Evaluation of the Performance of GFRP Dowels in Jointed Plain Concrete Pavement (JPCP) for Road/Airport under the Combined Effect of Dowel Misalignment and Cyclic Wheel Load

A thesis submitted to The University of Manchester for the degree of Doctor of Philosophy in the Faculty of Engineering and Physical Sciences

2013

BASIM HASSAN SHNAWA AL-HUMEIDAWI

SCHOOL OF MECHANICAL, AEROSPACE AND CIVIL ENGINEERING
TABLE OF CONTENTS

TABLE OF CONTENTS .......................................................................................................................... 2
LIST OF FIGURES ................................................................................................................................. 8
LIST OF TABLES .................................................................................................................................... 15
LIST OF NOTATIONS ............................................................................................................................ 17
ABSTRACT ............................................................................................................................................... 21
DECLARATION ......................................................................................................................................... 22
COPYRIGHT STATEMENT ....................................................................................................................... 23
PUBLICATIONS ....................................................................................................................................... 24
DEDICATION ............................................................................................................................................. 25
ACKNOWLEDGEMENTS ......................................................................................................................... 26
CHAPTER ONE INTRODUCTION ........................................................................................................... 27
  1.1. Introduction ....................................................................................................................................... 27
  1.2. Problem Statement ...................................................................................................................... 28
  1.3. Objectives of the Current Study ............................................................................................... 29
  1.4. Outline of the Thesis .................................................................................................................. 30
CHAPTER TWO LITERATURE REVIEW ............................................................................................... 32
  2.1. Introduction ....................................................................................................................................... 32
  2.2. Dowel Bar Theory ........................................................................................................................ 35
  2.2.1. Analytical investigation ......................................................................................................... 35
  2.2.2. Bearing stress at the dowel-concrete interface ...................................................................... 38
  2.3. Load Transfer Mechanism and Load Transfer Efficiency (LTE) ........................................... 39
  2.3.1. Factors affecting load transfer efficiency (LTE) .............................................................. 41
    2.3.1.1. Dowel looseness (DL) .................................................................................................. 41
    2.3.1.1.1. Experimental studies .............................................................................................. 41
    2.3.1.1.2. Numerical studies .................................................................................................. 42
CHAPTER THREE EXPERIMENTAL INVESTIGATION OF THE LOAD-DEFLECTION RESPONSE OF GFRP DOWELS ........................................... 61

3.1. Introduction .............................................................................................................. 61
3.2. Test Methodology .................................................................................................... 61
3.3. Material properties ................................................................................................... 64
3.3.1. Dowel bars ............................................................................................................... 64
3.3.1.1. Epoxy-coated steel dowel bars ................................................................................. 65
3.3.1.2. GFRP dowel bars ..................................................................................................... 65
3.3.2. Concrete ................................................................................................................... 68
3.4. Test Parameters ........................................................................................................ 68
3.5. Discussion of results ................................................................................................ 69
3.5.1. Deflection response of GFRP dowel bars under load .............................................. 69
3.5.2. Comparison of GFRP dowels with epoxy-coated steel dowel bars ......................... 70
3.5.3. Effect of concrete compressive strength on the deflection response of GFRP dowels ..................................................................................................................... 76
3.5.4. Effect of joint width on the deflection response of GFRP dowels ........................... 78
3.6. Summary and Conclusions ....................................................................................... 81

CHAPTER FOUR EXPERIMENTAL INVESTIGATION ON THE COMBINED EFFECTS OF DOWEL MISALIGNMENT AND WHEEL LOAD ........ 82

4.1. Introduction .............................................................................................................. 82
4.2. Experimental Plan ................................................................................................... 82
4.2.1. Dowel type and size ................................................................................................. 83
4.2.2. Dowel misalignment types ...................................................................................... 83
4.2.3. Dowel misalignment magnitude ............................................................................ 83
4.2.4. Number of dowel bars across the joint................................................................. 84
4.2.5. The Orientation of misaligned dowel bars ................................................. 84
4.3. Experimental setup ...................................................................................... 84
4.4. Test Sequence ............................................................................................... 93
4.5. Results and Discussions ................................................................................ 95
4.5.1. Dowel Misalignment for GFRP and steel dowel bars ................................ 95
4.5.1.1. Two misaligned dowels ............................................................................ 96
4.5.2. Three misaligned dowel bars ..................................................................... 101
4.5.3. Effect of dowel misalignment on relative deflection, dowel looseness (DL) and LTE ................................................................. 102
4.5.4. Results of the cyclic load test ..................................................................... 104
4.6. Pull-out Test ................................................................................................... 106
4.6.1. Objectives of the test .................................................................................. 106
4.6.2. Material and specimens fabrication ............................................................... 106
4.6.3. Test procedure ............................................................................................. 108
4.6.4. Results and discussion ................................................................................ 109
4.7. Summary and Conclusions .......................................................................... 111

CHAPTER FIVE DEVELOPMENT OF A 3D FINITE ELEMENT MODEL .......... 113
5.1. Introduction .................................................................................................... 113
5.2. Element Selection .......................................................................................... 113
5.2.1. Solid element ............................................................................................. 113
5.2.2. Spring element ........................................................................................... 115
5.3. Material Modelling ....................................................................................... 115
5.3.1. Steel dowel bars ....................................................................................... 115
5.3.2. GFRP dowel bars ...................................................................................... 116
5.3.3. Concrete Models ....................................................................................... 117
5.3.3.1. Input parameters for CDP ....................................................................... 118
5.3.3.1.1. Compressive behaviour ................................................................. 119
5.3.3.1.2. Tensile behaviour .......................................................................... 119
5.3.3.1.3. Plasticity parameters ...................................................................... 122
5.4. Dowel-Concrete Interaction ................................................................. 123
5.4.1. Modelling of load-deflection response of dowel bars across joints of JPCP ....... 123
5.4.2. Modelling of combined effect of dowel misalignment and wheel load .......... 124
5.5. Mesh Size .......................................................................................... 125
5.6. Loading Steps and Boundary Conditions ............................................... 128
5.6.1. Modelling of load-deflection response of dowel .................................... 128
5.6.2. Modelling of combined effect of dowel misalignment and wheel load ....... 128
5.7. Estimation of Coefficient of Friction ....................................................... 129
5.7.1. Steel dowel bars .................................................................................. 129
5.7.2. GFRP dowels ................................................................................. 130
5.8. Design of Steel Supporting Base ............................................................ 131

CHAPTER SIX NUMERICAL INVESTIGATION INTO THE LOAD-DEFLECTION RESPONSE OF GFRP DOWELS AND DEVELOPMENT OF DESIGN CONSIDERATIONS ............................. 134
6.1. Introduction ......................................................................................... 134
6.2. Current Design of Dowel Bars ............................................................. 134
6.3. Numerical Results ............................................................................... 136
6.3.1. Validation of FEM against the experimental results ......................... 136
6.3.2. Bearing stress at the dowel-concrete interface .................................. 138
6.3.3. Finite element modelling of a case study from literature ................. 141
6.4. Parametric Study of the Proposed GFRP Dowels’ Design Considerations .... 144
6.4.1. Evaluation criteria for GFRP dowels ............................................... 146
6.4.2. Proposed design considerations for the GFRP dowels ..................... 147
6.5. Summary and Conclusions ................................................................. 151

CHAPTER SEVEN NUMERICAL INVESTIGATION OF COMBINED EFFECT OF DOWEL MISALIGNMENT AND WHEEL LOAD ................................................. 152
7.1. Introduction ......................................................................................... 152
7.2. Importance of Numerical Simulation .................................................. 152
7.3. Parameters Included in Numerical Simulation .................................... 153
7.4. Results and discussion ....................................................................... 157
7.4.1. Verifying numerical results with current experimental results .................................. 157
7.4.2. Comparison of different misalignment cases of epoxy-coated steel dowels ........... 160
  7.4.2.1. Horizontal misalignment ...................................................................................... 160
  7.4.2.2. Vertical misalignment ............................................................................................ 164
  7.4.2.3. Combined misalignment ...................................................................................... 166
7.4.3. Comparison of GFRP dowels with epoxy-coated steel dowel bars ....................... 169
  7.4.3.1. Pull-out load and joint opening .............................................................................. 169
    7.4.3.1.1. Horizontal misalignment ................................................................................ 169
    7.4.3.1.2. Vertical misalignment .................................................................................... 172
    7.4.3.1.3. Combined misalignment ................................................................................ 174
  7.4.3.2. Damage initiation: investigation and comparison between steel and GFRP dowels ................................................................................................................................. 177
  7.4.3.3. Comparison of dowel looseness (DL) and LTE .................................................. 186
    7.4.3.3.1. Dowel looseness (DL) .................................................................................. 186
    7.4.3.3.2. Load transfer efficiency (LTE) ...................................................................... 189
7.4.4. Comparison of specimens containing two, and specimens containing three, GFRP dowels ............................................................................................................................. 192
  7.4.4.1. The pull-out load and joint opening ..................................................................... 192
  7.4.4.2. Damage initiation ................................................................................................ 195
  7.4.4.3. Comparison of dowel looseness and LTE ......................................................... 197
    7.4.4.3.1. Dowel looseness .......................................................................................... 197
    7.4.4.3.2. Load transfer efficiency (LTE) ...................................................................... 200
7.5. Summary and Conclusions ..................................................................................... 201

CHAPTER EIGHT EFFECT OF DOWELLED-JPCP DESIGN PARAMETERS ON PULL-OUT LOAD VERSUS JOINT OPENING BEHAVIOUR ........................................................................ 203

8.1. Introduction ............................................................................................................. 203
8.2. Effect of Concrete Compressive Strength ............................................................ 203
8.3. Effect of Dowel Bar Diameter ............................................................................. 205
8.4. Effect of Embedded Length of Dowel Bar ......................................................... 207
8.5. Effect of Concrete Pavement Thickness ............................................................. 208
8.6. Effect of Slab-Base Friction ................................................................. 209
8.7. Summary and Conclusions ................................................................. 212

CHAPTER NINE SUMMARY AND CONCLUSIONS ............................................. 213
9.1. Introduction ......................................................................................... 213
9.2. Experimental investigation into the Load-Deflection Response of GFRP Dowels ................................................................. 213
9.3. Numerical simulation of the Load-Deflection Response of GFRP Dowels ....... 214
9.4. Experimental Investigation into the Combined Effect of Dowel Misalignment and Wheel Load ................................................................. 214
9.5. Numerical Investigation into the Combined Effect of Dowel Misalignment and Wheel Load ................................................................. 215
9.6. Recommendations for future studies .................................................. 216

REFERENCES ................................................................................................. 217

APPENDIX A .................................................................................................. 224
LIST OF FIGURES

Figure 2.1. Typical section of a JPCP ...................................................................................... 33
Figure 2.2. Construction of a JPCP with joints and dowel bar basket .................................. 33
Figure 2.3. Some of the common distresses in concrete pavement (Jung et al. 2008; Miller and Bellinger 2003) .......................................................... 34
Figure 2.4. Relative deflections between adjacent pavement slabs ....................................... 38
Figure 2.5. Cross-sectional details of a joint showing DL (Davids 2000) ............................... 44
Figure 2.6. Joint spalling due to dowel misalignment (Khazanovich et al. 2009) .................. 46
Figure 2.7. Sectional view of JPCP with translation misalignments (a) Longitudinal; (b) Vertical ................................................................................................................. 47
Figure 2.8 Different types of skew misalignment: (a) Sectional view, non-uniform vertical misalignment; (b) Sectional view, uniform vertical misalignment; (c) Plan view, uniform horizontal misalignment; (d) Plan view, non-uniform horizontal misalignment; (e) Plan view, partial horizontal misalignment (h: slab thickness, s: dowel bars spacing, m: misalignment magnitude) .................................. 47
Figure 3.1. Test setup: (a) Complete test setup; (b) Schematic for the test setup ............... 62
Figure 3.2. Specimen during casting ........................................................................................ 64
Figure 3.3. Tensile strength test for steel dowel bars: (a) Test setup; (b) Stress-strain curve ................................................................................................................. 65
Figure 3.4. Double shear test set-up for GFRP dowels: (a) Schematic; (b) Photograph ....... 67
Figure 3.5. Results of double shear test for GFRP dowels .................................................. 67
Figure 3.6. Experimental and analytical results for the dowel deflection at 16 kN load for both steel and GFRP dowels: (a) GNH & SNH; (b) GWH & SWH; (c) GWL & SWL; (d) GNL & SNL .................................................................................. 72
Figure 3.7. Experimental results for the dowels deflections at 40 kN load for both steel and GFRP dowels ........................................................................................................ 73
Figure 3.8. Experimental results of the RD of specimens with steel and GFRP dowel bars ............................................................................................................................ 75
Figure 3.9. Experimental load-deflection behaviour of GFRP dowels for two different concrete grades at the following distances from the joint face: (a) Zero; (b)
20 mm; (c) 35 mm; (d) 50 mm; (e) 80 mm; (f) relative deflection of loaded side .......................................................... 77

Figure 3.10. Experimental dowel deflection at: (a) 16 kN load (b) 40 kN load .............. 78

Figure 3.11. Experimental load deflection behaviour of GFRP dowels for different joint opening (for a-f see caption of Figure 3.9) .......................................................... 80

Figure 3.12. Experimental dowel deflection at: (a) 16 kN load (b) 40 kN load .............. 81

Figure 4.1. Specimen of two dowel bars (GA2) ................................................................. 86

Figure 4.2. Specimen of three dowel bars (GV3P2) ........................................................... 87

Figure 4.3. Dowel misalignment calculations (GH2N4) .................................................... 87

Figure 4.4. Whole test setup (with steel-beam supporting base) ...................................... 90

Figure 4.5. Sketch of a specimen with all instrumentations ............................................. 91

Figure 4.6. Actual step with all instrumentations ............................................................. 91

Figure 4.7. Concrete slabs casting ................................................................................... 92

Figure 4.8. Concrete compressive strength test (cube test) .............................................. 92

Figure 4.9. Cyclic load on two slabs ................................................................................ 95

Figure 4.10. Effect of vertical misalignment for GFRP dowels- pull-out load versus joint opening for the specimens GA2 and GV2N2 ................................................. 97

Figure 4.11. Comparison of aligned GFRP and steel dowels – pull-out load versus joint opening for the specimens GA2 and SA2 .......................................................... 97

Figure 4.12. Effect of vertical misalignment for steel dowels – pull-out load versus joint opening for the specimens SA2-steel and SV2N2 ............................................ 97

Figure 4.13. Comparison between horizontal and vertical misalignment – pull-out load versus joint opening for the specimens of SV2N2, GV2N2 and GH2N4 .......... 98

Figure 4.14. Comparison of partially vertical and horizontally non-uniform misalignment with aligned dowels-pull-out load vs. joint opening for the specimens having three GFRP dowels ...................................................... 102

Figure 4.15. Comparison of relative deflection before and after slabs movement .......... 103

Figure 4.16. Comparison of LTE before and after slab movement .................................. 104

Figure 4.17. Relative deflection versus number of load cycles ........................................ 104

Figure 4.18. LTE versus number of load cycles ............................................................... 105

Figure 4.19. Comparison of dowels looseness due to slabs movement and cyclic load .... 106

Figure 4.20. Pull-out test: (a) Mould fabrication; (b) Preparing for casting ................. 107
Figure 4.21. Pull-out test specimens dimensions and test setup ........................................... 109
Figure 4.22. Pull-out load versus slip distance at ages of 3 and 28 days ......................... 110
Figure 4.23. Cyclic pull-out load versus slip for a vertically misaligned dowel bar ......... 111
Figure 5.1. Solid element formulation .............................................................................. 114
Figure 5.2. Spring element: (a) Connecting node to ground; (b) Connecting two nodes .... 115

Figure 5.3. Response of concrete to uniaxial loading (SIMULIA 2010). where, $E_0$: the initial (undamaged) elastic stiffness of the material; $\sigma_{t0}$: tension failure stress; $\varepsilon_t^{pl}$ & $\varepsilon_t^{el}$: Plastic and elastic tensile strains respectively; $\sigma_{co}$ & $\sigma_{cu}$: yield and ultimate stress in compression; $\varepsilon_c^{pl}$ & $\varepsilon_c^{el}$: Plastic and elastic compressive strains respectively; $d_t$: Damage variable in tension; $d_c$: Damage variable in compression. ............................................................... 118

Figure 5.4. Stress-strain relation for concrete under uniaxial compression (BS EN 1992-1-1 2004) ............................................................................................................. 119
Figure 5.5. Post-failure tensile response: (a) Stress-crack displacement, (b) Stress-fracture energy (SIMULIA 2010) ............................................................................. 120
Figure 5.6. Stress-crack opening diagram (CEB-FIP 1990 1993) .................................. 121
Figure 5.7. Inelastic behaviour of concrete ($f_{ck,cube} = 30$ MPa): (a) In compression (b) In tension ............................................................................................................ 121

Figure 5.8. The selection of dilation angle value (GH2N4) ............................................... 123
Figure 5.9. Stresses at misaligned dowel bars during opening of joint ......................... 125
Figure 5.10. Mesh size of load-deflection response model ............................................. 125
Figure 5.11. Mesh size of combined effect of dowel misalignment and wheel load model .................................................................................................................. 126
Figure 5.12. Sensitive analysis for mesh size of the model (SV2N2) .................................. 127
Figure 5.13. Prediction of coefficient of friction between epoxy coated steel dowels and concrete pavement (SV2N2) ........................................................................ 130
Figure 5.14. Parametric study on coefficient of friction between GFRP dowels and concrete pavement (GH2N4) ................................................................. 131

Figure 5.15. Comparison of dowel bar shear force for specimens SA2 on the steel-beam base and on the elastic foundation ..................................................... 132
Figure 5.16. Effect of different bases’ types on pull-out load for vertical misalignment (SV2N2) .............................................................................................................. 133

Figure 5.17. Effect of different bases’ types on pull-out load for horizontal misalignment (GH2N4) .............................................................................................. 133

Figure 6.1. Simulation of the load-deflection response of a GFRP dowel for the specimen GWH at the following distances from the joint face: (a) Zero; (b) 20 mm; (c) 35 mm; (d) 50 mm; (e) 80 mm; (f) RD of loaded to unloaded side of blocks .............................................................................................................. 137

Figure 6.2. Load versus principal compressive stress at the face of the joint, from the numerical simulation. ................................................................. 139

Figure 6.3. Bearing stress (MPa) (S22 – normal stress in the vertical direction) at the dowel-concrete interface for the steel and the GFRP dowels .............................................................................................................. 140

Figure 6.4. Effective area in Keeton and Bishop’s test (1957) .............................................................................................................. 142

Figure 6.5. Shear force in the central dowel .............................................................................................................. 143

Figure 6.6. Bending moment in the central dowel .............................................................................................................. 144

Figure 6.7. Schematic diagram of the concrete pavement with the dual wheel load position .............................................................................................................. 145

Figure 6.8. Effect of load repetitions (number cycles) on relative deflection for various dowel systems (Porter et al. 2001) .............................................................................................................. 146

Figure 6.9. Bearing stress (MPa) at the face of joints underneath the critical dowels. The abbreviations are used to define the different cases as follows. The first letter refers to dowel type: GFRP or steel; the second (number) refers to the joint width (in mm); the third number refers to the slab thickness (in mm); the fourth number refers to the spacing of the dowel bars (in mm); and the last number refers to the dowel bar diameter (in mm) .............................................................................................................. 150

Figure 7.1. Comparison of pull-out loads from the experimental test with the FEM: (a) SA2; (b) SV2N2; (c) GA2; (d) GV2N2; (e) GH2N4, (f) GA3; (g) GH3N2, (h) GV3P2 .............................................................................................................. 158

Figure 7.2. Simulation of concrete failure (specimen GH2N4) .............................................................................................................. 159

Figure 7.3. Comparison of different types and magnitudes of horizontal misalignment for specimens containing steel dowel bars: (a) Non-uniform; (b) Uniform; (c) Partial .............................................................................................................. 163

Figure 7.4. Vertical deflection of the slabs during the opening of the joint for SH2N3 ........ 163

Figure 7.5. Distribution of contact pressure (in MPa) at the dowel-concrete interface for specimen SH2N4: (a) Horizontal section of the slab-dowel assembly at joint

11
opening 4 mm; and on the developed outer surface of Dowel-2 at joint opening (b) 4 mm, (c) 7 mm

Figure 7.6. Comparison of different types and magnitudes of vertical misalignment for the specimens containing steel dowel bars: (a) Non-uniform; (b) Uniform; (c) Partial

Figure 7.7. Uplift of the slab and the RD across the joint during the opening of the joint for the specimens containing vertically misaligned dowels: (a) SV2N4, (b) SV2U4

Figure 7.8. Distribution of contact pressure (in MPa) at the dowel-concrete interface:........ 166

Figure 7.9. Comparison of different types and magnitudes of combined misalignment for specimens containing steel dowel bars: (a) Non-uniform; (b) Uniform; (c) Partial

Figure 7.10. Uplift of the slab and the RD across the joint during the opening of the joint for the specimens containing combined misaligned dowels: (a) SC2N3, (b) SC2U3

Figure 7.11. Comparison of different types and magnitudes of horizontal misalignment of steel and GFRP dowel bars: (a) Non-uniform; (b) Uniform; (c) Partial

Figure 7.12. Distribution of contact pressure (in MPa) at the dowel-concrete interface at the joint opening of 6 mm: (a) SH2N1, (b) GH2N4

Figure 7.13. Comparison of different types and magnitudes of the vertical misalignment of steel and GFRP dowel bars: (a) Non-uniform; (b) Uniform; (c) Partial

Figure 7.14. Distribution of contact pressure (in MPa) at the dowel-concrete interface of specimen GV2U4 at the joint opening: (a) 1.5 mm, (b) 12 mm

Figure 7.15. Uplift of the slab and the RD across the joint during the opening of the joint for the specimens containing vertically misaligned dowels: (a) GV2U1, (b) GV2U4

Figure 7.16. Comparison of different types and magnitudes of the combined misalignment of steel and GFRP dowel bars: (a) Non-uniform; (b) Uniform; (c) Partial

Figure 7.17. Comparison of the pull-out stress (MPa) of steel and GFRP dowels at the joint openings of 3 mm and 6 mm

Figure 7.18. Test specimen with highlighted control concrete volume

Figure 7.19. Tensile damage in the concrete surrounding dowel bars for specimens: (a) SH2N2; (b) GH2N2
Figure 7.20. The damage ratios of different misalignment cases for specimens that have steel dowels and specimens that have GFRP dowels at the joint openings of 3 mm, 6 mm and 12 mm. Note the log-scale in the above plot of the damage ratios ................................................................. 183

Figure 7.21. Average equivalent plastic strain within a control volume for different joint openings. Note the log-scale was used in the above plot .................................................. 185

Figure 7.22. DL due to slabs’ movement (horizontal misalignment): (a) Non-uniform, (b) Uniform, (c) Partial ................................................................. 187

Figure 7.23. DL due to slabs’ movement (vertical misalignment): (a) Non-uniform, (b) Uniform, (c) Partial .................................................. 188

Figure 7.24. DL due to slabs’ movement (combined misalignment): (a) Non-uniform, (b) Uniform, (c) Partial .................................................. 188

Figure 7.25. Reduction in LTE due to slabs’ movement (horizontal misalignment): (a) Non-uniform, (b) Uniform, (c) Partial (“B” before movement and “A” after movement) .................................................................................. 190

Figure 7.26. Reduction in LTE due to slabs’ movement (vertical misalignment): (a) Non-uniform, (b) Uniform, (c) Partial (“B” before movement and “A” after movement) .................................................................................. 191

Figure 7.27. Reduction in LTE due to slabs’ movement (combined misalignment): (a) Non-uniform, (b) Uniform, (c) Partial (“B” before movement and “A” after movement) .................................................................................. 191

Figure 7.28. Comparing different types and magnitudes of the horizontal misalignment for specimens containing two, and specimens containing three, GFRP dowels: (a) Non uniform; (b) Uniform; (c) Partial .................................................. 193

Figure 7.29. Comparing different types and magnitudes of the vertical misalignment for specimens containing two and specimens containing three GFRP dowels: (a) Non uniform; (b) Uniform; (c) Partial .................................................. 194

Figure 7.30. Comparing different types and magnitudes of the combined misalignment for specimens containing two and specimens containing three GFRP dowels: (a) Non uniform; (b) Uniform; (c) Partial .................................................. 195

Figure 7.31. The damage ratios of different misalignment cases for specimens containing two GFRP dowels and specimens containing three GFRP dowels at the joint openings of 3 mm, 6 mm and 12 mm. Note the log-scale in the values of the damage ratios................................................................................. 196
Figure 7.32. Average equivalent plastic strain within a control volume for different joint openings for specimens containing two, and specimens containing three, GFRP dowels. Note the log-scale in the values of the damage ratios .............. 197

Figure 7.33. DL due to slabs’ movement (horizontal misalignment) for specimens containing two and specimens containing three GFRP dowels: (a) Non-uniform, (b) Uniform, (c) Partial ................................................................. 198

Figure 7.34. DL due to slabs’ movement (vertical misalignment) for specimens containing two and specimens containing three GFRP dowels: (a) Non-uniform, (b) Uniform, (c) Partial ................................................................. 199

Figure 7.35. DL due to slabs’ movement (combined misalignment) for specimens containing two and specimens containing three GFRP dowels: (a) Non-uniform, (b) Uniform, (c) Partial ................................................................. 199

Figure 8.1. Pull-out load versus joint opening for various concrete compressive strength: (a) Steel dowels; (b) GFRP dowels. ................................................................. 205

Figure 8.2. Comparison of damage ratios of concrete surrounding the dowel bars for different concrete grades and different dowel types ............................................. 205

Figure 8.3. Comparison of pull-out load versus joint opening for different dowel bars sizes: (a) Steel dowels; (b) GFRP dowels ................................................................. 206

Figure 8.4. Comparison of damage ratios in the surrounding concrete pavement for different sizes of steel and GFRP dowel bars ................................................................. 207

Figure 8.5. Comparison of pull-out load for different lengths of dowel bars: (a) Steel dowels; (b) GFRP dowels ................................................................. 208

Figure 8.6. Comparison of damage ratio in the surrounding concrete pavement for different length of steel and GFRP dowels ................................................................. 208

Figure 8.7. Comparison of slab thickness effect on pull-out load for misaligned dowel bars: (a) Steel dowels; (b) GFRP dowels ................................................................. 209

Figure 8.8. Comparison of slab thickness effect on damage ratios in surrounding concrete pavement of misaligned steel and GFRP dowels ................................................................. 209

Figure 8.9. Validation of different bases types (FEM) with experimental results: (a) SV2N2; (b) GH2N4 ................................................................. 210

Figure 8.10. Comparison of effect of slab-base friction on pull-out load of misaligned steel dowel bars ................................................................. 211

Figure 8.11. Comparison of the effect of slab-base friction on concrete deterioration surrounding misaligned steel dowel bars ................................................................. 211
LIST OF TABLES

Table 2.1 Misalignment tolerances in mm per half length of the dowel bar (225 mm) according to various agencies (Rao et al. 2009) ................................................................. 48

Table 3.1. Mechanical properties of GFRP dowels supplied by the manufacturer .............. 66

Table 3.2. Experimental parameters .................................................................................. 68

Table 3.3. Deflection and elastic properties of the dowel bars ............................................ 70

Table 4.1. Test matrix for the experimental specimens .......................................................... 89

Table 5.1 Mechanical properties of GFRP bars used in the numerical simulations (Direction 1 is parallel to the dowel centre line and Directions 2 and 3 are the transverse directions). ................................................................................... 117

Table 5.2. Fracture energy (Nm/m² —(CEB-FIP 1990 1993)) ........................................... 121

Table 5.3. Coefficients $\alpha_F$ is related to maximum aggregate size (CEB-FIP 1990 1993)... 121

Table 5.4. Joint face deflection for different elements size (SV2N2) .................................... 127

Table 5.5. Comparison of deflection of concrete slab supported by different bases types.... 132

Table 6.1. Recommended steel dowel bar dimensions .......................................................... 135

Table 6.2. Comparison of ultimate deflection (mm) from the FEM and experimental tests ..................................................................................................................... 138

Table 6.3. Slab deflections at the face of the joint ................................................................. 142

Table 6.4. Shear force (kN) at the face of the joint of the loaded slab ................................. 143

Table 6.5. Maximum bending moments in different dowels ................................................. 143

Table 6.6. All combinations of dowel and pavement parameters for steel and GFRP dowels ....................................................................................................................... 148

Table 6.7. Design considerations for the GFRP dowels based on comparison with AASHTO (1993) guide and UK Highway Agency Requirements ....................... 149

Table 6.8. Bearing stress evaluation for the recommended design considerations of GFRP dowels ........................................................................................................... 149

Table 7.1. Simulation matrix for specimens containing two steel dowel bars ................. 154

Table 7.2. Simulation matrix for specimens containing two GFRP dowel bars ............... 155
Table 7.3. Simulation matrix for specimens containing three GFRP dowel bars ............... 156
Table 7.4. Comparison of joint face deflection (mm) from the experimental test with the FEM ................................................................................................................................. 160
Table 7.5. Maximum values of tensile plastic strain component (PEEQT) at each step of the analysis (see § 5.6.2 for steps definition) ........................................................................ 180
Table 7.6. Maximum values of compression plastic strain component (PEEQ) at each step of the analysis (see § 5.6.2 for steps definition) .............................................. 181
Table 7.7. Average damage ratios for different misalignment cases ................................ 183
Table 7.8. Comparison of LTE before “B” and after “A” slabs’ movement for specimens that contain two and specimens that contain three GFRP dowels ...... 200
Table 8.1. Material properties and layer depth ........................................................................................................... 210
LIST OF NOTATIONS

\( a \) Radius of contact area of tyre (mm)
\( d \) Dowel bar diameter (mm)
\( d_c \) Damage variable in compression.
\( d_l \) Deflection of loaded slab (mm).
\( d_{max} \) Maximum aggregate size (mm)
\( d_t \) Damage variable in tension
\( d_u \) Deflection of unloaded slab (mm).
\( f_c \) Compressive strength of concrete (MPa)
\( f_{ck} \) Characteristic compressive strength of concrete at 28 days (MPa)
\( f_{cm} \) Mean value of concrete cylinder compressive strength (MPa)
\( f_{ctm} \) Mean value of axial tensile strength of concrete (MPa)
\( h \) Thickness of pavement (mm)
\( k \) Modulus of subgrade reaction (MPa/mm)
\( k_o \) Modulus of dowel support (MPa/mm)
\( l \) Dimension of equivalent contact area of tyre (mm)
\( m \) Misalignment magnitude (mm)
\( s \) Spacing between dowels (mm)
\( u \) Magnitude of crack opening (mm)
\( u_1 \) Crack opening (mm) for \( \sigma_l = 0.15 f_{ctm} \), according to bi-linear stress-crack opening diagram for uniaxial tension (mm)
\( u_c \) Crack opening (mm) for \( \sigma_l = 0 \), according to bi-linear stress-crack opening model for uniaxial tension (mm)
Crack opening (mm) for $\sigma_t = 0$, according to stress-fracture energy model (mm)

Joint opening (mm).

Distance along dowel bar from the joint face (mm).

Deflection of dowel bar (mm)

Three Dimensional Finite Element Method

Cross-sectional area of dowel bar (mm$^2$)

American Association of State Highway and Transportation Officials

American Concrete Institute

Concrete Damaged Plasticity

Dowel Looseness (mm)

Young’s modulus of elasticity (MPa)

Longitudinal and transverse modulus of elasticity of GFRP dowels (MPa)

Initial (undamaged) elastic stiffness of the material (MPa)

Modulus of elasticity of the concrete pavement (MPa)

Secant modulus of elasticity of concrete (MPa)

Pull-out load per dowel bar (N)

Federal Highway Administration

Falling Weight Deflectometer

Shear modulus (MPa)

In-plane shear modulus (MPa)

Fracture energy (N.m/m$^2$).

Glass Fibre Reinforced Polymer

Second moment of area of dowel bar cross-section (mm$^4$)

Jointed Plain Concrete Pavement

embedded length of dowel bar (mm)

Load Transfer Efficiency

Load Transfer Efficiency based on deflection calculations
\( LTE_\sigma \) Load Transfer Efficiency based on stress calculations

LVDT Linear Variable Displacement Transducer

\( M \) Moment of the dowel bar (N. mm)

\( M_o \) Bending moment in dowel at the joint face (N. mm)

\( M_{\text{max}} \) Maximum bending moment of the dowel bar (N. mm)

\( N \) Number of load cycles

\( P \) Applied load (N)

PEEQ Equivalent plastic strain components in compression

PEEQT Equivalent plastic strain components in tension

\( P_T \) Total load transfer across the joint (N)

\( P_i \) Load transfer through the dowel bar (N)

RD Relative Deflection of joint faces (mm)

\( S_L \) Slab length of concrete pavement (the distance between the transverse joints) (mm)

TLE Transfer Load Efficiency

\( V \) Shear force at the dowel bar (N)

\( \alpha \) Coefficient of thermal expansion of concrete = 8 to 12 \( \times 10^{-6}/^\circ\text{C} \)

\( \alpha_F \) Coefficient relating fracture energy to the tensile strength and cracking displacement, depends on the maximum aggregate size

\( \beta \) Relative stiffness of the beam on elastic foundation (mm\(^{-1}\))

\( \delta \) Shear deflection of dowel bar (mm)

\( \varepsilon \) Strain

\( \varepsilon_c \) Compressive strain in concrete

\( \varepsilon_{c1} \) Compressive strain in concrete at peak stress

\( \varepsilon_d \) Coefficient of drying shrinkage of concrete 0.5 to 2.5 \( \times 10^{-4} \)

\( \varepsilon_{cu} \) Ultimate compressive strain in concrete

\( \varepsilon_c^{\text{in}} \) & \( \varepsilon_t^{\text{in}} \) Inelastic strain in compression and tension respectively
Plastic and elastic compressive strains respectively

Plastic and elastic tensile strains respectively

Form factor, equal to (10/9) for solid circular and ellipse sections

Coefficient of friction

Poisson’s ratio

Ratio of strain in transverse direction (2) to that in axial direction (1) for a stress in axial direction (1)

Ratio of strain in axial direction (1) to that in transverse direction (2) for a stress in transverse direction (2)

Allowable bearing stress of dowel bar (MPa)

Bearing stress of the dowel bar on the surrounding concrete (MPa)

Compressive stress in the concrete

Yield and ultimate stresses in compression

Stress in loaded concrete slab (MPa)

Stress in unloaded concrete slab (MPa)

Tensile stress in concrete

Tensile failure stress

In-plane shear stress

Bond stress at the dowel-concrete interface (MPa)

Eccentricity

Dilation angle

Additional joint opening due to temperature change and concrete shrinkage (mm)
ABSTRACT

Dowel bars are provided at the transverse joints of the Jointed Plain Concrete Pavement (JPCP) to transfer the load between adjoining slabs and to allow for expansion and contraction of the pavement due to temperature and moisture changes. The current study involved evaluation of the performance of Glass Fibre Reinforced Polymer (GFRP) dowels in JPCP as an alternative to the conventional epoxy-coated steel dowel bars, especially in the presence of dowel misalignment.

This research involved two main sets of experimental tests. The first set focused on the evaluation of load-deflection response of GFRP dowels using a scaled model of pavement slabs. The second set investigated the combined effect of dowel misalignment and cyclic wheel load on the performance of steel and GFRP dowels. The tested slabs (in the second set) were supported on a steel-beam base with stiffness such that the effects of the underlying layers of real pavements are incorporated. In both of these sets of experiment the GFRP dowels were compared with the steel dowels of similar flexural rigidity. The research also involved detailed numerical investigations using ABAQUS for all experimental tests in the current study. The validated numerical model was used to conduct three sets of parametric studies: to propose design considerations for the GFRP dowels; to simulate all important cases of dowel misalignment (111 cases) for steel and GFRP dowels and to give an insight into the damaged volume in the surrounding concrete pavement; and to investigate the effects of diameter, length and type of dowel bar, concrete grade, pavement thickness, and slab-base friction on the joint-opening behaviour.

The results from the first set of experiments showed that the 38 mm GFRP dowels perform better in terms of deflection response compared to the 25 mm steel dowels. Also, it was observed that the relative deflection (RD) is more sensitive to the changes in the joint width rather than the concrete strength. The numerical results from the first set showed a good agreement with the experimental results and showed lower magnitude and better distribution of stress in the concrete underneath the GFRP dowels as compared with the steel dowels. Finally, on the basis of a detailed parametric study (70 different cases), design considerations for GFRP dowels in JPCP were suggested.

The second set of experimental results showed that the GFRP dowels can withstand a cyclic traffic load and significantly reduce joint lockup and dowel looseness (DL) and can provide sufficient load transfer efficiency (LTE). It was also observed that the dowel misalignment affects DL significantly more than the repeated traffic load. Slab-base separation and the orientation of misaligned dowels have significant effects on the pull-out load required to open the joint. The numerical results from the second set indicated that the pull-out load was small for the vertical misalignment cases compared to the horizontal and combined misalignment cases. The results also indicated the occurrence of concrete spalling and deterioration at smaller joint openings for combined misalignment when compared to other misalignment types. The use of GFRP dowels significantly reduced the pull-out load and joint lockup when dowel misalignment exists. Consequently, the deterioration of the surrounding pavement substantially decreased. The long term performance of the JPCP fitted with GFRP dowels improves because of a reduction in the DL and the RD, and by maintaining a good LTE even for misaligned dowels. The numerical results also showed that the pull-out load increases significantly for an increase in the concrete compressive strength and the dowel bar diameter. Small increase in pull-out load was observed for higher embedded length of the dowel bar, whereas the increase was insignificant for an increase in the pavement thickness and slab-base friction.

In general, the study showed the GFRP dowel can be a potential alternative for the conventional steel dowel bars in JPCP.
DECLARATION

No portion of the work referred to in the thesis has been submitted in support of an application for another degree or qualification of this or any other university or other Institute of learning;
COPYRIGHT STATEMENT

I. The author of this thesis (including any appendices and/or schedules to this thesis) owns certain copyright or related rights in it (the “Copyright”) and he has given The University of Manchester certain rights to use such Copyright, including for administrative purposes.

II. Copies of this thesis, either in full or in extracts and whether in hard or electronic copy, may be made only in accordance with the Copyright, Designs and Patents Act 1988 (as amended) and regulations issued under it or, where appropriate, in accordance with licensing agreements which the University has from time to time. This page must form part of any such copies made.

III. The ownership of certain Copyright, patents, designs, trademarks and other intellectual property (the “Intellectual Property”) and any reproductions of copyright works in the thesis, for example graphs and tables (“Reproductions”), which may be described in this thesis, may not be owned by the author and may be owned by third parties. Such Intellectual Property and Reproductions cannot and must not be made available for use without the prior written permission of the owner(s) of the relevant Intellectual Property and/or Reproductions.

IV. Further information on the conditions under which disclosure, publication and commercialisation of this thesis, the Copyright and any Intellectual Property and/or Reproductions described in it may take place is available in the University IP Policy (see http://www.campus.manchester.ac.uk/medialibrary/policies/intellectual-property.pdf), in any relevant Thesis restriction declarations deposited in the University Library, The University Library’s regulations (see http://www.manchester.ac.uk/library/aboutus/regulations) and in The University’s policy on presentation of Theses
PUBLICATIONS

Journals


Conference

DEDICATION

This thesis is dedicated to my parents, brothers, wife and children.

For their endless love, support and encouragement
ACKNOWLEDGEMENTS

I wish to express my gratitude to my supervisor, Dr P. Mandal, for taking me under his wings and for his professionalism in providing guidance and keeping his eyes on my progress. Also, I would like to thank all laboratory staff for their help and Mr Tom Swailes for his valuable advice during the first set of my experimental work. I express my thanks to Dr Ahmed Abdullah, who was a PhD student in school of MACE for his advice on numerical modelling of concrete in ABAQUS.

I would like to acknowledge the financial support available from the Ministry of Higher Education, Iraq Government to conduct this research work. Acknowledgements are also due to Hughes Brothers, Inc. for supplying us the GFRP bars free of charge.

Finally, my deepest appreciation goes to my family especially my eldest brother Ali (a civil engineer) and to my loving wife. Without their help and support this work would not have been completed.

I am thankful for the endless love and support of my friends.
CHAPTER ONE
INTRODUCTION

1.1. Introduction

The Jointed Plain Concrete Pavement (JPCP) is one of the most widely used pavement types in highways, airports runways and aprons due to its ability to sustain high traffic wheel loads with low maintenance and high durability. JPCP consists of concrete slabs separated by transverse and longitudinal joints resting on one or more layers of foundation such that it possesses better resistance to failures of underlying layers and can handle the harshest conditions. The transverse joints can be classified into expansion joints and contraction joints. In case of expansion joints a small cap is introduced at the end of the dowel bar to allow for any movement during expansion of the pavement.

Concrete pavements suffer several types of volumetric strains due to temperature changes during the setting of concrete, shrinkage, daily and seasonal variations in ambient temperatures and moisture conditions. Transverse joints are typically included within the JPCP to relieve the longitudinal stresses due to these volumetric changes. The main objectives of creating joints in concrete pavements involve (Bussell and Cather 1995; Kelleher and Larson 1989; Packard 1973):

- Allowing for slab movement during concrete expansion and contraction due to changes in ambient temperature, moisture condition, and thermal gradient along the slab depth.
- Controlling transverse cracking (mid span) of concrete pavements.
• Additionally, joints are also introduced due to discontinuous construction activities.

The applied traffic load is transferred across these joints by dowel bars or aggregate interlocks or both. Therefore, these joints are either (i) non-dowelled, when the joint width is less than 1 mm, or when the road is serving low traffic volume or (ii) dowelled, where the joint width is greater than 1 mm or for serving moderate or heavy traffic volume.

Many highway agencies strongly recommend using dowel bars to transfer the load across the joints, minimizing the relative deflections of the pavements, providing smooth riding as well as safe and comfortable road usage. Often, pavements and joint distresses develop due to insufficient load transfer or improper functioning of the dowels (AASHTO 1993).

1.2. Problem Statement

Steel dowel bars are the most conventional load transfer devices used across the transverse joints of JPCP to distribute the load between the adjacent concrete pavement slabs and maintain the horizontal movement. These bars are usually given a corrosion protection coating at the time of installation, which usually wears off with age depending on the loading and environmental conditions, and corrosion takes place eventually requiring huge maintenance cost. The corrosion of steel dowels usually causes the dowels to expand and freeze in the concrete socket, and a flaky layer is initiated due to that expansion. Also, the degree of expansion is unlikely to be uniform along the whole length (Mancio et al. 2008); therefore gaps may develop at the dowel-concrete interface. Consequently, the corrosion contributes to dowel looseness around the dowel bars. This also affects the surface smoothness of steel dowels, the lack of which offers higher resistance to slab movements, generating a substantial amount of locked up stresses. Furthermore, due to the high modulus of elasticity of steel dowel bars, high bearing stress may be induced in the concrete surrounding the dowels owing to repeated traffic load. The combined effect of dowel looseness and high bearing stress causes rapid deterioration of pavements.
Aligned dowel bars are typically designed to be at a mid-depth of the pavement thickness, parallel to the centreline of the pavement, equally spaced, and of equal embedded length on each side of the joint. Dowel misalignment refers to tilted or misplaced configuration of dowels in the basket, or the movement of the basket itself during the casting of concrete or both. Dowel misalignment may produce joint lockup causing joint distresses and reduction in load transfer (Khazanovich et al. 2001; Tayabji 1986). Resistance to the slab movement due to dowel misalignment may also produce more localised deterioration of the concrete pavement surrounding the dowel bars, and thereby creating an additional enlargement of the dowel bar socket (dowel looseness).

The combined deterioration in concrete pavement surrounding the dowel bars due to corrosion, repeated traffic load and dowel misalignment have a significant effect on the long term performance of the transverse joints of a JPCP and thus on the pavement service life.

1.3. Objectives of the Current Study

The current study deals with the performance of dowel bars as load transfer devices in JPCP. It involves experimental and numerical investigations for different aspects which may affect that performance, and it also includes investigations into alternative corrosion free material to the conventional epoxy-coated steel dowel bars. Therefore, the main objectives of this study are:

1. Analysing the theory of load transfer mechanisms and the performance of dowel bars as a load transfer device in a transverse joint of JPCP by carrying out a detailed literature review on the subject, and clearly identifying the gaps in the knowledge.

2. Evaluating the suitability of using Glass Fibre Reinforced Polymer (GFRP) dowel bars as an alternative corrosion free material to conventional epoxy-coated steel dowels by experimental investigation. Load-deflection responses of GFRP dowels were determined for various parameters including joint opening widths, different grades of concrete and the ultimate load carried by these dowels for each case. The results were compared with those of steel dowels.
3. Conducting experimental tests to study the combined effect of dowel misalignment and wheel load on dowel bars performance in JPCP for various cases of GFRP and steel dowel misalignment. The experimental procedure aims to address important limitations of previous studies by introducing suitable representation for pavement foundation which can be considered as more realistic representation and allow for the investigation of the combined effect of dowel misalignment and wheel load. The tests will demonstrate the appropriateness of GFRP dowels to minimize pull-out load and joint lockup, and adequately transfer the applied load when it is subjected to cyclic load test.

4. Carrying out a detailed parametric study using a validated numerical model to develop design considerations for the GFRP dowel bars.

5. Developing a detailed numerical model to simulate the combined effect of dowel misalignment and wheel load on the dowel bars’ performance in JPCP for a range of values and types of dowel misalignment. The numerical simulation involved the assessment of damage occurrence in concrete pavement surrounding the dowel bars, which is difficult to achieve by experimental investigation. This analysis can clearly show the changes in stress/strain distribution due to the effects of wheel load and dowel misalignment, indicating the severity of each case of dowel misalignment, and the suitability of GFRP dowels as an improvement over the steel dowels. Consequently, this work will help to develop guidelines for allowable dowel misalignment tolerances.

6. Investigating the effects of different dowel and pavement parameters on the pull-out load - joint opening behaviour and associated distresses.

1.4. Outline of the Thesis

The dissertation “Evaluation of the Performance of GFRP Dowels in JPCP (Road/Airport) under the Combined Effect of Dowel Misalignment and Wheel Load” consists of nine chapters outlined as follows:

Chapter One gives a general background on JPCP and dowel bars as load transfer devices. It also states the problems and the objectives of the current study, and presents a thesis structure.
Chapter Two discusses the dowel bar theory, different aspects related to the load transfer mechanisms and its efficiency, and the alternative materials used for conventional steel dowel bars by reviewing previous studies in literature and identifying the limitations of the previous research.

Chapter Three presents a detailed experimental investigation of the load-deflection responses of the GFRP dowels as a corrosion-free alternative material compared to the epoxy-coated steel dowel bars.

Chapter Four introduces several experimental tests that show the combined effects of traffic wheel loads and dowel misalignment on the dowel bars’ performance in JPCP. The test methodology involved examining the use of GFRP dowels to minimize the pull-out load and the deterioration of the pavement.

Chapter Five illustrates the finite element modelling, and techniques which were used to simulate the experimental tests and to carry out the parametric studies.

Chapter Six presents a numerical simulation for the experimental investigation of the load-deflection responses of the GFRP dowels. Based on the validated numerical model a set of design considerations for the GFRP dowels were suggested.

Chapter Seven involves a detailed numerical analysis of the combined effect of traffic wheel loads and dowel misalignment on the dowel bars’ performance in JPCP. It includes realistic misalignment cases, and introduces new techniques to assess the damage on the concrete pavement surrounding the dowel bars. This analysis involved both steel and GFRP dowels.

Chapter Eight shows the effects of different dowel and pavement parameters on pull-out load – joint opening behaviour and associated distresses.

Chapter Nine highlights the achievements of the current research and lists the conclusions of the current study and gives recommendations for future studies.
CHAPTER TWO
LITERATURE REVIEW

2.1. Introduction

In general, typical section of JPCP consists of plain concrete slabs supported by one or two base layers and subgrade soil (existing soil) as shown in Figure 2.1. Transverse joints are constructed between these slabs to maintain the slab movement during concrete expansion and contraction due to changes in temperature and moisture. The applied traffic load is transferred across these joints by dowel bars and aggregate interlock (see Figure 2.2). Inadequate load transfer or improper function of the dowels may lead to several types of distress such as the following (AASHTO 1993; Jung et al. 2008; Miller and Bellinger 2003; UK Highway Agency 1994).

- **Faulting**, which is the difference in the vertical relative deflection of the transverse joint faces resulting mainly from inadequate load transfers.
- **Spalling**, which is the breaking or cracking of the concrete at the joint face due to joint lockup, traffic load, improper joint design or construction and/or freezing and thawing action.
- **Transverse cracking**, straight cracks which initiate at approximately right-angles to the pavement longitudinal centreline due to high tensile stress resulting from improper joint function, design or construction.
- **Corner break**, which is a pavement crack intersecting with the joint edges at the corner within no more than 1.83 m (6 ft.) from the slabs corner due to poor load transfer, thermal expansion and contraction and/or repeated traffic load.

See Figure 2.3 for the above mentioned distresses.
The general requirements of the dowel bars are to provide adequate load transfer across the joint (primarily by shear mechanism), to reduce the relative deflection at the joint face. The dowels should also be able to accommodate the contraction and expansion of the slabs, i.e. they should have low bond strength with concrete and very low coefficient of friction.
Figure 2.3. Some of the common distresses in concrete pavement (Jung et al. 2008; Miller and Bellinger 2003)
The current chapter presents a comprehensive literature review that includes dowel bar theory, load transfer mechanisms, dowel looseness, dowel misalignment and alternative dowel materials (GFRP dowel bars).

A brief description is included in this chapter for each of these subjects. Also, identifications for the limitations and shortcomings of the previous studies found in the literature are presented in this chapter.

2.2. Dowel Bar Theory

2.2.1. Analytical investigation

Timoshenko and Lessels (1925) developed an analytical model of dowel bar embedded in pavement slabs based on the original work of a beam on elastic foundation proposed by Winkler in 1867. According to this formula, the beam deflection can be calculated using the fourth order differential equation shown in Equation (2.1):

$$EI \frac{d^4 y}{dx^4} = -ky$$  \hspace{1cm} (2.1)

where, $x$: the length coordinate; $y$: the deflection; and $EI$: the flexural rigidity of the beam. The parameter $k$ represents foundation stiffness. The general solution for the above differential equation can be written as:

$$y = e^{\beta x}(A \cos \beta x + B \sin \beta x) + e^{-\beta x}(C \cos \beta x + D \sin \beta x)$$  \hspace{1cm} (2.2)

where,

$$\beta = \sqrt{\frac{k}{4EI}}$$  \hspace{1cm} (2.3)

The constants $A$, $B$, $C$ and $D$ can be obtained by substituting appropriate boundary conditions of the beam. According to the assumption of a semi-infinite beam with moment $M_o$, and point load $P_t$, Equation (2.2) can be re-written as:

$$y = \frac{e^{-\beta x}}{2\beta^3 EI} [P_t \cos \beta x - \beta M_o (\cos \beta x - \sin \beta x)]$$  \hspace{1cm} (2.4)
Here, $y$: dowel bar deflection; $E$: modulus of elasticity of dowel bar; $I$: second moment of area of dowel bar cross-section; $P_l$: load transfer by dowel bar; $k$: foundation modulus; $\beta$: relative stiffness of the dowel bar.

Accordingly, the bending moment and shear force can be written as follows:

\[
-Em \frac{d^2y}{dx^2} = M = -\frac{e^{-\beta x}}{\beta} [P_l \sin \beta x - \beta M_o (\sin \beta x + \cos \beta x)]
\]  
\[ (2.5) \]

\[
\frac{dM}{dx} = V = -e^{-\beta x} [(2\beta M_o - P_l) \sin \beta x + P_l \cos \beta x]
\]  
\[ (2.6) \]

The maximum bending moment occurs at the location of zero shear force ($dM/dx=0$). Hence, the maximum bending moment can be expressed as:

\[
M_{\max} = -\frac{P_l e^{-\beta x}}{2\beta} \sqrt{1 + (1 + \beta w)^2}
\]  
\[ (2.7) \]

where the joint opening width, $w = -2M_0/P_l$.

The deflection at the face of the joint can be calculated from Equation (2.4) by substituting $x=0$, and $M_o = -\frac{P_l w}{2}$.

\[
y = \frac{P_l}{4\beta^3EI} [2 + \beta w]
\]  
\[ (2.8) \]

Friberg (1940) replaced the foundation modulus ($k$) in Equation (2.3) by $k_o d$ in which $k_o$ is the modulus of the dowel support and $d$ is the dowel bar diameter (see Equation (2.9)). The modulus of the dowel support represents the elastic properties of the dowel bar-pavement system. It is defined as the pressure on the dowel bar that is necessary to cause 1 mm deflection in the concrete pavement at the face of the joint; hence, it is the reaction of the concrete under load (Friberg 1938). It is difficult to determine the value of $k_o$ analytically. However, the value of relative stiffness of dowel bar ($\beta$) can be estimated from the experimentally measured deflection of the dowel bar at the face of the joint using Friberg’s formula (Equation 2.10). The value of $k_o$ then can be obtained using Equation (2.9).
\[ \beta = \sqrt[4]{\frac{k_o d}{4EI}} \]  \hspace{1cm} (2.9)

\[ y = \frac{P_t - \beta M_o}{2\beta^3EI} \]  \hspace{1cm} (2.10)

There are many factors which influence \( k_o \), such as the load magnitude, the depth of the concrete under the dowel bar, the shape and size of the dowel, material properties of concrete (modulus of elasticity, compressive strength), and the yield strength of the dowel (Friberg 1938). Higher value of \( k_o \) leads to more load being transferred by the dowel across the joint.

As mentioned before, Friberg’s (1940) equation assumed that the dowel bar has semi-infinite length. In practice, dowel bars have a finite length, which means that this equation cannot be applied. However, a subsequent study has shown that this equation can be applied when \( \beta L \) is greater than 2 (Porter et al. 1992).

Friberg (1940) assumed that the relative deflection (RD) across the transverse joint of JPCP consists of three parts: the deflection of the dowel at the face of the joint \( (y) \), the deflection due to the slope of the dowel \( \left( \frac{w_d y}{2dx} \right) \) and the flexural deflection \( \left( \frac{P_t w^3}{12EI} \right) \).

Accordingly, the RD is obtained as given in Equation (2.11):

\[ RD = 2y + w\frac{dy}{dx} + \frac{P_t w^3}{12EI} \]  \hspace{1cm} (2.11)

Porter et al. (2001) reported that the RD across a joint has a fourth term called ‘shear deflection’ \( (\delta) \) and thus the RD (the total difference in deflection between two adjacent slabs) is presented in Equation (2.12), (see Figure 2.4):

\[ RD = 2y + w\frac{dy}{dx} + \delta + \frac{P_t w^3}{12EI} \]  \hspace{1cm} (2.12)

\[ \delta = \frac{\lambda P_t w}{A_d G} \]  \hspace{1cm} (2.13)

where, \( G \): shear modulus; \( A_d \): cross-sectional area of dowel bar; \( \lambda \): form factor, equal to (10/9) for solid circular and elliptical cross-sections.
For a small joint opening of around 3 mm (1/8 in.) which is common for contraction joints, the deflection due to slope and flexural deflection is very small and can be neglected (Porter et al. 2001). Therefore, Equation (2.12) can be written as:

\[ RD = 2y + \delta \]  

(2.14)

![Diagram of relative deflections between adjacent pavement slabs](image)

**Figure 2.4. Relative deflections between adjacent pavement slabs**

### 2.2.2. Bearing stress at the dowel-concrete interface

The bearing stress of dowel bars in concrete pavements at the face of a joint is an important parameter in order to assess their effectiveness as load transfer devices. A significant increase in bearing stress produces crushing of the concrete that is in contact with the dowel. The crushing increases with the repetitive traffic load and that creates a void underneath the dowel bar. This void also produces an additional deflection for the dowelled-joint, and consequently the load transfer does not start until the dowel comes into contact with the pavement. Hence, the transferred load across the joint decreases which lead to more localised pressure on the subgrade, creating joint faulting and the possibility of pumping of the fine material underneath the pavement.

According to the general assumption, the dowel bar behaves as a beam on an elastic foundation. A direct relationship had been established between the dowel bar deflection and bearing stress \( \sigma_b \) at the face of a joint as shown in Equation (2.15) (Friberg 1940):
\[ \sigma_B = k_o y \] (2.15)

The bearing stress in the above equation should not exceed the allowable value recommended by the American Concrete Institute (ACI) Committee 325 (1956), in order to minimize the crushing of concrete underneath the dowel. The allowable bearing stress can be determined using Equation (2.16):

\[ \sigma_a = \left( \frac{4 - \left( \frac{d}{25.4} \right)}{3} \right) f_c \] (2.16)

where, \( \sigma_a \) : Allowable bearing stress (MPa); \( f_c \) : Concrete compressive strength (MPa)

### 2.3. Load Transfer Mechanism and Load Transfer Efficiency (LTE)

Transverse joints are provided within the JPCP to relieve the longitudinal stresses due to volumetric changes in the pavement. These joints are mostly dowelled joints for joints greater than 1 mm or in pavements under high traffic roads. The dowels maintain the horizontal movement of the pavement during expansion and contraction and transfer the load between the adjoining slabs.

In general, when a wheel load is placed at the joint edge, it would be expected that part of this load will transfer to the subgrade and for the remaining load to be transferred across the joint. For an ideal load transfer mechanism, 50% of the applied load would be transferred directly to the subgrade under the loaded slab, and 50% transferred across the joint to the adjacent slab. However, due to repetitive traffic loads, environmental conditions, and limited durability of the dowel bars, the joint behaviour in the ideal manner (50% load transfer) cannot be expected. The amount of load transfer across the joint is a function of several parameters such as dowel diameter, dowel spacing, the embedded length of the dowel, joint width, dowel looseness (concrete deterioration or enlargement of the dowel bar socket), and the properties of the dowel and the concrete (Maitra et al. 2009). Yoder and Witczak (1975) stated that there are many factors that can cause a reduction in load transfer such as dowel looseness (DL) and dowel misalignment. These factors reduce the total load transfer across the joint by about 5% - 10%. Therefore, they suggested a 45% load transfer for the purpose of design. Brown
and Bartholomew (1993) stated that a range of 35% - 40% of the applied load transferring across a joint would be acceptable for roads used by heavy trucks.

The ability of a joint to transfer an applied load can be calculated using several formulas from the literature (AASHTO 1993; Hammons 1998; Ioannides and Korovesis 1992; Ioannides et al. 1990; Wadkar et al. 2011).

1- Load transfer efficiency based on deflection calculations (LTE$_\delta$).

\[
LTE_\delta = \frac{d_u}{d_l} \times 100\% = \frac{\text{Deflection of the unloaded side}}{\text{Deflection of the loaded side}} \times 100\% \tag{2.17}
\]

2- Transfer load efficiency (TLE)

\[
TLE = \frac{P_T}{P} \times 100\% = \frac{\text{Total load transferred}}{\text{Total applied load}} \times 100\% \tag{2.18}
\]

3- Load transfer efficiency based on stress calculations (LTE$_\sigma$).

\[
LTE_\sigma = \frac{\sigma_u}{\sigma_L} \times 100\% = \frac{\text{Bending stress of the unloaded side}}{\text{Bending stress of the loaded side}} \times 100\% \tag{2.19}
\]

LTE$_\delta$ and TLE are usually used to specify the joint effectiveness in the transferring part of the wheel load placed on a joint face of the approach slab to the adjacent leave slab. LTE$_\delta$ is a criterion suggested by the AASHTO (1993) to evaluate the effectiveness of load transfer devices in transferring wheel load across a joint of JPCP. It is calculated from the ratio of the deflection of the unloaded side ($d_u$) to the loaded side ($d_l$) at the outer edge of the wheel load, near the joint face as shown in Equation (2.17). The AASHTO (1993) guide states that a range of 70-100% of LTE$_\delta$ is adequate to obtain an appropriate load transfer across the joint for a wheel load of 40 kN. The TLE is another criterion for measuring joint effectiveness in JPCP, proposed by Ioannides et al. (1990). It represents the ratio between the total load transfers across the joint to the total applied load as shown in Equation (2.18). Third criterion is LTE$_\sigma$, which is the ratio of bending stress of the unloaded side to that of the loaded side of a slab. Since LTE$_\delta$ is easier to measure in the field using Falling Weight Deflectometer.
(FWD) and used in most of the previous studies (Wadkar et al. 2011), it is considered in
the current study and it is abbreviated henceforth as of LTE for the sake of brevity.

2.3.1. Factors affecting load transfer efficiency (LTE)

LTE is governed by many parameters as previously stated. Some of these are related to
the load transfer device such as the dimensions, specifications and spacing of the dowel
bars or are related to the concrete pavement dimensions, properties, load position and
the temperature of the slab (Shoukry et al. 2005). These factors are mostly related to the
JPCP design. The other factors causing reduction in load transfer are related to the
construction process of the concrete pavement or performance during the service life of
a road, such as dowel looseness (DL) and dowel misalignment. A brief description of
these factors is given in the following sections.

2.3.1.1. Dowel looseness (DL)

The dowel looseness (DL) refers to gaps or voids between a dowel bar and the
surrounding concrete pavement. Two types of looseness can be observed — (i) initial
looseness, which develops due to several factors including the coating of the dowel
(bond breaker), the air and water voids that can be found especially under the dowel,
and the shrinkage of the concrete during hardening; (ii) the looseness due to repeated
traffic load that causes the crushing of the concrete surrounding the dowel bar and the
displacement of the crushed particles (Teller and Cashell 1958). Corrosion of steel
dowel bars may also produce additional looseness. The shape and size of DL may vary
along the embedded length of the dowel bar. Several experimental and numerical
studies were carried out to investigate DL. A summary and discussion for these studies
are shown below.

2.3.1.1.1. Experimental studies

Laboratory investigations into the load transfer mechanism across the joints of JPCP
started in the early years of the 20th century. However, the investigations of the effect of
DL on dowelled-JPCP performance had been limited. Teller and Cashell (1958) carried
out laboratory tests to investigate the performance of the dowel bars at transverse joints
of JPCP. Their study consisted of two concrete slabs connected by four dowel bars that
were subjected to a repetitive load alternating on both sides of the joint. These slabs
were resting on steel supporting beams. The flexible support offered by those beams can represent the subgrade deflection when a load is applied to a pavement. The tests involved an investigation into various dowel diameters in the range of 16-32 mm (⅝-1¼ inch) and slabs of varying thicknesses ranging from 150-250 mm (6-10 in.), joint openings ranging from 0-25 mm (0-1 in.) and different embedded lengths. The case of a single dowel across the joint was also investigated. The results showed that initial DL (measured in terms of the increase in RD) across a joint developed as a result of concrete shrinkage, and air or water voids. The results also showed that DL increases with the number of load cycles. The initial looseness and looseness due to load cycles vary with the dowel diameter, embedded length of the dowel bar and load magnitude. A considerable loss in load transfer was observed for small DL value. A looseness magnitude of 0.2 mm produced an 8.5% reduction in the load transfer across the joint.

Buch and Zollinger (1996) conducted an experimental investigation into the effect of DL on the load transfer by dowel bars in the transverse joints of a concrete pavement. They investigated the effects of type, shape and texture of aggregates on LTE. The effects of the dowel diameter, spacing of the dowels, joint opening, and load magnitude and load cycles on LTE were also investigated. Based on a statistical analysis for the experimental results, an empirical formula was proposed to compute the reduction in LTE due to DL. The results showed that the LTE decreases from 80% to 60% by increasing the DL from 0.15 mm to 0.5 mm.

2.3.1.1.2. Numerical studies

DL was also investigated using a numerical approach in which several assumptions were made regarding the shape and size of DL. Channakeshava et al. (1993) modelled a dowelled-jointed pavement system by simulating the dowel bar as beam elements, and the concrete pavement as solid quadratic elements. The nonlinearity of the material, geometry, and the loading conditions were considered in this study. The dowel-concrete interaction was modelled by spring elements. A small gap was introduced around the dowel bar to represent the DL; consequently the load transfer began only after an initial dowel deformation equal to the amount of the gap that was introduced. The results indicated the occurrence of a high local deformation arround the dowel at the joint face.
due to stress concentration. This deformation produced a significant reduction in the load transfer.

In a subsequent attempt, Zaman and Alvappillai (1995) developed a finite element model (FEM) to investigate multiple-JPCP slabs subjected to a moving aircraft load. The concrete pavement was modelled with 4-noded thin plate elements, whereas the dowel bars were modelled by plane frame elements. The dowel-concrete interaction was represented by contact elements. One end of the dowel bar was considered fully embedded within the concrete pavement, while the other end was allowed to move by maintaining the dowel-concrete interaction that was specified in the analysis. The DL was assumed as the initial clearance within the contact elements between the free end of the dowel bar and the surrounding concrete pavement. The results indicated that an increase in DL produces an increase in the deflection of the loaded side of the pavement, and a decrease in LTE. The increase in DL from 0 mm to 0.127 mm caused a decrease in LTE from 99% to 70%.

Davids and Mahoney (1999) verified FEM software (Ever FE which was developed to simulate the load transfer in JPCP) with the experimental results of Hammons (1997). Pavement layers were modelled by 20-noded solid elements supported on a Winkler foundation while the dowels were modelled by 3-noded quadratic beam elements. All of these components were assumed to behave as a linear elastic material. A uniform gap of 0.08 mm around the dowel bars in the unloaded side of the slab was only provided as uniform DL along the embedded length of the dowel. They observed that their results agreed well with the experimental results when they considered this amount of DL compared with a no DL model. These results indicated an initiation of DL in the experimental test.

Another study was conducted by Davids (2000) to examine the effect of DL on JPCP response by introducing a different shapes of DL. The DL shape was assumed as shown in Figure 2.5, in which the author tried to simulate the crushing of concrete at the joint faces. According to the results of that study, LTE decreases from 95% for a zero gap to 65% - 60% for a 0.12 mm gap. For a gap size of 0.24 mm, LTE was about 50% - 55%. Also, the maximum vertical stress subjected to the base layer underneath the loaded slab changed from 107 kPa to 180 kPa when the gap changed from 0.00 to 0.12 mm.
In a subsequent study by Davids et al. (2003), the DL was assumed as a uniform gap around the dowel bar and it was symmetric on both the loaded and unloaded sides. The results of this study showed that the total load transfer decreased by 70% for the gap size of 0.2 mm. It should be mentioned that the assumption of similar DL in the loaded and unloaded sides is difficult to justify as the load transfer is never ideal in reality.

Shoukry et al. (2001) investigated the LTE of the dowel bars at transverse joints of JPCP by using falling weight deflectometer (FWD). Their analysis included using a 3D FEM to simulate the various conditions of contact such as full bond, sliding along the interface and the vibration of dowel due to the gap between concrete and dowel. They modelled all the components of the pavement system (pavement layers and dowels) with brick elements, and assumed a length-wise uniform gap only at the underside at both ends of the dowel bars. A linear elastic behaviour was assumed for all the materials in the JPCP system. The results showed that LTE significantly decreased with the increase in DL magnitude; also, the tensile stress at the bottom of the loaded slab increased by 10% when the maximum investigated value of the gap (0.2 mm) was reached. However, a uniform DL at both ends of the dowel is less realistic as mentioned in the previous paragraph.

Kim and Hjelmstad (2003) carried out a numerical analysis using ABAQUS (FE software) to investigate the structural behaviour of JPCP in airports for several load configurations, slab thicknesses, and DL values. They modelled the pavement and supporting layers by solid elements and used beam elements to model the dowel bars and a linear elastic material model was used for the concrete slabs. Dowel bars were embedded at the loaded side of the pavement while a unidirectional gap along the embedded length of each dowel was provided underneath the dowel at the unloaded side.
of the pavement by using a gap contact algorithm. The results indicated a significant variation and reduction in transferring load with an increase of gap size. It showed that load transfer reduced from 22% to 0% when the gap size increased from 0.00254 mm to 0.254 mm for a single wheel load case. For the tri-tandem load case, the transferring load decreased from 12.8% to 1.4% and the maximum bending stress in the loaded side increased by 18% when the gap size increased from 0.00254 mm to 1.02 mm.

Maitra et al. (2009) conducted a 3D FE analysis using ANSYS software to investigate the influence of dowelled-JPCP parameters (diameter, length and spacing of dowels, $k_o$, DL, concrete slab thickness, joint opening, and properties of the concrete and the subgrade) on LTE. They modelled the pavement layers by 8-noded solid brick elements while using 3D beam elements for the dowel bars. All of the pavement components were modelled as a linear elastic material. The dowel bars were embedded in the loaded side of the pavement while linear springs and a contact interface were used to simulate the dowel-concrete interaction at the unloaded side. A uniform gap along the embedded length (uniform around the dowels) of all the dowels on the unloaded side was provided as DL. In order to investigate the effect of DL on LTE, the size of the gap was varied from 0 mm to 0.1 mm. An empirical formula was proposed for the effect of DL on the LTE. According to that formula, the LTE decreases from 90% to 58% when the gap size (DL) increases from 0 mm to 0.1 mm for $k_o= 407$ MPa/mm.

Most of the previously mentioned studies modelled the dowel bars and concrete pavements as linear elastic materials. Although it is true that the stress due to wheel load is lower than the compressive strength of concrete, traffic wheel loads may produce high-localised stresses and deterioration at the dowel-concrete interface which creates DL. These studies assumed the DL as a predefined gap in which several assumptions were made for the size, shape and location of the DL. The DL was assumed to be due to the repetitive load only whereas additional looseness could arise due to dowel-concrete friction or due to dowel misalignment during the expansion and contraction of the pavement.

### 2.3.1.2. Dowel misalignment

Dowel bars’ misalignment or misallocation is a construction defect that may occur during the construction process of the joints, causing the dowel bars to deviate from the
pavement centreline vertically or horizontally or in both directions. This may also restrain the movement of slabs by locking the transverse joints that causes an increase in the effective length of the slab. The lockup of these joints can initiate problems including transverse cracks, corner breaks, joint spalling and faulting (see Figure 2.6) (Khazanovich et al. 2009; Tayabji 1986).

Dowel misalignments can be classified into the following two types: (a) translational misalignments, which occurs when the whole dowel bar is offset from the central plane of the concrete slab in any direction (horizontally, vertically or longitudinally) but it remains parallel to the centreline of pavement (see Figure 2.7); (b) skew or rotational misalignments, which occurs when the dowel remains at the central plane of the slab but it tilts horizontally or vertically or in both directions (see Figure 2.8) (Khazanovich et al. 2001). The focus of the current study is on the skew misalignments since they lead to more restraint in the slab’s movement and because they are more detrimental than the translational misalignments (Khazanovich et al. 2001; Prabhu et al. 2006).

Dowel misalignment tolerances vary according to the various highway agencies around the world. These tolerances are illustrated in Table 2.1.

![Figure 2.6. Joint spalling due to dowel misalignment (Khazanovich et al. 2009)](image-url)
Figure 2.7. Sectional view of JPCP with translation misalignments (a) Longitudinal; (b) Vertical

Figure 2.8 Different types of skew misalignment: (a) Sectional view, non-uniform vertical misalignment; (b) Sectional view, uniform vertical misalignment; (c) Plan view, uniform horizontal misalignment; (d) Plan view, non-uniform horizontal misalignment; (e) Plan view, partial horizontal misalignment (h: slab thickness, s: dowel bars spacing, m: misalignment magnitude).
Table 2.1 Misalignment tolerances in mm per half length of the dowel bar (225 mm) according to various agencies (Rao et al. 2009)

<table>
<thead>
<tr>
<th>Agency</th>
<th>Skew Misalignment</th>
<th>Translation Misalignment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Vertical</td>
<td>Horizontal</td>
</tr>
<tr>
<td>Great Britain</td>
<td>4.87</td>
<td>4.87</td>
</tr>
<tr>
<td>Germany</td>
<td>9.36</td>
<td>9.36</td>
</tr>
<tr>
<td>Multiple Agencies*</td>
<td>3.12</td>
<td>3.12</td>
</tr>
<tr>
<td>Montana</td>
<td>3.12</td>
<td>3.12</td>
</tr>
<tr>
<td>North Dakota</td>
<td>3.12</td>
<td>3.12</td>
</tr>
<tr>
<td>Tennessee</td>
<td>3.12</td>
<td>3.12</td>
</tr>
<tr>
<td>Nevada</td>
<td>6.24</td>
<td>6.24</td>
</tr>
<tr>
<td>Missouri</td>
<td>6.24</td>
<td>6.24</td>
</tr>
<tr>
<td>Kansas</td>
<td>4.74</td>
<td>4.74</td>
</tr>
<tr>
<td>Indiana</td>
<td>4.74</td>
<td>4.74</td>
</tr>
<tr>
<td>North Carolina</td>
<td>4.74</td>
<td>4.74</td>
</tr>
<tr>
<td>Illinois</td>
<td>2.37</td>
<td>2.37</td>
</tr>
<tr>
<td>Delaware</td>
<td>2.37</td>
<td>2.37</td>
</tr>
<tr>
<td>South Carolina</td>
<td>7.02</td>
<td>7.02</td>
</tr>
<tr>
<td>Georgia</td>
<td>7.02</td>
<td>7.02</td>
</tr>
<tr>
<td>Alabama</td>
<td>3.12</td>
<td>8.61</td>
</tr>
<tr>
<td>New York</td>
<td>N/A</td>
<td>2.00</td>
</tr>
<tr>
<td>Ohio</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Pennsylvania</td>
<td>2.87</td>
<td>2.87</td>
</tr>
<tr>
<td>Ministry of Transportation</td>
<td>3.00</td>
<td>3.00</td>
</tr>
<tr>
<td>Ontario</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Arkansas, Connecticut, Hawaii, Idaho, Kentucky, Minnesota, Texas, Utah, Wisconsin, Nebraska, Iowa, Michigan

+ N/A: Not Applicable

++ h: Pavement thickness

A comprehensive review of the previous dowel misalignment studies are presented in the following sections.

2.3.1.2.1 Experimental studies:

A few experimental studies have been conducted to evaluate the effect of dowel misalignment or misallocation on joint opening behaviour. Smith and Benham (1938) carried out laboratory tests on a small section of JPCP with 19 mm diameter dowel bars spaced at 300 mm. One side of the slab was pulled out while the other side was fixed to the testing frame. Various misalignment orientations were examined by opening the
joint after 28 days of the concrete casting. The results showed concrete spalling occurred for a 150 mm thick slab at a joint opening of 19 mm for a misalignment magnitude of 12.7 mm (0.5 in.) per half length of the dowel bar. Whereas for a 125 mm thick slab, a slight spalling was observed for a misalignment magnitude of 3.2 mm per half length of the dowel. The results indicated no significant spalling for a joint opening less than 12.7 mm and a misalignment magnitude of up to 19 mm. For all misalignment types, the pull-out load did not exceed 13.5 kN for a joint opening of 12.7 mm.

Segner and Cobb (1967) tested a concrete pavement section, 1.83 m (6 ft) wide by 1.68 m (5.5 ft) long and 0.25 m (10 in) thick which was fitted with 32 mm diameter steel dowel bars. The tests were conducted at 2 and 7 days after concrete casting. The results showed that the load required for opening the joint significantly increased for a misalignment magnitude greater than 3.2 mm per half length of the dowel bar compared with that of the aligned dowels. It also indicated that the combined misalignment required more pull-out load to open the joint by more than 12.7 mm as compared with the horizontal misalignment.

Tayabji (1986) carried out laboratory tests on a slab section of 0.92 m (3ft) wide, 2.1 m (7ft) long, 0.2, 0.25 m (8, 10 in) thick, and 3.2 mm (1/8 in.) transverse joint width. The test framework was built up using a channel shaped steel member. One half of the framework was held strongly to a rigid base while a hydraulic jack pulled the other side. The tests were conducted on specimens having a single misaligned dowel, and two non-uniformly misaligned dowels. Misalignment levels of 0, 3.2 mm, 6.4 mm, 12.7 mm, 25.4 mm and 50.8 mm per half-length of dowel (225 mm) were investigated. The slabs were pulled to form a joint opening of 6.4 mm. The results showed no significant difference in the pull-out load for single misaligned dowel specimens, whereas a significant increase in the pull-out load was observed for the specimens containing two misaligned dowels when the misalignment magnitude exceeded 12.7 mm. The maximum pull-out load for a single dowel was 5.5 kN at a joint opening of 6.4 mm. For the specimens having two misaligned dowels, the maximum average pull-out load per dowel was 4.5 kN for a misalignment magnitude less than 12.7 mm while it increased up to 18 kN for a misalignment magnitude greater than 12.7 mm.
Prabhu et al. (2006) carried out experimental investigations into the dowel misalignment effect on joint opening behaviour in order to produce guidelines for allowable dowel misalignment tolerances. The test specimen consisted of two concrete slabs measuring 1220 mm × 610 mm × 250 mm each and both were separated by a 3 mm thick steel plate to form the transverse joint with 32 mm diameter steel dowels connected through it. Each slab rested on a rigid steel plate supported by smooth rollers and two hydraulic jacks were used to push these slabs apart to open the joint by 25 mm (1 in.). Their investigations involved one, two, three and five steel dowel bars with different misalignment magnitudes (0, 6.35 mm, 12.7 mm, 19 mm, and 25.4 mm per half length of the dowel bar), different misalignment types (uniform and non-uniform) and different dowel bar orientations (vertical, horizontal, and combined). The results showed that all joints in a concrete pavement are opened when the load per dowel exceeds 5 to 7 kN. These also showed that for all misalignment types, the pull-out load increases with an increase in the magnitude and non-uniformity of the dowel misalignments. The results indicated that there were occurrences of concrete spalling and observable cracks especially for higher misalignment magnitudes and non-uniform misalignment. An increase in the number of misaligned dowels across the joint increased the severity of structural distress. However, in their test arrangement the slabs were fixed to the base, and any separation that may happen in reality was restricted and the effect of slab self-weight was minimal. For these reasons, there was insignificant difference in pull-out loads for vertical and horizontal misalignments.

Recently, (Hoegh and Khazanovich 2009) conducted an experimental study to investigate the permissible tolerance limits of dowel misalignment prescribed by the Federal Highway Administration (FHWA). The study also considered dowel-concrete friction, shear capacity and shear stiffness of misaligned dowel bars. The test consisted of a concrete beam (457 × 1200 × 203) mm which was fitted with four 38 mm round steel dowel bars (greased and ungreased) having different misalignment types and magnitudes. Each bar was 457 mm long; 229 mm of it was embedded in a concrete beam with different levels of misalignment. The test was designed to simulate a part of a concrete pavement slab having misaligned steel dowels and being subjected to axle load. This was carried out in two stages. In the first stage, the dowels were pulled-out individually in the longitudinal direction for 6.4 mm (0.25 inch) and the pull-out load
versus displacement was recorded. The second stage involved an evaluation of the shear capacity of the dowel bar by turning the beam on its side and applying direct shear on the outer dowels. The test also involved applying a repeated shear load of 13.3 kN to the outer dowels only at a frequency of 2 Hz for load cycles of 10,000-14,000. The experimental results suggested that the dowel misalignment has an insignificant effect on the pull-out load required to open the joint. However, for a higher misalignment magnitude, the results indicated a lower shear capacity for the misaligned dowel bar and a higher deterioration in the surrounding concrete. The results also clarified that the pull-out load of dowels without grease is about 200% of that for greased dowels. Since the test arrangement is similar to a direct pull-out test for individual dowels, no interaction among the misaligned dowel bars was possible. Hence the test arrangement was not representative of actual slab-openings in the field. The casting of the specimens in a vertical direction may have reduced the variations in shrinkage strains for different dowel orientations, which is an important factor in the development of bond strength. Their test arrangement excluded the possibility of additional bearing stress arising due to the constrained movement of the adjacent dowels.

2.3.1.2.2. Numerical studies

Over the past few decades, the dowelled-JPCP system has been modelled using FEM. One of the early examples of modelling is by Huang and Wang (1973), where the dowel bar was modelled by a shear spring directly attached to the two faces of a joint. Afterwards, Tabatabaie and Barenberg (1980) used beam elements to simulate the dowel bars, and the dowel-concrete interaction was represented by a vertical spring connecting the dowel bar to the concrete pavement. These models (Huang and Wang 1973; Tabatabaie and Barenberg 1980) ignored the modelling of embedded portions of the dowel bar by assuming it as a semi-infinite beam supported on an elastic foundation. A subsequent study modelled the dowel bar as three segments of a single flexural beam — two segments embedded in concrete slabs which were considered as an elastic medium and the middle segment as a standard beam (Nishizawa et al. 1989). A similar approach was adopted by Guo et al. (1995) in which the dowel segment across the joint was modelled by a shear flexural beam connecting two flexural beams embedded in concrete slabs. These studies, and the aforementioned studies in the DL section,
assumed the dowel bars to remain fully aligned across the joint. Therefore, any construction error relating to the dowel bars’ installation was not considered.

Riad et al. (2009) investigated the effect of a skewed joint on the performance of JPCP using 3D FEM. All the components of the dowelled-jointed concrete pavement were modelled using solid brick elements. The dowel-concrete interaction was modelled by a contact interface having a coefficient of friction of 0.05 at the free end of the dowel bar. A contact interface was also provided between the slab and the base layer with a coefficient of friction of 0.9. A 25-µm uniform gap was provided around the dowel bar as a representation of the initial DL. The results indicated that the skewing of the joint did not improve the JPCP performance, however the shear and tensile stress in the concrete surrounding the dowel bar was found to be greater for skew joints than for right angled joints. The increase in the stress level of the surrounding concrete was possibly due to the effect of the restriction of the slab movement during the expansion and contraction of the pavement, which is similar to the effect of dowel misalignment.

Over the last few years, a couple of FEM studies reported to have modelled misaligned dowels in JPCP. Khazanovich et al. (2001) carried out 2D and 3D FEM using ABAQUS software for single and multiple misaligned dowels. The 2D FEM was adopted to simulate a single misaligned dowel in the vertical or in the horizontal direction. For multiple dowel bars, 3D FEM was implemented by using solid brick elements representing the dowel bars and the concrete pavement with the dowel-concrete interaction modelled by contact elements. In order to develop contact pressure between the dowel bars and the concrete pavement so that they could simulate the effect of dowel misalignment, the analysis was conducted in two stages. In the first stage, only the dowel temperature was increased to develop contact pressure between the dowel bars and the concrete pavement. In the second stage, the slab temperature was decreased to get the required joint opening while the dowel temperature was kept constant. The authors modelled the concrete pavement and dowel bars as linear-elastic material. Their results revealed that the pull-out load increased with an increase in the dowel misalignment level.

A similar approach to that of Khazanovich et al. (2001) was recently used (Saxena et al. 2011, 2009) with the modification of using concrete damaged plasticity (CDP) to model
the concrete pavement surrounding the dowel bars. The results showed that the pull-out load and pavement distresses are more affected by the dowel-concrete friction than by the dowel misalignment. It also showed that an insignificant change in the pull-out load for the specimens with a single dowel was due to ignoring the effect of bearing stress produced by adjacent dowels during the pull-out test. These studies involved several assumptions and limitations (changing the dowel temperature, the coefficient of friction, the shear stress limit and the elastic slip), which are difficult to validate due to lack of appropriate experimental data. In these studies, the effect of the base layer and repeated wheel loads were not considered.

Leong et al. (2006) conducted a finite difference analysis using a programme called Fast Lagrangian Analysis of Continua 3D (FLAC3D) which had essentially been developed for geotechnical applications. Three misalignment cases were simulated: one misaligned dowel in a single plane, three misaligned dowels in a single plane and three misaligned dowels in the opposite direction (non-uniform misaligned dowels). The results showed that the pull-out load for the specimens having three misaligned dowels is greater than that for single dowel. It was also demonstrated that for the same number of misaligned dowels there is no significant difference in the pull-out load for different misalignment types. The same pull-out load for different misalignment types (horizontal, vertical, uniform and non-uniform) points out that the programme could not capture the effect of dowel misalignment on the joint-lockup in JPCP.

Prabhu et al. (2009, 2007) developed a 3D FE model to simulate dowel misalignment and its effect on the performance of JPCP using ABAQUS software. The dowel bars and concrete pavement were modelled using solid brick elements while the dowel-concrete interaction was simulated by using surface-to-surface contact and nonlinear spring elements. They used the CDP model to represent the concrete pavement material and an elastic material model for the steel dowel bars. Since these studies were simulation of the laboratory experiment that was conducted by the authors, the base layer was not modelled and any movement in the vertical direction was constrained. They carried out their analysis in two steps. Step one involved applying the slab’s self-weight as body force to develop the contact pressure between the dowel bars and the concrete pavement while in the second step, each side of the joint was opened by 6 mm as a representation of pavement contraction. Several misalignment types, dowel
orientations and dowel numbers were considered and two different coefficients of friction were adopted for each analysis case as idealistic and realistic values according to the authors. The results showed an increase in the pull-out load and joint distress with an increase in magnitude and in the non-uniformity of dowel misalignment. However, their results were affected by ignoring the underlying pavement layers. Additionally, their assumption of the damage criterion being the critical stress/strain occurring in a single element may have been affected by stress localisation due to irregular meshing. Consequently, a comparison between different misalignment cases and the establishing of dowel misalignment tolerances according to this criterion is difficult.

2.4. Alternative Materials for Steel Dowel Bars

Epoxy-coated steel dowel bars are the most conventional type of dowel bars that are used as load transfer devices across the joints of JPCP. These steel bars often get corroded which creates dowel looseness (DL) due to initiation of a non-uniform flaky layer around the dowel bar and freezing of the dowel (Mancio et al. 2008). As a result, high bearing stress is induced in the concrete surrounding the dowels owing to repeated traffic load. The expansion and freezing of steel dowels due to corrosion increases surface irregularities and generates a substantial amount of locked-up stresses. The combined effect of the above problems causes a rapid deterioration of pavements. The GFRP dowel bars, on the other hand, are made of a corrosion-proof material. Their surface is highly smooth and unlike steel they do not require greasing to lower the bond with concrete. The slab movements are less restricted due to the minimal bond that exists between the GFRP bars and concrete, resulting in significantly lower locked-up stresses.

Although GFRP dowels are approximately 50% higher in cost than the steel dowels (Bian 2009), the cost of their maintenance over the long term, their transportation costs and the cost of installation are less. Therefore, they can be a practical alternative to epoxy-coated steel dowels.

Many studies have been conducted to address the pavement distresses and improve the performance of JPCP over its service life. These improvements involve the investigation of various materials and shapes as alternatives to the conventional rounded
steel dowel bars. Although alternative shapes such as the elliptical dowel bar may improve the LTE and reduce the bearing stress on the concrete surrounding the dowel bars (Porter et al. 2006), the corrosion of dowel bars still remains a crucial problem.

Investigations into using GFRP bars as dowel bars in JPCP started in the early 1990s. A laboratory fatigue test on full scale pavement slabs fitted with GFRP dowels was conducted by Porter et al. (1993) up to 10 million load cycles. They observed the possibility of using GFRP dowels of 44.45 mm (1.75 in) diameter spaced at 200 mm to obtain a response similar to epoxy-coated steel dowel bars of 38 mm (1.5 in) spaced at 305 mm.

Other laboratory tests carried out by Porter et al. (2001) involved determination of the material properties of GFRP dowels, and undertaking direct shear tests and fatigue tests for a single dowel across the joint. These tests were conducted on aged and un-aged specimens to show the long term performance of GFRP dowel bars. The tests also involved two full-scale pavement specimens supported by a steel-beam base. Each specimen consisted of two concrete slabs of 1830 mm × 1830 mm × 300 mm dimensions, and was connected by 38 mm GFRP dowels across a 3 mm full depth joint. The GFRP dowel bars were spaced at 305 mm for the first specimen and at 152.5 mm for the second specimen. A 40 kN load was applied sinusoidally on both sides of the joint up to 5 million cycles and the deflections of the slabs were measured under quasi-static conditions at set intervals during the cyclic tests. The results showed that the 38 mm GFRP dowels spaced at 152.5 mm performed equally well to those of steel dowels of the same diameter spaced at 305 mm.

Eddie et al. (2001) conducted experimental investigations and a field evaluation into the performance of 38 mm diameter GFRP dowels compared to 32 mm epoxy-coated steel dowel bars. The experimental programme involved three phases. Phase one involved a static load test for slabs resting on a weak subgrade which was represented by steel springs with a stiffness of 3.6 MN/m^3. Phase two involved replacing the weak subgrade by a stiff subgrade of 300 mm thick compacted limestone with a stiffness of 133.3 MN/m^3. Phase three involved applying a cyclic load varying between 20-130 kN and at a frequency of 6 Hz until a total of 1 million load cycles was reached. The results of the static and cyclic tests revealed that the 38 mm GFRP dowel bars have a comparable
response to that of the 32 mm diameter epoxy-coated steel dowels. A field evaluation for GFRP dowels in an actual pavement was undertaken 8 months after its construction using FWD. The results showed that the LTE for the 38 mm GFRP dowel bars is similar to that of 32 mm diameter epoxy-coated steel dowel bars.

Shalaby and Murison (2001) carried out laboratory tests on GFRP dowels as an alternative material to steel dowels. Their tests involved a double shear test and a four points bending test for 32 mm diameter epoxy-coated steel dowels, 38 mm GFRP dowels and a 60 mm Concrete Filled CF-GFRP tube. The study also involved a field evaluation using GFRP dowels compared with steel dowels. The results showed that the GFRP dowels can achieve the same level of performance as steel dowels if a higher diameter is used for the GFRP dowels. The results also indicated that an optimal performance for the GFRP dowels can be obtained by an investigation into the orientation of the fibre along the GFRP dowels.

Murison et al. (2005) carried out an investigation into four types of dowels: 38 mm epoxy-coated steel, 38 mm GFRP, 50.8 mm and 63.5 mm CF-GFRP tubes. The test consisted of one slab having dimensions of 610 mm × 610 mm × 254 mm, with the load applied directly to the dowel bar. The test sequence involved two phases: the first phase included applying 1 million cycles of 12 kN load and the dowel bar deflections were recorded at 5 different positions along the embedded length of the dowel bar under a quasi-static load after each set of 250,000 load cycles. The second phase involved loading the dowels monotonically until failure of the concrete. Strain gauges were closely spaced in the concrete underneath the dowel bar. The results showed that, for a similar diameter, the GFRP dowels exhibited a higher deflection than the steel dowels due to a weakness in the transverse direction. The 63.5 mm CF-GFRP tube showed less deflection when compared with the other types due to its large cross-sectional area. However, this size of dowel may be restricted by the slab thickness requirement in the actual pavement. The authors themselves have reported that the strain gauge readings were not reliable. The non-uniformity of strain beneath the dowel and possible damage to the strain gauges used in the region underneath the dowel were the reasons for the results’ unreliability. The study also did not show the variation in relative deflection due to using GFRP dowels which is the most important factor in joints’ evaluation (Porter et al. 2001) and did not compare the deflection values for similar rigidity levels. Also, a
direct load was applied to the GFRP dowel that may have caused more localized stress in the dowel and splitting of the fibre and matrix, making it difficult to check the failure load.

Duan-Yi et al. (2006) conducted laboratory tests and analytical investigations on two types of GFRP tubes both having an outside diameter of about 60 mm and an inside diameter of about 50 mm and grouted with concrete having a 45 MPa compressive strength. The tests involved a double shear test and three bending tests. The results indicated that lower deflection and bearing stress could be achieved using these dowels when compared with 30 mm diameter epoxy-coated steel dowels. However, the size of these tubes may be restricted by the slab thickness requirement in real pavements. Another factor was the separation or degradation of the concrete inside these tubes which can be considered as a drawback for these dowels.

Porter and Pierson (2007) carried out experimental tests on six highway dowel types of different materials (steel, stainless steel and GFRP), different shapes (round and elliptical), different sizes and different joint widths (0, 3.2 mm and 12.7 mm). The experiments were conducted using the modified AASHTO T253 test (Porter et al. 2001) which is a modified test of the original AASHTO T253 test to determine the modulus of dowel support, $k_o$ (AASHTO 1993). Strain gauges were placed on the embedded length of some of the tested dowels to show the bending moment of the dowel bars. The measured data were compared with the theoretical model of Friberg (1940) for a dowel bar of semi-infinite length supported on an elastic foundation. The results showed that for similar dowel size, GFRP dowels produced more deflection than steel dowels. Also, elliptical shaped dowels reduced the dowel bearing stress as compared with rounded shaped ones. However, the results showed that for all dowel types and shapes, a 22.25 kN load can be transferred by each of these dowels before reaching the allowable stress as determined by the ACI Committee 325 (1956). It was observed that the measured bending strain was lower than that estimated by the Timoshenko model. These tests did not incorporate a cyclic load test to assess the long term performance of elliptical dowels and GFRP dowels.

Vijay et al. (2009) carried out experimental tests, a field evaluation and a theoretical investigation of the GFRP dowels’ response in JPCP. The experimental tests involved
two different types of dowel bars (steel and GFRP), two dowel diameters (25 mm and 38 mm), two different spacing (305 mm and 152.5 mm) and two slab thicknesses (279 mm and 305 mm). The contraction joint was a saw-cut joint with a maximum width of 6.35 mm which was constructed in the middle of a slab that was 3.05 m long and 0.305 m in width. This slab was supported by a compacted aggregate inside a wooden frame in which the modulus of subgrade reaction was determined from a plate bearing test. The performances of the dowel bars were measured by testing slabs for both a static and a cyclic load of up to 5 million cycles. The results showed that GFRP dowels could provide an adequate LTE (more than 75%) according to the AASHTO (1993) guide for most of the tested specimens. In terms of the RD, the results demonstrated that a smaller RD and a higher LTE can be obtained from GFRP dowels when spaced at 152.5 mm when compared with that of steel dowels positioned at 305 mm. The results also showed that a lower embedded length was required for the GFRP dowels compared with steel dowel bars. The field evaluation agreed with the experimental results.

Robert et al. (2010) conducted a physical, mechanical and durability evaluation of GFRP dowels in terms of their usage as dowel bars in JPCP. The investigation involved two types of matrix materials — vinlyester and polyester. The durability of the GFRP dowels based on each of these matrix materials was evaluated by testing embedded GFRP dowels in concrete while being immersed in water at a temperature of 60°C for 75 days to replicate the aging process. The shear strength and the flexural modulus were compared with reference un-aged samples. The results indicated that the GFRP dowels based on a vinlyester matrix have a higher shear strength and flexural modulus than those based on a polyester matrix. It also showed that the aging process produced an insignificant effect on the shear strength and on the flexural modulus of GFRP dowels for both matrix types; consequently a good long term performance can be expected for GFRP dowels in JPCP.

Löfsjögård (2005) carried out laboratory tests to investigate the effect of the dowel bars material (steel and GFRP), different dowel diameters (17 mm and 25 mm for steel dowels, and 19 mm and 25 mm for GFRP dowels) and different coating materials (polythene and bitumen for steel dowels, and uncoated for GFRP dowels) on the bond strength between dowel bar and concrete pavement. Each dowel was drawn out and
pushed back for four cycles for 1.5 mm; then the dowel was finally pulled and pushed back for 5 mm. The maximum average pull-out loads in the first pull-out cycle for similar dowel sizes were 14.9 kN and 5.4 kN for the steel and the GFRP dowels respectively. The result also showed that with the dowels of the same material and under the same conditions, the pull-out load increased with an increase in the dowel size.

Although few studies had reported that the GFRP bars are highly water absorbent (Morii et al. 1994; Nishizaki and Meiarashi 2002) which may cause increase in bar diameter and degradation in mechanical properties, experimental tests that had been conducted on GFRP dowels taken from a pavement section constructed by the Ohio Department of Transportation and Civil Engineering (after 15 years of field service) showed that GFRP dowel had been virtually unaffected. Also, the tests showed that there was no significant decrease in mechanical properties of these bars (Busel 2000).

### 2.5. Summary of the Literature Review

This chapter presents all the major studies that were carried out to investigate the performance of dowel bars in the transverse joints of JPCP. These studies involved theoretical and numerical investigations into the load transfer mechanisms within JPCP. The chapter also includes details on numerical and experimental investigations into DL and dowel misalignment which are two important factors that cause a decrease in LTE. Finally, investigations into the use of GFRP dowels as an alternative corrosion-proof material instead of the conventional epoxy-coated steel dowel bars are also presented.

From the review of previous studies, it can be summarised that the bearing stress at the dowel-concrete interface is an important aspect of the dowel bar design. Excessive bearing stress can cause crushing of concrete (DL) which leads to a decrease in load transfer across the joint. Consequently, more load needs to be absorbed by the edge of the loaded slab and the subgrade beneath it. This, in turn, significantly affects the pavement’s lifespan by causing joint distresses such as spalling, faulting and pumping. The review has also shown that the load-deflection response of the dowel bars in transverse joints of JPCP needs further investigations for different pavement parameters, especially for the GFRP dowels, in order to ensure that these dowels are
adequate as load transfer devices. The dowel looseness (DL) was investigated as an action of the enlargement of the dowel bars sockets (deterioration of concrete surrounding the dowel) due to a repetitive wheel load only. The studies in the literature showed that DL significantly affects the amount of load transfer across the joints. However, DL was presented as an initial gap at the dowel-concrete interface, and many different sizes and shapes for the DL were assumed. These assumptions may be significantly different to the actual behaviour, which needs verification. Therefore, the visualization from FEM for possible concrete deterioration at the dowel-concrete interface can provide a good indication of the DL effect on the performance of dowel bars in JPCP.

In terms of the dowel misalignment studies, there are some limitations and shortcomings in the pull-out load investigations such as in the base layer representation and in the assessment of damaged concrete volume. These limitations need to be addressed by further investigations. So far there is no study to-date on the combined effect of dowel misalignment and DL due to repeated wheel loads on LTE and RD. Additionally, there is no study on using GFRP dowels to minimize joint lockup and the associated distress in the presence of dowel misalignment.

Regarding the use of GFRP dowel bars in the transverse joints of JPCP, in addition to what has been mentioned in the earlier sections, all previous studies dealt with contraction joints only. The joint width had been 3.17 mm (1/8 in.), except for Vijay et al.(2009). There is no study to-date on using GFRP dowels in expansion joints. The behaviour of GFRP dowel bars can be affected significantly by the joint width. Moreover, the previous studies have not provided sufficiently detailed design guidelines for GFRP dowels. They have been limited in the sense that they investigated few parameters in isolation, and did not incorporate all possible combinations of dowel-concrete pavement system parameters. For example, parameters such as pavement thickness and the spacing between the bars for different joint widths especially for expansion joints (wide joints) have not been investigated before.
CHAPTER THREE
EXPERIMENTAL INVESTIGATION OF THE LOAD-DEFLECTION RESPONSE OF GFRP DOWELS

3.1. Introduction

This chapter presents an experimental investigation into the load-deflection response of the GFRP and steel dowel bars at the transverse joints of the Jointed Plain Concrete Pavement (JPCP). Details of the test methodology, test parameters and the properties of the materials involved are described in this chapter. The experimental results of the deflection response of the GFRP dowels were compared with that of the epoxy-coated steel dowels. Also, a comparison was conducted between the experimental results of both GFRP and steel dowel bars with the analytical procedure of Timoshenko (1925).

3.2. Test Methodology

The test consisted of examining one dowel bar across a transverse joint between two concrete blocks as a simple representation of the actual pavement. The dimensions of the blocks were determined from the standard spacing and slab depths according to the American Association of State Highway and Transportation Officials (AASHTO) guide (AASHTO 1993) and the UK Highways Agency guidelines (UK Highway Agency 2009). The dimensions of the loaded and the reacting blocks were 300 mm × 300 mm × 250 mm and 450 mm × 300 mm × 250 mm respectively. The width of the blocks was selected to represent a 300 mm spacing of the dowel bars while the additional length of the reacting block was used to fix the block to the base of the testing machine.
Two different sizes of dowel bars were used and they were 25 mm for the epoxy-coated steel dowel bars and 38 mm for the GFRP dowel bars. A larger size for the GFRP dowels was chosen to get an approximately similar flexural rigidity ($EI$) value to that of the steel dowels. The length was 458 mm for both types of bars as recommended by the AASHTO (1993) guide. A light lubricant was used to debond the embedded steel dowel bars from the loaded block, whereas the GFRP dowel bars were used without any lubricant or greasing.

The main aim of this investigation was to evaluate the deflection response of the GFRP dowel inside the concrete pavement and the stress at the joint face beneath the dowel bar. Therefore, the subgrade layer was omitted and its effect was included within the modulus of dowel support ($k_0$) using the beam on elastic foundation approach of Timoshenko (Timoshenko and Lessells 1925). The reacting block rested on the steel base of the testing machine while a 25 mm gap was introduced beneath the loaded block to allow for the deflection of the dowel bar and to transfer the load to the reacting side. The loaded block was loaded monotonically using an L-shaped frame attached to the block as shown in Figure 3.1. A line load was applied on the steel frame along the centreline of the joint.

![Figure 3.1. Test setup: (a) Complete test setup; (b) Schematic for the test setup](image)

Figure 3.1. Test setup: (a) Complete test setup; (b) Schematic for the test setup
The reacting block was held down to the base of the testing machine by one threaded bar through a hole in the concrete block. Twisting of the loaded block was prevented by clamping both sides of it to the reacting block. Also, two load cells were placed at the outer edge of the loaded block to measure the load transferred to the base of the machine and to prevent any initial backward tilting as shown in Figure 3.1. At the initial stage of the test, the load cells were in full contact with the loaded block, and provided the reaction force data. At about 10% of the applied load, the contact between the load cells and the blocks was lost due to tilting, and all the applied load was being transferred to the reacting block through the dowel bar.

The test arrangement is similar to the testing of steel dowels as found in literature (Mannava et al. 1999). This arrangement was chosen as it represents the dowel shear action similar to the field conditions. Moreover, there is no standard test for the dowel bar deflection inside a concrete pavement and there are no limitations or requirements for the magnitude of relative deflection (RD) of JPCP in the AASHTO (1993) guide. The AASHTO (1993) guide assumes that the RD between the pavements will be taken into account by considering the LTE. From the test arrangement it is plausible that the loaded block will have a finite rotation introducing bending in the dowel bar. A back-calculation from the LVDT readings at the face indicates that this may have affected the dowel displacement at the most by 3%. In the field, small rotation of the loaded side is also likely due to load concentration at the joint face, and curling of slabs as a result of temperature or moisture gradient.

The surface deflection of the dowel bar was measured using LVDTs through the 7 mm wide and 90 mm long slots in the reacting block. The deflection of the dowel was measured at five positions of the embedded length of the dowel bar at a distance of 80 mm from the joint face. Previous works (Mannava et al. 1999; Murison et al. 2005) had suggested that the deflection beyond this distance was negligible. The LVDTs were spaced at the distances of 0, 20 mm, 35 mm, 50 mm and 80 mm from the joint face.

A special frame was fabricated to hold the LVDT sensors inside the slot. This frame consisted of a plate with specific holes to hold the sensors; these holes were carefully drilled to give a minimum clearance so that the sensors remain vertical during the test. The plate was attached to the top surface of the reacting block using adhesives.
accuracy LVDTs were used to measure the deflection (accuracy ±0.001 mm). The LVDTs were calibrated using a long barrel micrometer that had an accuracy of ±0.002 mm. A trial specimen test was conducted to check the overall setup and to ensure that the LVDTs were giving satisfactory results.

Each specimen was cast in two stages: the first stage involved casting the reacting block with the dowel at the centre to ensure that the dowel was fixed at the appropriate position. The second stage involved removing the mould of the reacting block and casting the loaded block the next day. The face of the mould of the loaded side was designed as two halves to be able to slide from the sides after casting (see Figure 3.2).

![Figure 3.2. Specimen during casting](image)

### 3.3. Material properties

#### 3.3.1. Dowel bars

The flexural and shear properties of dowel bars were required for this investigation. These properties are well known for the epoxy-coated steel dowels, being an isotropic material. Since GFRP dowels are a non-isotropic material, using them as a load transfer
device for the JPCP, required direction-specific material properties at the design stage (numerical simulation).

3.3.1.1. Epoxy-coated steel dowel bars

The steel dowel bars used in this study were made of mild steel according to BS EN 13877-3 (2004) which requires the minimum tensile strength to be used as a dowel to be 250 MPa. The tensile strengths were tested according to BS EN 10002-1 (2001) as shown in Figure 3.3 (a). The average yield and ultimate tensile strength obtained from the tests were 275 MPa and 460 MPa respectively (see Figure 3.3 (b)). Although the dowel bars used in the concrete pavement are normally designed to be in the elastic range, the complete stress-strain behaviour was necessary for subsequent numerical simulation of dowel shear tests. The steel dowel bars were supplied by the manufacturer in a coated state (epoxy-coated) with a minimum coating thickness of 0.3 mm, according to the requirements of BS EN ISO 7253 (2001).

![Figure 3.3. Tensile strength test for steel dowel bars: (a) Test setup; (b) Stress-strain curve](image)

3.3.1.2. GFRP dowel bars

According to the manufacturer, Hughes Brothers Inc. (Nebraska, USA), the GFRP dowel bars used in the current study had been produced by the pultrusion process. It consists of the pultrusion of continuous ECR-glass filament with vinylester resin. This
resin matrix offers high resistance against corrosion, alkaline attack, acids and solvent (Barbero 1999) which are important aspects in the dowel bars’ longevity. The glass fibres are arranged in a unidirectional way, parallel with the longitudinal direction. In Table 3.1, the mechanical properties of the GFRP dowel bars are presented as supplied by the manufacturer.

The predominant mechanism of load transfer by dowels at the transverse joints of the JPCP is shear. GFRP dowels have higher strength and stiffness in the longitudinal direction (parallel with the direction of fibres). The tensile strength of the GFRP dowels in the longitudinal direction is higher than that of the steel bars, whereas the transverse properties are much weaker. The shear strength of GFRP is a critical issue in using it as a dowel bar, considering the weakness it has in the transverse direction.

<table>
<thead>
<tr>
<th>Diameter (mm)</th>
<th>38</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear strength in bending (kN) ASTM D4475-96</td>
<td>95.7</td>
</tr>
<tr>
<td>*Shear strength in direct shear (kN) ACI 440.3R-04-B.4</td>
<td>176.1</td>
</tr>
<tr>
<td>Shear stress (MPa)</td>
<td>151.7</td>
</tr>
<tr>
<td>Longitudinal modulus of elasticity $E_1$ (GPa)</td>
<td>40.8</td>
</tr>
<tr>
<td>Transverse modulus of elasticity $E_2$ (GPa)</td>
<td>10</td>
</tr>
<tr>
<td>% Weight fraction of fibre</td>
<td>70-75</td>
</tr>
<tr>
<td>Specific gravity of dowel</td>
<td>1.9-2</td>
</tr>
</tbody>
</table>

*Shear strength for single side

A double shear test was conducted for the GFRP dowels used in the current investigation according to BS ISO 10406-1 ISO/DIS 10406-1 (2007) (see Figure 3.4). Three samples were tested and the average shear strength of a single side was 193 kN (see Figure 3.5, and Equation (3.1)), which is about 10% higher than the shear strength listed by the manufacturer as shown in Table 3.1. The in-plane shear modulus ($G_{12}$) was calculated from this test using Equation (2.13) after subtracting the deflection due to bending from the total displacement (by assuming the middle part of the dowel as a beam with two fixed ends and loaded with uniformly distributed load). The value of in-plane shear modulus ($G_{12}$) is required for the numerical simulations presented in Chapters Six, Seven and Eight. The initial stiffness of all three bars was smaller,
because the inner diameter of the test piece holders (see Figure 3.4) holding the dowel have not been exactly the same as the outer diameter of the dowel bars. In fact a small clearance was made for ease in inserting the dowels considering that there may have been a small variation in the GFRP dowels’ diameter. At higher loads, contact areas between the dowel outer surface and the test piece holders’ inner surface increased, thereby raising the stiffness.

![Diagram](image_url)

**Figure 3.4. Double shear test set-up for GFRP dowels: (a) Schematic; (b) Photograph**

![Graph](image_url)

**Figure 3.5. Results of double shear test for GFRP dowels**

\[ \tau_{12} = \frac{P}{2A_d} \]  

(3.1)

where, \( P \): Applied Load; \( A_d \): Cross sectional area of dowel; \( \tau_{12} \): In-plane shear stress; \( G_{12} \): In-plane shear modulus.
3.3.2. Concrete

All the test specimens were cast in the concrete laboratory and the method of normal concrete mix design as outlined in Teychenne et al. (1997) was followed. Six cubes were cast for each specimen, three for the loaded block and three for the reacting block. The compressive strength for each block was measured for each sample by calculating the average compressive strength of these cubes on the day of the test, as shown in Table 3.2. Eurocode2 (EC2) was used to find the concrete properties according to the concrete grade (BS EN 1992-1-1 2004).

A special nomenclature or coding was used to define the test specimens as follows:

- G  GFRP dowels,
- S  epoxy-coated steel dowels,
- N  narrow joints (10 mm),
- W  wide joints (21 mm),
- H  higher concrete grade (28 MPa), and
- L  lower concrete grade (17 MPa).

Table 3.2. Experimental parameters

<table>
<thead>
<tr>
<th>Test code</th>
<th>Measured joint width (mm)</th>
<th>Nominal concrete strength (MPa)</th>
<th>Measured compressive strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>GNH</td>
<td>10.3</td>
<td>28</td>
<td>29.5</td>
</tr>
<tr>
<td>GWH</td>
<td>21.3</td>
<td>28</td>
<td>29</td>
</tr>
<tr>
<td>GWL</td>
<td>22.5</td>
<td>17</td>
<td>18</td>
</tr>
<tr>
<td>GNL</td>
<td>10.3</td>
<td>17</td>
<td>22.7</td>
</tr>
<tr>
<td>SNH</td>
<td>10.3</td>
<td>28</td>
<td>35.5</td>
</tr>
<tr>
<td>SWH</td>
<td>21.3</td>
<td>28</td>
<td>30.1</td>
</tr>
<tr>
<td>SWL</td>
<td>21.3</td>
<td>17</td>
<td>19.7</td>
</tr>
<tr>
<td>SNL</td>
<td>10.3</td>
<td>17</td>
<td>25.1</td>
</tr>
</tbody>
</table>

3.4. Test Parameters

Two parameters were considered in the test for both GFRP and steel dowel bars, which are the concrete compressive strength and the joint width. Two grades of concrete were selected: 28 MPa and 17 MPa. The selection of these concrete grades was made to obtain the distinct load-deflection response of the GFRP dowel bars that can be measured with certainty to demonstrate the effect of different concrete grades.
However, it should be mentioned that, a concrete grade of 17 MPa is not used in practice but it was selected to verify the results with data available in literature (Mannava et al. 1999).

Two joint widths were selected for the current investigation: 10 mm and 21 mm. These dimensions are typical for expansion joints in the Middle-Eastern countries like Iraq. Few studies have dealt with the behaviour of GFRP dowels at the expansion joints; therefore, the main concern of this study was to examine the deflection response of GFRP dowels at these joints. This may be a critical aspect in the design as the mechanical properties of the GFRP dowels in transverse directions are substantially lower compared to the steel dowels. Eight tests were carried out, four with GFRP dowel bars and four with epoxy-coated steel dowel bars according to the parameters shown in Table 3.2.

3.5. Discussion of results

3.5.1. Deflection response of GFRP dowel bars under load

The specimens included in the test were loaded monotonically until the failure of the specimens. The load-deflection readings for all the LVDTs inside the slot (deflection of the dowel bar surface) and the deflection of the loaded block were recorded during the test. Measurement of RD between the loaded and the reacting blocks is more important for GFRP dowels compared with the steel dowels. The measured RD between the loaded and the reacting blocks largely consisted of shear deformation of the GFRP dowels.

In order to obtain the deflection of the dowel bar analytically (theoretically) according to Timoshenko’s procedure, the value of the modulus of dowel support \( k_o \) is needed. It can be back-calculated from the deflection value of the dowel bar at the joint face at the end of the elastic range as described in Chapter Two of this study (Equations (2.8) and (2.9)). In the current study, a load level of 16 kN was used to compare the deflections of all the specimens. The numerical simulation (in Chapter Six of this study, Figure 6.2) exhibits elastic behaviour in the dowel-concrete interface up to this load level. The experimental results also confirmed this selection, as linear behaviour was observed until about 16 kN, even though this load is higher than the allowable load transfer by the
dowel bars according to AASHTO (1993). Similar observation was also found in the literature (Mannava et al. 1999; Murison et al. 2005; Porter and Pierson 2007). Table 3.3 gives a summary of the ultimate load carried by each specimen, the ultimate deflection, and the deflection at 16 kN load of the dowel bar at the joint face. The modulus of dowel support is also presented in this Table. The results in Table 3.3 show that the deflection of the dowel bars at the joint face varies according to the joint width and the concrete’s compressive strength. The results in Columns 3 and 4 of Table 3.3 show that the deflection at the ultimate load is about 4 to 9 times its value at the end of the elastic range due to an increase in crack-width at the post-peak zone.

<table>
<thead>
<tr>
<th>Test code</th>
<th>Ultimate load (kN)</th>
<th>Deflection at the Ultimate load (mm)</th>
<th>Deflection at 16 kN load (mm)</th>
<th>Modulus of dowel Support (MPa/mm)</th>
<th>$\beta$ (mm$^{-1}$)</th>
<th>$\beta L$</th>
</tr>
</thead>
<tbody>
<tr>
<td>GNH</td>
<td>57.7</td>
<td>0.46</td>
<td>0.06</td>
<td>492.8</td>
<td>0.0325</td>
<td>7.27</td>
</tr>
<tr>
<td>GWH</td>
<td>62</td>
<td>0.67</td>
<td>0.08</td>
<td>443.6</td>
<td>0.0317</td>
<td>6.92</td>
</tr>
<tr>
<td>GWL</td>
<td>59.7</td>
<td>1.17</td>
<td>0.18</td>
<td>143.3</td>
<td>0.0239</td>
<td>5.22</td>
</tr>
<tr>
<td>GNL</td>
<td>48.2</td>
<td>0.53</td>
<td>0.07</td>
<td>405.7</td>
<td>0.031</td>
<td>6.93</td>
</tr>
<tr>
<td>SNH</td>
<td>43.9</td>
<td>0.57</td>
<td>0.09</td>
<td>523.8</td>
<td>0.0304</td>
<td>6.81</td>
</tr>
<tr>
<td>SWH</td>
<td>40.1</td>
<td>0.46</td>
<td>0.12</td>
<td>404.6</td>
<td>0.0285</td>
<td>6.24</td>
</tr>
<tr>
<td>SWL</td>
<td>46.5</td>
<td>0.73</td>
<td>0.24</td>
<td>145</td>
<td>0.0221</td>
<td>4.94</td>
</tr>
<tr>
<td>SNL</td>
<td>41.6</td>
<td>0.71</td>
<td>0.14</td>
<td>231.3</td>
<td>0.0248</td>
<td>5.55</td>
</tr>
</tbody>
</table>

3.5.2. Comparison of GFRP dowels with epoxy-coated steel dowel bars

The comparison between GFRP dowels and epoxy-coated steel dowels was conducted in three stages. The first stage involved the comparison of the dowel bar deflection at the end of the elastic range (16 kN), at the ultimate load of the specimen having the least load carrying capacity (40 kN for SWH) as shown in Table 3.3 and with Timoshenko model at 16 kN load level. The 40 kN load is much greater than the load carried by dowel bars in an actual pavement. However, it was included in this study for better understanding of the behaviour of dowel bars at or near their ultimate load capacity. The second stage involved comparison of the RD of specimens containing GFRP dowels with corresponding specimens of steel dowels. Finally, the third stage compared the localization and distribution of stress at the dowel-concrete interface using numerical
simulation. The first and second stages are explained in this section while the third stage is explained later in Chapter Six.

The first stage involved two parts: the first part was a comparison of experimental results for both GFRP and steel dowels at 16 kN and 40 kN load levels. The second part compared both of them using Timoshenko’s analysis. According to Timoshenko’s analysis a dowel bar is assumed as a semi-infinite beam on an elastic foundation when $\beta L$ is greater than 2, which is applicable for the tested specimens as shown in Table 3.3.

The results of the first stage are presented in Figure 3.6. The experimental results show that a significant reduction can be achieved in a dowel bar deflection by using GFRP dowels with a flexural rigidity ($EI$) similar to that of steel dowels. The negative signs in the values along the vertical axes in Figure 3.6 and Figure 3.7 indicate downward displacements. To obtain similar $EI$, GFRP dowels with bigger diameters have been used. The increase in the contact area of the GFRP dowels due to higher diameter produces less stress concentration at the dowel-concrete interface which causes less deflection for the dowel bar at the face of the joint.

Comparisons using Timoshenko’s analysis (Timoshenko and Lessells 1925) are also presented in Figure 3.6 for both steel and GFRP dowels. The analysis generally predicts higher deflection for the steel dowel bars than the experimental values. The deflection calculations according to this method consider a single $k_o$ value that was back-calculated from the actual dowel bar deflection at the face of the joint. Higher deflections of the steel dowel bars at the face of the joint produce lower values of $k_o$ (see Equations (2.8), (2.9) and (2.10)). Using only the joint face deflection in the analytical approach may overestimate deflections in the interior parts of the dowel especially for specimens in Figure 3.6 (a) and Figure 3.6 (d). For these specimens, the narrow joint width produced higher stress localization and deflection for the dowel at the joint face.

For the GFRP dowels, lower bearing stress and deflection were obtained due to a larger contact area and a lower stiffness (as has already been stated before). Subsequently, a bigger $k_o$ value was achieved due to the lower joint face deflection of the dowel bar. The results presented in Figure 3.6 show that the deflected length is less for the GFRP dowels than that for the steel dowels in both experimental and analytical investigations. The analytical approach assumes that the dowels are supported uniformly along the
embedded length on springs of equal stiffness. This is far from reality and may explain
the difference in the deflected length profiles as shown in Figure 3.6.

Figure 3.6. Experimental and analytical results for the dowel deflection at 16 kN load for
both steel and GFRP dowels: (a) GNH & SNH; (b) GWH & SWH; (c) GWL & SWL; (d)
GNL & SNL

These results reveal the difficulty of adopting the analytical procedure of Timoshenko
for the design purposes of GFRP dowelled-JPCP, since the deflections are affected by
changes in the $k_o$ value which varies significantly with the geometry and the properties
of the pavement system. It also varies with the embedded length of the dowel bar and
the loading rate (even for the elastic range). The same difficulty also exists for the steel
dowels, since a wide range (81-407 MPa/mm) of $k_o$ has been specified by previous
researchers (Yoder and Witczak 1975). The bearing stress ($\sigma$) calculations for the
concrete beneath the dowel according to Timoshenko’s procedure related to $k_o$ (see
Equation (2.15)) and any inaccurate estimation for $k_o$ value could affect its value. The
present investigation demonstrates that the design procedure of the GFRP dowels should be based on an experimental or numerical analysis rather than the highly idealised analytical approach of Timoshenko.

Figure 3.7 shows the load-deflection response for both steel and GFRP dowels at 40 kN load level. The deflection increased for both steel and GFRP dowels when compared with their values at 16 kN but the 38 mm GFRP dowels still exhibit lower deflection values than the 25 mm steel dowels. A significant increase in dowel deflection in specimens SWH was observed compared with SNH (see Figure 3.7 (a) and Figure 3.7 (b)). This is because in addition to SWH having a wider joint, the specimen had a lower measured concrete compressive strength compared with SNH (see Table 3.2).

![Figure 3.7: Experimental results for the dowels deflections at 40 kN load for both steel and GFRP dowels](image)

The results in Figure 3.7 show approximately linear variation in displacement along the embedded length for the GFRP dowels. This indicates that the displacement was governed primarily due to shear. The steel dowels although had similar flexural stiffness to that of the GFRP dowels, the shear stiffness being about 15 times higher, caused the
deflection to be dictated by bending. The higher values of deflection for the steel dowels was due to concentrated bearing stress at the dowel-concrete interface which may have caused localised concrete damage, especially at the unconfined region of joint face. The GFRP dowels on the other hand produced more uniform bearing stress as the low transverse modulus (10 GPa compared to 200 GPa for steel) caused high cross-sectional deformation, which in turn increased the contact area and reduced the bearing stress. The low bearing stress produces high values of modulus of dowel support, which is equivalent in having high foundation stiffness for an equivalent beam on elastic foundation model. A perceived weakness of GFRP becomes helpful in this case. The effect of high bearing stress for the steel dowels was also pronounced due to the fact that the top of the dowels had a 7 mm × 90 mm gap that was required for placing the LVDTs. The stress concentration around the edges of this gap would have substantially more for steel dowels as they were of smaller diameters. For the specimen GWL the deflection pattern was not as described above due to the joint width being bigger increased flexural component, and also there may have been damage to the concrete at the face of the joint due to very low compressive strength.

The RD results of the loaded side to the reacting side are presented in Figure 3.8. The results indicate that for all specimens (except GWL) the relative deflections of GFRP dowels at a particular load level were equal to or slightly higher than that of the steel dowels, although the magnitudes of deflections at the joint face were less. This was due to the shear deformation of the GFRP dowels in the joint area. The shear deformation may not be captured correctly by using analytical expressions such as Equation (2.13), since the equation assumes that the half length of the dowel bar at the joint acts as a cantilever beam. However this is not the case in reality where the dowel deflects at the face of the joint.
For the specimen SWL shown in Figure 3.8 (c), the weaker concrete support may have contributed to a higher RD, because the low concrete compressive strength and a small contact area increases the bearing stress and the deflection of the dowel in this specimen. However, there is no big difference between the RD of 25 mm steel and 38 mm GFRP dowels in these specimens. For the specimens GNL, an initial deflection was observed, possibly due to voids at the upper surface of the dowel within the loaded block.

Although it is well known that the RD is a governing factor in dowel bars’ design, the results shown in Figure 3.8 confirm that it is more so for the GFRP dowels. Moreover, the results show that GFRP dowels compare favourably with the steel dowels of similar rigidity.
3.5.3. Effect of concrete compressive strength on the deflection response of GFRP dowels

As it has already been mentioned before in this experimental work; two different grades of concrete were selected: 17 MPa and 28 MPa. Comparisons between these specimens of different concrete grades for the same joint opening are illustrated in Figure 3.9 and Figure 3.10. These comparisons were made separately for all deflection measurements (Figure 3.9) to get a clearer understanding about the load-deflection response of GFRP dowel bars. In Figure 3.10, the comparisons were made at the end of the elastic range (16 kN) and at the ultimate load (40 kN) only to avoid repetition. It should be noted that the downward deflection in Figure 3.10 was marked by a minus sign to show the deformation of the dowel due to the load application.

Results in Figure 3.9 show a comparison of the deflection values of GFRP dowels in specimens GWH and GWL for two different concrete grades. The nominal joint width in both cases was 21 mm and the measured concrete compressive strength was 29 MPa and 18 MPa. High deflection was observed for the specimens having lower compressive strength (GWL). The increase in concrete compressive strength by 60% caused deflection to be decreased by 50%. For narrower joint widths, a difference in concrete strengths of 30% (see GNH and GNL in Figure 3.10), causes change in deflection for only about 13%.

Figure 3.9 (f) shows the RD between the loaded and the unloaded side. It can be observed that the concrete strengths do not have a significant effect in this case, which is primarily due to the joint width being wider in this case. In wider joints, the effect of shear deformation is predominant on the RD of the GFRP dowels.
Figure 3.9. Experimental load-deflection behaviour of GFRP dowels for two different concrete grades at the following distances from the joint face: (a) Zero; (b) 20 mm; (c) 35 mm; (d) 50 mm; (e) 80 mm; (f) relative deflection of loaded side.

Figure 3.10 presents the results of the deflection data for the GFRP dowels of the specimens GNH and GNL where the joint width is 10.3 mm and the concrete compressive strengths are 29.5 MPa and 22.7 MPa respectively. It can be seen that there
is only a small difference between the two specimens in the elastic range, as shown in Figure 3.10 (a), because for the smaller values of loads, concrete in both specimens offers approximately the same bearing resistance. With the increase of the load value beyond the elastic range, the deflection value for the specimens with weaker concrete (GNL) starts to increase more than GNH as shown in Figure 3.10 (b). A significant crushing and softening of the concrete beneath the bar may happen as predicted by the numerical simulation presented in Chapter Six of this study.

The displacement of the GFRP dowels diminished rapidly along the embedded length in specimens of higher compressive strength. The supporting concrete foundation for the embedded part of the dowel in this case remained in the elastic range due to a higher compressive strength. Consequently, it offered higher resistance to bearing stress induced by the dowel bar and these results were observed for both specimens with a higher concrete strength (GWH and GNH).

It can be concluded that the higher compressive strength of concrete does reduce dowel bars’ deflection, especially for the wider expansion joints.

![Figure 3.10. Experimental dowel deflection at: (a) 16 kN load (b) 40 kN load](image)

3.5.4. **Effect of joint width on the deflection response of GFRP dowels**

The experimental programme involved the investigation of two different widths of joints: 10 mm and 21 mm, which are typical sizes for expansion joints. There are no previous studies dealing with expansion joints. Figure 3.11 and Figure 3.12 illustrate the comparison between the experimental results for all specimens with GFRP dowels in
terms of different joint widths. Figure 3.11 shows the results of the comparison of the specimens GWL and GNL for different joint widths. They have a nominal compressive strength of 17 MPa. The narrower joint width of specimen GNL caused a reduction in the deflection of the embedded part of the dowel compared with specimen GWL. A higher RD was observed for specimen GNL at the beginning of the test due to initial deflection as shown in Figure 3.11 (f). The slope of the RD graph of specimen GNL increased rapidly after about 12% (6 kN) of ultimate load of specimen GNL (48.2 kN). The average slope of the RD graph of specimen GNL between those two load values (6 kN and 48.2 kN) is about 80% of that of specimen GWL, this shows that specimen GNL exhibited smaller RD.

Figure 3.12 shows the comparison between specimens GNH and GWH. They have a nominal concrete compressive strength of 28 MPa. A small difference in the dowel deflection at the joint face can be noticed between these specimens. The deflection values of the GFRP dowel at the joint face in specimen GNH are about 80% and 88% of that in specimen GWH at the end of the elastic range (Figure 3.12 (a)) and the ultimate load (Figure 3.12 (b)) respectively. Shorter deflected length can be observed for the specimen GNH with the narrower joint width compared with that of the dowel in specimen GWH.
Figure 3.11. Experimental load deflection behaviour of GFRP dowels for different joint opening (for a-f see caption of Figure 3.9)
Figure 3.11 and Figure 3.12 reveal that the deflected portion of the GFRP dowel increases with an increase in joint width due to dowel bending stress as well as the dowel bar deflection and the RD increases with increasing joint width.

3.6. Summary and Conclusions

The following conclusions can be obtained from the experimental and analytical (theoretical) investigations of the GFRP dowel bars behaviour at the transverse joints of JPCP:

- An experimental investigation was made to compare the load-deflection response of the GFRP and epoxy-coated steel dowels at different joint widths and for varying concrete compressive strengths.
- The experimental investigation showed that although 38 mm (1.5 in.) GFRP dowels have similar rigidity to 25 mm (1 in.) epoxy-coated steel dowels; the GFRP dowels generally produce a lower deflection.
- The deflection of the GFRP dowel was significantly affected by changing the concrete compressive strength and the joint widths.
- Between the two test parameters – concrete strength and joint width – the relative deflection was found to be more sensitive to the changes in joint width, especially for the GFRP bars.
CHAPTER FOUR
EXPERIMENTAL INVESTIGATION ON THE COMBINED EFFECTS OF DOWEL MISALIGNMENT AND WHEEL LOAD

4.1. Introduction

This chapter involves the experimental investigation for the combined effects of dowel misalignment and wheel load on the performance of steel and GFRP dowels in JPCP. It presents the plan, setup, and test sequence of the experimental programme. Detailed discussions of the experimental results for the effects of dowel misalignment on the pull-out load, joint lockup, dowel looseness (DL), and load transfer efficiency (LTE) are described. The discussions involve a comparison of DL and the reduction in LTE due to dowel misalignment with that due to the cyclic load effect.

This chapter also presents results of a standard pull-out test to show the effect of changing the distance from the exposure surface (due to dowel misalignment) on the pull-out load, and the usefulness of the pull-out test to investigate the dowel misalignment (§4.6).

4.2. Experimental Plan

The experimental plan involved applying a 40 kN quasi-static load before and after opening and closing of the joint to show the effect of dowel misalignment on LTE and DL. Subsequently, a cyclic load of 40 kN magnitude and one million cycles was applied to compare LTE and DL due to dowel misalignment.
In order to clarify the effect of dowel misalignment on the joint performance, the following parameters were experimentally investigated.

- Dowel type and size.
- Dowel misalignment types.
- Dowel misalignment magnitude.
- Number of dowel bars across the joint.
- Orientation of misaligned dowel bars.

### 4.2.1. Dowel type and size

Two different dowel types were involved in this investigation – the GFRP and the epoxy-coated steel dowel bars. Although steel dowel bars are more common in practice, there are several durability-related problems as mentioned in the earlier chapters. GFRP bars on the other hand have a smooth surface profile, low bond with concrete and they are corrosion-free. However GFRP has a lower Young’s modulus than steel. Hence higher diameter GFRP dowels (38 mm) were used to compare with the performance of the steel dowel bars (25 mm) of similar flexural rigidity.

### 4.2.2. Dowel misalignment types

Dowel misalignment can be classified into two categories that were explained in detail in Chapter Two (§2.3.1.2), translation and skew misalignment. The focus of this study is on the skew misalignments since they lead to more restraint and deterioration for pavement (Khazanovich et al. 2001; Prabhu et al. 2006). In addition to aligned dowel cases, three types of dowel misalignment were included in the current investigation — uniform misalignment, non-uniform misalignment and partial misalignment. Uniform misalignment type refers to parallel misaligned dowel bars, whereas non-uniform misalignment refers to the tilt of the dowels in an opposite direction. Partial misalignment refers to when only one of the dowels is misaligned while the other dowels are aligned (see Figure 2.8).

### 4.2.3. Dowel misalignment magnitude

Dowel misalignment magnitude refers to the magnitude of tilting per half length of the dowel bar. Three values of misalignment were chosen in this experimental
investigation: zero misalignment, 12.5 mm and 25 mm per half length of the dowel bar (see Table 4.1). The first value was chosen to represent a typical installation of aligned dowel bars at the transverse joints without any construction defect. The second value was adopted to represent misalignment magnitude more than the permissible limits according to most highways agencies (see Table 2.1). This selection was made to keep the distress surrounding the dowel bar localized without significant cracking during the opening of the joint by 12 mm (Prabhu et al. 2009) to allow for the evaluation of LTE under a cyclic load, after the slabs were pulled out and pushed back representing thermal movement. The third value of misalignment magnitude was adopted for only one specimen to simulate the possible failure of the specimens due to joint lockup. The opening of the joint by 12 mm represents the worst case scenario of pavement contraction, which is much greater than the typical movement of joints (6.4 mm) in JPCP (AASHTO 1993; Saxena et al. 2009).

4.2.4. Number of dowel bars across the joint

The number of dowel bars used in the current experimental programme was selected to show the effect of the interaction of the group of misaligned dowels on the load required to open the joint according to the sizes of the specimens and the testing machine. A total of two and three dowel bars were included in the specimens of dimensions (900 × 900 × 200) mm and (900 × 1200 × 200) mm respectively (see Table 4.1).

4.2.5. The Orientation of misaligned dowel bars

Several orientations for the dowel bars were adopted in the experimental investigation as follows: fully aligned dowels, vertically misaligned dowels and horizontally misaligned dowel bars. These orientations were adopted for specimens containing GFRP dowels and specimens containing epoxy-coated steel dowel bars.

4.3. Experimental setup

A total of eight specimens were tested in the current investigation. Six of these specimens were fitted with GFRP dowels and two were fitted with epoxy-coated steel dowels (see Table 4.1 for details). Each test specimen consisted of two concrete slabs connected by steel or GFRP dowel bars. A 3 mm smooth steel plate separated these two
slabs during the concrete casting and appropriate holes and slots were made in the separation plate to allow for the dowels to connect the slabs across the joint and also to facilitate the removal of the plate itself on the following day after casting. The dimensions of the slabs were 900 mm × 900 mm × 200 mm for specimens containing two dowel bars (see Figure 4.1) and 900 mm × 1200 mm × 200 mm for the specimens containing three dowel bars (see Figure 4.2). In these tests, two different types of dowel bars were used, one was a 25 mm diameter smooth and round epoxy-coated steel bar and the other a 38 mm diameter round GFRP bar. Both of these dowels had a length of 458 mm. These dowels were embedded for equal lengths of 227.5 mm on each side of the joint. The dimensions of the slabs and the steel dowels were selected according to the AASHTO (1993) guide. The diameter of the GFRP dowels was chosen according to the experimental investigation in Chapter Three, in which the 38 mm GFRP dowels exhibited similar deflection to that of the 25 mm steel dowel bars and this size also produces an equivalent flexural rigidity (EI) to that of the steel dowel bars. The tests were carried out for the slabs having two and three dowel bars to incorporate the effect of adjacent dowels as, it had been observed by Saxena et al. (2011) that testing of single dowel across the joint may not be adequate to quantify the effect of skew dowel misalignment.

The moulds were manufactured with 18 mm thick plywood except for the cut boxes’ sides that were of 36 mm thick plywood (see Figure 4.1). Cut boxes were made at both sides of the mould to make grooves to hold the hydraulic jacks while pushing the slab apart to create the required joint opening as shown in Figure 4.1. Small grooves were also made at the mould base and sides to fix the 3 mm thick separating plate. The slots in the separating plate beneath the dowels were closed using sticky plastic tape before pouring in the concrete. The plate was kept in the specimens until the concrete initially set and then it was removed the following day by pulling it in a vertical direction, the sticky tape underneath the dowels was later removed when the slab was being lifted for testing.
The dowel bars were held at their positions at the mid-depth of the specimen at a spacing of 300 mm before and during the casting process using 8 mm diameter smooth steel bars with 40 mm threaded ends. Both ends of each dowel bar were drilled and threaded to 40 mm. The steel bars were connected to the ends of the dowel bars and to holes at the sides of the mould (Figure 4.1). These bars were coated with grease so that they could be easily removed from the specimens after the initial setting of the concrete.

To achieve the required misalignment magnitude and orientation, simple trigonometric calculations were made to find the angles of inclinations and corresponding distances related to them (see Figure 4.3). These distances were measured and holes were drilled carefully on the sides of the mould. The misalignment magnitude and orientation were checked after installing the dowels and before the casting.

Figure 4.1. Specimen of two dowel bars (GA2)

The dowel bars were held at their positions at the mid-depth of the specimen at a spacing of 300 mm before and during the casting process using 8 mm diameter smooth steel bars with 40 mm threaded ends. Both ends of each dowel bar were drilled and threaded to 40 mm. The steel bars were connected to the ends of the dowel bars and to holes at the sides of the mould (Figure 4.1). These bars were coated with grease so that they could be easily removed from the specimens after the initial setting of the concrete.
Figure 4.2. Specimen of three dowel bars (GV3P2)

Figure 4.3. Dowel misalignment calculations (GH2N4)
In order to obtain a more realistic representation for an actual pavement resting on the ground, a supporting base was designed to represent the equivalent base for the underlying layers beneath the concrete pavement slabs. This base was developed to overcome the limitations and shortcomings of the previous dowel misalignment studies mentioned before. The supporting base consisted of four transverse beams simply supported on two longitudinal beams resting on a strong floor in the Heavy Structures Laboratory. The supporting beams were designed using a finite element analysis to represent a base layer with a subgrade reaction modulus (foundation stiffness) of 54 MN/m$^3$. Two sets of FE analysis were carried out for this purpose. Firstly, the slab was modelled with an elastic foundation of stiffness $k = 54$ MN/m$^3$, and secondly the slab was supported by four beams similar to the test arrangements as shown in Figure 4.4. The dimensions of the beams were selected so that the deflections of the slabs at eight LVDT positions (see Figure 4.5 and Figure 4.6) and the shear force and bending moment in the dowels would be of similar values from both sets of simulations. Subsequently, the second FE model was validated with the displacement values from experiments. The results of the finite element validations are presented in Chapter Five. The interior beams were UB 127 × 76 × 13 while the outer beams were UB 152 × 89 × 16; both beams had a clear length of 2000 mm. A thin layer of PTFE (Polytetrafluoroethylene) with a thickness of 1.5 mm was used underneath and on the sides of the slabs to minimize the effect of friction between the steel-beam base and the concrete slabs, in order to accurately measure the pull-out load. In practice, a separation membrane is placed between the slab and the base which reduces the friction (Maitra et al. 2009). Any in-plane rotation that may have occurred during the slabs’ movement was prevented by clamping the slabs as shown in Figure 4.6. This base arrangement allowed a slab-base separation during the slabs’ movement. The amount of separation that may have occurred depends on the orientation of the misaligned dowel bars.
### Table 4.1. Test matrix for the experimental specimens

<table>
<thead>
<tr>
<th>Specimen Dimensions (mm)</th>
<th>Number, diameter and material of dowel</th>
<th>Code of specimens</th>
<th>Misalignment type</th>
<th>Magnitude of misalignment per half length of dowel (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 slabs, each one (450 × 900 ×200)</td>
<td>2Ø38 mm GFRP</td>
<td>GA2</td>
<td>Aligned</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GH2N4</td>
<td>Horizontal</td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GV2N2</td>
<td>Vertical</td>
<td>12.5</td>
</tr>
<tr>
<td>2 slabs, each one (450 ×1200 ×200)</td>
<td>3Ø38 mm GFRP</td>
<td>GA3</td>
<td>Aligned</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GH3N2</td>
<td>Horizontal</td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GV3P2</td>
<td>Vertical</td>
<td>12.5</td>
</tr>
<tr>
<td>2 slabs, each one (450 × 900 ×200)</td>
<td>2Ø25 mm steel</td>
<td>SA2</td>
<td>Aligned</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SV2N2</td>
<td>Vertical</td>
<td>12.5</td>
</tr>
</tbody>
</table>

G H 2 N 4

- Misalignment magnitude: 1 → 6.25 mm, 2 → 12.5 mm, 3 → 19 mm, and 4 → 25 mm, per half of dowel length
- Misalignment type: U → Uniform, N → Non-uniform, and P → Partial, only one dowel misaligned
- Number of dowels across the joint
- Dowel bars orientation: A → Aligned, H → Horizontally misaligned, and V → Vertically misaligned
- Dowel bar type: G → GFRP, and S → Steel
Figure 4.4. Whole test setup (with steel-beam supporting base)
Figure 4.5. Sketch of a specimen with all instrumentations

Figure 4.6. Actual step with all instrumentations
The concrete mix design was carried out following the methods of a normal concrete mix design as outlined in Teychenne et al.(1997). The mix ratio (cement: fine aggregate: coarse aggregate) was 1:1.5:2.6 and a water/cement ratio of 0.55, and the maximum size of the coarse aggregate was 10 mm. This mix design was for a nominal concrete compressive strength of 28 MPa at 28 days. However, the average measured concrete compressive strength was about 33 MPa at 28 days. The concrete slabs used were cast on a large vibrating table in the concrete laboratory as shown in Figure 4.7. Three cubes were cast from the same concrete batch and these cubes were tested on the same day of the test to measure the actual concrete compressive strength (see Figure 4.8).

Figure 4.7. Concrete slabs casting

Figure 4.8. Concrete compressive strength test (cube test)
4.4. **Test Sequence**

The experimental programme was conducted in four major steps as follows:

- **Step one** involved a deflection measurement at the 8 positions shown in Figure 4.5 under a quasi-static wheel load of 40 kN. The 40 kN load is equivalent to a single axle load according to the AASHTO (1993) guide and BS 7533-12 (2006).

- **The second step** consisted of recording the load required to open the contraction joint by 12 mm when the slabs were pushed apart by two hydraulic jacks, and then pushed back to the original position. It should be noted that in expansion joints dowel caps are used to prevent the crushing of concrete at the end of the dowels during the pavement expansion.

- **The third step** involved repeating the deflection measurement similar to that taken in Step one under a quasi-static wheel load of 40 kN.

- **The fourth step** involved applying the 40 kN load for one million cycles to compare the effect of the cyclic wheel load on the GFRP dowels with that of the steel dowel bars and also the effects of dowel misalignment and joint lockup on the long term performance for both types of dowel.

The specimens were instrumented as shown in Figure 4.5 and Figure 4.6. The vertical deflections of the specimens were measured using LVDTs at 8 locations in two rows parallel to the dowel bar centre line as shown in Figure 4.5. The testing machine consisted of two controlled loading actuators held by transverse beams connected to the machine frame (see Figure 4.4). A thick steel plate 200 mm × 345 mm × 40 mm was attached to each actuator in order to apply the 40 kN load as a dual wheel load of a single equivalent axle load. The dimensions of the loaded area were selected to accommodate the actuator and to produce a tyre inflation pressure of 580 kPa as recommended by the American Concrete Pavement Association (ACPA) (Huang 1993).

The quasi-static load was applied on one side of the slab monotonically from 0-40 kN while the other side was clamped by the second actuator to prevent upward movement. The deflection data for the slab were recorded during the load application. The RD of the joint faces was calculated and then the LTE and the DL were computed from the deflection values.
During the pull-out step, the two halves of the specimens were pushed apart using two hydraulic jacks connected to one main pump to create a 12 mm additional joint opening. These jacks applied the load as pressure on the steel plates attached to the specimen sides in order to distribute the load on them with a loading rate of 0.5 mm/min. The load required to open the joint was recorded during this step. The load created by each jack was measured using load cells attached to the jack, while the joint opening was measured using two horizontal slider LVDTs attached to the top surface of the specimens (see Figure 4.5 and Figure 4.6). After measuring the load required to open the joint, the slabs were pushed back to maintain the initial joint opening by inserting a 3 mm thick separator plate which was used during casting. The RD of the joint was measured again under the same quasi-static load (40 kN) to show the effect of dowel misalignment and joint lockup on DL and LTE.

Since slab movement occurs immediately after concrete casting due to concrete shrinkage and changes in hydration temperature (before opening a road to traffic), this movement was applied in the test specimens before starting the cyclic load test. Although the expansion and contraction of slabs is a continuous process during a pavement’s life, in this experimental programme it was carried out only once at the start, since the objective of the study was to obtain a clear picture of the difference between the looseness due to slab movement and the looseness due to cyclic load.

The last step of the test included an evaluation of the long term performance of GFRP and steel dowel bars in JPCP under a cyclic load. The combined effect of DL due to slab movement (representation of thermal and shrinkage movement) and cyclic load (representation of repeated traffic load) on LTE was investigated in this step. The cyclic load was applied sinusoidally (see Figure 4.9) on both the slabs. The load amplitude ranged between 0.5-40.5 kN and the frequency of the load application was 4 Hz. The load from both actuators was out-of-phase to each other, i.e., when one actuator was providing a full load of 40.5 kN, the other actuator was providing 0.5 kN. The lower limit was set to 0.5 kN so that the actuator would restrain any upward movement (see Figure 4.9). The upward movement of the unloaded side of real slab would be counteracted by its self-weight, the effect of which was much lower for this scaled down test specimen. A rubber pad of 1 mm thickness was placed beneath each load actuator to prevent the crushing of concrete during the cyclic load. Deflections of the
concrete slabs were measured under a 40 kN quasi-static load after (i) 250,000 (ii) 500,000 (iii) 750,000 and (iv) 1 million cycles of the applied 40 kN load.

![Cyclic load on two slabs](image)

**Figure 4.9. Cyclic load on two slabs**

### 4.5. Results and Discussions

Firstly, the present set of experimental results provides a more realistic assessment of the effect of dowel misalignment. In previous experiments reported in the literature, the effect of the underlying layers has been completely ignored. In this study a base was designed to mimic the underlying layers. Secondly, the suitability of GFRP dowels was tested for whether they caused reduction in the required pull-out load and joint lockup. Finally, the results of the combined effect of dowel misalignment and of the traffic wheels load on DL and LTE are presented.

#### 4.5.1. Dowel Misalignment for GFRP and steel dowel bars

According to the standard pull-out load versus joint opening behaviour, the debonding of the dowel bar from a concrete pavement during slab movement consists of two parts: Part I - fully bonded and Part II - slipping region. Part I represents the fully bonded region from the start of the load application until the start of the initial slipping of the dowel bar from the concrete pavement. This region is governed by the bond stress between the dowel bar and the concrete pavement surrounding it, as shown by Equation (4.1). Part II represents the debonding region that starts after initial slipping. The slipping of the dowel bar starts when the dowel-concrete interface shear exceeds the
bond strength between the dowel bar and the surrounding concrete. Even for the dowel bars which are designed to have no bond with the concrete pavement, some bond exists due to chemical adhesion between the dowel bars and the concrete, mechanical interlocking due to the roughness of the dowel bar, and the frictional stress due to wearing or nicking of the dowel bar coating. The slipping region is affected significantly by the transverse interaction between the dowel bars and the concrete surrounding them. This interaction is governed by frictional or contact stress at the dowel-concrete interface and the bearing stress of the misaligned dowels on the concrete surrounding them.

\[ \tau = \frac{F}{\pi dL} \]  

(4.1)

where, \( \tau \): the bond stress (MPa); \( F \): pull-out load per dowel bar (N), \( d \): diameter of dowel (mm); \( L \): embedded length of dowel bar (mm).

4.5.1.1. Two misaligned dowels

Figure 4.10 to Figure 4.13 all illustrate the pull-out loads required to open the joint for the different cases of dowel misalignment for GFRP and epoxy-coated steel dowel bars. The general observations obtained from these figures are that the dowel misalignment produces an increase in pull-out load, and the slipping of the steel bars starts at a lower load compared with the GFRP dowels (see Figure 4.11 and Figure 4.13). The load required for the initial slipping is governed by the bond stress between the dowel bar and concrete. For similar bond stress the load required for the initial slipping increases with an increase in the dowel bar diameter as given by Equation (4.1). The initial slipping was calculated at a 0.05 mm joint opening, which was assumed as a threshold value for the start of slipping. This value was the smallest measured displacement achieved by the slider LVDTs with a high level of certainty in the experimental test. The initial slipping of the aligned steel dowel bars occurred at about 2.5 kN while for the GFRP it occurred at about 4 kN.
Figure 4.10. Effect of vertical misalignment for GFRP dowels- pull-out load versus joint opening for the specimens GA2 and GV2N2

Figure 4.11. Comparison of aligned GFRP and steel dowels – pull-out load versus joint opening for the specimens GA2 and SA2

Figure 4.12. Effect of vertical misalignment for steel dowels – pull-out load versus joint opening for the specimens SA2-steel and SV2N2
Figure 4.13. Comparison between horizontal and vertical misalignment – pull-out load versus joint opening for the specimens of SV2N2, GV2N2 and GH2N4

Figure 4.10 shows the effect of vertical misalignment for the GFRP dowels (GA2 and GV2N2). The results indicate that there is no significant difference between the specimens with aligned (GA2) and misaligned dowels (GV2N2). The maximum load required to open the joint for specimen GA2 is 8.35 kN while for the specimen GV2N2 it is 8.85 kN. Both specimens exhibited similar behaviour at the post-slipping stage. The load required to open the joint did not change significantly at the post-slipping stage for both specimens and it slightly decreased during the opening of the joint where the final values at the 12 mm joint opening were 7.1 kN and 7.5 kN for the specimens GA2 and GV2N2 respectively. This difference can be explained by the fact that, for misaligned dowels, part of the pulling force will be expended as bearing pressure onto the dowels, which will eventually require a higher pull-out force for the same amount of slip. Moreover, the drying shrinkage of the concrete varies with the depth from the exposure surface, as previous experimental studies have shown that higher values of shrinkage strain occur near the exposure surface (J. K. Kim and Lee 1998; Lim et al. 2009). For GV2N2, the tilted-down side would have experienced lower shrinkage strain, being farther from the exposure surface, which causes a reduction in contact pressure between the dowel bar and the concrete. Consequently, the load required to open the joint decreases in the case of vertical misalignment.

Figure 4.11 shows the pull-out load versus the joint opening for aligned GFRP and steel dowels. The graphs show that for the steel dowel bars (SA2) slip initiates at a lower load level than the GFRP dowels (GA2) due to a smaller circumferential area according
to the bond stress formula (Equation (4.1)). But the post-slip load (governed by friction) for steel dowels increases rapidly until it reaches a maximum value at a joint opening of around 4 mm which is 145% of that for the GFRP. Although the steel dowels are epoxy-coated, the surface irregularities and the chemical adhesion produce a higher frictional stress between the steel dowels and the concrete pavement. This load decays after about a 6 mm joint opening due to a complete break-down of the bond between the dowel and the concrete, and the starting of non-uniform frictional stress distribution that occurs after slipping due to the local crushing of concrete surrounding the dowel bar which can shear off the concrete inside the pits. Whereas, for the GFRP dowels in specimen GA2, a higher load was required to initiate slip since they had a larger diameter, but the load did not increase at the post-slip stage due to lower transverse interaction between the dowel and the concrete pavement. As mentioned before, this interaction is a function of the frictional stress between the dowel bar and the concrete pavement, which is very low for the GFRP dowels.

Figure 4.12 illustrates the comparison of the load-slip behaviour for aligned and vertically misaligned steel dowel bars (SA2 and SV2N2). Although, the misalignment of steel dowel bars and its effect on pull-out load and stress at the dowel-concrete interface had been investigated in literature, the combined effect of wheel load, dowel misalignment and the long-term performance of the dowel-joint system have not yet been explored. Moreover, most of the experimental programme in literature did not incorporate a realistic base layer. Therefore, an understanding of the effect of dowel misalignment is somewhat incomplete. The current test programme was carried out with the aim of providing a clearer picture of the steel dowel-concrete slab interaction that is to be used to compare the performance of the GFRP dowels. The results showed a significant increase in the average load per dowel required to open the joints of vertically non-uniform misaligned steel dowels (SV2N2) when compared with aligned dowels (SA2). The maximum load observed for SV2N2 was about 14.5 kN while for the SA2 it was about 10.8 kN. The reduction in pull-out load per dowel with joint opening was greater for the specimen SA2 than for the specimen SV2N2. In the case of the aligned dowel bar, pull-out load is primarily affected by frictional stress, whereas bearing stress is also significant for the misaligned dowel. By comparing the results with GFRP dowels as shown in Figure 4.10, it is revealed that the effect of vertical
misalignment is more significant for steel dowels due to a higher coefficient of friction and the different surface texture.

Figure 4.13 presents the comparisons of the results of specimens with non-uniform vertical misalignment (SV2N2 and GV2N2) for both steel and GFRP dowels, and for non-uniform horizontal misalignment for specimens of GFRP dowels (GH2N4). The results show that for the same type and magnitude of dowel misalignment (SV2N2 and GV2N2) the steel dowels debond at a lower load but need a significantly higher force to continue slipping. The debonding of the dowel bars is affected significantly by the dowel bar diameter as mentioned before. However, the slipping of the dowels is governed by the frictional stress between the dowel bar and the concrete pavement. Since, the friction caused by epoxy-coated steel dowels is much higher than that caused by GFRP (as shown later in Chapter Five), any increase in contact pressure due to bearing stress on the dowel bar produces a noticeable increase in the load required to open the joint. The maximum pull-out load for specimen SV2N2 can be observed at approximately 4 mm joint opening which is similar to a typical joint movement in an actual pavement. The load reached 14.6 kN and then decreased to an approximately stable value of 12.5 kN. For the specimens with GFRP dowels a maximum load of 8.85 kN was obtained after the start of slipping, then it linearly decreased to its final value of 7.5 kN at a 12 mm joint opening. For specimens with horizontal misalignment (GH2N4) and irrespective of the dowel misalignment magnitude, a higher load was needed to open the joint. Similar to the vertically misaligned dowels, bearing pressure induced frictional force increases for horizontally misaligned dowels too. However, unlike the vertically misaligned dowels, shrinkage strain variation due to the distance from the exposed surface did not cause any reduction in contact pressure for the horizontally misaligned dowels. Also, no lifting in the specimens was observed during the pull-out process. A large pull-out load was required for GH2N4 due to the above reasons and also the magnitude of misalignment was the highest one considered in this study. Cracks were observed at the concrete surface when the joint opening exceeded 6 mm and the concrete slab completely failed when the joint opening exceeded 9 mm. It can be observed from the results that the debonding phase for the steel dowels occurs gradually until about 4 mm of joint opening, hence the pull-out load joint opening behaviour becomes non-linear from the initiation of debonding. The non-linearity also
increases due to relatively high value of the coefficient of friction. The GFRP dowels on
the other hand, debond completely over very little joint opening, and as the coefficient
of friction is low, the pull-out load joint opening behaviour is virtually bi-linear, similar
to the elastic-perfectly plastic material property assumed in the elasto-plastic analysis of
structures. An analysis scheme for the GFRP dowels can be developed along this line.
However, it will be difficult to include the effect of concrete damage in such simplistic
model, and consequently any prediction of failure will not be possible.

4.5.2. Three misaligned dowel bars

Figure 4.14 presents the results of the specimens GA3, GV3P2 and GH3N2 that had
three GFRP dowel bars across the joint. The results show that for all specimens the
slipping of dowels started approximately at a similar load level. The differences can be
observed at the post-slip part of the joint opening. For specimen GA3, the pull-out load
was nearly stable with a small decrease until the joint was fully opened. While for
specimens GV3P2, the vertical tilt of the middle dowel bar led to a variation in the
contact pressure that consequently reduced the pull-out load required to open the joint.
However, after a specific joint opening of 4 mm the pull-out load became stable and
was approximately close to that of GA3 because after the breaking of the bond the pull-
out load is just the amount of load required to overcome kinetic friction. For the
specimen GH3N2, it can be noticed from the same figure that the load required to open
the joint increases with higher joint opening and the load magnitude is highest among
the specimens with three GFRP dowels of different orientations. This is because no
slab-base separation occurs for horizontal misalignment (as will be explained in Chapter
Seven of this study); hence there is no possibility in the reduction of contact pressure
between the slab and the dowels.
4.5.3. Effect of dowel misalignment on relative deflection, dowel looseness (DL) and LTE

In order to investigate DL due to the movement caused by expansion or contraction of the concrete pavement and its effect on the LTE, the deflections of the specimens were measured under a vertical quasi-static load before and after this movement. These tests were conducted to show the effect of slab movement on the specimens which had epoxy-coated steel dowel bars as compared with those specimens that had GFRP dowels, in order to investigate the suitability of using GFRP dowels to reduce joint lockup, RD and improve the LTE.

Figure 4.15 presents the results of the comparison between the RD for each specimen before and after the slab movement. It can be observed that for all the specimens the RD after the movement is higher than its value before the movement. The increase in RD for the specimens with misaligned dowels of both steel and GFRP dowels is higher than that of the specimens of aligned dowels. The specimens of GFRP dowels exhibited a smaller increase in RD due to slab movement compared with those of the epoxy-coated steel dowels. This is because the highly regular surface texture of the GFRP dowels creates lower frictional forces which leads to lower stresses at the dowel-concrete interface and consequently to a lower pull-out load. The dowel bar socket deteriorates less as a result of this.
The LTE is a criterion provided by the AASHTO (1993) guide to evaluate the load transfer by dowel bars across the joint. When a load is applied at the joint-face of a slab, both slabs are expected to deflect as a result of that. For an ideal load transfer across the joint, both loaded and unloaded slabs should exhibit the same joint-face deflection. Therefore, the LTE is measured as a ratio of the deflection of the unloaded side \( (d_u) \) to the deflection of the loaded side of the joint \( (d_l) \) as shown earlier in Equation (2.17), a detailed description of this can be found in Chapter two of this study.

Figure 4.16 shows a comparison of the LTE before and after the pulling-out and the pushing back of the slabs. Because the LTE is calculated as a function of joint-face deflections, a similar trend of results to those in Figure 4.15 is obtained. The reduction in LTE is smaller for specimens with aligned dowels than for those with misaligned dowels and the specimens with GFRP dowels exhibited a smaller reduction in LTE compared with that of steel dowels.
4.5.4. Results of the cyclic load test

The deflection readings were recorded under a quasi-static load of 40 kN after each set of 250,000 load cycles and the LTE was calculated according to these readings. The results presented in Figure 4.17 show that the 38 mm GFRP dowel produces a lower RD when compared to that of the 25 mm epoxy-coated steel dowel bars. The RD results of specimens with GFRP dowels show an increasing trend with a number of load cycles compared to the specimens with steel dowels. However, these values are still lower than those for the 25 mm steel dowel bars.

Figure 4.16. Comparison of LTE before and after slab movement

Figure 4.17. Relative deflection versus number of load cycles
Figure 4.18 illustrates the comparison of LTE for specimens with GFRP and steel dowels during the cyclic load test. The results show good LTE can be achieved using GFRP dowels during cyclic loading, principally due to the higher diameter of the GFRP dowels. The LTE for GFRP specimens decreased at a higher rate with an increasing number of load cycles when compared to the specimens with steel dowels. However, the LTE values still remained within a range of 70-100% which is sufficient for the load transfer across a typical joint (AASHTO 1993).

![Figure 4.18. LTE versus number of load cycles](image)

Figure 4.19 shows a comparison between the DL due to the slab movement and that due to the application of one million cycles of 40 kN load. The DL was calculated from the growth in RD of the joint face. The results reveal that the looseness due to slab movement is higher for the steel dowels than it is for the GFRP dowels, and for misaligned dowels than it is for aligned dowels. Pulling-out and pushing back the slabs during the test (which acts as a representation of the expansion and contraction of the concrete pavement) produced looseness for the epoxy-coated steel dowels higher than that due to one million load cycles; whereas, for the GFRP dowels, the looseness magnitude due to the slab movement is smaller than that due to the cyclic load effect. These performances are associated with the surface texture of both types of these dowels. The higher frictional stress between the steel dowels and the concrete causes more deterioration and enlargement (looseness) of the dowel bar socket than that of the GFRP dowels. The specimen GV2N2 shows a slightly greater looseness than that of
SV2N2 after a one million cycle of 40 kN load. The movement of slabs that have misaligned dowels leads to a looseness at the joint face that consequently increases the unsupported length of the dowel bar segment at the joint. This increase in unsupported length is more of a concern for the GFRP dowels than for the steel dowels because the GFRP dowels are weaker in the transverse direction. In spite of this looseness, the GFRP dowels show very high LTE (see Figure 4.18).

![Graph showing dowel looseness comparison]

**Figure 4.19. Comparison of dowels looseness due to slabs movement and cyclic load**

### 4.6. Pull-out Test

#### 4.6.1. Objectives of the test

The objectives of the pull-out test were:

- To check the suitability of the standard pull-out test i.e., individually pulling of dowel bars in an assessment of the dowel misalignment effect on the JPCP in terms of pull-out load magnitude and concrete deterioration.
- To investigate the effect of the orientation of misaligned dowel bars during concrete casting, and the distance from the exposure surface on the pull-out load magnitude.
- To investigate the effect of concrete age on the pull-out load required to cause slipping of the dowel bar from the concrete cube.

#### 4.6.2. Material and specimens fabrication

The experimental pull-out test programme involved six samples with different misalignment magnitudes. All samples were cast together in the same mould that had
six partitions as shown in Figure 4.20. The mould was fabricated so that the faces A1 & A2, and B1 & B2 were attached together before the mould assembly. These faces were drilled carefully at a specific angle for each specimen to obtain the required misalignment type and magnitude. Subsequently, these faces were assembled with the other parts of the mould with the dowel bars being fitted inside (see Figure 4.20). The misalignment magnitudes were checked again for the dowel bars within the mould before the sealant was applied to fix the position. The same concrete mix design was used as described before in this chapter.

Figure 4.20. Pull-out test: (a) Mould fabrication; (b) Preparing for casting
The samples were cast in the concrete laboratory on the vibrating table, and a total of six cubes were cast from the same concrete batch to measure the concrete compressive strength on the day of the test. Since the previous test showed that the GFRP dowel bars significantly reduced the pull-out load, only the 38 mm diameter GFRP dowels were involved in this test programme. The dimensions of each specimen were 300 mm × 200 mm × 200 mm in which the embedded length of the dowel bar was 227.5 mm as shown in Figure 4.20 and Figure 4.21.

### 4.6.3. Test procedure

The pull-out tests were carried out in the Heavy Structures Laboratory using a universal testing machine. The concrete block was held down to the base of the machine using a steel frame. This frame consisted of two transverse thick steel plates that held the top of the specimen to the machine base by two long threaded bolts. The free ends of these plates were supported on steel blocks on both sides of the specimen as shown in Figure 4.21. The steel plates were kept at a distance of 40 mm (according to a preliminary FE analysis) away from the dowel bar to avoid any bearing stress from these plates on the concrete surrounding the dowel bar. A special hinge was fabricated to connect the end of the dowel bar to the testing machine, so that the pull-out load direction always remained parallel with the concrete block centreline irrespective of the dowel bar orientation. The dowel bar slip was measured using the LVDT attached to the dowel bar. The load was applied with a constant loading rate of 0.5 mm/min.

Five specimens were pulled out to get a dowel bar slip of 12 mm. While, only one specimen was subject to four cycles of pull-out and push back to show the effect of this movement on any possible deterioration at the dowel-concrete interface which may prevent the dowel bar returning to the starting no-load position. This may have occurred due to the dislodging of some fine particles in the bottom of the hole due to the breakage of the bond between the dowel bar and the concrete pavement.
4.6.4. Results and discussion

As was stated before, the experimental programme involved six specimens. Two of these specimens contained aligned dowel bars while two contained horizontal misaligned dowel bars and two contained vertical misaligned dowel bars. It should be noted here that the terms “vertical misalignment” and “horizontal misalignment” refer to the orientations of the dowels during casting, as there was no difference in the load-application during the pull-out tests for all the specimens. The misaligned dowel bars had a misalignment magnitude of 19 mm per embedded length of the dowel bar. Dowel bars in specimens that contained aligned and horizontally misaligned dowel bars, had the same distance from the exposure surface of the specimen which is 100 mm from the centreline of dowel bar. The ends of the dowel bar in the specimens that contained vertically misaligned dowel bars were 119 mm from the exposure surface of the specimen. In order to show the effect of concrete hardening on the pull-out load, the tests were performed on one specimen from each type after 3 days while the other three specimens were tested after 28 days of concrete casting.

Figure 4.22 and Figure 4.23 present the pull-out loads for all tested specimens at ages of 3 and 28 days. The results show that there is no significant difference in pull-out loads for the specimens of the same age compared with the effect of dowel misalignment on
specimens containing two concrete slabs and two dowel bars across the joint, as described earlier in this chapter. This behaviour is attributed to the missing of the interaction of the group of dowel bars across the joint, which was described in earlier sections of this chapter. However, it can be observed that the pull-out loads for specimens that have horizontal misalignment are slightly higher than those of the aligned specimens, which could be due to the effect of bearing stress due to pull-out load on a misaligned dowel bar. The specimens that have a vertically misaligned dowel bar exhibited a slightly lower pull-out load compared to the aligned specimens, due to the effect of less shrinkage strain for further distance from the exposure surface (J. K. Kim and Lee 1998; Lim et al. 2009). This reduction due to shrinkage is more than the increase in higher bearing stress due to misalignment, as was observed in the case of horizontally misaligned specimens. The results also showed that the pull-out load increased with the age of the concrete.

Figure 4.22. Pull-out load versus slip distance at ages of 3 and 28 days
Figure 4.23. Cyclic pull-out load versus slip for a vertically misaligned dowel bar

Figure 4.23 shows that the repeated pull-out and push back of the specimen with a vertically misaligned dowel bar at an age of 28 days did not cause deterioration of the dowel-concrete interface, as the dowel bar can be easily returned to the initial position without increase in pull-out load. This behaviour is attributed to very low frictional stress and the higher uniformity of the GFRP dowel bar surface. This can be considered as a merit for the GFRP dowels because some tests in literature have shown a higher push back force compared with the pull-out force that was required for uncoated or bitumen coated steel dowel bars to push the dowel to initial position (Löfsjögård 2005; Shoukry et al. 2003). More push back force reflects concrete crushing at the dowel-concrete interface and displaced particles in the bottom of the hole of the dowel bar.

4.7. Summary and Conclusions

An experimental study was conducted to investigate dowel misalignment and joint lockup using epoxy-coated steel and GFRP dowels for realistic pavement conditions. The study considered the effects of the position of the dowel bar during concrete casting and evaluated the combined effect of dowel misalignment and traffic wheel load on LTE, RD and DL for both epoxy-coated steel and GFRP dowels. The GFRP dowels, as an alternative to epoxy-coated steel dowel bars, manifest a significant reduction in the load required for joint opening. The lower pull-out load reduces the stress at the dowel-
concrete interface, DL and the mid-span stress in the slab. Although only one cycle of push-out and push-back was conducted, the results illustrate that the axial movement of the slab produces a significant increase in DL and a decrease in LTE when compared with that due to the repetition of traffic wheel loads especially in the presence of dowel misalignment. The use of GFRP dowels can improve the pavement performance by minimizing the joint lockup and enhancing the LTE. Since the expansion and contraction of pavements are continuous during the service life of a road, more deterioration will occur in the concrete surrounding steel dowels as compared to GFRP dowels. This deterioration increases rapidly when dowel misalignment exists. It should be mentioned that although the experiments were designed after careful considerations of various parameters, the current set of results do not include other effects like the curling of slabs due to temperature changes. Even though the experimental results clearly indicate better performance of the GFRP dowels under the test conditions, more experimental data including duplication is necessary for a detailed quantitative evaluation.

The pull-out test results showed that the standard pull-out tests using single dowel cannot precisely quantify the effect of dowel misalignment. It also showed that changing the distance from the exposure surface may affect the pull-out load magnitude.
CHAPTER FIVE
DEVELOPMENT OF A 3D FINITE ELEMENT MODEL

5.1. Introduction

A numerical simulation was carried out to expand the experimental investigation by including most practical parameters and to give insight about stress-strain distribution and damage initiation. This chapter describes in detail the development of the finite element model (FEM) using ABAQUS software for the modelling of the load-deflection response of a dowel bar, and for modelling the combined effect of dowel misalignment and wheel load on a dowel bar’s performance in JPCP. The description involves element selection, material models, dowel-concrete interaction, meshing size, loading steps, boundary conditions, prediction of some required parameters, and the design of a steel-beam supporting base.

5.2. Element Selection

5.2.1. Solid element

The ABAQUS library contains several types of three dimensional (3D) continuum elements, such as the 8-noded (C3D8, linear) or 20-noded (C3D20, quadratic) elements as shown in Figure 5.1. These elements are capable of modelling linear or nonlinear stress-displacement analyses involving contact interaction and material plasticity (Cook 1995). Each node in these elements has three translational degrees of freedom. Both linear and quadratic solid elements can have reduced or full integration features. The reduced integration element has fewer integration points as shown in Figure 5.1. The
faces of these elements are connected together in a similar way to the arrangement of bricks; therefore they are often called brick elements.

In the current study the first order reduced integration solid elements (C3D8R) were used to model the concrete slabs, dowel bars and the steel-beam supporting base. The first-order elements were selected to avoid the convergence problem often encountered in contact formulation using second-order elements due to the presence of mid-side points at the edges of the element (SIMULIA 2010). The reduced integration elements were selected because the fully integrated linear (first-order) elements may exhibit shear locking, which means that the deformed shape of the element under pure bending includes spurious non-zero transverse shear strain. Although fewer nodes and fewer integration points are involved with the first order reduced integration element (see Figure 5.1), good results can be achieved by adopting a finer mesh (SIMULIA 2010). Consequently, by using C3D8R elements for the dowel bars, it was possible to simulate contact and associated frictional stress at the dowel-concrete interface.

![Figure 5.1. Solid element formulation](image)
5.2.2. Spring element

Spring elements were adopted in the current study to model the longitudinal bond between the dowel and the concrete during expansion and contraction. There are several types of spring elements that are available in the ABAQUS library. These springs can connect one node to the ground and are called SPRING1; or two nodes in a fixed direction (SPRING2); or two nodes along the direction from Node 1 to Node 2 (SPRINGA) (see Figure 5.2). The spring elements can be linear or nonlinear and the difference between these is that the linear spring element is defined by its stiffness only while for the nonlinear spring element the complete force-displacement relationship need to be prescribed using ABAQUS input file. SPRING2 with non-linear force-displacement relationships was used in this study.

![Spring element](image)

**Figure 5.2.** Spring element: (a) Connecting node to ground; (b) Connecting two nodes.

5.3. Material Modelling

The current investigation involved three different materials: steel dowels, GFRP dowels, and concrete. The modelling of each one of these materials is described below.

5.3.1. Steel dowel bars

The current study involved two major strands. The first part involved an investigation of the load-deflection response of the dowel bars across the transverse joints of JPCP and the second part involved the investigation of the combined effects of dowel misalignment and wheel load on the dowel bars performance in JPCP.

Since the loads in the first part were applied until reaching the failure point of the specimens, a classical metal plasticity model was used to model the steel dowel material. The yield and ultimate strength was taken as 275 MPa and 460 MPa respectively, whereas the modulus of elasticity was 200 GPa. These data were obtained
from the experimental test presented in Chapter Three of this study (§3.3.1.1); Poisson’s ratio was taken as 0.3.

In the second part of the experimental programme, the slab section that had two epoxy-coated steel dowel bars was subjected to a 40 kN load as equivalent to a single axle load. According to the AASHTO (1993) guide, the dowel bar materials remain linear elastic under this axle load. Therefore the linear elastic material model was used with the same material properties as mentioned above.

### 5.3.2. GFRP dowel bars

GFRP dowels are a unidirectional fibre-reinforced composite material consisting of continuous filaments in the longitudinal direction. The production method of these bars makes it possible to consider them as a transversely isotropic material. This means that the elastic constants such as Young’s modulus of elasticity and Poisson’s ratio are isotropic in the plane of the cross-section. In the direction normal to the cross-section the elastic constants are different to the in-plane values. For numerical simulations, the elastic constants were obtained from several sources including the properties supplied by the manufacturer, the experimental results from the shear test and the available theoretical and empirical models in the literature. The theoretical and empirical models that have been proposed in the literature to obtain the mechanical properties of GFRP bars are based on their volume fractions (Hull and Clyne 1996; Tsai and Hahn 1980). Glass fibre volume fraction was calculated as 0.545 according to the weight fraction listed by the manufacturer (Table 3.1). Since the derivations of these models are beyond the scope of this work only the final values of the elastic properties are shown in Table 5.1.

Five independent elastic parameters were used to define the material properties of the GFRP bars in ABAQUS. These parameters involved axial and transverse modulus of elasticity ($E_1$ and $E_2$), shear modulus ($G_{12}$), and Poisson’s ratios ($v_{12}$ and $v_{21}$). The modulus of elasticity $E_1$ and $E_2$, weight fraction of fibre, and specific gravity of dowels were chosen from the data supplied by the manufacturer and the shear modulus $G_{12}$ was adopted from the double shear test as illustrated in Chapter Three of the current study, while Poisson’s ratios were calculated using the empirical models from literature (Hull and Clyne 1996; Tsai and Hahn 1980). For these calculations, Poisson’s ratio and
density of the glass fibre were taken as 0.22 and 2620 kg/m$^3$ respectively, while the density of the matrix was taken as 1350 kg/m$^3$ from the data available in literature (CEB-FIP 2007).

Table 5.1 Mechanical properties of GFRP bars used in the numerical simulations
(Direction 1 is parallel to the dowel centre line and Directions 2 and 3 are the transverse directions).

<table>
<thead>
<tr>
<th>Property</th>
<th>Magnitude</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_1$ (GPa)</td>
<td>40.8</td>
<td>Manufacturer</td>
</tr>
<tr>
<td>$E_2 = E_3$ (GPa)</td>
<td>10</td>
<td>Manufacturer</td>
</tr>
<tr>
<td>$G_{12} = G_{13}$ (GPa)</td>
<td>3.62</td>
<td>Current test</td>
</tr>
<tr>
<td>$v_{12}$</td>
<td>0.29</td>
<td>Calculated</td>
</tr>
<tr>
<td>$v_{21}$</td>
<td>0.071</td>
<td>Calculated</td>
</tr>
</tbody>
</table>

5.3.3. Concrete Models

ABAQUS offers three different models for the modelling of concrete material under low confining pressures. These models are the Smear ed Cracking Model, Brittle Cracking Model and Concrete Damaged Plasticity (CDP) Model. The CDP model was used in the current study because a Brittle Cracking Model is only available for ABAQUS/Explicit, when structures are subjected to a dynamic load, whereas a Smeared Cracking Model involves significant simplifications of the actual behaviour of the concrete structure under a compressive load.

CDP is a versatile model for prediction of the response of all types of concrete structures and other quasi-brittle materials subjected to monotonic, cyclic and/or dynamic load. This model is available for both ABAQUS/Standard and ABAQUS/Explicit and it can be used for both plain and reinforced concrete structures. The two failure mechanisms that are assumed in this model are tensile cracking and compressive crushing. According to this model, the tensile and compressive behaviours of concrete are identified by damaged plasticity. For a uniaxial tension, stress-strain behaviour of concrete shows linear-elastic response until reaching the value of failure stress $\sigma_{f0}$. After this value, stress-strain softening occurs due to the initiation of cracks.
Similarly for the compressive behaviour, the linear elastic response is observed until reaching the yield value $\sigma_{co}$. Beyond this point, the stress hardening is seen until reaching the ultimate value of stress in compression $\sigma_{cu}$ followed by strain softening. The CDP model allows the user to consider the effect of stiffness recovery during cyclic loading and it can also show the degradation in the elastic stiffness of concrete by two damaged variables $d_i$ and $d_c$. These variables are function of the plastic strains ($\tilde{\varepsilon}_t^{pl}$ and $\tilde{\varepsilon}_c^{pl}$) and temperature. The values of the damage variables range from zero for undamaged material to one, in which the strength of the material is completely lost. Figure 5.3 shows the concrete behaviour under uniaxial compression and tension.

![Concrete Behaviour Diagram](image)

**Figure 5.3.** Response of concrete to uniaxial loading (SIMULIA 2010). where, $E_o$ :the initial (undamaged) elastic stiffness of the material; $\sigma_{t0}$ : tension failure stress; $\tilde{\varepsilon}_t^{pl}$ & $\varepsilon_t^{el}$ : Plastic and elastic tensile strains respectively; $\sigma_{co}$ & $\sigma_{cu}$ : yield and ultimate stress in compression; $\tilde{\varepsilon}_c^{pl}$ & $\varepsilon_c^{el}$ : Plastic and elastic compressive strains respectively; $d_i$ : Damage variable in tension; $d_c$ :Damage variable in compression.

### 5.3.3.1. Input parameters for CDP

The CDP model can characterise the concrete behaviour in tension and compression by defining the uniaxial compression response, uniaxial tensile response and plasticity parameters. A brief description of these parameters is mentioned below.
5.3.3.1.1. Compressive behaviour

In order to define the stress-strain response of the plain concrete in uniaxial compression beyond the elastic range, the post-yield compressive stress $\sigma_c$ and inelastic strain $\varepsilon_c^{\text{in}}$ should be provided in a tabular form; this definition describes both the hardening and softening regions. The inelastic strain is obtained by subtracting the compressive elastic strain $\varepsilon_c^{\text{el}}$ from the total strain in compression $\varepsilon_c$ as follows.

$$
\varepsilon_c^{\text{in}} = \varepsilon_c - \varepsilon_c^{\text{el}} \tag{5.1}
$$

where, $\varepsilon_c^{\text{el}} = \sigma_c / E_o$

Defining the compressive behaviour in the numerical analysis was based on the experimental data of concrete cube tests described in Chapters 3 and 4. The stress-strain relationship for concrete under uniaxial compression was adopted from EC2 as shown in Figure 5.4 (BS EN 1992-1-1 2004).

$$
\frac{\sigma_c}{f_{cm}} = \frac{k n - n^2}{1 + (k - 2)n} \quad n = \frac{\varepsilon_c}{\varepsilon_{cl}}
$$

$\varepsilon_{cl} = 0.0007 f_{cm}^{0.31} < 0.0028$

$k = 1.05 E_{cm} \times |\varepsilon_{cl}| / f_{cm}$

$E_{cm} = 22 [0.1 f_{cm}^{0.3}] \times 0.9$

(0.9 for limestone aggregate)

$\varepsilon_{cul} = 0.0035$ for $f_{cm} < 58$ MPa

$f_{cm} = f_{ck} + 8$

$f_{ck}$: Characteristic compressive strength of concrete at 28 days

Figure 5.4. Stress-strain relation for concrete under uniaxial compression (BS EN 1992-1-1 2004)

5.3.3.1.2. Tensile behaviour

To describe the tensile softening behaviour of concrete two approaches are available in ABAQUS. These approaches are the post-failure stress-strain relation in Figure 5.3 (b)
and the fracture energy-cracking criterion shown in Figure 5.5. The first approach causes convergence problems for low or unreinforced concrete structures (SIMULIA 2010) such as the current case of JPCP.

The second approach is the fracture energy criterion which was developed by Hillerborg (1976). This approach overcomes the shortcoming of the stress-strain approach. According to this criterion, the stress-displacement relation instead of the stress-strain relation describes the brittle behaviour of concrete. The required energy to open the unit area of the crack was defined by Hillerborg (1976) as a material parameter and called fracture energy ($G_f$). Numerically, the fracture energy equals the area under the stress-displacement curve. The fracture energy-cracking criterion can be defined either in a tabular format of post-failure stress and cracking displacement or directly specifying the fracture energy as a material property associated with the failure stress. The latter option assumes a linear loss of concrete strength after cracking (see Figure 5.5 (b)). The first option represents the loss of tensile strength in the structure by multi lines or bilinear curve as shown in Figure 5.5 (a) and Figure 5.6.

![Figure 5.5. Post-failure tensile response: (a) Stress-crack displacement, (b) Stress-fracture energy (SIMULIA 2010)](image)

In this thesis, the tensile post-failure behaviour of concrete was defined by a bilinear stress-crack opening procedure reported in CEB-FIP 1990 (1993) design code as shown in Figure 5.6. Expressions shown in Figure 5.6 were used to define the cracking displacement corresponding to post-failure stress. The fracture energy varies with concrete class and maximum aggregate size as shown in Table 5.2 and Table 5.3.

The post-failure behaviours of concrete in compression and tension were defined in this study according to the previous discussions and are shown in Figure 5.7.
Table 5.2. Fracture energy (Nm/m² — (CEB-FIP 1990 1993))

<table>
<thead>
<tr>
<th>Max. aggregate size $d_{max}$ (mm)</th>
<th>$G_F$</th>
</tr>
</thead>
<tbody>
<tr>
<td>C12</td>
<td>C20</td>
</tr>
<tr>
<td>8</td>
<td>40</td>
</tr>
<tr>
<td>16</td>
<td>50</td>
</tr>
<tr>
<td>32</td>
<td>60</td>
</tr>
</tbody>
</table>

Table 5.3. Coefficients $\alpha_F$ is related to maximum aggregate size (CEB-FIP 1990 1993)

<table>
<thead>
<tr>
<th>$d_{max}$ (mm)</th>
<th>8</th>
<th>16</th>
<th>32</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\alpha_F$</td>
<td>8</td>
<td>7</td>
<td>5</td>
</tr>
</tbody>
</table>

Figure 5.6. Stress-crack opening diagram (CEB-FIP 1990 1993)

Figure 5.7. Inelastic behaviour of concrete ($f_{ck,cube} = 30$ MPa): (a) In compression (b) In tension

121
5.3.3.1.3. Plasticity parameters

The following plasticity parameters are required input in the ABAQUS CDP model.

- **Dilation angle** \((\psi)\) is a material parameter measured in the plane of confining pressure \((p)\) and von Mises stress \((q)\) at a high confining pressure and shows the inclination of an incremental plastic strain. The dilation angle was found to be varied between \(20^0-40^0\). A better agreement with the experimental results was achieved for dilation angles between \(30^0-40^0\) (Malm 2009). In the current study the dilation angle was selected after carrying out a sensitivity analysis using validated FEM in ABAQUS as shown in Figure 5.8. In spite of the difference with the experimental results at the start of the dowel debonding, the dilation angle of \(30^0\) provided a good prediction of the initiation of failure. Hence this value was adopted in the concrete damage plasticity model. It can be observed from Figure 5.8 that ductility increased with an increase in dilation angle.

- **Eccentricity** \((\varepsilon)\) refers to the rate of change of plastic flow potential function. The default value of the eccentricity is 0.1, in which the dilation angle does not change over a wide range of confining pressure. A higher eccentricity value than 0.1 causes a rapid increase of the dilation angle for low confining pressure. While a lower value than 0.1 may produce a convergence problem when the material is subjected to the low confining pressure.

- **\(\frac{\sigma_{bo}}{\sigma_{co}}\)** is the ratio of initial equibiaxial compressive yield stress to initial uniaxial compressive yield stress. Its value is between 1.1 and 1.16. The latter value was used in current analysis as it is the default value in ABAQUS.

- **\(K\)** is the ratio of the second stress invariant on the tensile meridian \(q_{(TM)}\), to that on the compressive meridian \(q_{(CM)}\), at initial yield for any given value of the pressure invariant \(p\) such that the maximum principal stress is negative. Its value is between 0.5 and 1 (the default value is 0.6667).

- The viscosity parameter may be used in ABAQUS to improve the convergence in softening regime; the default value of it is set to zero in ABAQUS.
5.4. Dowel-Concrete Interaction

Several models were considered in the literature to model the dowel-concrete interface as illustrated in Chapter Two of this study. In the current FEM the dowel-concrete interface was represented by surface-to-surface contact for simulating the dowel bar response across the transverse joints of JPCP. A non-linear spring elements were also added in the simulation of the joint opening and closing for evaluation of the combined effect of dowel misalignment and wheel load on the dowels bar performance in JPCP. The following sections illustrate in detail the adopted techniques for representing the dowel-concrete interaction in each of the two adopted FEMs in this study.

5.4.1. Modelling of load-deflection response of dowel bars across joints of JPCP

The dowel-concrete interaction has been characterised in the current analysis using surface-to-surface contact interaction. Small sliding formulation has been used because the average sliding between dowel and concrete (contacting surfaces) is less than the element length in the contact region (SIMULIA 2010). The normal behaviour between interacting surfaces has been modelled by hard contact, whereas the tangential
behaviour has been addressed by defining the coefficient of friction between the dowel and concrete.

5.4.2. Modelling of combined effect of dowel misalignment and wheel load

The bond between the concrete and the dowel was simulated using two sets of interactions. The first interaction represented the longitudinal bond between the dowel and the concrete due to several factors such as chemical adhesion between the concrete and the dowel, the roughness or irregularities of the dowel surface, and other factors such as static friction. The second interaction represented the transverse interaction between contacting surfaces of the dowel and the concrete pavement. This interaction produces bearing stress on the adjacent concrete and an additional longitudinal bond due to increased frictional stress. The transverse interaction significantly affects the pull-out load per dowel in the presence of dowel misalignment.

The longitudinal interaction was modelled by using nonlinear spring elements connecting the coincident points of the dowel and the concrete slab along four lines on the surface of the dowel. The nonlinear force-displacement relationship was calibrated on the basis of experimental tests of similar cases with aligned dowels. Since the bond between the aligned dowel bar and the concrete pavement is primarily governed by the longitudinal bond whereas the contribution of the transverse interaction is minimal; three specimens with aligned dowel bars were tested: SA2, GA2 and GA3. The pull-out load per dowel was converted to uniform stress along the dowel bar as pull-out stress versus joint opening. The force carried by each spring was obtained by multiplying the pull-out stress by the corresponding area of each spring.

The transverse interaction represents the interaction between the contacting surfaces of the dowel and the concrete, resulting from frictional and bearing stresses. This interaction is particularly important when the dowels are misaligned (see Figure 5.9). This interaction was modelled in ABAQUS by surface-to-surface contact interaction with coulomb friction for the tangential behaviour and hard contact for the normal behaviour. The slipping of the dowel starts when the applied stress due to the joint opening force is greater than the contact pressure multiplied by the coefficient of friction. The hard contact between the dowel bar and the concrete produces high bearing stress at the dowel-concrete interface especially when the dowel bar is misaligned.
5.5. **Mesh Size**

Eight-noded first order reduced-integration continuum solid elements (C3D8R) with three translation degrees of freedom were used to represent all model components in both of: modelling load-deflection response of dowel bar in JPCP and modelling combined effect of dowel misalignment and wheel load on dowel bar performance. These elements allow for a clear and precise view for large deformations and stress-strain distribution at the dowel-concrete interface.

The adopted mesh size for load-deflection response model is shown in Figure 5.10 while the model of combined effect of dowel misalignment and wheel load on dowel bars performance in JPCP is shown in Figure 5.11.
A sensitivity analysis was carried out to select the appropriate mesh size so as not to affect the accuracy of the results significantly and at the same time keeping the computational time within a reasonable limit. Simulation results from three different meshes were compared with the experimental results. Since the modelling of combined effect of dowel misalignment and wheel load involves applying a quasi-static load and joint movement, the sensitivity analysis was carried out on this model, and similar size of elements were used for both models. Comparison of pull-out loads (Figure 5.12), and deflection of loaded side ($d_l$) and deflection of unloaded side ($d_u$) of slabs (Table 5.4) showed that the element size of $12.5 \text{ mm} \times 5 \text{ mm} \times 4.16 \text{ mm}$ for the dowel bar and the element size of $12.5 \text{ mm} \times 10 \text{ mm} \times 4.16 \text{ mm}$ for surrounding concrete pavement gives a good agreement with the experimental results. For the concrete away from the dowel bar coarser mesh was used.

The modelling of combined effect of dowel misalignment and wheel load on dowel bar’s performance in JPCP consisted of simulating two concrete slabs connected by 2 or
3 dowel bars and resting on the steel-beam base. Using the same element size for whole slab produces significant increase in element numbers and computational time. Therefore, each concrete slab was modelled as two parts tied together using tie constraint (SIMULIA 2010). More refinement (according to sensitivity analysis results) was provided for concrete pavement surrounding the dowel bars in order to assess concrete deformation due to dowel misalignment and wheel load, whereas a coarse mesh was used for the rest of the slab as shown in Parts (c) and (d) of Figure 5.11.

![Figure 5.12. Sensitive analysis for mesh size of the model (SV2N2)](image)

**Table 5.4. Joint face deflection for different elements size (SV2N2)**

<table>
<thead>
<tr>
<th>Element size</th>
<th>$d_l$</th>
<th>$d_u$</th>
</tr>
</thead>
<tbody>
<tr>
<td>25 mm $\times$ 10 mm $\times$ 6.25 mm</td>
<td>2.27</td>
<td>2.13</td>
</tr>
<tr>
<td>12.5 mm $\times$ 5 mm $\times$ 4.16 mm</td>
<td>2.27</td>
<td>2.13</td>
</tr>
<tr>
<td>10 mm $\times$ 5 mm $\times$ 4.16 mm</td>
<td>2.27</td>
<td>2.13</td>
</tr>
<tr>
<td>Experimental</td>
<td>2.45</td>
<td>2.28</td>
</tr>
</tbody>
</table>
5.6. Loading Steps and Boundary Conditions

5.6.1. Modelling of load-deflection response of dowel

FEM was carried out to simulate the experimental tests of load-deflection response of single dowel bar connecting two concrete blocks across a joint (see Figure 3.1 and Figure 5.10). The reacting concrete block was supported by a thick steel plate at the base, and another steel plate at the top for one-third length of the block was used to clamp the top surface. These two plates were connected by steel bolt through a hole in the reacting concrete block. The interaction between these two plates and concrete were modelled as surface-to-surface contact while each end of bolt was tied to each corresponding plate as representation for the actual tightened state.

For the loaded block, the steel loading frame was attached to the concrete at two bolted positions at the back, and the rest of the interaction was modelled as surface-to-surface contact. The interaction between the load cells and the loaded block (at the bottom) was modelled as contact interaction as well. The lateral movement of the loaded block was constrained in the simulation as clamps used in the test prevented any such movement. The dimensions of the model components were described in Chapter Three of this thesis.

Since the load was applied as a line load in the test, a small partition was made to apply the load as pressure on very narrow area (3 mm × 100 mm). This technique was used because the line load option in ABAQUS is not available for 3D analysis (SIMULIA 2010). Only one step was used in this model to apply the self-weigh of the model components and the monotonic load.

5.6.2. Modelling of combined effect of dowel misalignment and wheel load

The steps undertaken in the analysis were designed to model the actual experimental tests. Five steps were included in the current simulation as mentioned below:

- Step one involved loading one side of the slabs monotonically from 0-40 kN as a representation of the traffic wheel load at the edge of the slab and the slabs’ self-weights. The self-weight of the slabs and the dowels were applied as a body
force. In this step, the deflections of the loaded and unloaded sides were noted to compare with the experimental values of LTE.

• Step two comprised of suspending the quasi-static load of 40 kN and keeping only the self-weight from the previous step. This step was designed to release the elastic deformation of the slabs due to the quasi-static wheel load, and to develop a contact pressure between the slabs and dowels, which is necessary for the activation of the coulomb friction model during the pull-out stage.

• Step three included opening the joint by 6 mm from each side to model the concrete pavement contraction. A displacement control was used in this step to get the required joint opening. This step showed the pull-out load required to open the joint for different misalignment cases.

• Step four involved closing the joint up to the initial opening by reversing the displacement controls’ directions.

• Step five included the repetition of step one to evaluate the LTE of the dowel bars and to estimate the DL after the slab movement. The DL was calculated indirectly in terms of the difference in the RD of the joint edges before and after the slab movement.

An addition to what mentioned above, the lateral movement of the slabs was constrained to represent the clamping of slabs in the test. The interaction between steel-beam base and concrete slabs was modelled by surface-to-surface contact with coefficient of friction of 0.02 for numerical stability as PTFE has a very low coefficient of friction.

5.7. Estimation of Coefficient of Friction

5.7.1. Steel dowel bars

Experimental and numerical studies have been conducted to measure the coefficient of friction of steel dowel bars (Shoukry et al. 2003; William and Shoukry 2001). A FE investigation (William and Shoukry 2001) showed that the coefficient of friction can be taken as 0.05 for greased dowels. Another experimental study (Shoukry et al. 2003) showed that the coefficient of friction between 32 mm steel dowels and concrete is 0.076 for greased side and 0.384 for ungreased side, and 0.0986 for greased side and
0.343 for ungreased side for 38 mm steel dowel bars. In the current study, the coefficient of friction was selected according to a sensitivity analysis as shown in Figure 5.13. The results in Figure 5.13 show that the friction coefficient of 0.05 for greased side of epoxy-coated steel dowel gives a good agreement with the experimental results. The corresponding value for the coefficient of friction for the ungreased side of dowel was 0.35.

![Figure 5.13. Prediction of coefficient of friction between epoxy coated steel dowels and concrete pavement (SV2N2)](image)

**Figure 5.13. Prediction of coefficient of friction between epoxy coated steel dowels and concrete pavement (SV2N2)**

### 5.7.2. GFRP dowels

There is no previous work to estimate the coefficient of friction of GFRP dowels. A previous pull-out test (Löfsjögård 2005) showed that the pull-out load for GFRP dowels is significantly lower than that for greased steel dowels. Therefore, a lower coefficient of friction is expected. A sensitivity study was carried out to estimate this value and the results showed that the best agreement was obtained for a coefficient of friction of 0.016 as shown in Figure 5.14.
5.8. Design of Steel Supporting Base

A detailed description (dimensions and arrangement) for the steel-beam supporting base was presented in Chapter Four of this study. As mentioned before, this base was designed using a FE analysis to characterise the behaviour of a base layer with a stiffness (modulus of subgrade reaction) of 54 MN/m$^3$ (O’Flaherty 2002). Firstly, the slab was modelled with elastic foundation of stiffness $k = 54$ MN/m$^3$. Then the elastic foundation was removed and the slab was supported by four beams with the arrangements shown in Figure 4.4 (Chapter Four). The dimensions of the beams were selected such that the deflections of the slabs at eight LVDT positions (Figure 4.5-Chapter Four), and the shear force and bending moment in the dowels were of similar values in both the analyses. Subsequently, the second FE model was validated with displacement values from the experiments. The results showed that the shear force in the dowels in specimens supported by the steel-beam were very similar to the shear force from the specimens supported by an elastic foundation as shown in Figure 5.15.

The deflection values from both sets of simulations are also very similar to each other and agree well with the experimental results as shown in Table 5.5.
Figure 5.15. Comparison of dowel bar shear force for specimens SA2 on the steel-beam base and on the elastic foundation

Table 5.5. Comparison of deflection of concrete slab supported by different bases types

<table>
<thead>
<tr>
<th>Source of data for each Supporting type of slab</th>
<th>Joint face deflection (mm)</th>
<th>Outer edge of the slab (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Loaded side</td>
<td>Unloaded side</td>
</tr>
<tr>
<td>Expt – Steel-beam</td>
<td>- 2.29</td>
<td>- 2.12</td>
</tr>
<tr>
<td>FEM – Steel-beam</td>
<td>- 2.26</td>
<td>- 2.13</td>
</tr>
<tr>
<td>FEM - elastic foundation</td>
<td>- 2.18</td>
<td>- 2.03</td>
</tr>
</tbody>
</table>

The effect of the supporting base type on the pull-out load required to open the joint was also compared as shown in Figure 5.16 and Figure 5.17. The verifications also involved the modelling of a fixed base which was adopted in previous research (Prabhu et al. 2007). The current set of analysis shows the effect of omitting the representation of the base layers on the pull-out load, and consequently, on the damaged concrete volume. Figure 5.16 and Figure 5.17 show the FEM results of the pull-out load for the specimen supported by steel-beam, elastic foundation and fixed base compared with the experimental pull-out load for specimens resting on steel-beam base. These results show no significant difference in pull-out load for the specimens supported by the steel-beam and specimens supported by the elastic foundation compared with the experimental results. On the other hand, the fixed base case shows a significant difference in pull-out
load compared with the experimental results for the horizontal misalignment case, as shown in Figure 5.17.

**Figure 5.16. Effect of different bases’ types on pull-out load for vertical misalignment (SV2N2)**

![Graph showing effect of different bases' types on pull-out load for vertical misalignment.]

**Figure 5.17. Effect of different bases’ types on pull-out load for horizontal misalignment (GH2N4)**

![Graph showing effect of different bases' types on pull-out load for horizontal misalignment.]

133
CHAPTER SIX
NUMERICAL INVESTIGATION INTO THE LOAD-DEFLECTION RESPONSE OF GFRP DOWELS AND DEVELOPMENT OF DESIGN CONSIDERATIONS

6.1. Introduction

This chapter involves a numerical investigation to compare the performance of Glass Fibre Reinforced Polymer (GFRP) and steel dowel bars as load transfer devices across the transverse joints of a Jointed Plain Concrete Pavement (JPCP). It presents a numerical simulation of the experiments reported in Chapter Three to validate the models. From the numerical simulation it was possible to visualize the bearing stress distribution at the dowel-concrete interface for both steel and GFRP dowel bars. A simulation was also carried out for an experimental test of a full-scale pavement section found in the literature. Subsequently, the validated finite element model (FEM) was used to develop design considerations for GFRP dowel bars in JPCP by conducting a detailed parametric study involving all the important dowel bars and pavement parameters.

6.2. Current Design of Dowel Bars

The design of epoxy-coated steel dowel bars varies according to the requirements of different highway agencies. However, the spacing of 300 mm seems to be the common standard for most of these agencies. The AASHTO (1993) guide and the UK Highways Agency recommendations, as shown in Table 6.1, were considered for the purpose of the comparison between GFRP and steel dowels. The design procedure for steel dowel bars without any modifications is unlikely to be suitable for the GFRP dowels since
they have a relatively low stiffness in the transverse direction. Therefore, the current numerical investigation was carried out to propose design considerations for the GFRP dowels by comparing their behaviour with that of steel dowels for different pavement system parameters.

Table 6.1. Recommended steel dowel bar dimensions

<table>
<thead>
<tr>
<th>Pavement thickness $h$ (mm)</th>
<th>AASHTO (1993) guide</th>
<th>UK Highways Agency</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Diameter (mm)</td>
<td>Length (mm)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>150</td>
<td>19</td>
<td>458</td>
</tr>
<tr>
<td>200</td>
<td>25</td>
<td>458</td>
</tr>
<tr>
<td>250</td>
<td>32</td>
<td>458</td>
</tr>
<tr>
<td>300</td>
<td>38</td>
<td>458</td>
</tr>
<tr>
<td>350</td>
<td>44.45</td>
<td>458</td>
</tr>
<tr>
<td>$h$ &lt; 239</td>
<td>25</td>
<td>600</td>
</tr>
<tr>
<td>$h$ &gt; 239</td>
<td>32</td>
<td>600</td>
</tr>
</tbody>
</table>

As stated before, several studies have been conducted on GFRP dowels as load transfer devices in concrete pavements (Eddie et al. 2001; Porter et al. 2001; Porter et al. 1993; Vijay et al. 2009). These studies investigated, both experimentally and theoretically, JPCP with GFRP dowels for static and cyclic load. Their results found that the GFRP dowels are adequate load transfer devices when proper diameter and spacing are selected.

From a laboratory fatigue test on full scale pavement slabs, Porter et al. (1993) observed the possibility of using GFRP dowels of 44.45 mm diameter spaced at 200 mm to obtain a response similar to epoxy-coated steel dowel bars of 38 mm spaced at 305 mm. Subsequently, experiments conducted by Eddie et al. (2001) showed that the 38 mm GFRP dowels performed at a similar level to 32 mm epoxy-coated steel dowels. A discrepancy in the observed results was noticed between the studies of Porter et al.
(1993) and Eddie et al. (2001) since the earlier study showed a higher relative deflection (RD) in 38 mm GFRP dowels when used as an alternate for the 32 mm steel dowel. Another study carried out by Porter et al. (2001) showed that 38 mm GFRP dowels spaced at 150 mm performed equally to that of steel dowels of the same diameter spaced at 305 mm. However, all these studies considered a typical contraction joint of width 3.2 mm (1/8 inch).

A recent experimental and theoretical study (Vijay et al. 2009) demonstrated that a smaller RD and a higher load transfer efficiency (LTE) can be obtained from GFRP dowels when spaced at 152.5 mm compared with steel dowels positioned at 305 mm. The contraction joint was saw-cut with a maximum width of 6.4 mm.

This chapter introduces a detailed numerical study in order to simulate the load-deflection response of steel and GFRP dowels, and to develop design considerations for GFRP dowels. The study involved most of the important pavement system parameters and all possible combinations of dowel-concrete pavement system parameters. For example, parameters such as pavement thickness and the spacing between the bars for different joint widths especially for the expansion joints (wide joints), which were not included in any of the previous studies, have been considered in the current study.

6.3. Numerical Results

6.3.1. Validation of FEM against the experimental results

The FEM results were validated by the experimental results of the load-deflection response of the GFRP dowels which was presented in Chapter Three of this study. A reasonable agreement was achieved between the FEM results and the experimental values of displacements at the dowel surface and the loaded block, especially for the first three positions (0, 20 mm, and 35 mm from the joint face). Slight differences can be observed with the last two positions (50 mm and 80 mm from the joint face). This might be due to a small movement of the sensors, or due to any small rotation of the block that may have occurred during the test. Also, the presence of air-voids and voids due to shrinkage of the concrete during hardening may cause a small variation. The validation of the numerical results with the experimental results is shown in Figure 6.1 and Table 6.2. The finite-element results show a closer prediction for the experimental
response than that of the Timoshenko’s approach which has been presented in Chapter Three of this study.

Figure 6.1. Simulation of the load-deflection response of a GFRP dowel for the specimen GWH at the following distances from the joint face: (a) Zero; (b) 20 mm; (c) 35 mm; (d) 50 mm; (e) 80 mm; (f) RD of loaded to unloaded side of blocks
Table 6.2. Comparison of ultimate deflection (mm) from the FEM and experimental tests

<table>
<thead>
<tr>
<th>Distance from joint face</th>
<th>GNH Expt</th>
<th>GNH FEM</th>
<th>GWH Expt</th>
<th>GWH FEM</th>
<th>GWL Expt</th>
<th>GWL FEM</th>
<th>GNL Expt</th>
<th>GNL FEM</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 mm</td>
<td>0.462</td>
<td>0.487</td>
<td>0.639</td>
<td>0.695</td>
<td>1.17</td>
<td>1.158</td>
<td>0.528</td>
<td>0.448</td>
</tr>
<tr>
<td>20 mm</td>
<td>0.256</td>
<td>0.224</td>
<td>0.311</td>
<td>0.382</td>
<td>0.458</td>
<td>0.502</td>
<td>0.268</td>
<td>0.224</td>
</tr>
<tr>
<td>35 mm</td>
<td>0.135</td>
<td>0.112</td>
<td>0.172</td>
<td>0.180</td>
<td>0.346</td>
<td>0.262</td>
<td>0.150</td>
<td>0.116</td>
</tr>
<tr>
<td>50 mm</td>
<td>-0.004</td>
<td>-0.031</td>
<td>0.077</td>
<td>0.047</td>
<td>-0.244</td>
<td>0.085</td>
<td>0.046</td>
<td>0.038</td>
</tr>
<tr>
<td>80 mm</td>
<td>-0.001</td>
<td>-0.007</td>
<td>-0.005</td>
<td>-0.007</td>
<td>0.041</td>
<td>-0.011</td>
<td>0.007</td>
<td>0.009</td>
</tr>
<tr>
<td>Relative deflection</td>
<td>3.92</td>
<td>3.582</td>
<td>4.95</td>
<td>5.130</td>
<td>5.9</td>
<td>5.82</td>
<td>2.92</td>
<td>2.56</td>
</tr>
</tbody>
</table>

6.3.2. Bearing stress at the dowel-concrete interface

The finite element simulations for all the tested specimens were carried out and the bearing stress at the face of the joint of the specimen with the GFRP dowel was compared with that of the corresponding specimen of the steel dowel at the end of elastic range (16 kN load). This limit (16 kN or 40% of ultimate load) was selected, as explained in §3.5.1, to represent the elastic limit of concrete behaviour. Figure 6.2 shows that the hardening zone of concrete behaviour (for dowel-concrete interface) started around this load value for the specimens with less load carrying capacity (e.g., Specimen GNL).

From the numerical simulation, it was possible to visualize the stress distribution at the joint face and the stress localization underneath the dowel bar (see Figure 6.3). Comparison of the contour plots on the left (GFRP) with the contour plots on the right (steel) demonstrates that the GFRP dowels result in lower stress concentrations when compared with the steel bars of similar flexural rigidity. The higher concentration of stress under the steel dowels may cause flaking of the material at the joint face. From Figure 6.3, it can also be observed that the maximum bearing stress of each specimen with a GFRP dowel is about 60-70% of the specimen with a steel dowel. For example, the maximum bearing stresses for GWL and SWL are 25.3 MPa and 42.7 MPa respectively.
The distribution of stress in the specimens with a GFRP dowel is quite different to the distribution of stress in the specimens with a steel dowel. The wide bearing area and the low transverse modulus of elasticity of the GFRP dowel produces less stress concentration in the concrete beneath it. For the epoxy-coated steel dowel bar the stress was concentrated over a narrow area underneath the bar. More distant regions received only a very small amount of stress compared to the dowel-concrete interface (see Figure 6.3). Contrary to the specimens with GFRP dowels, the bearing stress underneath the steel dowel bars exceeded the allowable bearing stress recommended by the ACI Committee 325 (1956), which is calculated according to Equation (2.16).

The finite element analysis shows that using a bigger diameter of GFRP dowel can extend the pavement’s life by providing sufficient load transfer with lower bearing stress at the dowel-concrete interface. High bearing stress deteriorates concrete and may ultimately create a gap beneath the dowel (dowel looseness) which reduces the load transfer between the adjacent slabs and produces pavement distress.
Figure 6.3. Bearing stress (MPa) (S22 – normal stress in the vertical direction) at the dowel-concrete interface for the steel and the GFRP dowels
6.3.3. Finite element modelling of a case study from literature

The experimental programme in the current study (investigation of the load-deflection response of dowel bars – in Chapter Three) only dealt with scaled (1:10) specimens. In order to suggest the design considerations for GFRP dowel bars, the scaling effect needed to be considered. Unfortunately, there is no experimental data in the literature for full-scale pavements with GFRP dowel bars at the expansion joints. However, a similar study can be found for steel dowel bars (Keeton and Bishop 1957). Their study checked the load transfer mechanism at the expansion joints of an airfield pavement. A numerical analysis was carried out to simulate the behaviour of this test data.

Keeton and Bishop (1957) investigated two concrete slabs 7620 mm × 4572 mm × 254 mm connected by 28.6 mm rounded epoxy-coated steel dowel bars spaced at 305 mm across 19 mm expansion joints. The length of these bars was 508 mm. The weakened plain joint (without dowel bars) was constructed at the mid-length of each slab. It was mentioned that the subgrade beneath the pavement was a compacted layer with modulus of subgrade reaction 0.0543 MPa/mm. The dowels included in this experiment were greased and capped from one side only. The capping ends were arranged alternatively at both sides. The shear force and bending moment were estimated from the readings of the strain gauges attached to the bars and embedded in the concrete slabs. One of these slabs was loaded at the middle with a rectangular tyre imprint of size 396.24 mm × 406.4 mm by a static load of 222 kN with a tyre pressure of 1.38 MPa. The material properties were adopted as mentioned in the experiment by Keeton and Bishop (1957). The modulus of elasticity and Poisson’s ratio for concrete and steel dowels were 28 GPa and 200 GPa, and 0.15 and 0.3 respectively.

The experimental results of Keeton and Bishop (1957) illustrated that the shear force in the dowel at 1524 mm from the central dowel was virtually negligible. Consequently, the width of the effective area of the slab was considered as 3353 mm [2× (1524+ half dowel’s spacing)]. Also, the length of the slab beyond the weakened joint had an insignificant effect on the behaviour of the dowel bars at the joint. Therefore, to save computational time, the dimensions considered for the slabs in the FEM were 3810 mm × 3353 mm × 254 mm (see Figure 6.4). The experiments of Keeton and Bishop (1957) were also simulated by Maitra et al. (2009) using the ANSYS finite element
programme. The numerical results from the current study (using ABAQUS) have also been compared with their results. Table 6.3 shows a comparison of the deflection of the loaded and unloaded sides of the dowels at the face of the joint. The numerical results from the present study show a reasonable agreement with the experimental results. Table 6.4 and Table 6.5 compare the maximum shear force and the bending moment in the central dowel (beneath the load) and in the nearest three dowels to it. Shear force and bending moment diagrams for the central dowel bar are shown in Figure 6.5 and Figure 6.6 respectively. Keeton and Bishop (1957) reported that there was a void below the centre of the slab at the joint and the size of it was about 1.016 mm (possibly due to slab-warping). The size of this gap was added to the computed deflection values from the current analysis and a similar approach was also taken by Maitra et al. (2009).

![Figure 6.4. Effective area in Keeton and Bishop’s test (1957)](image)

<table>
<thead>
<tr>
<th>Distance from central dowel (mm)</th>
<th>Loaded side (mm)</th>
<th>Unloaded side (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>2.15</td>
<td>2.26</td>
</tr>
<tr>
<td>305</td>
<td>2.07</td>
<td>2.21</td>
</tr>
<tr>
<td>610</td>
<td>1.95</td>
<td>2.08</td>
</tr>
<tr>
<td>915</td>
<td>1.82</td>
<td>1.95</td>
</tr>
<tr>
<td>1220</td>
<td>1.7</td>
<td>1.7</td>
</tr>
</tbody>
</table>
Table 6.4. Shear force (kN) at the face of the joint of the loaded slab

<table>
<thead>
<tr>
<th>Distance from central dowel (mm)</th>
<th>Current analysis by ABAQUS</th>
<th>Experimental Keeton and Bishop (1957)</th>
<th>Maitra et al. (2009) by ANSYS</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>23.5</td>
<td>22.9</td>
<td>28</td>
</tr>
<tr>
<td>305</td>
<td>17.8</td>
<td>15.7</td>
<td>19.7</td>
</tr>
<tr>
<td>610</td>
<td>10</td>
<td>10.9</td>
<td>8.4</td>
</tr>
<tr>
<td>915</td>
<td>4.6</td>
<td>4.5</td>
<td>3.5</td>
</tr>
<tr>
<td>1220</td>
<td>1.2</td>
<td>2.7</td>
<td>1.3</td>
</tr>
</tbody>
</table>

Table 6.5. Maximum bending moments in different dowels

<table>
<thead>
<tr>
<th>Distance from central dowel (mm)</th>
<th>Maximum Positive Bending Moment (kN.m)</th>
<th>Maximum Negative Bending Moment (kN.m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0.30</td>
<td>0.29</td>
</tr>
<tr>
<td>305</td>
<td>0.25</td>
<td>0.26</td>
</tr>
<tr>
<td>610</td>
<td>0.16</td>
<td>0.17</td>
</tr>
<tr>
<td>915</td>
<td>0.09</td>
<td>0.18</td>
</tr>
<tr>
<td>1220</td>
<td>0.05</td>
<td>0.13</td>
</tr>
</tbody>
</table>

Figure 6.5. Shear force in the central dowel
6.4. Parametric Study of the Proposed GFRP Dowels’ Design Considerations

A parametric study was conducted using an assembly of two concrete slabs of 4.5 m × 3.5 m resting on an elastic foundation with modulus of subgrade reaction of 0.054 MPa/mm. The dimensions were selected to represent an actual pavement, where the lane width would be about 3.5 m and the distance between contraction joints being 4.5 m. Five different slab thicknesses of 150 mm, 200 mm, 240 mm, 250 mm and 300 mm, with five different dowel bar spacing of 300 mm, 250 mm, 200 mm, 150 mm and 100 mm, and three joint widths of 3 mm, 6.35 mm, and 19 mm were considered (see Table 6.6). The length of the dowel bars for all these cases was 600 mm (UK Highway Agency 2009). The maximum size of the GFRP dowel used in the analysis was 38 mm based on the study by Porter et al. (2001).

A dual wheel load of 40 kN was placed at the corner of the slab (see Figure 6.7(a)) as equivalent to a single axle load according to the AASHTO (1993) guide and BS 7533-12 (2006) with a uniform pressure of 550 kPa. The size of the contact area was determined according to the procedure in Yoder and Witczak (1975) as shown in Equation (6.1).
where, \( a \): the radius of contact area, \( P \): the total applied load, and \( p \): the tyre pressure.

Equation (6.1) was used to calculate the radius of a circular contact area. Then the circular area was converted to the shape shown in Figure 6.7 (b) using Equation (6.2) (Yoder and Witczak 1975). However, to simplify the modelling process, this area was assumed as an equivalent rectangular area with a dimension of \( l \times 0.6l \) (for each tyre). The selected dimensions for each tyre were 245 mm × 148 mm.

\[
l = \frac{\text{Area}}{0.5227}
\]  

(6.2)

Since a standard wheel load was used in this study which meant that both the pavement and the dowels remained in the elastic range, the material properties were considered as linear elastic. The concrete properties were obtained according to EC2 (BS EN 1992-1-1 2004) corresponding to a concrete cube compressive strength of 30 MPa. The corresponding modulus of elasticity and Poisson’s ratio were 31000 MPa and 0.15, respectively. In this numerical model (parametric study) the slab was subjected to quasi-static load only rather than cyclic load to save computational time. Previous studies (Porter et al. 2001; Porter et al. 1993) had shown that the relative performance of GFRP and steel dowels is similar under static and cyclic loading (see Figure 6.8).
6.4.1. Evaluation criteria for GFRP dowels

Four criteria were used to evaluate the comparative performance of the GFRP dowels. Two of these criteria are involved with the effectiveness of the joint in transferring part of the wheel load to the adjacent slab. These two criteria, which were used in the current analysis, are load transfer efficiency (LTE) and transfer load efficiency (TLE). These criteria were explained in detail in Chapter Two of this study (§ 2.3).

The third criterion used in the evaluation process was the Relative Deflection (RD). It is a major governing factor when the shear deflection of GFRP dowels is much greater than that of steel dowels. High RD produces road usage problems such as joint faulting, pumping and uncomfortable riding. Therefore, it was assumed that when the RD is equal to or less than that of the steel dowel bars, the joint effectiveness will be sufficient for transferring the load. Its value was calculated directly from the finite element analysis by subtracting the deflection of the unloaded side from the deflection of the loaded side.

The fourth criterion was the bearing stress at the joint face of the critical dowel (beneath the wheel load) which is one of the important parameters for the dowel bars’ design. The bearing stress of the selected GFRP dowels’ combinations was compared with that of the steel dowels. The joint effectiveness criteria (LTE and TLE) were used to check the adequacy of the joint to transfer the load while the bearing stress and RD were used to select the appropriate combination for design purposes.
6.4.2. Proposed design considerations for the GFRP dowels

A number of finite element simulation runs were carried out with different combinations of pavement thickness, dowel diameter, dowel spacing and joint widths, as shown in Table 6.6. Design considerations are recommended after comparing the above mentioned parameters (LTE, TLE, RD and bearing stress) with that of the steel dowels according to the AASHTO (1993) guide and the UK Highways Agency specifications. The LTE and TLE showed that the GFRP dowels are adequate as load transfer devices in the transverse joints of JPCP for most combinations investigated in the current study as shown in Table 6.6. It was assumed that the GFRP dowels produce similar behaviour to that of the steel dowels in JPCP when the RD and bearing stress are equal to or less than that of the steel dowels. The results in Table 6.6 can also be used to evaluate the sensitivity to changes in any parameter. Several combinations of GFRP dowels and pavement are possible which will achieve a performance equivalent to pavements with steel dowels. Based on the data presented in Table 6.6, a recommended subset is shown in Table 6.7 for different pavement thicknesses and joint types. The combinations were selected by comparing with the recommended design for steel dowel bars as given in the AASHTO (1993) guide and by the UK Highways Agency. For the purpose of the comparison, the maximum contraction joint width was taken as 6.35 mm (0.25 inch) and the expansion joint as 19 mm (0.75 inch), as recommended by the AASHTO (1993) guide.

The bearing stress of the recommended combinations of the GFRP dowels’ diameter, spacing and slab thickness were compared with that of epoxy-coated steel dowel bars (see Table 6.8 and Figure 6.9). The comparison was also made for the allowable bearing stress according to the ACI formula (1956) presented in Equation (2.16). It can be seen that a significant reduction can be achieved in the maximum bearing stress at the dowel-concrete interface in the joint face for the proposed design considerations of GFRP dowels when compared with the equivalent steel dowels. The reduction in the bearing stress of the dowel bars in the surrounding concrete is one of the most important factors in augmented pavement life and reduction in maintenance cost. The bearing stress at the dowel-concrete interface significantly contributes to the void beneath the dowel bars.
The results of the bearing stress beneath the critical dowel, as shown in Table 6.8, confirmed that the bearing stress did not exceed the allowable bearing stress calculated according to Equation (2.16). It should be noted that very low spacing may result in the formation of cracks along the line of the dowels across the joint face.

Table 6.6. All combinations of dowel and pavement parameters for steel and GFRP dowels

<table>
<thead>
<tr>
<th>W (mm)</th>
<th>h (mm)</th>
<th>d (mm)</th>
<th>Type</th>
<th>Dowel Bars Spacing (mm)</th>
<th>% LTE</th>
<th>% TLE</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>300</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>250</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>200</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>150</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

| 19     | 15     | 19     | Steel | 300 | 75.6 | 39.9 | 0.244 |
|        | 20     | 25     |       | 250 | 80   | 42   | 0.148 |
|        | 25     | 25     |       | 200 | 79.1 | 42.3 | 0.13  |
|        | 25     | 32     |       | 150 | 83.7 | 42.6 | 0.098 |
|        | 30     | 38     |       |     | 85.5 | 43.9 | 0.074 |

| 6.3    | 15     | 19     | GFRP  | 300 | 71.6 | 39.7 | 0.236 |
|        | 20     | 25     |       | 250 | 66.3 | 37.9 | 0.222 |
|        | 25     | 25     |       | 200 | 71.8 | 41   | 0.181 |
|        | 25     | 32     |       | 150 | 69.6 | 40.9 | 0.160 |
|        | 30     | 38     |       |     | 67.2 | 41.5 | 0.147 |

| 3      | 15     | 19     | Steel | 300 | 82.6 | 40.8 | 0.167 |
|        | 20     | 20     |       | 250 | 82.2 | 41.7 | 0.132 |
|        | 24     | 20     |       | 200 | 81.1 | 42   | 0.12  |
|        | 20     | 25     |       | 150 | 85.2 | 42.8 | 0.108 |
|        | 25     | 25     |       |     | 84.4 | 43.3 | 0.094 |
|        | 25     | 32     |       |     | 87.2 | 43.6 | 0.075 |
|        | 30     | 38     |       |     | 88.2 | 44.5 | 0.059 |

<table>
<thead>
<tr>
<th>100 mm spacing</th>
<th>Spacing 100 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>19</td>
<td>30 38</td>
</tr>
<tr>
<td>GFRP</td>
<td>85 43 0.07</td>
</tr>
</tbody>
</table>
Table 6.7. Design considerations for the GFRP dowels based on comparison with AASHTO (1993) guide and UK Highway Agency Requirements

<table>
<thead>
<tr>
<th>Pavement thickness, ( h ) (mm)</th>
<th>Expansion joints</th>
<th>Contraction joints</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Diameter ( d ) (mm)</td>
<td>Spacing ( s ) (mm)</td>
</tr>
<tr>
<td>150</td>
<td>32</td>
<td>300</td>
</tr>
<tr>
<td>200</td>
<td>32</td>
<td>150</td>
</tr>
<tr>
<td>200</td>
<td>38</td>
<td>200</td>
</tr>
<tr>
<td>250</td>
<td>38</td>
<td>150</td>
</tr>
<tr>
<td>300</td>
<td>38</td>
<td>100</td>
</tr>
</tbody>
</table>

Comparison with UK Highway Agency specifications

- \( h < 239 \): 38, 250, 38, 300
- \( h > 239 \): 38, 150, 38, 250

Table 6.8. Bearing stress evaluation for the recommended design considerations of GFRP dowels

<table>
<thead>
<tr>
<th>Dowel bar type</th>
<th>Pavement thickness ( h ) (mm)</th>
<th>Allowable Bearing stress (MPa) according to Equation (2.16)</th>
<th>Expansion joints</th>
<th>Contraction joints</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Diameter ( d ) (mm)</td>
<td>Spacing ( s ) (mm)</td>
</tr>
<tr>
<td>Steel</td>
<td>150</td>
<td>32.5</td>
<td>19</td>
<td>300</td>
</tr>
<tr>
<td>Steel</td>
<td>200</td>
<td>30.2</td>
<td>25</td>
<td>300</td>
</tr>
<tr>
<td>Steel</td>
<td>250</td>
<td>27.4</td>
<td>32</td>
<td>300</td>
</tr>
<tr>
<td>Steel</td>
<td>300</td>
<td>25</td>
<td>38</td>
<td>300</td>
</tr>
<tr>
<td>GFRP</td>
<td>150</td>
<td>27.4</td>
<td>32</td>
<td>300</td>
</tr>
<tr>
<td>GFRP</td>
<td>200</td>
<td>27.4</td>
<td>32</td>
<td>150</td>
</tr>
<tr>
<td>GFRP</td>
<td>200</td>
<td>25</td>
<td>38</td>
<td>200</td>
</tr>
<tr>
<td>GFRP</td>
<td>250</td>
<td>25</td>
<td>38</td>
<td>150</td>
</tr>
<tr>
<td>GFRP</td>
<td>300</td>
<td>25</td>
<td>38</td>
<td>100</td>
</tr>
</tbody>
</table>
Figure 6.9. Bearing stress (MPa) at the face of joints underneath the critical dowels. The abbreviations are used to define the different cases as follows. The first letter refers to dowel type: GFRP or steel; the second (number) refers to the joint width (in mm); the third number refers to the slab thickness (in mm); the fourth number refers to the spacing of the dowel bars (in mm); and the last number refers to the dowel bar diameter (in mm).
6.5. Summary and Conclusions

A detailed numerical simulation was conducted for a dowelled-jointed concrete pavement containing GFRP and steel dowels. The following conclusions can be drawn from the numerical analysis carried out in this chapter.

- The numerical results produced a good agreement with the experiments. From the finite element simulation it was possible to visualise the bearing stress distribution underneath the dowel at the joint face. A great reduction in bearing stress can be achieved by using GFRP dowels with equivalent rigidity to that of steel dowels. The reduced and uniform bearing stress obtained by using GFRP dowels has a significant effect on minimizing the dowel looseness problem and extending pavement life. The numerical analysis was also used in formulating design considerations for GFRP dowels.

- Design considerations have been developed for the GFRP dowel bars with different combinations of pavement thickness, dowel diameter and joint widths. LTE, TLE, RD and bearing stress at the joint face for GFRP dowels were compared with those of epoxy-coated steel dowel bars according to the AASHTO (1993) guide and the UK Highways Agency specifications. The results showed that GFRP dowels are sufficient for load transfer according to LTE and TLE for most combinations of parameters investigated. The RD and bearing stress were used as key parameters for comparing the performance of different combinations of a pavement system. It was assumed that the GFRP dowels behave similarly to steel dowel bars when the RD and bearing stress of the JPCP fitted with GFRP dowels are equal to or less than that of steel dowels.

- The numerical analysis (as well as the experimental investigation in Chapter Three of this study) provided a good overview of the behaviour of GFRP dowels at expansion joints. The parametric study using numerical analysis helped in producing the design considerations.
CHAPTER SEVEN
NUMERICAL INVESTIGATION OF COMBINED EFFECT OF DOWEL MISALIGNMENT AND WHEEL LOAD

7.1. Introduction

This chapter introduces a detailed numerical investigation into the combined effect of dowel misalignment and wheel load on the performance of dowel bars in JPCP. It also presents an investigation into the suitability of GFRP dowels as an alternative material to epoxy-coated steel dowels. The numerical analysis involves investigation of 111 misalignment cases for steel and GFRP dowel bars. The numerical results include the pull-out load required to open a joint, the concrete deterioration associated with each misalignment case, the stress-strain distribution due to wheel load and dowel misalignment, increase in dowel looseness (DL) and decrease in load transfer efficiency (LTE) due to the opening and closing of the transverse joints of JPCP.

7.2. Importance of Numerical Simulation

A numerical simulation was carried out to expand the scope of the experimental investigation by including all practical cases of dowel misalignment and to give an insight into the stress, strain and damage in the surrounding concrete pavement. These stresses are extremely difficult to assess experimentally without affecting the measurements or damaging the gauges. The numerical simulation can easily show the distribution of stress/strain due to the combined effect of dowel misalignment and wheel load. In addition, it provides an indication of the damaged concrete volumes due to dowel misalignment. Consequently, a clear visualisation can be obtained on the severity of each misalignment case and the deterioration associated with it.
7.3. Parameters Included in Numerical Simulation

The experimental investigation reported in Chapter Four addressed the dowel misalignment problem and its effect on the DL and LTE. However, a full understanding of the problem cannot be achieved unless a detailed investigation of all the parameters affecting dowel misalignment is carried out. This investigation is difficult to be conducted experimentally for all possible cases due to limited time and resources and technical difficulties. Therefore, a comprehensive numerical simulation has been carried out to consider these parameters using the validated model with the experimental results. These parameters are described below:

- Dowel bar types: the current investigation involved the conventional epoxy-coated steel dowel bars (S) and GFRP dowels (G) as an alternative material.
- Dowel bar orientations: three orientations were considered - Vertical (V), Horizontal (H) and Combined (C) (vertical and horizontal).
- Misalignment types: three possible misalignment types were investigated - Uniform (U), Non uniform (N) and Partial misalignment (P).
- Misalignment magnitudes: the simulation involved investigation of four misalignment magnitudes 6.25 mm (1), 12.5 mm (2), 19 mm (3) and 25 mm (4) per half length of the dowel bar.

A total of 111 misalignment cases were investigated for specimens containing two steel dowel bars, specimens containing two GFRP dowel bars and specimens containing three GFRP dowel bars. These cases are shown in Table 7.1, Table 7.2, and Table 7.3. Nomenclature of the specimens can be found in §4.3, Table 4.1. The shaded cells refer to the specimens which were investigated experimentally.
Table 7.1. Simulation matrix for specimens containing two steel dowel bars

<table>
<thead>
<tr>
<th>Specimen Dimensions</th>
<th>Number of dowels and Misalignment type</th>
<th>Specimens Code</th>
<th>Orientation of the dowel bars</th>
<th>Magnitude of misalignment in mm per half of the dowel length</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 slabs each one (450 × 900 × 200) mm</td>
<td>2 (Uniform)</td>
<td>SA2</td>
<td>Aligned</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SV2U1</td>
<td>Vertical</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SV2U2</td>
<td></td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SV2U3</td>
<td></td>
<td>19</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SV2U4</td>
<td></td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SH2U1</td>
<td>Horizontal</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SH2U2</td>
<td></td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SH2U3</td>
<td></td>
<td>19</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SH2U4</td>
<td></td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SC2U1</td>
<td>Combined</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SC2U2</td>
<td></td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SC2U3</td>
<td></td>
<td>19</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SC2U4</td>
<td></td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>2 (Non-Uniform)</td>
<td>SV2N1</td>
<td>Vertical</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SV2N3</td>
<td></td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SV2N4</td>
<td></td>
<td>19</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SH2N1</td>
<td>Horizontal</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SH2N2</td>
<td></td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SH2N3</td>
<td></td>
<td>19</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SH2N4</td>
<td></td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SC2N1</td>
<td>Combined</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SC2N2</td>
<td></td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SC2N3</td>
<td></td>
<td>19</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SC2N4</td>
<td></td>
<td>25</td>
</tr>
<tr>
<td></td>
<td>2 (Partial- One Bar Misaligned)</td>
<td>SV2P1</td>
<td>Vertical</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SV2P2</td>
<td></td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SV2P3</td>
<td></td>
<td>19</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SV2P4</td>
<td></td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SH2P1</td>
<td>Horizontal</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SH2P2</td>
<td></td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SH2P3</td>
<td></td>
<td>19</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SH2P4</td>
<td></td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SC2P1</td>
<td>Combined</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SC2P2</td>
<td></td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SC2P3</td>
<td></td>
<td>19</td>
</tr>
<tr>
<td></td>
<td></td>
<td>SC2P4</td>
<td></td>
<td>25</td>
</tr>
</tbody>
</table>
Table 7.2. Simulation matrix for specimens containing two GFRP dowel bars

<table>
<thead>
<tr>
<th>Specimen Dimensions</th>
<th>Number of dowels and Misalignment type</th>
<th>Specimens Code</th>
<th>Orientation of the dowel bars</th>
<th>Magnitude of misalignment in mm per half of the dowel length</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 slabs each one (450 × 900 ×200) mm</td>
<td>2 (Uniform)</td>
<td>GA2</td>
<td>Aligned</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GV2U1</td>
<td>Vertical</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GV2U2</td>
<td></td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GV2U3</td>
<td></td>
<td>19</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GV2U4</td>
<td></td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GH2U1</td>
<td>Horizontal</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GH2U2</td>
<td></td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GH2U3</td>
<td></td>
<td>19</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GH2U4</td>
<td></td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GC2U1</td>
<td>Combined</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GC2U2</td>
<td></td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GC2U3</td>
<td></td>
<td>19</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GC2U4</td>
<td></td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GV2N1</td>
<td>Vertical</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GV2N2</td>
<td></td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GV2N3</td>
<td></td>
<td>19</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GV2N4</td>
<td></td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GH2N1</td>
<td>Horizontal</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GH2N2</td>
<td></td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GH2N3</td>
<td></td>
<td>19</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GH2N4</td>
<td></td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GC2N1</td>
<td>Combined</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GC2N2</td>
<td></td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GC2N3</td>
<td></td>
<td>19</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GC2N4</td>
<td></td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GV2P1</td>
<td>Vertical</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GV2P2</td>
<td></td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GV2P3</td>
<td></td>
<td>19</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GV2P4</td>
<td></td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GH2P1</td>
<td>Horizontal</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GH2P2</td>
<td></td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GH2P3</td>
<td></td>
<td>19</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GH2P4</td>
<td></td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GC2P1</td>
<td>Combined</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GC2P2</td>
<td></td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GC2P3</td>
<td></td>
<td>19</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GC2P4</td>
<td></td>
<td>25</td>
</tr>
</tbody>
</table>
Table 7.3. Simulation matrix for specimens containing three GFRP dowel bars

<table>
<thead>
<tr>
<th>Specimen Dimensions</th>
<th>Number of dowels and Misalignment type</th>
<th>Specimens Code</th>
<th>Orientation of the dowel bars</th>
<th>Magnitude of misalignment in mm per half of the dowel length</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 slabs, each one (450 × 1200 × 200) mm</td>
<td>3 (Uniform)</td>
<td>GA3</td>
<td>Aligned</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GV3U1</td>
<td>Vertical</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GV3U2</td>
<td></td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GV3U3</td>
<td></td>
<td>19</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GV3U4</td>
<td></td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GH3U1</td>
<td>Horizontal</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GH3U2</td>
<td></td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GH3U3</td>
<td></td>
<td>19</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GH3U4</td>
<td></td>
<td>25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GC3U1</td>
<td>Combined</td>
<td>6.25</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GC3U2</td>
<td></td>
<td>12.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GC3U3</td>
<td></td>
<td>19</td>
</tr>
<tr>
<td></td>
<td></td>
<td>GC3U4</td>
<td></td>
<td>25</td>
</tr>
<tr>
<td>3 (Non-Uniform)</td>
<td>GV3N1</td>
<td>Vertical</td>
<td>6.25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GV3N2</td>
<td></td>
<td>12.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GV3N3</td>
<td></td>
<td>19</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GV3N4</td>
<td></td>
<td>25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GH3N1</td>
<td>Horizontal</td>
<td>6.25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GH2N2</td>
<td></td>
<td>12.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GH3N3</td>
<td></td>
<td>19</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GH3N4</td>
<td></td>
<td>25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GC3N1</td>
<td>Combined</td>
<td>6.25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GC3N2</td>
<td></td>
<td>12.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GC3N3</td>
<td></td>
<td>19</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GC3N4</td>
<td></td>
<td>25</td>
<td></td>
</tr>
<tr>
<td>3 (Partial-One Bar Misaligned)</td>
<td>GV3P1</td>
<td>Vertical</td>
<td>6.25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GV3P2</td>
<td></td>
<td>12.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GV3P3</td>
<td></td>
<td>19</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GV3P4</td>
<td></td>
<td>25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GH3P1</td>
<td>Horizontal</td>
<td>6.25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GH3P2</td>
<td></td>
<td>12.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GH3P3</td>
<td></td>
<td>19</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GH3P4</td>
<td></td>
<td>25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GC3P1</td>
<td>Combined</td>
<td>6.25</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GC3P2</td>
<td></td>
<td>12.5</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GC3P3</td>
<td></td>
<td>19</td>
<td></td>
</tr>
<tr>
<td></td>
<td>GC3P4</td>
<td></td>
<td>25</td>
<td></td>
</tr>
</tbody>
</table>
7.4. Results and discussion

7.4.1. Verifying numerical results with current experimental results

As mentioned earlier, the FE analyses were carried out to overcome the limitations and difficulties of the experimental investigation. The pull-out loads from the 3D FEM were compared with the experimental results to validate the FEM as shown in Figure 7.1.

The FE results agreed fairly well with the experimental results for all the aligned eight cases which were investigated. However, differences can be observed in specimens such as GH2N4 that had high level of misalignment. The FE results did not match at all at the start of debonding of the dowel from the concrete. In the experimental specimen, high bearing stress due to misalignment may have caused localised damage which consequently affected the value of the coefficient of friction. Whereas in the FE model, the friction coefficient was maintained at a constant value during the whole analysis, irrespective of any interfacial damage. Another possibility is that any small variation in misalignment between two dowels may have affected the pull-out load, causing small tilting of the slabs during the movement due to sticking of one dowel more than the other.

In case of Specimen GH2N4 significant cracks were observed in the test specimens and failure occurred due to joint lockup. The pull-out load from the FE simulation agrees with the experimental pull-out load especially during crack initiation and failure as shown in Figure 7.1 (e). Also, the FE simulation showed a good prediction of the crack pattern and the occurrence of failure as shown in Figure 7.2. Visualisation of the pattern and location of the cracks at different load levels makes the numerical simulation an effective tool for analysis. The model also provides the damaged volume of concrete which is difficult to estimate experimentally. The damaged volume gives a clear insight on the effect of dowel misalignment on the surrounding concrete and the stresses associated with it. Also, it helps to quantify the suitability of using GFRP dowels in terms of reducing the detrimental effect of dowel misalignment on JPCP.
Figure 7.1. Comparison of pull-out loads from the experimental test with the FEM: (a) SA2; (b) SV2N2; (c) GA2; (d) GV2N2; (e) GH2N4; (f) GA3; (g) GH3N2; (h) GV3P2
Figure 7.2. Simulation of concrete failure (specimen GH2N4)
Table 7.4 presents the results of a comparison of the joint face deflection from the FE analyses with the experimental investigations for all the tested specimens. The DL was calculated as the difference in RD before and after the pull-out and push-back of the slabs. The results show a close prediction for the joint face deflections by the FE simulation compared with that of the experimental investigation. It can be observed from Table 7.4 that the joint face deflections from the FE results increase after joint movement in a similar way to that in the experimental results. This response reflects the ability of the FE model to represent the concrete deformation and the enlargement of the dowel bar socket (dowel looseness) due to dowel misalignment.

Table 7.4. Comparison of joint face deflection (mm) from the experimental test with the FEM

<table>
<thead>
<tr>
<th>Specimen code</th>
<th>Before pull-out</th>
<th>After pull-out</th>
<th>DL Expt.</th>
<th>DL FEM</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Experimental</td>
<td>FEM</td>
<td>Experimental</td>
<td>FEM</td>
</tr>
<tr>
<td></td>
<td>$d_l$</td>
<td>$d_u$</td>
<td>RD</td>
<td>$d_l$</td>
</tr>
<tr>
<td>SA2</td>
<td>2.29</td>
<td>2.13</td>
<td>0.16</td>
<td>2.26</td>
</tr>
<tr>
<td>SV2N2</td>
<td>2.45</td>
<td>2.28</td>
<td>0.17</td>
<td>2.27</td>
</tr>
<tr>
<td>GA2</td>
<td>2.1</td>
<td>2.01</td>
<td>0.09</td>
<td>2.1</td>
</tr>
<tr>
<td>GV2N2</td>
<td>2.21</td>
<td>2.08</td>
<td>0.13</td>
<td>2.41</td>
</tr>
<tr>
<td>GH2N4</td>
<td>1.93</td>
<td>1.84</td>
<td>0.09</td>
<td>1.92</td>
</tr>
<tr>
<td>GA3</td>
<td>1.8</td>
<td>1.69</td>
<td>0.11</td>
<td>1.68</td>
</tr>
<tr>
<td>GV3P1</td>
<td>1.93</td>
<td>1.85</td>
<td>0.08</td>
<td>1.87</td>
</tr>
<tr>
<td>GH3N2</td>
<td>1.93</td>
<td>1.84</td>
<td>0.09</td>
<td>1.92</td>
</tr>
</tbody>
</table>

7.4.2. Comparison of different misalignment cases of epoxy-coated steel dowels

7.4.2.1. Horizontal misalignment

Figure 7.3 (a-c) presents the effect of different types and magnitudes of horizontal misalignment. The comparison involves three misalignment types: non-uniform where the dowels tilt in the opposite directions, uniform where the dowels tilt in the same directions, and partial where only one dowel is tilted. Also, it includes four misalignment magnitudes (1) 6.25 mm, (2) 12.5 mm, (3) 19 mm and (4) 25 mm per half length of the dowel bar.

This tilting may occur due to tilt or misplace configuration of dowels in the basket, or the movement of the basket itself during the casting of concrete or both (see Figure 2.2).
As shown in Figure 7.3 (a), the misalignment magnitude has an insignificant effect at the initial stage of slipping of the dowel bars. For all misalignment cases, the slipping starts at about 2.5 kN. As the joint opening increases the load per dowel bars rapidly increases. The increase in dowel load is related to the misalignment magnitude where it is higher for higher misalignment magnitude. The transverse interaction between the dowel and the concrete is the source of this augmentation. The transverse interaction initiates due to the contact pressure between the dowel bar and the concrete pavement. Since the contact pressure is a function of the bearing stress of the dowel bar on the surrounding concrete, higher contact pressure is obtained for higher misalignment magnitude. Vertical deflection of the slabs for specimen SH2N3 during the pull-out step is shown in Figure 7.4. The slabs as a whole remain horizontal. The vertical deflections mostly occur at the dowel-concrete interface only.

The current study shows that a higher pull-out load is required for misaligned steel dowels compared to the previous work publicised in the literature (Prabhu et al. 2007). This is because the flexible base used in this investigation allowed for dowel bar deflection due to slab self-weight; this factor was ignored in other research. The current results fall within the range of axial forces in dowels observed in a field investigation (Shoukry et al. 2003).

The vertical drops in the curves shown in Figure 7.3 (a) are associated with concrete spalling. The reduction in the load is accompanied by an excessive increase in plastic strain in tension and compression which reflects a significant cracking and crushing of the concrete surrounding the dowel bars. The damage and associated load-drop occur at a smaller joint opening for the specimens that have higher dowel misalignment such as SH2N4 and SH2N3 while the damage largely remains localized for the smaller misalignment magnitudes such as SH2N1 and the reduction in the load only occurs at high joint-opening. The drop in the load for specimen SH2N4 started at 4 mm joint opening then the final softening happened at 4.73 mm whereas for specimen SH2N3 the concrete spalling happened at 5.33 mm followed by tensile softening at 6.35 mm. The contact pressure between the dowel bar and the surrounding concrete significantly decreases after that load-drop as shown in Figure 7.5 (b) and (c). Figure 7.5 (b) and (c) show the contact pressure distribution at the developed outer surface of the dowel bar. These figures were plotted by writing a code using Matlab (2010) programme according
to the contact pressure data obtained from ABAQUS output file. The horizontal axis of these plots represents both sides of dowel bars normalised with respect to embedded length of the dowel bar, whereas the vertical axis represents the circumference of the dowel bar normalised with respect to half circumferential length. Figure 7.5 (a) illustrates a section of slab with horizontally misaligned dowels and shows the actual distribution for the contact pressure at the dowel-contact interface. The reduction in contact pressure can be linked to crack initiations and deterioration at the dowel-concrete interface. In spite of the drop in load for specimen SH2N2, the redistribution of stress allowed the simulation to continue. This scenario is in contrast to specimens SH2N4 and SH2N3 where the excessive stress and strain led to stopping the simulation from running after the minimum increment size had been reached.

The results of the horizontally uniform dowel misalignment cases in Figure 7.3 (b) show that the pull-out load increases with an increase in dowel misalignment magnitude. It can be observed from the results that the pull-out load is lower for uniform misalignment than for non-uniform misalignment especially for specimens of higher misalignment magnitude (SH2U3, SH2U4, SH2N3 and SH2N4). Uniform misalignment causes less concrete spalling (in terms of load-drop) than a similar magnitude of non-uniform misalignment, as shown in Figure 7.3 (a). Only specimen SH2U4 had a significant drop in the load and tensile failure. The specimens SH2U3 and SH2U2 showed slight drops in load due to concrete spalling at the joint opening of 7.6 mm and 8.7 mm respectively. However, specimens with uniformly misaligned dowels exhibited greater joint opening before the concrete spalling compared with the corresponding specimens of non-uniform misalignment.

For the partial misalignment cases shown in Figure 7.3 (c), the average pull-out load per dowel is less when compared to the corresponding specimens of uniform and non-uniform dowel misalignment in Figure 7.3 (a) and Figure 7.3 (b) because only one dowel bar is misaligned. In spite of the average load being less than that in the case of uniform misalignment, the load-drop was seen for all specimens except SH2P1 due to the stress concentration at the misaligned dowel. A significant load-drop for specimen SH2P4 occurred at joint opening of 4.4 mm. For specimens SH2P3 and SH2P2, a significant load-drop occurred at joint openings of 6 mm and 8.7 mm respectively. A
drop in the pull-out load was noticed at the smaller joint opening when compared with the corresponding specimens of uniform misaligned dowels due to stress concentration.

Figure 7.3. Comparison of different types and magnitudes of horizontal misalignment for specimens containing steel dowel bars: (a) Non-uniform; (b) Uniform; (c) Partial

Figure 7.4. Vertical deflection of the slabs during the opening of the joint for SH2N3
Figure 7.5. Distribution of contact pressure (in MPa) at the dowel-concrete interface for specimen SH2N4: (a) Horizontal section of the slab-dowel assembly at joint opening 4 mm; and on the developed outer surface of Dowel-2 at joint opening (b) 4 mm, (c) 7 mm

7.4.2.2. Vertical misalignment

Figure 7.6 (a-c) illustrates the results of the comparison among three types of vertical orientations of misaligned dowels: (a) non-uniform; (b) uniform, and (c) partial. For non-uniform vertical misalignments the pull-out loads slightly increase with an increase in the misalignment magnitude (see Figure 7.6 (a)). The increase of dowel misalignment from 6.25 mm to 25 mm for specimens SV2N1 and SV2N4 add only 4 kN for the maximum average pull-out load required to open the joint. Also, no significant drop or softening is observed in the pull-out load for all the specimens.

Figure 7.6 (b) presents the results for the specimens with different magnitudes of uniform vertical misalignment. Both dowel bars are parallel and tilt in a vertical direction. These results show very small differences in the pull-out load. Also, no drop in the pull-out load is observed. Figure 7.6 (c) shows the results of the comparison
between different magnitudes of partial misalignment. The results show the maximum difference in average pull-out load is about 2 kN. Likewise with other vertical misalignments types no drop can be seen in the pull-out loads required to open the joint.

Figure 7.6. Comparison of different types and magnitudes of vertical misalignment for the specimens containing steel dowel bars: (a) Non-uniform; (b) Uniform; (c) Partial

All types of vertical misalignments show significantly lower pull-out load when compared with the corresponding specimens of horizontal misalignment. The most likely cause of the lower pull-out load is the slab-base separation and the relative deflection (RD) across the joint during the opening of the joint as shown in Figure 7.7. This uplift movement is linked to the vertical orientations of the dowel bars and materializes due to the “wedge effect” of the inclined bars. Therefore, its value is higher for uniform misalignment than it is for non-uniform vertical misalignment (see Figure 7.7). As a consequence of this uplift, significant variations and reductions occurred in the contact pressure as shown in Figure 7.8. Consequently, this produced a significant reduction in the pull-out load required to open the joint especially for uniform misalignment. This behaviour had not been observed in previous research (Prabhu et al. 2007, 2006; Tayabji 1986) since the slab-base separation was not allowed. Therefore,
no significant change was observed between the vertical and horizontal misalignment in their studies.

Figure 7.7. Uplift of the slab and the RD across the joint during the opening of the joint for the specimens containing vertically misaligned dowels: (a) SV2N4, (b) SV2U4

Figure 7.8. Distribution of contact pressure (in MPa) at the dowel-concrete interface:
(a) SV2N4; (b) SV2U4

7.4.2.3. Combined misalignment

Figure 7.9 (a-c) illustrates the results of the pull-out loads for the different cases of combined misalignment. Figure 7.9 (a) shows the results of the pull-out loads for combined non-uniform misalignment, where each dowel is tilted in horizontal and vertical planes at the same time. The tilt directions for both the vertical and horizontal orientations of the dowel are opposite to each dowel. Figure 7.9 (b) presents the results of the combined uniform dowel misalignment in which both dowels have the same vertical and horizontal tilts. Figure 7.9 (c) shows the results of the combined partial misalignment where only one dowel is tilted in both the vertical and horizontal planes and the other bar is aligned.

As shown in Figure 7.9 (a) all specimens are characterised by an increase of the pull-out loads with an increase in the misalignment magnitude. The results reveal that all specimens have load-drops due to high stress and deterioration of the surrounding
concrete pavement (concrete spalling). The pull-out loads are significantly increased with opening of the joint for specimens SC2N3 and SC2N4 due to joint lockup. The maximum load values are 27.9 kN and 30.5 kN for specimens SC2N3 and SC2N4 respectively. These values are observed at the joint opening of 6 mm, however, the joint opening severity and load-drops for specimen SC2N4 is noticeable at a lower joint opening of 3.6 mm. The loads start to show sudden drops at 3.6 mm for specimen SC2N4 which was due to concrete deterioration (spalling) surrounding the misaligned dowels. The concrete spalling in specimens SC2N3 and SC2N4 was followed by the final softening of the pull-out load at the joint opening of 6 mm as reflected by significant cracks in the surrounding concrete. Specimens SC2N1 and SC2N2 showed load-drops at the joint opening of 11 mm as an indication of concrete spalling.

For the specimens of combined uniform misalignment shown in Figure 7.9 (b) a similar trend can be observed for increase in the pull-out load with the misalignment magnitudes. An uplift movement occurred for all the specimens of combined dowel misalignment due to the vertical inclinations of the dowels as shown in Figure 7.10. Therefore, there is no significant increase in pull-out load for the combined misalignment (uniform and non-uniform) when compared with the horizontal misalignment shown in Figure 7.3, although the interaction between the dowel bars across the joint for the combined misalignment is more complicated than that for the horizontal misalignment due to the uplift movement. However, the uplift movement for the specimens of combined misaligned dowels (see Figure 7.10) is less than that of the vertical misalignment shown before in Figure 7.7. The pull-out loads for the specimens with combined uniform misaligned dowels is less than the corresponding specimens with the combined non-uniform misalignment. The parallelization of the dowel bars produces more uplift movement and variation in contact pressure in the specimens of the combined uniform misalignment than the combined non-uniform misalignment which consequently reduces the pull-out load. Only specimen SC2U4 had a significant load-drop and cracks at an approximately 4.2 mm joint opening. Significant load-drops due to concrete spalling surrounding the misaligned dowels could be observed for specimens SC2U3 and SC2U2 at joint openings of 5.75 mm and 8.25 mm respectively.

Figure 7.9 (c) presents the results of the combined partial dowel misalignment. The average pull-out loads are lower than the corresponding specimens for combined
uniform and combined non-uniform misalignment due to the fact that only one dowel in the specimens was misaligned. The drop of load due to concrete deterioration (spalling) occurred in specimen SC2P4 only at the joint opening of 4.2 mm.

Figure 7.9. Comparison of different types and magnitudes of combined misalignment for specimens containing steel dowel bars: (a) Non-uniform; (b) Uniform; (c) Partial

Figure 7.10. Uplift of the slab and the RD across the joint during the opening of the joint for the specimens containing combined misaligned dowels: (a) SC2N3, (b) SC2U3
7.4.3. Comparison of GFRP dowels with epoxy-coated steel dowel bars

7.4.3.1. Pull-out load and joint opening

7.4.3.1.1. Horizontal misalignment

The current study involved an exploration into using GFRP dowels as an alternative material for the conventional epoxy-coated steel dowel bars. The investigations involved the same parameters which were investigated for steel dowel bars whereby three dowel orientations were included in this simulation: horizontal, vertical and combined. Also, three misalignment types were included: non-uniform, uniform and partial. The investigation involved four misalignment magnitudes: (1) 6.25 mm, (2) 12.5 mm, (3) 19 mm, and (4) 25 mm per half length of the dowel bar.

The results for the horizontal misalignment cases are shown in Figure 7.11 (a-c). The curves show that the GFRP dowels need slightly higher pull-out load to debond from the concrete pavement due to the bigger diameter of these dowels (38 mm) when compared with the tested steel dowels (25 mm). However, the post-slip stages show significant increase in pull-out loads for the steel dowels as compared to the GFRP dowels.

For non-uniform misalignment, high pull-out loads are observed for the specimens having steel dowels (Figure 7.11 (a)). This is due to joint lockup created by the high frictional stress between the steel dowels and the surrounding concrete caused by misalignment and the higher surface irregularity of the steel dowels. Although the surface irregularity was not simulated by changing the surface roughness, its effect was considered in the longitudinal bond behaviour of the dowel-concrete interaction based on the experimental tests reported in Chapter Four of this study. Specimen SH2N1, with the lowest misalignment magnitude among steel dowels considered in the current investigation, shows higher pull-out loads than GFRP dowels with four-times misalignment magnitude. The pull-out load at the 6 mm joint opening is 16.5 kN for SH2N1, compared with 8.6 kN, 10.6 kN, 13.3 kN, and 16.25 kN for specimens GH2N1, GH2N2, GH2N3, and GH2N4 respectively.

A substantial improvement can be achieved by using GFRP dowels when considering the decrease in concrete deterioration (spalling). The results in Figure 7.11 (a) show that
no load-drop occurs due to concrete spalling for specimens GH2N1 and GH2N2. Specimen GH2N3 shows a small load-drop due to concrete spalling at the joint opening of 10.5 mm, while significant cracks and failure occur for specimen GH2N4 at the joint opening of 8.5 mm. For the steel dowels, only specimen SH2N1 does not show noticeable concrete spalling, while significant concrete spalling in specimen SH2N2 can be observed at the joint opening of 9.5 mm. Significant cracks and failure occur for specimens SH2N3 and SH2N4 at joint openings of 6.35 mm and 4.7 mm respectively.

It can be observed that, for the GFRP dowels, the concrete deterioration (spalling) occurs in specimens of high misalignment magnitudes GH2N3 and GH2N4 only; also significant cracking only occurs in specimen GH2N4. However, these distresses occur at larger joint openings and lower pull-out loads compared to the steel dowels. Spalling and cracking at the dowel-concrete interface take place due to the transverse interaction between the misaligned dowels and the concrete. This interaction arises from the frictional and bearing stress at the interface. The frictional stress for the GFRP dowels is much lower than that of the steel dowels due to very small coefficient of friction for the GFRP dowels and a small contact pressure due to a higher contact area on account of a bigger diameter and lower stiffness for GFRP dowels (see Figure 7.12). However, the bearing stress at a large joint opening for high misalignment magnitude could be the source of these distresses. Figure 7.12 shows a similar contact pressure distribution to that of steel dowel bar shown in Figure 7.5, however the maximum contact pressure value is about 40% of that of steel dowel bar.

Figure 7.11 (b) shows the results for uniform horizontal misalignment, in which significantly lower pull-out loads for the GFRP dowels are observed. The load-drop due to concrete spalling can be observed for specimen GH2U4 only at the joint opening of 11 mm. Whereas, for the steel dowels, the spalling can be seen for specimens SH2U2, SH2U3 and SH2U4 at the joint openings of 8.7 mm, 7.6 mm and 5.3 mm respectively. Also, specimen SH2U4 shows significant cracks and failure at the joint opening of 7.7 mm. Specimens of horizontally uniform misaligned dowels exhibit less distress compared with the corresponding specimens of non-uniform misaligned dowels.

For the partial misaligned dowel bars, the results in Figure 7.11 (c) indicate that the average pull-out load for the GFRP dowels in each specimen is less than that of the steel
dowels and they are less than the corresponding specimens with uniform or non-uniform misalignments. The results also show no load-drop for all the specimens having GFRP dowels except for GH2P4. This specimen exhibited significant concrete spalling at the joint opening of 9 mm due to stress concentration arising from interface bearing stress. However, this joint opening is greater than the realistic joint opening value according to the AASHTO (1993) guide.

Figure 7.11. Comparison of different types and magnitudes of horizontal misalignment of steel and GFRP dowel bars: (a) Non-uniform; (b) Uniform; (c) Partial
Figure 7.12. Distribution of contact pressure (in MPa) at the dowel-concrete interface at the joint opening of 6 mm: (a) SH2N1, (b) GH2N4

7.4.3.1.2. Vertical misalignment

Figure 7.13 (a-c) shows the results for vertically misaligned steel and GFRP dowels. It can be observed from Figure 7.13 (a) that the debonding of the dowel bars occurs at a similar load to that of the horizontal misalignment cases shown in Figure 7.11, but at the post-slip stage the increase in pull-out load is significantly lower than the horizontal misalignment cases. This can be linked to the effect of the uplift and the slab-base separation which has been described earlier in this chapter. However, for all misalignment magnitudes, the GFRP dowels show a significantly smaller pull-out load when compared with that of the steel dowel bars. The pull-out loads at the joint opening of 6 mm are 7.9 kN, 8.5 kN, 9.1 kN, and 9.75 kN for the specimens GV2N1, GV2N2, GV2N3 and GV2N4 respectively compared with 12.1 kN, 13.2 kN, 14.6 kN and 15.8 kN for the specimens SV2N1, SV2N2, SV2N3 and SV2N4 respectively. Concrete spalling can be seen in specimen SV2N4 when the joint opening exceeds 6 mm, whereas a smaller load-drop due to concrete spalling can be observed in specimen GV2N4 at a joint opening of 9.7 mm.

Figure 7.13 (b) compares the results of specimens having vertically uniform misaligned GFRP and steel dowel bars. The results indicate that the pull-out loads for all the specimens (steel or GFRP dowels) are unaffected by the increase in dowel misalignment magnitude. The constant pull-out load values for GFRP dowels in Figure 7.13 (b) is because of the uplift movement of the slabs during the pull-out process which keeps the contact pressure approximately constant (see Figure 7.14). Figure 7.14 shows a similar contact pressure value and distribution during the opening of joint, and the maximum value of contact pressure is significantly lower than that of horizontal misaligned dowels shown in Figure 7.12 (b). The maximum contact pressure for
vertically misaligned dowel is concentrated at the top and bottom of the dowel at the face of joint due to the vertical inclination of the dowel bar. The uplift movement (the RD across the joint) increases with an increase in misalignment magnitude as shown in Figure 7.15. Moreover, the bearing stresses of misaligned dowels are minimal due to the parallel nature of the uniformly misaligned dowels.

Figure 7.13 illustrates the results of the comparison of vertical partial misalignment for the steel and GFRP dowels. The results reveal a small increase in the pull-out load of the GFRP and steel dowels. Also, it shows a significant reduction in the pull-out load that can be achieved by using GFRP dowel bars. The results show no concrete spalling in all specimens with GFRP dowels whereas concrete spalling can be observed in specimen SV2P4 at the joint opening of 6.5 mm. Figure 7.13 shows no considerable augmentation in pull-out loads during the opening of the joint for all the specimens that had GFRP dowels.
Figure 7.14. Distribution of contact pressure (in MPa) at the dowel-concrete interface of specimen GV2U4 at the joint opening: (a) 1.5 mm, (b) 12 mm

Figure 7.15. Uplift of the slab and the RD across the joint during the opening of the joint for the specimens containing vertically misaligned dowels: (a) GV2U1, (b) GV2U4

7.4.3.1.3. Combined misalignment

Figure 7.16 (a-c) presents the results of a comparison between combined misaligned steel and GFRP dowels. The results of the combined non-uniform misalignment in Figure 7.16 (a) show that the pull-out loads for the steel dowels are significantly higher than those for the GFRP dowels. These loads are close to the pull-out loads in horizontally non-uniform misaligned dowels as mentioned earlier in this chapter of the current study. However, more concrete deterioration (spalling) occurred for the specimens having combined misaligned dowels (steel or GFRP) than that for horizontal and vertical misalignment. The load-drops due to concrete deterioration (spalling) were noticed at a smaller joint opening for both steel and GFRP dowels than in the case of the horizontal non-uniform misalignment. The effects of concrete spalling were noticeable on the pull-out loads of specimens GC2N3 and GC2N4 at the joint openings of 7.5 mm and 4.6 mm respectively. Significant cracks and failure occurred for specimen GC2N4 at the joint opening of 8.1 mm. The results show that GFRP dowels produce a substantial improvement in dowel bars’ performance in terms of reducing the pull-out load and concrete deterioration. The most likely cause of higher concrete distress.
without a significant increase in pull-out loads for combined misalignment compared with other misalignment types is the complex interaction between the dowel bars at the joint. The occurrence of the uplift movement (the RD across the joint) during the pull-out process keeps the pull-out loads similar to that of horizontal misalignment.

Figure 7.16 (b) illustrates the results of the comparison between combined uniform misaligned steel and GFRP dowels. It can be seen that the pull-out loads for the GFRP dowels are significantly less than that for the steel dowels. Only specimen GC2U4 shows a small load-drop due to concrete spalling at the joint opening of 9.9 mm whereas for the steel dowels, the spalling is observed at joint openings of 10.8 mm, 8.25 mm, 5.75 mm and 3.7 mm for specimens SC2U1, SC2U2, SC2U3 and SC2U4 respectively. Moreover, significant cracks and failure can be seen for SC2U4 at the joint opening of 4.2 mm.

Figure 7.16. Comparison of different types and magnitudes of the combined misalignment of steel and GFRP dowel bars: (a) Non-uniform; (b) Uniform; (c) Partial
The results of the comparison of partially misaligned steel and GFRP dowels are shown in Figure 7.16 (c). A similar trend for lower pull-out loads for the GFRP dowels than for the steel dowels is observed. Also, less damage can be seen for the GFRP dowels compared with the steel dowels. As a consequence of the concrete spalling, a small load-drop can be observed at joint openings of 8.5 mm and 6.1 mm for the specimens GC2P3 and GC2P4 respectively. Whereas, for the steel dowels, the concrete spalling occurs at the joint openings of 10.9 mm, 9.3 mm, 6.25 mm and 4.2 mm for SC2P1, SC2P2, SC2P3 and SC2P4 respectively.

Since the diameters of the investigated steel and GFRP dowels are not the same, a comparison has been carried out between the pull-out stresses for all the specimens of steel and GFRP dowels and for all types and magnitudes of dowel misalignment at the joint openings of 3 mm and 6 mm as shown in Figure 7.17. It can be observed that the pull-out stresses for the GFRP dowels are significantly lower than that for steel dowels for all misalignment types and magnitudes. There are no significant differences in the pull-out loads of the GFRP dowels having a misalignment magnitude up to 12.5 mm for all types of dowel misalignment. On the other hand, a significant increase in pull-out load can be observed with the increase in the misalignment magnitudes for the specimens that had steel dowel bars. This increase is more noticeable for the non-uniform and partial misalignment than for the uniform misalignment cases. Also, it is more noticeable for the horizontal and combined misalignment than for the vertical misalignment. Generally, using GFRP dowel bars produces an average reduction in normalised pull-out load (pull-out stress) of 51%, 39% and 39% for vertical, horizontal and combined misalignments respectively, at the joint opening of 3 mm.
7.4.3.2. Damage initiation: investigation and comparison between steel and GFRP dowels

Damage to, and deterioration of, concrete pavements due to dowel misalignment are difficult to estimate from experimental investigations. Although the effect of any damage can be indirectly observed through the drop in pull-out load, the size and distribution of that damage are difficult, if not impossible, to quantify. To address this issue, researchers usually recourse to numerical techniques (Prabhu et al. 2009, 2007). However, the previous studies have been limited to only identifying the initiation of damage and did not include the effect of damage plasticity. Quantification of the damaged volume and visualization of the damage locations are important steps in understanding the behaviour of concrete pavements with misaligned dowels. Unfortunately this has not been addressed before.

In the current study, a full non-linear analysis using concrete damage plasticity model was employed to obtain the opening behaviour including any effect due to concrete cracking, spalling or other damage. The degree of damage or distress in the concrete pavement was represented as a ratio of damaged volume to a control volume of
600 mm × 227.5 mm × 200 mm surrounding the dowel bars (see Figure 7.18 for control volume). This control region was selected because no damage was observed within the other regions of the slab. The damage ratios are determined by dividing the sum of the volumes of the damaged elements by the total control volume. Elements were assumed as damaged when they had a non-zero value of equivalent plastic strain in uniaxial tension (PEEQT) and the maximum plastic strain value was positive. Consequently, the tensile damage variable in ABAQUS (DAMAGET) for these elements was positive. DAMAGET is a non-decreasing quantity and refers to the tensile failure of the material. This variable is a function of the plastic strain of the material; when its value is more than zero it indicates the initiation of cracks at the concrete slab (SIMULIA 2010). It should be noted here that plastic strain components in tension (PEEQT) can also be used to assess the location of the damaged elements. Because excessive damage and plastic strain were observed in the concrete pavement slabs due to dowel misalignment as compared to that due to wheel load (see Figure 7.19 and Table 7.5 and Table 7.6), the damage ratios are presented here for the pull-out step only. Appendix A shows the damage and plastic strain initiation and variation for all loading steps at the face of the joint of the specimens that contained epoxy-coated steel dowel bars and the specimens that contained GFRP dowel bars for all horizontal misalignment cases.

Figure 7.18. Test specimen with highlighted control concrete volume
Figure 7.19: Tensile damage in the concrete surrounding dowel bars for specimens: (a) SH2N2; (b) GH2N2
Table 7.5. Maximum values of tensile plastic strain component (PEEQT) at each step of the analysis (see § 5.6.2 for steps definition)

<table>
<thead>
<tr>
<th>Step</th>
<th>SH2N1</th>
<th>SH2N2</th>
<th>SH2N3</th>
<th>SH2N4</th>
<th>GH2N1</th>
<th>GH2N2</th>
<th>GH2N3</th>
<th>GH2N4</th>
</tr>
</thead>
<tbody>
<tr>
<td>One</td>
<td>0.0019</td>
<td>0.0019</td>
<td>0.0020</td>
<td>0.0022</td>
<td>0.0004</td>
<td>0.0004</td>
<td>0.0004</td>
<td>0.0004</td>
</tr>
<tr>
<td>Three</td>
<td>0.0046</td>
<td>0.0325</td>
<td>0.0669</td>
<td>0.0849</td>
<td>0.0016</td>
<td>0.0041</td>
<td>0.0240</td>
<td>0.0280</td>
</tr>
<tr>
<td>Four</td>
<td>0.0047</td>
<td>0.0326</td>
<td></td>
<td></td>
<td>0.0016</td>
<td>0.0041</td>
<td>0.0240</td>
<td></td>
</tr>
<tr>
<td>Five</td>
<td>0.0047</td>
<td>0.0326</td>
<td></td>
<td></td>
<td>0.0016</td>
<td>0.0041</td>
<td>0.0240</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step</th>
<th>SH2U1</th>
<th>SH2U2</th>
<th>SH2U3</th>
<th>SH2U4</th>
<th>GH2U1</th>
<th>GH2U2</th>
<th>GH2U3</th>
<th>GH2U4</th>
</tr>
</thead>
<tbody>
<tr>
<td>One</td>
<td>0.0019</td>
<td>0.0023</td>
<td>0.0033</td>
<td>0.0033</td>
<td>0.0004</td>
<td>0.0004</td>
<td>0.0004</td>
<td>0.0004</td>
</tr>
<tr>
<td>Three</td>
<td>0.0042</td>
<td>0.0222</td>
<td>0.0605</td>
<td>0.0644</td>
<td>0.0010</td>
<td>0.0032</td>
<td>0.0545</td>
<td>0.0343</td>
</tr>
<tr>
<td>Four</td>
<td>0.0042</td>
<td>0.0222</td>
<td>0.0605</td>
<td>0.0644</td>
<td>0.0010</td>
<td>0.0032</td>
<td>0.0545</td>
<td>0.0343</td>
</tr>
<tr>
<td>Five</td>
<td>0.0044</td>
<td>0.0223</td>
<td>0.0611</td>
<td></td>
<td>0.0010</td>
<td>0.0032</td>
<td>0.0545</td>
<td>0.0343</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step</th>
<th>SH2P1</th>
<th>SH2P2</th>
<th>SH2P3</th>
<th>SH2P4</th>
<th>GH2P1</th>
<th>GH2P2</th>
<th>GH2P3</th>
<th>GH2P4</th>
</tr>
</thead>
<tbody>
<tr>
<td>One</td>
<td>0.0029</td>
<td>0.0029</td>
<td>0.0029</td>
<td>0.0029</td>
<td>0.0003</td>
<td>0.0003</td>
<td>0.0004</td>
<td>0.0004</td>
</tr>
<tr>
<td>Three</td>
<td>0.0044</td>
<td>0.0225</td>
<td>0.0266</td>
<td>0.0277</td>
<td>0.0006</td>
<td>0.0025</td>
<td>0.0477</td>
<td>0.0051</td>
</tr>
<tr>
<td>Four</td>
<td>0.0044</td>
<td>0.0225</td>
<td>0.0266</td>
<td>0.0277</td>
<td>0.0006</td>
<td>0.0026</td>
<td>0.0477</td>
<td>0.0051</td>
</tr>
<tr>
<td>Five</td>
<td>0.0044</td>
<td>0.0225</td>
<td>0.0266</td>
<td>0.0277</td>
<td>0.0006</td>
<td>0.0026</td>
<td>0.0477</td>
<td>0.0051</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step</th>
<th>SV2N1</th>
<th>SV2N2</th>
<th>SV2N3</th>
<th>SV2N4</th>
<th>GV2N1</th>
<th>GV2N2</th>
<th>GV2N3</th>
<th>GV2N4</th>
</tr>
</thead>
<tbody>
<tr>
<td>One</td>
<td>0.0027</td>
<td>0.0024</td>
<td>0.0024</td>
<td>0.0023</td>
<td>0.0003</td>
<td>0.0004</td>
<td>0.0004</td>
<td>0.0004</td>
</tr>
<tr>
<td>Three</td>
<td>0.0028</td>
<td>0.0025</td>
<td>0.0025</td>
<td>0.0024</td>
<td>0.0003</td>
<td>0.0004</td>
<td>0.0004</td>
<td>0.0004</td>
</tr>
<tr>
<td>Four</td>
<td>0.0028</td>
<td>0.0025</td>
<td>0.0025</td>
<td>0.0024</td>
<td>0.0003</td>
<td>0.0004</td>
<td>0.0004</td>
<td>0.0004</td>
</tr>
<tr>
<td>Five</td>
<td>0.0028</td>
<td>0.0026</td>
<td>0.0026</td>
<td>0.0025</td>
<td>0.0003</td>
<td>0.0004</td>
<td>0.0004</td>
<td>0.0004</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step</th>
<th>SV2U1</th>
<th>SV2U2</th>
<th>SV2U3</th>
<th>SV2U4</th>
<th>GV2U1</th>
<th>GV2U2</th>
<th>GV2U3</th>
<th>GV2U4</th>
</tr>
</thead>
<tbody>
<tr>
<td>One</td>
<td>0.0027</td>
<td>0.0028</td>
<td>0.0028</td>
<td>0.0028</td>
<td>0.0003</td>
<td>0.0003</td>
<td>0.0004</td>
<td>0.0004</td>
</tr>
<tr>
<td>Three</td>
<td>0.0028</td>
<td>0.0025</td>
<td>0.0025</td>
<td>0.0024</td>
<td>0.0003</td>
<td>0.0004</td>
<td>0.0004</td>
<td>0.0004</td>
</tr>
<tr>
<td>Four</td>
<td>0.0028</td>
<td>0.0025</td>
<td>0.0025</td>
<td>0.0024</td>
<td>0.0003</td>
<td>0.0004</td>
<td>0.0004</td>
<td>0.0004</td>
</tr>
<tr>
<td>Five</td>
<td>0.0028</td>
<td>0.0026</td>
<td>0.0026</td>
<td>0.0025</td>
<td>0.0003</td>
<td>0.0004</td>
<td>0.0004</td>
<td>0.0004</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step</th>
<th>SC2N1</th>
<th>SC2N2</th>
<th>SC2N3</th>
<th>SC2N4</th>
<th>GC2N1</th>
<th>GC2N2</th>
<th>GC2N3</th>
<th>GC2N4</th>
</tr>
</thead>
<tbody>
<tr>
<td>One</td>
<td>0.0029</td>
<td>0.0031</td>
<td>0.0036</td>
<td>0.0044</td>
<td>0.0003</td>
<td>0.0004</td>
<td>0.0005</td>
<td>0.0005</td>
</tr>
<tr>
<td>Three</td>
<td>0.0108</td>
<td>0.0145</td>
<td>0.0951</td>
<td>0.0926</td>
<td>0.0015</td>
<td>0.0336</td>
<td>0.0598</td>
<td>0.0533</td>
</tr>
<tr>
<td>Four</td>
<td>0.0199</td>
<td>0.0205</td>
<td></td>
<td></td>
<td>0.0015</td>
<td>0.0337</td>
<td>0.0598</td>
<td></td>
</tr>
<tr>
<td>Five</td>
<td>0.0199</td>
<td>0.0205</td>
<td></td>
<td></td>
<td>0.0015</td>
<td>0.0337</td>
<td>0.0598</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step</th>
<th>SC2U1</th>
<th>SC2U2</th>
<th>SC2U3</th>
<th>SC2U4</th>
<th>GC2U1</th>
<th>GC2U2</th>
<th>GC2U3</th>
<th>GC2U4</th>
</tr>
</thead>
<tbody>
<tr>
<td>One</td>
<td>0.0028</td>
<td>0.0027</td>
<td>0.0026</td>
<td>0.0023</td>
<td>0.0003</td>
<td>0.0003</td>
<td>0.0004</td>
<td>0.0004</td>
</tr>
<tr>
<td>Three</td>
<td>0.0076</td>
<td>0.0221</td>
<td>0.0261</td>
<td>0.0730</td>
<td>0.0012</td>
<td>0.0066</td>
<td>0.0072</td>
<td>0.0108</td>
</tr>
<tr>
<td>Four</td>
<td>0.0183</td>
<td>0.0221</td>
<td>0.0262</td>
<td></td>
<td>0.0012</td>
<td>0.0067</td>
<td>0.0072</td>
<td>0.0110</td>
</tr>
<tr>
<td>Five</td>
<td>0.0183</td>
<td>0.0221</td>
<td>0.0262</td>
<td></td>
<td>0.0012</td>
<td>0.0067</td>
<td>0.0072</td>
<td>0.0110</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step</th>
<th>SC2P1</th>
<th>SC2P2</th>
<th>SC2P3</th>
<th>SC2P4</th>
<th>GC2P1</th>
<th>GC2P2</th>
<th>GC2P3</th>
<th>GC2P4</th>
</tr>
</thead>
<tbody>
<tr>
<td>One</td>
<td>0.0027</td>
<td>0.0028</td>
<td>0.0029</td>
<td>0.0028</td>
<td>0.0003</td>
<td>0.0004</td>
<td>0.0005</td>
<td>0.0004</td>
</tr>
<tr>
<td>Three</td>
<td>0.0066</td>
<td>0.0148</td>
<td>0.0197</td>
<td>0.0231</td>
<td>0.0012</td>
<td>0.0025</td>
<td>0.0348</td>
<td>0.0621</td>
</tr>
<tr>
<td>Four</td>
<td>0.0136</td>
<td>0.0148</td>
<td>0.0197</td>
<td>0.0232</td>
<td>0.0012</td>
<td>0.0025</td>
<td>0.0349</td>
<td>0.0632</td>
</tr>
<tr>
<td>Five</td>
<td>0.0136</td>
<td>0.0149</td>
<td>0.0197</td>
<td>0.0232</td>
<td>0.0012</td>
<td>0.0026</td>
<td>0.0349</td>
<td>0.0638</td>
</tr>
</tbody>
</table>
Table 7.6. Maximum values of compression plastic strain component (PEEQ) at each step of the analysis (see § 5.6.2 for steps definition)

<table>
<thead>
<tr>
<th>Step</th>
<th>SH2N1</th>
<th>SH2N2</th>
<th>SH2N3</th>
<th>SH2N4</th>
<th>GH2N1</th>
<th>GH2N2</th>
<th>GH2N3</th>
<th>GH2N4</th>
</tr>
</thead>
<tbody>
<tr>
<td>One</td>
<td>0.0018</td>
<td>0.0016</td>
<td>0.0017</td>
<td>0.0018</td>
<td>0.0012</td>
<td>0.0012</td>
<td>0.0011</td>
<td>0.0012</td>
</tr>
<tr>
<td>Three</td>
<td>0.0053</td>
<td>0.0117</td>
<td>0.0205</td>
<td>0.0316</td>
<td>0.0016</td>
<td>0.0055</td>
<td>0.0093</td>
<td>0.0097</td>
</tr>
<tr>
<td>Four</td>
<td>0.0053</td>
<td>0.0176</td>
<td></td>
<td></td>
<td>0.0016</td>
<td>0.0055</td>
<td>0.0108</td>
<td></td>
</tr>
<tr>
<td>Five</td>
<td>0.0058</td>
<td>0.0190</td>
<td></td>
<td></td>
<td>0.0016</td>
<td>0.0055</td>
<td>0.0112</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step</th>
<th>SH2U1</th>
<th>SH2U2</th>
<th>SH2U3</th>
<th>SH2U4</th>
<th>GH2U1</th>
<th>GH2U2</th>
<th>GH2U3</th>
<th>GH2U4</th>
</tr>
</thead>
<tbody>
<tr>
<td>One</td>
<td>0.0019</td>
<td>0.0032</td>
<td>0.0025</td>
<td>0.0025</td>
<td>0.0011</td>
<td>0.0011</td>
<td>0.0011</td>
<td>0.0011</td>
</tr>
<tr>
<td>Three</td>
<td>0.0050</td>
<td>0.0107</td>
<td>0.0199</td>
<td>0.0282</td>
<td>0.0015</td>
<td>0.0048</td>
<td>0.0083</td>
<td>0.0131</td>
</tr>
<tr>
<td>Four</td>
<td>0.0050</td>
<td>0.0127</td>
<td>0.0397</td>
<td>0.0424</td>
<td>0.0015</td>
<td>0.0048</td>
<td>0.0083</td>
<td>0.0213</td>
</tr>
<tr>
<td>Five</td>
<td>0.0055</td>
<td>0.0171</td>
<td>0.0429</td>
<td></td>
<td>0.0015</td>
<td>0.0048</td>
<td>0.0083</td>
<td>0.0216</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step</th>
<th>SH2P1</th>
<th>SH2P2</th>
<th>SH2P3</th>
<th>SH2P4</th>
<th>GH2P1</th>
<th>GH2P2</th>
<th>GH2P3</th>
<th>GH2P4</th>
</tr>
</thead>
<tbody>
<tr>
<td>One</td>
<td>0.0022</td>
<td>0.0022</td>
<td>0.0022</td>
<td>0.0022</td>
<td>0.0011</td>
<td>0.0011</td>
<td>0.0011</td>
<td>0.0011</td>
</tr>
<tr>
<td>Three</td>
<td>0.0041</td>
<td>0.0086</td>
<td>0.0134</td>
<td>0.0151</td>
<td>0.0015</td>
<td>0.0050</td>
<td>0.0086</td>
<td>0.0092</td>
</tr>
<tr>
<td>Four</td>
<td>0.0041</td>
<td>0.0087</td>
<td>0.0135</td>
<td>0.0152</td>
<td>0.0015</td>
<td>0.0050</td>
<td>0.0087</td>
<td>0.0092</td>
</tr>
<tr>
<td>Five</td>
<td>0.0041</td>
<td>0.0111</td>
<td>0.0177</td>
<td>0.0204</td>
<td>0.0015</td>
<td>0.0050</td>
<td>0.0087</td>
<td>0.0092</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step</th>
<th>SV2N1</th>
<th>SV2N2</th>
<th>SV2N3</th>
<th>SV2N4</th>
<th>GV2N1</th>
<th>GV2N2</th>
<th>GV2N3</th>
<th>GV2N4</th>
</tr>
</thead>
<tbody>
<tr>
<td>One</td>
<td>0.0022</td>
<td>0.0022</td>
<td>0.0021</td>
<td>0.0021</td>
<td>0.0011</td>
<td>0.0011</td>
<td>0.0012</td>
<td>0.0012</td>
</tr>
<tr>
<td>Three</td>
<td>0.0045</td>
<td>0.0059</td>
<td>0.0070</td>
<td>0.0182</td>
<td>0.0013</td>
<td>0.0041</td>
<td>0.0075</td>
<td>0.0087</td>
</tr>
<tr>
<td>Four</td>
<td>0.0046</td>
<td>0.0062</td>
<td>0.0073</td>
<td>0.0188</td>
<td>0.0013</td>
<td>0.0041</td>
<td>0.0077</td>
<td>0.0094</td>
</tr>
<tr>
<td>Five</td>
<td>0.0046</td>
<td>0.0062</td>
<td>0.0074</td>
<td>0.0188</td>
<td>0.0013</td>
<td>0.0041</td>
<td>0.0077</td>
<td>0.0095</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step</th>
<th>SV2U1</th>
<th>SV2U2</th>
<th>SV2U3</th>
<th>SV2U4</th>
<th>GV2U1</th>
<th>GV2U2</th>
<th>GV2U3</th>
<th>GV2U4</th>
</tr>
</thead>
<tbody>
<tr>
<td>One</td>
<td>0.0022</td>
<td>0.0022</td>
<td>0.0021</td>
<td>0.0021</td>
<td>0.0011</td>
<td>0.0011</td>
<td>0.0012</td>
<td>0.0012</td>
</tr>
<tr>
<td>Three</td>
<td>0.0022</td>
<td>0.0022</td>
<td>0.0021</td>
<td>0.0021</td>
<td>0.0011</td>
<td>0.0011</td>
<td>0.0012</td>
<td>0.0012</td>
</tr>
<tr>
<td>Four</td>
<td>0.0022</td>
<td>0.0022</td>
<td>0.0021</td>
<td>0.0022</td>
<td>0.0011</td>
<td>0.0011</td>
<td>0.0012</td>
<td>0.0012</td>
</tr>
<tr>
<td>Five</td>
<td>0.0022</td>
<td>0.0022</td>
<td>0.0022</td>
<td>0.0022</td>
<td>0.0011</td>
<td>0.0011</td>
<td>0.0012</td>
<td>0.0012</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step</th>
<th>SC2N1</th>
<th>SC2N2</th>
<th>SC2N3</th>
<th>SC2N4</th>
<th>GC2N1</th>
<th>GC2N2</th>
<th>GC2N3</th>
<th>GC2N4</th>
</tr>
</thead>
<tbody>
<tr>
<td>One</td>
<td>0.0023</td>
<td>0.0025</td>
<td>0.0023</td>
<td>0.0025</td>
<td>0.0011</td>
<td>0.0012</td>
<td>0.0012</td>
<td>0.0014</td>
</tr>
<tr>
<td>Three</td>
<td>0.0216</td>
<td>0.0308</td>
<td>0.0852</td>
<td>0.0953</td>
<td>0.0022</td>
<td>0.0189</td>
<td>0.0234</td>
<td>0.0247</td>
</tr>
<tr>
<td>Four</td>
<td>0.0708</td>
<td>0.0939</td>
<td></td>
<td></td>
<td>0.0022</td>
<td>0.0231</td>
<td>0.0307</td>
<td></td>
</tr>
<tr>
<td>Five</td>
<td>0.0708</td>
<td>0.0939</td>
<td></td>
<td></td>
<td>0.0022</td>
<td>0.0239</td>
<td>0.0318</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step</th>
<th>SC2U1</th>
<th>SC2U2</th>
<th>SC2U3</th>
<th>SC2U4</th>
<th>GC2U1</th>
<th>GC2U2</th>
<th>GC2U3</th>
<th>GC2U4</th>
</tr>
</thead>
<tbody>
<tr>
<td>One</td>
<td>0.0022</td>
<td>0.0022</td>
<td>0.0022</td>
<td>0.0022</td>
<td>0.0011</td>
<td>0.0011</td>
<td>0.0011</td>
<td>0.0011</td>
</tr>
<tr>
<td>Three</td>
<td>0.0209</td>
<td>0.0134</td>
<td>0.0268</td>
<td>0.1145</td>
<td>0.0016</td>
<td>0.0058</td>
<td>0.0107</td>
<td>0.0162</td>
</tr>
<tr>
<td>Four</td>
<td>0.0378</td>
<td>0.0426</td>
<td>0.0449</td>
<td></td>
<td>0.0016</td>
<td>0.0058</td>
<td>0.012</td>
<td>0.0199</td>
</tr>
<tr>
<td>Five</td>
<td>0.0378</td>
<td>0.0426</td>
<td>0.0524</td>
<td></td>
<td>0.0016</td>
<td>0.0058</td>
<td>0.0121</td>
<td>0.0201</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Step</th>
<th>SC2P1</th>
<th>SC2P2</th>
<th>SC2P3</th>
<th>SC2P4</th>
<th>GC2P1</th>
<th>GC2P2</th>
<th>GC2P3</th>
<th>GC2P4</th>
</tr>
</thead>
<tbody>
<tr>
<td>One</td>
<td>0.0021</td>
<td>0.0022</td>
<td>0.0024</td>
<td>0.0028</td>
<td>0.0011</td>
<td>0.0011</td>
<td>0.0012</td>
<td>0.0011</td>
</tr>
<tr>
<td>Three</td>
<td>0.0213</td>
<td>0.0298</td>
<td>0.0435</td>
<td>0.0231</td>
<td>0.0017</td>
<td>0.0069</td>
<td>0.0227</td>
<td>0.0300</td>
</tr>
<tr>
<td>Four</td>
<td>0.0685</td>
<td>0.0755</td>
<td>0.0784</td>
<td>0.0347</td>
<td>0.0017</td>
<td>0.0069</td>
<td>0.03</td>
<td>0.0377</td>
</tr>
<tr>
<td>Five</td>
<td>0.0685</td>
<td>0.0756</td>
<td>0.0784</td>
<td>0.0348</td>
<td>0.0017</td>
<td>0.0069</td>
<td>0.0306</td>
<td>0.0381</td>
</tr>
</tbody>
</table>
The damage ratios were calculated for three joint opening widths: 3 mm, 6 mm and 12 mm. The first two widths were selected to represent the real contraction of the pavement because the thermal movement of each concrete pavement slab of 4.5 m length is about 3 mm which can be calculated using Equation (7.1) (Darter and Barenberg 1977). For the contraction of two adjacent slabs, the joint opening will be 6 mm. The 12 mm width represents the worst case scenario of concrete contraction due to a high variation in temperature or high concrete shrinkage due to high water content or poor aggregate gradation (a large ratio of fine aggregate) or due to using some types of additives (accelerators).

\[
\Delta S_L = C S_L \left( \alpha \Delta T + \varepsilon_d \right)
\]  

(7.1)

where, \( \Delta S_L \): the additional joint opening due to temperature change and concrete shrinkage (mm); \( C \): the adjustment factor for the slab-base friction which can be taken as 0.8 for the granular base and 0.65 for stabilized base; \( S_L \): the slab length (the distance between the transverse joints) (mm); \( \alpha \): the coefficient of thermal expansion of concrete = \( 8 \) to \( 12 \times 10^{-6} \)/\( C^0 \); \( \Delta T \): temperature change in \( C^0 \); and \( \varepsilon_d \): the coefficient of the drying shrinkage of concrete which can be taken as \( 0.5 \) to \( 2.5 \times 10^{-4} \).

Figure 7.20 presents the damage ratios for all misalignment cases considered in the current study for steel and GFRP dowels at the joint openings of 3 mm, 6 mm and 12 mm. It clearly shows that vertical misalignment causes less damage to the surrounding concrete pavement compared with horizontal and combined dowel misalignment for both steel and GFRP dowels. The smaller pull-out loads for these specimens indicates less possibility of joint lockup and stress concentration in the dowel-concrete interface, consequently less damage occurred. However, the vertical misalignment can cause differential displacement for the joint faces which is similar to joint faulting, especially for uniform misalignment (see Figure 7.15). This displacement has a direct impact on the ride quality of the road. Also, as shown in Figure 7.20, uniform misalignment causes less damage compared with non-uniform misalignment for all misalignment cases. Combined misalignment exhibits higher damage in the concrete pavement surrounding the dowel as compared with horizontal misalignment. The damage ratios increase with an increase in the misalignment magnitude except for vertical uniform misalignment.
The damage ratios of different misalignment cases for specimens that have steel dowels and specimens that have GFRP dowels at the joint openings of 3 mm, 6 mm and 12 mm. Note the log-scale in the above plot of the damage ratios.

The average damage ratios for all misalignment types and magnitudes from Figure 7.20 are summarised in Table 7.7. It illustrates that the damage ratios significantly increase with an increase in the joint opening especially for horizontal and combined misalignments. The increase in deterioration for combined misalignment is more than that of horizontal misalignment as mentioned before. This can be linked to the effect of higher bearing stress created by combined misaligned dowel bars. Using GFRP dowel bars produces a substantial reduction in concrete deterioration, especially in the presence of dowel misalignment.

**Table 7.7. Average damage ratios for different misalignment cases**

<table>
<thead>
<tr>
<th>Misalignment type</th>
<th>% average damage ratios</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3 mm</td>
</tr>
<tr>
<td></td>
<td>Steel</td>
</tr>
<tr>
<td>Vertical</td>
<td>1.5</td>
</tr>
<tr>
<td>Horizontal</td>
<td>4.5</td>
</tr>
<tr>
<td>Combined</td>
<td>4.9</td>
</tr>
</tbody>
</table>
Although the ABAQUS parameter DAMAGET indicates the state of damage fairly well, it does not show the changes in plastic strain that occur during the stages of opening and closing. Hence equivalent plastic strain components in uniaxial tension and compression (PEEQT & PEEQ) were also used to identify the deterioration that occurs due to the combined effect of dowel misalignment and traffic wheel load, and to compare GFRP dowels with steel dowels. Since the maximum PEEQT & PEEQ values may be localized in only a few elements, the average PEEQT & PEEQ values were calculated for the control volume of the tested slabs.

Figure 7.21 presents a comparison of average PEEQT & PEEQ for both steel and GFRP dowels at the joint openings of 3 mm and 6 mm. The results show a similar trend to that achieved by plotting the damage ratios for a control volume as defined before. The vertical misalignment cases show significantly lower average values of PEEQT & PEEQ than that of the horizontal misalignment cases which are in turn slightly lower than the combined misalignment cases. Also, the average PEEQT & PEEQ values increase with an increase in non-uniformity and the magnitudes of misalignment. For all the investigated cases, the average PEEQT & PEEQ values for the specimens that had GFRP dowels are significantly lower than that of steel dowels.

Since more damage is observed in the specimens that have horizontal and combined misaligned dowels, the behaviour of the material during the pull-out process can be clarified by looking at the response of these specimens. Normally, dowel bars are designed to have free movement in an axial direction; therefore, no axial force is expected in the dowel bar. Dowel misalignment or improper debonding of the dowel bars from the concrete pavement may cause the initiation of such forces. It can be observed from the result in Figure 7.21 that for a lower misalignment magnitude (6.25 mm) such as for specimens SH2N1, SH2P1, SC2N1, SC2U1 and SC2P1 the average PEEQT & PEEQ values are approximately similar. This indicates a lower axial force initiation for a lower misalignment magnitude. In contrast, for the higher misalignment magnitudes significant differences can be seen in the average PEEQT & PEEQ values due to the axial force initiation.

As shown in Figure 7.21, the average PEEQ values are slightly higher than the average PEEQT values for most cases of horizontal and combined misalignment at joint opening
of 3 mm. This is because for a small joint opening, the bearing stress of the misaligned dowels creates compressive stress on the dowel-concrete interface. At this stage fewer tensile cracks are initiated. Consequently, the average PEEQ values are slightly higher than the average PEEQT values. As the joint opens more than 3 mm, the numbers and widths of the cracks increase significantly due to the frictional and bearing stress at the dowel-concrete interface. As a consequence of that the average PEEQT values rapidly increase especially for a higher misalignment magnitude for steel dowel bars whereas GFRP dowels remain affected by bearing stress (consequently, have more compressive plastic strain) due to a higher bearing area and significantly lower frictional stress.

The average plastic strain values PEEQT & PEEQ show that excessive damage occurs in the concrete pavement due to dowel misalignment during the opening of the joint. Also, they show the change in stress status at the surrounding concrete pavement during the opening of the joint, from compressive stress (due to the bearing of misaligned dowels at the beginning of the opening of the joint) to tensile stress (due to the frictional stress with the increase in the joint opening).

![Figure 7.21. Average equivalent plastic strain within a control volume for different joint openings. Note the log-scale was used in the above plot](image-url)
7.4.3.3. Comparison of dowel looseness (DL) and LTE

7.4.3.3.1. Dowel looseness (DL)

The previous investigations have shown that dowel misalignment has a significant effect on the deterioration of the concrete pavement surrounding dowel bars. This deterioration causes an enlargement of the dowel bar socket, which affects the LTE of the dowel bar. This section presents the results of the effect of dowel misalignment on the DL and on the LTE for steel and GFRP dowels. The results are illustrated for the four misalignment magnitudes of 6.25 mm, 12.5 mm, 19 mm and 25 mm, which were defined 1, 2, 3 and 4 respectively as the last character in the naming of the specimens. The number zero was used to refer to the aligned cases for both steel and GFRP dowels. The DL was computed indirectly by calculating the increase in RD after the pulling and pushing in of the concrete slabs.

The effect of dowel misalignment on the DL (enlargement of the dowel bars’ sockets) due to slab movement for those specimens having horizontally misaligned steel and GFRP dowels are presented in Figure 7.22. DL increases for both steel and GFRP dowels for higher misalignment magnitude. Misalignments of steel dowels lead to a significant looseness when compared with GFRP dowels. Figure 7.22 shows that for a similar misalignment magnitude the DL is higher for non-uniform rather than uniform misalignments, and for uniform rather than partial misalignments. A substantial increase in DL is noticed for a dowel misalignment greater than 6.25 mm especially for horizontally non-uniform dowel misalignment. The DL is 0.139 mm, 0.0469 mm and 0.0393 mm for the non-uniform, uniform and partial misalignments respectively for steel dowels having 12.5 mm dowel misalignment. For GFRP dowels, the DL is 0.0416 mm, 0.0194 mm and 0.0139 mm for the corresponding misalignment types and magnitude. DL for specimens SH2N4, SH2N3, SH2U4 and GH2N4 is not considered in this comparison because their analysis was halted as a result of significant cracks and failure.
Figure 7.22. DL due to slabs’ movement (horizontal misalignment): (a) Non-uniform, (b) Uniform, (c) Partial

Figure 7.23 presents the variation in DL with the misalignment magnitudes for vertically misaligned steel and GFRP dowels. It can be seen that the vertically uniform misalignment produces no DL for either steel and GFRP dowels (see Figure 7.23 (b)). For other types of vertical misalignments (non-uniform and partial), there is no significant increase in DL for dowel misalignment magnitudes less than 12.5 mm. DL increases for the vertically non-uniform and partial misalignment when the misalignment magnitude is greater than 12.5 mm for steel and GFRP dowels. However, the GFRP dowels still show lower DL when compared with steel dowels. Generally, vertical misalignment produces lower DL compared with horizontal misalignment, as shown in Figure 7.22 for both steel and GFRP dowels.

Figure 7.24 compares the variation in DL with the misalignment magnitude for the specimens that have combined misalignment. The trend is similar to the horizontal misalignment cases, where the steel dowels produce higher DL than the GFRP dowels and the DL increases for higher misalignment magnitude. However, no significant differences can be observed in the DL for dowels that have non-uniform combined misalignment due to the excessive damage in the dowel-concrete interface that occurs for this misalignment type. The DL values are not shown for the specimens SC2N3,
SC2N4, GC2N4 and SC2U4 because the simulation stopped due to the initiation of significant cracks and damage in these specimens.

Figure 7.23. DL due to slabs’ movement (vertical misalignment): (a) Non-uniform, (b) Uniform, (c) Partial

Figure 7.24. DL due to slabs’ movement (combined misalignment): (a) Non-uniform, (b) Uniform, (c) Partial

188
In summary, the trend in the variation of DL with misalignment magnitudes and the types of dowel misalignment is similar to that of the damage volume in the surrounding concrete pavement. Vertical dowel misalignment (Figure 7.23) shows lower DL than the corresponding specimens of horizontal and combined dowel misalignment for both steel and GFRP dowel bars. Also, the uniform misalignment exhibits significantly lower looseness than the non-uniform dowel misalignment. This variation in DL is directly linked to the damage in the surrounding concrete pavement which was predicted by the numerical simulation in the previous sections. This variation reveals the effect of dowel misalignment on the dowel bar performance, especially when such opening and closing occur innumerable times during the lifetime of a pavement.

### 7.4.3.3.2 Load transfer efficiency (LTE)

The LTE is one of the most important indicators given by the AASHTO (1993) guide to evaluate dowel bars’ performance in JPCP. The LTE is defined as the ratio of the deflection of the unloaded side ($d_u$) of the joint to the deflection of the loaded side ($d_l$) of the joint. According to the AASHTO (1993) guide, a sufficient load transfer across the joint is achieved when the LTE is between 70-100%. The current section compares the LTE for steel and GFRP dowels before and after the slabs’ movement to show the effect of dowel misalignment on the LTE.

Figure 7.25 illustrates the LTEs for the specimens that have horizontally misaligned steel and GFRP dowel bars. It can be observed that the LTE is slightly higher for the steel dowels than that for the GFRP dowels before the joint movement due to the lower stiffness of the GFRP dowels. However, the GFRP dowels still show very good LTE according to the AASHTO (1993) guide. The LTE decreases for both steel and GFRP dowels after the slabs’ movement as a consequence of developing DL. A significant decrease is observed for the steel dowels when compared with the GFRP dowels. This reduction is higher for the non-uniform misalignment cases when compared with the uniform misalignment cases, and similarly for the uniform cases compared with the partial misalignment cases. Also, the reduction in LTE increases with the increase in dowel misalignment magnitudes.

Figure 7.26 presents the results of a comparison of the LTEs of the specimens that have vertically misaligned steel and GFRP dowels. It shows that vertical uniform
misalignment has an insignificant effect on LTE whereas a noticeable reduction in LTE is seen for the vertically non-uniform and the partial misalignment cases. This reduction is clearly seen for the specimens that have misalignment magnitudes greater than 19 mm. Figure 7.26 indicates that, for all specimens that have vertically misaligned dowels (steel and GFRP), a lower reduction can be observed in LTE after the slabs’ movement compared with specimens that have horizontally misaligned dowels. This response is associated with the damage initiation due to each of these misalignment types.

The LTEs before and after the slabs’ movement for the specimens that have combined misaligned steel and GFRP dowels are shown in Figure 7.27. Similar to the other misalignment types, a reduction in the LTE can be seen for both steel and GFRP dowels after the slabs’ movement, and the reduction increases as the misalignment magnitude increases. The reduction in LTE is more noticeable for the specimens containing steel dowels especially for high misalignment magnitudes. A comparison has not been made for some higher misalignment types due to failure of these specimens.

![Graphs showing reduction in LTE due to slabs’ movement](image)

Figure 7.25. Reduction in LTE due to slabs’ movement (horizontal misalignment): (a) Non-uniform, (b) Uniform, (c) Partial (“B” before movement and “A” after movement)
Figure 7.26. Reduction in LTE due to slabs’ movement (vertical misalignment): (a) Non-uniform, (b) Uniform, (c) Partial (“B” before movement and “A” after movement)

Figure 7.27. Reduction in LTE due to slabs’ movement (combined misalignment): (a) Non-uniform, (b) Uniform, (c) Partial (“B” before movement and “A” after movement)
In summary, Figure 7.25, Figure 7.26 and Figure 7.27 show that the slabs’ movement produces more reduction in the LTE for steel dowels than for GFRP dowels especially for the high misalignment magnitudes. Also, they show that this reduction is greater for the combined and horizontally misaligned dowels than for vertically misaligned dowels and for the non-uniform misalignment than for the uniform and partial dowel misalignment. Although the steel dowel bars show slightly higher initial LTE (before slabs’ movement) compared with GFRP dowels, the decrease in LTE is lower for the GFRP dowels than that for steel dowels after just one opening and closing of the joint especially for the high misalignment magnitudes. This response is expected to increase as a consequence of repeating the expansion and contraction during the pavement service life.

7.4.4. Comparison of specimens containing two, and specimens containing three, GFRP dowels

Due to apparatus limitations in the laboratory, it was difficult to test a full width slab. The previous investigations have focused on the specimens containing two steel or two GFRP dowel bars across the joint. In order to investigate the effect of the numbers of the dowel bars at the transverse joint of JPCP, the current section presents a comparison between specimens containing two GFRP dowels and specimens containing three GFRP dowels. This comparison includes the effect of the number of dowel bars on the pull-out load and on the damage in the surrounding concrete pavement.

7.4.4.1. The pull-out load and joint opening

The results of the comparison of the average pull-out load per dowel bar for specimens containing two GFRP dowels with specimens containing three GFRP dowels are illustrated in Figure 7.28, Figure 7.29 and Figure 7.30. These figures present the results of the comparison for horizontal, vertical and combined misalignment types respectively. The trend of the results for the specimens that contained three GFRP dowels is similar to that of the results for specimens containing two GFRP dowels whereby the pull-out load increases with an increase in misalignment magnitude and it is significantly lower for vertical misalignment compared with horizontal and combined misalignments. These figures show that for all magnitudes and types the specimens containing three GFRP dowels required slightly higher pull-out load per dowel bar to
open the joint. The differences in pull-out loads are more noticeable for the non-uniform misalignment than for the uniform and partial misalignment.

![Graphs showing load per dowel bar versus joint opening for different types and magnitudes of horizontal misalignment.](image)

**Figure 7.28.** Comparing different types and magnitudes of the horizontal misalignment for specimens containing two, and specimens containing three, GFRP dowels: (a) Non uniform; (b) Uniform; (c) Partial

Although the pull-out loads for specimens containing three GFRP dowels are slightly higher than those of specimens containing two GFRP dowel bars, the results show that load-drops associated with the spalling of the concrete pavement surrounding the dowel bar are less. The results show the occurrence of the concrete spalling for specimens GH2N3, GH2U4 and GH2P4 at joint openings of 10.5 mm, 11.5 mm and 9 mm respectively where no significant load drops were observed in the corresponding
specimens containing three GFRP dowels. This could be linked to a better re-
distribution of the stresses in the dowel-concrete interface which may happen for the
specimens containing three GFRP dowels. Only specimen GH3N4 exhibited excessive
cracks and failure at the joint opening of 8.46 mm whereas for the specimens containing
two GFRP dowels, cracks and failure were also observed for specimens GH2N4 and
GC2N4. The failure of GH3N4 happened approximately at a similar joint opening to
that of the failure of specimen GH2N4.

Figure 7.29. Comparing different types and magnitudes of the vertical misalignment for
specimens containing two and specimens containing three GFRP dowels: (a) Non uniform;
(b) Uniform; (c) Partial
Figure 7.30. Comparing different types and magnitudes of the combined misalignment for specimens containing two and specimens containing three GFRP dowels: (a) Non uniform; (b) Uniform; (c) Partial

7.4.4.2. Damage initiation

The comparisons between the damage ratios of control volume for the specimens containing two GFRP dowels and the specimens containing three GFRP dowels are presented in Figure 7.31. It should be mentioned that the control volume of specimens containing three GFRP dowels is 900 mm × 227.5 mm × 200 mm. As with the specimens containing two GFRP dowels, the damage ratios for the specimens containing three GFRP dowels increase as the joint is being opened. It can be observed from Figure 7.31 that the damage ratios for the specimens containing two GFRP dowels are higher than those of the specimens containing three GFRP dowels. As mentioned
before, this could occur due to a better re-distribution of the stresses in the dowel-concrete interface for specimens containing three GFRP dowels. For the highest magnitudes of non-uniform dowel misalignment (combined and horizontal), the damage ratios at the 12 mm joint opening are approximately similar for the specimens containing two GFRP dowels and the specimens containing three GFRP dowels. This is attributable to the excessive damage that occurs at this large joint opening.

Figure 7.31. The damage ratios of different misalignment cases for specimens containing two GFRP dowels and specimens containing three GFRP dowels at the joint openings of 3 mm, 6 mm and 12 mm. Note the log-scale in the values of the damage ratios.

Figure 7.32 shows the average plastic strain components (PEEQT and PEEQ) for specimens containing two and specimens containing three GFRP dowels at the joint openings of 3 mm and 6 mm. It can be observed from Figure 7.32 that, at the joint opening of 3 mm, the average PEEQ values for specimens containing two GFRP dowels are higher than that for specimens containing three GFRP dowels especially for misalignment magnitude equal to or less than 12.5 mm except for specimens GC2N4 and GC2U4 whereas there is no significant differences in the average PEEQT values for specimens containing two and specimens containing three GFRP dowels. This behaviour was due to more compressive stress concentration occurring in the specimens.
containing two GFRP dowels while redistribution and less localization occurred for the specimens containing three GFRP dowels. For the tensile plastic strain (PEEQT), the values are approximately similar for both types of specimens due to low frictional stress, except for the highest misalignment magnitude (25 mm) where higher average PEEQT values can be observed for specimens containing two GFRP dowels due to crack initiations.

This behaviour approximately continued without significant changes as the joint opening increased to 6 mm because both types of specimens exhibited a similar post-slip response due to significantly low frictional stress.

![Figure 7.32. Average equivalent plastic strain within a control volume for different joint openings for specimens containing two, and specimens containing three, GFRP dowels. Note the log-scale in the values of the damage ratios](image.png)

7.4.4.3. Comparison of dowel looseness and LTE

7.4.4.3.1. Dowel looseness

Dowel looseness (DL) can arise at the dowel-concrete interface due to concrete deterioration in this area. In this section, DL is compared for the specimens containing
two misaligned GFRP dowels and the specimens containing three misaligned GFRP dowels after the opening and closing of the joint. The comparison involved all misalignment cases which were investigated in the current study.

It can be observed from Figure 7.33, Figure 7.34 and Figure 7.35 that, for all misalignment types (non-uniform, uniform and partial) and for all orientations of misaligned dowel bars (horizontal, vertical and combined), there is no significant differences in DL for the specimens containing two and the specimens containing three GFRP dowels when the misalignment magnitude is less than 12.5 mm except for GC2N2. For higher misalignment magnitudes, more DL is observed for the specimens containing two GFRP dowels due to a greater deterioration in these specimens which results from higher stresses’ localization as shown in Figure 7.31.

Figure 7.33, Figure 7.34 and Figure 7.35 show that the non-uniform misalignment exhibits more DL than uniform and partial misalignment. They also show that the vertical orientations of misaligned dowel bars cause lowest DL as compared to the other specimens with similar misalignment type and magnitude.

![Graphs showing DL due to slabs’ movement (horizontal misalignment) for specimens containing two and specimens containing three GFRP dowels: (a) Non-uniform, (b) Uniform, (c) Partial](image)

Figure 7.33. DL due to slabs’ movement (horizontal misalignment) for specimens containing two and specimens containing three GFRP dowels: (a) Non-uniform, (b) Uniform, (c) Partial
Figure 7.34. DL due to slabs’ movement (vertical misalignment) for specimens containing two and specimens containing three GFRP dowels: (a) Non-uniform, (b) Uniform, (c) Partial

Figure 7.35. DL due to slabs’ movement (combined misalignment) for specimens containing two and specimens containing three GFRP dowels: (a) Non-uniform, (b) Uniform, (c) Partial
7.4.4.3.2. Load transfer efficiency (LTE)

Table 7.8 presents the results of a comparison of LTE for specimens containing two and specimens containing three misaligned GFRP dowels before and after joint movement. It can be observed from this table that there is no significant difference in LTE before and after slab movement especially for misalignment magnitudes less than 12.5 mm. For misalignment magnitudes greater than 12.5 mm, there is a very small reduction in LTE for both types of specimens; however, this reduction is significantly lower than that for epoxy-coated steel dowel bars which was discussed earlier in this chapter.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>94.2</td>
<td>94.2</td>
<td>93.4</td>
<td>93.3</td>
<td>94.2</td>
<td>94.2</td>
<td>93.4</td>
<td>93.3</td>
<td>94.2</td>
<td>94.2</td>
<td>93.4</td>
<td>93.3</td>
</tr>
<tr>
<td>1</td>
<td>94.3</td>
<td>93.4</td>
<td>92.7</td>
<td>92</td>
<td>93.6</td>
<td>93</td>
<td>93.3</td>
<td>92.9</td>
<td>93</td>
<td>94</td>
<td>93.3</td>
<td>93</td>
</tr>
<tr>
<td>2</td>
<td>94.4</td>
<td>93</td>
<td>92.7</td>
<td>91</td>
<td>93.6</td>
<td>92.5</td>
<td>93.3</td>
<td>92.5</td>
<td>93.6</td>
<td>93</td>
<td>93.3</td>
<td>92.7</td>
</tr>
<tr>
<td>3</td>
<td>94.2</td>
<td>92</td>
<td>93</td>
<td>90</td>
<td>93.6</td>
<td>92.5</td>
<td>93.3</td>
<td>91.5</td>
<td>92.9</td>
<td>91.5</td>
<td>93.3</td>
<td>92.4</td>
</tr>
<tr>
<td>4</td>
<td>93.8</td>
<td>93.4</td>
<td>93.7</td>
<td>92.2</td>
<td>93.3</td>
<td>90</td>
<td>93.6</td>
<td>90.5</td>
<td>93.4</td>
<td>92.4</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Misalignment magnitude</th>
<th>GV3N-B</th>
<th>GV3N-A</th>
<th>GV2N-B</th>
<th>GV2N-A</th>
<th>GV3U-B</th>
<th>GV3U-A</th>
<th>GV2U-B</th>
<th>GV2U-A</th>
<th>GV3P-B</th>
<th>GV3P-A</th>
<th>GV2P-B</th>
<th>GV2P-A</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>94.2</td>
<td>94.2</td>
<td>93.4</td>
<td>93.3</td>
<td>94.2</td>
<td>94.2</td>
<td>93.4</td>
<td>93.3</td>
<td>94.2</td>
<td>94.2</td>
<td>93.4</td>
<td>93.3</td>
</tr>
<tr>
<td>1</td>
<td>93.8</td>
<td>92.7</td>
<td>93.2</td>
<td>92.8</td>
<td>93.8</td>
<td>93.7</td>
<td>93.2</td>
<td>93.2</td>
<td>93.7</td>
<td>93.8</td>
<td>93.2</td>
<td>93.2</td>
</tr>
<tr>
<td>2</td>
<td>93.8</td>
<td>92</td>
<td>93</td>
<td>92.3</td>
<td>93.8</td>
<td>93.7</td>
<td>93.2</td>
<td>93.1</td>
<td>94.1</td>
<td>93.8</td>
<td>93.2</td>
<td>92.7</td>
</tr>
<tr>
<td>3</td>
<td>93.8</td>
<td>92.9</td>
<td>93.2</td>
<td>89.8</td>
<td>93.8</td>
<td>93.7</td>
<td>93.1</td>
<td>93</td>
<td>94.1</td>
<td>93.6</td>
<td>93.1</td>
<td>91.7</td>
</tr>
<tr>
<td>4</td>
<td>93.7</td>
<td>92.7</td>
<td>93.2</td>
<td>89.3</td>
<td>93.7</td>
<td>93.7</td>
<td>93</td>
<td>93</td>
<td>94</td>
<td>92.9</td>
<td>93.1</td>
<td>91.3</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Misalignment magnitude</th>
<th>GC3N-B</th>
<th>GC3N-A</th>
<th>GC2N-B</th>
<th>GC2N-A</th>
<th>GC3U-B</th>
<th>GC3U-A</th>
<th>GC2U-B</th>
<th>GC2U-A</th>
<th>GC3P-B</th>
<th>GC3P-A</th>
<th>GC2P-B</th>
<th>GC2P-A</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>94.2</td>
<td>94.2</td>
<td>93.4</td>
<td>93.3</td>
<td>94.2</td>
<td>94.2</td>
<td>93.4</td>
<td>93.3</td>
<td>94.2</td>
<td>94.2</td>
<td>93.4</td>
<td>93.3</td>
</tr>
<tr>
<td>1</td>
<td>94.3</td>
<td>92.8</td>
<td>93.3</td>
<td>92.3</td>
<td>94.2</td>
<td>93.6</td>
<td>93.7</td>
<td>93.2</td>
<td>94.3</td>
<td>94</td>
<td>93.4</td>
<td>93.2</td>
</tr>
<tr>
<td>2</td>
<td>90.4</td>
<td>89.5</td>
<td>93.1</td>
<td>89.5</td>
<td>94.6</td>
<td>93.5</td>
<td>94.2</td>
<td>93</td>
<td>94.4</td>
<td>94</td>
<td>93.7</td>
<td>93.1</td>
</tr>
<tr>
<td>3</td>
<td>93.9</td>
<td>92</td>
<td>93.1</td>
<td>87.6</td>
<td>95</td>
<td>93.8</td>
<td>94.6</td>
<td>93</td>
<td>94.5</td>
<td>92.7</td>
<td>94</td>
<td>92.6</td>
</tr>
<tr>
<td>4</td>
<td>94.5</td>
<td>91.1</td>
<td>93</td>
<td>95.4</td>
<td>94.2</td>
<td>93.3</td>
<td>91.4</td>
<td>94.7</td>
<td>93.4</td>
<td>93.3</td>
<td>91.8</td>
<td></td>
</tr>
</tbody>
</table>
7.5. **Summary and Conclusions**

This chapter involved investigations into the effect of orientations, misalignment types and the misalignment magnitudes of steel and GFRP dowel bars on the pull-out load required to open the transverse joints of JPCP. The numerical simulation showed the deterioration accompanied with each case of dowel misalignment. It also showed the maximum plastic strain components due to wheel load and dowel misalignment. Consequently, a clear insight was obtained into the increase in DL and the decrease in LTE due to the combined effect of wheel load and dowel misalignment for each misalignment case. This chapter also illustrated the effect of the number of misaligned GFRP dowel bars across the joint on pull-out load, concrete deterioration, DL and LTE.

The following conclusions can be drawn from this chapter:

- The experimental and FE results of 38 mm GFRP dowels show a comparable, or even slightly better performance, to that of 25 mm epoxy-coated-steel dowels in terms of the DL and LTE after joint movement.
- Using a flexible base, in which slab-base separation is allowed during the pull-out process, has a significant effect on the pull-out load required to open the joint.
- Dowel misalignment has a significant effect on joint lockup, whereby the pull-out load increases by increasing the misalignment magnitude and/or the non-uniformity of the misaligned dowels especially for horizontal and combined misalignment.
- The results show significantly lower pull-out loads for vertical misalignment as compared with horizontal and combined misalignment. This can be linked to the relative deflection across the joint during the pull-out process when the slab-base separation is permitted. This relative deflection of the slab produces significant variation and reduction in the contact pressure between the dowel bars and the concrete pavement; consequently, the clamping force decreases.
- A substantial reduction in the pull-out load required to open the joint is achieved by using GFRP bars as dowel bars across the transverse joints of JPCP.
- Nonlinear finite element analysis is capable of simulating the concrete damage surrounding dowel bars. The quantity of the damaged volume and the equivalent plastic strains in uniaxial tension and compression (PEEQ & PEEQT) were used to
indicate the size and effect of this damage. Also, they were used to compare between the steel and GFRP dowels for different misalignment cases.

- Dowel misalignment produces significant damage on the surrounding concrete pavement compared with that due to wheel load only for both steel and GFRP dowels.

- Although the combined misalignment cases show approximately a similar pull-out load to that for horizontal misalignment cases, the damaged concrete volume is more in the combined misalignment cases especially for the steel dowels. The vertical misalignment cases exhibit significantly lower damaged volumes compared with the horizontal and combined misalignment cases. Also, the damage is less for uniformly misaligned specimens than for the non-uniformly misaligned specimens.

- A substantial reduction in concrete pavement deterioration can be achieved by using GFRP bars as an alternative to the epoxy-coated steel dowels. The GFRP bars have a much improved long term performance and thus can reduce JPCP maintenance cost.

- As a consequence of the lower deterioration that is caused in the concrete pavement surrounding the GFRP dowels, the GFRP dowels show lower DL and higher LTE than the steel dowels during the opening and closing of the joint especially when dowel misalignment exists.

- The investigated misalignment magnitudes are higher than the misalignment tolerances allowed by most highway agencies; however, the selection has been made to clearly show the differences between the misalignment types. Also, the opening and closing of the joint occurs innumerable times during the lifetime of a pavement and, therefore, the accumulative effect of lower misalignment may be similar to that of opening and closing the joint just once for a high misalignment magnitude.

- Although specimens containing three GFRP dowels showed that a slightly higher pull-out load per dowel was required to open the joint compared with specimens containing two GFRP dowels, the concrete pavement deterioration was slightly less for three dowelled specimens. This behaviour can be attributed to a better redistribution of the stresses surrounding the misaligned dowels.
CHAPTER EIGHT
EFFECT OF DOWELLED-JPCP DESIGN PARAMETERS
ON PULL-OUT LOAD VERSUS JOINT OPENING
BEHAVIOUR

8.1. Introduction

This chapter presents numerical investigations on the effects of size, embedded length
and type of dowel bar, concrete grade, pavement thickness, and slab-base friction on the
pull-out load required to open the joint of JPCP and associated concrete deterioration in
the presence of dowel misalignment. The pull-out load and damage in surrounding
concrete pavement were compared for all investigated parameters by adopting the same
procedure used in Chapter Seven.

8.2. Effect of Concrete Compressive Strength

Figure 8.1 shows the variation of average pull-out load per dowel bar with concrete
compressive strength. The investigation involved two concrete grades C30 and C40;
and two types of dowel bars - epoxy-coated steel and GFRP. This selection was made to
demonstrate the impact of the variation in the concrete compressive strength on the pull-
out load, joint lockup and associated concrete deterioration in the presence of dowel
misalignment.

As stated in Chapter Four, the load-slip behaviour consists of two stages: fully bonded
stage and slipping stage. In the first stage, the bond is governed by the adhesion
between the dowel and the concrete, which depends on the applied load and the size of
Dowel bar. The debonding occurs when the dowel-concrete interface shear exceeds the adhesion force. In the second stage, slipping occurs for the debonded dowels; and it is governed by frictional stress between the dowel bar and concrete pavement, and bearing stress of misaligned dowel bars.

Dowel bars are designed to debond from the pavement in order to maintain horizontal movement during expansion and contraction, and transfer the load between adjoining slabs primarily by shear mechanism. This behaviour is contrary to the most concrete structures where a good bond between the reinforcement and concrete is required for composite action. For the dowel bars, although there is little adhesion with concrete, an effective bond may result due to dowel misalignment and/or the frictional stress between the dowel and surrounding concrete due to “nicking” or improper coating of dowel bar. That may lead to joint lockup and pavement distresses.

The results in Figure 8.1 show that unlike the epoxy-coated steel dowel bars the pull-out load for the GFRP dowels increases little for an increase in concrete strength (see Figure 8.1). In general, the increase in bond strength between dowel bars and concrete is related to the increased tensile strength of concrete which increases with higher concrete compressive strength (BS EN 1992-1-1 2004). The bond strength between the plain steel dowel bar and the concrete is affected by local strain and the presence of cracks at the dowel-concrete interface (Park and Paulay 1975). For higher strength concrete, more pull-out load is required to develop shear stress at the dowel-concrete interface in order to mobilise the frictional stress between the dowel and concrete. This causes propagation of cracks and shearing off of the concrete inside small pits at the steel dowel surface. It can be observed from Figure 8.1(a) that the maximum pull-out load at joint opening of 6 mm increases by 10% when the concrete compressive strength increases by 33%, which corresponds approximately to the increase in tensile strength. For the GFRP dowels (see Figure 8.1(b)), there is no significant increase in pull-out load due to lower friction and high uniformity of dowel surface. Consequently, no shearing of damaged concrete is required and the slipping occurs under a steady load.

Figure 8.2 presents a comparison of the damage ratios of concrete surrounding the dowel bars. It clearly shows that the damaged volume decreases as the concrete strength increases for both the steel and the GFRP dowel bars. This response can be attributed to
an increase in concrete tensile strength with an increase in concrete compressive strength. Consequently, the pull-out load increases and the damaged volume decreases.

Figure 8.1. Pull-out load versus joint opening for various concrete compressive strength: (a) Steel dowels; (b) GFRP dowels.

Figure 8.2. Comparison of damage ratios of concrete surrounding the dowel bars for different concrete grades and different dowel types

8.3. Effect of Dowel Bar Diameter

The dowel bar size usually varies with the pavement thickness and joint width (AASHTO 1993; UK Highway Agency 2009). The effect of this variation on pull-out load of misaligned dowels is presented in this section. Figure 8.3 shows the results of pull-out loads for different sizes of epoxy-coated steel and GFRP dowel bars. It can be observed that the pull-load increases with an increase in dowel diameter for both steel and GFRP dowels in the post-slip stage. This increase in pull-out load is slightly higher for the GFRP dowels than for epoxy-coated steel dowel bars. At joint opening of
4.5 mm, an increase of steel dowel diameter by 28% produces an increase in pull-out load by 19%, whereas for the GFRP dowels a 19% increase in diameter increases the pull-out load by 23%. The increase in dowel diameter produces increase in dowel contact area, which attracts higher bearing stress due to the slab self-weight and wheel load. Higher bearing stress induces higher frictional stress between the dowel and the concrete, and consequently more pull-out load is required (for uniform slab self-weight, the increase in the dowel diameter produces more normal force). Also, the increase in dowel diameter causes increase in the bearing area of misaligned dowels on surrounding concrete pavement. Although the frictional stress is minimal for the GFRP dowels, the bond between the GFRP dowels and concrete pavement is substantially governed by longitudinal bond and bearing stress of misaligned dowels on surrounding concrete pavement; therefore the increase in dowel bar diameter produces increase in pull-out load due to more bearing area of misaligned dowels. However, the pull-out load for the GFRP dowels remains significantly less than corresponding specimens containing steel dowel bars.

![Comparison of pull-out load versus joint opening for different dowel bar sizes](image)

**Figure 8.3. Comparison of pull-out load versus joint opening for different dowel bar sizes: (a) Steel dowels; (b) GFRP dowels.**

Figure 8.4 shows the results of comparison of the damage ratio in the surrounding concrete pavement for different sizes of steel and GFRP dowel bars. The result reveals that damaged volume increases by increasing the dowel bar diameter for both steel and GFRP dowels. This comparison agrees with the previous comparison of pull-out load, where more axial load leads to more stress and deterioration at the dowel-concrete interface. The results also demonstrate that the damaged volume in the specimen
containing bigger GFRP dowels (38 mm) is about 50% of that for specimen containing smaller size of steel dowel bars (25 mm).

Figure 8.4. Comparison of damage ratios in the surrounding concrete pavement for different sizes of steel and GFRP dowel bars

8.4. Effect of Embedded Length of Dowel Bar

The embedded length of dowel bars considered in this study were 458 mm and 600 mm as they are the most common lengths adopted by many highway agencies and pavement associations (AASHTO 1993; UK Highway Agency 2009). Figure 8.5 illustrates that change in embedded length of the dowel bars produces less change in pull-out load compared with the change in concrete strength or dowel bars size. The results in Figure 8.5 show for joint opening less than 2 mm, the pull-out load remains unchanged for increase in embedded length for both steel and GFRP dowels. For joint opening greater than 2 mm, the pull-out load slightly increases with longer embedded length of the steel dowels, while it slightly decreases with longer embedded length of the GFRP dowels.

An increase in dowel bar length may effectively increase the contact area in the same fashion as an increase in dowel bar diameter would do. However, the longer embedded length produces no change in stress concentration at the face of the joint, which does change with changes in diameter. The longer embedded length also reduces concrete spalling at higher joint opening by allowing for more stress redistribution along the length.

Figure 8.6 presents a comparison of the damage ratios in surrounding concrete pavement for different embedded length of steel and GFRP dowels. The results in
Figure 8.6 illustrate that the damaged concrete volume decreases as the embedded length of the dowel bar increases especially for the higher joint opening (12 mm). Since the maximum local bond stress moves down with opening of the joint (dowel bar slip) (Feldman and Bartlett 2007), the longer embedded length may increase the possibility of moving the localised bar tension along the dowel length before concrete deteriorates.

**Figure 8.5. Comparison of pull-out load for different lengths of dowel bars: (a) Steel dowels; (b) GFRP dowels**

**Figure 8.6. Comparison of damage ratio in the surrounding concrete pavement for different length of steel and GFRP dowels**

8.5. **Effect of Concrete Pavement Thickness**

Figure 8.7 presents the results of comparison of the effect of pavement slab thickness on the pull-out load required to open the joint in the presence of dowel misalignment. It can be observed from the results that the pull-out load does not change when the slab thickness increases. However, less concrete spalling and load-drop can be noticed for
thicker concrete pavement slab especially for the steel dowel bars. For thinner slab the stresses due to dowel misalignment cause concrete deterioration and damage. In case of the thicker slab that was considered in this study, the stresses remain localised around the dowel-concrete interface only and are not sufficient to initiate damage. Figure 8.8 clearly shows the effect of the slab thickness on the damage ratio of concrete pavement surrounding the dowel bars. This demonstrates that the damage ratio significantly decreases for an increase in concrete slab thickness.

![Graph showing load per dowel bar vs joint opening for different slab thicknesses.](image)

**Figure 8.7.** Comparison of slab thickness effect on pull-out load for misaligned dowel bars:
(a) Steel dowels; (b) GFRP dowels

![Graph showing damage ratio vs joint opening for different slab thicknesses.](image)

**Figure 8.8.** Comparison of slab thickness effect on damage ratios in surrounding concrete pavement of misaligned steel and GFRP dowels

### 8.6. Effect of Slab-Base Friction

To demonstrate the effect of the slab-base friction on the pull-out load and associated stresses, the numerical results for the load-joint opening behaviour of slabs supported by the steel beam-base were verified with the experimental results. These results were also
compared with the numerical results for similar slabs resting on foundation layers. The material properties and dimensions of the underlying layers were obtained from a previous study in the literature, as shown in Table 8.1 (Riad et al. 2009), while the concrete pavement properties were adopted from the current experimental study described in Chapter Four. The widths of the base and sub-grade layers were extended beyond the slab dimensions to avoid the boundary effects. The slab-base interaction was modelled as a surface-to-surface contact with a coefficient of friction 0.9 according to the AASHTO (1993) guide recommendations while the base-subgrade interaction was modelled by a tie constraint. Zero-displacement boundary conditions were applied to the bottom of the subgrade layer.

Table 8.1. Material properties and layer depth

<table>
<thead>
<tr>
<th>Material</th>
<th>Young’s Modulus (MPa)</th>
<th>Poisson’s ratio</th>
<th>Density (kg/m$^3$)</th>
<th>Depth (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base layer</td>
<td>310</td>
<td>0.3</td>
<td>2100</td>
<td>208</td>
</tr>
<tr>
<td>Sub-grade layer</td>
<td>27</td>
<td>0.4</td>
<td>2040</td>
<td>1270</td>
</tr>
</tbody>
</table>

Comparisons of the pull-out loads for the specimens supported by foundation layers and specimens supported by steel-beam base are presented in Figure 8.9. The results show a reasonable agreement of pull-out load for the two cases. Experimental results have been added to the plots for comparison.

Figure 8.9. Validation of different bases types (FEM) with experimental results : (a) SV2N2; (b) GH2N4
Figure 8.10 presents the results of the comparison of pull-out load for misaligned dowel bars in concrete pavement slabs having different coefficients of friction (0.0 and 0.9) with base layer. The results reveal that slab-base friction has insignificant effect on the pull-out load required to open the joint and on the associated stress and damage at the dowel-concrete interface (see Figure 8.10 and Figure 8.11). The slab-base friction could restrain the slab movement during the expansion and contraction of the pavement. However, the pull-out load presented in Figure 8.10 is the load in the dowel bar only, which may need higher separation force for slab-base with higher coefficient of friction. Once the slabs start to move, same amount of axial load develops at the dowel bars due to dowel misalignment and friction between dowel bar and surrounding concrete pavement, irrespective of the coefficient of friction at the base.

Figure 8.10. Comparison of effect of slab-base friction on pull-out load of misaligned steel dowel bars

Figure 8.11. Comparison of the effect of slab-base friction on concrete deterioration surrounding misaligned steel dowel bars
8.7. Summary and Conclusions

This chapter presented the investigation of several pavement parameters affecting the pull-out load required to open the joint and associated concrete deterioration in the presence of dowel misalignment. These parameters involved concrete compressive strength, dowel bar diameter and embedded length, pavement thickness and slab-base friction. The investigation included both epoxy-coated steel and GFRP dowel bars. The results show that the pull-out load increases substantially for an increase in concrete compressive strength and dowel bar diameter, whereas the increase is less significant for increase in embedded length of the dowel bar. There is little difference in pull-out load for variation in pavement thickness and slab-base friction. The damage in the concrete pavement slabs decreases for higher grades of concrete, embedded length of the dowel bar, and concrete pavement thickness; while concrete deterioration increases with increase of dowel bar diameter. Varying the slab-base friction coefficient did not affect the pull-out load or associated stress and damage in the surrounding concrete pavement.
9.1. Introduction

This Chapter provides the summary and the main conclusions of the experimental and numerical investigations which were carried out in this study. The experimental investigations consisted of two parts: investigation of the load-deflection response of GFRP dowels in transverse joints of JPCP and investigation of the combined effect of dowel misalignment and wheel load on dowel bar performance at the transverse joints of JPCP. The numerical investigation involved simulations of each experimental work and conducting three sets of parametric studies. The first study was used to develop design considerations for the GFRP dowels in the transverse joints of JPCP. The second study was conducted to investigate different misalignment cases and their effect on RD, LTE and DL. The third study was carried out to demonstrate the effect of different dowel and pavement parameters on the pull-out load required to open the joint in the presence of dowel misalignment.

9.2. Experimental investigation into the Load-Deflection Response of GFRP Dowels

The test arrangement consisted of two concrete blocks connected by one dowel bar at the centre as a scaled down version of an actual pavement. The reacting block was supported by the steel base of the testing machine while a gap was provided underneath the loaded block. The load-deflection response of the GFRP dowels was compared with that of the steel dowels for two different dowelled-JPCP parameters: concrete strength and joint width. Since the GFRP dowels are relatively weak in the transverse direction, the wider joints were investigated in the current study as the worst case scenario.
The results showed that the GFRP dowel bars perform equally or slightly better than the steel dowel bars that have a similar flexural rigidity \((EI)\). For the two investigated parameters, the relative deflections of specimens containing GFRP dowels are more susceptible to joint width.

9.3. **Numerical simulation of the Load-Deflection Response of GFRP Dowels**

A finite-element simulation was carried out using the ABAQUS software to show the stress distribution at the dowel-concrete interface. These stresses are difficult to be assessed during the experimental investigations. The numerical results were validated with the corresponding experimental data and a good agreement was achieved. These results showed that significantly less stress concentration in concrete pavement underneath the dowel bar can be achieved using GFRP dowels as compared to the steel dowel bars.

The numerical simulation was used to develop design considerations for GFRP dowels as an alternative to the conventional epoxy-coated steel dowel bars. The design considerations involved all dowel and pavements’ parameters such as dowel diameter, dowel length, dowel bar spacing and slab thickness. These parameters were investigated for both contraction and expansion joints. The recommended design considerations for GFRP dowels were selected after comparing the GFRP dowels’ response with that of the epoxy-coated steel dowel bars.

9.4. **Experimental Investigation into the Combined Effect of Dowel Misalignment and Wheel Load**

This experimental study involved a detailed investigation on the pull-out load versus joint opening behaviour in the presence of dowel misalignment. It also showed the possible increase of DL and the decrease of LTE due to the combined effect of dowel misalignment and cyclic load (as a representation of repeated traffic load). The specimens were supported on a steel-beam base with stiffness such that the effects of the underlying layers of real pavements are incorporated. The study also included a comprehensive experimental investigation for using GFRP bars as alternative dowel bars to minimize joint lockup and the detrimental effect at the dowel-concrete interface.
as compared with epoxy-coated steel dowels. The results showed that the GFRP dowels can withstand the cyclic traffic load and significantly reduce joint lockup and DL and can provide sufficient LTE. It was also observed that dowel misalignment affects DL significantly more than the repeated traffic load. The slab-base separation and the orientation of misaligned dowel bars have significant effects on the load required to open the joint.

However, it was not possible to observe the deterioration in the surrounding concrete pavement from the experimental tests. Also, it could not include all misalignment cases due to limited resources and time. Therefore a numerical simulation was necessary.

9.5. Numerical Investigation into the Combined Effect of Dowel Misalignment and Wheel Load

The numerical investigation was carried out to address the limitations of the experimental programme. The validated FEM was used to include most likely cases of dowel misalignment (111 cases) for more realistic pavement representation (using an equivalent steel-beam support). It also clearly showed the stress-strain distribution in the concrete pavement surrounding the dowel bars and the damaged concrete volume associated with each case of dowel misalignment. The following conclusions can be made from the numerical results.

- The modelling of a base support equivalent to that of real pavement has a significant effect on the pull-out load and the deterioration of concrete pavement.
- Dowel misalignment significantly increased the possibility of joint lockup and consequently increased the pull-out load required to open the joint.
- Pull-out load and pavement deterioration increase by increasing misalignment magnitude and misalignment non-uniformity, especially for the horizontal and the combined misalignments. The vertical misalignment showed little increase in pull-out load and damage volume compared with the other types. The increased pull-out load and associated pavement deterioration due to misalignment, causes higher DL and lower LTE.

The use of GFRP dowels significantly improved the joint performance by minimizing joint lockup, pull-out load and concrete pavement deterioration. Consequently, less DL
and higher LTE after opening and closing the joint were achieved compared with the epoxy-coated steel dowel bars.

The numerical investigation also showed the effect of dowel and pavement parameters on the joint opening behaviour for both steel and GFRP dowels. The results of this investigation illustrated that the pull-out load increases by increasing the concrete grade, and the dowel bar size (diameter), whereas the increase is less significant for increase in embedded length of the dowel bar. There is little difference in pull-out load for variation in pavement thickness and slab-base friction. The concrete deterioration showed an approximately similar trend to that of the pull-out load.

9.6. Recommendations for future studies

The current study highlighted the combined effect of dowel misalignment and axle load on dowel bars’ performances in JPCP. It also showed the suitability of using GFRP dowel bars to minimize the pull-out required to open the joint and associated distress. However, future experimental studies can investigate the opening and closing of the joint after each specific interval of cyclic load application (250,000 cycles in the current study) for both steel and GFRP dowel bars. More dowel bars across the joint with bigger slab sizes and different stiffness of base layers can also be investigated.

Although the slab movement is caused by more than one reason such as temperature gradient, moisture gradient and concrete shrinkage, the modelling of temperature gradient could show the effect of slab curling on the pull-out load in the presence of dowel misalignment. Consequently, an extensive numerical simulation could be a useful tool to model this phenomenon. The numerical simulation could also be used to model the dowel looseness due to cyclic load effect in the presence of dowel misalignment, which would help in understanding the complex stress profile at the dowel concrete interface.

Future studies may involve in developing new software for JPCP analysis and design including the effects of dowel misalignment, DL, dowel-concrete interaction, concrete non-linearity and anisotropic properties of GFRP dowels. Such software will be extremely useful for the pavement designers.
REFERENCES


Jung, Y. S., Freeman, T. J., and Zollinger, D. G. (2008). "Guidelines for Routine Maintenance of Concrete Pavement." Texas Transportation Institute, the Texas A&M University system, College Station, Texas.


Khazanovich, L., Buch, N., Gotlif, A., and Eacker, M. "Mechanistic evaluation of vertical misalignment of dowel bars and their effect on joint performance." *Proc., 7th International Conference on Concrete Pavements*


APPENDIX A

DAMAGE INITIATION IN THE DOWEL-CONCRETE INTERFACE DUE TO COMBINED EFFECT OF WHEEL LOAD AND DOWEL MISALIGNMENT
Figure A-1. Comparison of damage and plastic strain at the joint face for specimens SH2N1 and GH2N1: (a) DAMAGET; (b) PEEQT; (c) PEEQ
(a)

(b)
Figure A-2. Comparison of damage and plastic strain at the joint face for specimens SH2N2 and GH2N2: (a) DAMAGET; (b) PEEQT; (c) PEEQ
Figure A-3. Comparison of damage and plastic strain at the joint face for specimens SH2N3 and GH2N3: (a) DAMAGET; (b) PEEQT; (c) PEEQ
Figure A-4. Comparison of damage and plastic strain at the joint face for specimens SH2N4 and GH2N4: (a) DAMAGET; (b) PEEQT; (c) PEEQ