Towards The Design of Gravity-Based Wind Turbine Foundations

by

Saif Naser Shaban

A thesis submitted in conformity with the requirements for the degree of Master of Applied Science
Graduate Department of Civil Engineering
University of Toronto

© Copyright by Saif Naser Shaban (2017)
Towards the Design of Gravity-Based Wind Turbine Foundations

Saif Naser Shaban

Master of Applied Science

Graduate Department of Civil Engineering
University of Toronto

2017

Abstract

Shear fatigue failure of reinforced concrete gravity-based wind turbine foundations is a major concern amongst designers, one that can potentially cause complete collapse of the whole turbine structure. In standard practice, such a failure is guarded against by using more concrete and steel, resulting in additional material and labour costs. Six SFRC and RC large-scale deep concrete beams were constructed and tested monotonically and cyclically in order to compare the contributions of conventional stirrups to the fatigue life of the beam with those of the steel fibres and verify the strain-based fatigue damage models developed at the University of Toronto. Results show that the contribution of the stirrups to the fatigue life of RC deep beams is marginal. The steel fibres, on the other hand, are a superior alternative both in terms of performance (fatigue life) and cost. Analytical results, using finite element analysis, show that the proposed fatigue models are reliable and superior to overly conservative code equations.
Acknowledgments

Life is a humbling journey with many milestones. Quite often, we focus on these milestones that we forget the journey itself. After all, it is the journey that teaches, improves, and shapes us: our identities, beliefs, goals, and aspirations. This journey has been very special for myself in every aspect of it: the joyous moments, hard work, sleepless nights, unexpected tantrums, and the deceiving feelings of pride. It has ‘distilled’ me into a better person, for it is “only in the darkest nights that the stars shine more brightly.” However, the benefits of such experience would have never been realized had it not been for the people who were part of it.

I would like to express my utmost appreciation, respect, and indebtedness to Professor Frank J. Vecchio. He was not only my supervisor, but also my mentor who has helped me in every way possible. The numerous meetings (with thought-provoking questions and effort-stimulating expectations) and group dinners I had with him have only increased my admiration of such a well-respected and helpful individual. My gratitude is also extended to Professor Paul Gauvreau for taking the time and effort to review this thesis and provide valuable feedback.

My colleagues have also had a tremendous impact on my experience. I would like to thank Siavash Habibi for continuously helping me throughout the past two years both in the courses we have taken together and this thesis. The countless coffee and lunch breaks we have had together added a special flavour to my experience. I would also like to thank Bernard Isojeh for his continuous mentorship and coaching throughout my experimental work, both in and outside the lab. This thesis would not have seen light if it was not for his technical guidance. A special appreciation is also extended to Farhad Habibi, who was always willing to guide and help me with his prudent advice and suggestions whenever I needed them. He has helped me adapt to the overwhelming lab environment. I would also like to thank my other colleagues who have helped me with several things and made this work environment fascinating: Anca Ferche, Vahid Sadeghian, Raymond Ma, Cong Liu, Mark Hunter, Andac Lulec, Stamatina Chasioti, Edvard Bruun, and Allan Kuan. The recommendations and guidance of Dr. Ali Amin, whom I had the pleasure of meeting, with regards to steel fibres have also helped me.

My time at the lab was filled with learning opportunities that enabled me to put the theory into practice and gain additional insights into the behaviour of reinforced concrete. I would like to
thank Renzo Basset, Giovanni Buzzeo, Xiaoming Sun, Michel Fiss, Bryant Cook, and Alan McClenaghan for their help during this time.

The financial support provided by Professor Frank J. Vecchio, the Department of Civil Engineering at the University of Toronto, and Hatch Ltd. is acknowledged and appreciated.

Last but not least, I would like to acknowledge the role that my family has played in this experience, knowing that no words will do them justice. My parents have unconditionally supported me in everything I have done. Their joy in seeing the final version of the thesis means the world to me, for I am nobody without them. My wife has been behind me in every step of this process, providing me with unreserved love and support throughout the whole way and making sure that I had everything I needed to succeed; I am forever grateful. Finally, my siblings (Hani and Durra) were behind me pushing me to exceed my limits; I could not have asked for a better family.
Table of Contents

Acknowledgments ........................................................................................................................ iii

Table of Contents ........................................................................................................................ v

List of Tables .............................................................................................................................. xi

List of Figures ............................................................................................................................. xiii

List of Appendices ...................................................................................................................... xxvii

Chapter 1: Introduction ............................................................................................................. 1

1 Introduction ............................................................................................................................. 1

1.1 Overview ............................................................................................................................. 1

1.1 Research Objectives ............................................................................................................ 4

1.2 Organization of the Thesis ................................................................................................. 5

Chapter 2: Literature Review .................................................................................................... 7

2 Literature Review .................................................................................................................... 7

2.1 Wind Turbine Foundation Types ........................................................................................ 7

2.1.1 Onshore Foundations ..................................................................................................... 7

2.1.2 Offshore Foundations ................................................................................................... 10

2.2 Loading on Wind Turbine Foundations .......................................................................... 11

2.2.1 Forces Transmitted from the Tower ............................................................................. 12

2.2.2 Soil Contact Pressure ................................................................................................. 12

2.2.3 Self-Weight of the Foundation ..................................................................................... 17

2.2.4 Earthquake Loading ..................................................................................................... 17

2.2.5 Wind Fatigue (Cyclic) Loading ..................................................................................... 19

2.3 Fatigue Loading .................................................................................................................. 20

2.3.1 Fatigue Load Parameters ............................................................................................ 20
<table>
<thead>
<tr>
<th>Section</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.3.2</td>
<td>General Fatigue Damage</td>
<td>21</td>
</tr>
<tr>
<td>2.3.3</td>
<td>S-N Curves</td>
<td>21</td>
</tr>
<tr>
<td>2.3.4</td>
<td>Miner’s Rule</td>
<td>22</td>
</tr>
<tr>
<td>2.3.5</td>
<td>Modified Goodman Diagram</td>
<td>23</td>
</tr>
<tr>
<td>2.3.6</td>
<td>Fracture Mechanics</td>
<td>25</td>
</tr>
<tr>
<td>2.4</td>
<td>FATIGUE OF REINFORCED CONCRETE</td>
<td>26</td>
</tr>
<tr>
<td>2.4.1</td>
<td>Fatigue of Plain Concrete</td>
<td>27</td>
</tr>
<tr>
<td>2.4.2</td>
<td>Fatigue of Reinforcement</td>
<td>33</td>
</tr>
<tr>
<td>2.4.3</td>
<td>Fatigue of Concrete-Reinforcement Bond</td>
<td>38</td>
</tr>
<tr>
<td>2.4.4</td>
<td>Fatigue of Reinforced Concrete Beams</td>
<td>41</td>
</tr>
<tr>
<td>2.5</td>
<td>OUTSTANDING ISSUES AND RESEARCH NEEDS</td>
<td>66</td>
</tr>
<tr>
<td>3</td>
<td>Current Design of Wind Turbine Foundations</td>
<td>68</td>
</tr>
<tr>
<td>3.1</td>
<td>Overview of the Design Process</td>
<td>68</td>
</tr>
<tr>
<td>3.2</td>
<td>Load Combinations</td>
<td>69</td>
</tr>
<tr>
<td>3.3</td>
<td>Soil Bearing Capacity and Settlement Considerations</td>
<td>73</td>
</tr>
<tr>
<td>3.4</td>
<td>Establishing the Geometry of the Foundation</td>
<td>74</td>
</tr>
<tr>
<td>3.5</td>
<td>Verifying the Size of the Foundation from Bearing Capacity and Stability Calculations</td>
<td>75</td>
</tr>
<tr>
<td>3.5.1</td>
<td>Trapezoidal Soil Pressure Distribution</td>
<td>78</td>
</tr>
<tr>
<td>3.5.2</td>
<td>Uniform Soil Pressure Distribution</td>
<td>78</td>
</tr>
<tr>
<td>3.5.3</td>
<td>Triangular Soil Pressure</td>
<td>82</td>
</tr>
<tr>
<td>3.6</td>
<td>Establishing Bending Moment and Shear Force Diagrams from Sectional Analysis of the Foundations</td>
<td>82</td>
</tr>
<tr>
<td>3.7</td>
<td>Design of Flexural Reinforcement Using Equivalent Stress Block</td>
<td>88</td>
</tr>
<tr>
<td>3.8</td>
<td>Design of the Foundation against One-Way (Beam) Shear</td>
<td>91</td>
</tr>
<tr>
<td>3.9</td>
<td>Design of the Foundation against Two-Way (Punching) Shear</td>
<td>94</td>
</tr>
<tr>
<td>Section</td>
<td>Page</td>
<td></td>
</tr>
<tr>
<td>----------------------------------------------</td>
<td>------</td>
<td></td>
</tr>
<tr>
<td>3.10 Tower-Foundation Connection</td>
<td>99</td>
<td></td>
</tr>
<tr>
<td>3.10.1 Insert Ring</td>
<td>100</td>
<td></td>
</tr>
<tr>
<td>3.10.2 Anchor Bolt Cage</td>
<td>104</td>
<td></td>
</tr>
<tr>
<td>3.11 Pedestal Detailing and Design</td>
<td>109</td>
<td></td>
</tr>
<tr>
<td>3.12 Transfer of Stresses from Pedestal to Footing</td>
<td>110</td>
<td></td>
</tr>
<tr>
<td>3.13 Using the Strut-and-Tie Method to Design the Footing</td>
<td>112</td>
<td></td>
</tr>
<tr>
<td>3.14 Development Length and Anchorage of the Reinforcement</td>
<td>114</td>
<td></td>
</tr>
<tr>
<td>3.15 Serviceability Limit State Checks</td>
<td>117</td>
<td></td>
</tr>
<tr>
<td>3.15.1 Deflections</td>
<td>117</td>
<td></td>
</tr>
<tr>
<td>3.15.2 Vibrations</td>
<td>118</td>
<td></td>
</tr>
<tr>
<td>3.15.3 Cracking</td>
<td>118</td>
<td></td>
</tr>
<tr>
<td>3.16 Durability Considerations of the Foundation</td>
<td>121</td>
<td></td>
</tr>
<tr>
<td>3.17 Fatigue Verification of the Foundation</td>
<td>122</td>
<td></td>
</tr>
<tr>
<td>3.17.1 Damage Equivalent Stress Range</td>
<td>122</td>
<td></td>
</tr>
<tr>
<td>3.17.2 Miner’s Rule</td>
<td>124</td>
<td></td>
</tr>
<tr>
<td>3.17.3 Isojeh et al. (2016) Fatigue Model</td>
<td>128</td>
<td></td>
</tr>
<tr>
<td>3.18 Finite Element Analysis of the Foundation</td>
<td>132</td>
<td></td>
</tr>
<tr>
<td>Chapter 4: Experimental Program</td>
<td>136</td>
<td></td>
</tr>
<tr>
<td>4 Experimental Program</td>
<td>136</td>
<td></td>
</tr>
<tr>
<td>4.1 Introduction</td>
<td>136</td>
<td></td>
</tr>
<tr>
<td>4.2 Test Specimens</td>
<td>136</td>
<td></td>
</tr>
<tr>
<td>4.3 Material Properties</td>
<td>139</td>
<td></td>
</tr>
<tr>
<td>4.3.1 Concrete</td>
<td>139</td>
<td></td>
</tr>
<tr>
<td>4.3.2 Steel Reinforcement</td>
<td>144</td>
<td></td>
</tr>
<tr>
<td>4.3.3 Steel Fibres</td>
<td>145</td>
<td></td>
</tr>
<tr>
<td>4.4 Specimen Construction</td>
<td>145</td>
<td></td>
</tr>
</tbody>
</table>
4.4.1 Formwork Construction ................................................................. 146
4.4.2 Reinforcement Preparation and Placement ........................................ 147
4.4.3 Concrete Casting ............................................................................. 149
4.5 Test Setup .......................................................................................... 152
4.6 Test Instrumentation .......................................................................... 155
  4.6.1 Strain Gauges ................................................................................. 155
  4.6.2 Linear Variable Differential Transducers (LVDTs) ................................ 158
4.7 Test Loading Protocol ......................................................................... 160

Chapter 5: Experimental Results ............................................................. 162
5 Experimental Results ........................................................................... 162
  5.1 Introduction ......................................................................................... 162
  5.2 LTM Results ....................................................................................... 162
    5.2.1 Test Observations ..................................................................... 163
    5.2.2 Experimental Response .............................................................. 169
  5.3 LTF Results ....................................................................................... 170
    5.3.1 Test Observations ..................................................................... 170
    5.3.2 Experimental Response .............................................................. 174
  5.4 LLM Results ....................................................................................... 180
    5.4.1 Test Observations ..................................................................... 180
    5.4.2 Experimental Response .............................................................. 185
  5.5 LLF Results ....................................................................................... 187
    5.5.1 Test Observations ..................................................................... 187
    5.5.2 Experimental Response .............................................................. 191
  5.6 LFM Results ....................................................................................... 194
    5.6.1 Test Observations ..................................................................... 195
    5.6.2 Experimental Response .............................................................. 202
5.7 LFF Results .................................................................203
  5.7.1 Test Observations .................................................204
  5.7.2 Experimental Response .............................................207

Chapter 6: Discussion of Experimental Results ....................................................213
  6 Discussion of Experimental Results .........................................................213
    6.1 Introduction ..................................................................................213
    6.2 Limitations and Sources of Error ......................................................213
    6.3 Comparison of Beams Subjected to Monotonic Loading .........................215
    6.4 Comparison of Beams Subjected to Fatigue Loading ..............................220

Chapter 7: Finite Element Modelling .................................................................227
  7 Finite Element Modelling ............................................................................227
    7.1 Introduction ..................................................................................227
    7.2 Finite Element Models ......................................................................228
      7.2.1 Meshing ..................................................................................228
      7.2.2 Support Conditions .................................................................229
      7.2.3 Loading Conditions ..................................................................229
      7.2.4 Analytical Models .................................................................232
    7.3 Results and Analysis ..........................................................................233
      7.3.1 Beams Subjected to Monotonic Loading .............................233
      7.3.2 Beams Subjected to Fatigue Loading ...........................................239

Chapter 8: Conclusions and Recommendations .............................................244
  8 Conclusions and Recommendations .........................................................244
    8.1 Conclusions ..................................................................................244
    8.2 Recommendations .........................................................................245

References ...........................................................................................................246

Appendix A: Steel Fibre Data Sheet and Mixing Recommendations .....................259
List of Tables

Table 2.1: Offshore wind turbine foundation types (International Renewable Energy Agency, 2012) .......................................................... 11

Table 2.2: Summary of test results for Higai experiment (Higai, 1978) ......................... 55

Table 2.3: Test beams for Isojeh and Vecchio (2016) ..................................................... 64

Table 3.1: Different load cases for the wind turbine foundation ...................................... 71

Table 3.2: Partial load safety factors given in IEC 61400 .................................................. 72

Table 3.3: Section boundaries for the analysis of the turbine foundation ....................... 84

Table 3.4: Crack width values suggested by ACI Committee 224. Taken from ACI 224R-01. 120

Table 3.5: Steel S-N curve parameters (European Committee for Standardization, 2004) .... 123

Table 3.6: Range of the modulus of subgrade reaction, ks. Note that qa refers to the allowable soil bearing capacity (Bowles, 1997) .......................................................... 134

Table 4.1: Test specimen details ...................................................................................... 137

Table 4.2: Concrete order specifications for set 1 and set 2 beams .................................. 139

Table 4.3: Concrete order specifications for set 3 beams .................................................. 139

Table 4.4: Concrete strength parameters ......................................................................... 144

Table 4.5: Parameters of the coupon tests ....................................................................... 145

Table 4.6: Original length of each LVDT in each shear span of the test beams .................. 160

Table 4.7: Fatigue load parameters used in the tests ......................................................... 161

Table 6.1: Summary of beams subjected to monotonic loading ...................................... 219

Table 6.2: Summary of beams subjected to fatigue loading ............................................ 226
Table 7.1: Fatigue loads used in modelling ............................................................... 232
Table 7.2: Analytical models used with the test beams ........................................... 233
Table 7.3: Summary of modelling beams subjected to fatigue loading .................. 243
List of Figures

Figure 1.1: Common components of a wind turbine (NewEn Inc., 2013) ............................................................... 2

Figure 1.2: Wind turbine anchor ring (Goransson & Nordenmark, 2011) ......................................................... 3

Figure 1.3: Collapsed wind turbine at the Fenner wind plant (The Syracuse, 2010) ................................. 4

Figure 2.1: Pile foundations: a) Pile group and cap; b) Solid mono-pile; c) Hollow mono-pile
(Ashlock & Schaefer, 2011) .......................................................................................................................... 7

Figure 2.2: Circular slab wind turbine foundation (Grasmere Wind Farm, 2011) .............................. 8

Figure 2.3: Octagonal spread foundation (Special Formwork, 2011) ......................................................... 8

Figure 2.4: Cylindrical gravity foundation (Maritime Journal, 2014) .............................................................. 9

Figure 2.5: Pile foundations: a) Plain slab b) Stub and pedestal; c) Stub tower embedded in tapered slab; d) Slab held down by rock anchors (Ashlock & Schaefer, 2011) ........................................ 9

Figure 2.6: Soil pressure distribution for concentric loading: a) uniform pressure, b) cohesive soil pressure, c) granular soil pressure (Brzev & Pao, 2006) ................................................................. 14

Figure 2.7: Soil pressure distribution due to pure bending (Brzev & Pao, 2006) .................................. 15

Figure 2.8: Soil pressure distribution for small eccentricity (Brzev & Pao, 2006) .............................. 16

Figure 2.9: Soil pressure distribution for the boundary eccentricity (Brzev & Pao, 2006) ......... 17

Figure 2.10: Soil pressure distribution for large eccentricities (Brzev & Pao, 2006) .................. 17

Figure 2.11: Fatigue forces acting on the wind turbine foundation and the induced fatigue failure ......................................................................................................................... 19

Figure 2.12: Typical number of cycles for different structures experiencing fatigue loading
(Goransson & Nordenmark, 2011) ........................................................................................................ 20

Figure 2.13: Fatigue loading (Caceres, 2004) ......................................................................................... 21
Figure 2.14: Sample S-N curve (Caceres, 2004) .................................................. 22

Figure 2.15: Graphical representation of Miner’s Rule (Guo, 2014) .......................... 23

Figure 2.16: Modified Goodman diagram (Mallet, 1991) ........................................ 24

Figure 2.17: Stress-strain curve of a concrete prism under repeated compression (Guo, 2014) ... 28

Figure 2.18: Fatigue failure envelope of concrete (Ekberg, Walther, & Slutter, 1957) ........... 28

Figure 2.19: Conditions around a crack in concrete (L = linear zone, N = nonlinear zone, F= fracture zone) (Bazant 1985) .................................................................................. 32

Figure 2.20: Development of a crack according to the fictitious crack model from (a) to (b) (Hillerborg, 1983) .................................................................................................................. 33

Figure 2.21: Evolution of fatigue in the cross section of the reinforcement (Rocha & Bruhwiler, 2012) ................................................................................................................................. 34

Figure 2.22: Cross section of fractured reinforcement (Guo, 2014) ..................................... 34

Figure 2.23: Lug base radius (r) and lug height (h) in a reinforcing bar (Afseth, 1993) ........ 36

Figure 2.24: Commonly used bond tensile fatigue test (Guo, 2014) ................................. 39

Figure 2.25: Typical S-N curve from a fatigue bond test (Guo, 2014) ................................. 39

Figure 2.26: Typical bond stress vs. slip behaviour of a bar (Guo, 2014) ............................. 40

Figure 2.27: Concrete damage (bond degradation) around a reinforcing bar with lugs (Afseth, 1993) .......................................................................................................................... 40

Figure 2.28: Failure of one beam under two-point fatigue bending test (Al-Rousan & Issa, 2011) ............................................................................................................................................. 43

Figure 2.29: Shear stress distribution in an uncracked rectangular beam (Wight & MacGregor, 2012) ................................................................................................................................. 43
Figure 2.30: Principal compressive stress trajectories in an uncracked beam (Wight & MacGregor, 2012) ................................................................. 44

Figure 2.31: Section of beam between two shear cracks (Wight & MacGregor, 2012) ............ 45

Figure 2.32: Free-body diagram at an inclined crack (Brzev & Pao, 2006)............................. 46

Figure 2.33: Shear-compression failure of a beam without shear reinforcement (Wight & MacGregor, 2012) ........................................................................................................................................................................ 47

Figure 2.34: Cracking pattern in diagonal tension failure of a beam without shear reinforcement (Wight & MacGregor, 2012) ........................................................................................................................................................................ 47

Figure 2.35: The effect of a/d on the failure of beams without shear reinforcement (Wight & MacGregor, 2012) ........................................................................................................................................................................ 48

Figure 2.36: Effect of the depth of the beam on its shear resistance (Collins and Kuchman, 1999) ........................................................................................................................................................................ 50

Figure 2.37: Shear fatigue failure of a reinforced concrete beam by fracture of reinforcement intersecting major crack (Zanuy et al., 2009) ........................................................................................................................................................................ 51

Figure 2.38: Experimental result of Zanuy (2008) experiment: a) evolution of cracks, b) failure by shear-compression .................................................................................................................................................................................................. 52

Figure 2.39: Number of cycles to diagonal cracking versus the maximum shear force (Chang & Kesler, 1958) .................................................................................................................................................................................................. 53

Figure 2.40: Failure patterns in the experiment by Chang and Kesler (1958): a) shear-compression, b) diagonal tension .................................................................................................................................................................................................. 53

Figure 2.41: Failure modes for different a/d ratios suggested by Higai (1978) ....................... 56

Figure 2.42: Free-body diagram at an inclined shear crack for a beam with stirrups (Wight & MacGregor, 2012) .................................................................................................................................................................................................. 57
Figure 2.43: Shear resistance in a beam with stirrups (Note: $V_{cy} =$ concrete shear resistance, $V_d =$ dowel action, $V_{ay} =$ aggregate interlock, $V_s =$ stirrups resistance) (Wight & MacGregor, 2012) 58

Figure 2.44: Truss model for a beam with shear reinforcement (Brzev & Pao, 2006) ............... 59

Figure 2.45: Arch action in a beam (Wight & MacGregor, 2012) ............................................. 61

Figure 2.46: Strut and tie truss for a simply supported beam subjected to concentrated load (Wight & MacGregor, 2012) ......................................................................................... 62

Figure 2.47: Strut and tie truss for a simply supported beam with transverse reinforcement (Wight & MacGregor, 2012) ......................................................................................... 62

Figure 2.48: Major shear crack in a deep beam (Zanuy et al. 2008) ........................................ 63

Figure 2.49: Crack patterns at failure of Isojeh and Vecchio (2016) beams ......................... 65

Figure 3.1: Key geometric parameters of the foundation .......................................................... 75

Figure 3.2: Collapsed wind turbine due to overturning of foundation (Maunu, 2008) ............ 75

Figure 3.3: Different possible loading distributions for the soil contact pressure ................. 76

Figure 3.4: Free body diagram of a wind turbine foundation subject to full trapezoidal soil contact pressure .................................................................................................................. 76

Figure 3.5: Free body diagram of a wind turbine foundation subject to uniform soil contact pressure ......................................................................................................................... 77

Figure 3.6: Free body diagram of a wind turbine foundation subject to triangular soil contact pressure ......................................................................................................................... 77

Figure 3.7: Effective area of a square foundation, Approach 1 (DNV/RISO, 2002) .............. 79

Figure 3.8: Effective area of a square foundation, Approach 2 (DNV/RISO, 2002) ............. 80

Figure 3.9: Effective area of a circular or an octagonal foundation (DNV/RISO, 2002) ........ 80
Figure 3.10: One meter wide strip for the sectional analysis of the foundation (Svensson, 2010) ................................................................. 83

Figure 3.11: Complete free body diagram of a gravity based foundation ........................................ 83

Figure 3.12: Sections considered for the sectional analysis of the turbine foundation .............. 84

Figure 3.13: Shear force and bending moment diagrams of the foundation ............................. 86

Figure 3.14: Shear force and bending moment diagrams of the foundation subject to uniform soil pressure ................................................................. 87

Figure 3.15: Equivalent concrete stress block (Brzev & Pao, 2006) ...................................... 88

Figure 3.16: Finding the moment resistance of a cross section using equivalent stress block (Brzev & Pao, 2006) ................................................................. 89

Figure 3.17: One-meter side section for flexural reinforcement design (Svensson, 2010) ....... 90

Figure 3.18: Reinforcement in circular foundations (Wind Farms Construction, 2012) ......... 90

Figure 3.19: Reinforcement in octagonal foundations (Sritharan, 2011) ............................... 91

Figure 3.20: Reinforcement in square foundations (B. McCaffrey & Sons Ltd., 2015) ........... 91

Figure 3.21: Critical section for one way shear for a rectangular foundation ...................... 92

Figure 3.22: One-way shear crack ....................................................................................... 92

Figure 3.23: Punching shear failure of the foundation (McCormac & Brown, 2014) ............ 95

Figure 3.24: Tributary area of two-way (punching) shear of a square foundation ................. 96

Figure 3.25: Headed shear reinforcement assembly (American Concrete Institute (ACI), 1999) .......................................................................................................................... 97

Figure 3.26: Typical arrangement of headed shear studs around a square column (American Concrete Institute (ACI), 1999) ................................................................. 98
Figure 3.27:Collapsed wind turbine tower due to detachment from foundation (Goodwin, 2014) ................................................................................................................................. 99
Figure 3.28: Typical insert ring (Hassanzadeh, 2012) ................................................................. 100
Figure 3.29: Top ring flange on top of the concrete foundation pedestal (Hassanzadeh, 2012) 101
Figure 3.30: Bolts through the flange of the turbine tower (Boltight Ltd.) ................................. 101
Figure 3.31: Cracking patterns of an insert ring with two embedded flanges (Hassanzadeh, 2012) ........................................................................................................................................ 102
Figure 3.32: U-hoops around the insert ring as anchor reinforcement (Svensson, 2010) ........ 102
Figure 3.33: The role of the U-hoops (suspension reinforcement) in resisting the transferred loads (Landen & Lilljegren, 2012) ............................................................................................................................... 103
Figure 3.34: Typical anchor bolt cage (Hassanzadeh, 2012) ...................................................... 104
Figure 3.35: Space left between tower base flange and pedestal surface (Hassanzadeh, 2012) 105
Figure 3.36: Bolts in PVC sleeves or grout holes (Hassanzadeh, 2012) ..................................... 105
Figure 3.37: Typical configuration of an anchor bolt cage (Vestas American Wind Technology Inc., 2004) ......................................................................................................................... 106
Figure 3.38: Anchor bolt cage configuration with a steel adaptor connected to the top flange (Maunu, 2008) .................................................................................................................. 106
Figure 3.39: Embedded steel ring plate of the anchor bolt cage ................................................ 108
Figure 3.40: Common layout for pedestal reinforcement in the presence of anchor bolt cage .. 110
Figure 3.41: The bearing capacity areas used in calculations (Bowles, 1997) .............................. 111
Figure 3.42: Dowels extending from the pedestal to the footing (McCormac & Brown, 2014) 112
Figure 3.43: Proposed strut-and-tie model for the foundation ..................................................... 113
Figure 3.44: Strut-and-tie model of the foundation proposed by Goransson and Nordenmark (2011) .......................................................... 113

Figure 3.45: Available anchorage length for a given tensile force, F_s. Taken from EN 1992.1.1-2004 .......................................................... 115

Figure 3.46: Crack control parameters in Gergely and Lutz (1968) equation (Brzev & Pao, 2006) .......................................................... 119

Figure 3.47: S-N curve used for steel by the EN (European Committee for Standardization, 2004) .......................................................... 123

Figure 3.48: Definitions of \( \sigma_{c1} \) and \( \sigma_{c2} \) (International Federation for Structural Concrete, 2010) .......................................................... 125

Figure 3.49: Variation of the longitudinal reinforcement damage with the reinforcement amount (Goransson & Nordenmark, 2011) .......................................................... 127

Figure 3.50: Variation of the shear reinforcement damage with the reinforcement amount (Goransson & Nordenmark, 2011) .......................................................... 127

Figure 3.51: The progression of fatigue damage in the cross section of the reinforcement (Isojeh & Vecchio, 2016) .......................................................... 130

Figure 3.52: Concrete strut principal compressive strain evolution (Isojeh & Vecchio, 2016) .......................................................... 131

Figure 4.1: Beams nominal dimensions (all dimensions are in mm) .......................................................... 137

Figure 4.2: Detailing of Set 1 beams (LTM and LTF) (all dimensions are in mm) .......................................................... 138

Figure 4.3: Detailing of Set 2 and Set 3 beams (all dimensions are in mm) .......................................................... 138

Figure 4.4: Consistency of the concrete mix for Set 1 and Set 2 beams .......................................................... 140

Figure 4.5: Consistency of the fibre-reinforced concrete mix .......................................................... 141

Figure 4.6: Cylinder compressive strength test set up .......................................................... 141
Figure 4.7: Crushing of the plain concrete cylinder ......................................................... 142
Figure 4.8: Crushing of the SFRC concrete cylinder ......................................................... 142
Figure 4.9: Modulus of rupture (MOR) test set up .......................................................... 143
Figure 4.10: Failure pattern of the MOR beam for Set 1 and Set 2 beams ......................... 143
Figure 4.11: Failure pattern of the MOR beam for Set 3 beams ........................................ 143
Figure 4.12: Hooked Bekaert steel fibre ........................................................................... 145
Figure 4.13: Formwork constructed .................................................................................. 146
Figure 4.14: Slots in the formwork where beams were cast .............................................. 147
Figure 4.15: Wooden posts placed after inserting the reinforcement cages into the forms ...... 147
Figure 4.16: Assembly of rebar cage using wooden chairs and ribbon ties ....................... 148
Figure 4.17: Welding of out-of-plane bar piece to maintain spacing between bars ............... 148
Figure 4.18: Reinforcement cage placed in the formwork ............................................... 149
Figure 4.19: Finished concrete surface ............................................................................ 149
Figure 4.20: Balling up of fibres in the concrete mix ....................................................... 150
Figure 4.21: High-speed portable concrete mixer .............................................................. 151
Figure 4.22: Rotating metal paddles in the mixer .............................................................. 152
Figure 4.23: Voids at the sides of the Set 3 beams ............................................................. 152
Figure 4.24: Intended support conditions of the beam (all dimensions are in mm) .............. 153
Figure 4.25: Roller used in the test .................................................................................. 153
Figure 4.26: Actual support conditions of the beam (all dimensions are in mm) ............... 153
Figure 4.27: Test setup front view ................................................................. 154
Figure 4.28: Test setup side view .................................................................. 154
Figure 4.29: Strain gauge used in the experiments ......................................... 155
Figure 4.30: Properties of the strain gauges used in the experiments ........... 156
Figure 4.31: Strain gauges in LLF and LFF beams (all dimensions are in mm) . 156
Figure 4.32: Strain gauges in the LTF beam (all dimensions are in mm) .......... 157
Figure 4.33: Strain gauge labels for LLF and LFF (all dimensions are in mm) 157
Figure 4.34: Strain gauge labels for LTF (all dimensions are in mm) .............. 157
Figure 4.35: LVDTs placed in each beam (all dimensions are in mm) .......... 158
Figure 4.36: Symbols for each LVDT used (all dimensions are in mm) ......... 160
Figure 5.1: LTM beam before cracking ............................................................ 165
Figure 5.2: LTM at the cracking load (186 kN) .............................................. 165
Figure 5.3: LTM at 195 kN ........................................................................... 166
Figure 5.4: LTM at 238 kN ........................................................................... 166
Figure 5.5: LTM at 278 kN ........................................................................... 167
Figure 5.6: LTM at 359 kN ........................................................................... 167
Figure 5.7: LTM at 400 kN ........................................................................... 168
Figure 5.8: LTM at failure (497 kN) .............................................................. 168
Figure 5.9: Load versus displacement experimental response for LTM .......... 169
Figure 5.10: Load versus maximum crack width for LTM .......................... 170
Figure 5.11: LTF after one cycle ................................................................. 172
Figure 5.12: LTF at failure (after 37316 cycles) ........................................ 172
Figure 5.13: LTF at failure from the back side ............................................. 173
Figure 5.14: Location of reinforcement fracture in LTF ................................ 173
Figure 5.15: Fractured reinforcement of LTF .............................................. 174
Figure 5.16: Mid-span deflection versus number of load cycles for LTF ........ 175
Figure 5.17: Strains on the left shear span of LTF ...................................... 176
Figure 5.18: Strains on the right shear span of LTF ..................................... 176
Figure 5.19: Evolution of principal tensile strain for LTF ............................ 177
Figure 5.20: Evolution of principal compressive strain for LTF .................... 177
Figure 5.21: Readings of strain gauges along longitudinal reinforcement of LTF ........ 178
Figure 5.22: Readings of strain gauges along the left transverse reinforcement of LTF .... 179
Figure 5.23: Readings of strain gauges along the right transverse reinforcement of LTF ...... 179
Figure 5.24: Uncracked LLM beam .............................................................. 182
Figure 5.25: LLM at the first crack load (206 kN) ........................................ 182
Figure 5.26: LLM at 226 kN ....................................................................... 183
Figure 5.27: LLM at 276 kN ....................................................................... 183
Figure 5.28: LLM at 300 kN ....................................................................... 184
Figure 5.29: LLM at 322 kN ....................................................................... 184
Figure 5.30: LLM at 370 kN ....................................................................... 185
Figure 5.31: LLM at failure (450 kN) ................................................................. 185
Figure 5.32: Experimental load versus mid-span deflection for LLM ...................... 186
Figure 5.33: Experimental load versus maximum crack width for LLM .................... 187
Figure 5.34: LLF after one load cycle .................................................................... 189
Figure 5.35: Potential spalling location at the right shear span of LLF ....................... 189
Figure 5.36: LLF after 5360 load cycles ................................................................. 189
Figure 5.37: LLF after 11504 load cycles ............................................................... 190
Figure 5.38: LLF at failure (after 36748 load cycles) .............................................. 190
Figure 5.39: Fractured longitudinal bar at failure of LLF ....................................... 190
Figure 5.40: Experimental mid-span deflection versus number of cycles for LLF ........ 191
Figure 5.41: Strains on the left shear span of LLF .................................................. 192
Figure 5.42: Strains on the right shear span of LLF ............................................... 192
Figure 5.43: Evolution of principal tensile strains for LLF ..................................... 193
Figure 5.44: Evolution of principal compressive strains for LLF ......................... 193
Figure 5.45: Strain gauge readings for LLF ............................................................. 194
Figure 5.46: LFM at 195 kN .................................................................................. 197
Figure 5.47: LFM at 230 kN .................................................................................. 197
Figure 5.48: LFM at 270 kN .................................................................................. 198
Figure 5.49: LFM at 282 kN .................................................................................. 198
Figure 5.50: LFM at 350 kN .................................................................................. 199
Figure 5.51: LFM at 376 kN after the drop from the peak load of 397 kN

Figure 5.52: LFM at 255 kN post-peak

Figure 5.53: Cracks at the left end of the beam indicating bond degradation for LFM at 200 kN post-peak

Figure 5.54: LFM at ultimate degradation and failure

Figure 5.55: Bursting of tension hooks at the left end of LFM

Figure 5.56: Major shear crack of LFM at the left support

Figure 5.57: Experimental load versus mid-span deflection for LFM

Figure 5.58: Experimental load versus maximum crack width for LFM

Figure 5.59: LFF after the first load cycle

Figure 5.60: The major shear crack of LFF after the first load cycle

Figure 5.61: The major shear crack of LFF at failure (after 4152 load cycles)

Figure 5.62: The major shear crack of LFF at failure from the back of the beam

Figure 5.63: Crushing of the concrete at the left support of LFF at failure

Figure 5.64: Vertical cracking at the left side of LFF at failure

Figure 5.65: Mid-span deflection versus number of cycles for LFF

Figure 5.66: Strains on the left shear span of LFF

Figure 5.67: Strains on the right shear span of LFF

Figure 5.68: Strains on the left shear span of LFF magnified

Figure 5.69: Principal tensile strain evolution of LFF

Figure 5.70: Principal compressive strain evolution of LFF
Figure 5.71: Strain gauges along the longitudinal reinforcement of LFF .......................... 212

Figure 6.1: Experimental response of beams subjected to monotonic loading .................. 215

Figure 6.2: Maximum crack width evolution of beams subjected to monotonic loading .... 216

Figure 6.3: Evolution of principal tensile strains for beams subjected to fatigue loading .... 220

Figure 6.4: Evolution of principal compressive strains for beams subjected to fatigue loading 221

Figure 6.5: Evolution of mid-span deflection for beams subjected to fatigue loading ........ 221

Figure 7.1: FE mesh of LLM, LLF, LFM, and LFF .................................................. 229

Figure 7.2: FE mesh of LTM and LTF ................................................................. 229

Figure 7.3: Fatigue special provision in VecTor2 job file ........................................ 230

Figure 7.4: Fatigue analysis parameters in VecTor2 .............................................. 231

Figure 7.5: Experimental and analytical responses for LTM ...................................... 234

Figure 7.6: Experimental and analytical cracking patterns for LTM ............................... 235

Figure 7.7: Analytical principal compressive stress contour for LTM .............................. 235

Figure 7.8: Experimental and analytical responses for LLM ........................................ 236

Figure 7.9: Experimental and analytical cracking patterns for LLM ............................... 237

Figure 7.10: Arching mechanism in LLM ................................................................. 237

Figure 7.11: Experimental and analytical responses for LFM ...................................... 238

Figure 7.12: Experimental and analytical cracking patterns for LFM .............................. 239

Figure 7.13: Fatigue life prediction of LTF ................................................................. 240

Figure 7.14: Fatigue life prediction of LLF ................................................................. 241
Figure 7.15: Fatigue life prediction of LFF .......................................................... 242
List of Appendices

Appendix A: Steel Fibre Data Sheet and Mixing Recommendations

Appendix B: Material Stress-Strain Curves

Appendix C: Grout Specifications
Chapter 1: Introduction

1 Introduction

1.1 Overview

Wind turbines are devices that capture the kinetic energy of the wind and transform it into electricity. They have been key machines in generating renewable energy, a sustainable practice that has the potential in the future to reduce or eliminate the dependence on fossil fuels to produce electricity. Wind turbines in Canada currently provide enough electricity to meet the needs of over three million Canadian homes, or six percent of the country’s electricity demand (The Canadian Wind Energy Association, 2015).

Most wind turbines today are horizontal-axis machines having a bladed rotor spinning in the vertical plane (US Office of Energy Efficiency and Renewable Energy, 2015). The basic components of such wind turbines include the rotor, nacelle, tower, and foundation (American Wind Energy Association, 2013). The rotor includes the blades which rotate due to the kinetic energy of the wind. They are typically made from materials that have a high strength-to-weight ratio (e.g. fiberglass), and are shaped in a way that creates differential pressure on different points of the blades, causing them to spin when facing the wind (American Wind Energy Association, 2013). The nacelle can be thought of as the “head” of the wind turbine, encasing most of the vital components that control the performance. These components include, but are not limited to, the low-speed shaft, gear box, high-speed shaft, generator, anemometer, and controller. The rotation of the rotor due to the wind enables the low-speed shaft to rotate. This shaft is connected to the gear box, which contains multiple gears that transfer the low-speed rotation due to the wind (about 20 rpm) to a high-speed rotation (about 1200 rpm) capable of producing the required electrical power (American Wind Energy Association, 2013). The high-speed rotation happens in the high-speed shaft, which is connected to the generator that produces electricity as a result. The anemometer is used to measure wind speed and direction. It sends the information to the controller, which is a computer system that controls the wind turbine. It adjusts the direction of the blades depending on the wind direction, and is capable of stopping the turbine under certain conditions, such as the occurrence of very high winds capable of damaging the blades. The controller can be accessed remotely from a computer to check the status of the system and make adjustments.
The tower of the wind turbine puts the blades at high elevations so that stronger winds can be encountered. The foundation holds the whole assembly together, maintains its stability, and prevents it from collapsing and overturning. It must be able to resist all the different loads imposed on it. Figure 1.1 shows the different common components of wind turbines.

![Diagram of wind turbine components](image)

**Figure 1.1: Common components of a wind turbine (NewEn Inc., 2013)**

The foundation is the backbone of the wind turbine. The tower, nacelle, hub, and rotor stand erect due to the foundation. It transmits different types of loads exerted by the wind turbine to the ground, provides stability to the structure, controls settlements, and prevents overturning. It must
be able to withstand the different loads imposed by the wind turbine structure, as well as the pressure exerted by the soil. In addition, displacement and rotation limits must be met. For example, the maximum rotation at the pile head and the maximum accumulated permanent rotation resulting from cyclic loading over the design life must be within the allowable tolerances (Malhotra, 2011). The choice of a specific foundation type and system depends on many factors including the soil conditions, size of the wind turbine, nature of the loading on the foundation, cost limitations, and field access limitations (Svensson, 2010). Note that the transfer of forces between the foundation and the tower is done through what is known as an anchor ring or bolt cage anchor, which is a rigid steel component connecting the foundation to the tower, as shown in Figure 1.2. It is embedded into the foundation and extends a certain level above the foundation surface, to which the tower is connected through prestressed bolts. The anchor ring can be thought of and idealized as a steel I-beam rolled circularly. Anchorage into the concrete foundation is provided by the flanges of the ring as well as the friction between the webs of the ring and the surrounding concrete. Grouting can also be used to provide further anchorage. Sometimes a soft layer is put under the bottom flange of the anchor ring to prevent a local punching failure through the concrete.

![Figure 1.2: Wind turbine anchor ring (Goransson & Nordenmark, 2011)](image)

Gravity-based foundations are typically reinforced concrete structures that rely on their mass to provide the required rigidity, stability, and resistance against overturning and sliding. The tower transmits vertical and horizontal forces as well as overturning and twisting moments to the foundation. In addition, cyclic forces are exerted on the foundation as a result of the rotation of the rotor blades, making the foundation susceptible to fatigue failure especially in the shear span that forms between the resultant of the soil reaction force and the compressive component of the cyclic forces. Shear fatigue failure of the foundation can be catastrophic if it is not considered properly, as shown in Figure 1.3. The design for fatigue resistance has always been through the consideration of independent material fatigue damage (i.e. steel and concrete separately) by linearly adding the
damages through what is known as Miner’s rule. However, the use of the global strain accumulation as a measure of fatigue damage has been proposed as an alternative and more refined method of design (Isojeh & Vecchio, 2016). The validity of these proposed models need to be verified and corroborated.

Conventionally, the wind turbine foundation is thickened (and sometimes more steel is used) to prevent fatigue failure, which increases the material costs; cost-effective means to increase the fatigue resistance of the foundation need to be investigated. Steel fibre-reinforced concrete (SFRC) is proposed as a possible solution to increase the fatigue resistance without the need to thicken the section or use conventional shear reinforcement. Accordingly, the behaviour of steel fibre-reinforced concrete under fatigue loading must be investigated to assess the contribution of steel fibres to the fatigue resistance of the section as compared to that of the conventional shear reinforcement.

1.1 Research Objectives

The main objectives of this thesis are the following:

1. To provide a detailed and coherent summary of the design procedure of reinforced concrete gravity-based wind turbine foundations.
2. To verify the validity and accuracy of the fatigue damage models proposed by Isojeh et al. (2016) and compare them with the traditional methods based on the S-N curves and Miner’s rule, so as to determine their suitability for use in fatigue design.

3. To examine the contribution of traditional shear reinforcement to the fatigue resistance of reinforced concrete deep beams in order to assess their reliability in improving the fatigue life of wind turbine foundations.

4. To investigate the possibility of using steel fibre-reinforced concrete (SFRC) as a superior alternative to traditional shear reinforcement for fatigue resistance and assess the contribution of steel fibres to the fatigue resistance of reinforced concrete.

In order to achieve the second and third objectives, an experimental program was designed and carried out in the structural laboratories at the University of Toronto. Six large-scale longitudinally reinforced concrete deep beams (4000 mm length x 1040 mm depth x 200 mm thickness) were constructed in three sets. All the sets contained the same amount of longitudinal reinforcement. Each set contained two beams of the same detailing. The first set consisted of plain concrete, while the second and third sets contained shear reinforcement and steel fibres, respectively. For each set, one beam was subjected to monotonic point loading while the other underwent fatigue loading. This enabled the assessment of the degree of fatigue damage, measured in terms of the degradation of strength and stiffness, as compared to the monotonic control cases. Since all the fatigue tests were performed under the same load levels, direct comparison of the performance of the beam containing shear reinforcement and the other containing steel fibres was possible. Finally, the fatigue damage models developed at the University of Toronto, Isojeh et al. (2016), can be verified by comparing their results with the experimental results of the beams.

1.2 Organization of the Thesis

This thesis summarizes the design process of reinforced concrete gravity-based wind turbine foundations and provides an experimental program to investigate the fatigue behaviour of steel fiber-reinforced concrete (SFRC) beams as they compare to that of reinforced concrete beams containing conventional shear reinforcement. Chapter 1 provided an overview of wind turbine foundations and discussed the research objectives.

Chapter 2 provides a literature review discussing: the different types of wind turbine foundations; the various loads acting on the gravity-based foundation; the different parameters and diagrams
used to describe the fatigue loading and response of a structure; the various factors affecting the fatigue strength of concrete, steel reinforcement, and the bond between them; the fracture mechanics of concrete and steel; and the monotonic and fatigue shear behaviour of slender and deep reinforced concrete beams.

Chapter 3 breaks down the design process of reinforced concrete gravity-based wind turbine foundations into detailed and coherent steps. In addition, the proposed fatigue models are presented and discussed in the section addressing the fatigue design of the foundation.

In Chapter 4, the experimental program is discussed. The details of the beams, including their dimensions and reinforcement layout, are given. Casting and curing procedures are addressed. The different types of instrumentations used are explained. The testing setup, including the supporting conditions and loading parameters, is described. Additionally, the results of the supplementary material tests (concrete cylinder compressive tests, modulus of rupture (MOR) tests, and steel coupon tests) are presented.

Chapter 5 presents the experimental results and the different plots of the tests performed. Detailed observations of the tests are presented, as well as photographs detailing the cracking patterns and failures of each test.

Chapter 6 provides a detailed analysis of the results including the discussion of the load-deflection response, cracking patterns, failure modes, degradation due to fatigue, and comparisons of the different responses of the beams under fatigue loading. Governing mechanisms that might have affected the results are also discussed.

Chapter 7 deals with the finite element analysis of the test beams, utilizing the proposed fatigue damage models incorporated in VecTor2. Note that VecTor2 is a nonlinear finite element software dedicated to the analysis of reinforced concrete structures, specifically two-dimensional membrane structures.

Chapter 8 provides conclusions drawn from the analysis of the experimental results and presents recommendations for future work.
Chapter 2: Literature Review

2 Literature Review

2.1 Wind Turbine Foundation Types

2.1.1 Onshore Foundations

For onshore wind turbines, there are two primary types of foundations: pile foundations and gravity-based foundations. A pile foundation is used when the near-surface soil has insufficient bearing capacity to withstand the loads transferred from the structure (e.g. clay). Hence, piles are driven, drilled, or jacked deep into the soil until a layer with sufficient bearing capacity is reached. The piles are then connected to a pile cap. The cap distributes the load from the structure into the ground and facilitates efficient sharing of the load by the piles (Ashlock & Schaefer, 2011). The connection between the piles and the pile cap falls between the two extremes of clamped rigid connection and hinged connection. Some variations of the pile foundations do not include a pile cap, such as a mono-pile foundation. The piles can be timber, steel, or concrete piles, although concrete piles are the predominantly used ones. Figure 2.1 shows some pile foundations variants.

![Figure 2.1: Pile foundations: a) Pile group and cap; b) Solid mono-pile; c) Hollow mono-pile (Ashlock & Schaefer, 2011)]

A gravity-based foundation is used when the top soil is strong enough to support the loads from the wind turbine. It is important to consider how far the water table is below the top soil when assessing the top soil’s capacity (DNV/RISO, 2002). A gravity-based foundation consists of a large area of concrete at the bottom of the wind turbine structure. This area can vary from rectangular and circular slabs (
Figure 2.2) to octagonal shallow mats (Figure 2.3) and cylindrical foundations (Figure 2.4). The slabs can be level or tapered, and are often placed concentrically under the tower. Tapered foundations slabs usually require less amount of concrete, so they are more economical. In addition, tapering the slab results in less congestion of rebar and ensures that water on the surface gets drained away (Goransson & Nordenmark, 2011). The bigger area and mass of gravity foundations provide stability and protect against overturning moments. The wide foundation brings the resultant of the soil forces closer to the tower, reducing overturning moment (Svensson, 2010). It also enables a smoother transition of the structure forces to the ground by having a large contact surface area, which ensures that the load-bearing capacity of the soil is not exceeded. This type of foundation is suitable even for soils with lower bearing capacities provided that the soil is stiff enough to prevent undesired settlements. The foundation must be able to resist the bending moment and shear force induced by the tower safely, hence proper detailing and dimensioning are required.

Figure 2.2: Circular slab wind turbine foundation (Grasmere Wind Farm, 2011)

Figure 2.3: Octagonal spread foundation (Special Formwork, 2011)
The gravity-based foundation is placed on the ground or below the ground at shallow levels. Hence, the excavation and refilling work required is minimal compared to pile foundations. The overturning moment is mainly resisted by the self-weight of the foundation. If the gravity-based foundation is built into the soil at shallow levels, the top soil might take some part in resisting the overturning moment, reducing the amount of concrete needed for the foundation, but at the expense of requiring more excavation and refilling of soil (Svensson, 2010). Figure 2.5 shows some variations of gravity-based foundation systems. Often in practice the gravity foundation consists of the individual tapered footing rigidly connected to a pedestal at the center, which holds the tower. The backfill usually covers the footing and the pedestal, so the ground level starts from the top of the pedestal.

Figure 2.4: Cylindrical gravity foundation (Maritime Journal, 2014)

Figure 2.5: Pile foundations: a) Plain slab b) Stub and pedestal; c) Stub tower embedded in tapered slab; d) Slab held down by rock anchors (Ashlock & Schaefer, 2011)
2.1.2 Offshore Foundations

In the recent years, offshore wind turbine power plants have emerged in many countries. Their higher required capital investment, compared to the onshore power plants, is offset by the higher generation capacities they have (International Renewable Energy Agency, 2012). The offshore environment provides a suitable place for wind power plants: the winds have higher speeds and lower turbulence. In addition, the space limitations are less and there is more proximity to the cities to which electricity is supplied, which reduces electricity transportation costs (International Renewable Energy Agency, 2012). However, offshore wind turbines require slightly more complex designs and considerations including dynamic water wave forces, ship impact loads, and corrosion vulnerability of the structure (International Renewable Energy Agency, 2012). Fortunately, as experience and research on these offshore wind turbines is accumulating, more specific designs and materials are being developed resulting in improved performance and durability. The offshore wind turbines usually have different foundation types to anchor them to the seabed. Table 2.1 lists the common types of such foundations. Note that the focus of this thesis will be on onshore concrete gravity wind turbine foundations, so offshore applications and other types of onshore foundations will not be discussed further.
Table 2.1: Offshore wind turbine foundation types (International Renewable Energy Agency, 2012)

<table>
<thead>
<tr>
<th>Foundation Type</th>
<th>Application</th>
<th>Advantages</th>
<th>Disadvantages</th>
</tr>
</thead>
<tbody>
<tr>
<td>Mono-piles</td>
<td>Most conditions, preferably shallow water and not deep soft material. Up to 4 m diameter. Diameters of 5-6 m are the next step.</td>
<td>Simple, light and versatile. Of lengths up to 35 m.</td>
<td>Expensive installation due to large size. May require pre-drilling a socket. Difficult to remove.</td>
</tr>
<tr>
<td>Multiple piles (tri-pile)</td>
<td>Most conditions, preferably not deep soft material. Suits water depths above 30 m.</td>
<td>Very rigid and versatile.</td>
<td>Very expensive construction and installation. Difficult to remove.</td>
</tr>
<tr>
<td>Concrete gravity base</td>
<td>Virtually all soil conditions</td>
<td>Float-out installation</td>
<td>Expensive due to large weight.</td>
</tr>
<tr>
<td>Steel gravity base</td>
<td>Virtually all soil conditions. Deeper water than concrete.</td>
<td>Lighter than concrete. Easier transportation and installation. Lower expense since the same crane can be used as for erection of turbine.</td>
<td>Costly in areas with significant erosion. Requires a cathodic protection system. Costly compared with concrete in shallow waters.</td>
</tr>
<tr>
<td>Mono-suction caisson</td>
<td>Sands and soft clays</td>
<td>Inexpensive installation. Easy removal.</td>
<td>Installation proven in limited range of materials.</td>
</tr>
<tr>
<td>Floating</td>
<td>Deep waters</td>
<td>Inexpensive foundation construction. Less sensitive to water depth than other types. Non rigid, so lower wave loads.</td>
<td>High mooring and platform costs. Excludes fishing and navigation from areas of farm.</td>
</tr>
</tbody>
</table>

2.2 Loading on Wind Turbine Foundations

There are many types of loads acting on a wind turbine shallow gravity spread foundation: in addition to its own self-weight and the soil contact pressure, the tower transmits force and moments to the foundation. Moreover, the dynamic forces of the wind (caused by the rotation of the blades) produce cyclic loading of variable parameters on the foundation.
2.2.1 Forces Transmitted from the Tower

The tower is connected to the foundation through the anchor ring, which acts more like a rigid connection allowing the transfer of vertical forces, horizontal forces, and bending and torsional moments. The vertical force is caused by the self-weight of the tower and its components, while the horizontal forces are caused by the lateral forces of the wind acting along the height of the tower. These lateral forces also induce an overturning moment on the foundation, which can displace the center of gravity of the wind turbine system from equilibrium, potentially leading to an overturning failure (Maunu, 2008). This moment is also caused by the rotation of the blades and is transferred to the foundation through the bolt cage connection in the form of a force couple. Since the wind can change its direction, the horizontal forces and moments can act on any side of the foundation (i.e. they are not restricted to one plane). Nonetheless, the forces in the vicinity of the anchor ring (i.e. the disturbance region where forces get transferred to the foundation) impose certain demands and stresses, which require special consideration and detailing (discussed in the design section). The demand imposed on the foundation by the tower (i.e. horizontal and vertical forces, as well as moments) is usually given by the wind tower generator supplier to be used in the foundation design.

The forces transmitted from the tower determine the distribution of the soil contact pressure, which significantly influences the design of the foundation as it determines its internal forces and stresses. For example, the distribution of the soil contact pressure under concentric vertical loading (i.e. without an overturning moment) is vastly different from the distribution under eccentric loading (i.e. with the presence of both a vertical force and an overturning moment) (Maunu, 2008).

2.2.2 Soil Contact Pressure

The soil exerts pressure on the foundation at the areas of contact. The soil must have sufficient bearing capacity and rotational stiffness to take in the loads transmitted, without causing excessive settlements. The rotational stiffness of the soil refers to its ability to control and limit rotations about the horizontal axes, preventing overturning. Horizontal stiffness of the soil is also required to prevent sliding. Essentially, the combined stiffness of the soil and the structure is checked to ensure stability; the overall foundation stiffness depends on the stiffness and strength of the soil as well as on the foundation structural elements (Svensson, 2010). The magnitude of the soil bearing capacity is a fundamental parameter in the design of the foundation and depends greatly on the
type of the soil and its properties. The geotechnical engineer will typically suggest allowable bearing capacity and minimum rotational and horizontal stiffness values that can be used in the design. Note that the determination of the bearing capacity of the soil by the geotechnical engineer takes into consideration not only the soil-specific conditions and parameters but also the nature of the structure and its loading (Brzev & Pao, 2006). Different equations are given for the bearing capacity of drained and undrained soil on which a circular, rectangular, or octagonal gravity based foundation is resting (See DNVGL-ST-0126 section G.4). These equations use shape, inclination, and bearing capacity factors (Bowles, 1997). The bearing pressure of the foundation is found by dividing the specified dead and live loads by the area of the foundation in contact with the soil. Surcharge pressure is also accounted for, which includes the service loads acting on the area directly above the foundation, the dead load of the foundation, and the soil overlay (backfill) over the foundation (Brzev & Pao, 2006). Design considerations of the bearing pressure will be discussed in the design section.

Site-specific soil investigations are carried out before the commencement of the foundation design process to obtain the relevant soil parameters (including the allowable bearing pressure) needed in the design. Such investigations normally include a site geological survey, topography survey of the soil surface, in-situ testing, soil sampling for subsequent cyclic and static laboratory testing, and shear wave velocity measurements for the assessment of the soil’s maximum shear modulus (DNV GL, 2016). The choice of soil investigation methods “shall take into account the phase of the project; the type, size, and importance of the wind turbine structure; the actual type of soil deposits and the complexity of soil and terrain conditions” (DNV GL, 2016). Essentially, the purpose is to ensure that the soil can safely hold the structure without catastrophic failure and undesirable differential and consolidation settlements. Hence, for the design of gravity-based foundations, the soil investigations should extend beyond the depth of any critical shear surface (DNV GL, 2016). All layers of soil affected by the structure’s settlements and contributing to its stiffness should also be thoroughly investigated. For seismically active regions, the depths of investigations are increased to reach areas that will have an influence on the design due to the propagation of shear waves as a result of earthquakes. Site-specific seismic parameters to determine the site class for the seismic force calculations are obtained. Scour and erosion of the soil are also issues that are addressed during the soil investigation. Essentially, the aim of these investigations and tests is to ensure that the soil (especially the shallow soil for the gravity-based
foundations) is able to withstand the structure and allow it to fulfill its design goals throughout the operational life. The level of the groundwater is also determined to assess buoyancy (uplift) effects on the foundation and the soil bearing pressure.

As stated earlier, the loads transmitted from the tower to the foundation (through the anchor ring) have a direct influence on the distribution of the soil contact pressure, which greatly determines the internal forces in the foundation, governing its design. Since these loads greatly depend on the wind speed and direction (which vary with time), the resulting soil pressure distribution varies both spatially and with time. Two cases are considered: concentric and eccentric loading of the foundation. Regardless of the resulting pressure distribution, the actual soil pressure (due to the imposed loads on the foundation by the tower) should be less than the allowable soil bearing pressure determined by the geotechnical engineer (Brzev & Pao, 2006).

2.2.2.1 Concentrically Loaded Foundations

Concentric loading on foundations consists of a vertical load transmitted from the tower to the foundation, without the presence of an overturning moment. In this case, the soil contact pressure is approximated as a uniformly distributed pressure (Figure 2.6(a)) although it is higher at the edges for cohesive soils such as clay (Figure 2.6(b)) or at the center for granular soil such as a sandy soil (Figure 2.6(c)) (Brzev & Pao, 2006). The typical rigidity of practical gravity foundations deems the approximation of uniform soil pressure appropriate (Bowles, 1997). The value of the pressure is found by dividing the axial load by the area of the foundation in contact with the soil.

![Figure 2.6: Soil pressure distribution for concentric loading: a) uniform pressure, b) cohesive soil pressure, c) granular soil pressure (Brzev & Pao, 2006)](image-url)
2.2.2.2 Eccentrically Loaded Foundations

In most cases, the vertical axis of the tower is coincident with that of the foundation; hence the vertical load transferred from the tower does not induce a moment in the foundation. Nevertheless, a moment is induced from the lateral wind loading on the tower, resulting in eccentric loading conditions where there is a force as well as a moment transferred from the tower. The resulting soil pressure distribution for this case can be found by the principle of superposition: the soil distributions for the concentric loading and the pure bending will be added together to give the total soil pressure distribution. The pressure distribution due to pure bending is shown in Figure 2.7. The value of the soil pressure is found by dividing the moment value by the section modulus of the area of the foundation in contact with the soil.

\[ q = \frac{M}{S} \]

Figure 2.7: Soil pressure distribution due to pure bending (Brzev & Pao, 2006)

Depending on the magnitude of the moment (i.e. the eccentricity), three slightly different soil pressures are possible: those corresponding to small eccentricities, large eccentricities, or boundary eccentricity. Brzev and Pao (2006) define the boundary eccentricity as that equal to one-sixth of the foundation’s length for a square foundation and one-quarter of the radius for a circular foundation. Note that the magnitude of the eccentricity is not only reflected by the deviation of the axial load from the neutral axis. For example, a large transferred moment coupled with a concentric axial load results in conditions identical to those for large eccentric loading.

For small eccentricities (i.e. small transferred moment), the total distribution of the soil pressure is trapezoidal, as shown in Figure 2.8. The distributions for boundary and large eccentricities are linear, as shown in Figure 2.9 and Figure 2.10, respectively. The deviation of the line of action of the resulting soil pressure from the neutral axis of the foundation reflects the amount of
eccentricity. Larger eccentricities are detrimental to the foundation because they result in larger overturning moments and bearing pressure on the soil. When the total resulting soil pressure contains negative stresses indicating that the soil is exerting tensile stresses on the foundation, these stresses are taken as zero because in reality the soil does not exert such tensile stresses (Brzev & Pao, 2006); (Maunu, 2008). The linear soil pressure distributions discussed are adequate simplifications of the complex soil-structure interaction, which were suggested by Meyerhof (1953). Other possible simplifications include a uniform or a variable nonlinear pressure over an effective contact area (Meyerhof, 1953). The nonlinear pressure distribution is accurate for fine-grained soils where the edge maximum pressure is exceeded, so the maximum pressure tends to redistribute inwards (Yilmaz, Schubert, Tinjum, & Fratta, 2014).

Yilmaz et al. (2014) installed pressures gauges beneath two octagonal gravity wind turbine foundations to monitor the bearing pressure on the soil underneath. Results from different stages in the service life of the wind turbines have shown that the pressure was distributed across most of the foundation-soil contact area. Furthermore, the pressure was not constant, both vertically and horizontally. The changes in the pressure values were related to the wind speed and direction.

![Figure 2.8: Soil pressure distribution for small eccentricity (Brzev & Pao, 2006)](image-url)
2.2.3 Self-Weight of the Foundation

Unlike other foundation types, gravity spread foundation are more massive to ensure smoother transfer of forces to the soil and resist overturning. Hence, the self-weight of the foundation is more pronounced and needs to be considered in the design. If the foundation is a level slab, then the self-weight will be a uniformly distributed load. However, foundations such as the octagonal spread foundation are tapered, which results in a self-weight distribution that is higher in areas where there is more mass concentration. In some cases, the weight of the backfill soil is also taken into consideration in design. The self-weight of the foundation reduces the eccentricity of the loading and a portion of it, which is behind the zero moment line, resists the overturning moment.

2.2.4 Earthquake Loading

Earthquake loading needs to be considered in the ultimate limit state as a principal load. The National Building Code of Canada (NBCC 2010), in section 4.1.8, permits the use of several
methods in the calculation of the earthquake load. The simplest method is the Equivalent Static Load method, in which the earthquake effects are translated into a lateral force given as follows:

\[ V = \frac{S(T_a) \cdot M_v \cdot I_E \cdot W}{R_d \cdot R_0} \]

where: \( V \) is the lateral earthquake force

\( T_a \) is the period of the structure

\( S(T_a) \) is the spectral acceleration corresponding to the period of the structure

\( M_v \) is a factor accounting for higher modes

\( I_E \) is the importance factor

\( W \) is the weight of the structure

\( R_d \) is the ductility factor related to the seismic force resisting system (SFRS)

\( R_0 \) is the over strength factor related to the seismic force resisting system (SFRS)

Over the height of the wind turbine structure above the ground level, the majority of the mass is concentrated at the hub height; the contribution of the mass of the tower is neglected. Therefore, the entirety of the lateral force due to the earthquake is assumed to be acting at the hub location, which induces an overturning moment on the foundation.

Empirical equations are given in the NBCC (section 4.1.8.11) to calculate the period of the structure. Soil amplification factors (\( F_a \) and \( F_v \)) are obtained from the geotechnical report and used to modify the spectral acceleration of the structure. The importance factor (\( I_E \)) as well as the factor accounting for higher modes (\( M_v \)) are obtained from the code. Finally, \( R_d \) and \( R_0 \) are estimated depending on the inherent ductility of the structure. The ultimate goal of the earthquake-resistant design is to prevent the collapse of the structure due to the expected earthquake. Although the Equivalent Static Force method simplifies the design against earthquakes and gives conservative results, more accurate analysis is possible through the response spectrum analysis. The increased accuracy better simulates the real response of the structure and can possibly result in more material savings, but at the cost of increased analysis time and complexity. This method involves
establishing a three-dimensional model of the structure in a structural analysis software (such as SAP2000) and performing modal and dynamic analysis on the model, which will produce the demands on the individual members of the structure due to the earthquake. The choice of which analysis method to use is determined by the designer.

2.2.5 Wind Fatigue (Cyclic) Loading

The rotation of the wind turbine blades due to the wind causes cyclic fatigue loading on the foundation, making it prone to fatigue failure (Isojeh & Vecchio, 2016); (Maunu, 2008); (Svensson, 2010); (Goransson & Nordenmark, 2011). The magnitude and parameters of such loading differ with varying wind speeds, direction, and amount of turbulence. On one side of the tower where the blades rotate towards the foundation, compressive fatigue loading will be exerted. On the other side where the blades are moving away from the foundation, tensile fatigue forces will result. The soil resultant force along with the compressive fatigue loading will create a shear span in the distance between them, inducing a possible shear fatigue failure of the foundation, as shown in Figure 2.11. This type of failure is critical for gravity foundations, especially for some typical octagonal spread foundations that act as deep beams. Hence, the behaviour of reinforced concrete deep beams under fatigue loading, especially shear fatigue behaviour, needs to be examined. In addition, safeguarding such foundations against fatigue failure needs to be included in the design.

Figure 2.11: Fatigue forces acting on the wind turbine foundation and the induced fatigue failure
2.3 Fatigue Loading

2.3.1 Fatigue Load Parameters

The concept of fatigue initially arose in the fracture analysis of metals. According to ASTM, fatigue is “the process of progressive localized permanent structural change occurring in material subjected to conditions which produce fluctuating stresses and strains at some point or points and which may culminate in crack or complete fracture after a sufficient number of fluctuations” (ASTM International, 1993). The fatigue damage is exhibited when the material fails under cyclic stresses. Usually, the cyclic stresses that cause fatigue failure are less than the static ultimate limit states. Figure 2.12 shows the range of number of cycles for different structures experiencing fatigue loading. Wind turbine foundations are typically subject to high-cycle fatigue.

![Figure 2.12: Typical number of cycles for different structures experiencing fatigue loading (Goransson & Nordenmark, 2011)](image)

The parameters for applied fatigue stresses include the maximum and minimum stresses, mean stress, range of stress, amplitude of stress, and the stress ratio (Caceres, 2004). The loading pattern and the parameters are shown in Figure 2.13. The following equations apply:

Mean Stress: \[ \sigma_m = (\sigma_{\text{max}} + \sigma_{\text{min}}) / 2 \] (1)

Stress Range: \[ \sigma_r = \sigma_{\text{max}} - \sigma_{\text{min}} \] (2)

Stress Amplitude: \[ \sigma_a = \sigma_r / 2 = (\sigma_{\text{max}} - \sigma_{\text{min}}) / 2 \] (3)
Stress Ratio: \[ \frac{\sigma_{\text{min}}}{\sigma_{\text{max}}} \] (4)

Figure 2.13: Fatigue loading (Caceres, 2004)

2.3.2 General Fatigue Damage

The damage caused by fatigue loading initiates locally with crack nucleation. Stress concentrations at an internal flow cause shear flow along slip planes, which eventually results in the initial crack. (Roylance, 2001). The stress concentrations at the crack cause it to propagate further, reducing the effective area of material resisting the load, until final fracture happens. This process is general to many materials but the mechanics and extent of each of the stages depend on the particular material under investigation. The detailed macro- and microscopic study of this damage process is studied and analyzed thoroughly in the field of fracture mechanics.

2.3.3 S-N Curves

When designing against fatigue, a convenient macro-structural representation of the fatigue life of the material is used. This representation, based on the stress-life method, utilizes what is known as the S-N curves (or the Wöhler Diagram), in which S refers to the cyclic stress amplitude while N refers to the number of cycles until fatigue failure. A sample S-N curve is shown in Figure 2.14. Note that each S-N curve is given for a constant minimum stress value. Another way the S-N is presented is in denoting S to be the maximum stress value. In this case, each S-N curve is given for a constant stress amplitude.
For a given number of cycles, the fatigue strength of the material is the stress amplitude at which the material fails in fatigue. On the other hand, the fatigue life is the number of cycles required for a material to fail in fatigue for a given stress amplitude. For some materials, especially some ferrous alloys, there is a fatigue limit (stress amplitude limit) below which the material does not fail no matter how many cycles of loading are applied (Roylance, 2001). The S-N curves of different materials are convenient in determining their approximate fatigue lives, since most fatigue tests use constant stress amplitudes. There are various standard S-N curves for different materials used by organizations and firms in the design against fatigue.

### 2.3.4 Miner’s Rule

The actual cyclic service loads and stresses on a structure have different and random stress amplitudes. This complicates the approximation of the fatigue life of the structure because there is no single S value to use. In this case, Miner’s Rule is used to approximate the fatigue strength and life of the material. It states that the total fatigue damage of a material under variable stress amplitudes can be obtained by linearly adding the damages caused by the different stress amplitudes acting on the material (Roylance, 2001). It relies on the concept of successive accumulation of fatigue damage. The fatigue loading on the material consists of m constant stress amplitudes. The material will fail when:
\[ \sum_{i=1}^{m} \frac{n_i}{N_i} = 1 \]

where \( n_i \) is the number of cycles applied of the \( i^{th} \) stress and \( N_i \) is the ultimate number of cycles to failure corresponding to the \( i^{th} \) stress (Guo, 2014). For every constant stress applied, \( N_i \) can be obtained from the S-N diagram of the material. A graphical representation of Miner’s Rule is shown in Figure 2.15. Although the conclusions given on its accuracy are not consistent, it is still used in many design codes. However, knowledge of its limitations and simplifications is essential when using it in design. For one thing, it neglects the stress redistribution and the irreversibility of fatigue damage and does not consider the effects of the loading sequence and strain accumulation.

Miner’s Rule assumes that structural failure occurs when the damage \( D \) (the value of the right hand side of Equation 5) is equal to 1, although many experiments have shown that the damage can vary between 0.79 and 1.53 (Sutherland, 1999). The service of a structure under fatigue loading can be approximated using Miner’s Rule by measuring the fatigue load cycles on the structure over some fixed period of time. The load cycles experienced during this period of time are assumed to be representative of the load cycles the structure will experience during its service life. As such, a damage rate, \( \Delta D \), is calculated for the fixed period of time by using Miner’s rule. This rate is nothing more than the damage incurred during this period of time. Then, the service life of the structure is calculated as the reciprocal of the calculated damage rate (\( T = 1/\Delta D \)) (Guo, 2014). This method assumes that the failure of the structure will happen when the damage is equal to one.

![Graphical representation of Miner’s Rule](Guo, 2014)

2.3.5 Modified Goodman Diagram

Although the S-N curves are convenient ways of representing the fatigue behaviour of members, they are only valid for the constant stress ratio or stress amplitude for which they were developed.
If the stress ratio corresponding to the fatigue loading on a certain member is changed, then a new S-N curve for the member corresponding to the new stress ratio should be constructed. The modified Goodman diagram shows the permitted stress ranges for a given number of load cycles before fatigue failure occurs (Mallet, 1991). In other words, the allowable maximum and minimum stress combinations for a given fatigue life are given. A sample modified Goodman diagram is shown in Figure 2.16. The maximum and minimum stress are shown as ratios of the ultimate static strength. The x-axis shows the minimum stress while the y-axis shows the maximum stress. The shaded region denotes the allowable stress combinations. The height of this at any specific minimum stress value represents the allowable stress range for the given fatigue life. If the loading falls within the shaded region, then the member will fail after N cycles, which is constant for every graph. If the loading falls above the shaded region, then the member will fail before reaching N load cycles. On the other hand, if the loading falls below the shaded region, then the member can sustain more than N load cycles.

Figure 2.16: Modified Goodman diagram (Mallet, 1991)
2.3.6 Fracture Mechanics

Fracture mechanics thoroughly addresses the initiation and propagation of cracks and the conditions and stresses surrounding them. There are two approaches to this field: the classical linear fracture mechanics, and the modern nonlinear fracture mechanics.

2.3.6.1 Linear Fracture Mechanics

Linear fracture mechanics assumes that the material is fully elastic and there is negligible or no plastification or nonlinearities in the vicinity of the crack (Afseth, 1993). The crack propagation is related to parameters such as the stress at the crack, the shape of the crack, and its size. Materials contain defects and irregularities that act as stress concentrations which facilitate the crack propagation.

Griffith (1920) proposed a criterion that relates the propagation of the cracks initiated, by the internal material defects and irregularities or applied forces, to the energy state of the material at the crack. Following the lowest energy path, the crack will propagate if the strain energy released during crack growth is equal to or greater than the energy required to expand the crack (Griffith, 1920):

\[ \delta W + \delta U \geq G_c \delta A \] (6)

where \( \delta W \) is the energy released when the applied load does work on propagating the crack, \( \delta U \) is the elastic (strain) energy released during crack growth, and \( G_c \delta A \) is the energy required to expand the crack, in which \( G_c \) is the surface energy per unit area of crack.

The Griffith criterion can be more conveniently and accurately expressed by the use of intensity factors. If the applied stress intensity factor (denoted as \( K \)) is equal to or greater than the critical intensity factor (denoted as \( K_c \)), then the crack will propagate. The critical intensity factor (also known as the fracture toughness) depends on parameters including the modulus of elasticity, crack surface energy, crack geometry, specimen geometry, mode of loading, and nature of deformations ahead of the crack (Ashby & Jones, 2011).

The propagation of a crack (after its initiation) does not necessarily occur linearly with the progression of the cyclic loading. The important pioneering work by Paris et al. (1963) established the relationship between the crack growth and the number of load cycles. It suggested that the rate
of crack growth with respect to the load cycle number is proportional to the $n^{th}$ power of the stress intensity factor range, in which $n$ is a material-specific constant:

$$\frac{da}{dN} = C \cdot \Delta K^n$$

in which $a$ is the crack size, $N$ is the number of cycles, $\Delta K$ is the stress intensity factor range, and $C$ and $n$ are material constants. The value of $\Delta K$ is given as:

$$\Delta K = Y * \Delta \sigma * \sqrt{\pi a}$$

in which $Y$ is the shape factor and $\Delta \sigma$ is the fatigue stress range. Integration can be performed on Eq. (7) to obtain the crack size (depth) as a function of the number of cycles (Paris & Erdogan, 1963).

### 2.3.6.2 Nonlinear Fracture Mechanics

Many materials contain nonlinear and plastic regions around the tip of a crack. In this case, using linear fracture mechanics is not sufficiently accurate; nonlinear fracture mechanics models were proposed to model and approximate such cases. While there are many nonlinear models that do not apply to concrete (such as J-integral path model and the crack opening displacement model), two nonlinear fracture mechanics models were developed specifically for concrete: crack band theory model and the fictitious crack model. These two models are briefly discussed in this literature review.

### 2.4 FATIGUE OF REINFORCED CONCRETE

Reinforced concrete structures are generally subject to two types of fatigue loading: low-cycle fatigue and high-cycle fatigue. The former refers to the fatigue loading which has high stress amplitudes that are usually enough to cause fatigue damage in a relatively lower number of cycles (e.g. earthquakes), while the latter contains low stress amplitudes but an extended number of cycles (e.g. service loads). Examining the fatigue of reinforced concrete requires looking at not only the fatigue behaviour of concrete and reinforcement as constituent parts, but also the complex interactions amongst them that affect the fatigue response of the member; the fatigue strength of reinforced concrete is not just simply the addition of its constituents’ fatigue strengths (Heffernan, 1997).
2.4.1 Fatigue of Plain Concrete

2.4.1.1 Mechanism of Fatigue in Plain Concrete

The fatigue failure of concrete has the same stages as that of other brittle materials. The first fatigue cracks initiate from stress concentrations and initial flaws in the specimen when the load is applied. Sometimes the initial cracks are already present due to the shrinkage of the cement paste before the application of the load. Then as the load cycles progress, these cracks that break the bond between the cement matrix and the aggregates propagate further until fracture happens. The propagation path of the cracks is highly variable, because the cracks need to find their way around the aggregates instead of cutting into them as this is the lowest energy path. As such, the aggregates act as crack arrestors in the sense that the energy required to further propagate the crack intersecting the aggregate increases (Afseth, 1993).

The stress-strain curve of a concrete prism undergoing cyclic compression stresses is shown in Figure 2.17. Note that $f_c^f$ refers to the fatigue limit of the specimen, which is the maximum stress value at which the specimen will not experience fatigue failure regardless of the number of cycles applied. Plastic deformations occur even when the applied stress is less than the fatigue limit (Guo, 2014). These deformations, however, tend to stabilize after certain number of cycles and the internal damage does not propagate further. Hence, subsequent cycles do not cause fatigue failure in the specimen. The areas of the hysteretic loops corresponding to such stress levels are small.

When the applied stress exceeds the fatigue strength, only $N$ number of cycles (obtained from the S-N diagram corresponding to the applied stress) can be applied before failure. Initially when the load cycles are less than 90% of the ultimate number of cycles to failure, the residual strains increase gradually while the areas of the hysteretic loops slightly decrease (Guo, 2014). In this stage, the cracks develop in a stable manner because the interactions between the micro-cracks and the aggregates tend to stabilize (Afseth, 1993). After many load cycles (e.g., more than $10^4$ cycles) as the damage brought by the cracks increases, strains become unstable and divergence occurs, culminating in the sudden failure of the structure (Guo, 2014). Note that the areas under the loops represent the energy dissipated during the loading process. This energy represents the irreversible energy of deformation, and it is the energy that is released when cracks propagate (Afseth, 1993). The envelope of the cyclic loading curve is very similar to the stress-strain curve corresponding to the monotonic loading (Collins and Mitchell, 1997). This is generally true for all cyclic loading.
curves of concrete. The microstructure and energy mechanics of the fatigue damage of concrete is addressed appropriately and thoroughly through fracture mechanics. A general fatigue failure envelope for concrete, given by Ekberg J. et al. (1957), is shown in Figure 2.18. The shaded area represents the region of allowable stress ranges without fatigue failure.

Figure 2.17: Stress-strain curve of a concrete prism under repeated compression (Guo, 2014)

Figure 2.18: Fatigue failure envelope of concrete (Ekberg, Walther, & Slutter, 1957)
2.4.1.2 Factors Affecting the Fatigue Strength of Plain Concrete

2.4.1.2.1 Level of Loading

Similar to many other materials, the fatigue strength of concrete decreases with increased maximum fatigue stress. In other words, the degradation in concrete strength due to applied fatigue loading is increased when the loading level is increased. When the level of fatigue loading applied is high compared to the static strength of concrete, it will take less cycles for the concrete to exhibit fatigue failure, hence the fatigue life is decreased (Guo, 2014). Such behavior is reflected in the downward slope of the S-N curve shown in Figure 2.14.

2.4.1.2.2 Moisture Content, Rate of Loading, and Age

Raithby and Galloway (1973) investigated the effects of moisture content, age, and rate of loading on the fatigue strength of plain concrete by performing constant amplitude sinusoidal fatigue tests on simply supported plain concrete beams. The moisture content had a predominant effect on fatigue strength: the oven-dried specimens exhibited the highest fatigue strength while the specimens that had partially dried in the laboratory before the test gave the lowest fatigue strength. The fully saturated specimens had intermediate fatigue strengths. While there is no direct explanation for the effect of the total moisture content of the specimen, the difference in fatigue strengths is most likely explained by the differential strains caused by the moisture gradients of the beams (Raithby & Galloway, 1973). The shrinkage strains caused by drying might have increased the internal friction, which made it more difficult for the cracks to propagate, causing an increase in the fatigue strength.

The effect of the rate of fatigue loading was investigated for two frequencies: 4 Hz and 20 Hz. Although the specimens loaded at a frequency of 20 Hz showed a slight increase in fatigue strength (within 5% increase), this increase was not statistically significant (Raithby & Galloway, 1973). This shows that the rates of loading used in fatigue tests do not have a significant effect on fatigue strengths, confirming the results of previous research (Kesler, 1953). The rate of fatigue loading has negligible effects when it is in the range of 1.2 to 7.3 Hz (Kesler, 1953). However, if the rate of fatigue loading is low enough to cause creep in the specimen (e.g. 0.1 Hz), then the fatigue strength is negatively affected, especially if the maximum applied stress is more than 75% of the static strength (Guo, 2014) (Naik & Singh, 1993).
The test specimens of Raithby and Galloway (1973) had various curing times up to 3 years. The results of the tests showed a substantial increase in fatigue strength and endurance with age of the specimen. The mean life to failure of the 2-year old specimens was 2000 times the life of the 4-week old specimens. The rate of increase as a result of age at higher stress levels seemed to be less. When a specimen has more time to cure, more hydration occurs and, hence, the strength increases. The porosity and defects, where unfavourable stress concentrations occur, of the specimen decrease when further curing is allowed to happen. This results in increased fatigue strength and endurance.

2.4.1.2.3 Stress Range

Murdock and Kesler (1958) investigated the effect of the stress range on the fatigue strength of plain concrete beams by performing three-point bending fatigue tests on 6 in. x 6 in. x 64 in. beams. The main variable was the stress ratio (R), which is the ratio of the minimum applied stress to the maximum applied stress. This ratio varied from 0.13 to 0.75 in the tests. The results showed that the fatigue strength increased from 56 percent of the modulus of rupture at R = 0.13 to 85 percent of the modulus of rupture at R = 0.75 (Murdock & Kesler, 1958).

2.4.1.2.4 History of Loading

The sequence and history of loading has an effect on the fatigue behaviour of plain concrete. While Miner’s Rule is used to approximate the damage caused by different amplitude loading, it ignores the sequence of loading: whether the higher amplitude stresses were applied in the beginning or the end is irrelevant. Hilsdorf and Kesler (1966) investigated the applicability of Miner’s Rule in flexural fatigue loading by performing two-stage constant amplitude loading. When the higher stress level was applied first, Miner’s Rule was conservative. However, when the lower stress level was applied first, Miner’s Rule was not safe (Hilsdorf & Kesler, 1966). This shows that the sequence of loading has a significant effect on the damage due to fatigue.

2.4.1.2.5 Stress Reversal

Zhang and Phillips (1989) performed fatigue tests on seventy 500 mm x 100 mm x 100 mm beams to investigate the effect of stress reversal, using different loading frequencies and stress ratios. The results indicate that stress reversal decreases the fatigue life of plain concrete, but this decrease is smaller when the stress ratio is positive (Zhang & Phillips, 1989). Other researchers concluded
that stress reversals have an insignificant effect on the fatigue strength (Murdock & Kesler, 1958); (Tepfers, 1982).

2.4.1.2.6 Rest Periods

Hilsdorf and Kesler (1966) subjected their specimens to rest periods between 1 and 27 minutes in which the specimens were under low sustained stresses. Results showed that rest periods equal to and less than 5 minutes increased the fatigue strength, while larger rest periods did not have further effects.

2.4.1.2.7 Aggregate Type

Klaiber et al. (1979) investigated the effect of the type of aggregate on the fatigue strength of plain concrete. The coarse aggregates considered were gravel and crushed limestone. The results showed that the type of coarse aggregate had a considerable effect on the fatigue strength of the specimens at high stress levels; the concrete made with gravel exhibited a higher fatigue strength than that made with crushed limestone. The effect of fine aggregates was less pronounced; higher quality fine aggregates resulted in only a slight increase in the fatigue strength (Klaiber, 1979).

2.4.1.3 Fracture Mechanics of Plain Concrete

When classical linear fracture mechanics was applied to concrete, poor results that deviated from measurements were reported (Bazant, 1985). This created the need to change the approach to nonlinear fracture mechanics suited to the behaviour of concrete. There are two major fracture mechanics models that were proposed for the behaviour of plain concrete: the crack band theory model and the fictitious crack model. Despite the development of these models, the applicability of fracture mechanics in predicting the response and crack pattern of concrete specimens (e.g. beams) is limited. This limitation arises from the nature of crack development in concrete specimens; instead of one crack developing, a multitude of cracks initiate and develop as a result of applied loading. Instead of fracture mechanics, damage models specific to concrete can be used to predict its fatigue behaviour (Isojeh & Vecchio, 2016).

2.4.1.3.1 Crack Band Theory Model

Bazant and Oh (1983) proposed the crack band theory to model the fracture of concrete. Concrete is modelled as a heterogeneous substance consisting of the aggregates and the hardened paste. Such a heterogeneous material is approximated by a homogeneous continuum of stresses and
strains (macro-stresses and macro-strains). These strains and stresses are defined as the averages of the micro-stresses and micro-strains over a representative volume, which must be large compared to the size of inhomogeneities (several times bigger than the maximum aggregate size) (Bazant, 1985). The fracture is modelled as a blunt smeared crack band due to the random nature of the microstructure (Bazant and Oh, 1983). This band is found at the fracture process zone, which is the strain-softened area at the tip of the crack (shown in Figure 2.19). This zone exhibits nonlinear behaviour due to the strain-softening happening due to the presence of the micro-cracks. Three parameters are needed for this model: the fracture energy, the uniaxial strength limit, and the width of the crack band.

![Image of Figure 2.19: Conditions around a crack in concrete (L = linear zone, N = nonlinear zone, F= fracture zone) (Bazant 1985)](image)

2.4.1.3.2 Fictitious Crack Model

The fictitious crack model, proposed by Hillerborg (1983), describes the fracture process zone at the tip of the crack. This zone consists of many discontinuous micro-cracks that are still capable of transferring stresses, hence forming the “fictitious crack” (Afseth, 1993). The micro-cracks form at the fracture process zone when the tensile strength of the material is exceeded. As more stress is applied on the region, the ability of the micro-cracks to transfer stresses decreases. In addition, the width of the fictitious crack increases. When the width reaches a critical value, the micro-cracks are no longer able to transfer stresses and a real crack is created. Prior to the formation of the fictitious crack, the normal stress-strain curve is used to describe the behaviour (Afseth, 1993). Figure 2.20 shows the stages of the development of the crack and the associated stresses according to this model.
2.4.2 Fatigue of Reinforcement

2.4.2.1 Mechanism of Fatigue of Reinforcement

The fatigue behaviour of the reinforcement follows that of metals. When the reinforcement is rolled, cold-worked, and heat treated, defects (e.g. fine cracks and impurities) result on the surface and the interior of the reinforcement (Guo, 2014). Such defects and the roots of the reinforcement ribs cause unfavourable stress concentrations. When the load is applied, the resulting stresses on these areas are magnified. When the stresses exceed the strength of the reinforcement on any plane, slipping of the steel crystal occurs, resulting in the formation of the initial cracks in the reinforcement (Guo, 2014). As the stresses applied on the reinforcement increase, the initial cracks expand and further slippage of crystal and movement of dislocations occur. The effective area of the uncracked reinforcement decreases, which increases the stress concentrations and facilitates further propagation of cracks, until the propagation becomes unstable and sudden fatigue failure of the reinforcement specimen occurs (Guo, 2014). The three stages of crack initiation,
propagation, and sudden failure are shown in Figure 2.21. When cracks propagate, friction causes a smooth and dark finish on the cross section of the reinforcement. On the other hand, the area of fracture in the cross section has a rough and granular appearance, as shown in Figure 2.22.

![Microscopic surface defect](image1)

**Figure 2.21:** Evolution of fatigue in the cross section of the reinforcement (Rocha & Bruhwiler, 2012)

![Rough and granular](image2)

**Figure 2.22:** Cross section of fractured reinforcement (Guo, 2014)

There are generally two fatigue tests that can be performed on the reinforcement. The first test is the bare bar axial fatigue test, in which the bare reinforcement bar is gripped from both of its ends and subjected to fatigue loading. This test is relatively easy to perform and enables the application of high load frequencies. However, the lack of perfect alignment of the testing machine can induce secondary and local stresses on the reinforcement. In addition, testing bare bars does not simulate the real operating conditions where the bars are bonded to the concrete (Tilly, 1979).

The second test that can be performed is the reinforced concrete beam fatigue bending test, in which a single bar is placed inside the beam (Tilly, 1979). The main advantage of this test is that it simulates the service conditions and accounts for the effects of the concrete and the bond. However, appropriate consideration of the concrete contribution to the strength is necessary to
accurately obtain the fatigue strength of the bar. In this test, only limited loading frequencies (e.g. 5 Hz) are possible, depending on the limitations of the testing equipment (Tilly, 1979).

2.4.2.2 Factors Affecting the Fatigue Strength of the Reinforcement

2.4.2.2.1 Magnitude of Fatigue Loading

Similar to plain concrete and most metals, the fatigue strength of the reinforcement, as well as its fatigue life, decreases with the increased level of fatigue loading (e.g. increased maximum applied stress): The degradation in the strength of the reinforcement and the rate of crack growth increase when the level of the applied loading is high compared to the static strength of the reinforcement. This is reflected in the downward slope of the S-N curve of the reinforcement.

2.4.2.2.2 Yield Strength

Tilly (1979) reviewed the factors affecting the fatigue behaviour of the reinforcement. It seems that increasing the yield strength of the bar more than the typically used value (420 MPa) has little influence on its fatigue strength; increasing the yield strength of a bar from 420 MPa to, for example, 700 MPa has negligible influence on its fatigue strength (Tilly, 1979). Pfister and Hognestad (1964) performed fatigue tests on 181 different reinforcement bars embedded in concrete beams and concluded that the yield strength of the bars, test beam cross section, and minimum stress levels do not have a significant influence on the fatigue strength of the bars up to 2 million cycles.

2.4.2.2.3 Bar Geometry

The geometry of the bar has a significant influence on its fatigue strength (Tilly, 1979); (Pfister & Hognestad, 1964). In their experiments, Pfister and Hognestad (1964) concluded that bent bars have considerably less fatigue strength than straight bars; the fatigue strength of the bars bent to 45 degrees was only half of that of straight longitudinal bars. The bend introduces unfavourable stress concentrations that facilitate the initiation of the fatigue cracks. This is also the case with the presence of ribs and manufacturer marks on the bars. Tilly (1979) compared the fatigue strengths of four different bar geometries; one of the bars was a plain bar while the other three bars had different rib and twist configurations. He concluded that while the plain bar had inferior pull-out and bond strengths, its fatigue strength was more than the other ribbed bars. When the ribs are placed in the vertical plane of the bar, the fatigue strength is up to 40% lower than when placed in
the horizontal plane (Burton & Hognestad, 1967). The presence of transverse lugs also decreases the fatigue strength. Pfister and Hognestad (1964) concluded that all fatigue cracks initiated at the roots of the lugs in their specimens. One of the most important factors influencing the stress concentrations is the ratio of the lug base radius to the lug height (shown in Figure 2.23). In addition, a sharper lug base radius is much more detrimental to the fatigue strength of the bar than a larger lug base radius (Afseth, 1993).

Reinforcement bars usually have the manufacturer marks in the form of raised features, which are the source of detrimental stress concentrations. When these marks were taken off, the fatigue strengths of the bars were increased by 100% (Tilly, 1979).

![Diagram of lug base radius (r) and lug height (h) in a reinforcing bar (Afseth, 1993)](image)

**Figure 2.23**: Lug base radius (r) and lug height (h) in a reinforcing bar (Afseth, 1993)

### 2.4.2.2.4 Manufacturing Practices and Methods

Matsumoto (1988) investigated the effects of cold work on the fatigue strength of reinforcing bars by performing fatigue tests on straight 9.53 mm diameter bars. The tests consisted of two phases: one to obtain the S-N curve for the normal bars, and the other to obtain the same curve for cold-worked bars. The results showed that for a given number of cycles, the fatigue strength of cold-worked bars is about 15 percent less than that of normal bars (Matsumoto, 1988).

Thandavamoorthy (1999) conducted fatigue bending tests on eleven 300 mm x 150 mm x 3000 mm concrete beams reinforced with high-ductility quenched and tempered bars to determine the fatigue strength and endurance of these bars. The common design criteria of 2,000,000 cycles was used to determine if the bars can safely endure such number of cycles. The frequency of loading used was 5 Hz for the most part and the stress ranges used were 0.1fy to 0.39fy and 0.2fy to 0.44fy, where fy refers the static yield strength of the bars. The results showed that all the quenched and tempered bars were able to safely withstand 2,000,000 cycles, with endurances very similar and
comparable to the normal mild bars; quenching and tempering the bars did not have a significant effect on the fatigue performance of the bars (Thandavamoorthy, 1999).

Tilly (1979) compared the results of different fatigue tests performed on reinforcement bars produced by rolls in the steel mills that had different degrees of wear. While the tests performed contained high variability of results, they all indicated that the reinforcement bars produced with unworn (new) rolls had lower fatigue strengths than those produced by worn rolls. Unworn rolls produce more defined transitions in the deformations of the reinforcement surface and hence the stress concentration are more pronounced.

2.4.2.2.5 Bar Diameter

Reinforcement bars with bigger diameters usually have lower fatigue strengths (Tilly, 1979). When the diameter is larger, there are certainly more defects and areas of stress concentration present in the bar, which make the process of crack initiation easier. While the effect of the size of the bar diameter on the fatigue strength of plain bars is relatively small, it is much more pronounced for ribbed bars (Tilly, 1979). For example, the fatigue strength of 40 mm diameter bars can be as much as 30% lower than that of 16 mm bars (Tilly, 1979). The bar diameter size effect is also more pronounced in axial tests compared to bending tests.

2.4.2.2.6 Corrosion

Corrosion is an electrochemical process by which expansive rust products are formed in the reinforcement. These products generally reduce the load-carrying capacity of the reinforcement. Li et al. (2011) performed axial tensile fatigue tests on fifteen carbonation-induced naturally corroded steel bars. The length of each specimen was 400 mm. The loading pattern was a constant amplitude sine wave ranging in frequency from 5 Hz to 10 Hz. The results showed that the brittle fracture surfaces of the specimens did not experience necking and contained unique even fracture characteristics. The fatigue life reduces greatly with the increase of the mass loss due to corrosion under the same applied stresses. With a mass loss of 31.5%, the fatigue life decreased by about 70% from 2704000 cycles (corresponding to 0% mass loss) to only 856000 cycles. The uneven corrosion products in the reinforcement cause big stress concentrations (Li et al., 2011).
2.4.2.3 Fracture Mechanics of Reinforcement

When the applied stress on the reinforcement is amplified locally by the presence of notches or defects in the microstructure of the reinforcement, local yielding which causes crack initiation occurs. The flow of stresses and dislocations cause crack propagation, which eventually leads to fatigue failure. At the tip of a crack, both elastic and plastic portions of the reinforcement are present. However, the portion of plastic material present is small, which deems the use of classical linear fracture mechanics for the reinforcement appropriate (Afseth, 1993). The Paris crack growth law can be used to estimate the extent of the fatigue crack in the reinforcement at a given load cycle. The material constants C and n in Eq. (7) are taken as $2 \times 10^{-13}$ and 3 for steel, respectively (Isojeh & Vecchio, 2016).

2.4.3 Fatigue of Concrete-Reinforcement Bond

The concrete-reinforcement bond is responsible for the transfer of the stresses from the concrete to the reinforcement. The effectiveness of reinforced concrete as a composite material system is governed by the strength and quality of the bond. When fatigue loading is applied on a reinforced concrete beam, the reinforcement experiences intermittent tensile stresses. The bond at this stage also experiences stresses, which causes a gradual bond degradation in terms of reduced strength and stiffness (Guo, 2014).

When there is considerable bond degradation, stress is no longer transferred from the concrete to the reinforcement; slippage in the reinforcement occurs. In this case, the concrete takes almost all the stress, which causes the formation of wide cracks in the concrete (Guo, 2014). In addition, the widening of the existing cracks is no longer sufficiently controlled, leading to sudden failure of the specimen.

The tensile fatigue bond test is usually performed on the tension specimen shown in Figure 2.24. In this configuration, the bar is embedded in the concrete only in the central portion of the specimen. Uniaxial tensile fatigue stresses are applied to assess the strength and response of the bond. A typical S-N diagram for the ultimate bond strength is shown in Figure 2.25. As can be seen, the fatigue bond strength decreases when the number of cycles applied increases. The strength of the concrete and the diameter of the reinforcement do not appear to have an obvious influence on the bond fatigue strength (Guo, 2014). The main factor influencing the fatigue...
The concrete-reinforcement bond experiences slip as the fatigue loading progresses. This slip is never recoverable and accumulates from each load cycle (Guo, 2014). In the beginning load cycles, the slip value is larger than those of the subsequent load cycles, resulting in more energy dissipation. However, after some load cycles, it tends to stabilize until the average bond stress becomes larger than the fatigue bond stress, at which point the reinforcement either pulls out or the concrete surrounding the reinforcement splits. This behaviour is shown in Figure 2.26. The lower energy dissipation during the latter cycles is explained by the local damages that happen at the interface. The damage of the concrete around a reinforcement bar containing transverse lugs, showing bond degradation, is shown in Figure 2.27.
Abbass et al. (2012) performed fatigue bending tests on three 1165 mm x 150 mm x 225 mm reinforced concrete beams to investigate the bond strength and behaviour. The fatigue loading applied was a low cycle loading, ranging from a total of 1 cycle to 100 cycles. All three beams were tested to failure. Results showed that the fatigue bond strength is only about 55% of the static bond strength. The use of high-strength concrete resulted in improved fatigue bond strengths.
2.4.4 Fatigue of Reinforced Concrete Beams

The fatigue behaviour of reinforced concrete beams depends on, in addition to the properties of the beams, the fatigue loading configuration. The fatigue failure can either be due to bending moments or due to shear forces. The mechanism and propagation of damage is somewhat different for each case.

The fatigue of reinforced concrete (and concrete in general) starts with crack initiation. Reinforced concrete structures are usually cracked under service loading, causing a reduction in their fatigue strengths (Guo, 2014). Repeated cycles from the service loads cause a propagation of the already initiated cracks, eventually leading to the fatigue failure of the structure. However, if the structure (or member) is not cracked under the repeated service loads, then repetitive cycles of such loads will not cause its fatigue failure regardless of the number of cycles (Guo, 2014); the magnitudes of the applied loads are too small to initiate cracks in the member, hence the process of fatigue will not be started. Generally, reinforced concrete beams are cracked when the design load is first applied; the fatigue loading will utilize the existing cracks until the fatigue failure of the beam occurs.

2.4.4.1 Flexural Fatigue Behaviour of Reinforced Concrete Beams

The cracking moment caused by fatigue loading is typically lower than the cracking moment caused by monotonic loading. Generally, for an average structural member experiencing 2 million cycles or more, the fatigue cracking moment is about 50% of the monotonic cracking moment (Guo, 2014). Fatigue loading causes a degradation in the tensile strength of the concrete, hence the process of crack initiation becomes easier compared to the monotonic case. When the applied fatigue stresses become equal to the tensile strength of the concrete beam in a certain plane, then cracking initiates perpendicular to the resultant of the applied stresses.

When fatigue loading is applied on a reinforced concrete beam that is already cracked by the first application of the service or design loads, the existing cracks will gradually widen in a stable manner (Guo, 2014). New cracks will also gradually form in between the existing cracks. If the fatigue loading applied has constant amplitude and maximum stress value, then the beam will experience fatigue failure when it reaches its fatigue life; that is, after the application of N load cycles for the particular load levels. After each load cycle, the stresses (and strains) in the tensile
reinforcement and compression fibres of concrete will increase, in addition to the deflection of the beam. The bond between the reinforcement and the concrete will also degrade accumulatively; there will be residual slip after each load cycle (Guo, 2014). If the average stress in the bond becomes larger than the fatigue bond strength, then there will be major slip leading to premature failure of the structure, shown by the development of sudden wide cracks in the concrete and/or the pull-out of the reinforcement. However, if adequate anchorage and interface friction resistance are provided, then bond slip will be minor, if any, and will not cause the failure of the beam.

Flexural fatigue failure of a beam is demonstrated by the fracture of the tensile reinforcement or the compressive fatigue crushing of the concrete. In most cases, the fracture of the reinforcement governs because most beams are designed as under-reinforced (Guo, 2014). As the beam is exposed to more cycles of load, the damage accumulates and cracks propagate further. Eventually, one of the tensile reinforcement bars will experience fracture in the most stressed location along its length, which is generally where it intersects a major concrete crack. After cracking, the remaining stress will be redistributed to the remaining reinforcing bars, which will imminently fracture as well, and the concrete neutral axis will be moved closer to the extreme compression fiber, reducing the effective area of concrete resisting compressive stresses. Sometimes the stress in the reinforcement is not high enough to cause its fracture, so concrete in the compression zone crushes first. The flexural fatigue failure of a reinforced concrete beam is denoted by either the fracture of the first reinforcing bar or the crushing of the extreme compression fiber concrete.

Al-Rousan and Issa (2011) performed two-point bending fatigue tests on 9 reinforced concrete beams externally bonded with CFRP sheets using four different stress ranges (0.25-0.35fy, 0.45-0.65fy, 0.65-0.9fy, and 0.45-0.9fy, where fy is the yield stress of the tensile reinforcement). The loading frequency was 4 Hz for most of the beams. Each beam contained 3 #4 (diameter = 12.7 mm) tensile bars. Results show that all beams except one withstood 2 million load cycles. The first flexural crack initiated during the first static load, and more cracks developed and propagated as the load cycles progressed. Eight beams failed by yielding of the reinforcement first, then partial rupturing of the CFRP sheets, and finally crushing of the concrete in the compression zone. Only one beam failed through rupturing of the tensile reinforcement. The presence of the CFRP sheets at the extreme tension fiber prevented the rupturing of the tensile reinforcement in most of the beams. Figure 2.28 shows the fatigue failure of one of the beams. The crushing of the concrete can clearly be seen at the mid-span of the beam.
2.4.4.2 Shear Fatigue Behaviour of Regular (Slender) Reinforced Concrete Beams

2.4.4.2.1 Behaviour of Beams without Shear Reinforcement

Shear stresses and forces exist in the beam where the bending moment changes its magnitude. These stresses are always coplanar to the cross section of the beam. Their distribution in an uncracked elastic rectangular cross section of a beam is shown in Figure 2.29. The shear span is the length of the beam between the applied force and the reaction force. It is in this area that shear failure is exhibited since shear stresses predominantly exist.

![Image](image.png)

(a) Flexural and shear stresses acting on elements in the shear span.

(b) Distribution of shear stresses.

Figure 2.29: Shear stress distribution in an uncracked rectangular beam (Wight & MacGregor, 2012)

Every concrete element is subject to both normal and shear stresses. In a specific orientation, the element experiences the largest and smallest possible normal stresses without any shear stresses. The normal stresses experienced in this orientation are called the principal stresses. The principal compressive stress trajectories in a shear span of a rectangular cross section beam is shown in Figure 2.30. These trajectories help predict the expected shear cracking pattern in the beam; concrete cracks when the principal tensile stress in any orientation exceeds the tensile strength of
concrete. The development of the cracks are then perpendicular to the tensile stresses, following the principal compressive stress trajectories. However, this relationship is not very accurate since the majority of shear cracks are preceded by vertical flexural cracks initiating at the bottom of the beam, which cause a drop in the beam’s tensile strength and a major redistribution of stresses (Wight & MacGregor, 2012).

![Figure 2.30: Principal compressive stress trajectories in an uncracked beam (Wight & MacGregor, 2012)](image)

Shear stresses cause diagonal tension cracks in beams. Most of these cracks originate as vertical flexure cracks but extend diagonally as shear cracks. These shear cracks exist in shear spans close to the ends of the beam (Wight & MacGregor, 2012). The equilibrium of a section of the beam between two shear cracks, along with its average shear stress distribution, is shown in Figure 2.31. Note that C refers to the compressive force produced by the concrete while T refers to the tension in the bars. The tensile force produced by the bars must be transferred to the adjacent sections by horizontal shear, which is equivalent to the vertical shear. About 30% of the vertical shear is transferred through the uncracked compression region while the rest is transferred across the cracks through aggregate interlock and the dowel action of the flexural reinforcement (Wight & MacGregor, 2012). The average value of this shear stress can be expressed as:

$$v = \frac{V}{b_w jd}$$  \hspace{1cm} (9)

where $b_w$ is the thickness of the web and $jd$ is the lever arm. Using the relationship between the bending moment and the shear force, this equation can be rewritten as:

$$V = \frac{dT}{dx} jd + \frac{d(jd)}{dx} T$$  \hspace{1cm} (10)

where $dT/dx$ is the shear flow across any horizontal plane between the reinforcement and the compression zone (Wight & MacGregor, 2012). When the shear flow is not disrupted by bond slip and/or diagonal tension cracks in the shear spans joining the load and the support, the shear is
transferred through beam action. Otherwise, arch action dominates, in which concrete forms compression struts while the longitudinal reinforcement forms the tension ties. Note that arch action is responsible for shear transfer in deep beams. In many beams, the shear transfer mechanism is a contribution of both the beam action and the arch action.

Beam action dominates as the method of shear transfer in normal “slender” beams. The free-body diagram of a section of a beam at an inclined crack due to the combined action of shear and flexure is shown in Figure 2.32. The applied moment is resisted by the couple generated by the tensile force in the reinforcement (T) and the compressive force in the concrete (C). Before the onset of shear cracking, shear stresses are resisted entirely by the compressive struts of concrete. After shear cracking, shear is transferred by three components: the shear force in the uncracked concrete in the compression zone (V_c), the shear force transmitted across the crack by aggregate interlock (V_a), and the dowel-shear force developed in the longitudinal reinforcement crossing the crack (V_d) (Brzev & Pao, 2006).
All shear failures exhibit inclined cracking. The nature and extend of the inclined cracks depend on, in addition to the loading applied, the shear span to beam depth (a/d) ratio (Wight & MacGregor, 2012). Generally, the diagonal tension cracks extend from pre-existing vertical flexure cracks and extend diagonally. The upper part of the diagonal crack extends towards the compression zone of the concrete. Sometimes, the concrete in compression on top of the crack crushes, forming what is termed as shear-compression failure, shown in Figure 2.33. This failure can be seen in short shear spans where a/d ranges from 1.0 to 2.5 (Wight & MacGregor, 2012). The lower portion of the diagonal crack usually extends horizontally on the level of the reinforcement towards the support. This horizontal extension causes splitting cracks along the reinforcement, which can cause bond or anchorage failures in the extreme cases where this mechanism governs (in short shear spans where a/d ranges from 1.0 to 2.5), shown in Figure 2.34. In more slender shear spans (a/d ranging from 2.5 to 6.0), the inclined cracking itself causes an abrupt disruption in equilibrium leading to the failure of the beam from the inclined cracking itself (Wight & MacGregor, 2012). For very slender shear spans (a/d more than 6), the beam will fail in flexure prior to the formation of the inclined cracks (Wight & MacGregor, 2012). As can be seen from Figure 2.35, the a/d ratio has a critical influence on the mode of failure of the beam. For lower ratios (especially those below 2.5), the shear resistance of the beam is increased but the mode of failure is governed by shear. For high ratios, the beam is able to attain its flexural capacity before failing in flexure, but the a/d ratio has little influence on the shear capacity.
Figure 2.33: Shear-compression failure of a beam without shear reinforcement (Wight & MacGregor, 2012)

Figure 2.34: Cracking pattern in diagonal tension failure of a beam without shear reinforcement (Wight & MacGregor, 2012)
In addition to the a/d ratio, the shear strength of a reinforced concrete beam without shear reinforcement depends on other factors such as the tensile strength of the concrete, longitudinal reinforcement ratio, size of the beam, applied axial forces, and the maximum coarse aggregate size (Wight & MacGregor, 2012). Since the development of inclined shear cracks occurs when the tensile strength of the concrete is exceeded along any plane, increasing the tensile strength of the concrete.
concrete results in an increase in the load required to initiate the shear cracks. When the tensile reinforcement ratio is low, flexural cracks develop wider and higher into the beam and eventually result in wider shear cracks. This reduces the value of the shear stresses (aggregate interlock and dowel action) transferred along the cracks, causing a reduction in the overall shear strength. Hence, when the tensile reinforcement ratio is increased, the shear strength is also increased. The size of the beam also has an effect on its shear strength, provided that there is no shear reinforcement. For a given concrete tensile strength, tensile reinforcement ratio, and a/d ratio, an increase in the depth of the beam results in a decrease of the shear at failure. When the beam is deeper, the distance of the tensile reinforcement from the neutral axis increases, which cause an increase in its strain. This increase in strain results in wider cracks, which decrease the shear resistance (aggregate interlock) of the beam, as seen in Figure 2.36. This effect is known as the size effect and has received more attention over the past two decades (Collins & Kuchman, 1999). For heavily reinforced members, this size effect is less pronounced. Axial forces also effect the shear resistance of the beam. Axial tensile forces increase the strains in the tensile reinforcement, which cause wider cracks and, hence, lower shear resistance. On the other hand, axial compressive forces delay the onset of flexural cracking and decrease their width, which increases shear resistance. The maximum aggregate size affects the aggregate interlock component of shear resistance. When the maximum aggregate size increases, the roughness of the crack surfaces increases as well, which enables more transfer of shear stresses across the cracks, increasing the shear resistance. The earlier equations of the ACI regarding the shear resistance of reinforced concrete beams without stirrups did not account for the size effects of the beam as well as the maximum aggregate size. The Modified Compression Field Theory (MCFT) developed by Vecchio and Collins at the University of Toronto provided an explanation for the transfer of shear stresses across shear cracks and took into account the crack widths and spacing in determining the shear strength of reinforced concrete beams without stirrups, which accounted for the maximum aggregate size as well as the size effect (Collins & Kuchman, 1999). The simplified equations of MCFT contained in CSA A23.3 will be addressed in the design section.
Similar to monotonic shear failure, the shear fatigue failure of a reinforced concrete beam is characterized by inclined cracking. When fatigue loading is applied on the beam, shear cracks initiate after a certain load level and number of cycles. Then, these cracks propagate (towards the compression zone and along the reinforcement to the support) and widen, and new shear cracks form. The majority of fatigue tests in the literature show that the shear fatigue failure of a reinforced concrete beam is ultimately governed by the fracture of the longitudinal reinforcement intersecting the major shear crack (Zanuy et al., 2009). This crack develops from the numerous resulting shear cracks as the most dominant (and wide) crack in the beam, exposing the reinforcement. The stress in the exposed portion of the reinforcement is higher partly due to the lack of concrete resistance provided by tension stiffening (Guo, 2014). Hence, fatigue cracks initiate and propagate in the exposed reinforcement, culminating in its fracture as shown in Figure 2.37. The shear fatigue failure can be also governed by the fatigue damage of the concrete in compression. However, this failure mechanism does not govern for low fatigue load levels, especially those that do not cause inclined shear cracks in the concrete (Sparks & Menzies, 1973). The degradation of the bond between the reinforcement and the concrete (i.e. bond slip) is also a possible shear fatigue failure mechanism. Fortunately, if proper anchorage is provided, then bond slip is not a major issue (Verna & Stelson, 1963). Overall, the shear fatigue failure of reinforced
concrete beams can be due to diagonal tension, shear-compression failure of beam by the fatigue compression failure of the concrete, anchorage failure of the reinforcement, or the fracture of the reinforcement intersecting the major shear crack.

![Image](image_url)

Figure 2.37: Shear fatigue failure of a reinforced concrete beam by fracture of reinforcement intersecting major crack (Zanuy et al., 2009)

For a reinforced concrete beam without shear reinforcement experiencing fatigue loading, the ultimate shear force value is about 60% of that under monotonic loading (Guo, 2014). The ratio of the fatigue to the monotonic ultimate shear forces \( \frac{V_{u_f}}{V_{u_m}} \) greatly depends on the corresponding tensile strength ratio \( \frac{f_{t_f}}{f_{t_m}} \) (Guo, 2014). Shear fatigue failure occurs when the beam is loaded \( N \) times at the corresponding shear fatigue limit. The value of the fatigue limit decreases as the number of cycles \( (N) \) increases.

Zanuy (2008) performed a four-point fatigue bending test on a reinforced concrete beam without shear reinforcement to assess the mode of failure. The beam was 300 mm in both width and depth, and 5 m in span length. The shear span-to-depth ratio \( (a/d) \) was 5.40. The concrete strength was 25 MPa, and the longitudinal reinforcement ratio was 0.025. The maximum and minimum applied loads were 60 kN and 25 kN, respectively. Results show that vertical flexural cracks developed in the first cycles along the full span of the beam, including the shear spans. After 77,000 cycles, a diagonal shear crack suddenly developed in the shear span from an existing flexural crack. This shear crack propagated towards the compression of the concrete and along the tensile reinforcement towards the support. The initiation and propagation of this crack did not cause failure in the beam until the size of the compression zone in the concrete was reduced to the point where it crushed under the applied loads, resulting in a shear-compression failure. The development of the cracks after a certain number of cycles \( (N) \) and the final failure of the beam are shown in Figure 2.38.
Chang and Kesler (1958) carried out fatigue tests on 64 reinforced concrete beams without shear reinforcement. The beams were 102 mm wide and 152 mm deep, and had an a/d ratio of 3.72. Three different values of longitudinal steel reinforcement ratio were used: 0.0102, 0.0186, and 0.0289. All the beams exhibited diagonal cracking during loading. Figure 2.39 shows the experimental number of cycles to diagonal cracking as a function of the normalized maximum shear force. For a given number of cycles to diagonal cracking, increasing the longitudinal reinforcement ratio increased the maximum shear force required to cause diagonal cracking. Many beams exhibited diagonal tension failure, while others contained significant residual strength after the diagonal cracking, failing due to the fatigue compression of the concrete (shear-compression failure). The scatter of the data with regards to the exact failure mode prevents reaching at clear conclusions with regards to critical factors causing the exact failure mode (whether diagonal tension failure or shear-compression failure). This further shows the complexity of the shear fatigue failure of reinforced concrete beams. The cracking patterns showing the diagonal tension failure and the shear-compression failure in the experiment are shown in Figure 2.40.
Schlafli and Bruhwiler (1998) performed four-point bending fatigue tests on 10 slender slab-like reinforced concrete beams without shear reinforcement. The beams were 150 mm in depth and 400 mm in width, with a span of 2.5 meters. The a/d ratio was 4.2. Three different longitudinal reinforcement ratios were used with the beams: 0.68%, 1.37%, and 1.60%. The loading pattern was a sinusoidal wave with a frequency of 4.5 Hz. The maximum loads used were 75-85% of the ultimate static strength load, while the minimum loads were 10% of the maximum loads. Tests results show that all the beams exhibited shear fatigue failure by the fracture of the reinforcement. Subsequent loading after fracture caused the plastification of the remaining bars (Schlafli & Bruhwiler, 1998). During the first thousand cycles, strains in the beam increased significantly and
crack propagation was rapid, resulting in significant cracks of 1-2 mm in width. At this stage, new cracks also developed, especially in the shear spans. The rate of increase of strains and crack propagation decreased in the subsequent cycles, until the final loading phases where fracture occurred. There was no crushing of concrete or spalling in the compression zone due to the fatigue loading for all reinforcement ratios used. In addition, significant reduction in the modulus of elasticity of concrete was observed due to the redistribution of stresses. The beams in the test exhibited shear failure in fatigue although they were not shear-critical beams under static loading.

To investigate the shear fatigue behaviour of reinforced concrete beams and its relation to the a/d ratio of the beam, Higai (1983) performed four-point bending fatigue tests on 23 rectangular reinforced concrete beams with different a/d ratios. The beams were divided into four sets, with each set having a different a/d ratio (ratios of 2, 4, 5, and 6.36). The maximum loading applied was 43-83% of the static shear failure load, while the minimum loading applied was 17-25% of the maximum load. The loading frequency used was 5 Hz. Table 2.2 summarizes the test results, with the last column showing the mode of failure for each beam. All beams exhibited shear failure. Those with an a/d ratio of 6.4 exhibited diagonal tension failure; they failed immediately with the development of the diagonal cracks. This failure mechanism was the same for static and fatigue loading modes in these beams. Beams with an a/d ratio of 2.0 exhibited arch failure. Although these beams developed diagonal cracking at the first loading cycle, they were able to resist more load cycles by the arching mechanism, in which the concrete in compression above the cracks formed the struts while the reinforcement formed the tension ties. The final failure of these beams was due to arching failure; either crushing of the concrete in compression or fracture of the reinforcement intersecting a major shear crack. This failure mechanism was the same for both static and fatigue loading of these beams, except for the fracture of the tensile reinforcement as it is not witnessed in failures due to static loads. For beams with an a/d ratio ranging from 2.0 to 6.4, the failure modes exhibited were diagonal tension and arching failure. The failure modes in these beams (i.e. with an a/d ratio from 2.0 to 6.4) were very complicated: the failure modes under fatigue loading were not the same as those under static loading. In addition, changing the value of the maximum fatigue force applied changing the failure mode; for example, one of the beams with an a/d of 5.0 exhibited arching failure when the maximum fatigue loading applied was 63% of the static shear failure load, but when the maximum applied loading was changed to 76% of the static shear failure load, the beam failed directly after the development of the diagonal crack (i.e.
diagonal tension failure). Higai (1978) suggested the graph in Figure 2.41 as a guide to the failure modes expected for different a/d ratios.

Table 2.2: Summary of test results for Higai experiment (Higai, 1978)

<table>
<thead>
<tr>
<th>a/d</th>
<th>No.</th>
<th>σc (MPa)</th>
<th>Pmax (kN)</th>
<th>Pmax/Ps</th>
<th>τ (MPa)</th>
<th>Nc (10^4)</th>
<th>Nu (10^4)</th>
<th>Failure mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>FC1</td>
<td>34.6</td>
<td>(99.1)</td>
<td>(4.13)</td>
<td>(4.13)</td>
<td></td>
<td></td>
<td>Static, A</td>
</tr>
<tr>
<td></td>
<td>FC2</td>
<td>35.1</td>
<td>(78.5)</td>
<td>(3.27)</td>
<td>(3.27)</td>
<td></td>
<td></td>
<td>Static, A</td>
</tr>
<tr>
<td></td>
<td>FC4</td>
<td>35.1</td>
<td>39.2</td>
<td>0.43</td>
<td>1.64</td>
<td>&gt; 200</td>
<td></td>
<td>A*</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>49.0</td>
<td>0.54</td>
<td>2.04</td>
<td>&gt; 200</td>
<td></td>
<td>A</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>58.8</td>
<td>0.64</td>
<td>2.45</td>
<td>95</td>
<td></td>
<td>DT</td>
</tr>
<tr>
<td></td>
<td>FC5</td>
<td>31.9</td>
<td>53.9</td>
<td>0.62</td>
<td>2.25</td>
<td>17</td>
<td>15</td>
<td>A*</td>
</tr>
<tr>
<td></td>
<td>FC6</td>
<td>31.9</td>
<td>53.9</td>
<td>0.82</td>
<td>1.64</td>
<td>59</td>
<td>17</td>
<td>A*</td>
</tr>
<tr>
<td></td>
<td>FC7</td>
<td>38.3</td>
<td>49.0</td>
<td>0.52</td>
<td>2.04</td>
<td>95</td>
<td>59</td>
<td>A</td>
</tr>
<tr>
<td></td>
<td>FC8</td>
<td>35.3</td>
<td>58.8</td>
<td>0.64</td>
<td>2.45</td>
<td>14</td>
<td>15</td>
<td>A</td>
</tr>
<tr>
<td>4</td>
<td>FT1</td>
<td>41.2</td>
<td>(84.3)</td>
<td>(1.06)</td>
<td></td>
<td></td>
<td></td>
<td>Static, DT</td>
</tr>
<tr>
<td></td>
<td>FT2</td>
<td>40.3</td>
<td>49.0</td>
<td>0.51</td>
<td>0.62</td>
<td>&gt; 200</td>
<td>&gt; 200</td>
<td></td>
</tr>
<tr>
<td></td>
<td>FT3</td>
<td>39.8</td>
<td>53.9</td>
<td>0.57</td>
<td>0.68</td>
<td>109</td>
<td>174</td>
<td></td>
</tr>
<tr>
<td></td>
<td>FT4</td>
<td>39.8</td>
<td>58.8</td>
<td>0.62</td>
<td>0.74</td>
<td>7</td>
<td>7</td>
<td>A</td>
</tr>
<tr>
<td></td>
<td>FT18</td>
<td>36.2</td>
<td>(68.6)</td>
<td>(1.07)</td>
<td></td>
<td></td>
<td></td>
<td>Static, DT</td>
</tr>
<tr>
<td></td>
<td>FT13</td>
<td>31.1</td>
<td>55.4</td>
<td>0.83</td>
<td>0.86</td>
<td>0.2</td>
<td>0.2</td>
<td>DT</td>
</tr>
<tr>
<td></td>
<td>FT14</td>
<td>29.1</td>
<td>49.0</td>
<td>0.76</td>
<td>0.77</td>
<td>0.3</td>
<td>0.3</td>
<td>DT</td>
</tr>
<tr>
<td></td>
<td>FT15</td>
<td>28.2</td>
<td>42.2</td>
<td>0.66</td>
<td>0.72</td>
<td>39</td>
<td>&gt; 106</td>
<td></td>
</tr>
<tr>
<td></td>
<td>FT16</td>
<td>32.8</td>
<td>46.1</td>
<td>0.67</td>
<td>0.72</td>
<td>34</td>
<td>79</td>
<td>A</td>
</tr>
<tr>
<td></td>
<td>FT17</td>
<td>31.1</td>
<td>42.2</td>
<td>0.63</td>
<td>0.66</td>
<td>241</td>
<td>&gt; 241</td>
<td></td>
</tr>
<tr>
<td></td>
<td>FT19</td>
<td>35.1</td>
<td>47.1</td>
<td>0.67</td>
<td>0.74</td>
<td>6</td>
<td>73</td>
<td>A</td>
</tr>
<tr>
<td>5</td>
<td>FT5</td>
<td>34.4</td>
<td>(74.5)</td>
<td>(1.13)</td>
<td></td>
<td></td>
<td></td>
<td>Static, DT</td>
</tr>
<tr>
<td></td>
<td>FT6</td>
<td>34.4</td>
<td>55.9</td>
<td>0.78</td>
<td>0.84</td>
<td>0.05</td>
<td>0.05</td>
<td>DT</td>
</tr>
<tr>
<td></td>
<td>FT7</td>
<td>35.5</td>
<td>48.1</td>
<td>0.65</td>
<td>0.73</td>
<td>44</td>
<td>44</td>
<td>DT</td>
</tr>
<tr>
<td></td>
<td>FT8</td>
<td>33.4</td>
<td>41.2</td>
<td>0.58</td>
<td>0.63</td>
<td>&gt; 120</td>
<td>&gt; 120</td>
<td></td>
</tr>
<tr>
<td></td>
<td>FT9</td>
<td>32.1</td>
<td>44.1</td>
<td>0.63</td>
<td>0.67</td>
<td>&gt; 106</td>
<td>&gt; 106</td>
<td></td>
</tr>
</tbody>
</table>

τ: Average shear stress under Pmax.
A: Arch failure
A*: Arch failure (fracture of bar)
DT: Diagonal tension failure
If a beam exhibits a certain failure mode under static loading, it is not necessarily true that it will exhibit the same failure mode under fatigue loading. This is true regardless whether the beam contains shear reinforcement or not, or whether it is a slender or deep beam. Verna and Stelson (1963) performed fatigue tests on reinforced concrete beams without shear reinforcement and compared the failure modes under both static and fatigue loading. They concluded that the beams that failed in compression and diagonal tension under static loading could fail in either modes of failure under fatigue loading. They also concluded that the beams which exhibited bond failure under static loading (perhaps due to poor anchorage) would exhibit the same bond failure under fatigue loading. Teng et al. (2000) conducted fatigue and static load tests on beams with an a/d ratio of 1.5. Some of these deep beams contained vertical shear reinforcement, while other did not contain any shear reinforcement. Although they failed in diagonal tension under static loading, these beams exhibited flexural failure (i.e. fracture of the tensile reinforcement) under fatigue loading (Teng, Ma, & Wang, 2000). Some other tests have shown that low fatigue loads could result in the flexural failure of otherwise shear-critical beams (Chang & Kesler, 1958). These tests reinforce the concept that the static failure mode does is not necessarily the same as the fatigue failure mode.

2.4.4.2.2 Behaviour of Beams Containing Shear Reinforcement

Shear reinforcement is provided to enhance the shear performance of the beams. When the shear load exceeds the shear capacity of the beam, shear reinforcement is provided to carry the excess shear beyond the concrete shear resistance (Brzev & Pao, 2006). In addition, shear reinforcement
acts to prevent the brittle shear failure of the beam by arresting the growth of the inclined cracking and preventing its penetration into the compression zone (Brzev & Pao, 2006). The forces of a beam with stirrups at an inclined crack is shown in Figure 2.42. It is the same as those in a beam without stirrups (Figure 2.32), except for $V_s$ which is the shear force carried by the stirrups. Prior to the onset of flexural cracking, the applied shear is carried entirely by the uncracked concrete. After flexural cracking, dowel action and aggregate interlock take part in resisting the shear (Wight & MacGregor, 2012). The shear reinforcement will not take part in resisting the shear until inclined shear cracking occurs, after which the shear reinforcement intersecting the inclined cracks takes part in resisting the shear. As the applied shear on the beam gets higher, the contribution of the shear reinforcement increases (until the yield of the stirrups) while the resistance due to concrete and tensile reinforcement decreases. The different components of shear resistance as functions of applied shear are shown in Figure 2.43. The most efficient position of the shear reinforcement is perpendicular to the inclined cracks, so that the tensile forces perpendicular to the cracks are along the axial length of the reinforcement (Brzev & Pao, 2006). However, shear reinforcement is usually placed as vertical stirrups for practical reasons.

![Figure 2.42: Free-body diagram at an inclined shear crack for a beam with stirrups (Wight & MacGregor, 2012)](image-url)
The shear transfer in a cracked reinforced concrete beam with shear reinforcement has been traditionally modelled by the truss analogy shown in Figure 2.44. The beam is modelled as a truss, with the longitudinal reinforcement acting as the tension ties, the concrete in the compression zone acting as the compression chord, the shear reinforcement acting as the vertical truss members, and the compression struts of the concrete between the inclined cracks acting as the diagonal truss members (Brzev & Pao, 2006). The angle between the diagonal members and the horizontal tension ties is represented by $\theta$, which is typically in the order of $45^\circ$. The height of the truss is equal to the lever arm ($d_v$) between the compression and tension forces forming the bending moment. The distance between adjacent vertical truss members is $d_v \cot(\theta)$. All the stirrups crossing a horizontal distance ($d_v\cot(\theta)$) are lumped into one member (Brzev & Pao, 2006). All external forces are applied at the pins of the truss. This system enables solving the forces in the truss using equilibrium concepts.
Stirrups that intersect shear cracks are subject to tensile stresses. It is desired that the stirrups yield before the beam fails in shear. Slender beams with small amounts of shear reinforcement will fail in diagonal tension; the stirrups will yield immediately after the opening of the inclined cracks, after which the beam will fail (Brzev & Pao, 2006). Beams with moderate shear reinforcement will exhibit shear-compression failure; the stirrups will yield first, followed by crushing of the concrete in the compression zone. However, if the amount of shear reinforcement provided is large, the concrete will crush before the stirrups will yield (Brzev & Pao, 2006).

The failure of shear-critical beams containing shear reinforcement in fatigue is similar to that of shear-critical beams without shear reinforcement. The only additional possible mode of failure in fatigue for beams containing stirrups is the fracture of the stirrups intersecting major shear cracks. When shear cracks widen under applied loading, cracks in the stirrups will initiate and propagate until the final fracture.
Chang and Kesler (1958) conducted fatigue tests on reinforced concrete beams containing different ratios of shear reinforcement but having the same a/d ratio (2.5) and similar amounts of longitudinal reinforcement. Results show that the beams with lower ratios of shear reinforcement exhibited fatigue failure by fracture of longitudinal reinforcement while those with the highest shear reinforcement ratios failed in fatigue by fracture of the stirrups. Although the bends of the stirrups have lower strengths than the vertical legs, fracture of stirrups can happen in either of these regions.

Chang and Chai (1989) performed fatigue tests on eleven reinforced concrete beams of varying amounts of shear reinforcement. The longitudinal reinforcement ratio (1.5%), cross-sectional dimensions (width and height of 240 mm), and spans (1.90 m) were the same for all the beams. The concrete compressive strength was 22.1 MPa. At a uniform loading frequency of 5 Hz, the minimum fatigue load applied was 39.2 kN, while the maximum load was varied for each beam. Test results showed that the strains of the stirrups generally increased when the number of load cycles increased. However, these strains underwent a temporary decrease on the onset of shear cracking due to stress redistribution. The stirrup strain was closely related to the diagonal crack formation, not to the bending crack formation. The failure modes of the beams did not clearly correlate with any of the test variables: three beams failed due to the fracture of the longitudinal reinforcement intersecting the major crack, and the remaining eight beams failed due to the fracture of the stirrups intersecting the major shear cracks (Chang & Chai, 1989).

2.4.4.3 Shear Fatigue Behaviour of Deep Reinforced Concrete Beams

According to ACI Code Section 10.7.1, a deep beam is defined as a member “loaded on one face and supported on the opposite face so that compression struts can develop between the loads and the supports, and having either clear spans equal to or less than four times the overall member height or regions loaded with concentrated loads within two times the height from the face of the support”. The Canadian concrete code (CSA A23.3-04) in clause 10.7.1 defines deep beams as those having a clear span to overall depth ratio less than 2. Such beams have various structural applications, including wind turbine foundations and transfer girders.

Deep beams are considered discontinuity regions in which the traditional elastic analysis does not apply. Prior to cracking, elastic analysis can be accurately performed to predict the behaviour of such beams. However, a major stress redistribution and reorientation of internal forces occur that
render elastic analysis inaccurate (Wight & MacGregor, 2012). In this case, plane sections do not remain plane, hence the strain compatibilities are violated. Instead, the beam is idealized as a “series of reinforcing steel tensile ties and concrete compressive struts interconnected at nodes to form a truss” capable of carrying all of the loads to the supports (Canadian Standards Association, 2004). The compressive struts consist of compressive stress trajectories acting in the direction of the strut (i.e. parallel to the span of the strut). Arch action is responsible for shear transfer in cracked deep beams, in which the concrete compressive strut is inclined throughout the shear span, and intersects the horizontal reinforcement tension tie at the support, as shown in Figure 2.45.

![Figure 2.45: Arch action in a beam (Wight & MacGregor, 2012)](image)

Different strut-and-tie configurations are formed for different applied loading configuration and reinforcement detailing. The strut-and-tie truss configuration for a simply supported beam without transverse reinforcement subjected to a concentrated load at mid-span is shown in Figure 2.46. In this beam, the concrete compressive struts connect the applied load with the support through diagonal struts, while the longitudinal reinforcement forms the tension tie connecting the two struts at the supports. The nodal zones are found at the ends of the compressive struts. The resulting strut-and-tie truss for a similar configuration beam but with transverse reinforcement is shown in Figure 2.47. As can be seen, compression fans exist at the vicinity of concentrated loads (applied load and support loads). These fans are compression struts that radiate out of the applied concentrated loads and are transferred to the local tension ties (e.g. transverse reinforcement) (Wight & MacGregor, 2012). Compression fields are parallel diagonal compressive struts combined with compression chords and tension ties (Wight & MacGregor, 2012).
There are three possible shear fatigue failures of deep reinforced concrete beams: diagonal tension failure (i.e. diagonal splitting of concrete), shear compression failure (crushing of concrete struts), and fracture of longitudinal reinforcement intersecting major cracks. Bond anchorage failure is not an issue if proper anchorage is provided. Experiments have shown that the contribution of stirrups in deep beams is less than that in regular beams; hence the fracture of the stirrups intersecting major cracks is not a common fatigue failure mode in such beams. This is related to the nature of shear cracks that form in deep beams; these cracks are steeper and have a higher angle with respect to the horizontal; that is, they are closer to vertical cracks than regular diagonal shear cracks (Wight & MacGregor, 2012). As such, there would be a less amount of stirrups crossing these cracks, which limits their contribution. A deep beam (a/d ratio of 1) which failed due to the fracture of the longitudinal reinforcement intersecting the major shear crack is shown in Figure 2.48. It can be seen in this beam that the major shear crack is more vertical and steep, and does not intersect any stirrup. In addition, the clamping stresses at the compressive struts decrease the tensile stresses in
the stirrups, which also limits their contribution. Nonetheless, transverse shear reinforcement acts to prevent the diagonal splitting of the concrete compression strut (Isojeh & Vecchio, 2016).

![Figure 2.48: Major shear crack in a deep beam (Zanuy et al. 2008)](image)

Teng et al. (2000) performed fatigue tests on 7 deep beams with an a/d ratio of 1.5. Shear reinforcements were placed in some of these beams. Results show that the failure of the beams was ultimately governed by the fracture of the longitudinal reinforcement crossing the major crack. The failure was flexural in nature. The shear reinforcement did not yield during the failure of the beams, which confirms their limited contribution to mitigating damage and arresting crack growth in deep beams. When identical beams were tested statically, they failed in shear through diagonal splitting. It was concluded through this experiment that there is a failure alteration for deep beams from shear failure under static loading to flexural failure under fatigue loading. This alteration is reversed for slender flexural beams according to many experiments; that is, their failure due to fatigue loading is a shear failure despite them failing in flexure when subjected to static loading.

Isojeh and Vecchio (2016) performed fatigue tests on 4 reinforced concrete beams, in addition to static tests of two identical control beams. The specimens had dimensions of 175 mm thickness, 250 mm depth, and 700 mm length. All the beams had a constant a/d ratio of 1.25, and contained a vertical shear reinforcement ratio of 0.2%. The longitudinal reinforcement ratio alternated between 0.45% and 0.90% in the beams. The fatigue loading consisted of a pulsating sinusoidal wave of 5 Hz frequency. The minimum fatigue load level was 5 kN, while the maximum load level alternated between 70% and 80% of the static strength load. The reinforcement and loading details of each beam are shown in Table 2.3. The resulting cracking pattern at failure for all the beams are shown in Figure 2.49. Under static loading, the control beam with the lower reinforcement ratio failed by fracture of the reinforcement crossing the major flexural crack, which was also
accompanied by major shear cracks. The other control beam failed by crushing of the concrete, exhibiting a shear compression mode of failure. In this beam, the applied stresses were able to crush the concrete, but were too low to cause yielding or fracture of the reinforcement. The four beams subjected to fatigue loading all failed by the fracture of the longitudinal reinforcement crossing the major crack.

Table 2.3: Test beams for Isojeh and Vecchio (2016)

<table>
<thead>
<tr>
<th>Sp.</th>
<th>$f'_c$, MPa</th>
<th>$\rho_t$ (%)</th>
<th>$\rho_v$ (%)</th>
<th>$P^o$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C7</td>
<td>63</td>
<td>0.45</td>
<td>0.2</td>
<td>70</td>
</tr>
<tr>
<td>C'7</td>
<td>63</td>
<td>0.9</td>
<td>0.2</td>
<td>70</td>
</tr>
<tr>
<td>C8</td>
<td>63</td>
<td>0.45</td>
<td>0.2</td>
<td>80</td>
</tr>
<tr>
<td>C'8</td>
<td>63</td>
<td>0.9</td>
<td>0.2</td>
<td>80</td>
</tr>
<tr>
<td>T1</td>
<td>55</td>
<td>0.9</td>
<td>0.2</td>
<td>Mono</td>
</tr>
<tr>
<td>T2</td>
<td>55</td>
<td>0.45</td>
<td>0.2</td>
<td>Mono</td>
</tr>
</tbody>
</table>
Figure 2.49: Crack patterns at failure of Isojeh and Vecchio (2016) beams

The fatigue failure of deep beams is a complex process depending on a multitude of factors, including (but not limited to) the shear span-to-depth ratio, amount and properties of longitudinal reinforcement, properties and section size of the concrete, and the nature and magnitude of the
fatigue loading. For beams with a/d ratios that are less than 1, the compressive struts developed are more vertical, approaching the behaviour of a concrete under uniaxial compression; hence, the failure of such beams will be ultimately governed by the crushing of the struts. For deep beam with higher a/d ratios, when the crack and damage propagation in the longitudinal reinforcement happens at a lower stress level or a faster rate than that of the concrete in compression, the reinforcement will fracture before the concrete crushes. This is particularly common for under-reinforced sections. When the section is heavily reinforced, large stress levels or more load cycles are required to fracture the reinforcement. Hence, the concrete crushes before the reinforcement is fractured. The fatigue limits of the concrete and the reinforcement can play a part in the failure of deep beams. For a certain loading level and behaviour, if the fatigue limit (i.e. endurance limit) of only one component is exceeded, then the ultimate fatigue failure will be governed by the fracture of this component. For example, if the endurance limit for the reinforcement is exceeded but that of the concrete is not in a deep beam subject to fatigue loading, then the ultimate fatigue failure of this beam will be governed by the fracture of the reinforcement intersecting the major crack. The diagonal splitting of the concrete can also be a mode of failure: if the diagonal crack propagates at rate fast enough to cause a drop in the beam capacity, which can be the case if this crack is very wide, the failure of the beam will be ultimate governed by it.

Fatigue loading on concrete causes stress redistribution, damage accumulation (strength and stiffness degradation), and strain irreversibility. The stresses induced in the longitudinal reinforcement intersecting a crack increase as a result of the irreversible strains in the concrete (Isojeh & Vecchio, 2016): prior to the application of the next cycle of load, there is a prestrain in the concrete (and the reinforcement) as a result of previous load cycles.

2.5 OUTSTANDING ISSUES AND RESEARCH NEEDS

Fatigue is a complex topic due to the various parameters that can affect the fatigue response of the structure: stress range, stress ratio, loading rate, rest periods, loading sequence, detailing of structure, fatigue strength of the concrete and the reinforcement, and many other parameters. In view of this, there are some outstanding issues in fatigue research that need to be addressed.

Relating the fatigue failure mode of reinforced concrete beams to the monotonic failure mode is an issue not discussed thoroughly in the literature. Although some experiments have briefly discussed this issue, such discussions were merely noting the difference between the failure modes
due to monotonic and fatigue loading based on observations. No discussions were provided as to why the divergence between the failure modes occurs.

The experiments performed to study the behaviour of reinforced concrete beams under fatigue loading were mostly small-scale experiments. To better simulate the behaviour of real structures subject to fatigue loading, which are typically large in size, and account for the difference in behaviour due to the size of the specimen, more large-scale experiments are needed. The larger costs of these experiments, compared to small-scale experiments, can be offset by more accurate and representative results.

Further verification of the model proposed for fatigue damage is required, especially using large-scale experiments. With additional verification, the confidence in the versatility and applicability of the proposed models to real structures subjected to fatigue loading, such as wind turbine foundations, will increase, potentially leading to their use in practice.
Chapter 3: Current Design of Wind Turbine Foundations

3 Current Design of Wind Turbine Foundations

3.1 Overview of the Design Process

The design of wind turbine foundations is not a ‘one-way’ approach, but rather an iterative process requiring the input of many experts and utilizing the information obtained from various investigations and analyses. After the site location for the wind turbine is chosen, various site investigations are carried out to determine important parameters relevant to the design. The investigation mostly relevant to the design of the foundation is that of the soil to determine its bearing capacity, stiffness, seismicity, groundwater level and other parameters. Then, the size and the concept of the foundation is chosen depending on the soil conditions, especially the bearing capacity, and the stability demands of the foundation (i.e. protection against overturning).

The pressure from the soil on the foundation and the forces transferred from the tower are analyzed to determine the critical loading combinations, which would produce distributions of the design bending moments, axial loads, and shear forces. These values determine the total demand on the foundation, which will enable the designer to identify the areas and the magnitudes of critical stresses and forces and design for them accordingly. Note that the forces transferred from the tower to the foundation (axial and horizontal forces as well as bending and twisting moments) are supplied by the contractors responsible for the tower, blade, and hub design. The standards and codes used in the foundation design include DNV-RISO: Guidelines for the Design of Wind Turbine Foundations, DNVGL-0126: Support Structures for Wind Turbines, International Electro-technical Commission IEC 61400, and CSA A23.3: Design of Concrete Structures.

The method used in the structural design of the foundation to achieve its prescribed level of safety is the limit states design; when the structure fails to perform its intended function and ceases to be safe from collapse, it has reached its limit state. In design terms, this method consists of ensuring that the factored design load effects (demand) do not exceed the factored design resistance (supply). The design load and resistance factors are prescribed based on structural reliability analyses to ensure that the variabilities and uncertainties involved are considered, which include uncertainties in the analytical models used and variabilities in the materials’ properties. The limit
states considered in the design of the foundation consist of the ultimate limit state, serviceability limit state, fatigue limit state, and accidental (abnormal) limit state (DNV/RISO, 2002). Note, however, that the abnormal limit state is often considered as part of the ultimate limit state. The ultimate limit state of the foundation corresponds to its safety, strength, stability, and prevention of large deformations that would affect its structural integrity. The serviceability limit state corresponds to the foundation’s satisfactory performance and the prevention of undesirable deformations, vibrations, and gapping. The limit states translate into critical load combinations and certain prescriptive measures dictated by the relevant codes, previous experience, and/or good practice. Durability of the foundation is also considered in the design to ensure that it is not critically affected by adverse environmental conditions.

3.2 Load Combinations

To verify that the limit states of the wind turbine foundation are not violated during different stages of its operation life, different loading cases reflecting various design life and operational stages are considered. These loading cases consist of combinations of factored loads to arrive at the most critical demand on the foundation. The International Electrotechnical Commission (IEC 61400) specifies (in Clause 7.3) that the following loads be considered:

- Gravitational and inertial loads: static and dynamic loads that result from gravity, vibration, rotation, and seismic activity
- Aerodynamic loads: static and dynamic loads that are caused by the airflow and its interaction with the stationary and moving parts of the wind turbine
- Actuation loads
- Other loads

Note that these loads are not only applicable to the foundation but to the entirety of the wind turbine assembly. Therefore, some of these loads are not applicable to the foundation design. The design load cases have different probabilities of occurrences depending on the operation and external conditions. Only those with reasonable occurrence probabilities should be used to verify the structural integrity of the foundation. According to IEC 61400, the design load cases should be found by combining:

- Normal design situations and appropriate normal or extreme external conditions
- Fault design situations and appropriate external conditions
- Transportation, installation, and maintenance design situations and appropriate external conditions

The different design load cases according to IEC 61400 are given in Table 3.1. The type of analysis considered is either for that of the ultimate loads (denoted by U) or fatigue loads (denoted by F). For ultimate load cases, they are classified as either normal (N), abnormal (A), or transport and erection (T) cases. Normal load cases occur frequently in the design life and do not cause major faults, unlike the abnormal cases which are less likely to occur but are more damaging. The following notation is used in Table 3.1:

- DLC: design load case
- ECD: extreme coherent gust with direction change
- EDC: extreme direction change
- EOG: extreme operating gust
- EWM: extreme wind speed model
- EWS: extreme wind shear
- NTM: normal turbulence model
- ETM: extreme turbulence model
- NWP: normal wind profile model
- V: wind speed
Table 3.1: Different load cases for the wind turbine foundation

<table>
<thead>
<tr>
<th>Design situation</th>
<th>DLC</th>
<th>Wind condition</th>
<th>Other conditions</th>
<th>Type of analysis</th>
<th>Partial safety factors</th>
</tr>
</thead>
<tbody>
<tr>
<td>1) Power production</td>
<td>1.1</td>
<td>NTM $V_{in} &lt; V_{hub} &lt; V_{out}$</td>
<td>For extrapolation of extreme events</td>
<td>U</td>
<td>N</td>
</tr>
<tr>
<td></td>
<td>1.2</td>
<td>NTM $V_{in} &lt; V_{hub} &lt; V_{out}$</td>
<td></td>
<td>F</td>
<td>*</td>
</tr>
<tr>
<td></td>
<td>1.3</td>
<td>ETM $V_{in} &lt; V_{hub} &lt; V_{out}$</td>
<td></td>
<td>U</td>
<td>N</td>
</tr>
<tr>
<td></td>
<td>1.4</td>
<td>ECD $V_{hub} = V_r - 2 \text{ m/s, } V_r$, $V_r + 2 \text{ m/s}$</td>
<td></td>
<td>U</td>
<td>N</td>
</tr>
<tr>
<td></td>
<td>1.5</td>
<td>EWS $V_{in} &lt; V_{hub} &lt; V_{out}$</td>
<td></td>
<td>U</td>
<td>N</td>
</tr>
<tr>
<td>2) Power production plus occurrence of fault</td>
<td>2.1</td>
<td>NTM $V_{in} &lt; V_{hub} &lt; V_{out}$</td>
<td>Control system fault or loss of electrical network</td>
<td>U</td>
<td>N</td>
</tr>
<tr>
<td></td>
<td>2.2</td>
<td>NTM $V_{in} &lt; V_{hub} &lt; V_{out}$</td>
<td>Protection system or preceding internal electrical fault</td>
<td>U</td>
<td>A</td>
</tr>
<tr>
<td></td>
<td>2.3</td>
<td>EOG $V_{hub} = V_r \pm 2 \text{ m/s and } V_{out}$</td>
<td>External or internal electrical fault including loss of electrical network</td>
<td>U</td>
<td>A</td>
</tr>
<tr>
<td></td>
<td>2.4</td>
<td>NTM $V_{in} &lt; V_{hub} &lt; V_{out}$</td>
<td>Control, protection, or electrical system faults including loss of electrical network</td>
<td>F</td>
<td>*</td>
</tr>
<tr>
<td>3) Start up</td>
<td>3.1</td>
<td>NWP $V_{in} &lt; V_{hub} &lt; V_{out}$</td>
<td></td>
<td>F</td>
<td>*</td>
</tr>
<tr>
<td></td>
<td>3.2</td>
<td>EOG $V_{hub} = V_{in}, V_r \pm 2 \text{ m/s and } V_{out}$</td>
<td></td>
<td>U</td>
<td>N</td>
</tr>
<tr>
<td></td>
<td>3.3</td>
<td>EDC $V_{hub} = V_{in}, V_r \pm 2 \text{ m/s and } V_{out}$</td>
<td></td>
<td>U</td>
<td>N</td>
</tr>
<tr>
<td>4) Normal shut down</td>
<td>4.1</td>
<td>NWP $V_{in} &lt; V_{hub} &lt; V_{out}$</td>
<td></td>
<td>F</td>
<td>*</td>
</tr>
<tr>
<td></td>
<td>4.2</td>
<td>EOG $V_{hub} = V_r \pm 2 \text{ m/s and } V_{out}$</td>
<td></td>
<td>U</td>
<td>N</td>
</tr>
<tr>
<td>5) Emergency shut down</td>
<td>5.1</td>
<td>NTM $V_{hub} = V_r \pm 2 \text{ m/s and } V_{out}$</td>
<td></td>
<td>U</td>
<td>N</td>
</tr>
<tr>
<td>6) Parked (standing still or idling)</td>
<td>6.1</td>
<td>EWM 50-year recurrence period</td>
<td></td>
<td>U</td>
<td>N</td>
</tr>
<tr>
<td></td>
<td>6.2</td>
<td>EWM 50-year recurrence period</td>
<td>Loss of electrical network connection</td>
<td>U</td>
<td>A</td>
</tr>
<tr>
<td></td>
<td>6.3</td>
<td>EWM 1-year recurrence period</td>
<td>Extreme yaw misalignment</td>
<td>U</td>
<td>N</td>
</tr>
<tr>
<td></td>
<td>6.4</td>
<td>NTM $V_{hub} &lt; 0,7 V_{ref}$</td>
<td></td>
<td>F</td>
<td>*</td>
</tr>
<tr>
<td>7) Parked and fault conditions</td>
<td>7.1</td>
<td>EWM 1-year recurrence period</td>
<td></td>
<td>U</td>
<td>A</td>
</tr>
<tr>
<td>8) Transport, assembly, maintenance and repair</td>
<td>8.1</td>
<td>NTM $V_{maint}$ to be stated by the manufacturer</td>
<td></td>
<td>U</td>
<td>T</td>
</tr>
<tr>
<td></td>
<td>8.2</td>
<td>EWM 1-year recurrence period</td>
<td></td>
<td>U</td>
<td>A</td>
</tr>
</tbody>
</table>
Different partial load safety factors are used with the load cases, depending on the nature of the load. These factors are given in Table 3.2. IEC 61400 permits the use of load factors from national and local building codes along with these factors, as long as the reliability and safety of the combined factors are more than the ones given in Table 3.2.

Table 3.2: Partial load safety factors given in IEC 61400

<table>
<thead>
<tr>
<th>Type of design situation (see Table 2)</th>
<th>Unfavourable loads</th>
<th>Favourable loads</th>
</tr>
</thead>
<tbody>
<tr>
<td>Normal (N)</td>
<td>1.35*</td>
<td>1.1</td>
</tr>
<tr>
<td>Abnormal (A)</td>
<td>1.1</td>
<td></td>
</tr>
<tr>
<td>Transport and erection (T)</td>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>All design situations</td>
<td>0.9</td>
<td></td>
</tr>
</tbody>
</table>

* For design load case DLC 1.1, given that loads are determined using statistical load extrapolation at prescribed wind speeds between $\gamma_{\text{in}}$ and $\gamma_{\text{out}}$, the partial load factor for normal design situations shall be $\gamma = 1.25$.

If for normal design situations the characteristic value of the load response $F_{\text{gravity}}$ due to gravity can be calculated for the design situation in question, and gravity is an unfavourable load, the partial load factor for combined loading from gravity and other sources may have the value

$$\gamma_{f} = 1.1 + \varphi z$$

where

- $\varphi = 0.15$ for DLC 1.1
- $0.25$ otherwise

$$z = \begin{cases} \frac{|F_{\text{gravity}}|}{F_{\text{gravity}}} & |F_{\text{gravity}}| > |F| \\ 1 & |F_{\text{gravity}}| > |F| \end{cases}$$

The nacelle, hub, tower, and blades manufacturer assesses the different wind speeds and their impacts on the loading cases and supplies the foundation designer with the critical forces transferred from the tower to the foundation. These forces consist of a vertical force, overturning moment, horizontal force, and a twisting moment. The foundation designer uses these loads, in addition to the other loads acting on the foundation (soil contact pressure, backfill weight, self-weight of the foundation, earthquake load, and other imposed loads) to determine the critical load cases on the foundation for the ultimate, serviceability and fatigue limit states.

Usually three different loading cases govern for the foundation: abnormal extreme loading case, normal extreme loading case, and a normal operation loading case. The first two cases are for the ultimate limit state and therefore are factored according to Table 3.2, while the last loading case (the normal operation loading) is for the serviceability limit state and, hence, the loads in it are not factored.
3.3 Soil Bearing Capacity and Settlement Considerations

The ultimate load path of the structural design is for the forces and stresses to be transferred from the superstructure to the foundation and ultimately to the underlying soil. Before ensuring that the foundation is designed adequately to withstand the demand from the superstructure, the required satisfactory performance of the soil for the given foundation concept and operational loads has to be verified. This performance means that the soil can safely take all the forces transferred from the foundation without collapse or undesired settlements. Each soil has an ultimate strength value, its bearing capacity, after which collapse (in the form of soil planes shearing against each other) occurs (Bowles, 1997). Even if the soil demonstrates enough bearing capacity to hold the transferred forces, these forces can cause settlements that are damaging to the structure. The geotechnical engineer determines several soil parameters from which the allowable soil pressure can be determined. The allowable soil pressure is the maximum pressure that can be applied to the soil. This is determined by dividing the bearing capacity of the soil by a suitable factor of safety. Note that the bearing capacity of the soil determined by the geotechnical engineer takes into account both the strength as well as the settlements of the soil; by not exceeding this capacity, the soil will safely support the foundation without undergoing undesirable settlements. There are several methods and equations given in geotechnical books (Bowles, 1997) that outline the procedure for calculating the bearing capacities of soils.

When the geotechnical engineer reports the allowable soil pressure, this value can either refer to the gross allowable pressure or to the net allowable pressure. If the bearing capacity of the soil is determined by the ultimate strength of the soil, then the gross pressure value is often reported. Otherwise when the bearing capacity is limited by settlement considerations, the net allowable pressure is reported (Bowles, 1997). The allowable gross pressure considers everything on top of the soil at the foundation-soil interface, including the backfill weight. On the other hand, the net allowable pressure refers to the pressure on top of the soil in excess of the overburden pressure. This means that the pressures due to the backfill and the volume of the soil occupied by the foundation are subtracted from the gross pressure. The reason that the net pressure is reported when settlements control is that the settlements are caused by pressures in excess of the steady state soil conditions that prevailed before excavations and construction of the foundation for over consolidated soils (Bowles, 1997).
3.4 Establishing the Geometry of the Foundation

The first step in the design of the foundation after obtaining the necessary information and loads (i.e. different load types and critical loading combinations) is to choose its preliminary dimensions. These dimensions are first chosen typically based on experience and similar projects. Gravity based foundations usually have a width of 15 to 20 meters (or a radius of 18 to 24 meters) and a thickness of 1.5 to 2.5 meters, and are typically embedded between 2.4 and 3.0 meters below finished grade (Goransson & Nordenmark, 2011).

The foundation contains a tapered base slab rigidly connected to a pedestal, to which the tower is anchored. The key parameters of the geometry of the foundation are shown in Figure 3.1. They are determined as a preliminary step and their adequacy is verified in the next stages of the design. These parameters include the radius or length of the pedestal (P), the height of the pedestal (Hp), the minimum and maximum thicknesses of the base slab (H1 and H2), the tapering slope, the distance from the edge of the pedestal to that of the foundation (Lb), and the total radius or length of the foundation (B). The levels of the soil backfill are also determined, which typically go up to almost the height of the pedestal, covering the entirety of the base slab and most of the pedestal. From these dimensions, and assuming that the foundation is circular, the self-weight of the foundation and the backfill can be determined from the following two equations (Svensson, 2010):

\[
W_f = \left[ \frac{\pi P^2}{4} (H_p + H_2 - H_1) + \frac{1}{2} \frac{\pi (B^2 - P^2)}{4} (H_2 - H_1) + \frac{\pi B^2}{4} H_1 \right] \gamma_c \tag{11}
\]

\[
W_b = \left[ \frac{1}{2} \frac{\pi (B^2 - P^2)}{4} (H_2 - H_1) + \frac{\pi (B^2 - P^2)}{4} H_p \right] \gamma_s \tag{12}
\]

where \(W_f\) and \(W_b\) refer to the weight of the foundation and backfill, respectively. \(\gamma_c\) refers to the unit weight of concrete, which is typically 24 kN/m³, while \(\gamma_s\) refers to that of the soil, which is around 19 kN/m³.
3.5 Verifying the Size of the Foundation from Bearing Capacity and Stability Calculations

Once the dimensions of the foundation are set, bearing capacity and stability calculations are performed to verify that these dimensions do not result in the overturning of the foundation and stressing the soil over its bearing capacity. Usually the main concern in the stability of the foundation is preventing it from overturning, which is catastrophic as shown in Figure 3.2. As such, this is the factor determining the size of the foundation, specifically the plan dimensions (Maunu, 2008).

The distribution used for the bearing pressure of the soil on the foundation can be different, depending on the amount of eccentricity (i.e. moment transferred from the tower) and the assumptions made, as shown in Figure 3.3. The distribution in Figure 3.3(a) refers to the case
where the amount of eccentricity of the vertical force (i.e. the moment) transferred from the tower is less than those in Figure 3.3(b) and Figure 3.3(c). Any of these distributions can be used in bearing capacity calculations, as long as consistency is maintained.

The free body diagrams of the three loading situations in Figure 3.3 are shown in Figure 3.4, Figure 3.5, and Figure 3.6. Note that these free body diagrams show only the resultant of most of the forces noted; the complete free body diagrams will be shown and discussed in the next section when performing the sectional analysis of the foundation. The equilibrium of the forces in the vertical direction as well as the equilibrium of the moments is used to ensure that no overturning occurs and that the maximum value of the soil contact pressure does not exceed the allowable limit, which is the soil bearing capacity (as determined by the geotechnical engineer) divided by some safety factor.
In these free body diagrams, $M_T$, $V_T$, and $H_T$ refer to the moment, vertical force, and horizontal force transferred from the tower to the foundation, respectively. These are usually supplied by the manufacturers of the tower, hub, and blades. Note that generally $H_T$ is assumed to act at the interface between the foundation and the soil, with a counteracting horizontal force from the soil denoted by $H_S$. This ensures that equilibrium at the horizontal direction is maintained. The moment transferred from the tower takes into account the eccentricities of both the horizontal and vertical forces, $H_T$ and $V_T$. $S_W$ and $F_B$ refer to the resultant of the self-weight of the foundation and the backfill, respectively. The soil contact pressure acts over a distance of $b$, with a maximum pressure value denoted by $q$. Finally, the width of the foundation is denoted by $B$. Different load factors can be applied to these loads depending on the limit state considered and the different codes employed. Depending on the soil contact pressure distribution chosen, slightly different calculations are
performed. The first check performed is to prevent the overturning of the foundation. This translates into ensuring that only a minor area of the foundation in contact with the soil is undergoing zero contact pressure and that the total overturning moments are sufficiently counteracted by the resisting moments. For ultimate limit state, the resulting soil contact pressure obtained from overturning moment calculations should cover at least half of the width of the foundation under the governing non-factored extreme loads (AWEA and ASCE, 2011). Under the serviceability limit state, no gapping should occur, therefore the resulting soil contact pressure should cover all of the foundation area under serviceability loading. This is to ensure that the foundation maintains adequate stiffness during operational loads and that the cyclic degradation of the foundation bearing materials is minimized (AWEA and ASCE, 2011). The second check performed is to ensure that the resulting maximum soil contact pressure value does not exceed the allowable limit.

3.5.1 Trapezoidal Soil Pressure Distribution

This soil pressure distribution, shown in Figure 3.3(a), represents the least critical case with regards to overturning since the eccentricity transferred from the tower is the lowest. The free body diagram of this case is shown in Figure 3.4. All the forces in this figure are known except for \( q_1 \) and \( q \), which represent the parameters of the soil contact pressure. These two unknowns are found by establishing two equilibrium equations: one asserting that the sum of vertical forces is equal to zero and the other being that the sum of the moments about the lower tip of the foundation should also equal to zero. If the values of \( q_1 \) and \( q \) are less than the allowable soil bearing pressure, then the chosen dimensions for the foundation are safe. Note that if \( q_1 \) is found to be negative, then the eccentricity imposed on the foundation is too critical to consider the trapezoidal soil pressure distribution. Instead, the triangular soil pressure distribution (Figure 3.3(c)) is more accurate to use, but the uniform distribution shown in Figure 3.3(b) can also be used, as specified in the DNV/RISO code.

3.5.2 Uniform Soil Pressure Distribution

The uniform soil pressure distribution, shown in Figure 3.3(b), is suggested by DNV/RISO. It represents a more critical case in terms of the eccentricity transferred from the tower and assumes that certain parts of the soil undergo plastification. In the free body diagram of Figure 3.5, all the parameters and loads are known except for \( q \) and \( b \), which represent the value of the soil contact
pressure and the width of the area over which it acts, respectively. The two equilibrium equations (vertical forces and moments) are used to find these unknowns. Note that the backfill forces are usually not included in the overturning calculations because they are considered to be favourable forces. It has to be verified that the value of \( b \) is at least 50\% of \( B \) and that \( q \) does not exceed the allowable bearing pressure [private correspondence with Hatch Ltd.]. If \( b \) is less than 50\% of \( B \), then the dimensions chosen for the foundations are unsatisfactory and, therefore, should be revised.

The DNV/RISO code utilizes an effective foundation area instead of using the whole area to calculate the soil bearing capacity and the demand on the foundation. Formulae for the foundation stiffness and the sliding resistance are also developed. The effective foundation area is established such that the load center of the vertical force transferred from the tower (acting at an eccentricity from the center of the foundation to produce the moment effect) coincides with its geometric center. The effective areas (\( A_{\text{eff}} \)) for square as well as circular and octagonal foundations are shown in Figure 3.7, Figure 3.8, and Figure 3.9.

![Figure 3.7: Effective area of a square foundation, Approach 1 (DNV/RISO, 2002)](image_url)
For square gravity base foundations, there are two approaches to calculating the effective foundation area, as shown in Figure 3.7 and Figure 3.8. The approach chosen for the bearing capacity calculations is the one that results in the most critical bearing demand. Figure 3.7
corresponds to Approach 1 for finding the effective area of a square foundation, with the following equations for its dimensions:

\[ b_{eff} = B - 2 \]  \hspace{1cm} (13)

\[ l_{eff} = B \]  \hspace{1cm} (14)

Figure 3.8 corresponds to Approach 2 for finding the effective area of a square foundation with the following equations:

\[ b_{eff} = l_{eff} = b - e\sqrt{2} \]  \hspace{1cm} (15)

Note that \( e \) refers to the eccentricity of the vertical load transferred from the tower.

For a circular foundation with a radius \( R \), its elliptical effective area (shown in Figure 3.9) is defined as:

\[ A_{eff} = 2 \left[ R2 \cos^{-1}\left(\frac{e}{R}\right) - e\sqrt{R^2 - e^2} \right] \]  \hspace{1cm} (16)

having the following major axes:

\[ b_e = 2(R - e) \]  \hspace{1cm} (17)

\[ l_e = 2R \sqrt{1 - \left(1 - \frac{b}{2R}\right)^2} \]  \hspace{1cm} (18)

The effective area formula for the circular foundation can be used for an octagonal foundation given that the radius used in the calculations is that of the polygon’s inscribed circle (DNV/RISO, 2002).

Since the tower transfers horizontal forces to the foundation, the sliding resistance of the soil has to be verified. This is achieved by satisfying the following criterion given in clause 8.2.1 in DNV/RISO:

\[ H < A_{eff} c + V \tan \phi \]  \hspace{1cm} (19)
in which H and V correspond to the horizontal and vertical forces transferred from the tower, respectively. Note that c and φ are soil-dependent parameters. In addition to this criterion, DNV/RISO requires that the horizontal load transferred from the tower (H) does not exceed 40% of the vertical load (V).

3.5.3 Triangular Soil Pressure

The free body diagram of the triangular soil pressure distribution is shown in Figure 3.6. Similar to the uniform distribution, all the values of the forces are known except for q and b, which represent the value of the soil pressure and the width over which it acts, respectively. Static equilibrium equations are used to find the value of these two parameters and verify that they are within the allowable limits (i.e. b being at least 50% of B and q being less than the allowable bearing pressure). This soil pressure distribution represents the case where the eccentricity imposed on the foundation is most critical.

3.6 Establishing Bending Moment and Shear Force Diagrams from Sectional Analysis of the Foundations

After the dimensions and the forces acting on the foundation are determined, sectional analysis is performed next. This consists of dividing the foundation into various sections and analyzing it section by section, starting from the left and going to the right. At each section, a vertical cut is made across the section, and the corresponding expressions for the shear force and bending moment are determined to establish the shear force and bending moment diagrams. These diagrams will identify the critical areas of the foundation experiencing the highest stresses and the values of such stresses. This will be the basis for the design of the reinforced concrete foundation. The sectional analysis is performed on one-meter wide strips extending from one edge of the foundation to the other and including the pedestal, as shown in Figure 3.10. Therefore, all the reported values for the shear force and bending moment will be per one meter width. Although the forces and moments from the tower are transferred to the foundation through the concentrated area of the pedestal, it is assumed that the internal forces are uniform throughout the whole width of the foundation (Landen & Lilljegren, 2012). A complete free body of the foundation is shown in Figure 3.11.
Figure 3.10: One meter wide strip for the sectional analysis of the foundation (Svensson, 2010)

Figure 3.11: Complete free body diagram of a gravity based foundation

$P$ refers to the width of the pedestal. $V_T$ is the vertical force transferred from the tower. The moment transferred from the tower to the foundation is modelled more accurately as a force couple rather than a concentrated moment (Eneland & Mallberg, 2013). This force couple is denoted by $F_T$ and $F_C$, having equal magnitudes but opposite directions. These forces are transferred from the tower to the pedestal of the foundation through the anchor bolts that run through the circumference of the tower-pedestal interface. A reasonable lever arm of the force couple is suggested to be two thirds of the pedestal width $(2/3 \, P)$. $q_{B1}$ and $q_{B2}$ refer to the forces due to the soil backfill. $S_{W1}$ and $S_{W2}$ refer to the parameters of the self-weight of the foundation. Note that the distribution of the self-weight has a shape similar to the tapered shape of the foundation. Finally, $q_1$ and $q_2$ refer to
the resulting soil pressure. Note that different soil pressure distributions can be used, as shown in Figure 3.3. The foundation is divided into many sections over which expressions for the shear force and bending moment diagrams are developed. Figure 3.12 shows the different sections used while Table 3.3 shows the dimensions of these sections.

![Diagram of sections considered for the sectional analysis of the turbine foundation](image)

Table 3.3: Section boundaries for the analysis of the turbine foundation

<table>
<thead>
<tr>
<th>Section Boundaries</th>
</tr>
</thead>
<tbody>
<tr>
<td>Section 1</td>
</tr>
<tr>
<td>Section 2</td>
</tr>
<tr>
<td>Section 3</td>
</tr>
<tr>
<td>Section 4</td>
</tr>
<tr>
<td>Section 5</td>
</tr>
<tr>
<td>Section 6</td>
</tr>
</tbody>
</table>

The typical shear force and bending moment diagrams for gravity based foundation are shown in Figure 3.13. The bending moment diagram is drawn on the compression side of the member: if the value of the bending moment is positive, the member is experiencing compression at the top and tension at the bottom. The critical areas with respect to both the bending moment and shear force are in the vicinity of the applied force couple. The force couple causes the sudden spike and drop in the shear force diagram at the pedestal area, which is in turn responsible for the sharp increase
in the bending moment diagram at the same location. The maximum shear force happens at the point of application of one of the force couples and extends to cover the lever arm between the couple. The maximum positive bending moment occurs where the compressive force of the couple acts, while the maximum negative bending moment occurs at the point of application of the tensile force. As the distance from the pedestal increases, both the bending moment and shear force diagrams gradually approach zero. From these diagrams, it is evident that the critical area of the foundation is that of the pedestal, specifically where the transfer of forces occurs between the tower and the foundation. The maximum shear and bending occur at this area and hence appropriate design procedures need to be implemented in order to ensure the smooth transfer of forces. Since the wind can act from any direction, the force couple from the tower is interchangeable in terms of the direction of the forces, which indicates that the diagrams developed can be rotated about the y-axis from the mid-span. The behaviour of the foundation can alternatively be seen as that of two cantilever beams joined at the pedestal.

The forces acting downward (the soil backfill, the self-weight, and the vertical force transferred from the tower ($V_T$)) decrease the maximum positive moment and increase the maximum negative moment.
Figure 3.13: Shear force and bending moment diagrams of the foundation

When considering the uniform soil pressure configuration shown in Figure 3.3(b), the resulting shear force and bending moment diagrams are similar to the ones in Figure 3.13 but not identical, as shown in Figure 3.14.
When the soil pressure distribution is uniform, the maximum absolute value of the bending moment seems to be coming from the negative moment rather than the positive moment. Comparing Figure 3.13 and Figure 3.14, it is clear that both soil pressure distributions give very similar values for the maximum moment, although one is positive and the other is negative. Both distributions also produce similar maximum values of the shear force. The critical areas of the foundation are the same in both distributions.

The sign (direction) of the maximum moment changes when different pressure distributions (and alternating force directions) are used. Therefore, the reinforcement configuration to be provided for the foundation to counter the stresses due to bending from the associated loads should be the
same in each direction and should be on both the top and the bottom, which is followed in common practice.

It is important to note that the different loads acting on the foundation can either increase or decrease the positive and the negative maximum bending moments. Therefore, suitable load combinations and factors need to be used when calculating the maximum positive and negative moments. The favourable loads in calculating the maximum negative moment are otherwise with regards to the maximum positive moment, and vice versa; different load combinations and soil pressure distributions need to be used to arrive the at the ultimate design moments and shear force.

### 3.7 Design of Flexural Reinforcement Using Equivalent Stress Block

The design of the flexural resistance treats the foundation as a set of two cantilever beams connected rigidly to the pedestal. If the clear span to depth ratio of these beams is at least 2, then the assumption of plane sections remain plane can be used, resulting in a linear strain profile over the depth of the cross section (CSA A23.3-04 Clause 10.7.1). The ultimate moment resistance of the cross section can then be found by calculating the moment arising from the compressive force due to the concrete in compression and the tensile force due to the reinforcement undergoing tension. The tensile contribution of cracked concrete (i.e. tension stiffening) is neglected. The compressive force due to the concrete is found by replacing the actual parabolic compressive stress distribution with an equivalent stress block, as shown in Figure 3.15. Figure 3.16 shows the process of finding the factored moment resistance of the section.

![Figure 3.15: Equivalent concrete stress block (Brzev & Pao, 2006)](image-url)
Figure 3.16: Finding the moment resistance of a cross section using equivalent stress block (Brzev & Pao, 2006)

As was seen previously, the maximum bending moment in the foundation happens at the point of application of the force couple close to the face of the pedestal. The critical section for the flexural design is therefore taken at the face of the pedestal (CSA A23.3-04 Clause 15.4.3) (Bowles, 1997). One-meter wide strips of the foundation extending from the face of the pedestal to the edge of the foundation are taken (shown in Figure 3.17) and, treated as a cantilever, designed for flexural resistance including the detailing (size, number, depth, clear cover, and yield strength) of the tensile reinforcement. Note that the twisting moments are added to the bending moments to account for the twisting action, as this procedure is commonly done in practice. Due to the changing direction of the load and the symmetry of the foundation, the reinforcement is placed both at the top and the bottom and in both directions of the foundation. For tapered foundations, the top reinforcement will follow the slope of the foundation; therefore, the tensile force due to the tapered reinforcement will be inclined. When reinforcement is present in the compression zone of the cross section, the inclusion of their contribution to the moment resistance is at the discretion of the designer.
The flexural reinforcement in circular foundations is radial, extending from the pedestal to the edge of the foundation as shown in Figure 3.18. The circumferential reinforcement seen in this figure is provided mainly for temperature and shrinkage control and to counteract any possible circumferential moment, which is usually minimal (Bowles, 1997). For a one-meter wide strip, the amount of the radial reinforcement decreases as the distance from the pedestal increases, therefore the amount of the reinforcement at several points along the length of the strip need to be checked to ensure that the moment resistance and the minimum reinforcement requirements are met. Square grid flexural reinforcement is provided for octagonal and square foundations, shown in Figure 3.19 and Figure 3.20. Some octagonal foundations contain radial reinforcement instead of the square grid reinforcement pattern.
3.8 Design of the Foundation against One-Way (Beam) Shear

The internal shear forces in the foundation make it prone to a brittle shear failure mainly characterized by diagonal cracking. This type of shear failure is known as one-way shear. The foundation needs to be analyzed (with the proposed dimensions) to ensure that sufficient shear capacity exists. If the concrete alone does not provide sufficient capacity (through aggregate interlock, uncracked compression zone, and dowel action by the longitudinal reinforcement) then either the depth of the foundation needs to be increased or transverse reinforcement needs to be supplied.

The maximum shear demand on the foundation happens between the force couple. In design, the critical section for the one-way shear is taken at a distance $dv$ from the face of the pedestal, where $dv$ is the effective shear depth and is calculated as either 90% of the depth of the longitudinal
reinforcement or 72% of the total height of the cross section of the beam, whichever is greater (CSA A23.3). The critical sections for one way shear for a square foundation are shown in Figure 3.21. Since the loading on the foundation can happen in any direction, the critical sections are shown for all the four directions. For a circular foundation containing a circular pedestal, the critical sections form a circle around the pedestal located at a distance of $dv$.

![Figure 3.21: Critical section for one way shear for a rectangular foundation](image)

The shear force at the critical section resulting from the critical load case is taken as the one-way factored shear demand, $V_f$, that the foundation must resist. The design for shear consists of ensuring that the shear resistance, $V_r$, of the foundation exceeds the factored shear demand. The shear resistance consists of the shear contribution by the concrete ($V_c$) and that of the transverse reinforcement ($V_s$). The one-way shear failure of the foundation consists of diagonal cracking originating from the pedestal and extending away from it, as shown in Figure 3.22.

![Figure 3.22: One-way shear crack](image)

The shear contribution of the concrete, $V_c$, takes into account the uncracked compressive strut of the concrete, concrete strength, dowel action of the longitudinal reinforcement, aggregate interlock, and the size effect (CSA A23.3 Clause 11.3.4):
\[ V_c = (\Phi_c)(\lambda)(\beta)(b_w)(d_v) \sqrt{f'_c} \]  
\[ (20) \]

where: $\Phi_c$ is a reduction factor reflecting the uncertainty in concrete

$b_w$ is the width of the web

$d_v$ is the effective shear depth

$f'_c$ is the compressive strength of concrete

$\lambda$ is a factor to account for the presence of light weight concrete, if any

$\beta$ is a factor accounting for the shear strength of the cracked concrete and takes into account parameters such as the crack spacing and the crack width (CSA A23.3 Clause 11.3.6)

If the concrete alone is not sufficient to resist the shear demand, then several options can be pursued. The depth of the foundation can be increased to give more shear resistance. Alternatively, a higher strength concrete can be used. However, for usually-deep foundations, the commonly used option is to provide transverse shear reinforcement. The additional shear resistance by this reinforcement is given in CSA A23.3 clause 11.3.5.1:

\[ V_s = \frac{(\Phi_s)(A_v)(f_y)(d_v)\cot\theta}{s} \]  
\[ (21) \]

where: $A_v$ is the cross sectional area of the transverse reinforcement

$f_y$ is the yield strength of the transverse reinforcement

$\theta$ is the inclination of the diagonal shear cracks relative to the longitudinal axis

$s$ is the spacing of the transverse reinforcement

The spacing of the transverse reinforcement is least at the critical sections and increases with the decrease of the shear demand away from the pedestal.
3.9 Design of the Foundation against Two-Way (Punching) Shear

The loads from the tower are transferred to the pedestal through the anchor ring. Then the pedestal distributes the load to the soil through the foundation slab. When the load is transferred from the pedestal to the slab, it is done so in the form of diagonal compression struts in the concrete. This transfer of concentrated forces puts the pedestal at the possibility of “punching” through the slab by diagonal shear cracks around the pedestal as shown in Figure 3.23. The punching shear is caused by the transfer of the concentrated loads from the pedestal, therefore it is only a matter of concern in the vicinity of the pedestal. Design codes (CSA A23.3 clause 13.3.3.1 and clause 15.5.1) specify that the critical section for two-way shear is to be taken at a section perpendicular to the plane of the slab a distance of d/2 from the face of the pedestal, where d is the distance from the extreme compression fiber to the centroid of the tension reinforcement. The total punching shear stress demand is found and then compared to the punching shear capacity given in the relevant codes. Referring to Figure 3.24, the plan view of the critical section for two-way shear forms a square at a distance of d/2 from the face of the pedestal. The cross-sectional area at the critical section is used to calculate the two-way shear demand. For a circular foundation and pedestal, the critical area forms a circle around the pedestal. The truncated shear failure plane is assumed to happen at the critical section, so the shear demand is calculated by either considering the downward shear force caused by the applied forces and moment at the center of the foundation inside the boundaries of the critical section (white area of Figure 3.24) or the upward shear force caused by the resulting soil pressure acting on the area outside of the perimeters of the critical section (the hatched area of Figure 3.24).

When using the central area to calculate the two-way shear demand on the foundation (Figure 3.24), the factored vertical downward force transferred from the tower needs to be taken into account fully. The moment transferred from the tower also increases the two-way shear demand. However, only part of the moment is transferred through shear to the foundation. The rest is transferred through flexure. For square and other symmetric equal-side foundations (e.g. circular and octagonal foundations), the code specifies that only about 40% of the applied moment will be transferred to the foundation by the eccentricity of shear, so only this portion is considered when calculating the shear demand (CSA A23.3 clause 13.3.5.3). According to CSA A23.3 clause 13.3.5.5, the factored shear stress, \( v_f \), is then calculated as:
\[ v_f = \frac{V_f}{b_0 d} + \left( \frac{Y_v M_{fx} e}{J_x} \right) + \left( \frac{Y_v M_{fy} e}{J_y} \right) \] (22)

where: 
- \( b_0 \) is the perimeter of the critical section
- \( d \) is the depth of the flexural reinforcement (Note that \( b_0 d \) refer to the shear area)
- \( Y_v \) is the portion of the applied moment that is transferred through shear eccentricity
- \( M_{fx} \) and \( M_{fy} \) refer to the applied moment in the x and y directions, respectively
- \( e \) is the distance from the centroid of the critical section for shear to the point where the shear stress is being calculated
- \( J_x \) and \( J_y \) both refer to the property of the critical section analogous to the polar moment of inertia

Figure 3.23: Punching shear failure of the foundation (McCormac & Brown, 2014)
The factored two-way shear stress without the presence of shear reinforcement should be at least equal to the two-way shear resistance of the concrete, $v_c$, given by CSA A23.3 Clause 13.3.4 and calculated as the minimum of the following three equations:

$$v_c = \left(1 + \frac{2}{\beta_c}\right)0.19\lambda\phi_c\sqrt{f'_c}$$  \hspace{1cm} (23)

$$v_c = \left(\frac{\alpha_c d}{b_0} + 0.19\right)\lambda\phi_c\sqrt{f'_c}$$  \hspace{1cm} (24)

$$v_c = 0.38\lambda\phi_c\sqrt{f'_c}$$  \hspace{1cm} (25)

where: $\Phi_c$ is the reduction factor for concrete

$\beta_c$ is the ratio of long side to short side of the column, concentrated load, or reaction area

$\alpha_c = 4$ for footing foundations containing one column (pedestal) at the center

$\lambda$ is the factor accounting for the presence of light-weight concrete, if any
When the factor governing the depth of the foundation is two-way shear (as is the case with many square foundations), the minimum allowable depth can be found by equating $v_c$ with $v_f$. The expression for $v_f$ would be expressed in terms of $d$ and the resulting equation will be quadratic in nature. Alternatively, if the preliminary dimensions are already laid out, then the equations for $v_c$ can be used to verify that the proposed depth is adequate. Otherwise, it should be increased or suitable shear reinforcement should be provided in the form of either stirrups or headed shear reinforcement. The maximum factored shear stress, $v_f$, acting on the slab is allowed to be about 35% more when headed shear reinforcement is used instead of stirrups (CSA A23.3 clause 13.3.8.2 and clause 13.3.9.2); some slipping might occur in the stirrups at the bends and some concrete in that area is subject to high localized compressive stresses that can cause it to crush, yielding lower resistance.

The headed shear reinforcement is a vertical shear stud that is mechanically anchored at each end by either a head or a plate as shown in Figure 3.25. It is used to intersect the cracks that originate from two-way shear and arrest their growth, preventing the brittle shear failure. The mechanical anchors used should enable the reinforcement to fully develop its yield strength (CSA A23.3 clause 13.3.8.1). The height of the stud should cover as much of the height of the cross section as possible, to reduce the chance of a shear crack passing around it. Such reinforcement is usually placed at concentric lines that are parallel to the column (or pedestal) cross section (clause 13.3.8.4).

![Figure 3.25: Headed shear reinforcement assembly (American Concrete Institute (ACI), 1999)](image-url)
When the concrete shear resistance, \( v_c \), measured at the critical section \( d/2 \) away from the pedestal face is less than the factored shear stress, \( v_f \), then shear reinforcement should be provided. The distance between the pedestal face and the first line of headed shear reinforcement should be a little less than the critical section distance. CSA A23.3 clause 13.3.8.6 specifies this distance to be 0.4d. The reinforcement should extend a safe distance from the critical section until the concrete resistance is sufficient by its own and the factored shear stress is not significant to warrant adding another line of headed reinforcement. Clause 13.3.7.4 specifies that the reinforcement should be extended at least a distance 2d from the face of the pedestal and should be extended to the section where the factored shear stress is not greater than \( 0.19 \lambda \phi_c \sqrt{f'_c} \). The contribution of the headed shear reinforcement to the two-way shear capacity, \( v_s \), is calculated according to clause 13.3.8.5:

\[
 v_s = \frac{\phi_s A_{vs} f_{yv}}{b_0 s} \tag{26}
\]

where: \( A_{vs} \) is the cross-sectional area of the headed reinforcement on a concentric line parallel to the parameter of the column

\( f_{yv} \) is the yield stress of the reinforcement

\( b_0 \) is the perimeter of the critical section

\( s \) is the spacing between concentric lines of headed reinforcement

It is important to note that in the zone reinforced by the headed shear reinforcement, the factored shear stress, \( v_f \), should not be taken more than \( 0.75 \lambda \phi_c \sqrt{f'_c} \) and the factored shear resistance of the concrete should be taken as \( 0.28 \lambda \phi_c \sqrt{f'_c} \) (clause 13.3.8.2 and clause 13.3.8.3). A typical arrangement of shear studs around a square column is shown in Figure 3.26.

![Figure 3.26: Typical arrangement of headed shear studs around a square column (American Concrete Institute (ACI), 1999)](image-url)
3.10 Tower-Foundation Connection

The critical forces arising from the wind turbine tower must be adequately transferred to the foundation in order to obtain the structural integrity of the structure. Therefore, critical attention needs to be paid to the design of the tower-foundation connection. Incorrect implementation of the design will create undesirable stress paths and concentrations that might eventually lead to the premature failure of the foundations, which would result in the collapse of the structure. A collapsed wind turbine due to the failure of the tower-foundation connection is shown in Figure 3.27. In this incident, the tower became detached from the foundation pedestal due to an inadequate tower-foundation connection detailing. The horizontal and vertical forces as well as the bending and torsional moments need to be effectively transferred to the foundation through the connection which forms the interface between the tower and the foundation at the pedestal. The connection, whose components are made of steel, has to possess sufficient yielding and ultimate strengths and resist pull-out and buckling from the applied loads. Effective anchorage into the pedestal is also required. Since fatigue loads will be transferred from the tower, the resistance of the connection to such loads needs to be verified. The components of the connection are obtained from companies specialized in its design and manufacturing.

Figure 3.27: Collapsed wind turbine tower due to detachment from foundation (Goodwin, 2014)

Although the tower-foundation connection comes in many forms and variations, it is usually either an embedded or a bolted connection. Embedded connections consist of a monolithic steel ring
(known as an insert ring) anchored inside the concrete foundation and extends above the pedestal a distance where its flange (known as a connection flange) is connected to the tower flange through prestressed bolts (Maunu, 2008). Bolted connections, on the other hand, consist of an embedded ring plate cast inside the pedestal connected to a base plate (or base flange) above the pedestal by stud bolts (AWEA and ASCE, 2011). A steel adaptor is mounted on top of the base plate, to which the tower is connected.

3.10.1 Insert Ring

A typical insert ring is shown in Figure 3.28. The bottom flange of the ring provides anchorage into the underlying pedestal and further distributes the tower forces instead of abruptly transferring them to the underlying concrete. Before casting of the foundation concrete, the ring is mounted with the reinforcement at the level desired. The main flexural reinforcement passes through the holes at the circumference of the ring (Hassanzadeh, 2012). The concrete covers all of the ring except a slight area below the top ring flange (shown in Figure 3.29), where the tower is connected. The bolt holes throughout the top flange of the ring enable the connection of the tower to the ring through bolts passing through the flange of the tower (shown in Figure 3.30) and that of the ring. Usually there is an empty space left between the concrete surface of the pedestal and the top flange of the ring to enable levelling of the tower, after which high compressive strength grout is injected into the space. Note that some insert rings contain two embedded flanges instead of one, providing further anchorage into concrete.

![Figure 3.28: Typical insert ring (Hassanzadeh, 2012)](image-url)
High compressive and tensile stresses will be induced in the concrete around the insert ring as a result of the transfer of high concentrated forces and moments. The downward vertical force from the tower will induce compressive stresses on the ring while the moment transferred will subject half of the ring to tensile stresses and the other half to compressive stresses. The possible resulting cracking patterns will be similar to those shown in Figure 3.31. Note that the anchor ring in this figure contains two flanges embedded in the concrete. As a result, one flange will apply the compressive forces (top flange) and the other will apply the tensile forces (bottom flange). If the anchor ring contained one embedded flange only, then both the compressive and tensile forces will be applied by the same flange.
To guard the pedestal against failures caused by these cracking patterns, anchor reinforcement is provided around the insert ring (Svensson, 2010). The main task of such reinforcement is to carry the excess tensile stress after the surrounding concrete has cracked. To be conservative, the contribution of the anchor reinforcement to compressive resistance should be ignored (Svensson, 2010). Such reinforcement will be placed as U-hoops around the insert ring as shown in Figure 3.32. Its role in resisting the forces transferred by the anchor ring is shown in Figure 3.33.
The stress exerted on the bottom embedded flange of the insert ring is calculated from the vertical force and moment transferred from the tower, assuming a linear elastic behaviour in the transfer of stresses from the steel to the concrete (Landen & Lilljegren, 2012):

$$\sigma = \frac{V_T}{A_f} \pm \frac{M}{S} = \frac{V_T}{2 \pi r d} \pm \frac{M}{\pi r^2 d}$$  \hspace{1cm} (27)

where: $V_T$ is the downward vertical force transferred from the tower

$A_f$ is the area of the embedded flange of the insert ring on which the force is acting

$S$ is the bending resistance of the flange (treated as a thin cylinder)

$r$ is the average radius of the insert ring

$d$ is the width of the flange (i.e. the difference between outer and inner radii)

The required amount of anchor reinforcement can be found from the stress value by multiplying this value by the area of the embedded flange ($A_f$) to find the total force exerted (Svensson, 2010). Then, this force can be multiplied by the desired safety factor and divided by the yield stress of the reinforcement to find the total area of the reinforcement needed.
3.10.2 Anchor Bolt Cage

An anchor bolt cage consists of a number of bolts (typically in the order of 100 bolts) that are connected from the bottom to a steel ring plate embedded in the foundation concrete. They are extended a certain distance above the level of the pedestal base to be attached to the tower base flange, which is welded to the tower base. A typical anchor bolt cage with the bottom steel ring is shown in Figure 3.34. The bolt cage is set in place with the reinforcement prior to casting of the foundation concrete. At this point, the depth of the steel ring placed under the pedestal surface and the length of the bolts above it are set. It is important that enough threaded distance of the bolts above the pedestal is left to allow the consequent attachment of the tower and the post-tensioning of the bolts (AWEA and ASCE, 2011).

![Figure 3.34: Typical anchor bolt cage (Hassanzadeh, 2012)](image)

After the foundation concrete is cast and cured, the tower becomes ready for attachment to the foundation. The tower is then connected to the foundation by inserting the bolts into the bolt holes in the tower base flange. Some empty space is left between the surface of the pedestal and the bottom of the tower base flange to enable levelling of the tower (as shown in Figure 3.35), which is extremely important to ensure optimum performance of the turbine and avoid undesired stresses (AWEA and ASCE, 2011). After the tower is levelled, washers and nuts are used to fasten it to the bolts. Post-tensioning of the bolts is also performed to ensure that no tensile stresses are exerted by the tower on the foundation (to prevent pullout failure) and provide additional anchorage of the tower. The empty space between the tower flange and the pedestal surface is then grouted with high compressive strength non-shrink grout. Besides enabling the levelling of the tower, grouting
ensures a smooth transition of stresses between the steel of the tower and the concrete of the pedestal and increases the fatigue resistance of the assembly (AWEA and ASCE, 2011).

![Figure 3.35: Space left between tower base flange and pedestal surface (Hassanzadeh, 2012)](image1)

The bolts in the cage are encased with hollow tubes or sleeves, as shown in Figure 3.36 and usually made from polyvinyl chloride (PVC), that prevent direct contact of the bolts with the poured concrete. This loss of contact provides the bolts with excess space to enable levelling of the tower and post-tensioning after the concrete has been poured (US Patent No. US8272181 B2, 2012). Then, grout is inserted into the sleeves to provide bond between the bolts and the surrounding concrete. The final configuration of the tower-foundation connection using an anchor bolt cage will be as shown in Figure 3.37. Another variation of this connection is to have, in addition to steel ring plate embedded in the concrete, another plate at the surface of the pedestal, to which a steel adaptor is connected, as shown in Figure 3.38. The tower is attached to this adaptor through bolts as well.

![Figure 3.36: Bolts in PVC sleeves or grout holes (Hassanzadeh, 2012)](image2)
The number of anchor bolt circles and the number of bolts in each circle are determined by experience, similar projects, or the recommendation of the wind tower generator manufacturer. The diameter of each anchor bolt circle, which passes through the center of the foundation pedestal, is also determined. After that, the anchor bolt post-tension force is determined such that under operating conditions no part of the anchor bolt cage is subject to tensile stresses resulting from the moment and the forces transferred from the tower [private correspondence with Hatch Ltd.]. This condition ensures that the possibility of tower pull-out at the connection, as well as the pull-out of the concrete at the pedestal, is minimized. Furthermore, overturning failure of the wind turbine is further mitigated. Because of the moment transferred from the tower to the foundation, half of the
Anchor bolts will be under tension due to the applied moment. The tensile force in each anchor bolt, $T_r$, due to the unfactored loads and moments transferred from the tower is calculated as:

$$T_r = -\frac{V}{N_c N_{bc}} + \frac{M}{D N_c N_{bc} / 4}$$

where: $V$ is the unfactored vertical force transferred from the tower

$M$ is the unfactored moment transferred from the tower

$N_c$ is the number of anchor bolt circles

$N_{bc}$ is the number of anchor bolts in each bolt circle

$D$ is the average diameter of the anchor bolt cage

The prescribed post-tensioning force for each anchor bolt takes into consideration the losses (assumed as a percentage of the load) as well as the tolerances (upward and downward) that may occur during installation [Private Correspondence with Hatch Ltd.]. It has to be ensured that this prescribed force is not less than the minimum post-tensioning load prescribed by the wind turbine generator manufacturer. The prescribed post-tensioning force is then calculated as follows:

$$PT = \max \left[ P_p, T_r (1 + PT_{loss}) + P_{tot} \right]$$

where: $PT$ is the prescribed post-tensioning force for each anchor bolt

$P_p$ is the minimum post-tensioning force specified by the manufacturer

$PT_{loss}$ is the assumed losses as a percentage of the load

$P_{tot}$ is the downward tolerance on the post-tensioning force

The capacity of each anchor bolt is then checked and verified against the prescribed post-tensioning force plus the specified upward force tolerance. The capacity of the steel ring plate embedded in the concrete is also checked. An outline of this plate with key parameters is shown in Figure 3.39. $R_{in}$ and $R_{out}$ refer to the outer and inner radii of the plate, respectively. $W$ and $t$ refer to the width and thickness of the plate. The distance between the centerlines of adjacent bolt holes,
whose diameters are $D_{bh}$, is denoted by $d_h$ while the distance between the centerline of the last bolt hole and the edge of the plate is denoted by $d_e$. In addition, $\theta_s$ refers to the angle of the sector of the plate corresponding to the tributary area of the set of the bolt holes along the same radius (two in the case of Figure 3.39). The arc length of this area is $S_s$. The area of each sector of the plate is found by the following formula:

$$A_s = \frac{\theta_s}{2} \cdot (R_{out}^2 - R_{in}^2) - \left(\frac{D_{bh}}{2}\right)^2 \cdot \pi \cdot 2$$ \hspace{1cm} (30)

The stress applied on the plate is then found by considering the stress resulting from either the factored governing design load case or the factored anchor bolt post-tensioning force plus a percentage of the external load, whichever is larger [private correspondence with Hatch Ltd.]. The stress, denoted by $\sigma_s$, is found by dividing the corresponding forces and moments by the area of the sector of the plate, $A_s$. Consequently, the capacity of the ring plate is verified in both the circumferential and radial directions.

In the circumferential direction, the steel ring plate is treated as a fixed beam subject to the distributed stress, $\sigma_s$. The maximum resulting bending moment on the ring plate is calculated as:

$$M = \frac{wL^2}{12}$$ \hspace{1cm} (31)
where: \( w \) = distributed load as a result of the stress = \( \sigma_s \ast W \)

\[ L = \text{the length of the plate sector net of the bolt hole diameter} = S_s - D_{bh} \]

The section modulus, \( S \), that is used to calculate the final stress value is calculated as:

\[
S = \frac{l}{y} = \frac{1}{12} (W - 2D_{bh}) \ast \frac{t^3}{t/2} \]

(32)

The resulting stress on the ring plate is calculated (\( \sigma = M/S \)) and compared to the capacity of the steel ring plate. Similar calculations are performed in the radial direction to check the capacity of the plate. In this case, the plate is also treated as a fixed beam and is divided into three sections: one section between the bolt holes, and two sections from each bolt hole to the edge [Private Correspondence with Hatch Ltd.].

### 3.11 Pedestal Detailing and Design

Attaching the steel tower directly to the footing embedded underground will make it susceptible to chemical attack and possible corrosion from the soil. The pedestal protects the steel tower by forming an interface between the footing and the tower. Parts of the pedestal will be underground while the rest will extend above the ground surface. The presence of carefully compacted backfill around the part of the pedestal underground will provide it with lateral bracing. The remaining unsupported length of the pedestal should not be more than three times the least lateral dimension of the pedestal to avoid possible buckling (ACI 318 clause 7.3). The design of the required reinforcement for the pedestal follows the same approach as that of reinforced concrete columns. The pedestals should contain sufficient bending and axial capacity to transmit the loads from the tower to the foundation footing. Resistance to pull-out and bursting failure should also be ensured.

Longitudinal bars should be provided as well as ties or spirals (i.e. transverse reinforcement) to confine the longitudinal bars and the concrete, providing more capacity and ductility. The use of transverse reinforcement will also ensure that the contribution of the longitudinal bars in resisting compressive loads can be counted on. In compression members (i.e. columns and pedestals), CSA A23.3 requires the use of transverse reinforcement whenever longitudinal reinforcement is used (clause 7.6.5.1). Such transverse reinforcement must be distributed within 125 mm of the top of the pedestal (clause 7.6.5.8) and shall be located at a maximum of one-half the tie spacing above
the footing (clause 7.6.5.3). After the dimensions and reinforcement layout are established, an interaction diagram for the pedestal can be established by means of hand calculations or software (such as Response 2000) and compared against the transferred axial load and moment from the turbine tower.

The number of longitudinal bars in the pedestal will depend on the desired capacity. However, it is common design practice, especially in circular pedestals, to provide one inner longitudinal bar circle inside the anchor bolt circle and another circle outside of it [private correspondence with Hatch Ltd.]. Ties for each longitudinal bar circle are provided as shown in Figure 3.40, paying attention to the minimum required tie spacing (clause 7.6.5.2). This will satisfy the requirement of CSA A23.3 that the anchor bolts in the pedestal be enclosed by transverse reinforcement that also surrounds at least four vertical bars of the pedestal (clause 7.6.5.8). Note that in general cases the code requires a minimum longitudinal reinforcement ratio of 1% of the column gross cross-sectional area (clause 10.9.1) and a maximum ratio of 8% (clause 10.9.2).

![Common layout for pedestal reinforcement in the presence of anchor bolt cage](image)

Figure 3.40: Common layout for pedestal reinforcement in the presence of anchor bolt cage

### 3.12 Transfer of Stresses from Pedestal to Footing

The forces exerted by the turbine are transmitted to the pedestal through the anchor bolt cage and then to the underlying footing. Compressive stresses are transferred to the footing by the pedestal bearing on it. The point of contact between the footing and the pedestal is equal to the latter’s cross-sectional area. The bearing capacity of the bottom part of the pedestal, where it touches the footing, needs to be checked against the transmitted compressive stresses. The bearing capacity of the top part of the footing touching the pedestal should also be checked against the compressive
stresses applied. Although the area of this part is equal to the cross-sectional area of the pedestal, the surrounding concrete of the footing provides it with lateral confinement, which increases its bearing capacity (McCormac & Brown, 2014). CSA A23.3 accounts for this increase by multiplying the concrete bearing capacity calculated for this area by $\sqrt{A_2/A_1}$, in which $A_1$ is the cross-sectional area of the pedestal and $A_2$ is the area shown in Figure 3.41. The factored bearing capacity of concrete is calculated as $0.85\Phi_c f'_c A_1$ according to CSA A23.3 clause 10.8.1, in which $\Phi_c$ is the resistance factor used for concrete.

\[
A_2 = (b + 4d)(c + 4d) \\
A_1 = b \times c
\]

Figure 3.41: The bearing capacity areas used in calculations (Bowles, 1997)

If the bearing resistance of the concrete in either the pedestal or the footing is less than the factored compressive stresses, dowels extending to the footing should be added to transmit the compressive stresses, as shown in Figure 3.42. The dependence on the pedestal vertical reinforcement to transmit the compressive stresses to the footing is not correct because these bars do not have adequate development length at the bottom of the column to allow the transmission of such stresses (McCormac & Brown, 2014). A common design practice is to splice the dowels with the pedestal longitudinal reinforcement to an adequate length above the pedestal-footing interface and bend them outward near the bottom of the footing to allow them to rest on the footing flexural reinforcement, as shown in Figure 3.42 (McCormac & Brown, 2014). The purpose is to allow these dowels to fully develop the required compressive stresses to be transferred to avoid bursting, crushing, and spalling in the bearing zones. The development length of the dowels should also be checked for developing tensile stresses, since these dowels are also used to ensure a proper transfer of tensile stresses (uplifts) and moments to the footing without causing splitting and pullout failures (McCormac & Brown, 2014).
Figure 3.42: Dowels extending from the pedestal to the footing (McCormac & Brown, 2014)

3.13 Using the Strut-and-Tie Method to Design the Footing

The strut-and-tie concept is a method used to model the flow of internal stresses and forces in a structure to reflect one possible path of these stresses after cracking of the concrete occurs, which disrupts the elastic stress field and causes a reorientation of the stresses and internal forces (Wight & MacGregor, 2012). The model embodies a system of internal forces that are in equilibrium with the externally applied forces, and the forces in each system of the model do not exceed the member design force. Therefore, it provides a lower bound on the strength of the actual structure, in which the design forces are lower than the collapse load of the structure (Wight & MacGregor, 2012). The model takes care of internal moment, shear forces, and axial forces simultaneously so that there is no need to design for each component separately. It consists of three components: struts, ties, and nodal zones. Struts represent the compression field of the concrete, which acts parallel to the struts. The ties represent the layers of the reinforcement in the same direction. Finally, the nodal zones represent the concrete surrounding the nodes, which are the points at which the forces in the model meet. Failure in the strut-and-tie model can be due to crushing of the strut, yielding of the tie, crushing of the nodal zones, anchorage failure of the ties, or diagonal cracking of the concrete (Wight & MacGregor, 2012). A strut-and-tie model of a simply supported beam subject to a point load at mid-span is shown in Figure 2.46. The rules and equations for establishing the model are given in CSA A23.3-14 clause 11.4. This model is especially useful in designing deep beams, whose shear span to depth ratio are less than 2.5.

Design using strut-and-tie modelling basically consists of establishing a truss capable of transferring the applied forces to the supports and making sure each component of the truss contains sufficient capacity. The compressive capacity of the nodal zones and the struts as well as
the capacity of the ties are checked. For each loading configuration, there are several possible strut-and-tie models. However, those models that provide the most direct and shortest path of the applied loads to the supports are the most efficient (CSA A23.3-04 N11.4.1). A proposed strut-and-tie model for the gravity wind turbine foundations is shown in Figure 3.43, in which the blue and black lines represent the struts and ties, respectively. The self-weight of the foundation is represented by two concentrated forces (SW/2 in the figure) acting at quarter-distance from each side. The resultant of the soil reaction, $R_{\text{soil}}$, is established assuming a uniform soil reaction as shown in Figure 3.3(b). Finally, $F_T$ and $F_C$ represent the force couple in the foundation, which represents the moment as well as the vertical forces transferred from the tower. An alternative strut-and-tie model for the foundation, given by Goransson and Nordenmark (2011), is shown in Figure 3.44. Landen and Lilljegren (2012) argue that a 3D strut-and-tie model for the foundation is more accurate in describing the real behaviour, since the pedestal is concentrated only at the center of the foundation. As such, they propose several 3D models for the design. Despite the applicability of strut-and-tie models in the design of the foundation, the current design practice is to use the traditional beam theory in its design (Landen & Lilljegren, 2012) [Private correspondence with Hatch Ltd.].

![Figure 3.43: Proposed strut-and-tie model for the foundation](image1)

![Figure 3.44: Strut-and-tie model of the foundation proposed by Goransson and Nordenmark (2011)](image2)
3.14 Development Length and Anchorage of the Reinforcement

The superior performance of reinforced concrete is greatly attributed to the composite action between the concrete and the steel reinforcement. In order for this composite action to occur, effective bond should exist between the concrete and steel. After concrete cracks, the steel reinforcement should be able to reach its full yield strength in the critical section(s) before the ultimate bond strength is reached (Brzev & Pao, 2006). This translates to ensuring that the steel extends beyond the critical section from both sides a distance that is at least equal to its development length, which is the minimum length of the steel that would enable the bar to yield before bond failure occurs.

CSA A23.3-04 in clause 12.1.1 states that “the calculated tension or compression in reinforcement at each section of reinforced concrete members shall be developed on each side of that section by embedment length, hook, or mechanical device, or by a combination thereof. Hooks may be used in developing bars in tension only.” In tension, the minimum development length should be 300 mm (clause 12.2.1). The general development length equation for bars in tension is given as follows:

\[
l_d = 1.15 \frac{k_1 k_2 k_3 k_4}{(d_{cs} + K_{tr})} \frac{f_y}{\sqrt{f_c'}} A_b
\]

(33)

where: \(l_d\) is the development length of the bar in tension

- \(k_1, k_2, k_3, k_4\) are bar location, coating, concrete density, and bar size factors, respectively
- \(d_{cs}\) is the distance between adjacent bars or concrete surface to the bar, whichever is smaller
- \(K_{tr}\) is a transverse reinforcement index
- \(f_y\) is the tensile yield strength of the bar being developed
- \(f_c'\) is the compressive strength of the concrete
- \(A_b\) is the area of an individual bar

The values for the different variables in the equation are found in CSA A23.3. For the wind turbine foundation, the critical moment section lies at the face of the pedestal. Hence, enough development
length from the face of the pedestal, in each direction, must be ensured. If the distance from the face of the pedestal to the end of the footing (where the reinforcement terminates) is less than the bars’ development length, then other means of anchorage, such as hooks or mechanical devices, must be used. The Euro Code (EN 1992.1.1), in clause 9.8.2.2, requires that every tensile force along the reinforcement of the footing be developed (either by embedment, hooks, or mechanical anchoring) in the remaining length of the reinforcement to the nearest footing edge. Referring to Figure 3.45, F_s is the tensile force in the reinforcement at a distance of x from the edge of the footing. The available anchorage length for this force is denoted by l_b. If the development length of the bar for F_s is less than l_b, then other means of anchorage must be employed (European Committee for Standardization, 2004).

For bars used to resist compressive stresses, the minimum development length that can be used is 200 mm (clause 12.3.1); the basic development length equation is given by CSA A23.3 clause 12.3.2 as follows:

\[ l_{db} = \frac{0.24d_b f_y}{\sqrt{f'_c}} \leq 0.044d_b f_y \]  

(34)

where: \( l_{db} \) is the development length of the bar in compression
\( d_b \) is the diameter of the bar

\( f_y \) is the compressive yield strength of the bar

\( f'_c \) is the compressive strength of the concrete

When the moment demand across the foundation reduces sharply in the areas further from the pedestal, a choice can be made to eliminate some reinforcing bars after a certain point. Theoretically, this point is when the moment demand falls to a point rendering some amount of the reinforcement unnecessary, which is known as the theoretical cut-off point. Good practice is to terminate a certain amount of the reinforcement at some point after the theoretical cut-off point to take into account the undesirable additional demands imposed by shear forces on the reinforcement, transfer of stresses from the larger number of bars to the smaller number of bars at the cut-off point, and allowances to account for errors and variations during construction (Brzev & Pao, 2006). CSA A23.3-04 clause 11.3.9.2 takes account of the effects of the shear and axial forces on the amount of the tensile longitudinal reinforcement required by specifying that the factored resistance of the reinforcement at all sections shall not exceed \( F_{lt} \) given as follows:

\[
F_{lt} = \frac{M_f}{d_v} + 0.5 N_f + (V_f - V_s - V_p) \cot \theta
\]  

(35)

where: \( M_f \) is the factored moment

\( N_f \) is the factored axial force

\( V_f \) is the factored shear

\( V_s \) is the shear resistance provided by the shear reinforcement

\( V_p \) is the shear resistance provided by the prestressed tendons, if any

\( d_v \) is the effective shear depth (greater of 0.9d or 0.72h)

\( d \) is the depth of the tensile reinforcement

\( h \) is the total height of the cross section
θ is the inclination of the diagonal compressive stresses to the longitudinal axis of the member.

The required resistance of the compression reinforcement, $F_{c}$, is described by the same equation but with the expression containing $M_f$ being negative. When splicing of the reinforcement is necessary in the foundation (e.g. the tensile reinforcement requires joining more than one bar to reach the desired length of reinforcement), precautions need to be taken to ensure smooth transfer of stresses across the bars. Whether using lap or welded splices, the requirements of clause 12.4 of CSA A23.3-04 have to be followed. Finally, the anchorage and detailing requirements of the transverse reinforcement, to ensure effective shear resistance, given in clause 12.13 must be followed as well.

3.15 Serviceability Limit State Checks

In addition to having sufficient strength to resist the factored applied loads, the wind turbine foundation should perform satisfactorily during its service life. According to CSA A23.3-04 (clause 8.1.4), three important criteria should be considered in the serviceability limit state: deflections, vibrations, and cracking.

3.15.1 Deflections

In the serviceability limit state, deflections are a major consideration in the design of floor slabs in buildings. Often times the limits set by the code on the permissible deflection control the thickness of the slabs, since the goal is to construct stiff-enough slabs that do not disrupt the occupants of the buildings by “waving” excessively, rendering the building unfit for occupancy and use. However, in the design of wind turbine foundations, such a concern is not present since there are no occupants. Rather, the concern in the excessive deflection of the foundation against the soil is to disrupt its stability and cause unfavourable second-order effects, possibly leading to the collapse of the wind turbine tower. The allowable soil bearing capacity, supplied by the geotechnical engineers, takes into account the unfavourable deflections and settlements. Since the foundation rests on the soil across all of its bottom surface, the differential deflection between the foundation and the soil is not an issue; the serviceability limit state load combinations are used to ensure that no gapping occurs between the foundation and the soil.
3.15.2 Vibrations

Similar to deflections, excessive vibrations might result in the formation of unanticipated second-order demands on the foundation. In addition, significant swaying of the wind turbine (along with the foundation) might result if the resulting vibration is close to the natural period of the structure, inducing excessive demands on the foundation with regards to strength and stability. Since any possible vibration comes from the loading transferred from the tower, the tower designer ensures that vibrations are kept within acceptable limits. As such, limiting vibrations is not an issue for the foundation designer. However, if doubt arises in the ability of the tower, in a specific design configuration, to control vibrations, then a dynamic analysis of the finite element model of the wind turbine structure would assess the specific case.

3.15.3 Cracking

Cracks in the reinforced concrete, especially those in the vicinity of the reinforcement, provide a pathway for chloride, moisture, oxygen, and carbon dioxide to reach the reinforcement and facilitate the electrochemical reaction resulting in the corrosion of the reinforcement. All design codes try to eliminate or minimize corrosion of the reinforcement by requiring a certain cover thickness, controlling the resulting cracks, or a combination of both.

There are several equations used to determine the maximum crack width resulting from a given set of applied loads, cross-sectional configuration, and reinforcement layout. AASHTO and CSA base their crack control on the simple empirical equation given by Gergely and Lutz (1968) for the maximum crack width:

\[ w_{\text{max}} = 2.2\beta\varepsilon_{\text{scr}} \sqrt{d_cA} \]  \hspace{1cm} (36)

where: \( w_{\text{max}} \) is the maximum crack width

\( \beta \) is a factor to account for strain gradient (ratio of the distance from the extreme tension fiber to the neutral axis to the distance from the tension steel to the neutral axis, taken as 1.0 for uniform strains)

\( \varepsilon_{\text{scr}} \) is the strain in the tension steel at the crack location

\( d_c \) is the distance from the extreme tension fiber to the center of the closest tension bar layer
A is the effective area of concrete surrounding each bar (refer to Figure 3.46)

![Figure 3.46: Crack control parameters in Gergely and Lutz (1968) equation (Brzev & Pao, 2006)](image)

The crack width calculated by Gergely and Lutz (1968) refers to the most probable width exceeding 90% of the widths of other cracks. CSA A23.3 and AASHTO combine $w_{max}$ and $\beta$ into a single crack control parameter, $z$, in the following manner:

$$z = f_s \sqrt[3]{d_c A}$$  \hspace{1cm} (37)

in which $f_s$ refers to the steel in the stress calculated at the maximum service load assuming an elastic stress distribution and cracked section properties. CSA A23.3 allows the approximation of $f_s$ as 60% of the yield stress of the reinforcement. According to CSA A23.3 clause 10.6.1, the value of $z$ should not exceed 30000 N/mm for interior exposure and 25000 N/mm for exterior exposure.

The Euro Code EN 1992-1-2 (European Committee for Standardization, 2004), in clause 7.3.4, provides an alternative formula for calculating the characteristic crack width which must be smaller than a prescribed maximum crack width:

$$w_k = s_{r, max} (\epsilon_{sm} - \epsilon_{cm}) \leq w_{k, max}$$ \hspace{1cm} (38)

where: $w_k$ is the characteristic crack width

$s_{r, max}$ is the maximum crack spacing

$\epsilon_{sm}$ is the mean reinforcement strain

$\epsilon_{cm}$ is the mean concrete strain between cracks (considering tension stiffening)
In common situations, where the centers of the tensile reinforcement are close together, the maximum crack spacing ($s_{r,\text{max}}$) can be calculated through the following equation:

$$s_{r,\text{max}} = k_3 c + k_1 k_2 \phi / \rho_{p,\text{eff}}$$

(39)

where: $k_1$ is the coefficient reflecting the bond characteristics

$k_2$ is the coefficient corresponding to the strain distribution

$k_3, k_4$ are values specific for each country

$\phi$ is the bar diameter

$\rho_{p,\text{eff}}$ is the ratio of the reinforcement in the effective concrete area

ACI suggests the maximum crack width values shown in Table 3.4 for different exposure conditions (ACI Committee 224, 2001). An essential goal of the crack control provision is to avert the collapse of the structure by the propagation of a single crack. Tensile reinforcement is provided to arrest crack growth. By providing the minimum required tensile reinforcement, sudden collapse by such crack growth is prevented. The desired ultimate pattern for the structure is to show several narrow cracks prior to failure. For deep structural members (those exceeding 750 mm in height according to CSA A23.3-04 Clause 10.6.2), skin reinforcement on the exposed side faces of the cross section is required to provide adequate crack control.

Table 3.4: Crack width values suggested by ACI Committee 224. Taken from ACI 224R-01

<table>
<thead>
<tr>
<th>Exposure Conditions</th>
<th>Crack Width</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dry air or protective membrane</td>
<td>0.016</td>
</tr>
<tr>
<td>Humidity, moist air, soil</td>
<td>0.012</td>
</tr>
<tr>
<td>Deicing chemicals</td>
<td>0.007</td>
</tr>
<tr>
<td>Seawater and seawater spray, wetting and drying</td>
<td>0.006</td>
</tr>
<tr>
<td>Water-retaining structures</td>
<td>0.004</td>
</tr>
</tbody>
</table>

In addition to structural distress, cracking in concrete can happen due to shrinkage of concrete, corrosion of reinforcement, cycles of freeze and thaw, sulphate attack, and other environmental demands on the structure. Therefore, adequate measures need to be taken to ensure the durability of the structure.
3.16 Durability Considerations of the Foundation

The environment’s demand on the foundation can significantly shorten its design life if proper measurements are not taken. Degradation mechanisms specific to foundations include corrosion, sulphate attack, freeze and thaw cycles, and shrinkage of the concrete. In general, using high quality concrete with low porosity and correct mixing, placing, and curing procedure will help with almost all of these mechanisms.

Corrosion is an electrochemical process that involves the oxidation of iron into ferrous ions in one part of the reinforcement, called the anode, and the formation of hydroxyl ions in another part, called the cathode, by consuming oxygen, moisture, and electrons (Collins & Mitchell, 1997). The high alkalinity of the cement paste forms a protective film around the reinforcement, which can be destroyed by the chloride ions. These ions can come from several sources such as salt water, deicing chemicals, and some admixtures. The passivity of the reinforcement can also be destroyed by carbon dioxide, facilitating the corrosion reaction. The products of corrosion induce an expansive stress on the surrounding concrete, causing spalling of the cover and possible splitting cracks running parallel to the reinforcement, which can affect its bond to the concrete. Ultimately, corrosion causes a reduction in the cross-sectional area of the reinforcement, significantly reducing its structural capacity. Depending on the severity of the environment, many measures can be taken to alleviate the rate of corrosion. These include using high quality concrete, protecting the foundation from moisture exposure, using epoxy-coated reinforcement, using cathodic protection in severe environments, limiting the crack widths to the acceptable limits, and refraining from using admixtures containing chlorides.

Sulphate attack on the concrete can happen when the surrounding soil and groundwater contain soluble sulphates (those of calcium, magnesium, and sodium) (Emmons, 1993). These sulphate react with the hydrated cement paste to form the expansive products of gypsum and ettringite, pressurizing the concrete. Scaling of the surface will follow, eventually leading to mass deterioration and disintegration of the concrete (Emmons, 1993). Protection against sulphate attack consists of using low water-cement ratio, air-entrained concrete, and sulphate-resisting cement.

Freeze and thaw cycles can cause unanticipated cracking in the concrete, especially in the presence of excessive moisture. Guarding against these cycles consist of using high quality and air-entrained concrete.
Shrinkage of the concrete causes it to crack, distressing the structure and providing a pathway for oxygen and carbon dioxide to the reinforcement. When concrete loses moisture, its volume decreases. This causes the structure to shrink, which will result in cracks if the shrinkage is resisted by restraining the structure (Collins & Mitchell, 1997). Such shrinkage can be greatly alleviated by the use of low shrinkage cement.

3.17 Fatigue Verification of the Foundation

The norm for fatigue resistance design has always been the consideration of independent material damage for the different materials constituting the structure (see EN 1992-1-1 6.8 and fib Model Code 2010 7.4.1). For a reinforced concrete beam containing shear reinforcement, this verification would consist of ensuring sufficient fatigue resistance for concrete in compression, longitudinal steel in tension, and shear reinforcement in tension as well. The fatigue resistance of concrete in tension is neglected because the concrete is often cracked at the loading stage(s) considered. For each material, fatigue verification is essentially an inequality: ensuring that the fatigue damage caused by the fatigue loading on the structure does not exceed a certain limit. Design codes use two methods to check for fatigue damage: damage equivalent stress range, and Miner’s Rule.

3.17.1 Damage Equivalent Stress Range

The first method consists of transforming the fatigue loading spectrum, which contains many load cycles at various stress ranges, into an equivalent \( N^* \) number of cycles (usually 1 million cycles for common reinforcing bars and concrete) and a corresponding damage equivalent stress range. The method of calculating the equivalent stress range depends on many factors including, but not limited to, the use of the structure, the types of loading imposed, and its service life (fib Model Code 2010). After calculating the damage equivalent stress range (which represents the fatigue demand on the structure), it is compared with the fatigue strength of the material corresponding to the same number of cycles, \( N^* \). This fatigue strength is determined empirically through certain experiments and curve fitting equations.

For reinforcing and prestressing steel, EN 1992-1-1 (2004) clause 6.8.5 specifies that there is enough fatigue resistance if the following condition is satisfied:

\[
γ_{F, fat} \Delta σ_{S, equ}(N^*) \leq \frac{Δσ_{Rsk}(N^*)}{γ_{S, fat}}
\]  

(40)
where: \( \gamma_{F,\text{fat}} \) and \( \gamma_{S,\text{fat}} \) are the fatigue load and resistance factors given in EN, respectively.

\( \Delta \sigma_{\text{S,equ}}(N^*) \) is the stress range for the \( N^* \) cycles given in the S-N curve specified in Figure 3.47 and Table 3.5. The relationship can be stated as \((\Delta \sigma_{\text{Rsk}})^b N = \text{constant}\).

\( \Delta \sigma_{\text{Rsk}}(N^*) \) is the damage equivalent stress range for steel for \( N^* \) cycles.

\[ E_{cd,\text{max,equ}} + 0.43 \sqrt{1 - R_{\text{equ}}} \leq 1 \] (41)

where:
\[ R_{equ} = \frac{E_{cd,\text{min,equ}}}{E_{cd,\text{max,equ}}} \]  

(42)

\[ E_{cd,\text{min,equ}} = \frac{\sigma_{cd,\text{min,equ}}}{f_{cd,fat}} \]  

(43)

\[ E_{cd,\text{max,equ}} = \frac{\sigma_{cd,\text{max,equ}}}{f_{cd,fat}} \]  

(44)

\[ f_{cd,fat} = k_1 \beta_{cc}(t_0) f_{cd} \left(1 - \frac{f_{ck}}{250}\right) \]  

(45)

\( R_{equ} \) is the stress ratio

\( E_{cd,\text{min,equ}} \) is the minimum compressive stress level

\( E_{cd,\text{max,equ}} \) is the maximum compressive stress level

\( \sigma_{cd,\text{min,equ}} \) is the lower stress of the ultimate amplitude for \( 10^6 \) cycles

\( \sigma_{cd,\text{max,equ}} \) is the upper stress of the ultimate amplitude for \( 10^6 \) cycles

\( f_{cd,fat} \) is the design fatigue strength of the concrete corresponding to \( 10^6 \) cycles

\( \beta_{cc}(t_0) \) is the coefficient for concrete strength at the first load application

\( t_0 \) is the time of the start of the cyclic loading on concrete in days

\( k_1 \) is the factor adjusted for the number of cycles considered (0.85 for \( 10^6 \) cycles)

\( f_{cd} \) is the design cylinder strength

\( f_{ck} \) is the characteristic cylinder strength

### 3.17.2 Miner’s Rule

Miner’s Rule assesses the fatigue damage in each material by linearly adding the damages caused by several cycles of load, each with a different amplitude. For each load cycle with a constant amplitude, the ratio of the actual number of cycles (denoted by \( n \)) to the number of cycles to fatigue
failure (denoted by N) is found. Then, all these ratios are added together, confirming that the material encompasses sufficient fatigue resistance if the value of the addition, which represents the total damage, is less than 1.0. The total damage is often limited a value below 1.0 to be conservative. This is especially necessary for concrete experiencing decreasing stress levels, which reduces the total damage to a value significantly lower than 1.0 (International Federation for Structural Concrete, 2010). According to the *fib* Model Code (2010) which outlines state-of-the-art design procedure and models, Miner’s Rule is the most refined method of verifying the fatigue resistance of reinforced concrete structures.

The number of load cycles to fatigue failure (N) is obtained from establishing an S-N relationship for the material in consideration, where S is the stress amplitude. For steel, this relationship is given in Figure 3.47. For concrete, the equations proposed by the *fib* Model Code (2010) will be outlined since the Euro Code (EN) does not give clear S-N relationships for concrete. The compressive stresses calculated in the S-N relationships given next are multiplied by an averaging factor, \( \eta \), to reflect the variation in the compressive stresses of the cracked concrete from the top compression fiber to 300 mm below it:

\[
\eta = \frac{1}{1.5 - 0.5 \frac{\sigma_{c1}}{\sigma_{c2}}} \tag{46}
\]

in which \( \sigma_{c1} \) and \( \sigma_{c2} \) are defined in Figure 3.48.

![Figure 3.48: Definitions of \( \sigma_{c1} \) and \( \sigma_{c2} \) (International Federation for Structural Concrete, 2010)](image)

For concrete under compression, the S-N relationships are as follows:

\[
\log N_1 = \frac{8}{Y - 1} \left( S_{cd,max} - 1 \right) \tag{47}
\]
\[
\log N_2 = 8 + \frac{8 \ln(10)}{Y - 1} (Y - S_{cd,\text{min}}) \log \left( \frac{S_{cd,\text{max}} - S_{cd,\text{min}}}{Y - S_{cd,\text{min}}} \right) 
\] (48)

\[
\log N = \begin{cases} 
\log N_1, & \log N_1 \leq 8 \\
\log N_2, & \log N_1 > 8 
\end{cases}
\] (49)

where:

\[
Y = \frac{0.45 + 1.8 S_{cd,\text{min}}}{1 + 1.8 S_{cd,\text{min}} - 0.3 S_{cd,\text{min}}^2}
\] (50)

\[S_{cd,\text{min}}\] is the minimum compressive stress level = \(\gamma_{Ed} \sigma_{cd,\text{min}} \eta / f_{cd,\text{fat}}\)

\[S_{cd,\text{max}}\] is the maximum compressive stress level = \(\gamma_{Ed} \sigma_{cd,\text{max}} \eta / f_{cd,\text{fat}}\)

\[\sigma_{cd,\text{max}}\] is the maximum compressive stress in the concrete

\[\sigma_{cd,\text{min}}\] is the minimum compressive stress in the concrete

\[f_{cd,\text{fat}}\] is the fatigue design strength given earlier

\[\gamma_{Ed}\] is the fatigue load factor

The maximum and minimum compressive stresses in the aforementioned fatigue equations refer to the internal stresses in the concrete that result from the applied fatigue loading. Depending on the aspect ratio of the foundation, these stresses can be found either by using the classical beam theory or the strut-and-tie method. The fatigue verification of the foundation consists of ensuring that the compression struts, longitudinal bars, and shear reinforcement of the footing possess sufficient fatigue resistance. In addition, the resistance of the anchor bolt cage as well as the underlying grout to the fatigue loading needs to be verified. The pedestal must also hold sufficient fatigue resistance, which is verified by checking the pedestal concrete and vertical reinforcement for fatigue resistance. If any of the fatigue checks are not satisfied, revisions to the design are necessary. This can consist of, but not limited to, using higher ratios of reinforcement, bigger
depths of concrete, or higher strength concrete or steel. Figure 3.49 and Figure 3.50, respectively, show the decrease in the damage (measured by Miner’s Rule) of the longitudinal and shear reinforcement of the footing as the amount of reinforcement increases.

Figure 3.49: Variation of the longitudinal reinforcement damage with the reinforcement amount (Goransson & Nordenmark, 2011)

Figure 3.50: Variation of the shear reinforcement damage with the reinforcement amount (Goransson & Nordenmark, 2011)
The serviceability limit state load combination for static loads is used in conjunction with the fatigue loads: for every number of cycles corresponding to a constant stress range and amplitude, the low and high amplitudes are used (along with the serviceability limit state static load combination) to produce the minimum and maximum internal stresses, respectively (European Committee for Standardization, 2004). In addition to the load demand, fatigue loading imposes higher deflection demands compared to the same level of static loading; the total progressive deflection due to fatigue loading after n load cycles is calculated by the following empirical equation given by the fib Model Code (2010):

\[ a_n = a_1 [1.5 - 0.5 \exp(-0.03n^{0.25})] \]  

(51)

where: \( a_n \) is the total deflection after n load cycles

\( a_1 \) is the deflection at the first cycle due to the application of the maximum load

While Miner’s Rule may provide a convenient way to readily assess the general fatigue damage of a structure, some studies have shown that it can give both conservative and unsafe estimations, since the fatigue damage can happen at any value between 0.79 and 1.53 (Sutherland, 1999). Moreover, this method neglects the stress redistribution and the irreversibility of fatigue damage and does not consider the effects of the loading sequence and strain accumulation. Finally, the consideration of independent material damage ignores the complex mechanisms that result from the interaction of concrete and steel. Therefore, there is a need for a more realistic and accurate way of designing against fatigue damage that would aim to capture the behaviour of reinforced concrete, as a composite material, under fatigue loading instead of relying on simple heuristic methods for independently checking the resistance of the constituent materials.

### 3.17.3 Isojeh et al. (2016) Fatigue Model

The fatigue failure of a reinforced concrete beam occurs when one of its constituents (concrete or steel) exhibits fatigue failure. For a more realistic evaluation of the fatigue performance of the structure during its service life, Isojeh and Vecchio (2016) proposed a model for the verification of fatigue design in terms of evolving deformation, which is used as a fatigue damage parameter. This would take into consideration the accumulated irreversible strains as a result of fatigue loading; these strains in the concrete accumulate with the progression of the loading, further increasing the strains and stresses of the reinforcement. In the same manner, the crack growth in
the reinforcement intersecting the major crack increases the strain in the reinforcement, which in turn increases the concrete strain (Isojeh and Vecchio 2016). The degradation of concrete strength and stiffness with the progression of fatigue loading is also accounted for in this model. As such, fatigue failure happens when the induced stress on the concrete or reinforcement is equal to the residual strength.

In the proposed model, linear fracture mechanics are used to estimate the fatigue failure point of the reinforcement bar. The Paris Crack Growth Law, given in Eq. (7) and Eq. (8) in the previous chapter, is used to determine the propagation of the reinforcement crack up to the point of fracture. Substituting Eq. (8) into Eq. (7) and integrating the number of cycles with respect to the crack depth yields the following expression:

\[
a_j = \left[ \frac{a_i^\alpha}{1 - \left[ N_{ij} \left( C \alpha \pi \frac{n}{2} Y n \Delta \sigma^n \alpha a_i^\alpha \right) \right]} \right]^{1/\alpha}
\]

where: \( \alpha = n/2 - 1 \)

\( Y \) is the shape factor

\( C \) and \( n \) are material constants (taken as \( 2 \times 10^{-13} \) and 3 for steel, respectively)

\( \Delta \sigma \) is the fatigue stress range

\( a_i \) and \( a_j \) are the smallest and largest crack depths for the interval of cycles \( N_{ij} \)

The initial crack depth for the reinforcement, \( a_0 \), is given as follows:

\[
a_0 = \frac{1}{\pi} \left( \frac{\Delta K_{th}}{Y \Delta \sigma_{lim}} \right)
\]

where \( \Delta K_{th} \) and \( \Delta \sigma_{lim} \) are the threshold stress intensity factor and fatigue limit of steel, respectively. Since the initial crack depth can be known through this equation, the crack depth corresponding to a particular number of cycles (\( a_j \) for \( N_{ij} \)) can be determined iteratively. From the crack depth leading to fatigue failure (denoted as \( a_y \)), the progressive area reduction of the reinforcement under fatigue loading can be estimated:
\[ A_i(a_y) = A_0 - A(a_y) \] (54)

where: \( A_i(a_y) \) is the remaining uncracked reinforcement cross-sectional area

\( A_0 \) is the cross-sectional area of the reinforcement

\( A(a_y) \) is the area of the fractured portion of the reinforcement

An expression for \( A(a_y) \) is given as follows:

\[ A(a_y) = \frac{\theta}{90} \pi r^2 - r \sin \theta \left( 2r - a_y \right) \] (55)

where \( r \) is the radius of the reinforcing bar and \( \theta \), shown in Figure 3.51, is given as:

\[ \theta = \cos^{-1}\left( \frac{r - 0.5a_y}{r} \right) \] (56)

To account for the fatigue damage and degradation of concrete, Isojeh and Vecchio (2016) proposed a modification to the strut-and-tie analysis in which the irreversible strain in the concrete strut is considered a progressively increasing pseudo load. The damage evolution in the strut-and-tie is based on the modified equilibrium, compatibility, and constitutive equations; the equilibrium of the forces is repeated for each loading cycle or interval, and the fatigue life of the beam corresponds to the point at which either the strut stress is equal to the limiting stress of the strut or the tie stress is equal to its corresponding yield stress. When the compressive strain evolution
of the concrete strut is plotted against the number of load cycles, the fatigue failure point of the strut corresponds to the number of cycles where the strain suddenly increases drastically, as shown in Figure 3.52.

![Figure 3.52: Concrete strut principal compressive strain evolution (Isojeh & Vecchio, 2016)](image)

The proposed model for the residual strength of the concrete strut is a modification of the strut limiting strength given by Collins and Mitchell (1997):

\[
f_c(i) = \frac{1 - D_s}{0.8 + 170\varepsilon_1}
\]

where: 

\( D_s \) is the concrete strength damage model

\( \varepsilon_1 \) is the principal tensile strain of the concrete strut, which is related to the effective compressive strain of the concrete strut (denoted as \( \varepsilon_2 \)) through the following compatibility equation:

\[
\varepsilon_1 = \varepsilon_x + (\varepsilon_x + \varepsilon_y)(\cot\theta)^2
\]

where \( \varepsilon_x \) and \( \varepsilon_y \) are the horizontal and vertical strains in the beam, respectively, and \( \theta \) is the angle between the strut and the tie. The effective compressive strain of the concrete strut due to fatigue loading (\( \varepsilon_2 \)) is somewhat different than the peak principal compressive strain due to monotonic
loading (denoted as $\varepsilon_p$) because it takes into consideration the irreversible strains of the concrete as well as the degradation in strength and stiffness due to fatigue loading. The following expression is suggested by Isojeh et al. (2017):

$$\varepsilon_2 = \varepsilon_p (1 + \sqrt{D_s}) - \varepsilon_0$$  \hfill (59)

where $\varepsilon_0$ is the irreversible compressive strain due to the previous load cycles. The analysis is done iteratively since the value of $\varepsilon_1$ is initially unknown; $\varepsilon_1$ and $\varepsilon_2$ are first assumed. From these assumptions, the values of $\varepsilon_x$ and $\varepsilon_y$ are found through strain compatibility equations. The value of the limiting strut stress is also found, and the validity of the assumptions for $\varepsilon_1$ and $\varepsilon_2$ are verified. Iterations of $\varepsilon_1$ and $\varepsilon_2$ continue until convergence of their values is achieved. The forces in the ties and struts are determined from the fatigue load applied. Then, the stresses in the struts and ties are found by dividing the forces found by the corresponding areas. The net area of the tie, $A_t(a_y)$, is used in the determination of the tie stress. At this stage, the irreversible strain, concrete damage, and the reinforcement crack depth for the next cycle is obtained. This analysis is carried on until the tie stress equals the yield stress or the strut stress equals its limiting stress. More detailed steps and explanations of the proposed model can be found in Isojeh et al. (2016). A subsequent paper was published (Isojeh, Vecchio, & El-Zeghayar, 2017) proposing a concrete fatigue damage model (and an expression for the irreversible compressive strain) and a simplified constitutive model for the fatigue behaviour of concrete in compression. A main objective of the experimental program carried out in this thesis is to verify the validity and accuracy of these models, which were incorporated into the nonlinear finite element program VecTor2 developed at the University of Toronto.

### 3.18 Finite Element Analysis of the Foundation

The complex geometry and detailing of a foundation, as well as the various kinds of forces acting on it make it difficult and time-consuming to perform hand calculations; the finite element method (FEM) provides a powerful tool to carry out detailed analysis of the foundation. Several commercial and educational software are available for this purpose such as STAAD Pro, SAP2000, ETABS, ANSYS, and VecTor2. After laying out the initial dimensions of the foundation (based on prior experience), the FEM can be used to establish the soil pressure distribution that spans in
both dimensions of the base of the foundation. Consequently, the bending moment and shear force
diagrams can also be established, which enable identification of the critical sections of the
foundations. After establishing the final design of the foundation including the concrete
dimensions and reinforcement amounts, a finite element model that includes the reinforcement and
connection detailing can be established. The relevant load combination can be applied to the model
with the suitable boundary conditions, simulating the real behaviour of the structure. This would
ensure that the foundation exhibits the required resistance and behaviour (within the design limits),
as well as confirm that the design layout is adequate for the load combinations considered.

The complexity of the FEM requires close attention to the input values and mesh layout, since the
large number of variables increases the probability of making mistakes. Engineering judgement is
also required to make simplifications in the model and determine some input values that are not
available. Assessment of the results and ensuring that they fall within the expected range is
paramount to the adequacy of the method.

The computational intensity of the FEM makes it important to carefully choose the type of
elements used. There is almost always a trade-off between the accuracy of the results and the
computational time. Simple low-powered elements produce less accurate results than more
complex high-powered elements, but require less computational time. For square foundations, the
use of rectangular elements with an aspect ratio close to one would yield optimal results. The use
of curvilinear elements with circular foundation might be more appropriate to capture the
geometry more accurately.

One of the most challenging yet critical parts of the finite element modelling of the foundation is
establishing the connection between the foundation and the underlying soil. The connection must
be able to reasonably simulate the soil behaviour as well as the foundation-soil interaction without
critically sacrificing the computational efficiency. One commonly used method to model the soil
and its interaction is the use of the modulus of subgrade reaction, which models the soil as a set of
elastic springs possessing vertical stiffness. The deflection at the soil level (or the soil settlement)
is directly proportional to its pressure. Although this method ignores the interaction between soil
particles and does not consider the continuity of the soil medium and its lateral stiffness, it
produces reasonable results in the modelling of the soil-foundation interaction and the resulting
soil pressure and internal foundation stresses as a result of that. The modulus of the subgrade
reaction is expressed in units of stiffness (force per displacement) per unit area. Its value depends on not only the type of soil, but also the allowable ultimate bearing capacity and the size of the foundation laying on the soil. There are no prescribed formulas to obtain a definite value of this modulus but rather suggested expressions and ranges which, combined with design experience, yield a suitable narrow range of its possible values that can be used in the modelling. Since the stiffness of the foundation is typically more than 10 times more than that of the soil, reasonable variations in the values of the modulus of the subgrade reaction will not have a significant effect on the stresses and bending moments of the foundation (Bowles, 1997). Table 3.6 presents a guide for the range of the modulus (denoted by $k_s$) values for different soil types. Bowles (1997) lays out different formulas that can be used to find suitable values of $k_s$. However, these formulas depend on either the modulus of elasticity of the soil or the size of loading plate used for load tests. In lieu of these equations, Bowles (1997) suggests the following general equation as a reasonable estimation:

$$k_s = (P)(FS) (q_a)$$  \hspace{1cm} (60)

where: $P$ is a factor used to account for the ultimate soil settlement (40 for an ultimate settlement of 25.4 mm, and 160 for an ultimate settlement of 6 mm)

$FS$ is the factor of safety

$q_a$ is the allowable soil bearing capacity defined as the ultimate bearing capacity divided by a factor of safety.

Table 3.6: Range of the modulus of subgrade reaction, $k_s$. Note that $q_a$ refers to the allowable soil bearing capacity (Bowles, 1997)

<table>
<thead>
<tr>
<th>Soil</th>
<th>$k_s$ (kN/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loose sand</td>
<td>4800 - 16000</td>
</tr>
<tr>
<td>Medium dense sand</td>
<td>9600 - 80000</td>
</tr>
<tr>
<td>Dense sand</td>
<td>64000 - 128000</td>
</tr>
<tr>
<td>Clayey medium dense sand</td>
<td>32000 - 80000</td>
</tr>
<tr>
<td>Silty medium dense sand</td>
<td>24000 - 48000</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>$q_a$ range</th>
<th>$k_s$ (kN/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>$q_a \leq 200$ kPa</td>
<td>12000 - 24000</td>
</tr>
<tr>
<td>$200 &lt; q_a \leq 200$ kPa</td>
<td>24000 - 48000</td>
</tr>
<tr>
<td>$q_a &gt; 800$ kPa</td>
<td>$&gt; 48000$</td>
</tr>
</tbody>
</table>

The value of the modulus of the subgrade reaction to be used in modelling is suggested by the geotechnical team. The equations available to calculate its value as well as Table 3.6 are used to
verify the adequacy of the suggested value. Other methods can be used to model the soil and its effects on the structure. These methods include using realistic models to represent the behaviour of the soil, such as the Mohr-Coulomb model and the Hardening Soil model (Svensson, 2010).
Chapter 4: Experimental Program

4 Experimental Program

4.1 Introduction

This chapter describes the experimental program performed. Each of six large-scale deep reinforced concrete beams was subjected to a three-point bending test (i.e. point load at the mid-span with two moment-relieving supports) of either monotonic or fatigue loading. In addition to assessing the fatigue degradation of the beams and the associated failure mode as compared to that under monotonic loading, these experiments were used to verify the validity and accuracy of the fatigue damage models proposed by Isojeh et al. (2016) and compare them with the traditional methods based on S-N curves and Miner’s rule, so as to determine their suitability for use in fatigue design. The experiments also enabled the investigation of the possibility of using steel fibre-reinforced concrete (SFRC) as an alternative to traditional shear reinforcement for fatigue resistance, and the assessment of the contribution of steel fibres to the fatigue resistance of reinforced concrete. The parameters required for the material properties were obtained by performing three supplementary material tests: coupon tests of the reinforcement, concrete modulus of rupture (MOR) bending tests, and concrete cylinder compression tests. The entirety of the experimental program, including specimen preparation and testing, was conducted in the structural laboratories at the University of Toronto over a span of 12 months, from July 2016 to July 2017.

Section 4.2 provides details on the specimen’s set up and detailing. Section 4.3 outlines the material tests and parameters. Sections 4.4 to 4.6 describe the specimens’ preparation, test instrumentation, and testing procedure (including important test parameters), respectively.

4.2 Test Specimens

Six large-scale reinforced concrete beams, divided into three sets, were constructed. Each set contained two beams with identical detailing, with the first beam subjected to monotonic loading until failure and the second beam subjected to fatigue loading. All beams were 4.0 m in length, 1040 mm in height, and 200 mm in thickness as shown in Figure 4.1; they contained the same
amount of longitudinal reinforcement and had a specified concrete compressive strength of 50 MPa.

Table 4.1 shows the properties and parameters of each beam. The naming convention for each beam consists of three letters: the first two letters describe the type of reinforcement while the last letter specifies the loading protocol. For instance, LTM refers to the beam with longitudinal (L) and transverse (T) reinforcement subjected to monotonic loading (M) while LLF refers to the beam with longitudinal reinforcement only subjected to fatigue loading.

<table>
<thead>
<tr>
<th>Set</th>
<th>Beam ID</th>
<th>Specified 28-day f’c (MPa)</th>
<th>Length (mm)</th>
<th>Height (mm)</th>
<th>Thickness (mm)</th>
<th>ρ_l (%)</th>
<th>ρ_v (%)</th>
<th>V_f (%)</th>
<th>Loading Protocol</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>LTM</td>
<td>50</td>
<td>4000</td>
<td>1040</td>
<td>200</td>
<td>0.38</td>
<td>0.15</td>
<td></td>
<td>Monotonic</td>
</tr>
<tr>
<td></td>
<td>LTF</td>
<td>50</td>
<td>4000</td>
<td>1040</td>
<td>200</td>
<td>0.38</td>
<td>0.15</td>
<td></td>
<td>Fatigue</td>
</tr>
<tr>
<td>2</td>
<td>LLM</td>
<td>50</td>
<td>4000</td>
<td>1040</td>
<td>200</td>
<td>0.38</td>
<td></td>
<td></td>
<td>Monotonic</td>
</tr>
<tr>
<td></td>
<td>LLF</td>
<td>50</td>
<td>4000</td>
<td>1040</td>
<td>200</td>
<td>0.38</td>
<td></td>
<td>0.75</td>
<td>Fatigue</td>
</tr>
<tr>
<td>3</td>
<td>LFM</td>
<td>50</td>
<td>4000</td>
<td>1040</td>
<td>200</td>
<td>0.38</td>
<td></td>
<td>0.75</td>
<td>Monotonic</td>
</tr>
<tr>
<td></td>
<td>LFF</td>
<td>50</td>
<td>4000</td>
<td>1040</td>
<td>200</td>
<td>0.38</td>
<td></td>
<td>0.75</td>
<td>Fatigue</td>
</tr>
</tbody>
</table>

ρ_l: longitudinal reinforcement ratio  
ρ_v: transverse reinforcement ratio  
V_f: volumetric ratio of the steel fibres

The detailing of the beams is shown in Figure 4.2 and Figure 4.3. The longitudinal reinforcement in all specimens consisted of four 15M deformed bars located at a depth of 1000 mm from the extreme compression face. For Set 2 beams containing transverse reinforcement (LTF and LTM), two 10M deformed bars were used in the compression face to support the stirrups; the contribution of these bars to the capacity of the beams was ignored. The stirrups were spaced at 300 mm apart throughout the length of the beam, except the last stirrup at the support where it was only 180 mm from the adjacent stirrup. Note that the beams’ detailing is symmetric about the mid-span. All the
longitudinal bars had sufficient development length to avoid bond failures and enable the bars to develop the required strength at the critical sections; tension hooks were used at both ends of the beam in order to provide the required development length for these bars. All hooks as well as stirrup bends satisfied the requirements of CSA A23.3. The cover and bar spacing also satisfied CSA A23.3 requirements.

The Set 3 beams (LFM and LFF) contained hooked-end steel fibres that were uniformly dispersed throughout the concrete. The amount of steel fibres used was 0.75% of the volume of the concrete for each beam. The specific details and parameters of the fibres will be discussed in the next section.
4.3 Material Properties

4.3.1 Concrete

Casting the beams was done in two stages: the first stage consisted of casting the Set 1 and Set 2 beams (i.e. the beams not containing steel fibres), while the second stage consisted of casting the Set 3 beams. All the concrete was ready-mix concrete obtained from Dufferin Concrete. The specifications of the concrete order for the first and second stages are given in Table 4.2 and Table 4.3, respectively.

Table 4.2: Concrete order specifications for set 1 and set 2 beams

<table>
<thead>
<tr>
<th>Product code</th>
<th>S5059220</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specified 28-day compressive strength</td>
<td>50 MPa</td>
</tr>
<tr>
<td>Volume of order</td>
<td>4.5 m³</td>
</tr>
<tr>
<td>Maximum aggregate size</td>
<td>14 mm</td>
</tr>
<tr>
<td>Slump</td>
<td>100 mm</td>
</tr>
<tr>
<td>Air Entrainment</td>
<td>None</td>
</tr>
<tr>
<td>Super plasticizer</td>
<td>To be sent with truck</td>
</tr>
</tbody>
</table>

Table 4.3: Concrete order specifications for set 3 beams

<table>
<thead>
<tr>
<th>Product code</th>
<th>S5059220</th>
</tr>
</thead>
<tbody>
<tr>
<td>Specified 28-day compressive strength</td>
<td>50 MPa</td>
</tr>
<tr>
<td>Volume of order</td>
<td>3 m³</td>
</tr>
<tr>
<td>Maximum aggregate size</td>
<td>14 mm</td>
</tr>
<tr>
<td>Slump</td>
<td>80 mm</td>
</tr>
<tr>
<td>Air Entrainment</td>
<td>None</td>
</tr>
<tr>
<td>Super-plasticizer</td>
<td>To be sent with truck</td>
</tr>
</tbody>
</table>

For the Set 1 and Set 2 beams, the specified 28-day compressive strength was 50 MPa. The maximum aggregate size of 14 mm, the minimum aggregate size that Dufferin was able to supply, was chosen to ensure that the concrete went smoothly in between the reinforcement. Although the specified slump was 100 mm, the actual achieved slump was 160 mm, which enabled the concrete to effectively go around and between the longitudinal reinforcement; there was no need to use the super plasticizer sent with the truck. The consistency of the mix is shown in Figure 4.4. After casting and setting, the beams were covered with a wet burlap and plastic to enable curing. The burlap was moistened continuously for 9 days, after which it was removed, along with the plastic, from the beams. The Set 1 beams were removed from the formwork and exposed fully to the lab environment, while the Set 3 beams were kept in the formwork although their surfaces were exposed to the lab environment. This slight difference in curing did not cause a significant change in the 28-day compressive strength, since the majority of the strength gain typically occurs in the
first three days after hardening; the actual 28-day compressive strengths for the Set 1 and Set 2 beams were 76 MPa and 78 MPa, respectively. The difference between the specified and the actual compressive strength of the concrete is not uncommon in practice since concrete companies overshoot the strength to be conservative.

![Figure 4.4: Consistency of the concrete mix for Set 1 and Set 2 beams](image)

The recommendations given by the steel fiber manufacturer (Bekaert) for the proper mixing of fibres with the concrete, shown in Appendix A, were followed for the casting of Set 3 beams. In order to make direct comparisons between the beams, the same strength (50 MPa) was specified for the Set 3 beams. The important part here was that all the beams would have very similar concrete strengths in order to assess the contribution of the fibres and the transverse reinforcement. A relatively low slump was specified (80 mm) for the fibrous concrete to avoid obtaining excessively high slumps that do not allow the fibres to disperse properly in the concrete, since the concrete supplier usually overshoots the specified slump. The actual slump obtained was 190 mm, which was enough for the proper dispersion of the fibres; there was no need to use superplasticizers with the mix. The consistency of the mix and the proper dispersion of the fibres are shown in Figure 4.5. After casting and setting, the beams were covered with wet burlap and plastic to enable curing. Similar to the Set 1 and Set 2 beams, the burlap was moistened continuously for 9 days, after which it was removed, along with the plastic, from the beams. Two weeks after the cast date, the beams were removed from the formwork. The actual 28-day compressive strength of the Set 3 beams was 56 MPa. Although this value is within 10 MPa of the specified strength, it is more than 20 MPa lower than the strengths of the Set 1 and Set 2 beams, thereby presenting a challenge in directly comparing the contributions of the fibres and the stirrups.
Cylinder compression tests were performed according to ASTM C39 in order to obtain the compressive strength of the concrete, denoted as $f'_c$. The cylinders were 12 inches in height (about 300 mm) and 6 inches in diameter (about 150 mm); they were subjected to the same curing conditions as the corresponding test beams and were tested at 28 days after casting and also on the day of testing of the corresponding beams. The cylinder compressive test set up is shown in Figure 4.6. The failure patterns obtained for the plain concrete (Set 1 and Set 2 beams) and fibre-reinforced beams are shown in Figure 4.7 and Figure 4.8, respectively. These failure patterns correspond to explosive crushing of the plain concrete cylinders and the spalling of their sides due to Poisson’s ratio effects. For the fibre-reinforced concrete, considerable toughness was witnessed in the response of the cylinders, so they underwent more deformation and did not fail explosively. Refer to Appendix B for the complete concrete compressive stress-strain curves on the day of testing for each of the beams. Table 4.4 gives the concrete properties for each test beam.
To obtain the tensile bending strength of the concrete, modulus of rupture (MOR) tests were performed on the day of testing (for LTF, LLF, and LFF specimens) according to ASTM C78-94. The MOR beams were subjected to the same curing conditions as those of the corresponding beams. The test set up is shown in Figure 4.9. The resulting failure patterns of the MOR beams for plain and fibre-reinforced concrete are shown in Figure 4.10 and Figure 4.11, respectively. The failures of these beams were sudden with a vertical crack at the mid-span vicinity where the bending moment was at its maximum value. Fibre-reinforced concrete experienced hardening after the first crack, and exhibited significant toughness in the curve as the capacity gradually dropped. Refer to Table 4.4 for a listing of the concrete properties for each test beam.
Figure 4.9: Modulus of rupture (MOR) test set up

Figure 4.10: Failure pattern of the MOR beam for Set 1 and Set 2 beams

Figure 4.11: Failure pattern of the MOR beam for Set 3 beams
Table 4.4: Concrete strength parameters

<table>
<thead>
<tr>
<th>Specimen ID</th>
<th>Curing Method</th>
<th>Time after Cast</th>
<th>$f'$ (MPa)</th>
<th>$f_r$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>LTM</td>
<td>wet burlap and plastic for 9 days, then beam removed from formwork and exposed to lab environment</td>
<td>28 days</td>
<td>76.0</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Testing day (96 days)</td>
<td>77.1</td>
<td>-</td>
</tr>
<tr>
<td>LTF</td>
<td>wet burlap and plastic for 9 days, then beam removed from formwork and exposed to lab environment</td>
<td>28 days</td>
<td>76.0</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Testing day (133 days)</td>
<td>78.2</td>
<td>6.5</td>
</tr>
<tr>
<td>LLM</td>
<td>wet burlap and plastic for 9 days, then beam kept in the formwork with surface exposed to lab environment</td>
<td>28 days</td>
<td>78.0</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Testing day (209 days)</td>
<td>82.4</td>
<td>-</td>
</tr>
<tr>
<td>LLF</td>
<td>wet burlap and plastic for 9 days, then beam kept in the formwork with surface exposed to lab environment</td>
<td>28 days</td>
<td>78.0</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Testing day (239 days)</td>
<td>84.4</td>
<td>6.5</td>
</tr>
<tr>
<td>LFM</td>
<td>wet burlap and plastic for 9 days, then beam kept in the formwork with surface exposed to lab environment</td>
<td>28 days</td>
<td>53.0</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Testing day (43 days)</td>
<td>54.7</td>
<td>-</td>
</tr>
<tr>
<td>LFF</td>
<td>wet burlap and plastic for 9 days, then beam kept in the formwork with surface exposed to lab environment</td>
<td>28 days</td>
<td>53.0</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Testing day (63 days)</td>
<td>55.7</td>
<td>6.6</td>
</tr>
</tbody>
</table>

4.3.2 Steel Reinforcement

Ductile and deformed steel reinforcement was used in the experiment. Two different reinforcement sizes were used: 15M for the tensile longitudinal reinforcement, and 10M for the stirrups and the compression face reinforcement holding them. The nominal diameters for these bars are 10 mm for the 10M bars and 15 mm for the 15M bars. The corresponding cross-sectional areas are 100 mm$^2$ and 200 mm$^2$, respectively. In order to obtain the parameters and the response of the reinforcing bars used, standard tensile coupon tests were performed using a 1000 kN MTS testing machine. For each reinforcement size, three coupons were tested and the average response of these coupons was obtained as the representative response of the reinforcement. Refer to Table 4.5 for the parameters of the coupon tests. Appendix B lists the stress-strain curves for 15M and 10M bars.
Table 4.5: Parameters of the coupon tests

<table>
<thead>
<tr>
<th>Bar Designation</th>
<th>Use</th>
<th>Sample</th>
<th>$f_y$ (MPa)</th>
<th>$f_u$ (MPa)</th>
<th>$\varepsilon_y \times 10^{-3}$</th>
<th>$\varepsilon_{sh} \times 10^{-3}$</th>
<th>$E$ (MPa)</th>
<th>$E_{sh}$ (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>15M</td>
<td>Longitudinal tensile reinforcement</td>
<td>1</td>
<td>450</td>
<td>589</td>
<td>2.68</td>
<td>19.2</td>
<td>168000</td>
<td>1250</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>448</td>
<td>585</td>
<td>2.76</td>
<td>19.2</td>
<td>162000</td>
<td>1240</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>428</td>
<td>569</td>
<td>2.37</td>
<td>18.8</td>
<td>181000</td>
<td>1190</td>
</tr>
<tr>
<td></td>
<td>Mean</td>
<td></td>
<td>442</td>
<td>581</td>
<td>2.60</td>
<td>19.1</td>
<td>170000</td>
<td>1230</td>
</tr>
<tr>
<td>10M</td>
<td>Stirrups and compression bars holding them</td>
<td>1</td>
<td>448</td>
<td>591</td>
<td>2.47</td>
<td>19.0</td>
<td>181000</td>
<td>1170</td>
</tr>
<tr>
<td></td>
<td></td>
<td>2</td>
<td>440</td>
<td>575</td>
<td>3.09</td>
<td>21.0</td>
<td>142000</td>
<td>985</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>454</td>
<td>581</td>
<td>2.91</td>
<td>21.7</td>
<td>156000</td>
<td>1010</td>
</tr>
<tr>
<td></td>
<td>Mean</td>
<td></td>
<td>448</td>
<td>582</td>
<td>2.82</td>
<td>20.6</td>
<td>160000</td>
<td>1060</td>
</tr>
</tbody>
</table>

4.3.3 Steel Fibres

Steel fibres were added to the Set 3 beams in order to assess their contribution to the fatigue resistance of reinforced concrete beams as compared to that of stirrups. The fibres used were Dramix RC-80/30-BP from Bekaert. They are hooked fibres (shown in Figure 4.12) with a total length of 30 mm and a cross-sectional diameter of 0.38 mm, resulting in an aspect ratio (i.e. length/diameter) of 79. The ultimate strength of these fibres is 3070 MPa, as given by Bekaert. Their product data are given in Appendix A.

![Figure 4.12: Hooked Bekaert steel fibre](image)

The volumetric ratio of the fibres used, compared to the total volume of the beams, was 0.75% which corresponds to 60 kg of fibres per one cubic meter of concrete. This ratio was chosen so that the contribution of the fibres to the performance of the concrete would not be nominal while at the same time a workable and consistent concrete mix would be achieved. It also corresponds to the ACI 318 recommendation for fibre volume needed to replace minimum shear reinforcement.

4.4 Specimen Construction

Constructing the large-scale test beams consisted of three main stages: building the formwork, constructing and placing the steel cage, and casting the concrete. The construction of the specimens took place at the structural laboratory of the University of Toronto in the period between July 2016 and April 2017.
4.4.1 Formwork Construction

The formwork that was constructed had the capacity of holding four beams at once, hence Set 1 and Set 2 beams were cast together. Plywood sheets of 19 mm (3/4") in thickness were used for the side and bottom panels of the formwork since the smooth surface of the plywood sheets gave a smooth concrete surface. This left the upper surface of the formwork open to insert the reinforcement cages into the formwork and cast concrete. Transverse stiffeners were used on the side plywood sheets to protect them from buckling outwards due to the hydrostatic pressure of fresh concrete. These stiffeners consisted of 50 x 100 mm (2"x4") wooden posts spaced at 400 mm from each other along the perimeter of the formwork, except at the shorter sides near the corner where the first posts after the corners were spaced at about 200 mm. Longitudinal stiffeners (also 2"x4" wooden posts) were also used for the side plywood sheets near the top and bottom surfaces. The constructed formwork is shown in Figure 4.13 and Figure 4.14. Note that after inserting the steel cages into the formwork, 2"x4" wooden posts that spanned across the width of the formwork were nailed at quarter-span points in order to further prevent the formwork from buckling, as shown in Figure 4.15.

![Figure 4.13: Formwork constructed](image-url)
Figure 4.14: Slots in the formwork where beams were cast

Figure 4.15: Wooden posts placed after inserting the reinforcement cages into the forms

4.4.2 Reinforcement Preparation and Placement

Preparation of the reinforcement first involved cutting the 6.0 m supplied 15M rebar into the appropriate lengths to form the longitudinal reinforcement. These bars were also bent at the ends to form the required tension hooks to sufficiently develop the bars. The supplied 10M bars were also cut and bent to form the required stirrups and compression longitudinal reinforcement. Wooden rebar chairs and rebar ribbon ties were used during the assembly of the cages to hold the bars in place, as shown in Figure 4.16. In addition, small out-of-plane pieces of rebar were welded to the longitudinal reinforcement at the tension hooks to maintain the spacing between the individual longitudinal bars, as shown in Figure 4.17. After the reinforcement cage had been constructed, strain gauges were attached at certain locations along the longitudinal and transverse reinforcement (the details of which are discussed in the testing instrumentation section). The reinforcement locations where the strain gauges would be attached were filed and polished to
produce a smooth surface to enable the strain gauges to bond appropriately to the reinforcement and hence give accurate readings. The strain gauges used gave measurements along their longitudinal direction, so they were bonded horizontally with the longitudinal reinforcement and vertically with the transverse reinforcement. After a liquid adhesive was used to bond the strain gauges to the reinforcement, aluminum foil and duct tape were used around the strain gauges to protect them from possible damage that might happen during concrete casting. The wires were channeled out along the reinforcement from the top corners of the beam since these were not critical sections. Finally, the finished reinforcement cage was put into the formwork ready for casting as shown in Figure 4.18. In order to elevate the longitudinal reinforcement the required distance above the bottom cover of the beam, rectangular concrete blocks of 40 mm thickness were used.

Figure 4.16: Assembly of rebar cage using wooden chairs and ribbon ties

Figure 4.17: Welding of out-of-plane bar piece to maintain spacing between bars
4.4.3 Concrete Casting

The final phase of specimen construction consisted of casting the concrete. For the Set 1 and Set 2 beams, a bucket was used to transport concrete from the ready-mix truck to the beams. Each beam was filled with four layers of concrete. After each layer, two large vibrators were used along the beam as the primary means of consolidation. There were no problems encountered regarding the passage of the coarse aggregate between the longitudinal reinforcement or the workability of the concrete mix containing the fibres. After all the beams were filled with concrete, the surface was levelled and finished using a trowel, which produced a smooth top surface as shown in Figure 4.19. Two hours after finishing the surface, wet burlap and plastic sheets were used to cover the beams in order to start the curing process.
Before successfully casting the fibre-reinforced Set 3 beams, there were two separate unsuccessful attempts at mixing the fibres with the concrete. In the first attempt, a 1.5% volumetric ratio of fibres (i.e. 120 kg of fibres per one cubic meter of concrete) was added to the concrete truck in the mixing plant. All the recommendations of Appendix A for mixing fibres were followed: the slump of the concrete was more than 120 mm, the fibres were added at a rate no more than 40 kg/min, and the truck drum rotation speed was close to 12 rpm. A conveyor belt was used to transport the fibres to the concrete mix in the truck as the drum was rotating. Unfortunately, the fibres balled up inside the concrete mix resulting in an inconsistent mix. Therefore, the casting was called off. Two weeks later, after consulting with industry professionals, it was decided that the fibre dosage be cut in half (i.e. using 0.75% volumetric ratio of fibres) in order to avoid fibre balling in concrete. Despite using half of the dosage and a slower rate of fibre addition combined with a more thorough mixing, fibres still balled up as shown in Figure 4.20. These were incompressible balls that consisted of fibres clumped together. This showed that the common concrete trucks used in industry are generally not suited to handle fibre dosages that exceed 20 kg/m$^3$ (i.e. 0.25% volumetric ratio) since these trucks neither have the necessary rotating mixing paddles inside them that disperse the fibres into the concrete nor do they provide the mixing speed necessary for SFRC mixes. An advocacy in the academia for using steel fibres in structural applications should be accompanied by a parallel move in the industry to acquire equipment that guarantees the efficiency and cost-effectiveness of such mixes.

Figure 4.20: Balling up of fibres in the concrete mix
To avoid fibre balling, a high-speed portable concrete mixer (shown in Figure 4.21) containing rotating metal paddles (shown in Figure 4.22) was rented to ensure proper mixing of concrete and steel fibres. The rotating speed of the mixer was at least 30 rpm. The concrete was poured from the truck to the mixer up to a predetermined volume, where a pre-measured amount of fibres (corresponding to 0.75% ratio) was added to the concrete and dispersed by the mixer. Then, the mixer was lifted by the crane and emptied into the formwork. Since the capacity of the mixer was only 140 liters, several batches were needed in order to fill the formwork and the cylinders. Upon visual examination of the SFRC mix and later on the examination of the cylinder test results, it was clear that there was uniform dispersion of fibres into the concrete. During casting, consistent minimal amounts of water were added to each batch in the mixer to counteract the reduction in slump caused by the addition of fibres in order to ensure the workability of the mix. Since the mixer was emptied from a height into the formwork and the mix had a slump of 190 mm, it was decided that no vibration to the mix in the formwork was necessary and that gravity was enough to ensure proper consolidation of the concrete. Unfortunately, this decision turned out to be incorrect since there were numerous small voids on the side surfaces of the two beams (see Figure 4.23). In addition, some longitudinal bars were exposed at a portion of the tension hooks at the end of the beams. Subsequently, a non-shrink high strength precision grout (Quikrete No. 1585-00) was used to patch the beams and provide bond to the exposed tension hooks. The specifications of this grout are given in Appendix C. Despite the use of the grout, the improper consolidation of the SFRC mix caused by not vibrating the mix had consequences that were seen in the test results discussed in the next chapters.

Figure 4.21: High-speed portable concrete mixer
Figure 4.22: Rotating metal paddles in the mixer

Figure 4.23: Voids at the sides of the Set 3 beams

4.5 Test Setup

The test beams were transported to the testing location in the lab through overhead cranes, steel chains, and lifting hooks that were embedded in the concrete at opposite ends of each beam. The intended test set up was for each beam to be simply supported and subjected to a point load at mid-span, as shown in Figure 4.24. The actual support conditions of each beam were two rollers (shown in Figure 4.25) instead of one roller and one pin; the horizontal restraint was provided by the loading plate at the mid-span as shown in Figure 4.26. These support conditions resulted in the same bending conditions as those shown in Figure 4.24. To prevent buckling and toppling of the beams, out-of-plane lateral supports were provided at about 320 mm from each end of the beam positioned close to the mid-height of the cross section. These supports did not affect the behaviour
of the beams or impose additional unanticipated bending constraints. The final test set up of each beam is shown in Figure 4.27 and Figure 4.28. Note that each beam was painted with an ultra-flat acrylic white paint in order to highlight the cracking patterns. The instrumentation used in the tests is discussed next.

![Figure 4.24: Intended support conditions of the beam (all dimensions are in mm)](image1)

![Figure 4.25: Roller used in the test](image2)

![Figure 4.26: Actual support conditions of the beam (all dimensions are in mm)](image3)
Figure 4.27: Test set up front view

Figure 4.28: Test set up side view
4.6 Test Instrumentation

During the test, a computer-controlled data acquisition system was used to obtain measurements from the test instrumentation used: strain gauges and linear variable differential transducers (LVDTs). Values measured by these instruments during the test were stored in text files and saved in the computer. The values obtained from the loading jack (i.e. jack load and displacement) were also included in the saved files.

4.6.1 Strain Gauges

Strain gauges were used to obtain the strains at their corresponding locations. They were used in the tests to obtain the strains of longitudinal reinforcement and stirrups, allowing a closer look at the behaviour of the test specimens. A sample strain gauge used in the experiment is shown in Figure 4.29. All the strain gauges used were of the same type and model; they were 5 mm in length and had the properties shown in Figure 4.30. They were obtained from Tokyo Sokki Kenkyujo Co. Ltd.

Figure 4.29: Strain gauge used in the experiments
The strain gauges were used only for the test beams subjected to fatigue loading, namely LTF, LLF, and LFF; putting strain gauges on the other beams subjected to monotonic loading was not justified since the main output needed from those beams was the ultimate load. The strain gauges were placed symmetrically about the mid-span of the beams; 7 gauges were used for LLF and LFF while 25 gauges were used for LTF as shown in Figure 4.31 and Figure 4.32. The strain gauges in the stirrups of LTF were positioned along the shear spans of both sides from the mid-span in order to capture the diagonal shear cracks during testing. The labels used for each strain gauge for LTF, LLF and LFF beams are shown in Figure 4.33 and Figure 4.34.

Figure 4.30: Properties of the strain gauges used in the experiments

Figure 4.31: Strain gauges in LLF and LFF beams (all dimensions are in mm)
Figure 4.32: Strain gauges in the LTF beam (all dimensions are in mm)

Figure 4.33: Strain gauge labels for LLF and LFF (all dimensions are in mm)

Figure 4.34: Strain gauge labels for LTF (all dimensions are in mm)
4.6.2 Linear Variable Differential Transducers (LVDTs)

Linear variable differential transducers (LVDTs) measure changes in length, which allows the determination of deflections. In many cases, average strains can also be calculated since the original fixed length of the LVDT is known. In the experiments performed, three LVDTs were used for each shear span and one LVDT was placed at the mid-span. In addition, one LVDT was placed at each support to account for any deformations at the supports. The support LVDTs are not integral to the test hence they are not shown in the drawings; the LVDTs configuration for each beam is shown in Figure 4.35. Note that the LVDTs were placed only on the front face of the beams and all of them had a stroke of ± 25 mm.

![Figure 4.35: LVDTs placed in each beam (all dimensions are in mm)](image)

The mid-span LVDT was used to obtain the deflection resulting from the load applied. For each shear span, three different LVDTs were placed. These LVDTs would enable the determination of the principal strains through Mohr Circle transformations. The method described by Isojeh et al. (2016) was used here to determine the principal strains for each shear span. Figure 4.36 shows the symbols used for each LVDT. Considering the left shear span, the following equations apply:

\[
\varepsilon_x = \varepsilon_{L3} - \varepsilon_{L2} + \varepsilon_{L1}
\]

\[
\varepsilon_y = \varepsilon_{L2}
\]

\[
\gamma_{xy} = \varepsilon_{L3} - \varepsilon_{L1}
\]

where:
$\varepsilon_x$ is the strain along the x (i.e. horizontal) direction

$\varepsilon_y$ is the strain along the y (i.e. vertical) direction

$\gamma_{xy}$ is the shear strain related to the x and y directions (taken as negative for the left shear span)

$\varepsilon_{L1}$ is the strain along L1

$\varepsilon_{L2}$ is the strain along L2

$\varepsilon_{L3}$ is the strain along L3

For the right shear span, the following equations are used:

\begin{align*}
\varepsilon_x &= \varepsilon_{R3} - \varepsilon_{R2} + \varepsilon_{R1} \\
\varepsilon_y &= \varepsilon_{R2} \\
\gamma_{xy} &= \varepsilon_{R1} - \varepsilon_{R3}
\end{align*}

where:

$\varepsilon_{R1}$ is the strain along R1

$\varepsilon_{R2}$ is the strain along R2

$\varepsilon_{R3}$ is the strain along R3

$\gamma_{xy}$ is taken as negative for the right shear span

For each shear span, the average principal strains ($\varepsilon_1$ and $\varepsilon_2$) are obtained through the following equation:

\begin{align*}
\varepsilon_1 &= \frac{1}{2}(\varepsilon_x + \varepsilon_y) + \frac{1}{2} \left( \sqrt{ (\varepsilon_x - \varepsilon_y)^2 + \gamma_{xy}^2 } \right) \\
\varepsilon_2 &= \frac{1}{2}(\varepsilon_x + \varepsilon_y) - \frac{1}{2} \left( \sqrt{ (\varepsilon_x - \varepsilon_y)^2 + \gamma_{xy}^2 } \right)
\end{align*}
During the tests, the LVDTs gave readings at every load stage or cycle. This enabled the determination of the principal strains at every load stage for each shear span, which provided an insight into the evolution of the principal strains after the onset of fatigue loading. It was ensured that the LVDTs covered a sufficient height of the beam in order to cover many cracks so that the average readings given would be more accurate. The original LVDT lengths used in the strain calculations are given in Table 4.6.

### Table 4.6: Original length of each LVDT in each shear span of the test beams

<table>
<thead>
<tr>
<th>LVDT Symbol</th>
<th>Original Length (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>L1</td>
<td>1245</td>
</tr>
<tr>
<td>L2</td>
<td>880</td>
</tr>
<tr>
<td>L3</td>
<td>1245</td>
</tr>
<tr>
<td>R1</td>
<td>1245</td>
</tr>
<tr>
<td>R2</td>
<td>880</td>
</tr>
<tr>
<td>R3</td>
<td>1245</td>
</tr>
</tbody>
</table>

### 4.7 Test Loading Protocol

Two types of loading protocols were used in the experiments: monotonic and fatigue loading. Referring to Table 4.1, three of the beams (i.e. LTM, LLM, and LFM) were subjected to monotonic loading, while the other three (i.e. LTF, LLF, and LFF) were subjected to fatigue loading. Each beam subjected to monotonic loading had its counterpart of the same dimensions and detailing subjected to fatigue loading, which enabled a direct comparison of the effects of fatigue and monotonic loading.

The monotonic loading tests performed were displacement-controlled at a rate of 0.005 mm/s. The loading was applied until the beams experienced failure, marked by a sharp drop in their load
capacities. At several points during the monotonic tests (outlined in the next chapter), the tests were paused briefly in order to write down observations, identify cracking patterns, take photographs, and measure crack widths.

Fatigue loading on the test beams was applied as a continuous pulsating sinusoidal waveform with a minimum load value of 5 kN and a maximum load value of either 80% or 88% of the monotonic capacity of the beam, as shown in Table 4.7. The values of the maximum fatigue loads were chosen in order to enable some comparison between the different beams subjected to fatigue loading in terms of their behaviours while not exceeding their monotonic capacities. Although such load percentages can be high for actual fatigue design of wind turbine foundations, they were chosen in order to expedite fatigue failure due to laboratory limitation. Nevertheless, the fatigue code previsions still apply to these percentages. The frequency of the loading used was 0.6 Hz as this was the maximum frequency that could be applied by the loading jack. The fatigue tests were not paused during any stage of the loading regime since rest periods do have an effect on the fatigue performance of specimens, as indicated in the literature review; test observations, photographs, and crack width measurements were all taken as the tests were continuing. The fatigue load parameters are summarized in Table 4.7.

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Fatigue Loading Frequency (Hz)</th>
<th>Fatigue Load Type</th>
<th>Monotonic Capacity (kN)</th>
<th>Minimum Fatigue Load (kN)</th>
<th>Maximum Fatigue Load (kN)</th>
<th>Maximum Fatigue Load as a Percentage of Monotonic Capacity</th>
</tr>
</thead>
<tbody>
<tr>
<td>LTF</td>
<td>0.6</td>
<td>Sinusoidal</td>
<td>500</td>
<td>5</td>
<td>400</td>
<td>80%</td>
</tr>
<tr>
<td>LLF</td>
<td>0.6</td>
<td>Sinusoidal</td>
<td>450</td>
<td>5</td>
<td>400</td>
<td>88%</td>
</tr>
<tr>
<td>LFF</td>
<td>0.6</td>
<td>Sinusoidal</td>
<td>397</td>
<td>5</td>
<td>350</td>
<td>88%</td>
</tr>
</tbody>
</table>
Chapter 5: Experimental Results

5 Experimental Results

5.1 Introduction

This chapter presents the results of the large-scale beam experiments performed including the test observations for each beam. Crack patterns, propagations, and widths for various load stages will be presented in addition to the damage modes and failure patterns. Various experimental response curves will be presented, utilizing the data obtained from the linear variable displacement transducers (LVDTs) and the strain gauges.

The six beams (LLM, LLF, LTM, LTF, LFM, and LFF) were subjected to either monotonic or fatigue loading through a point load applied at mid-span while being simply supported. Each beam subjected to monotonic loading had a counterpart with the same detailing subjected to fatigue loading. For each beam subjected to fatigue loading, LVDTs were placed in both the left and right shear spans to measure the evolution of the principal strains. In addition, strain gauges were placed at certain locations along the longitudinal bars and transverse reinforcement in order to capture the yield locations for the beams subjected to fatigue loading. Note that for all beams, one LVDT was placed at each support in order to account for any support movements. All the results presented take into account the support movements. The detailed experimental results and observations are discussed next for each beam.

5.2 LTM Results

LTM contained four 15M longitudinal bars at the tension face and stirrups spaced at 300 mm apart amounting to longitudinal and transverse reinforcement ratios of 0.38% and 0.15%, respectively. It also contained two 10M bars at the top just to hold the stirrups in place. The beam was subjected to a monotonic point load at the mid-span until failure. The test was displacement-controlled with a rate of 0.005 mm/s. Note that for LTM, LVDTs were placed in the shear spans as will be seen in the photos. However, these LVDTs were not used since it was decided that such LVDTs will only be used for beams subjected to fatigue loading.
5.2.1 Test Observations

From the beginning of the test and up to the cracking load, there were no visible cracks or displacements as shown in Figure 5.1. At 186 kN, the beam exhibited its first flexural crack at the mid-span between the left and the right shear spans. Despite originating closer to the right side, the crack extended closer towards the mid-span as it was progressing upwards towards the loading jack, as shown in Figure 5.2. The maximum crack width was 0.3 mm at this stage and the crack extended to about 70% of the beam height. The mid-span displacement was 1.56 mm at this stage.

At 195 kN and 1.8 mm mid-span deflection, more vertical flexural cracks developed around the mid-span from both sides and extended to about 50% of the beam height. The maximum crack width was still 0.3 mm at the first flexural crack. An inclined shear crack from each side of the beam close to the mid-span started developing between LVDTs L2 and L3 on the left side, and R2 and R3 on the right side as shown in Figure 5.3.

At 238 kN, the shear cracks that have developed earlier joined the flexural cracks and new shear cracks developed further away from the mid-span, as shown in Figure 5.4. The bottom of these cracks were forked cracks encompassing both flexural and shear cracks. The maximum crack width was now 0.4 mm. Two flexural cracks, one of which was the first crack, extended to more than 70% of the beam height towards the loading plate as the mid-span displacement was now 2.6 mm.

At 278 kN force and 2.9 mm mid-span displacement, the existing cracks widened and propagating further up towards the loading plate. The maximum crack width was now 0.5 mm. The shear cracks that have developed in the previous loading stage were now at about 60% of the height of the beam. New shear cracks also developed in both the left and right shear span close to LVDTs L1 and R1, respectively. More shear cracks joined existing flexural cracks as shown in Figure 5.5.

At 316 kN, the newly-formed shear cracks extended along the height of the beam to join some existing flexural cracks. Other shear cracks further away from the mid-span formed close to LVDTs L1 and R1. The maximum crack width at this stage was 0.6 mm. The crack propagation manner in the beam was consistent: up towards the loading plate. When the load was 359 kN, corresponding to a displacement of 4.6 mm, there were three major cracks that extended to more than 80% of the beam height: two shear cracks from the left and right shear spans and one flexural
crack which was the same as the first flexural crack. The maximum crack width was 0.7 mm occurring at the same flexural crack. The cracks in the beam did not go beyond the lower brackets of LVDTs L1 and R1 as shown in Figure 5.6; there was more widening and propagation of existing cracks than the formation of new ones at this stage. In addition, many cracks at the tension face of the beam (i.e. bottom) became parallel to the longitudinal axis. The orientation of the shear cracks approached 45°.

When the beam load reached 391 kN, there was much less force increase for an increase in displacement, signalling the plateauing of the response. This stage signalled the greatest increase in the crack width, with the maximum width being 2.5 mm at the location of the first flexural crack. Existing cracks were widening and propagating and few new cracks were forming. The major cracks were now at 90% of the beam height only 100 mm away from the loading plate.

At 400 kN, the displacement reached 10 mm. At this stage, the major flexural crack widened significantly to a maximum crack width of 6.5 mm and went further up to more than 95% of the height of the beam towards the loading plate. Many new cracks parallel to the longitudinal reinforcement were forming in the bottom part of the beam between the existing cracks as shown in Figure 5.7.

The maximum load the beam sustained was 497 kN at 108 mm of displacement; there was only an increase of 97 kN in force over the course of 100 mm of displacement. About 80% of the capacity of the beam was achieved in only 10% of the displacement. After 400 kN, flexural cracks were widened significantly and propagating towards the loading plate. New diagonal shear cracks closer to the supports also formed but their widths were a minor fraction of those of the flexural cracks. More bottom longitudinal cracks were forming parallel to the reinforcement, resulting eventually in the spalling of about 150 mm of the concrete around the mid-span, exposing the reinforcement. The majority of the cracks formed in the area between LVDTs L2 and R2. At 497 kN, the main flexural cracks went all the way to the compression face of the beam, joining the concrete that crushed beneath the loading plate, resulting in the failure of the beam. As can be seen from Figure 5.8, the beam at failure exhibited considerable displacement. The maximum crack width at failure was 30 mm, located at one of the flexural cracks that was joined by a shear crack. Compared to conditions at 400 kN, the number of cracks increased significantly at failure.
Figure 5.1: LTM beam before cracking

Figure 5.2: LTM at the cracking load (186 kN)
Figure 5.3: LTM at 195 kN

Figure 5.4: LTM at 238 kN
Figure 5.5: LTM at 278 kN

Figure 5.6: LTM at 359 kN
Figure 5.7: LTM at 400 kN

Figure 5.8: LTM at failure (497 kN)
5.2.2 Experimental Response

The experimental load versus displacement response for LTM is shown in Figure 5.9. The curve starts linearly until the cracking load of 186 kN is reached, after which the stiffness of the beam decreases resulting in a lower slope to the fairly linear portion of the graph afterwards. Then, the curve plateaus at about 391 kN signalling the yield of the longitudinal reinforcement. At the plateauing stage, there is a much lower gain in strength for every unit increase in displacement until the beam reaches its capacity of 497 kN, at which point the concrete crushes on the upper tip of the major flexural and shear cracks below the loading plate. At failure, both the flexural and shear cracks join the concrete crushing zone.

The evolution of the maximum crack width is shown in Figure 5.10. As can be seen, the widening of the crack happened at a steady rate from the crack load up to 391 kN (about 80% of the ultimate load), after which the maximum crack width increased significantly. The maximum crack width happened at the same flexural crack throughout the test duration up until shortly before the ultimate load, where another flexural crack had the maximum crack width.

![Load vs. Mid-Span Deflection Response for LTM](image)

Figure 5.9: Load versus displacement experimental response for LTM
5.3 LTF Results

LTF had the same detailing as LTM: four 15M longitudinal bars at the tension face and 10M stirrups spaced at 300 mm apart, amounting to reinforcement ratios of 0.38% and 0.15%, respectively. It was subjected to a pulsating sinusoidal fatigue loading at the mid-span while being simply supported at the ends. The minimum and maximum fatigue loads were 5 kN and 400 kN, respectively. The maximum fatigue load was 80% of the monotonic capacity of the beam.

LVDTs were placed on both the left and right shear spans in order to measure the evolution of the principal strains. In addition, strain gauges were placed at certain locations along both the longitudinal and transverse reinforcement shown in Figure 4.32.

5.3.1 Test Observations

Since the frequency of the loading was 0.6 Hz, it only took 1.7 seconds for the first cycle to be applied. The fast application of the load caused an abrupt and loud sound when the concrete cracked, which happened at 200 kN in the first cycle. After the first cycle, many flexural cracks formed around the mid-span, as shown in Figure 5.11. Shear cracks also formed in both shear spans, which extended up to 500 mm away from the supports. Many of these shear cracks were in fact flexural-shear cracks that had a fork-like appearance at the bottom of the beam. The strictly
diagonal cracks were more seen further away from the mid-span where less flexural cracks formed. Two major flexural cracks extended to 90% of the height of the beam towards the loading plate. One of these cracks, which formed at mid-span, exhibited the maximum crack width of 3.5 mm. Most of the widening happened at this crack, since the maximum crack width excluding this major crack was only 0.7 mm, which was 20% of the maximum width witnessed at the crack.

As more load cycles were applied, the major flexural cracks formed from the first cycle were widening. The shear cracks that formed further away from the mid-span were not widening but extending towards the loading plate. The maximum crack width always happened at the major flexural crack that exhibited the maximum crack width after the first cycle. No new major cracks, shear or flexural, formed as the loading cycles progressed. Almost all of the new cracks that formed were narrow flexural crack at the bottom of the beam around the mid-span area and longitudinal cracks spanning through the formed flexural cracks. After 37316 cycles, the beam failed due to fracture of the longitudinal reinforcement crossing one of the major flexural cracks. The location of the fracture was at the intersection of the longitudinal reinforcement with the stirrup at 300 mm away from the mid-span to the right. The maximum crack width at this stage was 15 mm at one of the major flexural cracks. However, the major cracks did not reach the extreme compression face of the beam, as shown in Figure 5.12; there was no crushing of the concrete. Figure 5.13 and Figure 5.14 show the back of the beam and the location where the longitudinal reinforcement fractured. Figure 5.15 shows the fracture of the reinforcement right at the intersection with the stirrup. Note that there was no damage or deformation observed to the stirrup at failure.
Figure 5.11: LTF after one cycle

Figure 5.12: LTF at failure (after 37316 cycles)
Figure 5.13: LTF at failure from the back side

Figure 5.14: Location of reinforcement fracture in LTF
5.3.2 Experimental Response

After the application of the 37316 load cycles that caused the fracture of one of the longitudinal bars, the beam’s ultimate mid-span deflection was 24.7 mm. Figure 5.16 shows the evolution of the mid-span deflection of the beam with the number of load cycles. As can be seen, the abrupt increases in deflection between one load cycle and the next happened in the first and last few cycles. During more than 99% of the cycles, the mid-span deflection changed by an amount less than 4 mm, which was about 15% of the total deflection value.
As mentioned previously, LVDTs were placed on both the right and left shear spans. They measured the strains averaged over the square element covered by the LVDTs in each span, which measured 880 x 880 mm. Figure 5.17 and Figure 5.18 show the horizontal, vertical, and shear strains in the left and right shear spans, respectively. The horizontal strain ($\varepsilon_x$) in both spans was tensile. However, the vertical strain ($\varepsilon_y$) was tensile in the left span and compressive in the right span. The shear strain had also different signs in each span. At the failure of the beam, all the strains diverged greatly, which is seen by the vertical jumps in the graphs. These strains were transformed to the principal tensile and compressive strains and plotted in Figure 5.19 and Figure 5.20 to examine the evolution of the principal strains. In these figures, the initial increase in the strains was due to the application of the first load cycle. Then, the strains stabilized and increased gradually with the number of load cycles. At the failure of the beam, their values spiked. As can be seen, the spike in the principal tensile and compressive strains at failure in the right shear span was more than that of the left shear span. Note that the longitudinal bar fracture was in the right span.
Figure 5.17: Strains on the left shear span of LTF

Figure 5.18: Strains on the right shear span of LTF
Figure 5.19: Evolution of principal tensile strain for LTF

Figure 5.20: Evolution of principal compressive strain for LTF
The readings of the strain gauges along the longitudinal reinforcement are shown in Figure 5.21. Note that some strain gauges malfunctioned at different stages of the test, so the erroneous readings were eliminated from the plot. Initially, only the portion of the rebar at the mid-span, where the MB strain gauge was located, yielded. In general, all the strains along the reinforcement were changing gradually. After about 22000 load cycles, the rebar at LLB3 yielded. Unfortunately, LLB3 malfunctioned shortly afterwards so further readings were not possible. Some gauges showed a decrease in the strains at some point during the test, which was due to stress redistribution and the closing and opening of new cracks. At failure, strain gauge RLB3 showed a sharp increase because it was at the location of the fracture. The readings of the strain gauges along the left and right transverse reinforcement are shown in Figure 5.22 and Figure 5.23. None of the stirrups yielded and most of them were undergoing tension, except for the stirrups containing the gauges LL1 and RL1; the stirrups had strains less than the yield strain by at least 35%. Again, some stirrups exhibited decreased tensile strains during the test due to stress redistribution.

![Strain Gauges along Longitudinal Reinforcement of LTF](image)

Figure 5.21: Readings of strain gauges along longitudinal reinforcement of LTF
Figure 5.22: Readings of strain gauges along the left transverse reinforcement of LTF

Figure 5.23: Readings of strain gauges along the right transverse reinforcement of LTF
5.4 LLM Results

LLM contained the same longitudinal bars as the previous beams: four 15M longitudinal bars at the bottom of the beam. However, it did not have any stirrups, compression reinforcement, or fibres. It was subjected to a downward monotonic point load at its mid-span while being simply supported at the ends. LVDTs or strain gauges were not used with this beam. The test was displacement-controlled with a rate of 0.005 mm/s.

5.4.1 Test Observations

Prior to the cracking load, there were no visible cracks on the face of the beam, as shown in Figure 5.24. The cracking load was 206 kN, at which point the first crack initiated at the mid-span area and extended to more than half the height of the beam, as shown in Figure 5.25. There was another crack that initiated at this stage, but it only extended 100 mm upwards. The mid-span deflection at this stage was 1.53 mm and the maximum crack width was 0.2 mm.

At 226 kN load and 2.86 mm deflection, the first flexural crack extended to about 90% of the beam height. Several other flexural cracks formed at the mid-span vicinity. The maximum crack width at this stage was 0.6 mm at about the mid-height of the first crack. All the new cracks that formed were directed towards the mid-span and upwards towards the loading plate, as shown in Figure 5.26. As the load progressed, existing cracks widened and splitting cracks that ran parallel to the reinforcement at the bottom and joined major vertical flexural cracks at mid-span started to form. By 276 kN of load and 4.76 mm of mid-span deflection, these splitting cracks were visible as shown in Figure 5.27. Such cracks joined existing flexural cracks at the bottom of the beam and formed a network of cracks extending towards the mid-span. At this stage, the first flexural crack, which was the major crack at this stage, extended to more than 95% of the beam height and had the maximum crack width, which was 1.8 mm. The cracking pattern showed that cracks headed to join this major flexural crack.

When the load reached 300 kN and the deflection was 7.08 mm, the splitting cracks that formed mostly in the left shear span had grown considerably in width and number; they now extended to about 100 mm away from the left support as shown in Figure 5.28. No new cracks formed in the mid-span area and the existing major flexural cracks did not go further up in height. The maximum crack width at this stage, which was 4.5 mm, occurred at the left splitting crack. On the right side,
there was barely any formation of new cracks or widening of existing cracks; the shear cracks on this side were about one meter away from the right support.

As the load progressed, splitting cracks further formed on the right side of the beam. No new cracks formed in the mid-span area or on the left side. The splitting cracks joined the major flexural crack, which was the first crack. Even the other major crack joined this flexural crack, which at this stage was the only crack extending upwards towards the loading plate; it was only 40 mm away from the extreme compression face of the beam. Figure 5.29 shows the beam at 322 kN of load and 11.22 mm of deflection. With the development of the splitting cracks in the right side, the cracking pattern on the beam was almost symmetrical, with the major flexural crack extending upwards at the mid-span. The maximum crack width at this stage was 5.5 mm, occurring at the right splitting crack.

As the load was increasing at this stage, the existing cracks widened. The only new cracks that formed were minor cracks at the bottom of the beam that started out as vertical cracks but joined the existing splitting cracks, forming potential spalling regions along the bottom of the beam. Most of the widening happened at the splitting cracks leading to the major flexural crack. By 370 kN of load and 14.22 mm of deflection, the maximum crack was 6.0 mm on both the left and right sides as shown in Figure 5.30.

The beam’s response began plateauing after about 386 kN of load. The major flexural crack widened considerably and approaching closer to the loading plate. Pieces of concrete at the bottom of the beam were spalling off, exposing the reinforcement, as cracks joined. The ultimate load of the beam was 450 kN, after which its capacity dropped signalling its failure. At this stage, the maximum crack width of the major flexural crack was 63 mm. Large sections of the concrete, with lengths of about one meter from each side, spalled off the beam. The cracks joined in a way such that the beam was divided into three big parts, as shown in Figure 5.31. The ultimate failure of the beam was caused by the major diagonal (splitting) crack that extended to almost all the way to the left support. The longitudinal bars yielded before this stage, signalling a flexural-shear failure. The ultimate displacement at failure was 82.6 mm.
Figure 5.24: Uncracked LLM beam

Figure 5.25: LLM at the first crack load (206 kN)
Figure 5.26: LLM at 226 kN

Figure 5.27: LLM at 276 kN
Figure 5.28: LLM at 300 kN

Figure 5.29: LLM at 322 kN
5.4.2 Experimental Response

The experimental load versus mid-span deflection for LLM is shown in Figure 5.32. As mentioned previously, the beam first cracked at 206 kN, before which the response was linear. After cracking, the stiffness of the beam decreased and eventually started plateauing after 386 kN as more cracks formed. The plateauing portion of the response occurred at about 80% of the deflection spectrum; it started at about 15 mm of mid-span deflection and continued to failure. However, more than 85% of the strength gain was developed in the first 15 mm of deflection. As the ultimate load was approached, the splitting cracks widened and considerable spalling occurred until the major crack widened to a point where the beam could not take more load, hence failing.
Figure 5.33 shows the evolution of the maximum crack width with the load. At 82% of the maximum load of the beam, the maximum crack width was only about 10% of the maximum crack width at failure, which shows that the majority of the widening of the cracks happened in the last 20% of the strength gain, i.e. after 370 kN, where the response was mostly plateauing. Although the maximum crack width increased with the applied load, that increase was highly nonlinear.

Figure 5.32: Experimental load versus mid-span deflection for LLM
5.5 LLF Results

LLF had the same detailing as LLM: four 15M longitudinal bars at the tension face amounting to a reinforcement ratio of 0.38%. The concrete used was plain concrete with no steel fibres embedded. The beam was subjected to a pulsating sinusoidal fatigue loading at the mid-span, with a loading frequency of 0.6 Hz, while being simply supported at the ends. The minimum and maximum fatigue loads were 5 kN and 400 kN, respectively. The maximum fatigue load was about 88% of the monotonic capacity of the beam, which was measured to be 450 kN by testing LLM as seen previously.

LVDTs were placed on both the left and right shear spans in order to measure the evolution of the principal strains. In addition, strain gauges were placed at certain locations along both the longitudinal and transverse reinforcement shown in Figure 4.33.

5.5.1 Test Observations

After the application of one load cycle, the cracking pattern of the beam was almost identical to that of its monotonic counterpart, LLM; splitting cracks, which formed on both shear spans joined the major flexural crack at the mid-span and extended to more than 95% of the beam height, as shown in Figure 5.34. The cracks were almost symmetrical about the mid-span of the beam and
extend all the way to the supports. The splitting cracks that formed were closer to the bottom of the beam than to its mid-height, running parallel to the longitudinal reinforcement and joined by several small vertical cracks along their way. The maximum crack width at this stage was 8.5 mm, which happened at the right splitting crack, where R1 LVDT was located, as shown in Figure 5.35. Spalling of the concrete at this location appeared imminent. The maximum crack width at the major crack at mid-span was 6 mm.

As more cycles were applied, the existing cracks were opening up at the maximum load and closing when the load was released. No new cracks formed but the existing cracks were gradually widening. A portion of concrete with a length of 600 mm at the bottom of the beam in the right shear span, where the maximum crack width was located, spalled off as shown in Figure 5.36, exposing the reinforcement. This occurred after 5360 load cycles. The maximum crack width was 6.5 mm after the concrete spalled. It is interesting to note that the spalling crack passed through one bolt of the bottom bracket holding the R1 LVDT.

After 11504 load cycles, the crack propagation was still gradual with barely any formation of new cracks, as shown in Figure 5.37. The maximum crack width at this stage was still 6.5 mm at the same position, which was at the major flexural crack at the mid-height of the beam. This crack, which was close to the loading plate, did not propagate further upwards. Interestingly, a crack close to the left support actually decreased in width compared to the previous load stage, from 2.0 mm to 1.6 mm. Almost all the LVDT brackets at this stage had cracks passing through them. No further spalling of concrete was observed at this stage.

Further applications of the load cycles caused more widening of the existing cracks. More cracks formed at the bottom of the beam in the right shear span, next to where the concrete previously spalled. The spalling was now progressing from the right shear span towards the mid-span. The major vertical crack was widening significantly closer to failure. After 36748 cycles, the beam failed by fracture of the longitudinal reinforcement that was already exposed prior to failure. The location of the fracture was at the right shear span 400 mm away from mid-span. The beam at failure is shown in Figure 5.38. At this stage, the concrete spalled at the bottom throughout the majority of the right shear span, exposing the reinforcement; the cracks at the bottom of the beam joined to cause such spalling. The major crack at mid-span had a maximum crack width of 50 mm.
Compared to the right side, the left shear span experienced no spalling. There was no crushing of concrete observed at this stage. The fracture of the rebar is shown in Figure 5.39.

Figure 5.34: LLF after one load cycle

Figure 5.35: Potential spalling location at the right shear span of LLF

Figure 5.36: LLF after 5360 load cycles
Figure 5.37: LLF after 11504 load cycles

Figure 5.38: LLF at failure (after 36748 load cycles)

Figure 5.39: Fractured longitudinal bar at failure of LLF
5.5.2 Experimental Response

The evolution of the experimental mid-span deflection of LLF with the number of load cycles is shown in Figure 5.40. As can be seen, the mid-span deflection increased from 0 mm to 12 mm in the first 3 cycles. However, it only increased by 4 mm over a span of 36733 load cycles afterwards from 12 mm to 16 mm. Finally, the last few cycles increased the ultimate mid-span deflection value to 42 mm. More than 99.9% of the load cycles, 36700 cycles, caused only a 9.5% increase in mid-span deflection; the majority of the deflections were caused by the beginning and ending cycles. This was consistent with the observation that the majority of crack widening and disintegration of the beam occurred towards the ending cycles.

![Mid-Span Deflection vs. Number of Cycles for LLF](image)

Figure 5.40: Experimental mid-span deflection versus number of cycles for LLF

LVDTs were placed in each shear span to measure the evolution of strains during the test. Figure 5.41 and Figure 5.42 show the horizontal, vertical, and shear strains in the left and right shear spans, respectively. The shear spans exhibited different signs for the strains. The left side had a compressive horizontal strain, while the right side had a tensile horizontal strain. On the other hand, the vertical strains were tensile in the left shear span and compressive in the right shear span. Despite the directions of the strains, the right shear span exhibited larger strains, which was expected since the majority of the spalling and widening of cracks happened on the right side. The majority of the strain increase on both sides happened in the first few cycles, after which it became
gradual. Finally, the strains diverged when the beam failed. Note that there could be seen some strain decreases at certain points during the test, which is normal due to stress redistribution. This is corroborated by some cracks reducing in width during the test.

Figure 5.41: Strains on the left shear span of LLF

Figure 5.42: Strains on the right shear span of LLF
Figure 5.43 and Figure 5.44 show the evolution of the principal tensile and compressive strains, respectively, during the test. These strains increased considerably during the first few cycles, after which they stabilized and changed gradually. At failure, they diverged greatly. Since the failure and majority of the spalling and cracking happened there, the principal strains on the right side were at least double those on the left side.

![Figure 5.43: Evolution of principal tensile strains for LLF](image1)

![Figure 5.44: Evolution of principal compressive strains for LLF](image2)
The values of the strain gauges along the longitudinal reinforcement during the experiment are shown in Figure 5.45. Note that the values are given for the number of cycles up to almost 300, as the strain gauges malfunctioned after that point. In the first 100 cycles, the bars at RLB3 (100 mm away from the point of fracture and 300 mm away from mid-span) and LLB2 yielded. After about 125 cycles, the longitudinal bars at the mid-span yielded. The values of the remaining strain gauges remained below the yield strain. The biggest increase in the strains of the longitudinal bars happened during the first few cycles, after which the strains stabilize.

![Strain Gauges along the Longitudinal Reinforcement of LLF](image)

**Figure 5.45: Strain gauge readings for LLF**

### 5.6 LFM Results

LFM contained four 15M longitudinal bars at the bottom amounting to a reinforcement ratio of 0.38%. The concrete contained hooked-end steel fibres with a volumetric fiber ratio of 0.75%. The fibres had a diameter of 0.38 mm and a length of 30 mm. The fiber tensile strength was 3070 MPa.

The beam was subjected to a monotonic point load at its mid-span while being simply supported. No LVDTs or strain gauges were used. The test was displacement-controlled with a rate of 0.005 mm/s.
5.6.1 Test Observations

Prior to the cracking load, there were no abrupt sounds as the beam was loading. The first crack was observed at 195 kN. It was located at mid-span with a width of 0.10 mm, as shown in Figure 5.46. At this stage, there were no other cracks. As the load increased, the crack progressed along the height of the beam towards the loading plate, with no new cracks forming. By 230 kN of load, the crack had reached almost mid-height of the beam, as shown in Figure 5.47, with the same crack width, 0.10 mm. After this stage, two new flexural cracks from both sides of the mid-span formed with a maximum crack width of 0.1 mm. By 270 kN of load, these two cracks were still below the mid-height of the beam, as shown in Figure 5.48.

When the load reached 282 kN, a major shear crack suddenly developed that caused the beam’s capacity to drop to 255 kN, as shown in Figure 5.49. The development of this shear crack was sudden, as there were no signs of such a crack prior to this load value. Originating halfway between the mid-span and the left support, this crack extended to about 80% of the height of the beam with the maximum crack width of 0.5 mm. The other existing cracks did not propagate further or increase in width.

After the drop in the load, the beam picked up capacity. Existing cracks were gradually widening and propagating at this stage but no new cracks formed. After about 340 kN of load, the crack width of the major shear crack increased rapidly, with hardly any change to the other existing cracks. The beam at 350 kN of load is shown in Figure 5.50. The maximum crack width at this stage was 1.4 mm, which was at the mid-height of the shear crack. Two new vertical flexural cracks formed, which brought the total number of cracks in the beam to six. Between 350 kN and 380 kN, no new cracks formed. The existing flexural cracks did not widen further nor progress further towards the loading plate; all the crack widening was happening at the major shear crack. Close to 390 kN, another shear crack formed next to the major shear crack but closer to the left support. However, the beam’s capacity was still increasing. After it reached the peak load of 397 kN, the beam’s capacity dropped to 366 kN because of the development of the other major shear crack extending from the edge of the left support up to the mid-height of the beam. The mid-span deflection at this stage was 5.0 mm.

After the beam’s capacity dropped to 366 kN, it never regained the peak load of 397 kN. The maximum load it reached after the capacity drop was 388 kN, which happened at a mid-span
deflection of 5.9 mm. At this stage, there was little propagation of existing cracks. Almost all of
the energy release was through the widening of the two major shear cracks. There was considerable
softening of the beam’s response after this stage with the beam’s response curve being close to
horizontal. At 376 kN of load and 8.2 mm of mid-span deflection, the maximum crack width at
was 5.5 mm, which occurred at the second major shear crack as shown in Figure 5.51. The crack
widths of the existing flexural cracks did not change, nor did they propagate further upwards
towards the loading plate.

After about 12.9 mm of mid-span deflection, there was a considerable drop in the beam’s capacity
from about 360 kN to 267 kN over a span of 1.2 mm of mid-span deflection. This was happening
as the two major shear cracks were widening considerably, with hardly any change to the existing
flexural cracks. At 255 kN of load and 14.3 mm of mid-span deflection, the second major shear
crack widened considerably to 20 mm as shown in Figure 5.52. The widening of the crack caused
further drops in the capacity of the beam. After this stage, signs of bond degradation appeared
close to the major shear crack. There was vertical cracking on the left side of the beam as shown
in Figure 5.53.

When the load reached 85 kN, the mid-span displacement was 22 mm. At this point, there was
complete degradation of the beam with the major shear crack widening to 50 mm as seen in
Figure 5.54. The tension hooks at the left end of the beam burst out as shown in Figure 5.55. At
the major shear crack, the fibres’ crack-bridging ability was not sufficient as can be seen in
Figure 5.56. The failure mode of this beam was a shear failure characterized by a diagonal tension
crack that disrupted the flow of internal forces and caused the drop in the capacity. Although there
were no signs of bond slip prior to the drop in the capacity of the beam, the bond slip that occurred
post-peak below 250 kN of load caused a significant widening of the major shear crack.
Figure 5.46: LFM at 195 kN

Figure 5.47: LFM at 230 kN
Figure 5.48: LFM at 270 kN

Figure 5.49: LFM at 282 kN
Figure 5.50: LFM at 350 kN

Figure 5.51: LFM at 376 kN after the drop from the peak load of 397 kN
Figure 5.52: LFM at 255 kN post-peak

Figure 5.53: Cracks at the left end of the beam indicating bond degradation for LFM at 200 kN post-peak
Figure 5.54: LFM at ultimate degradation and failure

Figure 5.55: Bursting of tension hooks at the left end of LFM

Figure 5.56: Major shear crack of LFM at the left support
5.6.2 Experimental Response

The experimental load versus mid-span deflection of LFM is shown in Figure 5.57. There are three linear portions in the response: before the onset of the first crack, prior to the first major shear crack, and prior to the second major shear crack. The beam exhibited the highest stiffness before the first crack which occurred at 195 kN. After that, the response was still linear until the first major shear crack, which occurred at 282 kN. After this point, the stiffness of the beam further reduced but the response maintained its linearity until the formation of the second major shear crack around 397 kN, which was the peak load of the response. After this peak, the beam’s stiffness greatly reduced as reflected by the softening portion of the response, which was almost horizontal. At around 13 mm of mid-span deflection, the widening of the major shear cracks caused a significant drop in the response of the beam, which was a drop of 80 kN over a displacement of only 1.4 mm. Typical of steel fibre-reinforced concrete (SFRC) beams, the beam showed considerable toughness in its post peak behaviour by coming gradually to its failure.

The load versus maximum crack width for LFM is shown in Figure 5.58. The majority of the crack width gain occurred in the post-peak response; prior to the peak load, the maximum crack width of the beam was 1.6 mm, which is only about 3% of the maximum crack width of 50 mm at failure. At 68% of the peak load, the maximum crack width was only 0.1 mm. In transitioning from the pre-peak to the post-peak regimes, the maximum crack width increased from 1.6 mm at around 390 kN to 4.5 mm at 376 kN. The mid-span deflection only increased by 3 mm during this transition. It is interesting to note that the maximum crack width increase in the post-peak regime is almost linear. Throughout the test, it was observed that the change in the maximum crack width increased as the effectiveness of steel fibres in crack-bridging decreased.
5.7 LFF Results

LFF had the same detailing as LFM: steel fibre-reinforced concrete (SFRC) with 0.75% fibre volumetric ratio and four 15M longitudinal rebar without any stirrups. Note that the concrete of
LFF is at least 23 MPa lower than those of the other beams subjected to fatigue loading, namely LTF and LLF.

The support conditions were the same as the other beams: simply supported with a point load at mid-span. The fatigue load that was applied (with a frequency of 0.6 Hz) was a sinusoidal pulsating load that ranged from 5 kN to 350 kN, which was 88% of the monotonic capacity of the same beam, as seen in LFM results. LVDTs were placed in both the left and right shear spans and strain gauges were put in certain locations along the longitudinal reinforcement, identical to the locations of those used with LLF.

5.7.1 Test Observations

After the application of the first load cycle, there was a total of six noticeable cracks on the beam as seen in Figure 5.59. Four of these cracks were steep flexural cracks around the mid-span area. There was another vertical crack on the right shear span of the beam. From the flexural cracks, there was one in the mid-span vicinity (more towards the left shear span) that extended to about 70% of the beam height towards the loading plate. It had a maximum crack width of 0.5 mm. The final sixth crack was a major diagonal shear crack in the left shear span running almost parallel to the LVDT L1, with an orientation of about 45°, and extended to about 300 mm away from the left support. It propagated to more than 75% of the height of the beam as shown in Figure 5.60. This was the major crack of the beam as the maximum crack width was 1.6 mm. As was the case with LFM, all of the stress redistribution and crack widening were occurring in the left shear span. As the number of load cycles was increasing, the major shear crack on the left was further widening and extending towards the loading support; the remaining cracks were neither widening nor propagating further. No new cracks formed on either shear span. After about 3850 cycles, signs of premature failure of the beam started appearing. The crack width of the major shear crack was widening at a faster rate than the previous cycles. The mid-span deflection started increasing considerably after every cycle as compared to the previous cycles. Cracks started appearing at the left support area.

After 4152 load cycles, the beam failed due to bond slip, which caused further opening of the major shear crack. At this stage, none of the bars fractured. The major shear crack reached a maximum crack width of 45 mm as shown in Figure 5.61. However, no crushing of the concrete was seen at the loading plate area. The other existing cracks did not widen or propagate further as
compared to the first cycle. The cracking at the major shear crack extended to the support area, which can be seen in Figure 5.62 when looking at the beam from the backside. There was also crushing of the concrete visible at the left support as seen in Figure 5.63. Finally, there were vertical cracks visible from the left side face of the beam, shown in Figure 5.64. All these were signs that the beam failed prematurely due to bond slip. The beam at this stage was not able to sustain the maximum fatigue load of 350 kN.
Figure 5.61: The major shear crack of LFF at failure (after 4152 load cycles)

Figure 5.62: The major shear crack of LFF at failure from the back of the beam
5.7.2 Experimental Response

The mid-span deflection versus the number of load cycles for LFF is shown in Figure 5.65. As was the case with the other beams subjected to fatigue loading, the majority of the deflection increase happened in the first few and last few cycles. In this case, the mid-span deflection was 1.4 mm after the first load cycle. By the second cycle, it has tripled to 4.6 mm. After about 25
cycles, the increase in the mid-span deflection became more gradual; over a span of 3850 cycles, the deflection only increased by 5 mm. However, in the last 300 cycles, it increased by more than 30 mm. The big increase in the deflection happened as the widening of the major shear crack and bond slip caused a significant reduction in the beam’s stiffness.

The horizontal, vertical, and shear strains for the left and right shear spans of LFF are shown in Figure 5.66 and Figure 5.67, respectively. The major shear crack in the left shear span caused both the horizontal and vertical strains in the same span to be tensile. A closer examination of these strains is shown in Figure 5.68. In the right shear span, the vertical and horizontal strains were also both tensile. However, the values of the strains as well as their increases in value in the left shear span were greater than those in the right shear span because of the major shear crack. Because the failure happened at the left side, it can be seen that the values of the strains in the left diverged significantly at failure. However, there was no divergence of strain values in the right shear span since there were no major shear cracks there. The failure on the left side did not affect the values on the other side.
Figure 5.66: Strains on the left shear span of LFF

Figure 5.67: Strains on the right shear span of LFF
The principal tensile and compressive strains are shown in Figure 5.69 and Figure 5.70, respectively. Due to the wide shear crack, the principal strains on the left shear span were both tensile. Even the average principal compressive strain was positive, i.e. tensile, although its value was close to zero. In addition, the principal tensile strain of the left shear span was greater than that of the right shear span. At failure, the values of the strains diverged. The change in strain values in the left shear span, especially that of the principal tensile stress, during the test was greater than those of the right shear span.
Figure 5.69: Principal tensile strain evolution of LFF

Figure 5.70: Principal compressive strain evolution of LFF

The values of the strain gauges along the longitudinal reinforcement are given in Figure 5.71. Note that the readings of some strain gauges after they malfunctioned were eliminated. All the strain
values were below the yield strain of the reinforcement; none of the bars yielded during the test. Some strain gauges (MB, LLB3, and RLB2) exhibited abrupt differences in the readings close to failure. The other strains gauges malfunctioned, unfortunately, before reaching failure. The longitudinal reinforcement had the largest increase in strains during the first few cycles, after which the strains stabilized and increased gradually. Note that for LFF, strain gauges RLB1, RLB2, and RLB3 were all placed along the left shear span while strain gauges LLB1, LLB2, and LLB3 were placed along the right shear span. MB was placed at mid-span. Strain gauge RLB1 was in the vicinity of the major shear crack, hence it had the largest strain values compared to the other gauges. At the beginning of the bond slip, its readings increased abruptly then dropped, as can be seen in the graph.

Figure 5.71: Strain gauges along the longitudinal reinforcement of LFF
Chapter 6: Discussion of Experimental Results

6 Discussion of Experimental Results

6.1 Introduction

This chapter analyzes and discusses the experimental results presented in the previous chapter. Load-deflection response plots, LVDT values, strain gauge readings, and experiment photographs will all be utilized in the analysis, which will focus on the relative behaviour of the beams during the tests, the progression of damage and cracking, the effects of reinforcement on the response, and failure mechanisms. The behaviours of the beams subjected to monotonic loading will be compared, followed by a comparison of the behaviours of the beams subjected to fatigue loading. The limitations and sources of errors are identified next.

6.2 Limitations and Sources of Error

During the experiments, strain gauges and LVDTs were used to record data at certain locations in order to gain more insight and make several conclusions about the behaviours of the beams. Although these tools enabled gathering more data, certain errors and limitations were associated with them. Knowing such errors and limitations will enable a more informed and accurate interpretation of the data gathered and a better appreciation of its limitations.

Strain gauges were placed at various locations along the longitudinal and transverse reinforcement in order to measure their strains. While these strain gauges provided useful information about the strains of the reinforcement at these locations, their readings were local: they only gave an indication of the strains at the specific locations, with no insight to the strains of the surrounding concrete. This meant that their readings were sensitive to local cracking conditions; if a crack cut through the concrete at one of the strain gauge locations, then the strain reading of that gauge would be amplified. In addition, if a strain gauge at a stirrup intersected one of the major shear cracks, it would show significantly increased strain values indicative of high shear stresses at this location. However, the rest of the stirrup was not necessarily under the same level of stress. More representative readings were obtained from the strain gauges when the cracks were well distributed. Strain gauges were also sensitive to the internal stresses in the beams during the test,
especially since their wiring had to be safely channelled out of the beam; many of them malfunctioned after certain levels during the tests.

The LVDTs were used during fatigue tests to capture the evolution of strains in both shear spans of the beam. Three LVDTs were installed in each span in order to be able to obtain the principal strains. The gauge zone of the LVDTs in each span was a square element measuring 880 mm x 880 mm. There was the inevitable human random error of laying out the dimensions of this element and aligning the LVDT brackets. During the tests, the bending of the beams might have also slightly affected the LVDTs’ alignment. In addition, the cracks that passed through the bolted LVDT brackets could have affected their readings. However, all these possible sources of error for the LVDTs were deemed negligible enough to obtain accurate readings. It is important to note that the strain readings of the LVDTs represented only the average strain values measured over their lengths.

There were two main factors in the experimental work that provided certain limitations in the analysis. First, there was a considerable difference in the compressive strengths of the SFRC and the plain concrete beams, despite both having a specified compressive strength of 50 MPa. The SFRC beams (Set 3) had an 28-day strength of 57 MPa while that of the plain concrete beams (Set 1 and Set 2) was 78 MPa. The plain concrete beams were about 27% stronger than the SFRC beams. This difference in strength requires certain considerations when directly comparing the responses of the fibre-reinforced and plain concrete beams. For instance, a lower observed strength of SFRC beams does not necessarily mean that the fibres were less effective than the stirrups. The reduced strength could well be attributed to the weaker concrete matrix. All the factors and parameters of the experimental beams need to be considered before such comparisons and conclusions can be made.

The second factor complicating the analysis is the premature failure of the SFRC beam subjected to fatigue loading, namely LFF, at considerably less load cycles than the other beams. It will be difficult to directly compare the performance of steel fibres under fatigue loading to stirrups and plain concrete. Other methods, such as comparing the strain evolution and utilizing finite element modelling, must be employed. Again, all the parameters of the beams must be carefully examined before reaching a conclusion. Overall, the limitations and errors mentioned did not prevent from making insightful and practical inferences and conclusions from the experimental data.
6.3 Comparison of Beams Subjected to Monotonic Loading

Three beams were subjected to monotonic loading: LLM containing plain concrete with no stirrups, LTM with 0.15% reinforcement ratio of stirrups, and LFM with 0.75% volumetric ratio of steel fibres. All of the beams contained four 15M longitudinal bars, amounting to a reinforcement ratio of 0.38%. LLM and LTM had concrete strengths of about 79 MPa while LFM’s concrete strength was 57 MPa. Figure 6.1 and Figure 6.2 show the experimental response of these beams and the evolution of their maximum crack widths, respectively. Upon examination of the response of the beams and the cracking patterns and failure modes (and later confirmed by finite element modelling), it was clear that the transfer of internal stresses in the beams was through arching action rather than beam action. The a/d ratio of the beams was slightly below 2.0, which supports this conclusion.

![Experimental Response of Beams Subjected to Monotonic Loading](image)

Figure 6.1: Experimental response of beams subjected to monotonic loading
The cracking load of LLM was 206 kN, after which its stiffness reduced but the response was still fairly linear up until the load reached about 300 kN. In this range, the majority of the cracks were still flexural cracks, with no major shear cracks visible yet. The drop in the capacity after 300 kN was due to the formation of shear splitting cracks that started out as diagonal cracks but quickly became almost horizontal running parallel to the reinforcement. After the drop in capacity, there was a reorientation of internal forces in the beam that has enabled it to pick up capacity shortly after; there was an increase in the load from 280 kN to about 386 kN in a linear manner, indicating that the steel has not yielded yet. The slope of this linear portion was a combination of the reduced stiffness of the concrete due to the flexural and shear cracks and the elastic modulus of the steel. After 386 kN, the tension ties yielded, which caused a plateau in the curve. This meant that the ties would exhibit more strain energy before taking more load, i.e. their stiffness reduced considerably, which is typical at the strain hardening stage. This meant that the plasticity of the ties reduced their crack-arresting ability, which can be seen in the big increase of the maximum crack width at the major crack after 400 kN and the spalling of the concrete at the ties location; more than 80% of the maximum crack width value at failure was incurred during the plateauing stage. During the plateauing stage, the concrete compressive struts were able to transfer more force to the supports, which allowed the ties to strain harden without failing the beam. As the strain in the ties was increasing, the maximum crack width of LLM was also increasing. This meant that
there was less shear friction between the cracks along the compressive struts, eventually leading to crack slip. The beam failed due to the disintegration of the compressive strut that disrupted the flow of internal stresses from the loading plate to the supports.

The cracking load of LTM was 186 kN, which was about 10% less than that of LLM. This was expected since LLM was tested 95 days after testing LTM, so the beam gained some strength meanwhile. Similar to LLM, LTM’s response after cracking remained linear, but with a lower stiffness (slope) post-cracking. However, there was no drop in strength prior to reaching the plateauing stage; the beam picked up strength in a linear manner up to about 390 kN, after which the response plateaued. At 270 kN, shear cracks became visible, which was at 10% lower load than the 300 kN load of LLM at which shear cracks became visible. This was expected since there was about 10% difference in the cracking load. The relative similarity in these loads show that the stirrups used with LTM became effective only after the formation of the shear cracks; the presence of transverse reinforcement prior to that stage was largely irrelevant. One major difference in the shear cracks between LLM and LTM was that the majority in the former were pure shear cracks propagating diagonally from the bottom of the beam, while most of those in the latter were flexural-shear cracks. The presence of the stirrups enabled LTM to pick up strength up to the plateauing stage without any drop in capacity due to diagonal cracking. These stirrups enabled more stresses to be transferred across the diagonal crack and arrested the cracks’ growth to prevent shear slip. The crack growth arrest abilities of the stirrups can be clearly seen in Figure 6.2; prior to plateauing of the response, the maximum crack width of LTM was only 30% of that of LLM, which is less by an amount of more than 5 mm, at the same corresponding load. The maximum crack width of LTM at failure was 50% of that of LLM, despite the former having 30 mm more mid-span deflection. The propagation of cracks in LTM, especially prior to plateauing, was also more stable than that of LLM. From Figure 6.1, it can be seen that both beams plateaued at around the same load value. However, the corresponding mid-span deflections were considerably different: LTM plateaued at 6 mm, which was only 40% of that of LLM. This suggests that the stirrups helped reduce the degradation of the stiffness of the beam. After plateauing, both beams had almost the same rate of strain hardening. However, since LTM plateaued at a lower mid-span deflection value and did not fail until 30 mm after the failure of LLM, it was able to pick up more capacity. In other words, the stirrups delayed the failure of the compression strut by transferring stresses across the diagonal cracks, which allowed the beam to pick up more capacity as the tension
ties were strain hardening. However, the increased propagation and widening of cracks after the yielding of the longitudinal bars caused the cracks in the compression struts to extend all the way to the compression face at the loading plate and eventually crush the concrete, resulting in the failure of the beam. It is interesting to note that the widest cracks in both beams were steeper than the regular 45-degree shear cracks. This limited the contribution of the stirrups to the capacity of the beams as there was a lesser force component crossing these cracks, explaining why the addition of the stirrups increased the capacity of the beam by only about 10%; LTM had a capacity of 497 kN, compared to the capacity of 450 kN for LLM. Using stirrups in more slender beams would be more effective since the stirrups would better bridge the more shallow (i.e. less steep) cracks typically found in such beams.

The effect of the steel fibres on the first cracking load can be clearly seen in LFM. It first cracked when the load value was 195 kN, which is between the load values of LTM and LLM, despite having a concrete compressive strength that was lower by at least 20 MPa from both other beams. Unlike stirrups, fibres became effective even before the first crack, by bridging the micro-cracks and shrinkage cracks found in the concrete. Notwithstanding, the amount of fibres used in the beam (0.75% volumetric fraction) is considered to be the minimum allowed fraction for the structural use of fibres. After cracking, the response of the beam was still linear, despite the lower stiffness. The number of cracks that formed on the beam was considerably fewer than those formed in LLM and LTM; at any given load, the number of cracks was less than half of those that formed in the other two beams. The crack widths were also considerably lower. Prior to 282 kN of load, the maximum crack width of LFM was 0.1 mm, which is only 20% of that of LTM at the same load, as can be seen in Figure 6.2. At 282 kN, a major shear crack developed that caused the beam’s capacity to drop about 20 kN. At this stage, the fibres restrained this crack from widening and causing a failure in the beam; the principal tensile stresses at the crack were less than the stresses that would cause the fibres to pull out, hence the fibres were effective in bridging it. This enabled the beam to pick up more capacity, by maintaining the shear stress transfer through aggregate interlock, in a linear manner with a slope (i.e. stiffness) not much less than that after the first cracking load. The linear portion of the response continued until the beam reached its maximum capacity, 397 kN, as can be seen in Figure 6.1. At this load, a second major shear crack developed that caused a drop in the response. However, this drop happened gradually as the response plateaued. The plateauing of the response can be attributed to a combination of factors. At the
maximum load, it is very likely that the longitudinal steel yielded, which caused considerable degradation in the beam’s stiffness, causing the cracks to open up at a faster rate, as is seen in Figure 6.2 when the maximum crack width increased substantially after the maximum load of 397 kN. The possibility of yielding of the reinforcement is further confirmed by the yield of the reinforcement for the other two beams at similar load values; the plateauing of the response for all three beams happened around the same load range. The formation of the second major shear crack further degraded the concrete to a point where it could no longer transfer the required amount of force across the cracks (to the supports) in order to gain capacity; there is no possible reorientation of the internal stresses that would facilitate an increase in the load. Finally, after the maximum load value, the fibres started to pull out from the concrete at the major shear cracks. The energy absorbed by the beam as the fibres were pulled out of the concrete was what caused the considerable toughness of the response typical of steel fiber reinforced concrete. As the fibres were being pulled out of the concrete, they were becoming less effective in bridging the concrete cracks, which explains the considerable widening of the cracks after the maximum load. The maximum load of LFM was 89% of that of LLM (only plain concrete) and 80% of that of LTM (with 0.15% stirrups), despite its concrete being more than 27% weaker than the concrete of the other beams, which shows the strength enhancement brought by the fibres. However, the lower strength of the concrete did not allow the beam to experience enough deflection to guarantee strength gain through strain hardening of the reinforcement, contrary to the behaviour of the other two beams. Table 6.1 compares the critical values of the responses of LLM, LTM, and LFM during various stages of the tests. The final column normalizes the maximum shear stress sustained by each beam with respect to the corresponding beams concrete compressive strengths; the equal values for LTM and LFM suggest that the fibres’ contribution to the strength was comparable to that of the stirrups.

Table 6.1: Summary of beams subjected to monotonic loading

<table>
<thead>
<tr>
<th>Beam</th>
<th>f’c (MPa)</th>
<th>ρ_v</th>
<th>V_f</th>
<th>Cracking load, P_cr (kN)</th>
<th>First plateau load (kN)</th>
<th>Max. load, P_u (kN)</th>
<th>Ultimate displacement (mm)</th>
<th>Failure mode</th>
<th>V_{cr}^*</th>
<th>V_{u}^*</th>
</tr>
</thead>
<tbody>
<tr>
<td>LLM</td>
<td>78</td>
<td>-</td>
<td>-</td>
<td>206</td>
<td>390</td>
<td>451</td>
<td>82</td>
<td>Flexural-shear</td>
<td>0.06</td>
<td>0.13</td>
</tr>
<tr>
<td>LTM</td>
<td>76</td>
<td>0.15%</td>
<td>-</td>
<td>186</td>
<td>391</td>
<td>500</td>
<td>109</td>
<td>Flexural</td>
<td>0.05</td>
<td>0.14</td>
</tr>
<tr>
<td>LFM</td>
<td>54</td>
<td>-</td>
<td>0.75%</td>
<td>195</td>
<td>397</td>
<td>397</td>
<td>24</td>
<td>Diagonal splitting</td>
<td>0.07</td>
<td>0.14</td>
</tr>
</tbody>
</table>

* V_{cr} = P_{cr}/2, V_u = P_{u}/2
6.4 Comparison of Beams Subjected to Fatigue Loading

Three beams were subjected to fatigue loading: LLF, LTF, and LFF. All of these beams had the same detailing as their monotonic counterparts; LLF contained only plain concrete, while LTF and LFF contained stirrups ($\rho_v = 0.15\%$) and steel fibres ($V_f = 0.75\%$), respectively. As before, LLF and LTF had a concrete compressive strength of 78 MPa while that of LFF was 57 MPa. All beams contained the same amount of longitudinal reinforcement. Figure 6.3, Figure 6.4, and Figure 6.5 show the evolution of principal tensile strains, principal compressive strains, and the mid-span deflection with the number of cycles, respectively.

![Evolution of Average Principal Tensile Strains for Beams Subjected to Fatigue Loading](image)

**Figure 6.3:** Evolution of principal tensile strains for beams subjected to fatigue loading
As can be seen in the evolution of the strains for the beams subjected to fatigue loading, the biggest increase in strain happened after the first cycle. The crack propagation during the first few cycles was rapid, with almost all the cracks in the beams forming at this stage. Between the first and last
few load cycles, the beam exhibited very gradual strain and deflection increases, which is seen in the almost flat lines in the strain and deflection curves in Figure 6.3, Figure 6.4, and Figure 6.5. The irreversible fatigue strain (and deflection) can be calculated by subtracting the strains after the first cycle from the strains at the stage just prior to the divergence of the values. The crack propagation between the first and last few cycles stabilized with less energy released during loading. There was little or no formation of new cracks at this stage. Rather, the existing cracks widened steadily. Local failure by stress concentration at one crack was prevented at this stage by the stress redistribution that happened in the beams, which enabled them to withstand more load cycles. It can be seen in the strain evolution graphs above that the strain values decreased slightly at various points between the first and last cycles, which is attributed to the stress redistribution. At some points, stress redistribution even caused some cracks to decrease in width (i.e. ‘close up’). Towards the last cycles, there was stress concentration at the major crack for all the beams. The higher degradation and crack width at the major crack, compared to the other cracks, caused more stress concentration. At this point, stress redistribution was not possible since the major crack was widening significantly, and thus the beams failed. LLF and LTF both failed by the fracture of the longitudinal reinforcement crossing the major crack, while LFF failed prematurely by bond slip.

LLF failed after 36748 load cycles. Comparing the first and last cycles, the cracking patterns of the beam at failure were evident from the first load cycle: a major vertical crack extending towards the loading plate at mid-span connected to splitting cracks running parallel to the longitudinal reinforcement. This was the same cracking pattern as the beam’s monotonic counterpart. As the load cycles progressed, the crack widened and gradually propagated. Closer to failure, the splitting cracks at the bottom of the beam caused significant spalling of concrete close to the reinforcement, which exposed the reinforcement. As is the case with all concrete beams, the exposed reinforcement (or the reinforcement intersecting the cracks) is subjected to increased local stresses at that location since there is no concrete to transfer some portion of the stress through tension stiffening; the entirety of the force has to be transmitted by the reinforcement. Therefore, it is no surprise that the fracture of the longitudinal reinforcement for beams subjected to fatigue always happens at the intersection of the reinforcement with the major crack. However, when there are many dominant cracks, it is not clear which crack will ultimately govern the behaviour of the beam; stress redistribution can render an actively widening crack dormant and transfer the stresses to another crack at which the reinforcement will fracture. The high strength of the concrete
protected LLF from failing by the crushing of the concrete. The beam could have experienced the fatigue failure of the concrete (i.e. shear-compression failure) if the maximum applied fatigue load was above the fatigue limit of the concrete but below that of the steel. Even with fatigue loading, the load transfer mechanism was through arch action; the ultimate failure was the disintegration of this mechanism by the fracture of the tension ties.

Despite having 0.15% ratio of stirrups, LTF failed after 37316 load cycles, which was only 1.5% more load cycles than LLF; adding stirrups to the beam did not significantly improve its fatigue life. The ultimate failure was caused by the fracture of the longitudinal stirrups located in the vicinity of the mid-span region, similar to LLF. However, there were no splitting cracks running parallel to the reinforcement, but rather shear-flexural cracks extending towards the loading plate. The addition of the stirrups did not extend the fatigue life for two main reasons. First, the nature of the cracks observed in deep beams is steeper than that of those prevalent in more slender beams, i.e. the cracks in deep beam are closer to the vertical than the horizontal. This significantly reduces the contribution of the stirrups since there is now less component of the stirrups that can transfer the principal tensile stresses running perpendicular to the cracks. The more parallel the stirrup is to the diagonal shear crack, the less its effectiveness. The major cracks observed in both LLF and LTF (i.e. those with the highest crack widths) were steeper in nature and exceeded 45 degrees, approaching the vertical. Therefore, the stirrups did not provide the resistance necessary to improve the fatigue life of LTF. Second, the arching mechanism responsible for the transfer of stresses in deep beams induces clamping stresses in the vicinity of the stirrups caused by the compressive struts. These stresses reduce the tensile strains (and stresses) that can be carried out by the stirrups across the shear crack, which limits the stirrups’ contribution. In the case of LTF, a failure mode by the fracture of the stirrups would have been possible if the amounts of both longitudinal reinforcement and stirrups were increased. This way, the increased number of longitudinal bars would result in less stress on each bar, thus increasing the number of cycles to failure. The increased number of stirrups would increase the likelihood of the stirrups intersecting the major cracks, which would make them more active in resisting the fatigue loads. The strain gauges placed on the stirrups at different location across both shear spans of the beams showed that none of the stirrups yielded, which confirms their limited contribution.

Despite the inability to increase the fatigue life of LTF, the stirrups were indeed effective in bridging cracks; the maximum crack width for LTF at failure was 15 mm, which was only 30% of
that of LLF. The principal compressive and tensile strains of LTF were also considerably less than those of LLF, being 4.3 times and 9.0 times less, respectively. The addition of the stirrups has also increased the stiffness of LTF, which exhibited 20% less mid-span deflection than LLF. The increase in stiffness is an indirect effect of stirrups, since they bridge the shear cracks that form on the beam, which reduces the degradation of the beam’s stiffness. By examining the graphs above, it can also be seen that the rate of degradation of LTF, which can be measured by the slope of the lines of the strain and deflection evolutions between the first and last few cycles, is less than that of LLF.

LFF, containing 0.75% steel fibres, exhibited less principal strains and deflection values when it was loaded compared to both LLF and LTF. Although the maximum fatigue load applied was 88% of the fatigue loads applied on LLF and LTF, the concrete strength of LLF was 27% lower. Therefore, the contribution of the fibres in reducing the extent of the propagation of cracks and lowering the values of the principal strains can be clearly seen; LFF’s principal tensile and compressive strains were 50% and 100% lower than those of LTF after the first load cycle, respectively. The crack-bridging ability of the fibres restrained the cracks from widening significantly. The contribution of the fibres with regards to restraining the strains is greater than that of the stirrups. This is due to the random orientation of the fibres; they are more likely to cross cracks at different orientations compared to the stirrups. In addition, they can also bridge out-of-plane cracks. The orientation, extent, and location of the cracks are somewhat irrelevant when fibres are present since such fibres are randomly distributed. On the other hand, the contribution of the stirrups is greatly dependent on the orientation and location of the cracks; there is a possibility that a crack may develop between stirrups, which reduces their contribution. These characteristics of fiber reinforced concrete make steel fibres a superior alternative to stirrups in increasing the fatigue life of reinforced concrete beams. It was seen from this set of experiments, both under monotonic and fatigue loading, that the crack bridging abilities of steel fibres are superior to those of the stirrups; the fatigue failure of concrete in compression was reduced by the steel fibers absorbing a major fraction of the fatigue load energy. In addition, for beams that will fail by the fracture of the longitudinal reinforcement in fatigue, adding a sufficient amount of steel fibres will reduce the size of the major crack at the location where the reinforcement may potentially fracture. In addition, the steel fibres crossing, or bridging, that crack will take part in transferring some stresses across the crack, relieving the reinforcement of a portion of the stress.
that it has to transfer. As a result, the total amount of the stress that the reinforcement carries decreases, which increases its fatigue life. Nonetheless, it has to be ensured that there is a sufficient amount of fibres to generate the required crack bridging and stress transfer abilities. Good anchorage of fibres is also necessary to ensure that they will not de-bond prematurely during fatigue loading. As such, hooked fibres are preferred to straight ones. Finally, the increased toughness brought by the fibres will also help increase the fatigue life of the beam by enabling it to absorb the energy required by fatigue loading.

Despite containing steel fibres, LFF failed prematurely after 4152 load cycles, which is only 11% of the number of load cycles it took LTF to fail. Unlike the other two beams subjected to fatigue loading, the failure was due to bond slip of the longitudinal bars; there was neither a fracture of any bar nor the crushing of concrete. The strain gauges also showed that none of the bars yielded during the test, which made them unlikely to fracture under the imposed loading. The steeper curves for LFF compared to LTF and LFF in Figure 6.3, Figure 6.4, and Figure 6.5 are indicative of the bond slip. The poor anchorage of the bars due to problem encountered during casting led to the bond failure. During casting of the beams containing fibres, it was erroneously decided that the fresh concrete in the beams’ formwork did not need any mechanical vibration since it was dropped from a height of more than one and a half meters and exhibited a slump of about 190 mm. Vibration was avoided because it was feared that the vibrator might have disrupted the even distribution of fibres. This caused numerous small internal and external voids in the concrete. Some portions of the bars, especially at the tension hooks, where exposed. High-strength non-shrink grout was used to fill the voids that were seen. Even though the fibre-reinforced beam subjected to monotonic loading failed as expected, the increased demand on the bond brought on by the fatigue loading caused LFF to fail prematurely; the concrete was not fully bonded to the ribs of the reinforcement. The voids also caused to beam to crush at the support area. This premature failure has unfortunately made it more challenging to directly demonstrate how effective the fibres were in increasing the fatigue life of reinforced concrete beams as compared to stirrups. Fortunately, the contribution of the fibres to the evolution of the principal strains (which are good indicators of fatigue performance) can be seen in the graphs showing the response prior to failure.

The challenges faced during this experimental work with casting steel fiber reinforced concrete (SFRC) is a call for the concrete industry to adjust some of their equipment and practices in order
to make steel fibres a more viable alternative to conventional reinforcement. Currently, various talks and discussions held with industry experts confirmed that steel fibres are predominantly used for non-structural crack control in some types of floors, especially industrial flooring. The power and mixing mechanism of the available truck mixers make it possible to only use a dosage of fibres that does not exceed 40 kg per cubic meter of concrete, which amounts to a volumetric ratio of 0.5%. Mixing trucks with a higher rotation power and more dispersive mixing paddles are required to produce acceptable SFRC mixes and avoid balling of fibres. This way, the need for small mixers to mix the fibres with the concrete coming from the truck is eliminated. Producing efficient SFRC mixes can reduce the cost of the project by eliminating the labor cost associated with detailing and laying out the stirrups as well as the material cost of the stirrups.

In comparing the monotonic and fatigue behaviour of the beams tested, it can be seen that there is no correlation between the monotonic and fatigue modes of failures. If a beam fails in shear under monotonic loading, it can very well fail in flexure under fatigue loading. Two of the beams failed due to the fracture of the longitudinal bars under fatigue loading despite them having different failure modes under monotonic loading: one failed in shear-compression and the other failed due to diagonal splitting. This point further reinforces the complexity of fatigue loading. All the factors of the experiment, ranging from the beam detailing to the parameters of the loading, have an effect on the fatigue life of the beam as well as on its mode of failure. Care has to be taken in choosing the control variables. Table 6.2 shows summarizes the results of the beams subjected to fatigue loading.

<table>
<thead>
<tr>
<th>Beam</th>
<th>f'c (MPa)</th>
<th>Shear reinforcement type</th>
<th>Shear reinforcement ratio</th>
<th>Maximum fatigue load (kN)</th>
<th>Number of load cycles to failure</th>
<th>Failure mechanism</th>
</tr>
</thead>
<tbody>
<tr>
<td>LLF</td>
<td>84</td>
<td>None</td>
<td>-</td>
<td>400</td>
<td>36700</td>
<td>Fracture of rebar</td>
</tr>
<tr>
<td>LTF</td>
<td>78</td>
<td>Stirrups</td>
<td>0.15%</td>
<td>400</td>
<td>37300</td>
<td>Fracture of rebar</td>
</tr>
<tr>
<td>LFF</td>
<td>57</td>
<td>Steel Fibres</td>
<td>0.75%</td>
<td>350</td>
<td>4100</td>
<td>Bond slip</td>
</tr>
</tbody>
</table>

Table 6.2: Summary of beams subjected to fatigue loading
Chapter 7: Finite Element Modelling

7 Finite Element Modelling

7.1 Introduction

Finite element modelling is a practical tool to verify the accuracy of the reinforced concrete fatigue models developed at the University of Toronto (Isojeh et al. 2016 and 2017) and assess their ability in predicting the response of large-scale reinforced concrete deep beams, such as the ones tested in this project. As indicated previously, the proposed models treat concrete as a continuum material, taking into account the interaction of the concrete and the reinforcement, and taking into account the irreversible fatigue strains and damage. Finite element modelling provides a convenient way to check for the fatigue resistance of structures without resorting to overly conservative empirical code equations.

The finite element modelling in this chapter was performed using VecTor2, a nonlinear finite element software developed at the University of Toronto and dedicated to modelling two dimensional reinforced concrete membrane elements subjected to in-plane normal and shear stresses. It models concrete as an orthotropic material with smeared, rotating cracks, and utilizes an incremental total load, iterative secant stiffness algorithm to produce realistic results that include the post-cracking behaviour, cracking patterns, and failure mode of the structure (Wong, Vecchio, & Trommels, 2013). The underlying analytical models for the software are the Modified Compression Field Theory (MCFT) and the Disturbed Stress Field Model (DSFM), which incorporate compatibility, equilibrium, and constitutive relationships for cracked reinforced concrete, and are able to assess the local stresses transferred across the cracks.

A finite element model of each test beam was developed and subjected to the same loading conditions as imposed during testing. First, different aspects of building the model will be discussed. Then, the analyses and results will be presented.
7.2 Finite Element Models

7.2.1 Meshing

The finite element (FE) mesh was constructed using FormWorks, a pre-processor for VecTor2. Plane stress rectangular elements were used to model concrete. An aspect ratio of 1.0 was used for these elements in order to avoid potential convergence problems. The depth of the beams was covered by 26 elements, with each element having a size of 40 x 40 mm. In total, there were 2647 rectangular elements representing concrete for each beam, which ensured that cracking patterns and stress contours were accurately captured. Employing a high number of low-powered rectangular elements, as compared to higher order elements, ensured computational efficiency without sacrificing accuracy. Only one type of material was used for concrete in each test beam, since the concrete and thickness were constant across the beam.

All steel reinforcement, both longitudinal and transverse, was modelled by truss elements. The low number of stirrups used made it easy to model them discretely and obtain more accurate results as compared to smearing them. However, for LFM and LFF, the fibres were modelled as smeared reinforcement since they were uniformly dispersed within the concrete. Since there was sufficient anchorage provided for the reinforcement bars and the results of the tests (for LLF and LTF) did not show any slippage of bars, perfect bond was assumed between the reinforcement and the concrete. The bond slip that happened in LFF was due to poor consolidation of concrete rather than insufficient anchorage, hence the bond in this beam was also assumed to be perfect. Two types of reinforcement were used: 15M bars for the longitudinal reinforcement and 10M bars for the stirrups and the compression reinforcement. The FE meshes of the test beams are shown in Figure 7.1 and Figure 7.2. Note that all the material parameters of concrete, steel bars, and steel fibres were obtained from the actual material tests and specifications.

The loading and bearing plates at the mid-span and supports were modelling using the unidirectional bearing material type. Structural steel material type could have also been used, as it was seen in the results that it did not restrain the concrete beneath it from bending or cause it to split; the stresses were transferred in the expected manner from the mid-span to the supports.
7.2.2 Support Conditions

All the beams were simply supported, so there was restraint in the x- and y-directions at one of the supports and a restraint in the y-direction only at the other. To avoid creating bending restraints, nodal support restraints were created at only one point for each support.

7.2.3 Loading Conditions

7.2.3.1 Beams Subjected to Monotonic Loading

The loading test for LLM, LTM, and LFM was displacement controlled. Therefore, a downward unit displacement was imposed at the mid-span to simulate the actual loading condition. Throughout all the monotonic loading simulations, the initial displacement factor used was 0 mm, with an increment of 0.1 mm. For every displacement, the corresponding load value of the beam was determined, producing the load-deflection curve.
7.2.3.2 Beams Subjected to Fatigue Loading

For the beams subjected to fatigue loading (LLF, LTF, and LFF), the tests were load-controlled with the minimum and maximum fatigue loads set. In VecTor2, a special provision called “fatigue” was added to the job file in which one can choose to consider fatigue loading and edit its parameters, as shown in Figure 7.3 and Figure 7.4. In the fatigue load parameters, six inputs are required: number of load cycles, loading frequency, loading ratio, fatigue waveform, permissible error, and interval of loading cycles. For sinusoidal loads, a waveform value of 0.15 should be input. A suitable permissible error value is 0.01. The interval of loading cycles refers to the interval after which the software assesses the fatigue damage. Isojeh recommends this value to be 0.01 times the number of load cycles. This way, the increased computational efficiency would not sacrifice accuracy.

Figure 7.3: Fatigue special provision in VecTor2 job file
For all the beams subjected to fatigue loading, a downward nodal load was placed at the mid-span. The initial value of this load was set to the maximum fatigue load, since the software assesses the fatigue damage on the beam at the first load cycle, after which the beam is loaded monotonically to failure. The value of the load at failure represents the residual capacity of the beam after fatigue damage. Consequently, the fatigue life of the beam is determined when its residual capacity is equal to the maximum fatigue load, i.e. when it cannot be loaded further than the applied fatigue loading. Therefore, different simulations, representing different fatigue load cycles, were performed for each beam in order to obtain its fatigue life.

In order to accurately reflect the fatigue damage of the beams, the ratio of the maximum fatigue load used in the models to the analytical monotonic capacity of each beam matched the experimental ratio used. This meant that the maximum fatigue load used in modelling was not necessarily the same as that of the experiment. However, in the case where the experimental and analytical monotonic capacities of the beams were similar (i.e. within 5%), the same maximum fatigue load was used in modelling. Table 7.1 shows the maximum fatigue load used with every
beam. For each beam, a load step size of 0.5 kN was used in the simulations. In order to capture the failure point more accurately, the step size was reduced to 0.1 kN close to the failure point.

Table 7.1: Fatigue loads used in modelling

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Experimental Monotonic Capacity (kN)</th>
<th>Experimental Max Fatigue Load Used (kN)</th>
<th>Analytical Monotonic Capacity (kN)</th>
<th>Analytical Max Fatigue Load Used (kN)</th>
<th>Max Fatigue Load / Monotonic Capacity (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>LLF</td>
<td>450</td>
<td>400</td>
<td>473</td>
<td>410</td>
<td>88%</td>
</tr>
<tr>
<td>LTF</td>
<td>500</td>
<td>400</td>
<td>511</td>
<td>400</td>
<td>80%</td>
</tr>
<tr>
<td>LFF</td>
<td>400</td>
<td>350</td>
<td>410</td>
<td>350</td>
<td>85%</td>
</tr>
</tbody>
</table>

7.2.4 Analytical Models

Although VecTor2 provides a variety of models for each aspect of the response, special attention must be paid to the applicability and suitability of the various models; models that are appropriate for certain concrete strengths and loading conditions, for example, might not be suitable for other cases. In order to directly compare the performances of the test beams through finite element analysis, the same set of analytical models, listed in Table 7.2, were used for all the test beams. All of these models were the default models except for the concrete compression pre- and post-peak models. Since the strength of the concrete for the beams was higher than 50 MPa, it was more appropriate to use Popovics model for high strength concrete for both pre- and post-peak behaviour. These models were used with both monotonic and fatigue loading.
### Table 7.2: Analytical models used with the test beams

<table>
<thead>
<tr>
<th>Behaviour</th>
<th>Model Used</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Concrete Models</strong></td>
<td></td>
</tr>
<tr>
<td>Compression pre-peak</td>
<td>Popovics (HSC)</td>
</tr>
<tr>
<td>Compression post-peak</td>
<td>Popovics/Mander</td>
</tr>
<tr>
<td>Compression softening</td>
<td>Vecchio 1992</td>
</tr>
<tr>
<td>Tension stiffening</td>
<td>Modified Bentz 2003</td>
</tr>
<tr>
<td>Tension softening</td>
<td>Linear</td>
</tr>
<tr>
<td>Confined strength</td>
<td>Kupfer / Richart</td>
</tr>
<tr>
<td>Dilation</td>
<td>Variable – Orthotropic</td>
</tr>
<tr>
<td>Cracking criterion</td>
<td>Mohr-Coulomb (Stress)</td>
</tr>
<tr>
<td>Crack stress calculation</td>
<td>Basic (DSFM/MCFT)</td>
</tr>
<tr>
<td>Crack width check</td>
<td>Agg./2.5 Max Crack Width</td>
</tr>
<tr>
<td>Crack slip calculation</td>
<td>Walraven</td>
</tr>
<tr>
<td>Hysteretic Response</td>
<td>Nonlinear with Plastic Offsets</td>
</tr>
<tr>
<td><strong>Reinforcement Models</strong></td>
<td></td>
</tr>
<tr>
<td>Hysteretic response</td>
<td>Bauschinger Effect (Seckin)</td>
</tr>
<tr>
<td>Dowel action</td>
<td>Tassios (Crack Slip)</td>
</tr>
<tr>
<td>Buckling</td>
<td>Akkaya 2012</td>
</tr>
<tr>
<td>Concrete Bond</td>
<td>Eligehausen</td>
</tr>
<tr>
<td><strong>Analysis Models</strong></td>
<td></td>
</tr>
<tr>
<td>Strain History</td>
<td>Previous Loading Considered</td>
</tr>
<tr>
<td>Geometric Nonlinearity</td>
<td>Considered</td>
</tr>
<tr>
<td>Cracking Spacing</td>
<td>CEB-FIP 1978 – deformed bars</td>
</tr>
</tbody>
</table>

### 7.3 Results and Analysis

The results, including the load-deflection responses and cracking patterns, were viewed using Augustus, a post-processor for VecTor2 developed by Professor Evan Bentz.

#### 7.3.1 Beams Subjected to Monotonic Loading

##### 7.3.1.1 LTM

The analytical and experimental load-deflection curves for LTM are shown in Figure 7.5. Initially when the strength increased linearly, both responses had very similar stiffness values seen by the almost parallel linear loading curve. The plateauing of the curves, at which the longitudinal bars yielded, happened analytically at a load of 422 kN, which was 6% higher than the corresponding experimental load. Between 10 and 80 mm of mid-span deflection, both curves experienced plateauing. However, the strain hardening of the reinforcement experimentally at this interval was more pronounced than in the analytical model. After 80 mm, the model showed significant strain...
hardening. The maximum load value for both responses happened at the ultimate deflection value. The analytical maximum load was 508 kN, which was only 2% higher than the experimental load. Both responses showed a shear-compression failure in which the shear cracks extended to the extreme compression face at the mid-span where the concrete crushed. The cracking patterns of the model and the test beam are compared in Figure 7.6. It can be seen that VecTor2 accurately captured the cracking patterns due to the applied loading, with the major cracks being more vertical in nature. Examining the principal compressive stress contours of the model, shown in Figure 7.7, confirmed that the load transfer in the beam was primarily through arching mechanism. Despite accurately predicting the maximum load, failure mechanism, and cracking patterns of the beam, VecTor2 overestimated the ductility of the beam considerably, with an ultimate deflection value 42% more than the corresponding experimental value.

![Load vs. Mid-Span Deflection Response for LTM](image)

Figure 7.5: Experimental and analytical responses for LTM
7.3.1.2 LLM

When compared with the experimental response, the FE analysis results for LLM showed more stiffness during the linear portion of the graph, as shown in Figure 7.8, despite having provided the actual stiffness value from the material test. Even after the first flexural crack, there did not seem to be an observable degradation in the stiffness. After the analytical response reached its peak, which coincided with the peak of the linear portion of the curve, a major vertical crack
connected to a splitting crack caused a drop of more than 35% in the load value, after which there was plateauing of the curve until failure. The analytical model did not show any increase in the load value due to strain hardening of the reinforcement. On the other hand, the experimental response experienced an initial drop of less than 25% after the linear portion of the curve. However, the drop in capacity was followed eventually by a smoother increase in capacity and plateauing of the curve, during which strain hardening of the reinforcement also contributed to the load increase. VecTor2 overestimated the drop in the capacity of the beam due to shear cracking and did not fully account for the subsequent increase in capacity due to stress redistribution and arching mechanism. The maximum load capacity of the beam in the FE model, which was 473 kN, was 5% higher than the experimental load. However, the ultimate analytical displacement was only 66% of its experimental counterpart. Contrary to LTM, VecTor2 underestimated the ductility of the beam by underestimating the residual capacity of the concrete. Similar to the experimental response, the analytical cracking pattern consisted mainly of major vertical cracks connected to splitting cracks, as shown in Figure 7.9. The failure mode was also predicted accurately: neither the concrete crushed nor the reinforcement fractured; the extensive cracking of the beam caused the capacity to drop. Due to the absence of stirrups that would reduce the values of the principal compressive strains, the arching mechanism in LLM is even more pronounced as seen in the principal compressive stress contours in Figure 7.10.

![Graph](image.png)

Figure 7.8: Experimental and analytical responses for LLM
7.3.1.3 LFM

The analytical response of LFM is compared with the experimental response in Figure 7.11. As was the case with LLM, the initial and post-cracking stiffness was overestimated by the model by about 30%. The analytical maximum load, 411 kN, was within 4% of the experimental value. The plateauing portion of the model was significantly greater than that of the experimental response; the final deflection value before the final capacity drop in the FE model was more than 3 times greater than that of the experiment, which shows that the contribution of the fibres to the shear strength of the concrete was somewhat over-estimated in this case. Unlike the previous two beams, there was a divergence in the failure mode between the experiment and the FE model. While the
model showed that the fibres were able to prevent the diagonal splitting of the beam by causing a shear-compression failure, the experimental failure mode was indeed diagonal splitting. Consequently, the experimental and analytical cracking patterns differ, as shown in Figure 7.12.

The finite element modelling of the beams subjected to monotonic loading showed that VecTor2 was able to capture their capacities extremely accurately, all well within 10% of the experimental values. Failure modes were generally predicted accurately, except for the SFRC beam. On the other hand, the ductility of the beams was not accounted for by the models.

![Graph: Load vs. Mid-Span Deflection Response for LFM](image)

Figure 7.11: Experimental and analytical responses for LFM
7.3.2 Beams Subjected to Fatigue Loading

The fatigue damage models for plain concrete, reinforcement, and steel fiber reinforced concrete developed at the University of Toronto by Isojeh et al. (2017) were incorporated into VecTor2. As mentioned previously, these models will account for the fatigue damage on the structure at the first load cycle, after which the load increases normally up to the structure’s residual capacity. The number of load cycles to failure in this case corresponds to the number of cycles at which the residual capacity is equal to the maximum applied fatigue load. This value is most important when a given structure, whose fatigue life is unknown, is subject to known fatigue load parameters. For every number of load cycles, a simulation must be run until the fatigue capacity is reached. Comparing the analytical and experimental results would enable an assessment of the applicability of these models to large-scale beams, since they have already been verified by Isojeh et al. (2017) on small-scale beams, and consequently validate their potential as useful design tools for reinforced concrete wind turbine foundations.

Figure 7.13 shows the degradation of LTF under different numbers of load cycles. The maximum load for each number of cycle represents the residual capacity of the beam. As the number of cycles was increased, the residual capacities decreased. The analytical fatigue life of LTF under the applied maximum fatigue load (400 kN) was 40000 cycles, which was only 7% higher than the
experimental fatigue life. Considering the complex nature of fatigue loading and the multitude of parameters that could affect the results, this estimation of the fatigue life is highly accurate. Similar to the experimental results, the failure mode of the beam was the fracture of the longitudinal reinforcement.

![Analytical Response of LTF under Fatigue Loading](image)

Figure 7.13: Fatigue life prediction of LTF

Similar to the experimental outcome, adding stirrups to the plain concrete beam did not improve its fatigue life; LLF failed after the same number of load cycles as LTF. Figure 7.14 shows the degradation of LLF as different load cycles were applied. The analytical fatigue life of 40000 load cycles was 9% higher than the experimental fatigue life of LLF. The failure mode was also the same: fracture of the longitudinal reinforcement.

These results confirmed that the proposed fatigue models are applicable to large-scale reinforced concrete beams, as they were able to accurately capture their fatigue lives. They are superior to the empirical fatigue equations provided in difference codes since they consider the irreversible fatigue strains, interaction between the concrete and reinforcement, and the residual beam capacity. Therefore, they can be used to model wind turbine foundations subject to known fatigue loads. Of course, load and resistance factors still need to be used to be conservative.
Since LFF failed prematurely due to bond slip, there is no significance in comparing the experimental and analytical fatigue lives. The latter will give an idea of the capacity of LFF had it not failed due to bond slip, which will help reinforce the conclusion that steel fibres do indeed increase the fatigue life of reinforced concrete beams as compared to stirrups. The analytical result of LFF will be of significance since the accuracy of the models were verified with the other two beams.

Figure 7.15 shows the degradation of LFF with different number of load cycles. As can be seen, the beam withstood about 1.2 million cycles before it failed due to the fracture of the longitudinal reinforcement. The degradation of the beam’s residual capacity can be seen as the number of load cycles were increased until the failure point, at which the residual capacity was identical to the applied fatigue load. The improved fatigue life brought on by the addition of steel fibres make the use of SFRC a superior alternative to using stirrups, especially in deep beams. In addition to the superior performance of the fibres under fatigue loading, replacing stirrups would save the cost of bending and placing in the field. In the case of wind turbine foundations, using steel fibres brings an additional advantage: it will reduce the congestion of the reinforcement especially at the footing-pedestal interface. Table 7.3 summarizes the outcome of modelling the beams subjected to fatigue loading and shows the fatigue life predictions of the test beams using conventional S-N curve procedures found in design codes. As can be seen, the equations used in the design codes
underestimate the fatigue life of the test beams. In addition to the load and resistance factors specific for fatigue, the linear S-N curve equations used for steel and the empirical equations used for concrete are conservative in predicting the number of cycles to failure due to the consideration of the serviceability limit state. Although the structure might withstand more number of load cycles without structural failure, the crack widths after those cycles might surpass the maximum crack widths imposed by the serviceability limit state. For LLF, the theoretical design fatigue life is more than 7 times less than the actual one, although the failure mode is correctly identified. LTF had a more accurate design fatigue life of 29000 cycles, 78% of the actual value. Theoretically, the decreased demand on the ties (i.e. longitudinal reinforcement) brought by the tensile action of the stirrups enhanced the fatigue life of the beam. Of course, this was not the case in reality as the stirrups had little effect in enhancing the fatigue life of the test beams. Finally, since the design codes do not have fatigue life predictions for SFRC beams and do not take into account the reduced stress on the reinforcement due to the bridging action of the steel fibres, the design fatigue life for LFF was the same as that of LLF. These results reinforce the need to adopt realistic, theoretically-grounded fatigue models for reinforced concrete structures.

Figure 7.15: Fatigue life prediction of LFF
Table 7.3: Summary of modelling beams subjected to fatigue loading

<table>
<thead>
<tr>
<th>Beam ID</th>
<th>Analytical (FE) Fatigue Life</th>
<th>Experimental Fatigue Life</th>
<th>Design Code Fatigue Life</th>
<th>Analytical Failure Mode</th>
<th>Experimental Failure Mode</th>
<th>Design Code Failure Mode</th>
</tr>
</thead>
<tbody>
<tr>
<td>LTF</td>
<td>40000 cycles</td>
<td>37300 cycles</td>
<td>29000 cycles</td>
<td>Fracture of rebar</td>
<td>Fracture of rebar</td>
<td>Fracture of rebar</td>
</tr>
<tr>
<td>LLF</td>
<td>40000 cycles</td>
<td>36700 cycles</td>
<td>5000 cycles</td>
<td>Fracture of rebar</td>
<td>Fracture of rebar</td>
<td>Fracture of rebar</td>
</tr>
<tr>
<td>LFF</td>
<td>4100 cycles</td>
<td>1200000 cycles</td>
<td>5000 cycles</td>
<td>Fracture of rebar</td>
<td>Fracture of rebar</td>
<td>Fracture of rebar</td>
</tr>
</tbody>
</table>
Chapter 8: Conclusions and Recommendations

8 Conclusions and Recommendations

8.1 Conclusions

1. Current equations for fatigue of reinforced concrete used in design codes neglect the accumulation of fatigue damage and, being empirical in nature, do not consider stress redistribution and the interaction between the concrete and reinforcement. Therefore, rather than accurately predicting the fatigue life of reinforced concrete elements, they provide overly inefficient conservative results.

2. Due to the steep nature of cracks in deep beams and the compressive stresses of the struts, the contribution of traditional shear reinforcement (stirrups) to the fatigue life of reinforced concrete deep beams is marginal, incurring additional labor and material costs without providing significantly improved fatigue life. However, the shear reinforcement does help reduce the values of principal strains compared to plain concrete.

3. Using steel fibres within the concrete helps reduce the maximum crack width and principal strain values, compared to plain concrete and concrete with stirrups, and increase the toughness of the structure.

4. Steel fibres are a superior alternative to traditional shear reinforcement in improving the fatigue life of reinforced concrete deep structures. Their random distribution and crack-bridging abilities reduce the stress on the longitudinal reinforcement at the major cracks, enhancing their fatigue performance, and reducing the rate of degradation of the concrete due to fatigue loading. In addition to the improved performance, steel fibres are more cost effective since they do not require labor work associated with cutting, bending, and placing the stirrups. The congestion of the reinforcement is also reduced when steel fibres are used.

5. Despite the advantages, steel fibres present certain challenges in the casting of concrete, especially with introducing the fibres into the concrete mix. It must be ensured that proper equipment and practices are employed in order to avoid balling and improper distribution of fibres.
6. The fatigue models developed by Isojeh et al. (2017) for reinforced concrete and SFRC are more realistic than the other models that rely on Miner’s rule, since they take into account the irreversible fatigue strains and the interaction between the concrete and the reinforcement and include newly-developed and verified concrete damage models. They provide accurate predictions on the fatigue life and failure modes of large-scale reinforced concrete deep beams, which makes them a powerful tool for use in the design for fatigue.

8.2 Recommendations

1. Further research is required to quantify the contribution of the stirrups to the fatigue life of reinforced concrete beams for several aspect ratios, percentages of shear and longitudinal reinforcements, and beam sizes. This would produce specific guidelines as to when exactly the contribution of stirrups becomes advantageous.

2. Although the fatigue models proposed by Isojeh et al. (2017) accurately predict the fatigue life of structures, further work is required in order to make these models more ‘design-friendly’. This would include improving the user interface, removing the sensitivity of such models to certain material inputs, and ensuring that the results always underestimate the fatigue life.

3. In order to facilitate the wide usage of steel fibres in the industry, proper infrastructure and equipment for producing reliable and cost-efficient SFRC systems are required for concrete companies instead of using the traditional low-energy concrete mixers.
References


Gergely, P., & Lutz, L. A. (1968). Maximum Crack Width in Reinforced Concrete Flexural Members. Causes, Mechanisms, and Control of Cracking in Concrete, SP-20, American Concrete Institute, 87-117.


Sparks, P. R., & Menzies, J. B. (1973). *The Effect of Rate of Loading upon the Static and Fatigue Strength of Plain Concrete in Compression*. Department of the Environment, Building Research Establishment.


Appendix A: Steel Fibre Data Sheet and Mixing Recommendations

RC-80/30-BP Fibres Data Sheet

Recommendations - mixing

1. General
- Preferably add fibres in the mixer at the batching plant.
- Recommended maximum dosage:

<table>
<thead>
<tr>
<th>Dosage (kg/m³)</th>
<th>Aggregate size</th>
</tr>
</thead>
<tbody>
<tr>
<td>pour</td>
<td>70</td>
</tr>
<tr>
<td>pump</td>
<td>55</td>
</tr>
</tbody>
</table>

- A continuous grading is preferred.
- Mix until all glued fibres are separated into individual fibres. Fibres do not increase mixing time significantly.
- If special cements or admixtures are used, a preliminary test is recommended.

2. Fibre addition
2.1. In batching plant mixer
- Never add fibres as first component in the mixer.
- Fibres can be introduced together with sand and aggregates, or can be added in freshly mixed concrete.

- Only for drum mixer: unopened degradable bags can be thrown directly in the mixer.

2.2. Truck mixer
- Put mixer on maximum drum speed: 12-18 rpm.
- Adjust slump to a min. of 12 cm (preferably with water reducing agents or high water reducing agents).
- Add fibres with maximum speed of 40 kg/min.
- Unopened degradable bags can be added provided that drum speed is min. 12 rpm.
- Optional equipment: belt hoist elevator.
- After adding the fibres, continue mixing at highest speed for 4-5 min. (± 70 rotations).

2.3. Automatic dosing
- Fibres in bulk can be dosed at rates up to 3.5 kg/sec with a specially developed dosing equipment.

Recommendations - storage
- Protect the pallets against rain.
- Do not stack the pallets on top of each other.

N.V. Bekoart S.A. - Beukenstraat 2 - 8550 Zwevegem - Belgium
Tel. +32 (0) 50 / 76 60 86
Fax: +32 (0) 50 / 76 79 47

Values are indicative only. Modifications reserved. All details describe our products in general form only. For ordering and design only use official specifications and documents. N.V. Bekoart S.A., 2000.
### Fibres Mixing Recommendations

<table>
<thead>
<tr>
<th>Handling</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Dramix</strong>&lt;sup&gt;®&lt;/sup&gt; Glued</td>
</tr>
<tr>
<td>- Gloves and eye protection must be used!</td>
</tr>
<tr>
<td>- Keep dry</td>
</tr>
<tr>
<td>- No stacking</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Before Adding Fibres</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Dramix</strong>&lt;sup&gt;®&lt;/sup&gt;</td>
</tr>
<tr>
<td>Maximum dosage depends on:</td>
</tr>
<tr>
<td>- Concrete composition</td>
</tr>
<tr>
<td>- Raking method</td>
</tr>
<tr>
<td>- Type of application</td>
</tr>
<tr>
<td>Bekker recommendations:</td>
</tr>
<tr>
<td>- Preferably use a central batching plant mixer</td>
</tr>
<tr>
<td>- A continuous grading and sieving curve</td>
</tr>
<tr>
<td>- Sufficient fines and mortar content</td>
</tr>
<tr>
<td>Optimum slump before fibre addition &gt; 12 cm</td>
</tr>
<tr>
<td>Note:</td>
</tr>
<tr>
<td>- Depending on dosage and fibre type, fibres reduce the slump</td>
</tr>
<tr>
<td>- Adjust required consistency only with (super) plasticisers</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Dosing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bags are non-degradable</td>
</tr>
<tr>
<td><strong>Plant mixer</strong></td>
</tr>
<tr>
<td>- Introduce fibres with sand and aggregates</td>
</tr>
<tr>
<td>- Add fibres to fresh mixed concrete</td>
</tr>
<tr>
<td>- Never add fibres as a first component</td>
</tr>
<tr>
<td><strong>Truck mixer</strong></td>
</tr>
<tr>
<td>- Never add fibres as a first component</td>
</tr>
<tr>
<td>- Never fill drum completely with concrete in order to achieve even fibre distribution</td>
</tr>
<tr>
<td>- Add fibre continuously at a maximum of 40 kg/min</td>
</tr>
<tr>
<td>If fibre blast equipments are used (not needed for Dramix® fibres) the suitability should be tested on beforehand</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Mixing</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>DO: drum rotation speed &gt; 12 rpm</strong></td>
</tr>
<tr>
<td>- Mixing times: after adding all fibres, mix 1 minute/m³ concrete and not less than 5 min</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Quality Control</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Before using fibre concrete, a preliminary test must be done</strong></td>
</tr>
<tr>
<td>- Workability</td>
</tr>
<tr>
<td>- Air content</td>
</tr>
<tr>
<td>- Separation of fibre bundles when using glued fibres</td>
</tr>
<tr>
<td>- Homogeneous fibre distribution in the concrete</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Pumping</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Hose diameter &gt; 1,5 x fibre length</strong></td>
</tr>
<tr>
<td>For complicated pump lines or concrete compositions, a trial is recommended prior to execution</td>
</tr>
</tbody>
</table>
Appendix B: Material Stress-Strain Curves

LTM Concrete Stress-Strain Curve

Note that the uniaxial compression test for LTM was displacement-controlled. Hence in order to avoid the explosive failure of the cylinder, the test was stopped after the concrete response had peaked, so the descending portion of the graph was not obtained.

Also note that at the point where the response changes from being linear to almost parabolic, the LVDTs were removed so the parabolic parts of the response are obtained from the displacement given by the jack rather than the LVDTs; adjustments have been made in order to match the displacements of the LVDT and the jack.
LTF Concrete Stress-Strain Curve

Uniaxial Compression Test for LTF at Testing Day

\[ f'c = 78 \text{ MPa} \]

LLM Concrete Stress-Strain Curve

Uniaxial Compression Test for LLM at Testing Day

\[ f'c = 82 \text{ MPa} \]
LLF Concrete Stress-Strain Curve

Uniaxial Compression Test for LLF at Testing Day

\[ f'c = 84 \text{ MPa} \]

LFM Concrete Stress-Strain Curve

Uniaxial Compression Test for LFM at Testing Day

\[ f'c = 55 \text{ MPa} \]
LFF Concrete Stress-Strain Curve

![Uniaxial Compression Test for LFF at Testing Day](image)

- $f_c' = 56$ MPa

15M Steel Stress-Strain Curve

![Tensile Coupon Test for 15M Steel](image)

- $f_y' = 442$ MPa
- $\varepsilon_y = 2.6 \times 10^{-3}$
- $f_u = 581$ MPa
- $\varepsilon_{sh} = 19.1 \times 10^{-3}$
- $E = 170447$ MPa
10M Steel Stress-Strain Curve

![Tensile Coupon Test for 10M Steel](image)

**Results:**
- $f'_y = 448$ MPa
- $\varepsilon_y = 2.82 \times 10^{-3}$
- $f_u = 582$ MPa
- $\varepsilon_{sh} = 20.6 \times 10^{-3}$
- $E = 159951$ MPa
- $E_{sh} = 1055$ MPa
Appendix C: Grout Specifications

Non-Shrink Precision Grout

Product No. 1585-00

Product Description
QUICKRETE® Non-Shrink Precision Grout is a high strength, non-metallic, Portland cement based material with expansive additives designed for grouting all types of machinery, steel columns, bearing plates, pre-cast concrete, and anchoring applications.

Product Use
Typical applications for QUICKRETE® Non-Shrink Precision Grout include grouting of:
- All types of machinery
- Steel columns
- Bearing plates
- Precast concrete
- Other anchoring conditions that require high in-service strength

The non-shrink characteristics of Non-Shrink Precision Grout make it stable and capable of handling high load transfers.

Sizes
- QUICKRETE® Non-Shrink Precision Grout – 50 LB (22.7 kg) bags

Yield
- Each 50 lb (22.7 kg) bag of QUICKRETE® Non-Shrink Precision Grout will yield 0.45 cu ft (12.7 L) at flowable consistency.

Technical Data

APPLICATION STANDARDS
- ASTM International
  - ASTM C827 Standard Test Method for Change in Height at Early Ages of Cylindrical Specimens of Cementitious Mixtures
  - ASTM C939 Standard Test Method for Flow of Grout for Preplaced-Aggregate Concrete (Flow Cone Method)
  - ASTM E488 Standard Test Methods for Strength of Anchors in Concrete and Masonry Elements

U.S. Army Corps of Engineers (USACE) - CRD 621

Physical / Chemical Properties
QUICKRETE® Non-Shrink Precision Grout complies with all properties of ASTM C1107 and CRD 621 producing the results shown in Table 1.

Table 1 Typical Physical Properties at 73°F (23°C)

<table>
<thead>
<tr>
<th>Compressive strength, ASTM C109 modified per ASTM C1107</th>
<th>Plastic Consistency</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 day</td>
<td>3000 psi (20.7 MPa)</td>
</tr>
<tr>
<td>3 days</td>
<td>9500 psi (65.5 MPa)</td>
</tr>
<tr>
<td>7 days</td>
<td>10,000 psi (68.9 MPa)</td>
</tr>
<tr>
<td>28 days</td>
<td>14,000 psi (96.5 MPa)</td>
</tr>
</tbody>
</table>

| Height change, ASTM C1090 1, 3, 7 and 28 days | 0 - 0.2% |
| Height change, ASTM C827 Flowable Consistency | + 0.8% |

| Height change, ASTM C1090 1, 3, 7 and 28 days | 0 - 0.2% |
| Height change, ASTM C827 Fluid Consistency | + 0.4% |

| Height change, ASTM C1090 1, 3, 7 and 28 days | 0 - 0.2% |
| Height change, ASTM C827 Pull-out Strength, ASTM E488 | + 0.3% |

1 1 1/4" (31 mm) bolts embedded 9" (225 mm) deep in 3" (75 mm) hole in 2000 psi (13.8 MPa) concrete.
INSTALLATION
SURFACE PREPARATION
Surfaces to receive the grout must be clean and free of any type of foreign matter, grease, paint, oil, dust or efflorescence. In some cases it may be necessary to roughen smooth surfaces or etch old ones with acid. The area should be flushed and soaked with clean water prior to grouting leaving no standing water. Place the grout quickly and continuously using light rodding to eliminate air bubbles.

MIXING
Add the minimum amount of water necessary to produce the desired flow characteristics as indicated in Table 2. Do not add more water than the amount needed to produce a 20-second flow per ASTM Test Method C 939. QUIKRETE® Non-Shrink Precision Grout should be mechanically mixed for a minimum of 5 minutes.

TABLE 2
APPROXIMATE WATER REQUIRED FOR 50 LB (22.7 KG) OF GROUT

<table>
<thead>
<tr>
<th>Type</th>
<th>Water Required</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plastic</td>
<td>1 gal (3.8 L)</td>
</tr>
<tr>
<td>Flowable</td>
<td>1 gal + 1 pt (4.3 L)</td>
</tr>
<tr>
<td>Fluid</td>
<td>1 gal + 3 pt (5.2 L)</td>
</tr>
</tbody>
</table>

WORKING TIME
When properly mixed to a fluid consistency QUIKRETE® Non-Shrink Precision Grout will comply with all portions of ASTM C1107 and CRD 621 and retain a fluid consistency for the maximum usable working times stated in Table 3.

TABLE 3
WORKING TIME

<table>
<thead>
<tr>
<th>Temperature</th>
<th>Working Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>50°F (10°C)</td>
<td>25 min</td>
</tr>
<tr>
<td>73°F (23°C)</td>
<td>25 min</td>
</tr>
<tr>
<td>90°F (32°C)</td>
<td>15 min</td>
</tr>
</tbody>
</table>

CURING
A damp cure of at least 3 days is necessary to control the non-shrink characteristics and maintain strength levels.

PRECAUTIONS
• Additions of cement or other materials will eliminate the designed product qualities
• Water quantities may be affected by temperature, mixing method and batch size
• QUIKRETE® Non-Shrink Precision Grout should not be re-tempered
• Grout temperature should be maintained from 50 - 90 degrees F (10 - 32 degrees C) to achieve specified results. Use cold water in hot weather or hot water in cold weather to achieve desired grout temperature.
• Do not pour grout if temperature is expected to go below 32 degrees F (0 degrees C) within a 12 hour period.
• Mix no more than can be used in 30 minutes

WARRANTY
The QUIKRETE® Companies warrant this product to be of merchantable quality when used or applied in accordance with the instructions herein. The product is not warranted as suitable for any purpose or use other than the general purpose for which it is intended. Liability under this warranty is limited to the replacement of its product (as purchased) found to be defective, or at the shipping companies’ option, to refund the purchase price. In the event of a claim under this warranty, notice must be given to The QUIKRETE® Companies in writing. This limited warranty is issued and accepted in lieu of all other express warranties and expressly excludes liability for consequential damages.

The QUIKRETE® Companies
One Securities Centre
3490 Piedmont Rd., NE, Suite 1300, Atlanta, GA 30305
(404) 634-9100 • Fax: (404) 842-1425

* Refer to www.quikrete.com for the most current technical data, SDS, & guide specifications

Revised 28 August 2015