shows the moment-curvature curves for the four composite steel-concrete beams with FSC (CCBF). As concluded before, composite steel-concrete beams with higher span to radius of curvature ratios had higher flexural moment capacities than those with lower ratios. However, CCBF-2 with a slightly lower span to radius of curvature ratio than CCBF-1 had achieved a peak flexural moment capacity of 207 kNm compared to the 155 kNm reached by CCBF-1. This case could be just an isolated case due to the imperfection of horizontal curvature of the test
specimen. Nevertheless, compared with the other composite steel-concrete beams with much higher ratio, both CCBF-1 and CCBF-2 with low span to radius of curvature ratios achieved much higher flexural moment capacities than both CCBF-3 and CCBF-4.

Figure 4.33 shows the moment-curvature curves for the four composite steel-concrete beams with PSC (CCBP). There is no such problem of having an isolated case with those composite steel-concrete beams with PSC. CCBP-1 achieved the highest flexural moment capacity of 182 kNm which was greater than the rest of the CCBP beams. Composite steel-concrete beams with PSC had lower flexural moment capacities with an increase to their span to radius of curvature ratios.

In terms of torsional moment capacity, Figure 4.34 shows the torque-twist curves for all the eight composite steel-concrete beams measured at the mid span. The torque moment capacities for all composite steel-concrete beams seemed to hover around the 34 to 40 kNm region. Every composite steel-concrete beam had a different level of twist in the diagram in Figure 4.34. We can conclude that for all the composite steel-concrete beams that were tested, the failure mode was the combined bending and twisting of the beams since the torque moment capacities reached by all test specimens were similar.

From Figures 4.35 and 4.36, the torque-twist curves for CCBF and CCBP measured at the mid span have been shown respectively. From Figure 4.35, except for CCBF-2, every other composite steel-concrete beam CCBF-1, CCBF-3 and CCBF-4 had achieved similar torque moment capacities of 24 kNm. CCBF-2 has obtained the highest torsional moment capacity of 33 kNm. This could be due to the presence of flexure in the load combination of CCBF-2.
From Figure 4.36, all the CCBP composite steel-concrete beams had similar peak torque moment capacities. However, their twist rotation at failure varied from 15 to 100 mRad. The reason was that the conclusion of the tests was very subjective to the excessive twisting of the test specimens. The twisting of the test specimens was ductile due to the presence of steel beams. However, the twist rotation could be too large and caused the separation of the contact between the test specimens and the hydraulic jack. Table 4.13 summarises all the maximum strengths such as the flexural and torsional moment capacities of the test specimens.

### 4.9.2.2 Neutral axis for composite steel-concrete beams

Figures 4.37, 4.38, 4.39 and 4.40 show the mid span strain distributions for CCBF-1, CCBF-2, CCBF-3 and CCBF-4 respectively. The composite neutral axes had a range from 215 to 228 mm from the base of the composite steel-concrete beams for the CCBF series. The neutral axis occurred in the concrete slab which was normal for a composite steel-concrete beam with FSC and a span length of 6 m. The strain distribution was linear for CCBF composite steel-concrete beams during the early stage of the tests as shown. There was not partial composite action or any longitudinal interface slip noticed from the strain distributions which indicated that the composite steel-concrete beams had full composite actions throughout the tests. The steel beams were also in tension throughout the tests as well which prevented torsional lateral buckling of the steel webs.

Figures 4.41, 4.42, 4.43 and 4.44 show the mid span strain distributions for CCBP-1, CCBP-2, CCBP-3 and CCBP-4 respectively. There were two set of strain distributions in each composite steel-concrete beam section, one at the concrete slab and the other at the steel beam. The two different strain distributions were separated
by the interface line between the concrete slab and steel beam. The neutral axes at concrete slabs of CCBP-1, CCBP-2, CCBP-3 and CCBP-4 had a range from 241 to 270 mm, whilst there were no neutral axes at the steel beams. Therefore, all the steel beams were under tension throughout the tests.

Interestingly, the strain distributions for concrete slabs behaved differently from the strain distributions for steel beams during the tests. While this was normal to have longitudinal interface slips between the concrete slabs and steel beams from the strain distributions, but there were different curvatures between the concrete slabs and steel beams in all the strain distributions for CCBP composite steel-concrete beams. The strain distributions of the concrete slabs had exhibited different curvatures compared with the curvatures of the steel beams. Due to the presence of PSC, the concrete slabs had seemed to be vertically separate from the steel beams. The difference in the curvatures was greater for CCBP-4 than CCBP-1. The reason could be that torque was induced earlier to CCBP-4 than CCBP-1 which could cause the composite steel-concrete beams to twist earlier and in turn induce tensile axial force to the shear studs at the mid span. The shear studs might be subjected to a combination of significant axial force due to the twisting and shear force due to the bending. However, CCBP-4 has achieved a similar torsional moment capacity with CCBP-1. Therefore, the only two conclusions adopted from here were that different curvatures appeared when torque was applied to the composite steel-concrete beams with PSC and the difference in curvatures was greater with an increase in the span to radius of curvature ratio.
4.9.2.3 Torsional interface slip for composite steel-concrete beams

The difference in the curvatures of the concrete slab and the steel beam has been named as the “torsional interface slip”. This difference in the curvatures was due to the difference in the twist rotation of the concrete slab and steel beam during the tests when torque was applied to the composite steel-concrete beams. Due to this torsional interface slip, the assumption or condition where “plane sections remain plane” was not valid in these tests. The strain distributions from CCBP-1 to CCBP-4 have reinforced this finding.

The torsional interface slip could be measured and illustrated from Figure 4.45. This slip could be affected by several factors such as the level of shear connection, the shear connector arrangement and the amount of applied torque. Since the strain distributions were taken from the mid span when torque was assumed to be zero, the level of torsional interface slip could be greater at the end supports than the mid span where the applied torque was the largest as shown in Figure 4.23. Therefore, the assumption “plane sections remain plane” should not be used in any numerical analysis when composite steel-concrete beams were subjected to torsion.

4.9.2.4 Longitudinal interface slip for composite steel-concrete beams

Using linear variable differential transducers (LVDT), the longitudinal interface slip between the concrete slab and the top steel flange was measured and summarised in Table 4.14 and plotted in Figure 4.46. These measurements were taken at the end supports of the composite steel-concrete beams where the longitudinal interface slip was the highest. Relative longitudinal interface slip was recorded as well when the maximum applied load was reached. As expected, the overall longitudinal interface slip measurements from the composite steel-concrete
beams with FSC were much lower than those with PSC. The relative longitudinal interface slip for FSC registered an average of 1.4 and 2.7 mm for PSC. Higher longitudinal interface slip was required for the composite steel-concrete beams when PSC was used. For example, CCBP-1 reached a relative longitudinal interface slip of 6.9 mm compared with that of 1.3 mm for CCBF-1.

With higher span to radius of curvature ratios, the relative longitudinal interface slips decreased for both the CCBF and CCBP. This could be explained due to the lower flexural moment achieved by those composite steel-concrete beams with higher span to radius of curvature ratios. For flexural moment to increase, greater shear force was required to transfer from the concrete slab to the steel beam. Therefore, since those composite steel-concrete beams with higher span to radius of curvature ratios reached the maximum torsional moment capacities faster, less flexural moment resistance was required.

4.10 SUMMARY

The test results of the experimental series II of eight curved in plan composite steel-concrete beams designed with varying levels of shear connection have been described in this chapter. The composite steel-concrete beams were subjected to combined torsion and flexure during the tests. The following outlines the important findings obtained from the beam tests.

(1) The push-out tests from experimental series II has reinforced that the average ultimate shear capacities and the load-slip characteristics for both FSC and PSC push-out tests were not affected even though different number of shear studs was used. The ductility of the shear studs was similar as well for both FSC and PSC push-out tests.
(2) In terms of stiffness, the initial stiffness of the curved in plan composite steel-concrete beams were different from one another due to the different in the profile of their horizontal curvature geometries. Curved in plan composite steel-concrete beams with higher span to radius of curvature ratios tend to have lower initial stiffness.

(3) In terms of load-carrying and flexural moment capacities, curved in plan composite steel-concrete beams with higher span to radius of curvature ratios tend to have lower load-carrying and flexural moment capacities. This was because with the increase of the span to radius of curvature ratio, the main failure criteria changed from bending to the combined action of bending and twisting. The critical factor would be their torsional moment capacities.

(4) Different levels of shear connection did not affect the torsional moment capacities of the test specimens. Moreover, curved in plan composite steel-concrete beams with PSC (50 % of FSC) seem to perform better than those with FSC in terms of load-carrying capacity.

(5) In terms of ductility, curved in plan composite steel-concrete beams such as CCBF-3, CCBF-4, CCBP-3 and CCBP-4 have lower ductility than those with lower span to curvature radius. The failure of CCBF-3, CCBF-4, CCBP-3 and CCBP-4 was earlier than other test specimens due to the increase in applied torque. This could be a major issue for curved in plan composite steel-concrete beams because there was no early visible warning such as excessive deflection before failure occurred.

(6) Separation of the concrete slab from the steel beam had been observed from the tests. This separation caused the difference in the curvature of the strain
distributions from both concrete slab and steel beam. The term “torsional interface slip” has been used to identify this difference in curvatures.

(7) The assumption “plane sections remain plane” should not be used in any numerical analysis when composite steel-concrete beams were subjected to torsion due to the torsional interface slip. This torsional slip would increase with the decrease in the level of shear connection.

(8) The longitudinal interface slip capacity of the shear connectors was not a critical parameter in the case for curved in plan composite beams especially when the span to radius of curvature was high. The failure modes of those beams tend to be concrete torsional cracking or torsional lateral buckling of the steel web. Therefore, torsional failure was more likely to occur than flexural failure. However, if the span to radius of curvature was low and the curved in plan beams were close to straight beams such as CCBF-1 and CCBP-1, flexural failure occurred.
### Table 4.1 Curved in plan composite steel-concrete beam specimen details

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Span / Radius</th>
<th>Radius (m)</th>
<th>Lever arm (mm)</th>
<th>Degree of shear connection (%)</th>
<th>Stud spacing (mm)</th>
<th>Stud section (19 mm studs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CCBF-1</td>
<td>0.291</td>
<td>20.65</td>
<td>232</td>
<td>100</td>
<td>460</td>
<td>2</td>
</tr>
<tr>
<td>CCBF-2</td>
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<td>19.79</td>
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<td>100</td>
<td>460</td>
<td>2</td>
</tr>
<tr>
<td>CCBF-3</td>
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<td>14.14</td>
<td>338</td>
<td>100</td>
<td>460</td>
<td>2</td>
</tr>
<tr>
<td>CCBF-4</td>
<td>0.632</td>
<td>9.49</td>
<td>501</td>
<td>100</td>
<td>460</td>
<td>2</td>
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<td>9.47</td>
<td>502</td>
<td>50</td>
<td>460</td>
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- **Number of specimens**: 8
- **Concrete slab size**: 6200 x 500 x 120 mm
- **Concrete strength**: 32 N/mm²
- **Concrete cover**: 25 mm
- **Steel beam**: 200 UB 29.8 (BHP-300PLUS)
- **Longitudinal reinforcement**: N12 – Grade 500 N/mm²
- **Transverse reinforcement**: R10 - Grade 500 N/mm²
- **Shear studs**: 19 mm headed studs
Table 4.2 Push-out specimen details

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<th>Specimen</th>
<th>Stud section (19 mm studs)</th>
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<th>Stud spacing (mm)</th>
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<td>460</td>
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Number of specimens: 4

Concrete slab size: 600 x 600 x 120 mm

Concrete strength: 32 N/mm²

Concrete cover: 25 mm

Steel beam: 200 UB 29.8 (BHP-300PLUS)

Reinforcement: N12 – Grade 500 N/mm²

Shear studs: 19 mm headed studs
Table 4.3 Cylinder compression test results in 28 days

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<th>Age (Day)</th>
<th>Sample</th>
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<th>Avg. diameter (mm)</th>
<th>Height (mm)</th>
<th>Type of cap</th>
<th>Max. load (kN)</th>
<th>Compressive strength (N/mm²)</th>
<th>Avg. strength (N/mm²)</th>
<th>Comment</th>
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<td>100.0</td>
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<td>99.5</td>
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<td>Rubber</td>
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Table 4.4 Cylinder compression test results for CCBF beam tests

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<th>Avg. diameter (mm)</th>
<th>Height (mm)</th>
<th>Type of cap</th>
<th>Max. load (kN)</th>
<th>Compressive strength (N/mm$^2$)</th>
<th>Avg. strength (N/mm$^2$)</th>
<th>Comment</th>
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<td>101.8</td>
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<td>99.9</td>
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### Table 4.5 Cylinder compression test results for CCBP beam tests

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<th>Avg. diameter (mm)</th>
<th>Height (mm)</th>
<th>Type of cap</th>
<th>Max. load (kN)</th>
<th>Compressive strength (N/mm$^2$)</th>
<th>Avg. strength (N/mm$^2$)</th>
<th>Comment</th>
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<td>100.0</td>
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### Table 4.6 Cylinder compression test results for push-out tests

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<th>Height (mm)</th>
<th>Type of cap</th>
<th>Max. load (kN)</th>
<th>Compressive strength (N/mm$^2$)</th>
<th>Avg. strength (N/mm$^2$)</th>
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<td>38.0</td>
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<td>CCPT-P1 &amp; CBPT-P2</td>
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<td>101.6</td>
<td>203.0</td>
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### Table 4.7 Splitting test results

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<th>Diameter (mm)</th>
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<th>Height (mm)</th>
<th>Max. load (kN)</th>
<th>Tensile strength (N/mm$^2$)</th>
<th>Comment</th>
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187
Table 4.8 Structural steel tensile test results

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<th>Width (mm)</th>
<th>Avg. width (mm)</th>
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<th>Avg. thickness (mm)</th>
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<th>Yield strength (N/mm²)</th>
<th>Tensile strength (N/mm²)</th>
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<td>Tensile strength (N/mm$^2$)</td>
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<td>586</td>
<td>683</td>
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Table 4.10 Stirrup bar tensile test results

<table>
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<tr>
<th>Stirrup specimen</th>
<th>Diameter (mm)</th>
<th>Avg. diameter (mm)</th>
<th>Area (mm$^2$)</th>
<th>Max. load (kN)</th>
<th>Yield stress (N/mm$^2$)</th>
<th>Tensile strength (N/mm$^2$)</th>
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</thead>
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<tr>
<td>1</td>
<td>10.00</td>
<td>10.00</td>
<td>10.00</td>
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<td>9.95</td>
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<td>9.95</td>
<td>9.97</td>
<td>78.1</td>
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<td>396</td>
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<td>5</td>
<td>10.00</td>
<td>10.00</td>
<td>10.00</td>
<td>78.5</td>
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<td>398</td>
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Avg. 399 498
Table 4.11 Headed shear stud tensile test results

<table>
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<tr>
<th>Stud specimen</th>
<th>Diameter (mm)</th>
<th>Avg. diameter (mm)</th>
<th>Area (mm²)</th>
<th>Max. load (kN)</th>
<th>0.2 % Proof stress (N/mm²)</th>
<th>Tensile strength (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>8.95</td>
<td>8.95</td>
<td>8.95</td>
<td>62.9</td>
<td>30</td>
<td>391</td>
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<tr>
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<td>8.90</td>
<td>8.90</td>
<td>62.2</td>
<td>31</td>
<td>393</td>
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<td>8.90</td>
<td>8.90</td>
<td>62.2</td>
<td>32</td>
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<td>4</td>
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<td>8.95</td>
<td>8.95</td>
<td>62.9</td>
<td>32</td>
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<td>Avg.</td>
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Table 4.12 Push-out test results

<table>
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<tr>
<th>Push-out test specimen</th>
<th>Max. load (kN)</th>
<th>Max. load per stud (kN)</th>
<th>Avg. slip at max. load (mm)</th>
<th>Avg. slip at fracture (mm)</th>
</tr>
</thead>
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<tr>
<td>CBPT-F1</td>
<td>536</td>
<td>134</td>
<td>13.4</td>
<td>16.6</td>
</tr>
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<td>CBPT-F2</td>
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<td>134</td>
<td>11.6</td>
<td>15.4</td>
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<tr>
<td>Avg.</td>
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<td></td>
<td>12.5</td>
<td>16.0</td>
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<td>CBPT-P1</td>
<td>290</td>
<td>145</td>
<td>12.8</td>
<td>14.7</td>
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<tr>
<td>CBPT-P2</td>
<td>278</td>
<td>139</td>
<td>10.9</td>
<td>12.2</td>
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<tr>
<td>Avg.</td>
<td></td>
<td></td>
<td>11.9</td>
<td>13.5</td>
</tr>
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</table>
Table 4.13 Curved beam test results

<table>
<thead>
<tr>
<th>Beam specimen</th>
<th>Ultimate applied load (kNm)</th>
<th>Deflection at maximum load (mm)</th>
<th>Deflection at failure (mm)</th>
<th>Ultimate torque moment (kNm)</th>
<th>Twist at maximum load (mRad)</th>
<th>Twist at failure (mRad)</th>
<th>Ultimate flexural moment (kNm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CCBF-1</td>
<td>104</td>
<td>86</td>
<td>96</td>
<td>24</td>
<td>63</td>
<td>171</td>
<td>155</td>
</tr>
<tr>
<td>CCBF-2</td>
<td>138</td>
<td>95</td>
<td>99</td>
<td>33</td>
<td>53</td>
<td>93</td>
<td>207</td>
</tr>
<tr>
<td>CCBF-3</td>
<td>72</td>
<td>45</td>
<td>49</td>
<td>24</td>
<td>69</td>
<td>90</td>
<td>107</td>
</tr>
<tr>
<td>CCBF-4</td>
<td>49</td>
<td>42</td>
<td>59</td>
<td>24</td>
<td>49</td>
<td>105</td>
<td>72</td>
</tr>
<tr>
<td>CCBP-1</td>
<td>122</td>
<td>115</td>
<td>121</td>
<td>29</td>
<td>42</td>
<td>46</td>
<td>182</td>
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<tr>
<td>CCBP-2</td>
<td>114</td>
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<td>171</td>
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<tr>
<td>CCBP-3</td>
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<td>47</td>
<td>49</td>
<td>28</td>
<td>52</td>
<td>69</td>
<td>116</td>
</tr>
<tr>
<td>CCBP-4</td>
<td>56</td>
<td>42</td>
<td>63</td>
<td>28</td>
<td>40</td>
<td>99</td>
<td>82</td>
</tr>
</tbody>
</table>
Table 4.14 Interface slips for curved in plan composite steel-concrete beams

<table>
<thead>
<tr>
<th>Beam specimen</th>
<th>Relative interface slip (mm)</th>
<th>Maximum interface slip (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CCBF-1</td>
<td>1.3</td>
<td>2.7</td>
</tr>
<tr>
<td>CCBF-2</td>
<td>2.4</td>
<td>2.7</td>
</tr>
<tr>
<td>CCBF-3</td>
<td>0.8</td>
<td>1.1</td>
</tr>
<tr>
<td>CCBF-4</td>
<td>1.1</td>
<td>3.4</td>
</tr>
<tr>
<td>CCBP-1</td>
<td>6.9</td>
<td>7.3</td>
</tr>
<tr>
<td>CCBP-2</td>
<td>2.3</td>
<td>2.7</td>
</tr>
<tr>
<td>CCBP-3</td>
<td>0.9</td>
<td>1.0</td>
</tr>
<tr>
<td>CCBP-4</td>
<td>0.7</td>
<td>1.6</td>
</tr>
</tbody>
</table>
Figure 4.1 Cross-sectional view for CCBF-1, CCBF-2, CCBF-3 and CCBF-4
Figure 4.2 Plan view for CCBF-1, CCBF-2, CCBF-3 and CCBF-4
Figure 4.3 Elevation view for CCBF-1, CCBF-2, CCBF-3 and CCBF-4
Figure 4.4 Cross-sectional view for CCBP-1, CCBP-2, CCBP-3 and CCBP-4
Figure 4.5 Plan view for CCBP-1, CCBP-2, CCBP-3 and CCBP-4
Figure 4.6 Elevation view for CCBP-1, CCBP-2, CCBP-3 and CCBP-4
Figure 4.7 Push-out test specimens for CBPT-F1 and CBPT-F2
Figure 4.8 Push-out test specimens for CBPT-P1 and CBPT-P2
Figure 4.9 Reinforcement cage

Figure 4.10 Curved composite beam test specimens after concrete pouring
Figure 4.11 Curved composite steel-concrete beam set-up

Figure 4.12 Restraining system at the support
Figure 4.13 Steel web stress-strain curves
Figure 4.14 Steel flange stress-strain curves
Figure 4.15 Longitudinal reinforcing bar stress-strain curves
Figure 4.16 Stirrup bar stress-strain curves
Figure 4.17 Shear stud tensile test specimen

Figure 4.18 Shear stud tensile test
Figure 4.19 Shear stud stress-strain curves
Figure 4.20 Push-out test summarised results
Figure 4.21 Load-horizontal displacements for push-out tests
Figure 4.22 Diagonal torsional cracks for CCBF-1

Figure 4.23 Separation between the concrete slab and the steel beam
Figure 4.24 Twisting at mid span for CCBF-2

Figure 4.25 Few flexural concrete cracks for CCBF-3
Figure 4.26 Diagonal torsional cracks for CCBF-4

Figure 4.27 Uplift of the steel beam for CCBF-4
Figure 4.28 Diagonal torsional cracks for CCBP-2

Figure 4.29 Longitudinal interface slip for CCBP-3
Figure 4.30 Load-mid span deflection for curved composite beams
Figure 4.31 Moment-mid span curvature for curved composite beams
Figure 4.32 Moment-mid span curvature for CCBF
Figure 4.33 Moment-mid span curvature for CCBP
Figure 4.34 Torque-twist for curved composite steel-concrete beams
Figure 4.35 Torque-twist for CCBF
Figure 4.36 Torque-twist for CCBP
Figure 4.37 Strain distribution at mid span for CCBF-1
Figure 4.38 Strain distribution at mid span for CCBF-2
Figure 4.39 Strain distribution at mid span for CCBF-3
Figure 4.40 Strain distribution at mid span for CCBF-4

Strain (x10³ µε) vs. Sectional Depth (mm) for different applied loads (0 kN, 10 kN, 20 kN, 30 kN, 40 kN). The graph shows the strain distribution for Steel-Concrete Interface and Composite NA conditions.
Figure 4.41 Strain distribution at mid span for CCBP-1
Figure 4.42 Strain distribution at mid span for CCBP-2
Figure 4.43 Strain distribution at mid span for CCBP-3
Figure 4.44 Strain distribution at mid span for CCBP-4
Figure 4.45 Torsional interface slip
Figure 4.46 End support interface slips for curved composite beams
CHAPTER 5

5 FLEXURE-TORSION INTERACTION RELATIONSHIP

5.1 INTRODUCTION

This chapter presents the flexure-torsion interaction relationship developed from the two experimental series I and II from the previous Chapters 3 and 4. Flexure–torsion interaction relationship can be represented by the taking the ultimate strengths of flexural and torsional moments from each test specimen from the tests and plotted into a curve to define the ultimate limit for flexural and torsional moment capacity for different load combinations. The curve defines the strength of the composite steel-concrete beams subjected to combined flexure and torsion. This chapter will also consider the effects of PSC in the interaction relationship curves for both straight and curved in plan composite steel-concrete beams.

5.2 PREVIOUS RESEARCH

Colville (1973) performed a series of tests on composite steel-concrete beams that were curved in plan to investigate the behaviour of headed shear studs. From his investigation, he proposed an interaction relationship between the torsional and flexural moments for composite steel-concrete beams. This relationship was governed by Equation 5.1 as shown below.
\[
\left( \frac{M_F}{M_U} \right)^2 + \left( \frac{T_F}{T_U} \right)^2 = 1
\]  

(5.1)

where,

\[ M_F = \text{flexural moments at failure in test} \]
\[ M_U = \text{theoretical values of the ultimate flexural moments} \]
\[ T_F = \text{torsional moments at failure in test} \]
\[ T_U = \text{theoretical values of the ultimate torsional moments} \]

Other researchers also came out with their own flexure-torsion interaction relationship curves. The nest significant research was done by Singh and Mallick (1977) to further investigate the effects of torsion on composite steel-concrete beams. Instead of testing curved in plan beams, they carried out the investigation on straight composite steel-concrete beams subjected on combined flexure and torsion. Eight composite steel-concrete beams of identical cross-sectional dimensions under varying ratios of torsion and flexure were tested. They concluded that in their flexure-torsion interaction relationship curve, an increase in the flexural moment capacity beyond the ultimate flexural moment capacity in the presence of torsion and an increase in the torsional moment capacity beyond the ultimate torsional moment capacity in the presence of flexure were observed. Ghosh and Mallick (1979) later improved the interaction relationship from Singh and Mallick (1977) with six more composite steel-concrete beams at the region where \( M/M_u \) was less than 0.8. They also concluded the same phenomenon where the ultimate torsional and flexural moment capacities increased in the presence of flexure and torsion respectively. Finally, Ray and Mallick (1980) took the interaction relationship curve from Ghosh and Mallick (1979) and improved it with six more composite steel-concrete beams. A
final improvement had been made to the flexure-torsion interaction relationship curve done by Singh and Mallick (1977), Ghosh and Mallick (1979) and Ray and Mallick (1980) as shown in Figure 5.1 based on a total of 20 full-scale experimental tests.

Nie et al. (2000) had investigated the interaction relationship with four composite steel-concrete beams subjected to combined flexure and torsion. They had discovered that the flexural moment capacities of all the four beams did increase in the presence of torsion and 10 to 50% increase in torsional moment capacity in the presence of flexure. The following equations were proposed by Nie et al. (2000) for the flexure-torsion interaction relationship curve.

\[
(T/T_u)^2 = 1 + 3.17(M/M_u) 
\]  \hspace{1cm} (5.2)

when \( M/M_u \leq 0.65 \) and \( T/T_u \geq 1.0 \)

\[
(T/T_u) = 3.42 - 2.55(M/M_u) 
\]  \hspace{1cm} (5.3)

when \( M/M_u > 0.65 \) and \( T/T_u > 0.6 \)

\[
(M/M_u) - (T/0.6T_u) = 1.0 
\]  \hspace{1cm} (5.4)

when \( M/M_u \leq 0.65 \) and \( T/T_u \leq 0.6 \)

where,

\[ T_u \] = torsional moment capacity of the composite steel-concrete sections under pure torque

\[ T \] = torsional moment capacity of the composite steel-concrete sections

\[ M_u \] = flexural moment capacity of the composite steel-concrete sections
under pure bending

\[ M = \text{flexural moment capacity of the composite steel-concrete sections} \]

At the moment, there is no flexure-torsion interaction relationship curve designed directly for curved in plan composite steel-concrete beams. The interaction relationship curve for straight beams could be used to represent curved in plan beams but this has to depend on the loading condition.

### 5.3 EXPERIMENTAL SERIES I

From Chapter 3, the ultimate strengths for both flexural and torsional moment capacities had been obtained from six straight composite steel-concrete beams, three steel beams and three concrete slabs. Using the ultimate strengths, the flexure-torsion interaction relationship curves had been plotted for straight composite steel-concrete beams with FSC and PSC. Those curves had been compared with previous interaction relationship curves for comparison. A simplified design model was presented towards the end of this section.

#### 5.3.1 Ultimate torsion equation

Since there are not design rule for the calculation of the torsional moment capacity of a composite steel-concrete beam in the Australian Standards AS2327.1 (Standards Australia, 2003) or any other international codes. The following Equation 5.4 provides the basis to determine the ultimate torsional moment capacity of the composite steel-concrete beams. Figure 5.1 also illustrates the contribution of each component of the equation.
\[ T_U = T_C + T_S + (\leq T_J) \]

where,

- \( T_U \) = theoretical values of the ultimate torsional moments
- \( T_C \) = contribution of the concrete towards ultimate torque
- \( T_S \) = contribution of the torsional reinforcement towards ultimate torque
- \( T_J \) = contribution of the steel beam towards ultimate torque

Hsu et al. (1985),

\[ T_C + T_S = 2x_0y_0 \left( \frac{A_{sw} f_{syf}}{s} \right) \]

where,

- \( x_0 \) = smaller distance between corner bars in the rectangular cross-section
- \( y_0 \) = longer distance between corner bars in the rectangular cross-section
- \( A_{sw} \) = cross-sectional area of the bar forming a closed tie
- \( f_{syf} \) = yield strength of the fitments
- \( s \) = centre-to-centre spacing of the shear or torsional reinforcement

Plastic collapse analysis,

\[ T_J = \left( b_f t_f^2 - \frac{t_f^3}{3} + \frac{b_w t_w^2}{2} + \frac{t_w^3}{6} \right) \tau_f \]

where,

- \( b_f \) = flange width
- \( t_f \) = flange thickness
- \( b_w \) = web height
\[ t_w = \text{web thickness} \]
\[ \tau_y = 0.6 \times \text{steel beam yield stress} \]

The contributions for the ultimate torsional moments can be separated into two sections. One section represents by the reinforced concrete slab \((T_c \text{ and } T_s)\) where the concrete slab and the torsional reinforcement in the experimental series I and II is the R10 reinforcing stirrup. Equation 5.6 is used to calculate the contributions from the reinforced concrete slab.

The other section is represented by the steel beam \((T_J)\). For the steel beam, it is possible to ignore the contribution from the steel web and only include the steel flanges as the main contributors for torsional resistance. Nevertheless, the steel web contribution can still be taken into account as shown in Equation 5.7.

Even though the above Equation 5.5 can be true when the composite steel-concrete beams are under pure torsion, the equation must not be used when flexural loading is applied. From Table 5.1, if the torsional contributions of concrete, torsional reinforcement from CS-2 and steel beam from SB-2 are added, the total torsional capacities of both CBF-2 and CBP-2 should be close to 5 kNm. However, the ultimate torsional moment capacities for CBF-2 and CBP-2 were 17 and 16 kNm, which were more than 3 times the calculated capacities from the Equation 5.5. This is also true for test specimens CBF-3 and CBP-3. The torsional contributions of CS-3 and SB-3 were 5.9 kNm, whilst the ultimate torque for CBF-3 and CBP-3 were 28 and 24 kNm. CBF-3 and CBP-3 also had capacities equal to more than 4 times the amounts from the addition of CB-3 and SB-3 using Equation 5.5.

One reason that can be given is the rate of flexural contribution of the concrete slab to the flexural moment capacity of the composite steel-concrete beams.
has been reduced to allow the rate of torsional contribution to increase. As the ratio of torsional to flexural loading increases, less flexural contribution of the concrete slab is needed, therefore the torsional moment capacity of the composite steel-concrete beams can increase as shown for CBF-2 and CBF-3 or CBP-2 and CBP-3.

5.3.2 Flexure-torsion interaction relationship

Figure 5.2 shows flexure-torsion interaction relationship ratio curves for CBF and CBP test specimens compared with the curve suggested by Equation 5.1 and other flexure-torsion interaction relationship curves suggested by other researchers.

Colville’s (1973) method provided a lower bound interaction relationship between the torsional and flexural moments for composite steel-concrete beams. Moreover, other researchers have suggested their own interaction relationship curves. Nie et al. (2000) and Ray and Mallick (1980) had concluded that there was an increase in torsional strength in the presence of flexure and an increase of flexural strength in the presence of torsion for composite steel-concrete beams. However, from the flexure-torsion interaction relationship curves of the CBF and CBP series obtained from the experimental results, there was no significant increase in flexural moment capacity in the presence of torsion. The flexural strength ratio was almost unchanged before the ultimate torsional moment capacity was reached. On the other hand, there was a minor increase in torsional strength in the presence of flexure for both the CBF and CBP series. Other findings were that the CBF series had a higher increase in torsional strength ratio than the CBP series in the presence of flexure. This illustrated that the contribution from the steel beam, $T_J$ towards the ultimate torque reduced when the level of shear connection decreased. In addition, the CBP series had a slight increase in flexural strength ratio, whilst the flexural strength ratio
of the CBF series reduced slightly in the presence of torsion. This illustrated that there was no difference in flexural strength ratios when the level of shear connection was reduced.

5.3.3 Modified design models

Based on the experimental test results, Figure 5.3 provides a conservative modified flexure-torsion interaction relationship model for straight composite steel-concrete beams with FSC and PSC (50 % of FSC). The flexure-torsion interaction relationship equations can be written as following.

For straight composite steel-concrete beams with FSC (CBF),

\[
\frac{T}{T_U} = 0.4 \left( \frac{M}{M_U} \right) + 1 \quad \text{when } 0 \leq \frac{M}{M_U} < 1 \text{ and } \frac{T}{T_U} \geq 1
\] (5.8)

\[
\frac{M}{M_U} = 1 \quad \text{when } \frac{M}{M_U} \geq 1 \text{ and } 0 \leq \frac{T}{T_U} \leq 1.4
\] (5.9)

For straight composite steel-concrete beams with PSC (50 % of FSC) (CBP),

\[
\frac{T}{T_U} = 0.2 \left( \frac{M}{M_U} \right) + 1 \quad \text{when } 0 \leq \frac{M}{M_U} < 1 \text{ and } \frac{T}{T_U} \geq 1
\] (5.10)

\[
\frac{M}{M_U} = 1 \quad \text{when } \frac{M}{M_U} \geq 1 \text{ and } 0 \leq \frac{T}{T_U} \leq 1.2
\] (5.11)

Equations 5.8 and 5.9 have provided a simplified model to represent the flexure-torsion interaction relationship for FSC in composite steel-concrete beams,
whilst Equations 5.10 and 5.11 have been represented for PSC (50 % of FSC) composite steel-concrete beams.

5.4 EXPERIMENTAL SERIES II

From Chapter 4, the ultimate strengths for both flexural and torsional moment capacities had been obtained from eight curved in plan composite steel-concrete beams. Using the ultimate strengths, the flexure-torsion interaction relationship curves had been plotted for curved in plan composite steel-concrete beams with FSC and PSC. Those curves had been compared with experimental series I and previous interaction relationship curves for comparison. A simplified design model was presented towards the end of this section.

5.4.1 Flexure-torsion interaction relationship

Figure 5.4 shows the flexure-torsion interaction relationship curves of CCBF and CCBP test specimens compared with others interaction relationship suggested by Colville (1973), other researchers and the curves from Section 5.3.

Colville (1973) method still could provide a lower bound interaction relationship between the torsional and flexural moments for composite steel-concrete beams as shown in Figure 5.4. CCBP and CCBF series were very similar to the flexure-torsion interaction curve of Nie (2000) when \( \frac{M}{M_U} < 1 \), as there was an increase in the torsional strength in the presence of flexure. Ray and Mallick (1980) presented a flexure-torsion interaction curve that had a much higher increase in torsional strength in the presence of flexure than both the CCBF and CCBP series. But from the flexure-torsion interaction relationship diagram, both CCBF and CCBP
series have illustrated again that there was no significant increase in flexural moment capacity in the presence of torsion. Therefore, the previous tests for straight composite steel-concrete beams subjected to combined flexure and torsion, CBF and CBP series did validate this conclusion as well.

Figure 5.5 has brought together CBF, CBP, CCBF and CCBP flexure-torsion interaction relationship curves. From the diagram, all curves showed a nearly unchanged flexural strength ratio when \( \frac{M}{M_U} = 1 \). There was no increase in the flexural moment capacities in the presence of torsion for both series. But compared to CBF and CBP series, the increase in torque capacities with the presence of flexure was greater than in the cases for CCBF and CCBP series respectively.

5.4.2 Modified design models

Based on the test results, Figure 5.6 presents new modified flexure-torsion interaction relationship models for curved in plan composite steel-concrete beams with FSC and PSC (50 \% of FSC). Changes have been made to the previous model since more specimens have been tested in the range where \( 0 \leq \frac{M}{M_U} < 1 \). The interaction relationship equations can be written as followed.

For curved in plan composite steel-concrete beams with FSC (CCBF),

\[
\frac{T}{T_U} = 0.7 \left( \frac{M}{M_U} \right) + 1 \quad \text{when } 0 \leq \frac{M}{M_U} < 1 \text{ and } \frac{T}{T_U} \geq 1 \tag{5.12}
\]

\[
\frac{M}{M_U} = 1 \quad \text{when } \frac{M}{M_U} \geq 1 \text{ and } 0 \leq \frac{T}{T_U} \leq 1.7 \tag{5.13}
\]
For curved in plan composite steel-concrete beams with PSC (50 % of FSC) (CCBP),

\[
\frac{T}{T_U} = 0.4\left(\frac{M}{M_U}\right) + 1 \quad \text{when } 0 \leq \frac{M}{M_U} < 1 \text{ and } \frac{T}{T_U} \geq 1
\]  

(5.14)

\[
\frac{M}{M_U} = 1 \quad \text{when } \frac{M}{M_U} \geq 1 \text{ and } 0 \leq \frac{T}{T_U} \leq 1.4
\]  

(5.15)

Equations 5.12 and 5.13 have provided a simplified model to represent the flexure-torsion interaction relationship for FSC composite steel-concrete beams. Equations 5.14 and 5.15 have been represented for PSC (50 % of FSC) composite steel-concrete beams.

### 5.5 SUMMARY

This flexure-torsion interaction relationship curves had been plotted using the test results obtained from Chapters 3 and 4. Those curves were based on the ultimate strengths in flexure and torsion obtained from the straight and curved in plan composite steel-concrete beams. From the interaction relationship, the phenomenon of having an increase in the torsional moment capacity of the composite steel-concrete beams in the presence of flexure had been reinforced. However, this phenomenon was not seen for the flexural moment capacity when torsion was presence in the equation. The flexural moment capacity remained the same for both straight and curved in plan composite steel-concrete beams. Simplified models to represent the flexure-torsion interaction relationship for both straight and curved in plan composite steel-concrete beams subjected to combined torsion and flexure have been presented in this chapter.
Another finding was that the commonly used equation represented in Equation 5.2 could not be used when a load combination of flexure and torsion was applied to the composite steel-concrete beams. Equation 5.2 defined the ultimate torsional moment capacity of a composite steel-concrete section as the total contribution from the torsional resistance of the reinforced concrete slab and the steel beam. However, due to the composite action of the composite steel-concrete sections, the test torsional strengths were much more than the theoretical torsional strengths.
Table 5.1 Theoretical and test ultimate torsional moments

<table>
<thead>
<tr>
<th>Beam specimen</th>
<th>Contribution of the reinforced concrete slab (kNm)</th>
<th>Contribution of the steel beam (kNm)</th>
<th>Theoretical ultimate torsional moment (kNm)</th>
<th>Test ultimate torsional moment (kNm)</th>
<th>Test / Theory</th>
</tr>
</thead>
<tbody>
<tr>
<td>CBF-2</td>
<td>1.6</td>
<td>3.3</td>
<td>4.9</td>
<td>17</td>
<td>3.5</td>
</tr>
<tr>
<td>CBF-2</td>
<td>1.9</td>
<td>3.0</td>
<td>4.9</td>
<td>28</td>
<td>5.7</td>
</tr>
<tr>
<td>CBP-2</td>
<td>1.6</td>
<td>3.3</td>
<td>4.9</td>
<td>16</td>
<td>3.3</td>
</tr>
<tr>
<td>CBP-3</td>
<td>1.9</td>
<td>3.0</td>
<td>4.9</td>
<td>24</td>
<td>4.9</td>
</tr>
</tbody>
</table>
Figure 5.1 Contributions for ultimate torsional moment capacity
Figure 5.2 Flexure-torsion interaction relationship for straight composite beams
Figure 5.3 Modified CBF and CBP flexure-torsion interaction relationship
Figure 5.4 Flexure-torsion interaction relationship for curved composite beams
Figure 5.5 Comparison between CBF, CBP, CCBF and CCBP
Figure 5.6 Modified CCBF and CCBP flexure-torsion interaction relationship
CHAPTER 6

6 FINITE ELEMENT MODELLING

6.1 INTRODUCTION

This chapter describes the development of a 3-D finite element model (FEM) capable of simulating composite steel-concrete beams subjected to combined flexure and torsion with the influence of PSC. The model was developed using the finite element software ABAQUS. The important of the nonlinear material properties and the longitudinal and torsional interface slips between the concrete slab and steel beam were considered in the model. The model also presented additional ability to consider the torsional behaviour of the composite steel-concrete beams. The model results were been used for comparison with the test results described in Chapters 3 and 4.

6.2 DESCRIPTION OF ABAQUS

ABAQUS is a general purpose finite element analysis program for use in the numerical modelling of structural response. Stress problems can be divided into two types, static and dynamic response, depending upon whether inertial effects are significant. It permits the same analysis to be used for both the static and dynamic phases. The program is designed for ease of use on complex problems, and has a simple input language, with comprehensive data checking as well as a wide range of preprocessing and post processing output display options. Therefore, it was used to
execute the numerical analysis for the design and behaviour of composite steel-concrete beams subjected to combined flexure and torsion. ABAQUS modules consist of ABAQUS/Standard, ABAQUS/Explicit and ABAQUS/CAE.

ABAQUS has a wide range of element types such as continuum elements which consist of one-dimensional, two-dimensional and three-dimensional beams, membranes and shells. Element formulations in ABAQUS are suitable for large displacements, rotations and strains. The material models can be used for metals, sand, clay, concrete, jointed rock, plastics and rubber. The user-defined subroutines permit the inclusion of additional material models and element types. ABAQUS/Standard was employed in this thesis, as it is an ideal solution technology for static event where highly accurate stress solutions are critically important such as the experiment series I and II.

6.3 MODELLING PROCEDURE IN ABAQUS

Due to the increase computing power available in the present situation, numerical analysis using finite element method is within the reach of both academic research and engineers in practice. One of the most commonly used finite element analysis package is ABAQUS. Using ABAQUS/CAE, a wide range of input options is able to be input into the software for modelling such as geometry, element types, material properties, solution controls, loads, graphic user interfaces, automatic meshing, boundary conditions, contact and post processing controls.

The modelling procedure in ABAQUS can be broken up into four different major steps.

(1) Step 1 - Geometry and material modelling
Step 2 - Boundary and constraint conditions

Step 3 - Output analysis

Step 4 - Post-processing of results

Figure 6.1 shows the modelling procedure of ABAQUS for the thesis. Step 1 consists of the part, material property, assembly and mesh fields of the procedure.

- The part field consists of the concrete slab, shear studs, structural steel beam and reinforcing steel.
- The material property field is consisted of the input of the nonlinear material stress-strain curves of each component in the part field.
- The mesh field involved the meshing of the components using different element types and assigned the number of mesh needed for the analysis.

Step 2 provides the constraints, contacts and surface interaction model used in ABAQUS. Load and boundary conditions are also assigned in this step.

Step 3 describes how to start the analysis and obtain the output from ABAQUS after the analysis process such as the stress distribution of the composite steel-concrete beam test, ultimate strengths and deflections. A step field is created in this stage to input the load case, the time period of the step and time increment of the analysis.

Step 4 will process the model results into figures and tables for validation and comparison with test results.
6.4 MATERIAL CONSTITUTIVE RELATIONSHIPS

The material constitutive laws are used to define the stress-strain characteristics of materials used in ABAQUS. The accuracy of the analysis is dependent on the constitutive laws used to define the mechanical behaviour of the components. The objective in this section is to develop a reliable understanding of the mechanical behaviour for developing finite element composite steel-concrete models that can accurately predict their behaviour and ultimate strengths when they are subjected to combined flexure and torsion. The main components affecting the behaviour of composite steel-concrete beams are the concrete slab, steel beam, reinforcing bars and shear connectors. These components have to be carefully modelled to obtain an accurate result from the finite element analysis.

6.4.1 Concrete

One of the core components of the composite steel-concrete beams is the concrete slab. The property of the concrete can be obtained from concrete cylinder compression and splitting tests according to AS1012.9 (Standards Australia, 1999) and AS1012.10 (Standards Australia, 2000) respectively. However, only the average concrete compressive and tensile strengths are determined from the two tests. Therefore, in order to input the full stress-strain property of concrete into the ABAQUS, a concrete property model is required.

6.4.1.1 Concrete property

For the material property, the concrete stress-strain curve proposed by Carreira and Chu (1985) were recommended for plain concrete at ambient temperature. The concrete stress in compression was assumed to be linear up to 0.4
of its maximum compressive strength. Beyond this point, the concrete stress was represented as a function of strain according to Equation 6.1.

\[
\sigma_c = \frac{f'_c \gamma \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)}{\gamma - 1 + \left( \frac{\varepsilon_c}{\varepsilon'_c} \right)^\gamma}
\]

where,
- \(\varepsilon_c\) = concrete compressive strain
- \(\varepsilon'_c\) = concrete strain corresponding to \(f'_c\)
- \(f'_c\) = characteristic compressive strength of the concrete
- \(\sigma_c\) = concrete compressive stress
- \(\gamma\) = parameter used for stress-strain curve for concrete

\[= \left( \frac{f'_c}{32.4} \right)^3 + 1.55\]

For concrete in tension, the tensile stress was assumed to vary linearly with an increase in the tensile strain up to 0.1 of its concrete compressive strength. After the concrete cracked, the tensile stress decreased linearly to zero. The value of concrete strain at zero stress was taken to be 10 times the strain at failure. This assumption allowed for the continuity of the analysis even though the bottom concrete layer cracked in tension.

However, for the purpose of modelling the failure due to torsion, two different concrete material properties were needed to input into ABAQUS. The concrete slab had been divided into four different layers as shown in Figure 6.2. The concrete stress-strain curve from Carreira and Chu (1985) as shown in Figure 6.3 was used for the bottom three layers of the concrete slab. As for the top layer, a
simple modification was made to the tensile region of the concrete stress-strain curve proposed by Carreira and Chu (1985). Instead of having the concrete tensile stress to decrease linearly to zero, it was assumed to drop to zero immediately as shown in Figure 6.4. The reason was to detect failure due to torsion for the concrete slab.

6.4.1.2 Concrete smeared cracking

ABAQUS has two main options for plasticity models for concrete. Karlsson and Sorensen (2006a) stated that these plasticity models are applicable to model the inelastic behaviour of concrete. Most of these models are incremental and in that the total strain is separated into elastic and plastic parts. The solution in ABAQUS to model the nonlinear problems is to apply the loading in steps where the load in the each step is being divided into increments. Using the Newton-Raphson method, the response of the structure to a load increment is solved by iteration.

Concrete smeared cracking model has been tried out initially to model concrete. This model does not track individual macro cracks. Constitutive calculations are independently at each interaction of the FEA model to consider the presence of cracks by the way in which the cracks affect the stress and material stiffness associated with the integration point. An isotropically hardened yield surface is active when the stress is dominantly compressive and an independent crack detection surface is used to determine if a point fails by cracking. The failure surface is a linear relationship between the equivalent pressure stress and the Von Mises equivalent deviatoric stress. Once a crack forms, crack orientation is stored for subsequent calculations and subsequent cracking at the same point is orthogonal to this direction. No more than three cracks can occurs at any point. The failure ratios option in ABAQUS can be used to define the shape of the biaxial failure surface by
specifying four ratios for ultimate stress and strain values of biaxial and uniaxial stress states.

6.4.1.3 Concrete damaged plasticity

However, concrete damaged plasticity model was used later in this thesis over concrete smeared cracking model because it was better to represent the inelastic behaviour of concrete. Concrete damaged plasticity model uses an isotropic damaged elasticity in combination with isotropic tensile and compressive plasticity. This option in ABAQUS is used to define yield function, flow potential and viscosity parameters.

Lubliner et al. (1989) proposed to use the concrete model uses the yield function with the modifications suggested by Lee and Fenves (1998) to consider different progression of strength characteristics under tension and compression. The evolution of the yield surface is defined by hardening variables, known as equivalent tensile and compressive plastic strains. The equivalent tensile and compressive plastic strains can be automatically calculated by ABAQUS after the definitions of elastic material behaviour. The tensile and compressive stress–strain behaviour outside the elastic range uses concrete tension stiffening and concrete compression hardening options respectively. The tensile and compressive damage uses the concrete tension damage and concrete compression damage options respectively. This concrete model employs non-associated plastic flow potential using the Drucker-Prager hyperbolic function for the flow potential.
6.4.2 Structural steel

The material properties for steel beam and reinforcing steel are other core components of the model. The stress-strain curved for both steel beam and reinforcing steel can be obtained from the steel tensile tests according to AS 1391 (Standards Australia, 1991). For the test specimens from experimental series I and II, the stress-strain curves were obtained and presented in Chapters 3 and 4. The data was input into two different material behaviours, elastic and plastic options of the ABAQUS.

In the event where material test results were not available such the modelling for parametric study in Chapter 8, a stress-strain curve for structural steel was proposed by Loh et al. (2004b). This curve was a simple elastic-plastic model with strain hardening. The model behaviour is initially elastic after which yielding and strain hardening develops as shown in Figure 6.5. Loh et al. (2004b) found that simple linear lines were found to be sufficiently accurate to represent the stress-strain relationship. It was also assumed that the stress-strain curve was similar for structural steel in compression.

6.4.3 Shear studs

The most common type of shear connectors used in composite construction is the headed shear studs. In both the experimental series I and II, 19 mm diameter headed shear studs were used. The connectors provide the composite action between steel beam and concrete slab and also prevent vertical separation between them at their interface.
From Chapters 3 and 4, the headed shear studs were tested according to AS 1391 (Standards Australia, 1991) and their stress-strain curves were plotted. Similarly to the structural steel, the material inputs of the shear studs were divided into elastic and plastic region based on the stress-strain relationship from the tests. Since the 19 mm diameter headed shear studs were also used in the parametric study in Chapter 8, the material property of the shear studs was remained the same.

6.5 CONTACT INTERACTIONS AND BOUNDARY CONDITIONS

Contact interactions and boundary conditions are important aspects in FEM. Since the numerical simulations have to consider the physical processes in the surface to surface interactions and boundary conditions. Improper definition of boundary conditions may introduce non-physical influences into the simulation especially in this thesis, where there were more than two components been considered in the simulation such as the concrete slab, steel beam, reinforcing steel and shear studs.

Boundary conditions can be used to specify the values of basic solution variables such as displacements, rotations, warping amplitude, fluid pressures, temperatures, electrical potentials, normalised concentrations or acoustic pressures at nodes. In this thesis, the boundary conditions are the simply supported ends at the steel beam and the twisting restraints at the concrete slab.

Karlsson and Sorensen (2006c) stated that most contact problems are modelled by using surface-based contact. The structures can be either 2-D or 3-D and they can undergo either small of finite sliding for example the interface surface between the concrete slab and steel beam. Contact interactions can also be some types of kinematic constraints such surface-based tie and surface-based coupling.
constraints. Even boundary conditions are also a type of kinematic constraint in stress analysis because they define the support of the structure or given fixed displacements at nodal points. The contact interactions for the FEM are shown in Figure 6.6.

6.5.1 Concrete slab and steel beam interface

As most contact problems are modelled by using surface-based contact, therefore it was used for modelling the slipping contact interface between the concrete slab and steel beam. In order to simulate the contact interface between these two different components, the concrete had been considered as the master surface and the steel as the slave surface. Small-sliding formulation was used to allow for large rotations but that a slave node will interact with the same load area of the master surface throughout the analysis. To further define the interface relationship, a “hard” contact relationship was used to minimise the penetration of slave nodes into the master surface and to disallow the transfer of tensile stress across the interface. As for the frictional behaviour of the interface, a frictionless contact property was used.

6.5.2 Concrete slab and reinforcing steel interface

The contact interface between the concrete and reinforcing steel was of less important compared to others interface. It was assumed that there was no slippage between the concrete and reinforcing steel during the analysis. Therefore, the embedded constraint method was used in the FEM. This embedded technique was used to specify the reinforcing bar elements that lie embedded in the host element which in this case was the concrete slab to be constrained. When a node of the reinforcing truss element lied within the host element, the degrees of freedom at the
node would be eliminated and the node would become an “embedded node”. The degrees of freedom of the reinforcing steel embedded node were constrained to the interpolated values of the degrees of freedom of the host element.

6.5.3 Concrete slab and shear studs interface

Similar constraint technique was used to simulate the contact interface between the concrete slab and shear studs. Since the one of the objectives of this thesis is to investigate the behaviour of composite steel-concrete beams under the influence of partial shear connection, therefore the FEM has to be able to model or consider the longitudinal interface slip of the shear studs.

The host element was the concrete slab while only the top surfaces of the shear studs were embedded into the concrete slab.

6.5.4 Steel beam and shear studs interface

Since longitudinal interface slip was observed in the tests, therefore the FEM has to be able to show in the analysis. There were two contact interactions for the shear studs. One of the contacts was the embedded top region of the shear studs in the concrete slab and the other was the connection of the shear studs to the top flange of the steel beam. A surface-based tie constraint was used for this contact interaction. It constrained each of the nodes on the shear stud bottom surface to have the same translational and rotational motions as well as all other active degrees of freedom as the point on the steel beam surface to which it was closest.
6.5.5 Simply supported and twisting restraint conditions

Since the composite steel-concrete beams were designed as simply supported at both ends and the concrete slab had to be restrained from twisting, therefore prescribed boundary conditions had been assigned to each support. For simply supported boundary condition, one end support of the steel beam were prescribed to have its translational displacements U1, U2 and U3 restrained and the other end support with only U2 and U3 restrained.

For the twisting restraint condition, both end supports were restrained of its translational displacements U2 and U3 to prevent the concrete slab from twisting. The boundary conditions of FEM are as shown in Figure 6.7.

6.6 LOAD APPLICATIONS

Since this thesis was investigating composite steel-concrete beams subjected to combined flexure and torsion, the load applications played an important role to define different levels of parameter. Different combinations of applied torque and bending loads have significant effects on the behaviour of the composite steel-concrete beams.

6.6.1 Applied load method

There are two ways of specifying distributed loads in ABAQUS: element-based distributed loads and surface-based distributed loads. The element-based loading is used to define distributed loads on element faces and bodies, whilst the surface-based loading prescribes a distributed load on a surface. For this thesis, the surface-based loading was used because it provided a more realistic loading compared to element-based loading. Therefore, the applied load was defined using a
pressure load onto a surface specified by the loading surface area of the steel plate in
the experimental series I and II.

6.6.2 Modified Riks analysis method

The modified Riks method with standard Newton-Raphson method in
ABAQUS was employed to solve the nonlinear problems and trace the nonlinear
load-deflection curve of this thesis. The standard Newton-Raphson method solves the
nonlinear equation incrementally and iteratively by using a tangent stiffness matrix.
The modified Riks method is generally used to solve buckling involving unloading
response which uses the load magnitude as an additional unknown and solves
simultaneously for displacement. This is accomplished by using the arc length along
with the static equilibrium path in the load-displacement space. The initial
increments will be adjusted if the finite element model fails to converge. Finally, the
value of load after each increment is computed automatically. The final result will be
either the maximum value of the load or the maximum displacement value.

6.7 FINITE ELEMENT TYPE AND MESH

Karlsson and Sorensen (2006a) discussed the basic modelling concepts such
as defining nodes and surfaces, the conventions and inputs formats that should be
followed when ABAQUS was used. After the input of material properties in the
several parts created for each component, the assembly of the model was followed.
Finally, the next step was the meshing of the assembly. A good detailed mesh is a
major issue in FEM. The finer the mesh given to the component, the better the end
results will be. However, the number of mesh in the model determines the
computation time that is requested to complete the simulation. A good mesh should
have well-shaped elements with mild distortion and moderate aspect ratios. The meshing profile for all components has been shown in Figure 6.8.

6.7.1 Solid elements

3-D solid elements are volume elements which are composed of a single homogeneous material or can include several layers of different material. Karlsson and Sorensen (2006b) stated that the solid elements can be used for both linear and complex nonlinear simulations involving contact, plasticity and large deformations. The first-order interpolation elements such as hexahedral exhibit potential stiff behaviour with a slow convergence rate but prevent potentially mesh locking when a reduced integration analysis procedure is used, whereas second-order elements provide higher accuracy. However, first-order elements were used to accurately model the contact surfaces and prevent compact contact condition.

For the concrete slab, structural steel beam and shear studs, a 3-D eight node element (C3D8R) as shown in Figure 6.9 with linear approximation of displacement, reduced integration with hourglass control, eight nodes and three translational degrees of freedom was used. Stress at various points throughout the thickness of the element can be provided at each integration nodes. The element profiles for the concrete slab, steel beam and shear studs are shown in Figure 6.8.

6.7.2 Truss elements

Truss elements are used in 2-D and 3-D to model slender, line-like structures that support loading only along the axis or the centerline of the element. No moment or forces perpendicular to the centerline are supported. A 2-node straight truss element which uses linear interpolation for position and displacement and has a
constant stress is available in ABAQUS/Standard. In addition, a 3-node curved truss element which uses quadratic interpolation for position and displacement so that the strain varies linearly along the element is available in ABAQUS/Standard too.

For the reinforcing steel, 3-D two-node truss element (TSD2) with linear approximation of displacement, two nodes and three translational degrees of freedom was used as shown in Figure 6.10.

### 6.8 Sensitivity Analysis

A sensitivity analysis was carried out for the finite element modelling of the composite steel-concrete beams in this thesis. Since the main failure criterion for composite steel-concrete beams subjected to combined flexure and torsion was either the cracking of the concrete slab due to torsion or the compressive crushing of the concrete slab due to flexure, therefore the concrete material property was important in this study.

In additional, the mesh size for the concrete slab, the steel beam and shear connectors was a great importance especially in the number of concrete slab layer given to the model. The number of concrete slab layer could determine the failure of the concrete slab due to torsion, whilst the mesh size of all components could influence the initial behaviour of the model in terms of its stiffness and the also the final ultimate strength.

#### 6.8.1 Sensitivity to concrete compressive strength

The concrete compressive strength of the concrete slab was important when the composite steel-concrete beams were subjected mainly to flexural loading such as CBF-1, CBP-1, CCBF-1 and CCBP-1. To investigate the sensitivity of the model due
to compressive concrete strength, three different methods were used to predict the stress-strain behaviour of the concrete in compression. The first method was a direct method of obtaining the material property of the concrete by using strain gauges on concrete cylinder during a compression test according to AS1012.9 (Standards Australia, 1999). This method is illustrated in Figure 6.11 where strain gauges were attached to the concrete cylinder at the mid span regions.

The second method was the one being used in the finite element model of this thesis. It was based on the Equation 6.1 from Carreira and Chu (1985) mentioned in Section 6.4.1.1. The stress-strain curve from Carreira and Chu (1985) was plotted against the curve obtained from the direct method in Figure 6.12. The curve from Carreira and Chu (1985) fitted perfectly to the direct method obtained from the concrete compression test.

The third method was suggested by Lam and El-Lobby (2005) where the concrete was treated as an elastic-plastic material as shown in Figure 6.12. Lam and El-Lobby suggested that their FEM could not predict the unloading cycles due to the cracking of the concrete, therefore a bilinear stress-strain curve for the concrete compressive strength was used to predict the behaviour of their test specimens.

Figure 6.13 has shown good agreement between the experimental and model results obtained from any of the three methods. The error percentages calculated were 0.4, 1 and 1% for the direct method, Carreira and Chu (1985) and Lam and El-Lobby (2005) respectively. Therefore, the Carreira and Chu (1985) method was used since it was the best fitted curve to the actual stress-strain curve of the concrete obtained by the direct method.
6.8.2 Sensitivity to mesh size

To investigate the sensitivity of mesh size to the finite element model of composite steel-concrete beams subjected to flexure and torsion with the effects of PSC. Different mesh configurations were given to three of the major components of the composite steel-concrete beams which are the shear connectors, concrete slab and steel beam.

6.8.2.1 Shear connectors

In Figure 6.14, four different mesh configurations of the shear connectors were provided to investigate the sensitivity of the shear connecters’ mesh size to the results of the finite element model. The different mesh configurations were based on 15, 20, 25 and 30 mm mesh sizes for meshing the shear connectors.

The obtained load-deflection curves are shown in Figure 6.15 for the sensitivity of the mesh size for shear connectors in the finite element model. The results are compared with the experimental result of CBF-1. From Figure 6.15, the error percentages calculated were 3, 1, 1 and 6% for shear connectors’ mesh size of 15, 20, 25 and 30 mm respectively. The result for the ultimate load capacity starts to converge from 30 mm to 15 mm mesh size. However, the optimum mesh size with sufficient accuracy for the shear connectors was clearly 20 mm with 1% of error for the ultimate load capacity.

6.8.2.2 Concrete slab

In Figure 6.16, the sensitivity to mesh size of the concrete slab was investigated using 100, 200, and 300 mm mesh sizes. From Figure 6.17, the results for different mesh sizes of the concrete elements are plotted against the experimental
result of CCBF-1. The error percentages were 4, 11 and 14% for the concrete slab’s mesh size of 100, 200 and 300 mm respectively. Therefore, the optimum size for the concrete slab was the 100 mm mesh size for the finite element model.

6.8.2.3 Steel beam

As for the steel beam, the mesh configurations given were 20, 40 and 80 mm mesh sizes as shown in Figure 6.18. The load-deflection curves obtained from different mesh configurations are plotted against the experimental result of CFP-1 in Figure 6.19. From the comparison, the error percentages were 1, 6 and 20% for steel beam’s mesh size of 20, 40 and 80 mm respectively. As shown in the Figure 6.19, the mesh size of 40 and 80 mm was not adequate to predict the behaviour of CBP-1 in terms of ductility and load capacity. Therefore a mesh size of 20 mm for the steel beam was used in the finite element model.

6.9 SUMMARY

Finite element analysis is a number technique for finding approximate solutions of partial differential equations as well as of integral equations. The solution approach is based either on eliminating the differential equation completely or rendering the partial differential equations into an approximating system of ordinary differential equations. These differential equations are then numerically integrated using standard techniques such as Euler’s or Runge-Kutta methods. In this thesis, it was used to model the behaviour of composite steel-concrete beams subjected to combined flexure and torsion.

The FEM was classified down into four different steps which included the geometry and material modelling, boundary and constraint conditions, output
analysis and post-processing of results. This chapter emphasises the importance of providing accurate material properties and choosing the right element and mesh types for all different components of the composite steel-concrete beams. In addition, the boundary and constraints conditions of the FEM provide the same external environments as the experimental series I and II. The contact interactions between each component are explained in detail in this chapter to allow for the modelling of the interaction between each component during analysis. Ultimately, this FEM had the ability to consider partial shear connection and investigate the nonlinear behaviour of the composite steel-concrete beams subjected to combined flexure and torsion.
Figure 6.1 Modelling procedure of ABAQUS
Figure 6.2 Concrete slab material properties allocation
Figure 6.3 Stress-strain curve for concrete, (Carreira and Chu, 1985) (Material 1)
Figure 6.4 Modified stress-strain curve for concrete (Material 2)
Figure 6.5 Stress-strain curve for structural steel, (Loh et al., 2004b)
Figure 6.6 Contact interactions for FEM

Concrete-stud interface: Embedded
Concrete-steel interface: Surface to surface contact
Concrete-rebar interface: Embedded
Steel-stud interface: Tie constraint
Figure 6.7 Boundary conditions for FEM
Figure 6.8 Finite element types and mesh for FEM

- Shear stud
- Concrete slab
- Steel beam
Figure 6.9 C3D8R 3-D solid element, (Mirza and Uy, 2009a)

Figure 6.10 3-D 2-node truss elements (T3D2) for reinforcing steel
Figure 6.11 Direct method to determine stress-strain curve for concrete
Figure 6.12 Stress-strain curves for concrete in compression
Figure 6.13 Sensitivity to concrete compressive strength
Figure 6.14 Different mesh sizes for shear connectors
Figure 6.15 Sensitivity to mesh size for shear connectors
Figure 6.16 Different mesh sizes for concrete slab
Figure 6.17 Sensitivity to mesh size for concrete slab
Mesh size of 20 mm

Mesh size of 40 mm

Mesh size of 80 mm

Figure 6.18 Different mesh sizes for steel beam
Figure 6.19 Sensitivity to mesh size for steel beam
CHAPTER 7

COMPARISONS BETWEEN FEM AND EXPERIMENTAL TESTS

7.1 INTRODUCTION

In this chapter, the accuracy of the finite element model described in Chapter 6 is validated herein by comparison with experimental results of composite steel-concrete beams subjected to combined flexure and torsion incorporating the effects of PSC. There were two different series of experimental tests conducted in this thesis. The first experimental series I was carried out to investigate the behaviour of six straight composite steel-concrete beams, three concrete slabs and three steel beams subjected to combined flexure and torsion in Chapter 3. The next experimental series II was dealing with curved in plan composite steel-concrete beams also subjected to combined flexure and torsion in Chapter 4. The complete details of the test specimens with their corresponding material properties are summarised in their own respective chapters. Comparisons were made between each test specimen with their own respective model. The reliability of the FEM was established by comparing the experimental results with the model results in terms of the load-deflection response and the ultimate strength capacities.
7.2 COMPARISONS BETWEEN MODELS AND EXPERIMENTAL SERIES I

To validate the FEM, comparisons were made with the experimental results of the six straight composite steel-concrete beams (CBF1 to CBF-3 and CBP-1 to CBP-3), three concrete slabs (CS-1 to CS-3) and three steel beams (SB-1 to SB-3) presented earlier in Chapter 3. The beam details including the material properties of the components obtained from relevant material tests are outlined in Tables 7.1, 7.2 and 7.3. The results for the beams are compared in terms of the load-deflection response at the mid span. Each comparison has been discussed in the Section 7.2.1.

7.2.1 Validation of the finite element model

7.2.1.1 Composite steel-concrete beam CBF-1

CBF-1 was a composite steel-concrete beam designed with FSC and subjected to only pure flexure in the experimental series I. CBF-1 has a span length of 4.1 m and was loading with a line load at the mid span of the composite steel-concrete beams. It was simply-supported and restrained from twisting at both support ends. CBF-1 had achieved an ultimate flexural moment capacity of 220 kNm for the test. From the 3-D-model using ABAQUS, a predicted flexural moment capacity of 218 kNm was obtained. From Figure 7.1, the load-deflection curve from the model has good agreement with the experimental results for CBF-1. The curve has provided similar peak load and shown similar ductility as CBF-1. The model also indicated a drop in the applied load due to the failure of the shear connectors similar to CBF-1.
7.2.1.2 Composite steel-concrete beam CBF-2

CBF-2 was a composite steel-concrete beam designed with FSC and subjected to combined flexure and torsion in the experimental series I. CBF-2 was loading with two point loads at different locations to induce torque to the composite steel-concrete beams. It was simply-supported and restrained from twisting at both support ends as well. From the test, CBF-2 achieved both ultimate flexural and torsional moment capacities of 214 and 17 kNm respectively. From the 3-D model, the predicted flexural and torsional moment capacities were 198 and 15 kNm respectively. From Figure 7.2, the load-deflection curve from the model has good agreement with the experimental results for CBF-2. The curve has provided slightly lower peak load and shown similar ductility until the failure point as the CBF-2. The initial stiffness of the curves is similar.

7.2.1.3 Composite steel-concrete beam CBF-3

CBF-3 was similar to CBF-2 except for the applied load combination. More torque was applied to the composite steel-concrete beams in the test. From the test, CBF-3 achieved both ultimate flexural and torsional moment capacities of 197 and 28 kNm respectively. From the 3-D model, the predicted flexural and torsional moment capacities were 193 and 25 kNm respectively. From Figure 7.3, the load-deflection curve from the model has good agreement with the experimental results for CBF-3. The curve has provided similar peak load and shown higher ductility than CBF-3. The initial stiffness of the model curve is similar to those of the test curve. However, the model curve tends to lower its stiffness as the load increased. Nevertheless, both curves obtained similar peak load at slightly different mid span deflection.
7.2.1.4 Composite steel-concrete beam CBP-1

CBP-1 was a composite steel-concrete beam designed with PSC and subjected to only pure flexure in the experimental series I. CBP-1 had achieved an ultimate flexural moment capacity of 188 kNm for the test. From the 3-D-model using ABAQUS, a predicted flexural moment capacity of 186 kNm was obtained. From Figure 7.4, the load-deflection curve from the model has excellent good agreement with the experimental results for CBP-1. The curve has provided similar peak load and shown similar ductility as CBP-1. The model curve stops at the same mid span deflection as CBP-1. Compared to CBF-1 model curve, the model curve for CBP-1 has a slightly lower load carrying capacity.

7.2.1.5 Composite steel-concrete beam CBP-2

CBP-2 was a composite steel-concrete beam designed with PSC and subjected to combined flexure and torsion in the experimental series I. From the test, CBF-3 achieved both ultimate flexural and torsional moment capacities of 194 and 16 kNm respectively. From the 3-D model, the predicted flexural and torsional moment capacities were 193 and 14 kNm respectively. From Figure 7.5, the load-deflection curve from the model has good agreement with the experimental results for CBP-2. The curve has provided similar peak load and shown slightly lower ductility than CBP-2.

7.2.1.6 Composite steel-concrete beam CBP-3

CBP-3 was a composite steel-concrete beam designed with PSC and subjected to combined flexure and torsion in the experimental series I. From the test, CBF-3 achieved both ultimate flexural and torsional moment capacities of 177 and
24 kNm respectively. From the 3-D model, the predicted flexural and torsional moment capacities were 159 and 20 kNm respectively. From Figure 7.6, the load-deflection curve from the model has good agreement with the experimental results for CBP-3. The curve has provided similar peak load and shown similar ductility as CBP-3.

7.2.1.7 Comparisons of the finite element model

From Figures 7.7 and 7.8, cross-sectional views of the FEM at the point load for CBF and CBP are presented respectively. It can be clearly seen that the stress regions for the shear connectors were the greatest when the composite steel-concrete beams were subjected to pure flexure for example CBF-1 and CBP-1. However, the stress regions for the shear connectors on CBF-3 and CBP-3 were greater than CBF-2 and CBP-2 due to excessive twisting of the concrete slab especially at the head regions of the shear connectors.

Furthermore, the twist angles of the concrete slabs were totally different from the steel beams as shown in Figures 7.7 and 7.8 for composite steel-concrete beams subjected to combined flexure and torsion such as CBF-2, CBF-3, CBP-2 and CBP-3. This observation supported the finding that the curvatures of the concrete slab and steel beam were different in a composite steel-concrete beam that was subjected to combined flexure and torsion during the test.

From Figures 7.9 and 7.10, isotropic views of the FEM for CBF and CBP are presented. The stress regions for both CBF-1 and CBP-1 were perpendicular to the concrete slabs, whilst the stress regions for CBF-2, CBF-3, CBP-2 and CBP-3 were parallel to the concrete slabs. This clearly indicating flexural concrete cracking for
CBF-1 and CBP-1, whilst a combination of torsional and flexural concrete cracking for CBF-2, CBF-3, CBP-2 and CBP-3.

7.2.1.8 Concrete slabs CS-1, CS-2 and CS-3

CS-1, CS-2 and CS-3 were concrete slabs and subjected to pure flexure and combined flexure and torsion in the experimental series I. CS-1, CS-2 and CS-3 have a span length of 4.1 m. It was simply-supported and restrained from twisting at both support ends. From Figures 7.11 to 7.13, the load-deflection curve from the model has good agreement with the experimental results for CS-1, CS-2 and CS-3 respectively. The curve has provided similar curve profile but lower similar ductility than the test curves. Another point to take note is that the models did not indicate any increase in the load carrying capacity in the presence of torsion as shown for the test curves. From Figures 7.14 and 7.15, there is no excessive twisting of the concrete slabs or parallel stress region at the isotropic views. All these observation indicated that the concrete slabs have failed in flexure.

7.2.1.9 Steel beams SB-1, SB-2 and SB-3

SB-1, SB-2 and SB-3 were steel beams and subjected to pure flexure and combined flexure and torsion in the experimental series I. SB-1, SB-2 and SB-3 have a span length of 4.1 m and was loading with the lever arm welded on to their top flange. It was simply-supported and restrained from twisting at both support ends. From Figures 7.16 to 7.18, the load-deflection curve from the model has good agreement with the experimental results for SB-1, SB-2 and SB-3 respectively. Especially with SB-1, the model curve perfectly matched the curve profile of the test results. The stiffness, strength and ductility of the model curve are the same as the test curve. As for SB-2, the model has a lower peak load than the test curve. For SB-
3, the model curve has slightly higher peak load than the test curve. Nevertheless, all models were able to determine the failure of the steel beam due to torsion for SB-2 and SB-3 and flexure for SB-1. From Figure 7.19, the cross-sectional and isotropic views of the models at point load are presented. The FEM has indicated clearly that the steel beam has better resistance to bending than to twisting. The deflection of the SB-1 was clearly greater than SB-2 and SB-3. The torsional resistance of the steel beams was clearly due to the top and bottom flanges indicated by their high stress level compared to the web.

### 7.2.2 Discussion

There was generally good agreement between the experimental results and the finite element models in terms of the load-deflection relationship over the entire loading profile until failure. The initial stiffness, ultimate flexural and torsional moment capacities were closely modelled. For beams that were governed by torsional failure, it can be observed that the analysis was able to predict this with reasonable accuracy. Even for beams under pure flexure, ductility or post failure profile was shown in the models for CBF-1, CBP-1 and SB-1.

Based on the accuracy of results from the experimental-numerical comparisons, it can be concluded that the numerical model from ABAQUS in Chapter 6 is suitable and reliable to simulate the behaviour of straight composite steel-concrete beams, concrete slabs and steel beams. The load-deflection relationship was able to be modelled accurately. For beams subjected to combined flexure and torsion, the failure mode was not governed by connector fracture, but the torsional cracking of the concrete slab. This was also predicted reasonably well by the model.
From Table 7.7, it presents a summary of the numerical comparison between the experimental and analysis results in terms of the flexural and torsional moment capacities. The comparison was accurate with the model to test result ratios being close than one. The initial stiffness of the model was deemed to be satisfactory in view of the presence of high nonlinearity in the behaviour of the composite steel-concrete beams.

The behaviour of a partial strength was shown in the model with difference in the ultimate flexural strength of the composite steel-concrete beams CBF-1 and CBP-1. Longitudinal interface slip was observed from the model for both FSC and PSC composite steel-concrete beams, even though the load-slip relationship was not compared in this chapter.

### 7.3 COMPARISONS BETWEEN MODELS AND EXPERIMENTAL SERIES II

To further validate the numerical model, comparisons were also made with the experimental results of the eight curved in plan composite steel-concrete beams (CCBF1 to CCBF-4 and CCBP-1 to CCBP-4 presented earlier in Chapter 4. The beam details including the material properties of the components obtained from relevant material tests are outlined in Tables 7.4, 7.5 and 7.6. The results for the beams are compared in terms of the load-deflection response at the mid span. Each comparison has been discussed in the Section 7.3.1.
7.3.1 Validation of the finite element model

7.3.1.1 Composite steel-concrete beam CCBF-1

CCBF-1 was a curved in plan composite steel-concrete beam designed with FSC and subjected to combined flexure and torsion in the experimental series II. CCBF-1 has a span length of 6 m and was loading with a line load at the mid span. It was simply-supported and restrained from twisting at both support ends. From the test, CCBF-1 achieved both ultimate flexural and torsional moment capacities of 207 and 33 kNm respectively. From the 3-D model, the predicted flexural and torsional moment capacities were 216 and 33 kNm respectively. From Figure 7.20, the load-deflection curve from the model has good agreement with the experimental results for CCBF-1. The curve has provided higher peak load and shown lower ductility than CCBF-1.

7.3.1.2 Composite steel-concrete beam CCBF-2

From the test, CCBF-2 achieved both ultimate flexural and torsional moment capacities of 155 and 24 kNm respectively. From the 3-D model, the predicted flexural and torsional moment capacities were 162 and 26 kNm respectively. From Figure 7.21, the load-deflection curve from the model has good agreement with the experimental results for CCBF-2. The initial stiffness of the model curve is similar to those of the test curve. However, the slope of the test curve decreased, which could be due to the crushing of the concrete that are not reflected in the model. More detailed meshing of the concrete slab could improve the profile of the model curve. However, with more meshes in the model, more computing time will be required. The model curve did obtain similar peak load as the test curve.
7.3.1.3 Composite steel-concrete beam CCBF-3

From the test, CCBF-3 achieved both ultimate flexural and torsional moment capacities of 107 and 24 kNm respectively. From the 3-D model, the predicted flexural and torsional moment capacities were 107 and 24 kNm respectively. From Figure 7.22, the load-deflection curve from the model has good agreement with the experimental results for CCBF-3. Since the failure mode of CCBF-3 was torsional failure. The model was stopped as soon as the concrete slab reached its limit.

7.3.1.4 Composite steel-concrete beam CCBF-4

From the test, CCBF-4 achieved both ultimate flexural and torsional moment capacities of 72 and 24 kNm respectively. From the 3-D model, the predicted flexural and torsional moment capacities were 77 and 26 kNm respectively. From Figure 7.23, the load-deflection curve from the model has good agreement with the experimental results for CCBF-4. The same observation can be seen from CCBF-3 and CCBF-4 models, the failure of both the composite steel-concrete beams was torsional failure. Therefore, the models were unable to continue to identify further failure such as the failure of the shear connectors of the buckling of the steel beams.

7.3.1.5 Composite steel-concrete beam CCBP-1

CCBP-1 was a curved in plan composite steel-concrete beam designed with PSC and subjected to combined flexure and torsion in the experimental series II. Since CCBP-1 has the smallest curvature compared to CCBP-2 to 4. The failure mode of the CCBP-1 was expected to be flexural failure. From Figure 7.24, the load-deflection curve from the model has good agreement with the experimental results for CCBP-1. The curve has provided higher peak load. But similar ductility is shown
in the figure. From the test, CCBP-1 achieved both ultimate flexural and torsional moment capacities of 182 and 29 kNm respectively. From the 3-D model, the predicted flexural and torsional moment capacities were 198 and 29 kNm respectively.

7.3.1.6 Composite steel-concrete beam CCBP-2

The failure mode of the CCBP-2 was expected to be torsional failure. From Figure 7.25, the load-deflection curve from the model has excellent agreement with the experimental results for CCBP-2. The curve has provided similar peak load and ductility as CCBP-2. As mentioned before, once the model has reached or failed due to torsional failure, the model would not able to continue to monitor other failure after the torsional moment capacity has been reached. As shown in the Figure 7.25, there is a step profile of the test curve, which is not shown in the model curve. From the test, CCBP-2 achieved both ultimate flexural and torsional moment capacities of 171 and 25 kNm respectively. From the 3-D model, the predicted flexural and torsional moment capacities were 170 and 27 kNm respectively.

7.3.1.7 Composite steel-concrete beam CCBP-3

From the test, CCBP-3 achieved both ultimate flexural and torsional moment capacities of 116 and 28 kNm respectively. From the 3-D model, the predicted flexural and torsional moment capacities were 129 and 31 kNm respectively. From Figure 7.26, the load-deflection curve from the model has good agreement with the experimental results for CCBP-3. The curve has provided higher peak load. But similar ductility is shown in the figure. Their curve profiles are very similar to each other.
7.3.1.8 Composite steel-concrete beam CCBP-4

From Figure 7.27, the load-deflection curve from the model has good agreement with the experimental results for CCBP-4 at the first half of the curve. Since the first reduction of the test curve was due to the torsional failure, then model would have stopped at the point. However, the test was continued with shearing off of the concrete slab, which could not be modelled with our 3-D finite element model. However, the model did successfully predict the peak load of the test. From the test, CCBP-4 achieved both ultimate flexural and torsional moment capacities of 82 and 28 kNm respectively. From the 3-D model, the predicted flexural and torsional moment capacities were 77 and 26 kNm respectively.

7.3.1.9 Comparisons of the finite element model

From Figures 7.28 and 7.29, the cross-sectional views of the FEM for CCBF and CCBP are presented respectively. Beams with smaller span to radius ratio experienced lateral buckling of the steel web as shown for CCBF-1 and CCBP-1. The stress level of the shear connectors was higher as well. The failure of beams with smaller span to radius ratio depended on the strength of the shear connectors and the steel beam, since their failure mode was flexural failure. For beam with higher span to radius ratio, the stress regions for both steel beam and shear connectors were not high at the failure load, which indicated that the failure criterion was torsional failure. The reason was that from Figures 7.30 and 7.31, the isotropic views of the FEM for CCBF and CCBP are presented with all the concrete slabs stressed to their maximum compressive stress at the mid span during the failure.
7.3.2 Discussion

The finite element model proposed herein is sufficiently accurate and suitable to model composite steel-concrete beams subjected to combined flexure and torsion. The behaviour of straight and curved in plan composite steel-concrete beams as well as steel beams and concrete slabs was adequately simulated until failure was reached. The load-deflection response and the ultimate strengths were able to be predicted with certain accuracy. For curved in plan composite steel-concrete beams with low level of curvature where the primary failure mode was governed by the flexural failure was well predicted.

A summary of results from the experimental tests and finite element models is presented in Table 7.7. The average predicted flexural moment capacity was established to be 98% of the ultimate flexural moment capacities achieved by the beams. In addition, the average predicted torsional moment capacity was established to be 97% of the ultimate torsional moment capacities achieved by the beams. From these results, it can be observed that the failure criterion indicated in the concrete slab used to predict the occurrence of concrete failure due to torsion which was described in Section 6.4.1 was reasonably accurate. It was found during the analysis that the torsional failure criterion was accurate when more than four layers were used in the modelling of the concrete slab.

7.4 SUMMARY

The analysis results based on the finite element model described in Chapter 6 generally compared well with the experimental behaviour of various composite steel-concrete beams tested in Chapters 3 and 4. Those composite steel-concrete beams were subjected to combine flexure and torsion during the tests. The full
flexural and torsional response of the beams was adequately simulated until failure. Failure modes of the concrete slabs and shear connector fracture occurred in the experimental tests were accurately traced and observed. For straight and curved in plan composite steel-concrete beams subjected to high torque where the primary failure mode was governed by the torsional failure of the concrete slab, this was also well predicted. Based on the comparisons between the ultimate strengths obtained from the finite element models and the experimental series I and II, it is observed that they are in good agreement.

The finite element models have provided further proof that due to the twisting of the composite steel-concrete beams, different curvatures for the steel beam and concrete slab can occur when torsion is presence. The cross-sectional views of the finite element models had also provided evidence that the shear connectors were subjected to uplift forces when the twisting angle of the steel beam was different from the twisting angle of the concrete slab. Lastly, lateral buckling of the steel web was presented for straight composite steel-concrete beams and curved in plan composite steel-concrete beams with low span to radius curvature in the finite element models.
Table 7.1 Composite beam details for experimental series I (Part 1)

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Table 7.3 Steel beam and concrete slab details for experimental series I

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<tr>
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</tr>
<tr>
<td>Yield stress (N/mm$^2$)</td>
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</tr>
<tr>
<td>Ultimate stress (N/mm$^2$)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Reference</td>
</tr>
<tr>
<td>------------------------</td>
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</tr>
<tr>
<td>Beam identification</td>
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</tr>
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<td>Type (Bottom)</td>
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</tr>
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<td>Ultimate stress (N/mm$^2$)</td>
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<td>Stirrup steel</td>
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<td>Ultimate stress (N/mm$^2$)</td>
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Table 7.7 Comparison table in terms of ultimate strengths

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<tr>
<th>Beam</th>
<th>$B$ (%)</th>
<th>Ultimate flexural moment (kNm)</th>
<th>Ultimate torsional moment (kNm)</th>
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<tr>
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<td>218</td>
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<td>CBF-2</td>
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<td>198</td>
</tr>
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<td>CBF-3</td>
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<td>CBP-1</td>
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<td>CBP-3</td>
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<td>159</td>
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<tr>
<td>SB-1</td>
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<td>114</td>
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<td>CCBP-4</td>
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<td>82</td>
<td>77</td>
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Avg.   0.98                  0.97
Std. Dev. 0.11              0.15
Figure 7.1 Comparison between FEM and CBF-1
Figure 7.2 Comparison between FEM and CBF-2
Figure 7.3 Comparison between FEM and CBF-3
Figure 7.4 Comparison between FEM and CBP-1
Figure 7.5 Comparison between FEM and CBP-2
Figure 7.6 Comparison between FEM and CBP-3
Figure 7.7 Cross-sectional views of FEM at point load for CBF
Figure 7.8 Cross-sectional views of FEM at point load for CBP
Figure 7.9 Isotropic views of FEM for CBF
Figure 7.10 Isotropic views of FEM for CBP
Figure 7.11 Comparison between FEM and CS-1
Figure 7.12 Comparison between FEM and CS-2
Figure 7.13 Comparison between FEM and CS-3
Figure 7.14 Cross-sectional views of FEM at point load for CS
Figure 7.15 Isotropic views of FEM for CS
Figure 7.16 Comparison between FEM and SB-1
Figure 7.17 Comparison between FEM and SB-2
Figure 7.18 Comparison between FEM and SB-3
Figure 7.19 Cross-sectional and isotropic views of FEM at point load for SB
Figure 7.20 Comparison between FEM and CCBF-1
Figure 7.21 Comparison between FEM and CCBF-2
Figure 7.22 Comparison between FEM and CCBF-3
Figure 7.23 Comparison between FEM and CCBF-4
Figure 7.24 Comparison between FEM and CCBP-1
Figure 7.25 Comparison between FEM and CCBP-2
Figure 7.26 Comparison between FEM and CCBP-3
Figure 7.27 Comparison between FEM and CCBP-4
Figure 7.28 Cross-sectional views of FEM at point load for CCBF
Figure 7.29 Cross-sectional views of FEM at point load for CCBP
Figure 7.30 Isotropic views of FEM for CCBF
Figure 7.31 Isotropic views of FEM for CCBP

CCBP-1

CCBP-2

CCBP-3

CCBP-4

341
CHAPTER 8

8  PARAMETRIC STUDY

8.1  INTRODUCTION

The proposed 3-D finite element model has been validated with experimental results described in Chapter 7. It was proved to be able to simulate the overall behaviour of concrete slabs, steel beams and composite steel-concrete beams subjected to combined flexure and torsion. Therefore, a parametric study was undertaken using the model that was properly calibrated to investigate other parameters for both the straight and curved in plan composite steel-concrete beams subjected to flexure and torsion.

This chapter will present the results for the effects of different span length and PSC on the behaviour of the composite steel-concrete beams subjected to flexure and torsion. Several models have been developed to each span length of 6, 8, 10, 12 and 14 m. The levels of shear connection taken into consideration of the parametric study were 50 and 100 %. All the ultimate strengths from the models would then be used to develop the flexure-torsion interaction relationships for each span length and each level of shear connection. Comparisons were be made between the interaction relationship for discussion.
8.2 BEAM SELECTION FOR PARAMETRIC STUDY

The beam selection for the parametric study was based on a typical arrangement of a 3-bay by 3-bay storey frame as shown in Figure 8.1. The beam specimens were assumed to be the primary beams in the floor system and the span of the primary beam was taken as the designated span length. Uniformly distributed office loading was assumed and taken as 3 kN/m$^2$ according to Australian Standards AS/NZS1170.1 (Standards Australia, 2002) with an additional dead load of 1 kN/m$^2$. Only internal simply supported primary beams were being considered in the design. The rigid plastic analysis method was used to determine the beam section size.

The initial concrete slab was assumed to be 120 mm in thickness with a nominal compressive strength of 30N/mm$^2$. The effective width of the concrete slab was based on the Eurocode 4 (British Standards Institution, 2005). Shear connectors were assumed to be 19 mm in diameter with a yield stress of 500 N/mm$^2$, whilst the reinforcing steel bars were taken as 12 mm in diameter with a yield stress of 500 N/mm$^2$ too. The stirrup reinforcing steel bars were taken as 10 mm in diameter with a yield stress of 500 N/mm$^2$ too.

Serviceability limit state checks for deflection, vibration and crack control were undertaken as part of the design. However, the design of the specimens was mostly based on the strength limit state for flexure. The selected beam sizing was summarised Tables 8.1 and 8.2 with respect to the designated span length.
8.3 PARAMETRIC STUDY FOR STRAIGHT COMPOSITE STEEL-CONCRETE BEAMS

Around 20 finite element models were used for each span length to simulate the behaviour of the composite steel-concrete beams subjected to combined flexure and torsion with partial shear connection. As mentioned, 5 different span lengths had been chosen for the parametric study. There were 6, 8, 10, 12 and 14 m in composite steel-concrete beam span length. Two different levels of shear connection (100 and 50 %) were designed to investigate the effects of partial shear connection. The strength interaction curves were plotted for each span length using several different FEM subjected to different level of applied flexure and torque onto the straight composite steel-concrete beams. Each co-ordinate was obtained from the ultimate flexural and torsional moment capacities of each FEM. Unified strength interaction ratio curves were also plotted using the strength interaction curves to allow the proposed interaction relationship to be applied to other beam sizing with the same span length.

8.3.1 6 m straight composite steel-concrete beams

For 6 m straight composite steel-concrete beams subjected to combined flexure and torsion, the flexure-torsion interaction relationship curves and the flexure-torsion interaction relationship ratio curves have similar profile between the FSC and PSC beams. From Figure 8.7, the strength interaction ratio curves for both FSC and PSC are actually the same. However, the flexural strength of the FSC composite steel-concrete beams was higher than PSC composite steel-concrete beams as illustrated in Figure 8.2. Both the figures show the phenomenon that in the presence of flexure, the torsional moment capacity of the composite steel-concrete
beams increases, while the flexural moment capacity remains the same in the presence of torsion.

8.3.2 8 m straight composite steel-concrete beams

For 8 m straight composite steel-concrete beams, the flexure-torsion interaction relationship curves and the flexure-torsion interaction relationship ratio curves also have similar profile. From Figure 8.8, the strength interaction ratio curves for both FSC and PSC are actually the same. However, the flexural strength of the FSC composite steel-concrete beams was higher than PSC composite steel-concrete beams as illustrated in Figure 8.3. Both the figures show the phenomenon that in the presence of flexure, the torsional moment capacity of the composite steel-concrete beams increases, while the flexural moment capacity remains the same in the presence of torsion.

8.3.3 10 m straight composite steel-concrete beams

For 10 m straight composite steel-concrete beams, the strength interaction ratio curves for both FSC and PSC are actually the same again as shown in Figure 8.9. The flexural strength of the FSC composite steel-concrete beams was higher than PSC composite steel-concrete beams as shown in Figure 8.4. Both the figures show the phenomenon that in the presence of flexure, the torsional moment capacity of the composite steel-concrete beams increases, while the flexural moment capacity remains the same in the presence of torsion.

8.3.4 12 m straight composite steel-concrete beams

For 12 m straight composite steel-concrete beams, the strength interaction ratio curves for both FSC and PSC are actually the same as shown in Figure 8.10.
However, the flexural strength of the FSC composite steel-concrete beams was higher than PSC composite steel-concrete beams as shown in Figure 8.5. Both the figures show the phenomenon that in the presence of flexure, the torsional moment capacity of the composite steel-concrete beams increases, while the flexural moment capacity remains the same in the presence of torsion.

8.3.5 14 m straight composite steel-concrete beams

For 14 m straight composite steel-concrete beams, the strength interaction ratio curves for both FSC and PSC are actually the same as shown in Figure 8.11. The flexural strength of the FSC composite steel-concrete beams was higher than PSC composite steel-concrete beams as shown in Figure 8.6. Both the figures show the phenomenon that in the presence of flexure, the torsional moment capacity of the composite steel-concrete beams increases, while the flexural moment capacity remains the same in the presence of torsion.

8.3.6 Discussion

8.3.6.1 Effects of the beam span length

From Figures 8.12 and 8.13, the ultimate flexural and torsional moment capacities are represented by the strength interaction curves for FSC and PSC respectively. It is clearly shown that the composite steel-concrete beams have an increase in the torsional and flexural moment capacities with an increase in span length. This is due to the increase in the steel beam and concrete slab sizes as illustrated in Tables 8.1 and 8.2.

However, from the strength interaction ratio curves plotted in Figures 8.14 and 8.15 for both FSC and PSC respectively, the phenomenon that in the presence of
flexure, the torsional moment capacity of the composite steel-concrete beams increases, while the flexural moment capacity remains the same in the presence of torsion was seen. The benefit for the torsional strength also increases with span length as shown in the non-dimensional interaction ratio curves. Interestingly, the benefit or the rate to increase in torsional strength drops from 8 to 10 m span lengths and that increase back again from 10 to 14 m span lengths. This observation was noticed for both FSC and PSC straight composite steel-concrete beams. The reason is that the neutral axes for both the 6 and 8 m span lengths situated at the concrete slab section, whilst the neutral axes for 10, 12 and 14 m span lengths situated at the steel beam section. The change in the location of the neutral axis affects the rate of increase in the torsional strength of the straight composite steel-concrete beams.

8.3.6.2 Effects of the degree of shear connection

As for the effects of PSC, from Figures 8.2 to 8.6, the torsional strength of the straight composite steel-concrete beams were the similar between the FSC and PSC composite steel-concrete beams. The PSC did not affect the torsional strength of the composite steel-concrete beams. In terms of flexural strength, it is a norm that FSC design achieved higher flexural strength than PSC design. In additional, the difference in the ultimate flexural strength was at most 19 % for the 6 m span length composite steel-concrete beams. This difference was deceasing with the increase in the span length with the difference being 2 % for the 14 m span length.

From the strength ratio interaction curves, the curves are the same for both PSC and FSC design beams. This indicates that the rate of increase and decrease of the ultimate flexural and torsional moment capacities are the same for both PSC and FSC design beams.
8.3.7 Proposed design models

Based on the FEM results, Figure 8.16 provides a proposed flexure-torsion interaction relationship model for 6 and 8 m straight composite steel-concrete beams. The interaction relationship model will not be affected by the level of shear connection as discussed in Section 8.3.6.2. To use this proposed design model, the neutral axis of the composite steel-concrete section has to be at the concrete slab section before the model can be used. The flexure-torsion interaction relationship equations can be written as following.

For 6 m straight composite steel-concrete beams,

\[
\frac{T}{T_U} = 0.1 \left( \frac{M}{M_U} \right) + 1 \quad \text{when} \quad 0 \leq \frac{M}{M_U} < 1 \quad \text{and} \quad \frac{T}{T_U} \geq 1 \tag{8.1}
\]

\[
\frac{M}{M_U} = 1 \quad \text{when} \quad \frac{M}{M_U} \geq 1 \quad \text{and} \quad 0 \leq \frac{T}{T_U} \leq 1.1 \tag{8.2}
\]

For 8 m straight composite steel-concrete beams,

\[
\frac{T}{T_U} = 0.3 \left( \frac{M}{M_U} \right) + 1 \quad \text{when} \quad 0 \leq \frac{M}{M_U} < 1 \quad \text{and} \quad \frac{T}{T_U} \geq 1 \tag{8.3}
\]

\[
\frac{M}{M_U} = 1 \quad \text{when} \quad \frac{M}{M_U} \geq 1 \quad \text{and} \quad 0 \leq \frac{T}{T_U} \leq 1.3 \tag{8.4}
\]

Equations 8.1 and 8.2 have provided a simplified and conservative model to represent the flexure-torsion interaction relationship for 6 m straight composite steel-concrete beams, whilst Equations 8.3 and 8.4 have been represented for 8 m straight
composite steel-concrete beams. Based on the beam selection calculation from Section 8.2, the optimum beam sizing for the designated span length of 6 and 8 m have neutral axis location at the concrete slab.

For span length of 10, 12 and 14 m, Figure 8.17 provides another proposed flexure-torsion interaction relationship model. The interaction relationship model will also not be affected by the level of shear connection as discussed in Section 8.3.6.2. To use this proposed design model, the neutral axis of the composite steel-concrete section has to be at the steel beam section before the model can be used. The flexure-torsion interaction relationship equations can be written as following.

For 10 m straight composite steel-concrete beams,

\[
\frac{T}{T_u} = 0.2 \left( \frac{M}{M_u} \right) + 1 \quad \text{when} \ 0 \leq \frac{M}{M_u} < 1 \ \text{and} \ \frac{T}{T_u} \geq 1 \quad (8.5)
\]

\[
\frac{M}{M_u} = 1 \quad \text{when} \ \frac{M}{M_u} \geq 1 \ \text{and} \ 0 \leq \frac{T}{T_u} \leq 1.2 \quad (8.6)
\]

For 12 m straight composite steel-concrete beams,

\[
\frac{T}{T_u} = 0.3 \left( \frac{M}{M_u} \right) + 1 \quad \text{when} \ 0 \leq \frac{M}{M_u} < 1 \ \text{and} \ \frac{T}{T_u} \geq 1 \quad (8.7)
\]

\[
\frac{M}{M_u} = 1 \quad \text{when} \ \frac{M}{M_u} \geq 1 \ \text{and} \ 0 \leq \frac{T}{T_u} \leq 1.3 \quad (8.8)
\]

For 14 m straight composite steel-concrete beams,
\[
\frac{T}{T_U} = 0.5 \left( \frac{M}{M_U} \right) + 1 \quad \text{when } 0 \leq \frac{M}{M_U} < 1 \text{ and } \frac{T}{T_U} \geq 1 \quad (8.9)
\]

\[
\frac{M}{M_U} = 1 \quad \text{when } \frac{M}{M_U} \geq 1 \text{ and } 0 \leq \frac{T}{T_U} \leq 1.5 \quad (8.10)
\]

Equations 8.5 and 8.6 have provided a simplified and conservative model to represent the flexure-torsion interaction relationship for 10 m straight composite steel-concrete beams, whilst Equations 8.7 and 8.8 have been represented for 12 m straight composite steel-concrete beams. From 14 m straight composite steel-concrete beams, Equations 8.9 and 8.10 were be used. For the both proposed models, interpolation of the curves can be used for other span lengths such as 7 or 9 m.

8.4 PARAMETRIC STUDY FOR CURVED IN PLAN COMPOSITE STEEL-CONCRETE BEAMS

Parametric study also covered the curved in plan composite steel-concrete beams. Around 20 finite element models were also used for each span length to simulate the behaviour of the composite steel-concrete beams subjected to combined flexure and torsion with partial shear connection. Same set of span length of 6, 8, 10, 12 and 14 m was used for the study. Two different levels of shear connection (100 and 50 %) were also designed to investigate the effects of partial shear connection. Both the strength interaction curves and the unified strength interaction ratio curves were plotted for each span length using several different FEM subjected to different level of applied flexure and torque onto the straight composite steel-concrete beams.
8.4.1 6 m curved in plan composite steel-concrete beams

From Figure 8.18, the strength interaction curves for both FSC and PSC are actually the same except for the pure bending where the flexural strength of the FSC is 13 % more than PSC. It also shows the phenomenon that in the presence of flexure, the torsional moment capacity of the composite steel-concrete beams increases, while the flexural moment capacity remains the same in the presence of torsion for FSC. There is an increase in the flexural strength for PSC in the presence of torsion. From Figure 8.24, the increase in flexural strength has been observed from the strength interaction ratio curve for the PSC composite steel-concrete beams.

8.4.2 8 m curved in plan composite steel-concrete beams

Same observations were seen for 8 m curved in plan composite steel-concrete beams. From Figure 8.19 and 8.25, the torsional strength for both FSC and PSC has increased in the presence of flexure, but only PSC has an increase in flexural strength in the presence of torsion. Even though, the increase was about 9 % of its ultimate flexural strength.

8.4.3 10 m curved in plan composite steel-concrete beams

For 10 m curved in plan composite steel-concrete beams, the flexural strength of the PSC composite steel-concrete beams was higher that FSC composite steel-concrete beams as shown in Figure 8.20. Both the figures does not showed the phenomenon that in the presence of flexure, the torsional moment capacity of the composite steel-concrete beams increase, while the flexural moment capacity remains the same in the presence of torsion. The reason was due to the premature
lateral buckling failure of the steel web as illustrated in Figure 8.21. The interaction ratio curves from Figure 8.26 are affected as well for FSC beams.

### 8.4.4 12 m curved in plan composite steel-concrete beams

From Figures 8.20 and 8.27, the phenomenon that in the presence of flexure, the torsional moment capacity of the composite steel-concrete beams increases was cut short due to the lateral torsional buckling failure of the steel web. The lateral buckling failure has also affected the PSC beams as well. However, it is clearly now that the lateral torsional buckling occurred when a medium level of torque and large amount of flexure were applied to the composite steel-concrete beams. When the composite steel-concrete beams were subjected to high torque with low flexure or high flexure with low torque, the premature failure due to lateral buckling of the steel web was avoided.

### 8.4.5 14 m curved in plan composite steel-concrete beams

From Figures 8.21 and 8.28, same observation were obtained from the strength interaction curves and the strength interaction ratio curves for 14 m curved in plan composite steel-concrete beams. Same premature failure of the lateral buckling of the steel web occurred when half of the ultimate torque was applied with high flexure loading. Increase in flexural strength in the presence of low applied torque was presented for both FSC and PSC beams. PSC beams seem to be less affected by the lateral buckling failure compared to the FSC beams.
8.4.6 Discussion

8.4.6.1 Effects of the beam span length

From Figures 8.29 and 8.30, the ultimate flexural and torsional moment capacities of the curved in plan composite steel-concrete beams for FSC and PSC are plotted respectively. There was an increase in both torsional and flexural strengths with an increase in span length. Same reason as before, the size of the steel beam and concrete slab was increased for an increase in span length from 6 to 14 m.

From Figures 8.31 and 8.32, the strength interaction ratio curves have provided some evidences that there were an increase in the torsional strength in the presence of flexure. The rate of increase for the torsional strength increased with the increase of span length. However, due to lateral buckling of the steel web when the span length increased, the rate could be clearly identified from the curves. Web stiffeners should be designed for the curved in plan composite steel-concrete beams especially when the steel web depth was large. Lateral buckling of the steel web was resulted from the large steel web that was presented when the span length increased to 10 m and above.

8.4.6.2 Effects of the degree of shear connection

From Figures 8.29 and 8.30, it showed that the performance of PSC beams were better than the FSC beams. For the PSC beams, the flexural strength had increased in the presence of low torsion for all span lengths and the reduction of strength due to lateral torsional buckling of the steel web was lesser compared to FSC beams. Both FSC and PSC beams were affected by lateral torsional buckling of the steel web in the high span length range.
From the comparisons between FSC and PSC beams at every span length, the effects of PSC have been investigated as well in the parametric study. 50 and 100% levels of shear connection have been chosen for this purpose. The only difference is the slight increase of flexural capacity for FSC design beams. From the ultimate strength ratio interaction, the curves are the same for PSC and FSC design beams.

### 8.4.7 Proposed design models

Based on the FEM results, Figure 8.33 provides a proposed flexure-torsion interaction relationship model for 6, 8, 10, 12 m straight composite steel-concrete beams. The interaction relationship model will not greatly be affected by the level of shear connection as discussed in Section 8.4.6.2. The 14 m span length was omitted in the proposed model due to huge reduction of ultimate strength in the strength interaction curves. Until web stiffeners were designed and computed again with the FEM, the 14 m span length curved in plan composite steel-concrete beams have to been calculated separately. The flexure-torsion interaction relationship equations can be written as following.

For 6 m curved in plan composite steel-concrete beams,

\[
\frac{T}{T_U} = 0.3 \left( \frac{M}{M_U} \right) + 1 \quad \text{when} \ 0 \leq \frac{M}{M_U} < 1 \quad \text{and} \quad \frac{T}{T_U} \geq 1
\]  \hspace{1cm} (8.11)

\[
\frac{M}{M_U} = 1 \quad \text{when} \ \frac{M}{M_U} \geq 1 \quad \text{and} \quad 0 \leq \frac{T}{T_U} \leq 1.3
\]  \hspace{1cm} (8.12)
For 8 m curved in plan composite steel-concrete beams,

\[
\frac{T}{T_U} = \left(\frac{M}{M_U}\right) + 1 \quad \text{when } 0 \leq \frac{M}{M_U} < 1 \text{ and } \frac{T}{T_U} \geq 1 \quad (8.13)
\]

\[
\frac{M}{M_U} = 1 \quad \text{when } \frac{M}{M_U} \geq 1 \text{ and } 0 \leq \frac{T}{T_U} \leq 2 \quad (8.14)
\]

For 10 m curved in plan composite steel-concrete beams,

\[
\frac{T}{T_U} = 1.25\left(\frac{M}{M_U}\right) + 1 \quad \text{when } 0 \leq \frac{M}{M_U} < 1 \text{ and } \frac{T}{T_U} \geq 1 \quad (8.15)
\]

\[
\frac{M}{M_U} = 1 \quad \text{when } \frac{M}{M_U} \geq 1 \text{ and } 0 \leq \frac{T}{T_U} \leq 2.25 \quad (8.16)
\]

For 12 m curved in plan composite steel-concrete beams,

\[
\frac{T}{T_U} = 1.5\left(\frac{M}{M_U}\right) + 1 \quad \text{when } 0 \leq \frac{M}{M_U} < 1 \text{ and } \frac{T}{T_U} \geq 1 \quad (8.17)
\]

\[
\frac{M}{M_U} = 1 \quad \text{when } \frac{M}{M_U} \geq 1 \text{ and } 0 \leq \frac{T}{T_U} \leq 2.5 \quad (8.18)
\]

Equations 8.11 to 8.18 have provided a simplified and conservative model to represent the flexure-torsion interaction relationship for 6 to 12 m curved in plan composite steel-concrete beams. Interpolation of the curves can be used for other span lengths such as 7 or 9 m.
8.5 SUMMARY

A parametric study was carried out to investigate the behaviours of the straight composite steel-concrete beams with an increase in beam span length. The chosen spans were 6, 8, 10, 12 and 14 m. Composite steel-concrete beams have shown an increase in the torsional and flexural capacities with an increase in span length due to the increase in steel beam and concrete slab sizes. Additionally, the benefit for the torsional strength also increases with the span length as shown in the non-dimensional interaction curves.

For straight composite steel-concrete beams, the phenomenon that in the presence of flexure, the torsional moment capacity of the composite steel-concrete beams increases, while the flexural moment capacity remains the same in the presence of torsion was observed from the FEM. The rate of increase depended on the location of the neutral axis and the span length. The rate increased with the increase in span length and the movement of the neutral axis from steel beam to concrete slab. There was no significant impact from the reduction of the level of shear connection to the strength interaction relationship except for the ultimate flexural strength.

For curved in plan composite steel-concrete beams, there was an increase in the torsional strength in the presence of flexure and also an increase in flexural strength in the presence of torsion. However, as the span length increased with the increase in the beam size, lateral torsional buckling of the steel web had affected the ultimate strength when the composite steel-concrete beams were applied with more than 50 % of the ultimate torque and high flexure. This reduction was more serious for FSC beams than PSC beams.
Proposed design models are presented in this chapter to illustrate the flexure-torsion interaction relationship for straight and curved in plan composite steel-concrete beams. These proposed design models provide a conservative upper limit for the ultimate flexural and torsional strengths of the composite steel-concrete beams.
### Table 8.1 Beam selection for each span length (Part 1)

<table>
<thead>
<tr>
<th></th>
<th>Span length (mm)</th>
<th>6000</th>
<th>8000</th>
<th>10000</th>
<th>12000</th>
<th>14000</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Concrete slab</strong></td>
<td>Thickness (mm)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>120</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Width (mm)</td>
<td>1500</td>
<td>2000</td>
<td>2500</td>
<td>3000</td>
<td>3500</td>
</tr>
<tr>
<td></td>
<td>Compressive strength (N/mm(^2))</td>
<td></td>
<td></td>
<td>30</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Steel beam</strong></td>
<td>Type</td>
<td>360UB56.7</td>
<td>530UB92.4</td>
<td>700WB150</td>
<td>1000WB215</td>
<td>1000WB296</td>
</tr>
<tr>
<td></td>
<td>Gross area (mm(^2))</td>
<td>7240</td>
<td>11800</td>
<td>19100</td>
<td>27400</td>
<td>37800</td>
</tr>
<tr>
<td></td>
<td>Total depth (mm)</td>
<td>359</td>
<td>533</td>
<td>710</td>
<td>1000</td>
<td>1016</td>
</tr>
<tr>
<td><strong>Steel flange</strong></td>
<td>Thickness (mm)</td>
<td>13</td>
<td>15.6</td>
<td>25</td>
<td>20</td>
<td>28</td>
</tr>
<tr>
<td></td>
<td>Yield stress (N/mm(^2))</td>
<td></td>
<td></td>
<td>300</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Ultimate stress (N/mm(^2))</td>
<td></td>
<td></td>
<td>500</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Steel web</strong></td>
<td>Thickness (mm)</td>
<td>8</td>
<td>15.6</td>
<td>10</td>
<td>16</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>Yield stress (N/mm(^2))</td>
<td></td>
<td></td>
<td>300</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Ultimate stress (N/mm(^2))</td>
<td></td>
<td></td>
<td>500</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
### Table 8.2 Beam selection for each span length (Part 2)

<table>
<thead>
<tr>
<th>Span length (mm)</th>
<th>6000</th>
<th>8000</th>
<th>10000</th>
<th>12000</th>
<th>14000</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Longitudinal reinforcing steel</strong></td>
<td>N12</td>
<td>N12</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type (Top)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type (Bottom)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spacing (mm)</td>
<td>471</td>
<td>638</td>
<td>805</td>
<td>971</td>
<td>1138</td>
</tr>
<tr>
<td>Concrete cover (mm)</td>
<td>25</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Yield stress (N/mm²)</td>
<td>500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ultimate stress (N/mm²)</td>
<td>600</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Stirrup steel</strong></td>
<td>R10</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Concrete cover (mm)</td>
<td>25</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spacing (mm)</td>
<td>100</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Yield stress (N/mm²)</td>
<td>500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ultimate stress (N/mm²)</td>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Shear connection</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Type</td>
<td>Headed stud</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Diameter (mm) x length (mm)</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Number of rows</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spacing for 100 % shear connection</td>
<td>107 86 71 71 71</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Spacing for 50 % shear connection</td>
<td>214 172 142 142 142</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Yield stress (N/mm²)</td>
<td>500</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ultimate stress (N/mm²)</td>
<td>600</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
FIGURES

Figure 8.1 Plan view of a typical composite floor system
Figure 8.2 Strength interaction curves for 6 m straight composite beams
Figure 8.3 Strength interaction curves for 8 m straight composite beams
Figure 8.4 Strength interaction curves for 10 m straight composite beams
Figure 8.5 Strength interaction curves for 12 m straight composite beams
Figure 8.6 Strength interaction curves for 14 m straight composite beams
Figure 8.7 Strength interaction ratio curves for 6 m straight composite beams
Figure 8.8 Strength interaction ratio curves for 8 m straight composite beams
Figure 8.9 Strength interaction ratio curves for 10 m straight composite beams

**Figure 8.9 Strength interaction ratio curves for 10 m straight composite beams**
Figure 8.10 Strength interaction ratio curves for 12 m straight composite beams
Figure 8.11 Strength interaction ratio curves for 14 m straight composite beams
Figure 8.12 Strength interaction curves for FSC straight composite beams
Figure 8.13 Strength interaction curves for PSC straight composite beams
Figure 8.14 Strength interaction ratio curves for FSC straight composite beams
Figure 8.15 Strength interaction ratio curves for PSC straight composite beams
Figure 8.16 Design model for 6 and 8 m straight composite beams
Figure 8.17 Design model for 10, 12 and 14 m straight composite beams
Figure 8.18 Strength interaction curves for 6 m curved composite beams
Figure 8.19 Strength interaction curves for 8 m curved composite beams
Figure 8.20 Strength interaction curves for 10 m curved composite beams
Figure 8.21 Lateral buckling of the steel web for curved in plan composite beams
Figure 8.22 Strength interaction curves for 12 m curved composite beams
Figure 8.23 Strength interaction curves for 14 m curved composite beams
Figure 8.24 Strength interaction ratio curves for 6 m curved composite beams
Figure 8.25 Strength interaction ratio curves for 8 m curved composite beams
Figure 8.26 Strength interaction ratio curves for 10 m curved composite beams
Figure 8.27 Strength interaction ratio curves for 12 m curved composite beams
Figure 8.28 Strength interaction ratio curves for 14 m curved composite beams
Figure 8.29 Strength interaction curves for FSC curved composite beams
Figure 8.30 Strength interaction curves for PSC curved composite beams
Figure 8.31 Strength interaction ratio curves for FSC curved composite beams
Figure 8.32 Strength interaction ratio curves for PSC curved composite beams
Figure 8.33 Design model for 6, 8, 10 and 12 m curved in plan composite beams
CHAPTER 9

9 CONCLUSIONS AND FURTHER RESEARCH

9.1 SUMMARY

The primary objective of this thesis was to study the effects of PSC to the behaviour of composite steel-concrete beams subjected to combined flexure and torsion. The research objective was divided into eleven components summarised below:

(1) To carry out an experimental investigation on straight concrete slabs subjected to combined flexure and torsion.

(2) To carry out an experimental investigation on straight steel beams subjected to combined flexure and torsion.

(3) To carry out an experimental investigation on straight composite steel-concrete beams subjected to combined flexure and torsion with the influence of PSC.

(4) To carry out an experimental investigation on curved in plan composite steel-concrete beams subjected to combined flexure and torsion with the influence of PSC.

(5) To develop a 3-D finite element model to simulate the behaviour of straight concrete slabs subjected to combined flexure and torsion.
(6) To develop a 3-D finite element model to simulate the behaviour of straight steel beams subjected to combined flexure and torsion.

(7) To develop a 3-D finite element model to simulate the behaviour of straight composite steel-concrete beams subjected to combined flexure and torsion with the influence of PSC.

(8) To develop a 3-D finite element model to simulate the behaviour of curved in plan composite steel-concrete beams subjected to combined flexure and torsion with the influence of PSC.

(9) To extend the 3-D finite element model to complete a parametric study for straight composite steel-concrete beams subjected to combined flexure and torsion with the influence of PSC.

(10) To extend the 3-D finite element model to complete a parametric study for curved in plan composite steel-concrete beams subjected to combined flexure and torsion with the influence of PSC.

(11) To develop flexure-torsion interaction relationship curves for both straight and curved in plan composite steel-concrete beams subjected to combined flexure and torsion with the influence of PSC and span length.

These eleven components had been carried out throughout the thesis. In Chapters 3 and 4, two full-scales experimental series I and II were undertaken to provide the experimental testing needed to investigate both straight and curved in plan composite steel-concrete beams respectively. Chapter 5 provided the initial modified design models for mapping the flexure-torsion interaction relationship for both straight and curved in plan composite steel-concrete beams with a span length of 4 and 6 m respectively.
The development of a 3-D finite element model was described in Chapter 6 with the ability of incorporating the PSC designs into the simulation and nonlinear stress-strain profile of each component of a composite steel-concrete section had been provided to model the plastic behaviour of the beams. More importantly, the model was used to monitor the failure criterion due to torsion.

In Chapter 7, the comparison of the FEM and the experimental results had validated the 3-D FEM in terms of load-deflection response and ultimate strengths. Using the validated model, a parametric study was carried in Chapter 8 with parameters such as different span length and the different levels of shear connection. All these chapters had carried out all the eleven objectives of this thesis with detailed and comprehensive execution.

9.2 CONCLUDING REMARKS

Limited experimental work and the lack of research of the issue of partial shear connection (PSC). For composite steel-concrete beams subjected to combined flexure and torsion was evident from the detailed review of the research literature done in Chapter 2. Moreover, curved in plan beams are often replaced by straight beams in construction to avoid the check for the torsional moment capacity. Therefore, this thesis had looked into straight and curved in plan composite steel-concrete beams subjected to combined flexure and torsion. This thesis had also provided a nonlinear finite element model using ABAQUS model capable of modelling composite steel-concrete beams and incorporating the effects of PSC.

To carry out an experimental investigation on straight concrete slabs, steel beams and composite steel-concrete beams subjected to combined flexure and
torsion with the influence of PSC, experimental series I was conducted in Chapter 3. Several findings were obtained from both push-out and beams tests.

Firstly, the average ultimate shear capacities and the load-slip characteristics for both FSC and PSC push-out tests were similar even though different number of shear studs was used. Therefore, we can conclude that the number of shear studs did not affect the average ultimate load capacity of shear studs in the push-out tests based on Eurocode 4 (British Standards Institution, 1992).

Secondly, in the presence of torsion, there was no increase or decrease of the flexural strength of the straight composite steel-concrete beams. However, in the presence of flexure, there was an increase in torsional strength.

Thirdly, for straight composite steel-concrete beams using only 50 % of shear connection, the reduction in their flexural strength was less than 10 % compared to FSC straight composite steel-concrete beams. This reinforced the use of PSC in composite steel-concrete designs.

Fourthly, the greater was the applied torque, the lower ductility would be for both straight composite steel-concrete beams and steel beams subjected to combined flexure and torsion. As the composite steel-concrete beams and steel beams tend to fail in torsion, whilst concrete slabs tend to fail in bending.

Lastly, even with FSC in composite steel-concrete beam designs, there was partials composite action at some sections of the beam span as longitudinal interface slip had been found from the strain distributions.

To carry out an experimental investigation on curved in plan composite steel-concrete beams subjected to combined flexure and torsion with the influence of PSC,
this thesis had presented an extended series of curved in plan composite steel-concrete beams subjected to combined flexure and torsion in Chapter 4. Several another findings were obtained from both push-out and curved in plan composite steel-concrete beams tests.

Firstly, the initial stiffness of the curved in plan composite steel-concrete beams were different from one another due to the different in the profile of their horizontal curvature geometries. Curved in plan composite steel-concrete beams with higher span to radius of curvature ratios tend to have lower initial stiffness.

Secondly, curved in plan composite steel-concrete beams with higher span to radius of curvature ratios tend to have lowered load-carrying and flexural moment capacities. This was because with the increase of the span to radius of curvature ratio, the main failure criteria changed from bending to the combined action of bending and twisting. Therefore, the critical factor would be their torsional moment capacities.

Thirdly, different levels of shear connection did not affect the torsional moment capacities of curved in plan composite steel-concrete beams. Moreover, curved in plan composite steel-concrete beams with PSC (50% of FSC) seem to perform better than those with FSC in terms of load-carrying capacity.

Fourthly, in terms of ductility, curved in plan composite steel-concrete beams had lower or zero ductility when the span to radius of curvature ratio increased such as CCBF-3, CCBF-4, CCBP-3 and CCBP-4. This issue could be a major when curved in plan composite beams were considered in designs to resist torsional moments.
Fifthly, there was separation of the concrete slab from the steel beam observed from the tests. This separation caused the difference in the curvature of the strain distributions from both concrete slab and steel beam. The term “torsional interface slip” has been used to identify this difference in curvatures. In addition, due to this torsional interface slip, the assumption “plane sections remain plane” could not be used in any numerical analysis when composite steel-concrete beams were subjected to torsion. Moreover, this torsional slip would increase with the decrease in the level of shear connection.

Lastly, the required amount of longitudinal interface slip was not important for curved in plan composite steel-concrete beams subjected to combined flexure and torsion such as CCBF-3, CCBF-4, CCBP-3 and CCBP-4. Since those composite steel-concrete beams with higher span to radius of curvature ratios reached their maximum torsional moment capacities faster before their full flexural moment resistance was reached. It was only required when the span to radius of curvature ratio was very low and PSC was used.

After the completion of both experimental series I and II, the test results were used to plot the flexural-torsion interaction relationship curves for straight and curved in plan composite steel-concrete beams in Chapter 5. From the curves, the phenomenon of having an increase in the torsional moment capacity of the composite steel-concrete beams in the presence of flexure had been reinforced. However, this phenomenon was not seen for the flexural moment capacity when torsion was presence in the equation. Simplified models to represent the flexure-torsion interaction relationship for both straight and curved in plan composite steel-concrete
beams subjected to combined torsion and flexure have been plotted based on the test results.

In additionally, the summation of all the torsional contributions from the concrete, steel beams and the reinforcement could provide and predicted ultimate torsional capacity for composite steel-concrete beams only when the beams were under pure torsion. This assumption should not be used when other forces such as shear, flexure or axial were involved. Due to the complex stress distribution when the composite steel-concrete beams, the torsional capacity was influenced by other parameters such as the level of shear connection, the beam sizes and the beam span lengths.

In Chapters 6 and 7, the development and the validation of a 3-D finite element model (FEM) were carried out respectively to simulate the behaviour of straight concrete slabs, straight steel beams, straight and curved in plan composite steel-concrete beams subjected to combined flexure and torsion with the influence of PSC. The commercial software package ABAQUS was used for this FEM.

The development of the 3-D FEM was classified into four different steps which included the geometry and material modelling, boundary and constraint conditions, output analysis and post-processing of results. The importance of providing accurate material properties and choosing the right element and mesh types for all different components of the composite steel-concrete beams was emphasised. In addition, the boundary and constraints conditions of the FEM provide the same external environments as the experimental series I and II. The contact interactions between each component had been explained in detail to allow for the modelling of the interaction between each component during analysis. Ultimately, this FEM had
the ability to consider PSC and investigate the nonlinear behaviour of the composite steel-concrete beams subjected to combined flexure and torsion.

As for the validation process, comparisons between the FEM and the experimental results were carried out. From the FEM, full flexural and torsional response of the composite steel-concrete beams was adequately simulated until failure. Failure modes of the concrete slabs and shear connector fracture occurred in the experimental tests were accurately traced and observed. For straight and curved in plan composite steel-concrete beams subjected to high torque where the primary failure mode was governed by the torsional failure of the concrete slab, this was also well predicted. Based on the comparisons between the ultimate strengths obtained from the FEM and the experimental series I and II, they were in good agreement.

The FEM have provided further proof that due to the twisting of the composite steel-concrete beams, different curvatures for the steel beam and concrete slab can occur when torsion is presence. The cross-sectional views of the FEM had also provided evidence that the shear connectors were subjected to uplift forces when the twisting angle of the steel beam was different from the twisting angle of the concrete slab. Lastly, lateral buckling of the steel web was presented for straight composite steel-concrete beams and curved in plan composite steel-concrete beams with low span to radius curvature in the FEM.

Finally, a parametric study was conducted to extend the 3-D FEM for straight and curved in plan composite steel-concrete beams subjected to combined flexure and torsion with the influence of PSC. This parametric also provided the tool to develop flexure-torsion interaction relationship curves for both straight and curved in plan composite steel-concrete beams.
Therefore, two parameters were used for the parametric study. Different span lengths of 6, 8, 10, 12 and 14 m were chosen to investigate the realistic behaviour of the composite steel-concrete beams out in the industry. 100 and 50 % of the shear connection were also chosen for the other parameter to investigate the influence of PSC.

From the parametric study, composite steel-concrete beams have shown an increase in the torsional and flexural capacities with an increase in span length due to the increase in steel beam and concrete slab sizes. Additionally, the benefit for the torsional strength also increases with the span length as shown in the non-dimensional interaction curves.

For straight composite steel-concrete beams, in the presence of flexure, the torsional moment capacity of the composite steel-concrete beams increased, while the flexural moment capacity remained the same in the presence of torsion. The rate of increase depended on the location of the neutral axis and the span length. The rate increased with the increase in span length and the movement of the neutral axis from the steel beam to the concrete slab. Moreover, there was no significant impact from the reduction of the level of shear connection to the strength interaction relationship except for the ultimate flexural strength.

For curved in plan composite steel-concrete beams, there was an increase in the torsional strength in the presence of flexure and also an increase in flexural strength in the presence of torsion. However, as the span length increased with the increase in the beam size, lateral torsional buckling of the steel web affected the ultimate strength when the composite steel-concrete beams were applied with more
than 50 % of the ultimate torque and high flexure. This reduction was more serious for FSC beams than PSC beams.

Proposed design models had been presented to illustrate the flexure-torsion interaction relationship for straight and curved in plan composite steel-concrete beams. These proposed design models provided a conservative upper limit for the ultimate flexural and torsional strengths of the composite steel-concrete beams.

In retrospect, this thesis has achieved its goals and objectives. The main issues of interests and scopes of investigation had been considered. The experimental results from this research provided useful information in the area of combined loadings for composite steel-concrete beams and the finite element model provided a new method to simulate the behaviour of composite steel-concrete beams.

9.3 RECOMMENDATIONS FOR FUTURE RESEARCH

The following points highlight several pertinent issues which may be useful for further investigation. They offer the possibilities to extent and improve on the present word carried out in this thesis.

(1) Additional set of full-scale curved in plan composite beams may be tested to further validate the findings herein. From these, the flexure-torsion interaction relationship can be evaluated and validated for different span lengths. The sudden drop in the flexural moment capacity of for span length of 10 m and greater should be investigated in depth with full-scale tests. Web stiffeners should be included to prevent the lateral-torsional buckling of the steel web in the tests.
(2) The 3-D finite element model may also be extended to include other parameters such as the material properties of the concrete, the reinforcing bar and the different types of shear connectors. Profiled steel sheeting can also be added into the model since it is commonly used in the building construction.

(3) Different levels of shear connection should also be investigated to have a completed picture of the influence of PSC. There may be a relationship between the levels of shear connection with the torsional slip in the case for curved in plan composite steel-concrete beams.

(4) It is recognised that standard push-out tests are conservative to represent the slip deformation behaviour of the shear connectors in composite steel-concrete beams. In this thesis, the push-out test results were also found to be suitable and conservative to predict the shear connector behaviour in composite steel-concrete beams as proved. However, the study on the localised behaviour of the connector in the concrete medium which is cracked is limited. It may be of interest to evaluate the effects of shear and axial forces being imposed onto the shear connectors as a result of having applied torsion to the concrete slab. This issue should be studied with both intensive experimental and analytical works.

(5) As mentioned before, profile steel sheeting is commonly used in most composite construction. The composite steel-concrete beam tests herein have not considered the inclusion of any form of steel sheeting. In fact, with the removal of concrete section could result in a lower torsional
moment capacity and higher torsional slip due to the interface contact between the steel flange and the steel sheeting.

(6) Composite steel-concrete beams under hogging moments should also be investigated. The flexure-torsion interaction relationship for composite steel-concrete beams under hogging moments is as important as those under sagging moments carried out in this thesis. The fact that the concrete slab is under tension could worsen the ability of the composite steel-concrete beams from torsion resistance. The critical failure of the composite steel-concrete beams under hogging moments could be the reduction of torsional moment capacity in the presence of flexure.


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