Experiments in
tunnel–soil–structure interaction

Stefan Ritter
Department of Engineering
University of Cambridge

This dissertation is submitted for the degree of
Doctor of Philosophy

Girton College

September, 2017
I hereby declare that this dissertation is the result of my own work and includes nothing which is the outcome of work done in collaboration except where specified in the text. It is not substantially the same as any that I have submitted, or, is being concurrently submitted for a degree or diploma or other qualification at the University of Cambridge or any other University or similar institution. Permission to exceed the recommended limits of 65,000 words was granted by the Board of Graduate Studies. This dissertation is presented in less than the revised limits of 72,000 words and 150 figures.

Stefan Ritter
September, 2017
Acknowledgements

This work would not have been possible without the support, guidance, inspiration and encouragement of numerous people.

First, I am particularly grateful to my supervisor, Dr Matt DeJong, for his desire to improve the understanding of the response of masonry to tunnelling-induced settlements which made this research possible. Specifically, his extensive knowledge, generosity with his time and efficient problem solving was invaluable, but also set a true example for my future career. I also thank him for the opportunity to acquire teaching experience, for the freedom to work remotely, for the discussions about football and for his guidance and support to explore rewarding challenges in the future.

Second, I am indebted to Dr Giorgia Giardina, from whom I learned a great deal. Her research enthusiasm and continuous academic and moral support were a great inspiration and helped me through the rough path to finish this thesis. I am also very grateful to her support and encouraging words during the challenges faced during centrifuge testing. Moreover, I thank Giorgia for sharing her expertise in starting an academic career; her excellent guidance is invaluable.

Third, I am thankful to several others who directly contributed to this research and to Crossrail for financial support. I am grateful to Professor Lord Robert Mair for his interest in this study, for sharing knowledge and for inspiration that extended far beyond the world of tunnelling. Many thanks go to Dr Ruaidhri Farrell for sharing his data. I am also indebted to Professor Giulia Viggiani for numerous discussion about tunnel–soil–structure interaction and for the unique experience to present at the 3xV workshop. I am grateful to Dr Stuart Haigh who has been extremely helpful with questions about physical modelling and soil mechanics. Many thanks also to Professor Gopal Madabhushi for various bits of wisdom and for the enjoyable opportunity to instruct the centrifuge experiment of 5R5. I am grateful to Dr Charlie Heron for excellent advice when starting with centrifuge modelling and to Dr Andrea Franza for endless discussions and his desire to push this research a step further. Thanks to all the technicians for their assistance with the experimental aspects of this research. Specifically, to John for sharing his extensive experience, to Mark for assuring that everything on my centrifuge package was
thoroughly fixated, to Kristian for optimizing the model preparation and educating me about how to pronounce essential words such as ‘solenoid valve’, to Chrissie for the uncountable wiring of my MEMS and to Richard for installing the tunnel and for taking care of my bike. Many thanks also to Stan Finney for his support with 3D printing.

Finally, I am thankful for the countless experiences I have had while being at Cambridge. Particularly, I am grateful to the various researchers who shared the last four years with me: Abdul, Alex, Alba, April, Deryck, Fiona, Francesca, Gianmaro, Gue, James, Jeffrey, Krishna, Livia, Matthew, Mehdi, Mike, Njemile, Orestis, Peter, Raz, Stephan, Sinan and Talia. Our many conversations, ranging from geotechnical engineering to any other subject one could imagine, were extremely enjoyable. Being part of the committee of the Geotechnical Society was also a great pleasure and inspiration. Through Girton College I have met many good friends including my homies from 7a Parker Street: Constantine, Felix, and Francesco, and also Aisling, Jose, Leah, Luca, Olympia, Sam and Shane. Thanks for being my friends and for sharing this unique experience with me. I am also thankful to the Girtonian Wolves and the Girton football team for keeping me fit and for showing me the Cambridge Undergraduate experience. However, all these experiences would not have been possible without the endless support of my family. I am thankful to my parents and grandparents, back home in Austria, for their understanding, generosity and encouragement. Most of all, I thank Kathrin for supporting me whenever needed, inspiring me to follow my dreams and for showing me how beautiful life is. It would not have been possible without her.
Abstract

Urbanisation will require significant expansion of underground infrastructure, which results in unavoidable ground displacements that affect the built environment. Predicting the interaction between a tunnel, the soil and existing structures remains an engineering challenge due to the highly non-linear behaviour of both the soil and the building.

This thesis investigates the interaction between a surface structure and tunnelling-induced ground displacements. Specifically, novel three-dimensionally printed building models with brittle material behaviour, similar to masonry, were developed and tested in a geotechnical centrifuge. This enabled replication of building models with representative global stiffness values and realistic building features including strip footings, intermediate walls, a rough soil–structure interface, building layouts and façade openings.

By varying building characteristics, the impact of structural features on both the soil and building response to tunnelling in dense sand was investigated. Results illustrate that the presence of surface structures considerably altered the tunnelling-induced soil response. The building-to-tunnel position notably influences the magnitude of soil displacements and causes localised phenomena such as embedment of building corners. An increase of the façade opening area and building length reduces the alteration of the theoretical greenfield settlements, in particular the trough width. Moreover, the impact of varying the building layout is discussed in detail.

For several building–tunnel scenarios, building distortions are quantified and the crucial role of building features is demonstrated. Structures spanning the greenfield inflection point experienced more deformation than identical structures positioned in either sagging or hogging, and partitioning a structure either side of the greenfield inflection point is shown to lead to unconservative damage assessments. Results also quantify the significant extent to which structural distortions increase as façade openings and building length increases. Observed building damage and cracking patterns confirm the reported trends.

The experimental results are used to evaluate the performance of available methods to assess the behaviour of buildings to tunnelling. Predictions ignoring soil–structure interaction are usually overly conservative, while approaches based on the relative stiffness of a structure
and the soil result in inconsistent predictions, though some methods performed better than others. Practical improvements to consider structural details when assessing this tunnel–soil–structure system are finally proposed.
## Contents

### Contents

List of figures  
List of tables  

### 1 Introduction

1.1 Background  
1.2 Motivation  
1.3 Research objectives and methodology  
1.4 Thesis outline and related publications  

### 2 Literature review

2.1 Ground deformations caused by tunnelling  
2.1.1 Empirical framework  
2.1.1.1 Transverse behaviour  
2.1.1.2 Trough width  
2.1.2 Greenfield soil movements of tunnels in sand  
2.1.2.1 Alternative curves  
2.1.2.2 Key parameters affecting ground movements of tunnels in sand  
2.1.2.3 Volumetric behaviour of granular material around tunnels  
2.1.2.4 Empirical approach to predict greenfield settlements of tunnels in sand  
2.1.2.5 Empirical framework to estimate the volume loss at soil surface  
2.1.3 Summary  
2.2 Response of buildings to tunnelling-induced settlements 

---

<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>Contents</td>
<td>ix</td>
</tr>
<tr>
<td>List of figures</td>
<td>xv</td>
</tr>
<tr>
<td>List of tables</td>
<td>xxiii</td>
</tr>
<tr>
<td>1 Introduction</td>
<td>1</td>
</tr>
<tr>
<td>1.1 Background</td>
<td>1</td>
</tr>
<tr>
<td>1.2 Motivation</td>
<td>2</td>
</tr>
<tr>
<td>1.3 Research objectives and methodology</td>
<td>4</td>
</tr>
<tr>
<td>1.4 Thesis outline and related publications</td>
<td>5</td>
</tr>
<tr>
<td>2 Literature review</td>
<td>9</td>
</tr>
<tr>
<td>2.1 Ground deformations caused by tunnelling</td>
<td>9</td>
</tr>
<tr>
<td>2.1.1 Empirical framework</td>
<td>13</td>
</tr>
<tr>
<td>2.1.1.1 Transverse behaviour</td>
<td>13</td>
</tr>
<tr>
<td>2.1.1.2 Trough width</td>
<td>16</td>
</tr>
<tr>
<td>2.1.2 Greenfield soil movements of tunnels in sand</td>
<td>18</td>
</tr>
<tr>
<td>2.1.2.1 Alternative curves</td>
<td>18</td>
</tr>
<tr>
<td>2.1.2.2 Key parameters affecting ground movements of tunnels in sand</td>
<td>20</td>
</tr>
<tr>
<td>2.1.2.3 Volumetric behaviour of granular material around tunnels</td>
<td>21</td>
</tr>
<tr>
<td>2.1.2.4 Empirical approach to predict greenfield settlements of tunnels in sand</td>
<td>23</td>
</tr>
<tr>
<td>2.1.2.5 Empirical framework to estimate the volume loss at soil surface</td>
<td>24</td>
</tr>
<tr>
<td>2.1.3 Summary</td>
<td>24</td>
</tr>
<tr>
<td>2.2 Response of buildings to tunnelling-induced settlements</td>
<td>25</td>
</tr>
</tbody>
</table>
## Contents

2.2.1 Deformation parameters ................................................. 25
2.2.2 Damage criteria neglecting soil–structure interaction ............... 27
  2.2.2.1 Empirical damage criteria ........................................ 28
  2.2.2.2 Semi-empirical damage criteria ................................. 30
2.2.3 Three-stage building risk assessment ................................. 38
  2.2.3.1 Preliminary assessment ........................................... 39
  2.2.3.2 Second stage assessment ......................................... 40
  2.2.3.3 Detailed evaluation ............................................. 40
2.2.4 Soil–structure interaction .............................................. 40
2.2.5 Building stiffness - the relative stiffness methods ................... 41
  2.2.5.1 Estimating the global building stiffness ....................... 46
  2.2.5.2 The dominant mode of building deformations ................. 50
2.2.6 The influence of the building weight .................................. 51
2.2.7 Centrifuge tests quantifying the soil–structure interaction ........ 52
2.2.8 Summary ..................................................................... 53
  2.2.8.1 Discussion of the limiting tensile strain method ............. 53
  2.2.8.2 Discussion of the relative stiffness methods .................. 54
2.3 Performance of masonry to tunnelling-induced settlements .......... 56
  2.3.1 Mechanics of masonry ............................................... 56
  2.3.2 The mode of deformation ............................................. 57
  2.3.3 Damage patterns ..................................................... 60
  2.3.4 Summary .................................................................. 61

3 Experimental method and equipment ........................................ 63
  3.1 Introduction to centrifuge modelling .................................... 63
  3.2 Errors and limitations in centrifuge modelling ......................... 64
    3.2.1 Variation in gravity field .......................................... 64
    3.2.2 Influence of the Earth’s gravity field ............................ 65
    3.2.3 Particle size effects ................................................ 65
    3.2.4 Boundary conditions .............................................. 66
  3.3 Experimental setup and equipment ........................................ 67
    3.3.1 Model geometry and instrumentation ............................ 67
    3.3.2 Model tunnel and tunnel excavation simulation ................ 69
    3.3.3 Soil model and preparation ....................................... 72
    3.3.4 Building model .................................................... 72
3.4 Digital image correlation ............................................. 75
3.5 Data acquisition ....................................................... 78
3.6 Testing procedure ..................................................... 78
3.7 Overview of centrifuge test series ................................. 79
3.8 Summary ................................................................. 79

4 3D printed building models ........................................... 83
4.1 Background .............................................................. 84
4.2 3D printing technique .................................................. 85
  4.2.1 3D printing process ............................................... 86
  4.2.2 Coordinate system ................................................. 88
  4.2.3 Material composition ............................................. 88
  4.2.4 Microstructure effects ........................................... 89
4.3 Building models and specimen ..................................... 90
4.4 Mechanical tests ...................................................... 93
  4.4.1 Specimen .......................................................... 93
  4.4.2 Test procedure .................................................... 94
  4.4.3 Results and analysis ............................................. 95
  4.4.4 Effect of different curing temperature ......................... 100
4.5 Global building stiffness ........................................... 100
  4.5.1 Bending stiffness ................................................ 101
    4.5.1.1 Neutral axis of centrifuge model buildings .............. 101
    4.5.1.2 Plane-strain relative building stiffness measures .... 101
    4.5.1.3 Approach by Melis and Rodriguez Ortiz (2001) .......... 103
    4.5.1.4 Approach by Son and Cording (2005, 2007) ............ 104
    4.5.1.5 Approach by Pickhaver et al. (2010) .................... 106
  4.5.2 Axial stiffness .................................................. 110
  4.5.3 Comparison to field data and previous research ............ 112
4.6 Summary ............................................................... 115

5 Centrifuge modelling effects and boundary conditions .......... 117
5.1 Background .......................................................... 117
5.2 Spin-up phenomena ................................................. 118
  5.2.1 Tunnel pressure control ....................................... 119
  5.2.2 Impact of tunnel excavation simulation technique ........ 119
  5.2.3 Near surface soil and structure vertical displacements .. 120
<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.2.4 Impact on building models</td>
<td>122</td>
</tr>
<tr>
<td>5.3 Boundary effects</td>
<td>127</td>
</tr>
<tr>
<td>5.3.1 Friction effects between the PMMA window and the soil model</td>
<td>127</td>
</tr>
<tr>
<td>5.3.2 Out of PMMA window plane movement of building model</td>
<td>128</td>
</tr>
<tr>
<td>5.4 Conclusions</td>
<td>132</td>
</tr>
<tr>
<td>6 The influence of a surface structure on tunnelling-induced subsidence</td>
<td>133</td>
</tr>
<tr>
<td>6.1 Background</td>
<td>133</td>
</tr>
<tr>
<td>6.2 Tunnel stability</td>
<td>135</td>
</tr>
<tr>
<td>6.3 Effect of building-to-tunnel position</td>
<td>136</td>
</tr>
<tr>
<td>6.3.1 Vertical soil response</td>
<td>136</td>
</tr>
<tr>
<td>6.3.1.1 Ground loss</td>
<td>138</td>
</tr>
<tr>
<td>6.3.1.2 Vertical surface soil displacements</td>
<td>139</td>
</tr>
<tr>
<td>6.3.2 Surface trough width</td>
<td>142</td>
</tr>
<tr>
<td>6.3.3 Horizontal surface soil displacements</td>
<td>144</td>
</tr>
<tr>
<td>6.3.4 Subsurface soil displacements</td>
<td>148</td>
</tr>
<tr>
<td>6.3.4.1 Vertical subsurface soil response</td>
<td>148</td>
</tr>
<tr>
<td>6.3.4.2 Horizontal subsurface soil response</td>
<td>150</td>
</tr>
<tr>
<td>6.3.4.3 Subsurface trough width</td>
<td>150</td>
</tr>
<tr>
<td>6.3.4.4 Shear and volumetric strains</td>
<td>151</td>
</tr>
<tr>
<td>6.4 Effect of building characteristics</td>
<td>155</td>
</tr>
<tr>
<td>6.4.1 Influence of building characteristics on the vertical soil response</td>
<td>155</td>
</tr>
<tr>
<td>6.4.1.1 Ground loss</td>
<td>157</td>
</tr>
<tr>
<td>6.4.1.2 Surface trough width</td>
<td>158</td>
</tr>
<tr>
<td>6.4.2 Influence of building characteristics on the horizontal soil response</td>
<td>160</td>
</tr>
<tr>
<td>6.5 Conclusions</td>
<td>163</td>
</tr>
<tr>
<td>7 Building response to tunnelling-induced subsidence</td>
<td>167</td>
</tr>
<tr>
<td>7.1 Background</td>
<td>167</td>
</tr>
<tr>
<td>7.2 Volume loss</td>
<td>169</td>
</tr>
<tr>
<td>7.3 Vertical building response</td>
<td>170</td>
</tr>
<tr>
<td>7.3.1 Vertical building displacements</td>
<td>170</td>
</tr>
<tr>
<td>7.3.2 Vertical soil–structure interaction</td>
<td>172</td>
</tr>
<tr>
<td>7.3.3 Vertical building distortions</td>
<td>176</td>
</tr>
<tr>
<td>7.4 Horizontal building response</td>
<td>185</td>
</tr>
<tr>
<td>7.4.1 Horizontal building displacements</td>
<td>185</td>
</tr>
</tbody>
</table>
7.4.2 Horizontal soil–structure interaction ........................................... 187
7.4.3 Horizontal building distortions .................................................. 188
7.5 Effect of building features on building response and damage ................. 194
  7.5.1 Global building response ...................................................... 196
  7.5.2 Local building response ....................................................... 198
  7.5.3 Building damage ............................................................... 202
7.6 Building response for hogging and sagging separation .......................... 208
7.7 Effect of building characteristics on shear and bending deformations ......... 211
  7.7.1 Building length effects ....................................................... 213
  7.7.2 Building opening effects .................................................... 215
7.8 Summary ................................................................................... 217

8 Evaluation of current damage assessment methods .................................... 221
  8.1 Background .............................................................................. 221
  8.2 Performance of criteria to estimate building strains .............................. 224
    8.2.1 Limiting tensile strain method .............................................. 224
    8.2.2 State of strain concept ....................................................... 229
    8.2.3 Comparison of criteria to estimate building strains ................. 233
  8.3 Performance of the relative stiffness methods ...................................... 237
    8.3.1 Estimating the relative stiffness ............................................ 237
      8.3.1.1 Soil stiffness .............................................................. 237
    8.3.2 Relative stiffness .............................................................. 241
      8.3.2.1 Relative stiffness expressions with focus on bending deflection 242
      8.3.2.2 Relative stiffness expression with focus on shear deflection  244
    8.3.3 Relative stiffness methods with focus on bending deflections ....... 245
      8.3.3.1 Modification factors for vertical building distortions .......... 245
      8.3.3.2 Modification factors for horizontal building distortions ... 252
    8.3.4 Relative stiffness method with focus on shear deflections ........... 255
      8.3.4.1 Predictions of the angular distortion ............................. 257
      8.3.4.2 Predictions of the horizontal strain ............................... 260
    8.3.5 Comparison of relative stiffness methods .................................. 262
  8.4 Summary ................................................................................... 270
    8.4.1 Criteria to estimate building strains .................................... 270
    8.4.2 Greenfield predictions ...................................................... 271
    8.4.3 Relative stiffness method predictions .................................... 271
Contents

9 Recommendations for practical implementation 273
  9.1 Accounting for the building layout 273
  9.2 Accounting for the building-to-tunnel position 281
  9.3 Accounting for façade openings 284
  9.4 Accounting for the building length 286
  9.5 Design recommendations to account for building characteristics 288
  9.6 Summary 290

10 Conclusions 293
  10.1 Main findings 293
    10.1.1 Influence of building characteristics on tunnelling subsidence 293
    10.1.2 Influence of building characteristics on structural behaviour 295
  10.2 Scientific contributions 297
  10.3 Limitations and applicability of results 299
  10.4 Future research 299

References 301

Appendix A Building geometry 315
List of figures

1.1 Thesis outline. .................................................. 7
2.1 Sources of ground movement during shield tunnelling (after Shirlaw et al., 2003). .................................................. 10
2.2 Vertical greenfield surface displacements described by a Gaussian curve. .... 13
2.3 Horizontal surface displacements, horizontal strain and vertical surface displacements (after Franzius, 2003). .................................................. 15
2.4 Surface and subsurface transverse greenfield settlement troughs for tunnels in sand (after Marshall et al., 2012). .................................................. 17
2.5 Modified Gaussian curve (after Vorster et al., 2005). .................................................. 19
2.6 Relationship between tunnel volume loss and shear strain (after Marshall et al., 2012). .................................................. 22
2.7 Contraction/dilation effects affecting the behaviour of ground surrounding tunnels (after Marshall et al., 2012; soil data from Zhao, 2008). .................................................. 22
2.8 Building deformation parameters (after Burland and Wroth, 1974). .................................................. 26
2.9 Representation of an actual building through a simple beam model (after Burland and Wroth, 1974; Farrell, 2010). .................................................. 33
2.10 Ratio of L/H affecting the predominate strain mode (after Burland and Wroth, 1974). .................................................. 34
2.11 Relative building dimensions and potential soil–structure interaction. .... 36
2.12 Damage category diagrams. .................................................. 37
2.13 Damage criteria based on the state of strain concept (after Son and Cording, 2005). .................................................. 38
2.15 Geometry of the soil–structure interaction problem after Potts and Addenbrooke (1997). .................................................. 42
<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.16</td>
<td>Design curves proposed by Franzius et al. (2006) (after Farrell, 2010).</td>
<td>44</td>
</tr>
<tr>
<td>2.17</td>
<td>Design curves for modification factors.</td>
<td>45</td>
</tr>
<tr>
<td>2.18</td>
<td>Design curves for modification factors focusing on shear distortions (Boscardin and Cording, 1989; Son and Cording, 2005).</td>
<td>47</td>
</tr>
<tr>
<td>2.20</td>
<td>Components of masonry (after Lourenco, 1996).</td>
<td>56</td>
</tr>
<tr>
<td>2.21</td>
<td>Typical stress-displacement curves of masonry (after Lourenco, 1996).</td>
<td>58</td>
</tr>
<tr>
<td>2.22</td>
<td>Masonry stress arches in sagging (after Liu, 1997).</td>
<td>59</td>
</tr>
<tr>
<td>2.23</td>
<td>Crack pattern in hogging and sagging (after BRE, 1995).</td>
<td>60</td>
</tr>
<tr>
<td>2.24</td>
<td>Maximum principal strain distribution of masonry walls with different amount of window openings Giardina et al. (2015c).</td>
<td>61</td>
</tr>
<tr>
<td>3.1</td>
<td>Framework to assess building response to tunnelling.</td>
<td>68</td>
</tr>
<tr>
<td>3.2</td>
<td>Front view of the centrifuge model indicating the image-based deformation measurement equipment.</td>
<td>69</td>
</tr>
<tr>
<td>3.3</td>
<td>Model tunnel: (a) cross-section through tunnel centreline, (b) cross-section of model tunnel and (c) image of model tunnel.</td>
<td>70</td>
</tr>
<tr>
<td>3.4</td>
<td>Tunnel control system.</td>
<td>71</td>
</tr>
<tr>
<td>3.5</td>
<td>Sand pouring arrangement.</td>
<td>73</td>
</tr>
<tr>
<td>3.6</td>
<td>Building model (dimensions in mm).</td>
<td>74</td>
</tr>
<tr>
<td>3.7</td>
<td>Rough soil-structure interface.</td>
<td>75</td>
</tr>
<tr>
<td>3.8</td>
<td>Tracking of soil movements using PIV (after Take, 2003).</td>
<td>77</td>
</tr>
<tr>
<td>3.9</td>
<td>Centrifuge test series with varying building length, $L$, building eccentricity, $e$, façade openings, $O$, and building layout, $G$.</td>
<td>80</td>
</tr>
<tr>
<td>4.1</td>
<td>Overview of the 3DP procedure: (a) before printing, (b) throughout printing process and (c) finalised print job (after Feng et al., 2015).</td>
<td>85</td>
</tr>
<tr>
<td>4.2</td>
<td>Overview of 3D printer and main steps of creating the 3D printed building models.</td>
<td>87</td>
</tr>
<tr>
<td>4.3</td>
<td>Coordinate system adopted for the 3D printing procedure (after Feng et al., 2015).</td>
<td>88</td>
</tr>
<tr>
<td>4.4</td>
<td>Surface structure of 3D printed building model.</td>
<td>90</td>
</tr>
<tr>
<td>4.5</td>
<td>Orientation of building model and specimen in print bed.</td>
<td>91</td>
</tr>
<tr>
<td>4.6</td>
<td>Building model preparation after 3D printing.</td>
<td>92</td>
</tr>
<tr>
<td>4.7</td>
<td>Building model: (a) structural details and (b) cross-section (dimensions in mm).</td>
<td>93</td>
</tr>
<tr>
<td>4.8</td>
<td>Application of dead load bars for test G.</td>
<td>94</td>
</tr>
</tbody>
</table>
List of figures

4.9 3D printed specimen. Dimensions in mm. ....................................... 94
4.10 Four-point-bending test setup. ....................................................... 95
4.11 Stress-strain curve of the 3D printed material. ............................ 97
4.12 Reference scenario to estimate neutral axis of building models. ....... 102
4.13 Cross-section of building models of tests A to F in prototype scale (dimensions in m). .............................................................. 103
4.14 Calculation of effective height after Pickhaver et al. (2010). .......... 107
4.15 Comparison of EI values after applying window opening reduction. .. 109
4.16 Equivalent area considering façade openings (after Pickhaver et al., 2010). 111
4.17 Global building stiffness values of the centrifuge model buildings in prototype scale compared to field data and previous research. .......... 113

5.1 Centrifuge tests A, B, E and F used to address spin-up phenomena. ..... 118
5.2 Pressure in the tunnel pressure and volume loss control system during spin-up. .............................................................. 119
5.3 Comparison between tunnel pressure (σy) and vertical (σv) and horizontal soil stresses (σh). ................................................................. 120
5.4 Vertical (left) and horizontal (right) soil displacements (in mm) adjacent to the model tunnel. ............................................................. 121
5.5 Spin-up surface soil and base structure movements. ........................ 123
5.6 Building movement parameters to estimate spin-up building response. .. 124
5.7 Structure response after spin-up. Deflection ratio (DR), slope (s) and average horizontal base strain (εh, tensile strains are positive) are presented at 75g. .. 126
5.8 Comparison between LVDTs/lasers and GeoPIV measurements of test B. 128
5.9 Position of micro-electro mechanical systems (MEMS) accelerometers on top building models. MEMS not to scale. ......................... 130
5.10 Rotation of building models out of PMMA plane along tunnel volume loss. 131

6.1 Tunnel pressure versus tunnel volume loss. .................................... 135
6.2 Building position effects on vertical displacement contours at V_{t,t} = 2.0%. 137
6.3 Effect of building eccentricity (constant L and O) on soil volume loss, V_{l,s}, versus tunnel volume loss, V_{t,t}. ........................................... 138
6.4 Building position effects on vertical surface soil displacement profiles at V_{t,t} = 0.5%, 1.0%, 2.0% and 4.0%. .............................................. 140
6.5 Effect of building eccentricity on vertical surface soil displacement profiles at V_{t,t} = 2.0%. ................................................................. 141
List of figures

6.6 Effect of building eccentricity (constant L and O) on the ratio between maximum surface soil settlements, \( S_{v,max} \), and maximum greenfield surface soil settlements, \( S^{GF}_{v,max} \), versus tunnel volume loss, \( V_{l,t} \). ......................................................... 142
6.7 Building position effects on surface trough width parameter \( K^* \) versus tunnel volume loss, \( V_{l,t} \). ................................................................. 143
6.8 Building position effects on horizontal surface soil displacement profiles at \( V_{l,t} = 0.5\%, 1.0\%, 2.0\% \) and 4.0\% ................................. 145
6.9 Building position effects on horizontal surface soil displacements at a tunnel volume loss of 2.0\% ................................................................. 145
6.10 Effect of building eccentricity on surface displacement vectors at a tunnel volume loss of 2.0\% ................................................................. 147
6.11 Effect of building position on soil settlements at different depths (\( z/z_t = 0.03, 0.13 \) and 0.26) at \( V_{l,t} = 2.0\% \). ................................................................. 149
6.12 Effect of building eccentricity on vertical profiles of horizontal soil displacements at \( x = 70 \) mm and \( V_{l,t} = 2.0\% \). ................................................................. 150
6.13 Effect of building position on trough width parameter \( K^* \) versus soil depth. ................................................................. 151
6.14 Building position effects on engineering shear strain, \( \gamma_{xz} \), at \( V_{l,t} = 2.0\% \). ................................................................. 153
6.15 Building position effects on volumetric strain, \( \varepsilon_{vol} \), at \( V_{l,t} = 4.0\% \). ................................................................. 154
6.16 Building characteristic effects on vertical surface soil displacement profiles at \( V_{l,t} = 0.5\%, 1.0\%, 2.0\% \) and 4.0\%. ................................................................. 156
6.17 Building characteristic effects on the vertical surface soil displacement profiles at \( V_{l,t} = 2.0\% \). ................................................................. 157
6.18 Effect of building characteristics on the ratio between maximum surface soil settlements, \( S_{v,max} \), and maximum greenfield surface soil settlements, \( S^{GF}_{v,max} \). ................................................................. 158
6.19 Effect of building characteristics on soil volume loss, \( V_{l,s} \), versus tunnel volume loss, \( V_{l,t} \). ................................................................. 158
6.20 Effect of building characteristics on surface trough width parameter \( K^* \) versus tunnel volume loss, \( V_{l,t} \). ................................................................. 159
6.21 Inflection points of the modified Gaussian curves fitted to surface soil settlements. ................................................................. 160
6.22 Effect of building characteristics on horizontal surface soil displacement profiles at \( V_{l,t} = 0.5\%, 1.0\%, 2.0\% \) and 4.0\% ................................................................. 161
6.23 Building characteristic effects on the horizontal surface soil displacement profiles at \( V_{l,t} = 2.0\% \). ................................................................. 162
6.24 Effect of building characteristics on horizontal soil displacement, $S_h$, profiles at $x = 70$ mm and a tunnel volume loss, $V_{lt}$, of 2.0%. ........................................ 163

7.1 Structure vertical displacement contours at $V_{lt} = 2.0%$. ........................... 171

7.2 Vertical base structure displacements compared to vertical soil surface displacements at $V_{lt} = 0.5\%$, 1.0\%, 2.0\% and 4.0\%. .......................... 174

7.3 Separation of building model and soil surface at tunnel centreline ($x = 0$ mm) with increasing tunnel volume loss for test A. ................................. 175

7.4 Vertical structure displacements at base, neutral axis and top of structure at $V_{lt} = 2.0\%$. .......................................................... 177

7.5 Vertical base structure displacements fitted with modified Gaussian curves at $V_{lt} = 0.5\%$, 1.0\%, 2.0\% and 4.0\%. .......................... 178

7.6 Position of inflection points of the modified Gaussian curves fitted to the vertical base building displacements versus tunnel volume loss. ............. 180

7.7 Deflection ratios versus tunnel volume loss. ........................................... 181

7.8 Modification factors for the deflection ratios versus tunnel volume loss. .... 184

7.9 Structure horizontal displacement contours at $V_{lt} = 2.0\%$. Left horizontal displacements are negative while right horizontal displacements are positive. 185

7.10 Horizontal base structure displacements compared to horizontal soil surface displacements at $V_{lt} = 0.5\%$, 1.0\%, 2.0\% and 4.0\%. Left horizontal displacements are negative while right horizontal displacements are positive. .... 189

7.11 Horizontal structure displacements at base, neutral axis and top of structure at $V_{lt} = 2.0\%$. ...................................................... 190

7.12 Average horizontal building strains obtained at the location of the building’s neutral axis (tension is positive). ................................. 191

7.13 Modification factors for the horizontal strains. .......................................... 193

7.14 Building deformation parameters. .......................................................... 195

7.15 Subdivision of building at partition walls into building bays and notation of corner points for a building of 260 mm length. ................................. 196

7.16 Displacements of corner points (CP) of Bay 1 of test F. ............................. 196

7.17 Global building deformation parameters. ................................................. 197

7.18 Top horizontal strain for building bays. Positive strains indicate tension. ... 200

7.19 Angular distortion for building bays. ...................................................... 201

7.20 Cracking of test F. ................................................................. 204

7.21 Crack initiation and location. .............................................................. 205
List of figures

7.22 Subdivision of building models spanning hogging and sagging region (tests C, E and F) and comparison to buildings placed in hogging regions (tests B and D). 209
7.23 Building deformation parameters for the hogging parts of buildings with 20% façade openings (tensile strains are positive). 210
7.24 Building deformation parameters for the hogging parts of buildings with 40% façade openings (tensile strains are positive). 211
7.25 Building deformation parameters for the sagging parts (tensile strains are positive). 212
7.27 Scenarios to study building length effects on shear and bending deformations. Tunnel position and diameter are not to scale. 214
7.28 Influence of increasing $L/H$ on bending and shear deflections. 214
7.29 Scenarios to study building opening effects on shear and bending deformations. Tunnel position and diameter are not to scale. 215
7.30 Influence of increasing the opening percentage on bending and shear deflections. 216

8.1 Maximum building tensile strains based on measured building distortions and related greenfield predictions by adopting the limiting tensile strain method (Burland et al., 1977; Burland and Wroth, 1974). 227
8.2 Performance of the LTSM as a ratio between building tensile strains based on observed building distortions and greenfield distortions. 228
8.3 Maximum principal tensile strains based on measured building distortions and related greenfield predictions by adopting the state of strain criteria proposed by Son and Cording (2005). 230
8.4 Performance of the SoS as a ratio between building tensile strains based on observed building distortions and greenfield distortions. 232
8.5 Comparison between maximum building tensile strains derived with the limiting tensile strain method (LTSM) and the state of strain (SoS) concept at $V_{t,t} = 2.0\%$. 233
8.6 Damage parameter index (DPI) for limiting tensile strain (LTSM) and state of strain (SoS) criteria. 235
8.7 Comparison between LTSM and SoS predictions and derived building tensile strains versus $V_{t,t}$. 236
8.8 Degradation of soil stiffness with induced tunnel volume loss. Triaxial test data from Zhao (2008). 239
<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.9</td>
<td>Estimation of relative stiffness formulations according to literature. 243</td>
</tr>
<tr>
<td>8.10</td>
<td>Estimation of the relative stiffness according to Son and Cording (2005) and Boscardin and Cording (1989). 245</td>
</tr>
<tr>
<td>8.11</td>
<td>Predicted versus measured $M_{DR_{log}}$ along $V_{t,t}$. 247</td>
</tr>
<tr>
<td>8.12</td>
<td>Predicted versus measured $M_{DR_{avg}}$ along $V_{t,t}$. 250</td>
</tr>
<tr>
<td>8.13</td>
<td>Performance of relative stiffness methods to assess $M_{DR}$. 251</td>
</tr>
<tr>
<td>8.14</td>
<td>Predicted versus measured $M_{Eh}$ along $V_{t,t}$. 255</td>
</tr>
<tr>
<td>8.15</td>
<td>Performance of relative stiffness methods to assess $M_{Eh}$. 256</td>
</tr>
<tr>
<td>8.16</td>
<td>Measured versus predicted angular distortion according to Son and Cording (2005) along tunnel volume loss. 258</td>
</tr>
<tr>
<td>8.17</td>
<td>Performance of Son and Cording (2005) to assess $\beta$. 259</td>
</tr>
<tr>
<td>8.18</td>
<td>Measured versus predicted horizontal strain according to Boscardin and Cording (1989). 261</td>
</tr>
<tr>
<td>8.19</td>
<td>Performance of Son and Cording (2005) to assess $\varepsilon_{ht}$. 262</td>
</tr>
<tr>
<td>8.20</td>
<td>Upper bound building tensile strains based on relative stiffness predictions and experimentally obtained building distortions. 264</td>
</tr>
<tr>
<td>8.21</td>
<td>Lower bound building tensile strains based on relative stiffness predictions and experimentally obtained building distortions. 265</td>
</tr>
<tr>
<td>8.22</td>
<td>Ratio between building strains based on measured building distortions ($\varepsilon_{t,Str}$) and upper bound predictions ($\varepsilon_{t,RSM}$). 267</td>
</tr>
<tr>
<td>8.23</td>
<td>Performance of current relative stiffness methods to predict building strains caused by tunnel excavation at $V_{t,t} = 2.0%$. 268</td>
</tr>
<tr>
<td>9.1</td>
<td>Comparison between test F and G in the Goh and Mair (2011a) design chart for the estimation of modification factors based on relative building stiffness. 275</td>
</tr>
<tr>
<td>9.2</td>
<td>Contours of vertical stress changes beneath strip footings using linear, homogeneous, isotropic elastic theory (Boussinesq, 1885). 276</td>
</tr>
<tr>
<td>9.3</td>
<td>Estimation of participating soil width. 278</td>
</tr>
<tr>
<td>9.4</td>
<td>Revised $EI$ and $EA$ values for test G and comparison to field data. 279</td>
</tr>
<tr>
<td>9.5</td>
<td>Revised assessment of test G compared to test F in the Goh and Mair (2011a) design chart. 280</td>
</tr>
<tr>
<td>9.6</td>
<td>Observed modification factors for the deflection ratio in the Goh and Mair (2011a) design chart. 282</td>
</tr>
<tr>
<td>9.7</td>
<td>Field and previous centrifuge test data in the Goh and Mair (2011a) design chart indicating building position effects. 283</td>
</tr>
</tbody>
</table>
List of figures

9.8 Increase of modification factors for the deflection ratio with window opening percentage. ................................................................. 285
9.9 Normalised ratio between the global bending stiffness of tests with 40% façade openings and tests with 20% openings. ......................... 285
9.10 Design guidance to estimate the change of trough width due to soil–structure interaction mechanisms. ........................................ 287
9.11 Comparison between relative stiffness values based on greenfield building lengths (*i = 60 mm) and modified building lengths due to soil–structure interaction (SSI). ................................................. 287
9.12 Results of an elastic continuum-based two-stage analysis method (Franza and DeJong, 2017) exemplifying the narrowing of the relation between \( M^{DR} \) and \( \rho \) when considering soil–structure interaction mechanisms (adopted from Franz et al., 2017). ......................................................... 288
9.13 Recommendations to account for building characteristics when applying the Goh and Mair (2011a) framework to estimate building response to tunnelling-induced settlements. ................................................. 289

A.1 Building geometry for tests with \( L = 200 \text{ mm} \): (a) front, (b) side and (c) top view (dimensions in mm). ........................................ 316
A.2 Building geometry for tests with \( L = 260 \text{ mm} \): (a) front, (b) side and (c) top view (dimensions in mm). ........................................ 317
A.3 Facade geometry for tests A-C (dimensions in mm). .......................... 318
A.4 Facade geometry for tests D (dimensions in mm). .............................. 318
A.5 Facade geometry for test E (dimensions in mm). ................................. 319
A.6 Facade geometry for tests F and G (dimensions in mm). ................. 319
# List of tables

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.1</td>
<td>Relevant publications.</td>
<td>8</td>
</tr>
<tr>
<td>2.1</td>
<td>Volume loss at surface of example EPB projects.</td>
<td>12</td>
</tr>
<tr>
<td>2.2</td>
<td>Curves to characterise settlement profiles above tunnels (adopted after Marshall et al., 2012).</td>
<td>19</td>
</tr>
<tr>
<td>2.3</td>
<td>Classification of visible damage after Burland et al. (1977).</td>
<td>28</td>
</tr>
<tr>
<td>2.4</td>
<td>Angular distortion limits by Skempton and MacDonald (1956).</td>
<td>29</td>
</tr>
<tr>
<td>2.5</td>
<td>Deflection ratio limits by Polshin and Tokar (1957).</td>
<td>29</td>
</tr>
<tr>
<td>2.6</td>
<td>Building slope and settlement limits by Rankin (1988).</td>
<td>30</td>
</tr>
<tr>
<td>2.7</td>
<td>Limiting tensile strain values linked to building damage (after Boscardin and Cording, 1989).</td>
<td>32</td>
</tr>
<tr>
<td>2.8</td>
<td>Example variables influencing the soil–structure interaction (after Boone, 2008).</td>
<td>41</td>
</tr>
<tr>
<td>2.9</td>
<td>Bending stiffness reduction factors due to façade openings and $L/H$ ratio (Melis and Rodriguez Ortiz, 2001).</td>
<td>49</td>
</tr>
<tr>
<td>3.1</td>
<td>Scaling laws relevant for this research (after Kutter, 1992).</td>
<td>64</td>
</tr>
<tr>
<td>3.2</td>
<td>Leighton Buzzard fraction E silica sand properties (Tan, 1990).</td>
<td>73</td>
</tr>
<tr>
<td>3.3</td>
<td>Details of the test series.</td>
<td>81</td>
</tr>
<tr>
<td>4.1</td>
<td>3D printed material properties compared to typical masonry properties from Giardina et al. (2015c).</td>
<td>99</td>
</tr>
<tr>
<td>4.2</td>
<td>Effect of different curing temperatures on 3D printed material properties.</td>
<td>100</td>
</tr>
<tr>
<td>4.3</td>
<td>Position of neutral axis for the centrifuge tests.</td>
<td>102</td>
</tr>
<tr>
<td>4.4</td>
<td>Global building bending stiffness following Melis and Rodriguez Ortiz (2001).</td>
<td>105</td>
</tr>
<tr>
<td>4.5</td>
<td>Global building shear stiffness following Son and Cording (2005).</td>
<td>106</td>
</tr>
<tr>
<td>4.6</td>
<td>Global building bending stiffness following Pickhaver et al. (2010).</td>
<td>108</td>
</tr>
<tr>
<td>4.7</td>
<td>Adopted opening reduction factors.</td>
<td>108</td>
</tr>
<tr>
<td>4.8</td>
<td>Adopted opening reduction factors.</td>
<td>109</td>
</tr>
</tbody>
</table>
### List of tables

<table>
<thead>
<tr>
<th>Number</th>
<th>Table Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>4.9</td>
<td>Global axial building stiffness.</td>
<td>111</td>
</tr>
<tr>
<td>4.10</td>
<td>Global axial building stiffness according to Boscardin and Cording (1989).</td>
<td>112</td>
</tr>
<tr>
<td>4.11</td>
<td>$EA$ and $EI$ of centrifuge model buildings.</td>
<td>115</td>
</tr>
<tr>
<td>7.1</td>
<td>Visible cracking.</td>
<td>207</td>
</tr>
<tr>
<td>8.1</td>
<td>Damage criteria for tunnelling-induced building damage assessments (adapted from Finno et al., 2005).</td>
<td>222</td>
</tr>
<tr>
<td>8.2</td>
<td>Input data for analytical assessment according to the limiting tensile strain method.</td>
<td>225</td>
</tr>
<tr>
<td>8.3</td>
<td>Assessment of the performance of the limiting tensile strain method.</td>
<td>229</td>
</tr>
<tr>
<td>8.4</td>
<td>Assessment of the performance of the state of strain concept.</td>
<td>232</td>
</tr>
<tr>
<td>8.5</td>
<td>Soil stiffness values used to estimate relative stiffness.</td>
<td>241</td>
</tr>
<tr>
<td>8.6</td>
<td>Assessment of the performance of the relative stiffness methods.</td>
<td>268</td>
</tr>
<tr>
<td>8.7</td>
<td>Performance of relative stiffness methods to predict damage categories.</td>
<td>270</td>
</tr>
<tr>
<td>9.1</td>
<td>Revised $EA$ and $EI$ values of test G.</td>
<td>279</td>
</tr>
</tbody>
</table>
Nomenclature

Roman Symbols

\( A \)  
Area

\( a \)  
Tunnel radius (Clough and Schmidt, 1981)

\( A^* \)  
Area considering window openings according to Pickhaver et al. (2010)

\( A_E \)  
Material constant (Lehane and Cosgrove, 2000)

\( a_j \)  
Area of a horizontal strip after Pickhaver et al. (2010)

\( B \)  
Building width parallel to tunnel

\( b \)  
Width

\( B_s \)  
Participating soil width parallel to tunnel advance

\( C_t \)  
Depth of tunnel cover

\( c_a - c_d \)  
Coefficients used to normalise \( V_{l,t} \) with \( V_{l,I} \) (Marshall et al., 2012)

\( D \)  
Mid-span deflection (ASTM Standards, 1986)

\( d \)  
Grain size

\( D_{ci} \)  
Cavity diameter (Kutter et al., 1994)

\( P \)  
Depth (height) according to ASTM Standards (1986)

\( DR \)  
Deflection ratio

\( D_t \)  
Tunnel diameter

\( E \)  
Young’s modulus
### Nomenclature

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>$e$</td>
<td>Building eccentricity</td>
</tr>
<tr>
<td>$e_{max}$</td>
<td>Maximum voids ratio</td>
</tr>
<tr>
<td>$e_{min}$</td>
<td>Minimum voids ratio</td>
</tr>
<tr>
<td>$E_s$</td>
<td>Soil stiffness</td>
</tr>
<tr>
<td>$E_v'$</td>
<td>Secant axial stiffness (Lehane and Cosgrove, 2000)</td>
</tr>
<tr>
<td>$E_{v0}$</td>
<td>Maximum stiffness of soil at very low strain (Lehane and Cosgrove, 2000)</td>
</tr>
<tr>
<td>$f_c$</td>
<td>Compressive strength</td>
</tr>
<tr>
<td>$F(e)$</td>
<td>Function of void ratio, $e$, used to determine soil stiffness (Lehane and Cosgrove, 2000)</td>
</tr>
<tr>
<td>$f_t$</td>
<td>Flexural strength (ASTM Standards, 1986)</td>
</tr>
<tr>
<td>$f_t$</td>
<td>Tensile strength</td>
</tr>
<tr>
<td>$G$</td>
<td>Building layout (or geometry)</td>
</tr>
<tr>
<td>$G$</td>
<td>Shear modulus</td>
</tr>
<tr>
<td>$g$</td>
<td>Earth’s gravity field</td>
</tr>
<tr>
<td>$G_c$</td>
<td>Fracture energy in compression</td>
</tr>
<tr>
<td>$G_f$</td>
<td>Fracture energy in tension</td>
</tr>
<tr>
<td>$G_s$</td>
<td>Specific gravity</td>
</tr>
<tr>
<td>$H$</td>
<td>Building height</td>
</tr>
<tr>
<td>$I$</td>
<td>Moment of inertia</td>
</tr>
<tr>
<td>$i$</td>
<td>Inflection point</td>
</tr>
<tr>
<td>$I_{A−A}$</td>
<td>Second moment of area of solid cross-section</td>
</tr>
<tr>
<td>$I^*$</td>
<td>Second moment of area considering window openings according to Pickhaver et al. (2010)</td>
</tr>
<tr>
<td>$I_D$</td>
<td>Soil relative density</td>
</tr>
</tbody>
</table>
Nomenclature

$I_{sm}$ Second moment of area according to the strip method (Pickhaver et al., 2010)

$K$ Trough width parameter

$k$ Reduction factor for the second moment of area after Pickhaver et al. (2010)

$K^*$ Trough width parameter based on $x^*$ (Marshall et al., 2012)

$K_{s^*}$ Trough width parameter for surface displacements based on $x^*$ (Marshall et al., 2012)

$K_{s,C/D_t}^{*\text{int}}$ intercept of $K_{s,V_l}^{*\text{slope}}$ values with $C/D_t$ (Marshall et al., 2012)

$K_{s,C/D_t}^{*\text{slope}}$ slope of $K_{s,V_l}^{*\text{slope}}$ with $C/D_t$ line (Marshall et al., 2012)

$K_{s,V_l}^{*\text{slope}}$ slope of $K_s^*$ against $V_{l,t}$ (Marshall et al., 2012)

$L$ Building length perpendicular to tunnel

$L$ Support span (ASTM Standards, 1986)

$M$ Modification factor

$m$ Slope of the tangent to the initial straight-line portion of the load-deflection curve (ASTM Standards, 1986)

$m$ Subsurface trough width parameter (Moh et al., 1996)

$N$ Centrifuge acceleration

$n$ Empirical constant adopted to modify the shape of soil stiffness degradation curves (Lehane and Cosgrove, 2000)

$n$ Shape function parameter for modified Gaussian curve (Vorster et al., 2005)

$O$ Façade openings

$P$ Arbitrary point (Boussinesq, 1885)

$P$ Peak load (ASTM Standards, 1986)

$p_{atm}$ Atmospheric pressure (=100 kPa)
Nomenclature

\( q \) Building load per unit area (Boussinesq, 1885)

\( R^2 \) Coefficient of determination

\( S \) Distance between two strip footings

\( s \) Spacing factor

\( s \) slope

\( S_h \) Horizontal displacements

\( S_v \) Vertical displacements

\( t \) Distance of the neutral axis from the edge of the beam in tension

\( V_{ci} \) Volume of cavity (Kutter et al., 1994)

\( V_{cr} \) Volume of crater (Kutter et al., 1994)

\( V_l \) Volume loss

\( V_{l,s} \) Volume of soil settlement trough

\( V_{l,t} \) Tunnel volume loss

\( x^* \) Horizontal distance from \( x = 0 \) to point on fitted curve where \( S_v = 0.606S_{v,max} \) (Marshall et al., 2012)

\( x^{**} \) Horizontal distance from \( x = 0 \) to point on fitted curve where \( S_v = 0.303S_{v,max} \) (Marshall et al., 2012)

\( z \) Soil depth

\( \bar{z} \) Centroid of building model

\( z_t \) Tunnel depth

Greek Symbols

\( \alpha \) Angular strain

\( \alpha_{red} \) Bending stiffness reduction factor due to window openings

\( \alpha \) Shape parameter for modified Gaussian curve
Nomenclature

\[ \alpha^* \] Relative axial stiffness

\[ \beta \] Angular distortion

\[ \beta \] Parameter to normalise \( V_{l,z} \) with \( C/D_t \) (Marshall et al., 2012)

\[ \beta \] Angle between two lines connecting \( P \) with foundation end points (Boussinesq, 1885)

\[ \Delta \] Relative building deflection

\[ \delta \] Angle between the vertical and the line between \( P \) and right hand side foundation endpoint (Boussinesq, 1885)

\[ \Delta GS \] Change of the ground slope (Son and Cording, 2005)

\[ \varepsilon_a \] Axial strain (triaxial test)

\[ \varepsilon_{b,\text{max}} \] Maximum bending strain (= extreme fibre strain)

\[ \varepsilon_{br} \] Resultant extreme fibre strain

\[ \varepsilon_{\text{crit}} \] Critical tensile strain

\[ \varepsilon_{d,\text{max}} \] Maximum diagonal tensile strain

\[ \varepsilon_{dr} \] Resultant diagonal tensile strain

\[ \varepsilon_{el} \] Strain at which soil stiffness to strain relation becomes non-linear (Lehane and Cosgrove, 2000)

\[ \varepsilon_h \] Horizontal strain

\[ \varepsilon_{hc} \] Horizontal strain in compression

\[ \varepsilon_{hr} \] Horizontal strain in tension

\[ \varepsilon_{lim} \] Limiting tensile strain

\[ \varepsilon_p \] Maximum principal tensile strain

\[ \varepsilon_r \] Strain when \( E'_i \) is half of \( E_{\varepsilon 0} \) (Lehane and Cosgrove, 2000)

\[ \varepsilon_{t,\text{crack}} \] Cracking strain of structure (Son and Cording, 2005)
### Nomenclature

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\varepsilon_{ult}$</td>
<td>Ultimate strain to failure (ASTM Standards, 1986)</td>
</tr>
<tr>
<td>$\varepsilon_{vol}$</td>
<td>Volumetric strain</td>
</tr>
<tr>
<td>$\varepsilon_{xx}$</td>
<td>Principal strain component in the $x$ plane</td>
</tr>
<tr>
<td>$\varepsilon_{zz}$</td>
<td>Principal strain component in the $z$ plane</td>
</tr>
<tr>
<td>$\gamma$</td>
<td>Unit weight of soil</td>
</tr>
<tr>
<td>$\gamma_{xz}$</td>
<td>Engineering shear strain</td>
</tr>
<tr>
<td>$\gamma_{w}$</td>
<td>Unit weight of water</td>
</tr>
<tr>
<td>$\lambda$</td>
<td>Window opening reduction factor for axial building stiffness</td>
</tr>
<tr>
<td>$\omega$</td>
<td>Angular velocity</td>
</tr>
<tr>
<td>$\phi$</td>
<td>Friction angle</td>
</tr>
<tr>
<td>$\rho$</td>
<td>Density</td>
</tr>
<tr>
<td>$\rho^*$</td>
<td>Relative bending stiffness</td>
</tr>
<tr>
<td>$\rho_{hog}$</td>
<td>Relative bending stiffness in hogging (Goh and Mair, 2011a)</td>
</tr>
<tr>
<td>$\rho_{sag}$</td>
<td>Relative bending stiffness in sagging (Goh and Mair, 2011a)</td>
</tr>
<tr>
<td>$\sigma_h$</td>
<td>Horizontal soil stress</td>
</tr>
<tr>
<td>$\sigma_t$</td>
<td>Internal tunnel pressure</td>
</tr>
<tr>
<td>$\sigma_{r,n}$</td>
<td>Normalised tunnel pressure</td>
</tr>
<tr>
<td>$\sigma_v$</td>
<td>Vertical soil stress</td>
</tr>
<tr>
<td>$\theta$</td>
<td>Tilt</td>
</tr>
<tr>
<td>$\theta_{max}$</td>
<td>Direction of crack formation (angle between the vertical and the plane on which $\varepsilon_p$ acts) according to Son and Cording (2005)</td>
</tr>
</tbody>
</table>

### Superscripts

<table>
<thead>
<tr>
<th>Superscript</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>$I$</td>
<td>Mode $I$ failure of masonry</td>
</tr>
</tbody>
</table>
II       Mode II failure of masonry

Subscripts

\(crit\)    Critical state
\(eff\)     Effective
\(eq\)      Equivalent
\(f\)       Foundation or footing
\(l\)       Loss
\(max\)     Maximum
\(mG\)      Modified Gaussian curve
\(min\)     Minimum
\(mod\)     Modified
\(red\)     Reduced
\(SC\)      Son and Cording (2005)
\(sec\)     Secant
\(sm\)      Strip method (Pickhaver et al., 2010)
\(su\)      Spin-up
\(t\)       Tunnel
\(tot\)     Total
\(w\)       Façade wall

Acronyms / Abbreviations

3D       Three-dimensional
3DP      Three-dimensional printing
\(BC\)    Boscardin and Cording (1989)
<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>CP</td>
<td>Corner points</td>
</tr>
<tr>
<td>DBM</td>
<td>Deep beam method</td>
</tr>
<tr>
<td>DIC</td>
<td>Digital image correlation</td>
</tr>
<tr>
<td>DPI</td>
<td>Damage parameter index (Schuster et al., 2009)</td>
</tr>
<tr>
<td>EPB</td>
<td>Earth pressure balanced</td>
</tr>
<tr>
<td>Fra.</td>
<td>Franzius et al. (2006)</td>
</tr>
<tr>
<td>G&amp;M</td>
<td>Goh and Mair (2011a,c)</td>
</tr>
<tr>
<td>GF</td>
<td>Greenfield (where no buildings are present)</td>
</tr>
<tr>
<td>hog</td>
<td>Hogging</td>
</tr>
<tr>
<td>LBM</td>
<td>Laminated beam model</td>
</tr>
<tr>
<td>LTSM</td>
<td>Limiting tensile strain method</td>
</tr>
<tr>
<td>LVDT</td>
<td>Linear variable differential transformers</td>
</tr>
<tr>
<td>MRO</td>
<td>Melis and Rodriguez Ortiz (2001)</td>
</tr>
<tr>
<td>NATM</td>
<td>New Austrian tunnelling method</td>
</tr>
<tr>
<td>P&amp;A</td>
<td>Potts and Addenbrooke (1997)</td>
</tr>
<tr>
<td>PAI</td>
<td>Prediction accuracy index (Schuster et al., 2009)</td>
</tr>
<tr>
<td>PMMA</td>
<td>Poly(methyl methacrylate)</td>
</tr>
<tr>
<td>PPT</td>
<td>Pore water pressure transducer</td>
</tr>
<tr>
<td>RP</td>
<td>Rapid prototyping</td>
</tr>
<tr>
<td>RSM</td>
<td>Relative stiffness method</td>
</tr>
<tr>
<td>S&amp;C</td>
<td>Son and Cording (2005)</td>
</tr>
<tr>
<td>sag</td>
<td>Sagging</td>
</tr>
<tr>
<td>SCL</td>
<td>Shotcrete lining</td>
</tr>
<tr>
<td>Acronym</td>
<td>Description</td>
</tr>
<tr>
<td>---------</td>
<td>-------------</td>
</tr>
<tr>
<td>SD</td>
<td>Standard deviation</td>
</tr>
<tr>
<td>SoS</td>
<td>State of strain concept</td>
</tr>
<tr>
<td>SSM</td>
<td>Strain superposition method</td>
</tr>
<tr>
<td>Str</td>
<td>Structure</td>
</tr>
<tr>
<td>TBM</td>
<td>Tunnel boring machine</td>
</tr>
</tbody>
</table>
Chapter 1

Introduction

1.1 Background

With continuing population growth and increasing urbanization expected in the next decades, there is an urgent need for a next generation of infrastructure. It is likely that extensive parts of this infrastructure will be underground. According to Collins (2013), about 1,000 km of tunnels were expected to be excavated in 2014 and a large part are dug underneath congested cities. This trend will most likely continue within coming years and further tunnels for transportation, water distribution, sewer systems, power tunnels and flood control will be excavated. Examples of such urban transportation tunnelling projects currently under construction are the Crossrail and the Thames Tideway Tunnel project in London, the Doha Metro, the Follo Line in Oslo, the Delhi Metro Phase III project and the Grand Paris Express.

However, tunnelling in an urban environment comes with significant challenges. Subsurface obstacles such as existing tunnels, buried utilities such as water, gas and sewer pipelines or deep foundations can significantly affect the tunnel design. On the other hand, buildings at the surface also interact with the tunnel construction. Tunnel excavation generates a change of the stress state of the soil above a tunnel and ground movements towards the tunnel cannot be avoided. These ground movements translate into surface settlements and may have an impact on existing buildings. Along a major urban tunnelling scenario such as, for example, the Crossrail project, more than 1,200 buildings are located within a possible influence area of the tunnel construction (Torp-Petersen and Black, 2001).

Building damage can be a significant contribution to the cost of tunnel construction projects. For instance, the construction of the Dublin Port Tunnel generated 334 claims of building damage in the form of hairline cracks (40), cracks (256), and damage of windows (8) (Brennan, 2007). According to Clarke and Laefer (2014), these claims add about €3.5 million
Introduction

To the project costs of €754 million and about 1 in every 8 houses along the tunnel alignment was affected. This example shows the direct impact of building damage on the project costs and highlights that tunnelling-induced settlements often result in minor damage (e.g. cracking) affecting the serviceability of buildings. Specifically, masonry buildings, which comprise a majority of the global building stock and are often of considerable heritage value, are particularly vulnerable to differential displacements. Predicting minor building damage is difficult and requires understanding of the nature of the soil–structure interaction. Clearly, examples of building damage caused by tunnelling lead to a negative public opinion about urban tunnel construction projects. Thus, it is essential to reliably predict the impacts of tunnel construction which would potentially occur, and if necessary to provide adequate mitigation measures.

From extensive research, useful models to predict tunnelling-induced settlement impact on buildings exist. These methods are generally applied using a three-staged process, with increasing detail of the risk scenario considered at each stage (Mair et al., 1996). In the first stage ground movements along the route are predicted for greenfield scenarios (where the existing buildings are neglected). If the soil deformations exceed defined thresholds, buildings located within this area qualify for further evaluation. Building tensile strains caused by the greenfield settlements, predicted in the first stage, are estimated within this second stage following the limiting tensile strain method (LTSM) of Burland and Wroth (1974) and Boscardin and Cording (1989). If these tensile strains indicate unacceptable building damage, buildings are assessed in further detail in the third stage, where building features and soil–structure interaction effects are taken into account. Numerical methods such as finite element analysis are often applied in the third stage and it is widely accepted that this third stage is complex, time consuming and costly.

1.2 Motivation

The above introduced three-stage approach was, for instance, applied to the Crossrail project. Torp-Petersen and Black (2001) reported that 428 buildings (208 listed buildings) along the tunnel route were identified to be above the accepted risk level of the second stage and detailed evaluation in the third stage was necessary. However, the results of the third stage led to a significant reduction of the predicted damage of many of these buildings (Torp-Petersen and Black, 2001). This shows that the methods used in the second stage, which neglects any interaction between the soil and the structure, generally lead to overly conservative predictions of the building damage. However, researchers have also reported that the second stage assessment in the form of the LTSM can also result in underestimation of the building damage when
the physical state of buildings such as a large amount of façade openings or pre-existing damage are not adequately considered within the damage assessment (Clarke and Laefer, 2014; Giardina et al., 2012). These shortcomings indicate a need for refinement. Researchers tackled this issue by providing relationships which account for soil–structure interaction effects through a measure of the building stiffness relative to the soil stiffness (i.e. Franzius et al., 2006; Goh and Mair, 2011a; Potts and Addenbrooke, 1997; Son and Cording, 2005). These so-called relative stiffness methods (RSM) might be applied in the second stage of this assessment approach and are a valuable contribution to refine predictions.

These alternative procedures, however, have not been extensively applied in practice due to concerns about their accuracy and reliability. Specific uncertainties which might have detrimental effects on the accuracy of the predicted damage are, for instance, the assessment of the overall stiffness of an affected building per metre run, the normalisation with the soil stiffness and the relevant building dimensions. In particular, much uncertainty still exists about the relationship between building details (e.g. building layout, façade openings, etc.) and the overall building stiffness (Haji et al., 2018). Consequently, recent literature has emerged that reported inconsistent results when applying the relative stiffness formulations available (DeJong et al., 2016; Giardina et al., 2017).

Although an extensive set of case studies exists, there is a lack of detailed building monitoring data to accurately evaluate the performance of the relative stiffness methods available. Moreover, field data is inherently affected by numerous uncertainties related to the ground, the asset and the tunnelling process. Because of this, various assumptions are often required to isolate important parameters affecting the interaction between a building and the soil during the tunnel excavation. Recent research pointed out that far too little informative building monitoring data exists to accurately assess the building response and thus reliably validate the relative stiffness formulations available (DeJong et al., 2016). For this reason, there is an urgent need to provide controlled experimental data that investigates the response of complex buildings to tunnelling-induced subsidence.

Most research related to soil–structure interaction during the tunnelling process can be related to the influence of the building stiffness on the ground and building response. However, the surface structures modelled in numerical (Franzius et al., 2006; Goh and Mair, 2011c; Potts and Addenbrooke, 1997) and experimental studies (Al Heib et al., 2013; Caporaletti et al., 2005; Taylor and Grant, 1998; Taylor and Yip, 2001) were mainly simplified plate or beam models. Specifically, existing centrifuge model tests, which accurately capture the self-weight stress state throughout the structure and the soil, were limited to simple structure models in the form of rubber, aluminium, micro concrete and masonry plates or beams (Capo-
Introduction

raletti et al., 2005; Farrell, 2010; Taylor and Grant, 1998; Taylor and Yip, 2001). More recent numerical modelling studies, however, showed the vital role of building characteristics such as, for example, stress localisation effects in the vicinity of façade openings (Burd et al., 2000; Giardina et al., 2013; Pickhaver et al., 2010; Son and Cording, 2007; Yiu et al., 2017) and the non-linear behaviour of the building material (Amorosi et al., 2014; Boonpichetvong and Rots, 2005; Giardina et al., 2013; Son and Cording, 2005) but these building feature effects have been disregarded to a large extent in physical modelling research. Consequently, experimental data about the influence of building features on this tunnel–soil–structure interaction problem are still missing.

1.3 Research objectives and methodology

The main objective of this research is to conduct a wide-ranging experimental investigation into the effects of tunnelling on masonry buildings to quantify the soil–structure interaction. More precisely, this research aims to provide a deeper understanding of:

- the influence of the building-to-tunnel position on the soil and building response during the tunnel construction process.

- the effects of façade openings on this soil–structure interaction problem.

- the influence of building length differences on the behaviour of surface structures to tunnelling-induced ground movements.

- the three-dimensional nature of this tunnel–soil–structure interaction problem by exploring building layout effects.

It is intended to achieve these objectives by performing a series of centrifuge tests using more complex building models. Small-scale three-dimensionally printed structures for centrifuge testing were designed and fabricated to model full-scale masonry structures on shallow foundations. The centrifuge tests focused on isolating effects of building characteristics including façade openings, the building-to-tunnel position and different building layouts.

The practical aim of this research is to use the comprehensive set of benchmark data, and the improved understanding obtained throughout interpretation of that data, to provide guidance on how to better predict the response of buildings subject to tunnelling induced settlements. In particular, the results of this centrifuge testing program provide data to validate and improve both computational models and methods to predict building damage. Consequently,
the obtained results provide a framework to refine the procedures for predicting tunnelling-induced settlement damage.

Another main objective of this research is to disentangle some of the uncertainties related to applying simplified methods to predict building response to tunnelling-induced soil displacements including the LTSM and the RSMs. To achieve this objective, the results of the centrifuge tests will be compared to predictions of these damage assessment procedures available. Evaluating these available methodologies provides required insight into the accuracy and reliability of these procedures. The improved understanding is then translated into recommendations for practical implementations.

1.4 Thesis outline and related publications

The remaining part of this thesis proceeds as follows:

• **Chapter Two** reviews existing research on building behaviour to tunnelling-induced soil displacements. This involves a summary of ground and building response to tunnelling. Available methods to predict building damage due to tunnelling settlements are reviewed.

• **Chapter Three** is concerned with the methods and equipment used during the centrifuge testing programme. Details of the instruments and techniques used to monitor the building and ground displacements throughout the experiments are presented. Additionally, an outline of the conducted centrifuge tests is shown.

• **Chapter Four** discusses the novel 3D printed centrifuge building models. This addresses the applied 3D printing technique and summarises the derived material properties of the 3D printed material and the building models. Results of a preliminary investigation to improve the strength properties of the 3D printed material are presented. Additionally, Chapter 4 discusses various methods of estimating the overall building stiffness of the small-scale building models.

• **Chapter Five** reports challenges observed during the conducted centrifuge test series. Specifically, soil and building displacements during the centrifuge acceleration are presented. The obtained results highlight the interaction between the model tunnel, the soil and the building model as the g-level increases. Additionally, boundary effects observed throughout the testing programme are quantified.
• **Chapter Six** addresses building effects on the soil response during the tunnel excavation stage. More specifically, centrifuge test results highlighting the influence of the position of the surface structure relative to the tunnel, the opening percentage of the building models, the building dimensions and the building layout on the soil behaviour are presented. This includes a quantification of the ground loss, the maximum surface settlements, the shape of surface and subsurface settlement troughs, the restraining effect of buildings to horizontal ground displacements and the development of shear and volumetric soil strains.

• **Chapter Seven** is concerned with the building response to tunnelling-induced ground displacements. The effects of building characteristics on building deformation parameters are presented. Furthermore, the approach of analysing a building separately on either side of the greenfield inflection point is assessed and the crucial role of building features on the governing mode (i.e. shear or bending) of building deformations is pointed out. Cracking damage of the brittle surface structure is reported.

• **Chapter Eight** evaluates existing procedures to predict building damage due to tunnelling-induced soil displacements by comparing their predictions with centrifuge test results, which were presented in Chapter 7. In particular, the reliability of available methods to predict building damage caused by tunnelling works is discussed.

• **Chapter Nine** translates the key findings of this experimental investigation on tunnel–soil–structure interaction effects into recommendations for practical application. This includes a novel framework to account for out of plane-strain building geometry differences when assessing the building stiffness per metre run and suggestions to consider building characteristics when applying the relative stiffness expressions proposed by Goh and Mair (2011a).

• **Chapter Ten** provides the conclusions of the previous chapters, addresses the applicability of the obtained results and indicates future research paths.

An overview of the dissertation outline is presented in Figure 1.1 while Table 1.1 lists publications that relate to certain chapters of the thesis.
1.4 Thesis outline and related publications

Fig. 1.1 Thesis outline.

Chapter 1
INTRODUCTION
Background, motivation, objectives and outline

Chapter 2
LITERATURE REVIEW
Available methods to predict building damage due to tunnelling subsidence

Chapter 3
EXPERIMENTAL TECHNIQUES

Chapter 4
3D PRINTED BUILDINGS

Chapter 5
MODELLING EFFECTS
Spin-up phenomena, boundary effects

Chapter 6
SOIL RESPONSE
Effect of building characteristics on soil displacements

Chapter 7
BUILDING RESPONSE
Effect of building characteristics on structural distortions, building damage

Chapter 8
EVALUATION
Evaluation of current building damage assessment methods

Chapter 9
RECOMMENDATIONS
Recommendations for practical implementations

Chapter 10
CONCLUSIONS
Summary of results, limitations and future research
### Table 1.1 Relevant publications.

<table>
<thead>
<tr>
<th>Publication</th>
<th>Chapter</th>
</tr>
</thead>
</table>
Chapter 2

Literature review

This chapter provides an overview of the available concepts of assessing the interaction between a surface structure and tunnelling-induced ground displacements. In the following, the causes of soil deformations due to tunnelling are first presented, after which empirical procedures to characterise these displacements are reviewed. Subsequently, concepts that describe the soil–structure interaction during the tunnelling procedure are presented. Finally, the response of masonry to tunnelling-induced subsidence is discussed.

2.1 Ground deformations caused by tunnelling

Rankin (1988) distinguished between two mechanisms of tunnelling-induced ground movements, which are arranged by occurrence in time related to the tunnel construction: (1) short-term settlements caused by stress relief due to the tunnel excavation and (2) long-term settlements due to consolidation effects. The shape and magnitude of these soil displacements are affected by various factors including the mechanical properties of the ground surrounding the tunnel, the applied construction technique and the quality of the workmanship (Rankin, 1988).

Tunnelling techniques in soft ground may be classified as open face tunnelling and closed face tunnelling based on the demand of support needed (Mair, 2013). Closed face tunnelling techniques using earth pressure balance (EPB) or slurry shields generally result in significantly lower ground movements than open face tunnelling techniques (e.g. open shield tunnelling or the sprayed concrete lining (SCL) method without face support). Figure 2.1 depicts potential sources of ground movements caused by shield tunnelling (Attewell, 1978; Mair and Taylor, 1997; Shirlaw et al., 2003; Viggiani and Soccodato, 2004) which can be summarised as follows:
1. **Face loss**: Deformation of the soil and movement of the soil towards the tunnel heading as a result of stress state changes around the tunnel face.

2. **Shield loss**: Radial ground movement around the tunnel as a consequence of overexcavation, issues with TBM attitude control or pushing of boulders.

3. **Tail void loss**: Radial ground loss of soil due to the closer of the physical void between the shield and the tunnel lining which can be minimised by timely tail void injection.

4. **Lining deflection loss**: Radial movement of the tunnel lining until an equilibrium between the lining support and the overburden pressure is reached. In urban tunnelling with relatively shallow overburden, a stiff support is usually installed; thus, lining deflections are generally small (Viggiani and Soccodato, 2004).

5. **Consolidation loss**: Long-term settlements as a consequence of pore water pressure changes and associated changes of the effective soil stresses.

For SCL tunnelling, which is often also refereed to as the New Austrian Tunnelling Method (NATM), the face loss (1), the lining deflection loss (4) and the consolidation loss (5) is applicable.

A common term to describe the sum of the short-term ground movements (1 to 4) is ‘volume loss’, $V_l$, which is specified as the ratio of the volume (per metre length) of soil moving into the tunnel, $V$, and the original volume of the tunnel (per meter length). For a circular...
2.1 Ground deformations caused by tunnelling

tunnel the volume loss, $V_l$, can be expressed as

$$V_l = 100 \frac{4V}{\pi D_t^2} \%$$

(2.1)

where $D_t$ is the tunnel diameter. For practical reasons, $V$ is generally estimated through measurements at soil surface level, neglecting that the soil volume moving into the tunnel may be different from the volume of the settlement trough measured at soil surface (Marshall et al., 2012). The reasons for this change of soil volume loss with distance from the tunnel is further discussed in Section 2.1.2.3. The volume loss at soil surface is often termed $V_{l,s}$ and generally the volume loss measured in practice. In this thesis, $V_{l,s}$ is used to refer to the volume loss at soil surface or at different soil depth, while $V_{l,t}$ is used to define the volume loss experienced by the tunnel.

The parameter volume loss is widely accepted when assessing the performance of tunnel excavation. Mair and Taylor (1997) extensively analysed field data and classified the reported settlement data with respect to the used tunnelling technique and ground condition. This previous work showed that open face tunnelling in stiff clay and SCL tunnelling without face support often result in volume losses of 1.0% to 2.0% and 0.5% to 1.5% respectively. For pressurised closed face tunnelling, EPB or slurry shield tunnelling, a high degree of ground control can be achieved and volume losses between 1.0% and 2.0% in soft clays and as low as 0.5% for sand were reported by Mair and Taylor (1997). Recent advancements in EPB and slurry shield tunnelling techniques have resulted in further reduction of $V_l$, as shown in Table 2.1. This is confirmed by a recent theoretical study of Vu et al. (2016) to estimate volume loss for shallow tunnels in different ground conditions.

By contrast, Shirlaw et al. (2003) conducted an extensive study on the occurrence of large settlements such as sinkholes and or loss of ground during EPB tunnelling for the North East Line in Singapore. Although the volume loss could generally be well controlled for this tunnelling case (see Table 2.1), local areas showed volume losses exceeding 5.0%. Shirlaw et al. (2003) reported that with EPB and also slurry shields similar localised scenarios of significant magnitude of $V_l$ occurred throughout the world, which can be attributed to certain events including the launching and docking of the TBM, the learning curve until the labourers are familiar with the TBM drive, mixed face conditions and long stoppages. These findings match well with observations reported by Sirivachiraporn and Phienwej (2012) for the Bangkok Blue Line (see Table 2.1) where higher volume losses than 2.0% were measured for local sections in sand or mixed face conditions (layered sand and clay) and when the tunnel excavation was launched after long stoppages. Mair (2013) also stated that tunnel drives in mixed face soil
Table 2.1 Volume loss at surface of example EPB projects.

<table>
<thead>
<tr>
<th>Tunnelling case</th>
<th>Reference</th>
<th>Ground conditions</th>
<th>Volume loss (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bangkok, Blue Line</td>
<td>Srivachiraporn and Phienwej (2012)</td>
<td>clay, sand</td>
<td>0.5 - 2.0</td>
</tr>
<tr>
<td>Kaohsiung, CR3 Taiwan</td>
<td>Hsiung (2011)</td>
<td>sand</td>
<td>0.20 - 1.27</td>
</tr>
<tr>
<td>London, CTRL(^a) Contract 220</td>
<td>Wongsaroj et al. (2005)</td>
<td>wide range(^b)</td>
<td>&lt; 0.5</td>
</tr>
<tr>
<td>London, Jubilee Line Extension</td>
<td>Dimmock and Mair (2008)</td>
<td>wide range(^c)</td>
<td>0.49 - 0.7(^d)</td>
</tr>
<tr>
<td>Milan, line 5</td>
<td>Fargnoli et al. (2013)</td>
<td>gravel, sand</td>
<td>0.27 - 0.82</td>
</tr>
<tr>
<td>Naples, line 6</td>
<td>Russo et al. (2012)</td>
<td>sand, silty sand</td>
<td>0.07 - 0.36</td>
</tr>
<tr>
<td>Singapore, North East Line</td>
<td>Shirlaw et al. (2002)</td>
<td>Old Alluvium(^e)</td>
<td>&lt;1.0</td>
</tr>
</tbody>
</table>

\(^a\)CRTL is the abbreviation of the Channel Tunnel Rail Link which is now called HS1.  
\(^b\)Geology encountered: Lambeth Group, Thanet Sands, chalk and London clay (see Mair, 2008).  
\(^c\)The tunnels were driven through sections of dense clayey silt, stiff silty clay, coarse gravel with sandy clay, dewatered dense silty sands, dewatered dense clayey sand and dense clayey sand (after Dimmock and Mair, 2008).  
\(^d\)The volume loss of 0.7% is the sum of the volume loss of both running tunnels (Dimmock and Mair, 2008).  
\(^e\)The Old Alluvium in Singapore consists of dense to very dense cemented clayey sands.

conditions are sensitive to greater volume losses. Vu et al. (2016) also showed that volume losses as high as 5.5% may occur for shallow tunnels if the face support pressure cannot be controlled.

In summary, modern tunnelling techniques generally provide a high degree of ground stability in a wide range of ground conditions with volume loss values of running tunnels typically below 1.0% (Mair, 2013). Larger volume losses encountered are the result of specific problems during the tunnel construction process. Current volume loss limits specified in tunnelling contracts account for the advancements in the tunnelling techniques. For instance, the thresholds of the allowable volume losses for the Crossrail project in London are set as 1.0% and even as low as 0.5% for certain areas (Reynolds, 2010). However, the excavation of stations usually requires several platform, passenger tunnels and caverns of varying geometry and thus alternative tunnelling methods (e.g. SCL) are usually applied which can result in larger volume losses. For stations, the risk of tunnelling-induced settlements on the built environment is often mitigated through measures such as compensation grouting which can significantly add to the project costs.
2.1 Ground deformations caused by tunnelling

2.1.1 Empirical framework

This section discusses both surface and subsurface greenfield ground movements. A widely accepted simple empirical relationship is introduced which is often applied to predict ground deformations caused by tunnelling. The emphasis is placed on transversal settlements. A variety of analytical solutions have been proposed to estimate tunnelling-induced soil displacements and are described elsewhere (Franza and Marshall, 2015; Marshall, 2009; Pinto and Whittle, 2013).

2.1.1.1 Transverse behaviour

Figure 2.2 illustrates a tunnelling-induced transverse surface settlement trough, which can be reasonably described by means of an inverted normal Gaussian distribution curve (Peck, 1969; Schmidt, 1969). This empirical approach is widely accepted in practice for various soil properties, groundwater conditions, excavation techniques, tunnel depths and tunnel diameters (Viggiani and Soccodato, 2004).

The vertical displacements $S_v(x)$ at a horizontal offset $x$ from the tunnel centreline can be expressed by:

$$S_v(x) = S_{v,max}e^{-\frac{x^2}{2i^2}}$$

(2.2)

where $S_{v,max}$ is the maximum vertical deformation measured above the tunnel centreline and $i$ the horizontal distance from the tunnel axis to the point of inflection (Figure 2.2). Equation 2.2 indicates that the greenfield settlement profile transverse to the tunnel heading is fully confined by two parameters $S_{v,max}$ and $i$, defining the magnitude and the shape of the settlement profile.
The inflection point $i$ defines the maximum slope of the settlement trough and divides the settlement trough into the hogging ($|x| > i$) and sagging ($|x| < i$) zone.

A widely accepted expression to define $i$ was given by O’Reilly and New (1982) and Mair et al. (1993) and can be written as:

$$i = K(z_t - z)$$  \hspace{1cm} (2.3)

where $K$ is the trough width parameter and $z_t$ is the tunnel depth. At surface level $z$ is equal to zero and Equation 2.3 simplifies to $i = Kz_t$. Field data showed that for tunnels in sand $K$ ranges between 0.25 and 0.45 and for tunnels in undrained clay $K$ ranges between 0.4 to 0.6 (O’Reilly and New, 1982; Mair and Taylor, 1997). Consistent with this literature, Fargnoli et al. (2013) illustrated that $K$ values between 0.25 to 0.45 provide an adequate fit to monitored settlement data for EPB tunnelling in coarse-grained soils. Rankin (1988) also stated that for soft silty clays and soft clays $K$ can be as high as 0.7. Recently reported average $K$ values of 0.6 for the construction of the Naples Underground Line 6, which was excavated in sand and silty sand, however, highlight the difficulty of estimating the trough width parameter (Russo et al., 2012). Therefore, Viggiani and Soccedato (2004) and Devriendt (2010) suggested to estimate this parameter based on experience from tunnelling projects in similar ground conditions and excavation methods as well as correlation with data obtained from case histories. Recent research shed further light on the estimation of $K$, which is discussed in Section 2.1.1.2.

The volume of the surface settlement trough per unit length can be estimated by:

$$V_{l,s} = \int_{-\infty}^{\infty} S_v(x)dx = \sqrt{2\pi}iS_{v,max}.$$  \hspace{1cm} (2.4)

After combining Equations 2.2 and 2.4, the transverse greenfield settlement trough can be written in terms of volume loss:

$$S_v(x) = \frac{\sqrt{\pi}V_{l,s}D_t^2}{2}e^{-\frac{x^2}{2i}}.$$  \hspace{1cm} (2.5)

For a preliminary assessment, it has become relatively common to carry out greenfield settlement analyses based on the expressions above using reasonable thresholds of volume loss (Reynolds, 2010).

Figure 2.3 provides the horizontal displacements of transverse settlements in greenfield conditions. For clays, O’Reilly and New (1982) provided a relationship to calculate the horizontal displacements using the equations of the empirical approach discussed above. Equation 2.6 is based on assumptions that the soil displacement vectors point directly towards the cen-
2.1 Ground deformations caused by tunnelling

![Diagram of ground deformation caused by tunnelling](image)

Fig. 2.3 Horizontal surface displacements, horizontal strain and vertical surface displacements (after Franzius, 2003).

tre of the tunnel axis (i.e., ‘point-sink’) and that the trough width parameter, $K$, stays constant with depth (Grant and Taylor, 2000).

$$S_h(x, z) = \frac{x}{z_t} S_v(x, z)$$

(2.6)

where $z$ indicates that the horizontal displacements $S_h(x, z)$ can be calculated at surface and subsurface level. Further widely recognised approaches of estimating the vector orientation of the tunnelling induced soil displacements were provide by Taylor (1995b), Attewell and Yeates (1984) and New and Bowers (1994). Specifically, the so-called ‘ribbon sink’ approach according to New and Bowers (1994) is often adopted for tunnels in clay, and assumes that the displacement vectors point towards a horizontal ribbon located at the tunnel invert. For sand, Farrell (2010) and Marshall et al. (2012) demonstrated that the displacement vectors are not pointing towards a distinct focal point and that the vector direction changes with the cover-to-diameter ($C/D_t$) ratio, $V_i$ and $x$. Consequently, an accurate description of the horizontal displacements based on the vector direction is still missing for tunnelling in sand.

The horizontal strain profile perpendicular to the tunnel heading is also illustrated in Figure 2.3 and can be estimated by combining Equations 2.2 and 2.6 and subsequent differentiation with respect to $x$. The obtained result can be expressed as:

$$\varepsilon_h = \frac{S_{v,max}}{z_t - z} e^{-\frac{x^2}{2\beta^2}} \left(1 - \frac{x^2}{\beta^2}\right)$$

(2.7)
Literature review

Figure 2.3 provides the transverse, horizontal strain profile above a tunnel in greenfield conditions and indicates a compression, $\varepsilon_{hc}$, and tension, $\varepsilon_{ht}$, zone. Compressive strains are related to the sagging zone while tensile strains are predominant in the hogging region.

2.1.1.2 Trough width

The inflection point $i$ and the related trough width parameter $K$ (Equation 2.3) can be used to describe the settlement trough width. As a rule of thumb, Rankin (1988) suggested that the trough width for tunnels in clay can be approximated by about three times the depth of the tunnel. Typically, $K$ was found by fitting Gaussian curves to field data and it has been found that $K$ varies due to various factors provided below.

Change of trough width with tunnel diameter

In contrast to the widely recognised Equation 2.3, Clough and Schmidt (1981) related $K$ to the tunnel diameter which can be written as:

$$\frac{i}{a} = \left(\frac{z_t}{2a}\right)^{0.8}$$

where $a$ is the radius of the tunnel. Moh et al. (1996) further expanded this work as discussed below.

Change of trough width with depth

Mair et al. (1993) identified that Gaussian curves can be used to approximate subsurface settlements and that the width of settlement profiles is underestimated at increasing depth when applying Equation 2.3. As a result, Mair et al. (1993) proposed a new relationship that implies a change of $K$ with $z$ which is expressed as:

$$K = \frac{0.175 + 0.325 \left(1 - \frac{z}{z_t}\right)}{1 - \frac{z}{z_t}}$$

(2.9)

This equation is the result of field data of surface and subsurface settlement profiles of tunnels in London clay and centrifuge tests of tunnels in soft clay, and considers that the change of $i$ with depth is linear with at a slope of $\partial i/\partial z = -0.325$, as illustrated in Figure 2.4. Grant and Taylor (2000) carried out centrifuge tests on tunnels in clay which showed good agreement with Equation 2.9. Combining Equations 2.3 and 2.9 provides a relationship in which the inflection point $i$ only depends on $z$ and $z_t$. 

16
2.1 Ground deformations caused by tunnelling

Fig. 2.4 Surface and subsurface transverse greenfield settlement troughs for tunnels in sand (after Marshall et al., 2012).

However, based on data of EPB tunnelling in silty sands and silty clays, Moh et al. (1996) showed that Equation 2.3 leads to much steeper and narrower subsurface troughs than observed in practice. Moh et al. (1996) argues that the trough width depends on the tunnel diameter and changes with soil depth and thus expanded Equation 2.8 to a relationship which additionally considers a change of the trough width with depth:

\[ i = \left( \frac{D_t}{2} \right) \left( \frac{z_t}{D_t} \right)^{0.8} \left( \frac{z_t - z}{z_t} \right)^m \]  

(2.10)

where \( m \) is the so-called 'subsurface trough width parameter' and \( D_t \) the tunnel diameter which shows that this equation also accounts for the tunnel size. Moh et al. (1996) proposed to use \( m = 0.4 \) for tunnels in silty sands. For tunnels in silty clays, \( m = 0.8 \) is recommended which results in the same expression as Equation 2.8. The suggested change of \( m \) with soil type indicates that the trough width is a function of the ground conditions.

Change of trough width with volume loss

Jacobsz et al. (2004), Vorster (2005), Marshall (2009) and Farrell (2010) carried out centrifuge experiments in dry sand and showed that the trough width decreases as the tunnel volume increases. This narrowing of the settlement trough can be explained by a mechanisms which is similar to a ‘chimney’ or ‘silo’ type failure above a tunnel (Farrell, 2010). Hergarden et al.
(1996) investigated the behaviour of mixed face tunnelling (sand overlain by clay) and also reported a decrease of the settlement trough with the magnitude of tunnel volume loss.

By contrast, Grant and Taylor (2000) conducted centrifuge experiments of tunnels in clay and found that the trough width is independent of the magnitude of tunnel volume loss. The authors reported that the shape and width of the settlement trough does not alter over a range of volume losses of 2% to 20%. This finding emphasises the different behaviour of clay and sand during the tunnel construction.

This research focuses on tunnels in sand which will be addressed next.

### 2.1.2 Greenfield soil movements of tunnels in sand

Differences between tunnelling in sand and clay have been raised by various researchers (e.g. Mair and Taylor, 1997, Marshall et al. (2012), Celestino et al. (2000)). The reason for this is the different nature of these two types of soil. For tunnels in clay, undrained behaviour with constant volume conditions is predominant. This behaviour of clay results in a reasonable agreement between observed settlement troughs and predictions using the Gaussian distribution as shown by several authors (e.g. Grant and Taylor, 2000). By contrast, tunnelling scenarios in drained soils, such as gravels and sands, show volume changes as a consequence of dilation/contraction effects of granular materials (Hansmire and Cording, 1985; Marshall et al., 2012). As a result, the Gaussian curve has not always provided a reasonable fit to settlement data of tunnels in sand (e.g. O’Reilly and New, 1982; Celestino et al., 2000; Vorster et al., 2005; Marshall et al., 2012; Zhou et al., 2014).

#### 2.1.2.1 Alternative curves

Following Marshall et al. (2012), the alternative curves can be classified into two and three degree of freedom curves. Table 2.2 provides these additional relationships and also recalls the Gaussian curve. Jacobsz et al. (2004) proposed a two degree of freedom modified settlement trough equation based on settlement data from a number of centrifuge experiments (Table 2.2) while Celestino et al. (2000) suggested yield density curves with an additional degree of freedom (Table 2.2). Vorster et al. (2005) proposed a modified Gaussian curve with the so-called ’shape function parameter’ $n$, which is defined by the shape parameter $\alpha$ (Table 2.2). Figure 2.5 illustrates the modified Gaussian curve and shows that $n$ controls the trough width whereas $i$ restrains the position of the inflection point (Vorster et al., 2005). Note that if $n = 1$ the modified Gaussian curve is equal to the Gaussian curve. Figure 2.5 demonstrates that the modified Gaussian curve notably affects the shape of the shoulders of the settlement profile.
Table 2.2 Curves to characterise settlement profiles above tunnels (adopted after Marshall et al., 2012).

<table>
<thead>
<tr>
<th>Reference (notation, symbol)</th>
<th>Degree of Freedom</th>
<th>Equation</th>
<th>Features</th>
</tr>
</thead>
<tbody>
<tr>
<td>Peck (1969) (Gaussian, G)</td>
<td>2 ((S_v,_{\text{max}}, i))</td>
<td>(S_v(x) = S_{v,\text{max}} e^{\frac{x^2}{2i^2}})</td>
<td>(S_v(i) = 0.606S_{v,\text{max}})</td>
</tr>
<tr>
<td>Celestino et al. (2000) (yield density, YD)</td>
<td>3 ((S_v,_{\text{max}}, a, b))</td>
<td>(S_v(x) = \frac{S_{v,\text{max}}}{1 + \left(\frac{x}{B}\right)^b})</td>
<td>(i = aB, B = (\frac{b-1}{b+1})^{1/b})</td>
</tr>
<tr>
<td>Jacobsz et al. (2004) (Jacobsz, J)</td>
<td>2 ((S_v,_{\text{max}}, i))</td>
<td>(S_v(x) = S_{v,\text{max}} e^{-1/3\left(\frac{x}{i}\right)^{1.5}})</td>
<td>(S_v(i) = 0.717S_{v,\text{max}})</td>
</tr>
<tr>
<td>Vorster et al. (2005) (modified Gaussian, mG)</td>
<td>3 ((S_v,_{\text{max}}, i, n))</td>
<td>(S_v(x) = \frac{nS_{v,\text{max}}}{(n-1)^{1/\alpha}} e^{\frac{a}{x^{2\alpha}}})</td>
<td>(n = 1 + \frac{e^{\alpha(2\alpha-1)}}{2\alpha+1})</td>
</tr>
</tbody>
</table>

![Modified Gaussian curve](image)

**Fig. 2.5** Modified Gaussian curve (after Vorster et al., 2005).

while keeping the section above the tunnel almost constant. This additional flexibility is an advantage when chimney-like mechanisms, which result in narrower settlement troughs, are expected.

Marshall et al. (2012) evaluated the curves presented in Table 2.2 with surface and subsurface field data (reported by Dyer et al., 1996) and data of centrifuge experiments modelling tunnels in a dry, dense sand.

This previous work demonstrated that a Gaussian curve provides a reasonable fit for deep tunnels and low volume loss. However, with increasing volume loss and decreasing tunnel depth the fit becomes increasingly poor. Therefore, Marshall et al. (2012) concluded that greenfield settlements of tunnels driven in sand may not always be adequately characterised.
by a Gaussian curve. Three degree of freedom curves such as the relationships suggested by Celestino et al. (2000) and Vorster et al. (2005) result in a more flexible shape of the settlement trough, hence providing a better match to the variable nature of settlement profiles for tunnels in sand. However, the higher degree of freedom curves add an additional level of complexity to the description of settlement profiles (Marshall et al., 2012).

Although this comprehensive work of Marshall et al. (2012) investigates the characteristics of settlement troughs of tunnels in sand considering data of a case history and centrifuge experiments, this research suffers from the fact that a limited amount of field data is considered. Another drawback is that the volume loss of the considered field data ($V_{l,s}$ ranges between 17.9% to 21.9%) is significantly higher than recently reported values (Table 2.1). These unrepresentative high $V_{l,s}$ values imply an advanced chimney-like mechanism that characterises the settlement profile.

2.1.2.2 Key parameters affecting ground movements of tunnels in sand

Using data of centrifuge experiments of tunnels in dry dense sand, Marshall et al. (2012) studied three key parameters influencing the settlements above tunnels in sand: (i) relative depth ($z_t - z$), (ii) tunnel size in the form of the cover-to-diameter ratio ($C/D_t$) and (iii) the tunnel volume loss ($V_{l,t}$). Following this previous research, Zhou et al. (2014) and Franza (2017) studied the effect of changing soil relative density ($I_D$) on greenfield settlements. The main findings of their research can be summarised as follows:

- The width of settlement troughs above tunnels in sand decreases with depth from the soil surface ($z$), a decrease of the cover-to-diameter ratio ($C/D_t$) and increasing tunnel volume losses ($V_{l,t}$).

- A general trend of narrowing of the settlement trough with higher $V_{l,t}$ was observed which can be described with a chimney-type mechanism.

- A decrease of the soil relative density tends to increase the magnitude of the settlements (Franza, 2017; Zhou et al., 2014). A decrease in $I_D$ tends to widen the settlement trough of deep tunnels while the trough width of shallow tunnels was less affected by $I_D$ (Franza, 2017).

- The ratio between the tunnel volume loss ($V_{l,t}$) and the soil volume loss ($V_{l,s}$) depends on the volumetric behaviour of granular soils during shearing.

These findings can be explained by the complex volumetric behaviour of drained ground surrounding a tunnel excavation which will be addressed next.
2.1 Ground deformations caused by tunnelling

2.1.2.3 Volumetric behaviour of granular material around tunnels

Tunnel excavation results in ground movement, and shearing of the soil surrounding a tunnel can be observed. These shear strains affect the volumetric behaviour of granular soils through contraction/dilation effects (Hansmire and Cording, 1985; Marshall et al., 2012). Volumetric change of soils depends on many variables such as the soil composition, the relative density and the confining stress. Figure 2.6 shows that the magnitude of shear strains developing around a tunnel is affected by the amount of tunnel volume loss and the position relative to the tunnel. From Figure 2.7 it can be seen that dense soil tends to contract at low shear strains, $\gamma$, while dilation effects can be observed as $\gamma$ develops. Compressive volumetric strains ($\varepsilon_{vol} < 0$ in Figure 2.7) result in soil volume losses ($V_{l,s}$) greater than the tunnel volume loss ($V_{l,t}$). This mechanism is valid for deep, small tunnels with high cover-to-diameter ratios (Figure 2.7) or for tunnels in loose sand. By contrast, for larger and shallower tunnels in dense sand, higher shear strains can be observed and the dense soil tends to dilate. Because of this dilative behaviour, $V_{l,s}$ can be less than $V_{l,t}$. These mechanisms highlight that for tunnels in sand the volumetric behaviour under shear, which is a function of the soil relative density, the tunnel size and depth have a direct impact on $V_{l,s}$ and $V_{l,t}$.

The findings of Marshall et al. (2012), Zhou et al. (2014) and Franza (2017) show that the development of shear strains is a function of the amount of tunnel volume loss, the tunnel size, the position within the affected soil body and intrinsic soil properties such as the relative density. This complex interaction between the tunnelling process and the behaviour of the surrounding soil leads to uncertainties when predicting the magnitude and shape of soil

Fig. 2.6 Relationship between tunnel volume loss and shear strain (after Marshall et al., 2012).
Contraction/dilation effects affecting the behaviour of ground surrounding tunnels (after Marshall et al., 2012; soil data from Zhao, 2008).

Deformations above a tunnel in drained soils. New empirical relationships were proposed by Marshall et al. (2012) which account for the relation between the shape and magnitude of settlement troughs of tunnels in sand and the tunnel size, depth and magnitude of volume loss which are outlined below.

2.1.2.4 Empirical approach to predict greenfield settlements of tunnels in sand

Based on data of centrifuge model tests, Vorster et al. (2005) and Marshall et al. (2012) suggested a set of new relationships, which consider that the shape and magnitude of greenfield settlement profiles of tunnels in sand are a function of the tunnel size, depth and volume loss. First, equation 2.9 was given in a more general form:

\[
K = K_s + \left( \frac{\partial i}{\partial z} \right) \left( \frac{z}{z_i} \right) \left( 1 - \frac{z}{z_i} \right) \tag{2.11}
\]

where \(K_s\) is the value of the trough width parameter \(K\) at the surface and \(\frac{\partial i}{\partial z}\) is the slope of the inflection point \(i\) with depth (Figure 2.4). Subsequently, the parameters \(x^*, x^{**}\) and \(K_s^*\) were introduced which are the horizontal offset from the tunnel centreline to the point on a fitted modified Gaussian curve where \(S_v = 0.606S_{v,max}\) (comparable with the inflection point \(i\).
2.1 Ground deformations caused by tunnelling

of the Gaussian curve), the horizontal offset from the tunnel centreline to the point on a fitted modified Gaussian curve where $S_v = 0.303S_{v,\text{max}}$ and the trough width parameter for surface displacements based on the parameter $x^*$, respectively. The parameters $x^*$ and $x^{**}$ vary for the curves presented in Table 2.2, though the curves have the same inflection point which is an advantage when qualitatively evaluating the goodness of the fit of individual curves. The parameter $K^*$ may be derived by applying

$$K^* = K^{**} + \left( \frac{\partial x^*}{\partial z} \right) \left( \frac{z}{z_i} \right) \frac{C_D}{C} t$$

(2.12)

$$K^{**} = K^{**}\text{int} + K^{**}\text{slope} \left( \frac{C}{D_t} \right) + K^{**}\text{slope} V_{l,t}$$

(2.13)

where the parameters $K^{**}\text{int} = 0.440$, $K^{**}\text{slope} = 0.055$, $K^{**}\text{slope} = -0.041$ and $\left( \frac{\partial x^*}{\partial z} \right) = -0.44$ which was specified using the data from the centrifuge experiments discussed in Marshall et al. (2012). Clearly, this approach is more complex than the previous empirical relationships to derive $K$, and as previously discussed, based on a limited amount of data and a single type of soil with a constant relative density of about 90%. More recent work by Zhou et al. (2014) and Franza (2017) reported that the relative density significantly changes the magnitude and shape of greenfield settlement troughs. For this reason, Franza (2017) suggested a new empirical approach that also accounts for $I_D$. These empirical frameworks better capture the influence of the tunnel depth, size and volume loss on the predicted greenfield settlement profiles of tunnels in dense sand, which are affected by the highly non-linear behaviour of granular materials.

2.1.2.5 Empirical framework to estimate the volume loss at soil surface

To describe the relation between $V_{l,s}$ and $V_{l,t}$ (Figure 2.4), Marshall et al. (2012) suggested the following empirical formulation for tunnels in dry, dense sand with $I_D = 90\%$:

$$V_{l,s} = \left( \frac{C}{D_t} \right)^\beta \left[ c_a + c_b \exp\left( -(V_{l,t} - \frac{c_c}{c_d})^2 \right) \right]$$

(2.14)

where $\beta = 0.5$, $c_a = 2.0$, $c_b = -3.7$, $c_c = -2.8$ and $c_d = 3.6$. Zhou (2015) accounted for alterations of $I_D$ by relating $\beta$ to $I_D$, which can be written as:

$$\beta = 1.75 - 1.5I_D$$

(2.15)
Literature review

Recently, Franza (2017) extended this framework by making use of a wider dataset with further variations of $C/D_t$ and $I_D$.

2.1.3 Summary

This section covered experimental methods of describing greenfield ground movements caused by tunnel construction. After introducing the sources of volume loss around a tunnel, a widely accepted empirical framework of characterising settlement profiles above tunnels was discussed. Finally, recent research investigating the complex behaviour of granular materials affected by a tunnelling excavation was presented. From this section the following main conclusions can be drawn:

- For practical reasons, greenfield settlements caused by tunnel construction are generally described by means of a Gaussian curve. This simple empirical approach is widely accepted and thus used for infrastructure projects throughout the world. However, a Gaussian curve does not always adequately characterise settlement profiles of tunnels in sand.

- For tunnels in sand, it was observed that the Gaussian curve reasonably describes settlement profiles at low volume loss up to about 1%. This is in particular true for near surface settlement profiles where Gaussian curves perform similar to more complex curves such as the modified Gaussian curve. However, as volume loss increases a narrowing of the settlement trough (chimney-like mechanism) was observed and the modified Gaussian curve performs significantly better by providing extra flexibility to describe the trough shape.

- Tunnelling scenarios in sandy ground are characterised by complex volumetric changes of the soil surrounding a tunnel. The resulting ground deformations are a function of the tunnel depth, the tunnel size, the magnitude of volume loss and the soil relative density. Based on this finding, an alternative empirical relationships was suggested by Marshall et al. (2012) which considers these parameters when determining the trough width. Although this expression is considerably more complex and based on limited data, it provides an adequate fit to settlement data of tunnels in sands, particularly as volume loss develops and chimney mechanisms evolve.
2.2 Response of buildings to tunnelling-induced settlements

This section focuses on the response of buildings to tunnelling-induced ground deformations. Building damage is generally a function of differential foundation movement (BRE, 1995) which causes stresses and strains within the building. Damage of buildings is often visible as cracks, jamming of doors, broken windows, damaged connections between adjacent buildings or fractured service pipes. To quantify the movement and distortion of a building, deformation parameters are commonly used. After introducing these parameters, systems to classify damage of buildings are discussed. Then, a widely recognised three-stage framework to assess the risk of buildings to damage caused by tunnelling subsidence will be presented. Finally, damage assessment methods which account for soil–structure interaction effects will be discussed.

2.2.1 Deformation parameters

Widely adopted parameters to describe ground and building distortions according to Burland and Wroth (1974) are shown in Figure 2.8 and defined below:

- **Settlement**, $S_v$, is the vertical movement of a point of a structure (Figure 2.8a). Downward movements are assigned positive values.
• **Relative Settlement**, $\delta S_v$, describes the differential settlement between two points of a structure (Figure 2.8a).

• **Rotation**, $s$, gives the change in gradient of a line joining two points on the structure (i.e. point A and B in Figure 2.8a). In different literature and within this thesis the rotation is often termed slope.

• **Angular strain**, $\alpha$, shown in Figure 2.8a, is given by $\delta S_{v,BA}/L_{AB} + \delta S_{v,BC}/L_{BC}$. It is defined to be positive for sagging mode deformations (upward concavity) and negative for hogging mode deformations (downward concavity).

• **Relative Deflection**, $\Delta$, is defined by the maximum settlements relative to a straight line connecting two points on a structure (i.e. A and D in Figure 2.8b).

• **Deflection Ratio**, $\Delta/L$, is expressed by the ratio of the relative deflection and the corresponding distance of the two points used to calculate the relative deflection (i.e. $L_{AD}$ in Figure 2.8b). The deflection ratio is often related to bending distortions and is usually abbreviated with $DR$.

• **Tilt**, $\theta$, describes the rigid body rotation of a structure. The tilt can be calculated by dividing the relative settlements between the edges of a structure with the horizontal length of the structure (Figure 2.8c).

• **Angular Distortion**, $\beta$, refers the rotation of the line joining two points with respect to the tilt (Figure 2.8c). The angular distortion can also be calculated by subtracting the tilt from the slope ($s - \theta$).

• **Horizontal Displacement**, $S_h$, describes the magnitude of the horizontal movement of points on the structure.

• **Horizontal Strain**, $\varepsilon_h$, is termed the quotient of the differential horizontal displacement between two points on the structure and the horizontal distance between these two points.

Different schools of thought exist about which parameters should best be used to assess potential damage of buildings (Burland et al., 2004). This will be demonstrated by presenting widely accepted damage criteria, which limit these deformation parameters to certain thresholds in order to avoid building damage.
2.2 Response of buildings to tunnelling-induced settlements

2.2.2 Damage criteria neglecting soil–structure interaction

Damage criteria to assess potential damage of buildings caused by ground deformation can be subdivided into empirical and semi-empirical relationships. Before discussing these criteria it is important to classify building damage. Skempton and MacDonald (1956) subdivided building damage into three categories: (i) architectural damage (affects the appearance of the property), (ii) functional damage (affects the usability or serviceability of the structural asset) and (iii) structural damage (affects the stability of the building). Nowadays, this general approach is still in use. For instance, the BRE (1995) classifies damage in three broad categories, ‘aesthetic’, ‘serviceability’ and ‘stability’, which shows the similarities to the work of Skempton and MacDonald (1956).

Burland et al. (1977) introduced a classification of damage which is based on the ease of repair and crack width, as shown in Table 2.3. This classification system is generally related to plasterwork and brickwork masonry and is in a slightly modified form recommended by the BRE (1995) to assess damage in low rise building. The damage categories proposed by Burland et al. (1977) can also be related to the general damage categories discussed above. Damage categories 0 to 2 represent ‘aesthetic’ damage, categories 3 and 4 ‘serviceability’ damage and category 5 ‘stability’ damage.

Mair et al. (1996) pointed out that the transition between damage category 2 and 3 is an important threshold. Damages below this threshold can either be related to causes which are linked to the structure, such as, thermal movements, material shrinkage, poor workmanship and design (BRE, 1995) or minor ground movements. However, damage exceeding category 2 is frequently caused by severe ground movements and thus of importance when dealing with tunnelling-induced building damage.

2.2.2.1 Empirical damage criteria

Based on field observations, various authors related visible damage to building deformation parameters. These damage criteria were mainly related to deformation of buildings due to self-weight movements (e.g. Skempton and MacDonald, 1956; Polshin and Tokar, 1957; Bjerrum, 1963; Burland and Wroth, 1974).

Skempton and MacDonald (1956) proposed a damage criterion which is related to the angular distortion (Table 2.4). Within this study Skempton and MacDonald (1956) monitored a total of 98 buildings and 40 of them showed damage. Following Boone (2008), a criticism of this damage criteria is that building features such as the length or height of structures or ground conditions are not taken into account.
Table 2.3 Classification of visible damage after Burland et al. (1977).

<table>
<thead>
<tr>
<th>Category of damage</th>
<th>Normal degree of severity</th>
<th>Description of typical damage (Ease of repair in italic)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Negligible</td>
<td>Hairline cracks less than about 0.1 mm</td>
</tr>
<tr>
<td>1</td>
<td>Very Slight</td>
<td>Fine cracks which are easily treated during normal decoration. Damage generally restricted to internal wall finishes. Close inspection may reveal some cracks in external brickworks or masonry. Typical crack widths up to 1 mm.</td>
</tr>
<tr>
<td>2</td>
<td>Slight</td>
<td>Cracks easily filled. Re-decoration probably required. Recurrent cracks can be masked by suitable linings. Cracks may be visible externally and some repointing may be required to ensure weathertightness. Doors and windows may stick slightly. Typical crack width up to 5 mm.</td>
</tr>
<tr>
<td>3</td>
<td>Moderate</td>
<td>The cracks require some opening up and can bepatched by mason. Repointing of external brickwork and possibly a small amount of brickwork to be replaced. Doors and windows sticking. Service pipes may fracture. Weathertightness often impaired. Typical crack widths are 5 to 15 mm or several up to 3 mm.</td>
</tr>
<tr>
<td>4</td>
<td>Severe</td>
<td>Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows. Windows and door frames distorted, floor sloping noticeably. Walls leaning or bulging noticeably, some loss of bearing in beams. Service pipes disrupted. Typical crack widths are 15 to 25 mm but also depends on the number of cracks.</td>
</tr>
<tr>
<td>5</td>
<td>Very severe</td>
<td>This requires a major repair job involving partial or complete rebuilding. Beams lose bearing, walls lean badly and require shoring. Windows broken with distortion. Danger of instability. Typical crack widths are greater than 25 mm but depends on the number of cracks.</td>
</tr>
</tbody>
</table>

Note: Crack width is only one factor in assessing category of damage and should not be used on its own as a direct measure of it.

Note: Local deviation of slope, from the horizontal or vertical, of more than 1/100 will normally be clearly visible. Overall deviations in excess of 18/150 are undesirable.

Table 2.4 Angular distortion limits by Skempton and MacDonald (1956).

<table>
<thead>
<tr>
<th>Damage description</th>
<th>Limiting angular distortion (β)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cracking in walls and partitions</td>
<td>1/300</td>
</tr>
<tr>
<td>Cracking of particularly sensitive brick structures</td>
<td>1/1000</td>
</tr>
<tr>
<td>Structural damage of façade</td>
<td>1/150</td>
</tr>
</tbody>
</table>

28
### 2.2 Response of buildings to tunnelling-induced settlements

Polshin and Tokar (1957) related self-weight building damage to the deflection ratio (Table 2.5). This previous work took different ground conditions and ratios of building length and height \((L/H)\) into account. Additionally, Polshin and Tokar (1957) provided maximum allowable slope values for steel and concrete frame infilled structures of \(1/500\) and about \(1/200\) for structures without infill or infill not susceptible to damage. Polshin and Tokar (1957) also related the onset of cracking to a certain tensile strain value. For masonry, this limiting tensile strain was found to be \(0.05\%\). Burland and Wroth (1974) showed that the Skempton and MacDonald (1956) and Polshin and Tokar (1957) damage criteria might be unsafe for unreinforced load-bearing masonry in hogging. Although these empirical damage criteria provide a useful framework to assess critical building damage, the deformations caused by tunnel construction are of different nature than the self-weight deformations. Thus, Attewell (1988) pointed out that tunnelling-induced settlements emerge within considerably shorter amount of time compared to deformations caused by the own-weight. For this reason, these damage criteria might underestimate the risk of damage of buildings in the vicinity of tunnel excavations.

Rankin (1988) adopted these self-weight criteria by providing a guide to predict the risk of building damage caused by underground construction which is based on critical slope and settlement values (Table 2.6). This previous study is based on a limited database of building damage due to tunnel excavation. Rankin (1988) suggested to calculate building rotation and maximum building settlement based on predictions of greenfield settlements through applying the Gaussian curve. A comparison between these calculated values and the critical deformation values (Table 2.6) classify the potential damage of a building. For buildings identified to be of damage category 3 and 4 (and sometimes category 2) a detailed assessment is necessary which has to consider the ground conditions, the building load and building characteristics (Rankin, 1988).

Eurocode 7 (EC7) also provides an annex (Annex H) which defines limiting structural deformation values (CEN, 2007). It is of interest that the EC7 relates structural damage to the angular distortion of a building. Following the EC7, the allowable maximum angular distort-

---

**Table 2.5 Deflection ratio limits by Polshin and Tokar (1957).**

<table>
<thead>
<tr>
<th>Building length to height</th>
<th>Limiting deflection ratio</th>
<th>Ground conditions</th>
</tr>
</thead>
<tbody>
<tr>
<td>(L/H \leq 3)</td>
<td>1/3300</td>
<td>sand</td>
</tr>
<tr>
<td></td>
<td>1/2500</td>
<td>soft clay</td>
</tr>
<tr>
<td>(L/H \geq 5)</td>
<td>1/2000</td>
<td>sand</td>
</tr>
<tr>
<td></td>
<td>1/1430</td>
<td>soft clay</td>
</tr>
</tbody>
</table>
### Literature review

#### Table 2.6 Building slope and settlement limits by Rankin (1988).

<table>
<thead>
<tr>
<th>Risk Category</th>
<th>Maximum Slope of Building</th>
<th>Maximum Settlement of Building (mm)</th>
<th>Description of Risk</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.</td>
<td>Less than 1/500</td>
<td>Less than 10</td>
<td>Negligible: superficial damage unlikely.</td>
</tr>
<tr>
<td>2.</td>
<td>1/500 to 1/200</td>
<td>10 to 50</td>
<td>Slight: Possible superficial damage which is unlikely to have structural significance.</td>
</tr>
<tr>
<td>3.</td>
<td>1/200 to 1/50</td>
<td>50 to 75</td>
<td>Moderate: Expected superficial damage and possible structural damage to buildings, possible damage to relatively rigid pipelines</td>
</tr>
<tr>
<td>4.</td>
<td>Greater than 1/50</td>
<td>Greater than 75</td>
<td>High: Expected structural damage to buildings. Expected damage to rigid pipelines, possible damage to other pipelines.</td>
</tr>
</tbody>
</table>

For open framed structures, infilled frames and load bearing or continuous brick walls are between a range of 1/2000 and 1/300 for the serviceability limit state. This subdivision of the maximum allowable angular distortion based on different building types reflects the different behaviour of, for example, load-bearing masonry and framed concrete structures when affected by ground movements. The EC7 further states that the ultimate limit state of structures is reached when the angular distortion is about 1/150, which is identical to the value suggested by Skempton and MacDonald (1956). The proposed values of the EC7 are related to the sagging mode of deformations and have to be halved if the structure is situated within the hogging zone. This additional recommendation accounts for the fact that buildings are generally more vulnerable to tension. The EC7 does not provide guidelines for ground movements caused by underground construction which, as mentioned above, emerge much faster than self-weight ground movements. Hence, the application of the EC7, which is more in line with the damage criteria of Skempton and MacDonald (1956) and Polshin and Tokar (1957), is considerably limited when assessing potential building damage caused by tunnelling.

#### 2.2.2.2 Semi-empirical damage criteria

Semi-empirical damage criteria are based on field observations, deep beam theory or the theory about the state of strain within building sections. Before introducing widely used concepts of estimating strains caused by ground movements, the relation between the onset of building damage in the form of cracking and tensile strains is discussed.
2.2 Response of buildings to tunnelling-induced settlements

Table 2.7 Limiting tensile strain values linked to building damage (after Boscardin and Cording, 1989).

<table>
<thead>
<tr>
<th>Category of damage</th>
<th>Normal degree of severity</th>
<th>Limiting Tensile strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Negligible</td>
<td>0 - 0.05</td>
</tr>
<tr>
<td>1</td>
<td>Very slight</td>
<td>0.05 - 0.075</td>
</tr>
<tr>
<td>2</td>
<td>Slight</td>
<td>0.075 - 0.15</td>
</tr>
<tr>
<td>3</td>
<td>Moderate^a</td>
<td>0.15 - 0.3</td>
</tr>
<tr>
<td>4 to 5</td>
<td>Severe to Very Severe</td>
<td>&gt; 0.3</td>
</tr>
</tbody>
</table>

^Note following Mair et al. (1996): Boscardin and Cording (1989) describe the damage corresponding to $\varepsilon_{lim}$ in the range 0.15 - 0.3% as 'moderate to severe'. However, none of the cases quoted by them exhibit severe damage for this range of strains. There is therefore no evidence to suggest that tensile strains up to 0.3% will result in severe damage.

Critical and limiting tensile strain

By drawing on the concept of Polshin and Tokar (1957) that tensile strain is a key parameter when assessing the damage of building damage, Burland and Wroth (1974) introduced the term 'critical tensile strain' ($\varepsilon_{crit}$), which is the magnitude of tensile strain when cracking becomes visible. The critical tensile strain describes the onset of visible cracking and thus might not result in any serviceability limits of the building (Burland and Wroth, 1974). Making use of data from mechanical analysis of different building materials, Burland and Wroth (1974) defined the following critical tensile strain ranges:

- Brickwork and blockwork: $\varepsilon_{crit} = 0.05 - 0.1\%$
- Reinforced concrete beams: $\varepsilon_{crit} = 0.03 - 0.05\%$

Burland et al. (1977) pushed the concept of the critical tensile strain a step further by introducing the 'limiting tensile strain' ($\varepsilon_{lim}$). The advantage of this parameter is that it can be related to various serviceability limit states (Burland et al., 2004). Thus, the limiting tensile strain is also called a 'serviceability parameter'. This view is supported by Boscardin and Cording (1989) who extended Table 2.3 through linking limiting tensile strain values to each damage category as shown in Table 2.7. This damage classification is notable because quantitative building deformation ($\varepsilon_{lim}$) is related to severity of building damage and hence provides an important framework when assessing potential damage of buildings due to tunnelling-induced settlements.
Literature review

Limiting tensile strain method

After limiting tensile strain values were linked to damage categories (Table 2.7), a semi-empirical framework to calculate building strains will be discussed next. This methodology is frequently called the limiting tensile strain method (LTSM).

Building strains due to vertical displacements: Burland and Wroth (1974) showed that the onset of cracking of simple beams of various $E/G$ values in hogging and sagging agrees well with case records of buildings. This finding resulted in a method that simplified a building façade as a simple rectangular, weightless beam of length $L$ and height $H$. Figure 2.9 illustrates this approach and also shows individual bending and shearing deformation modes. By assuming that a central point load acts on this beam and that the beam performs under isotropic linear-elastic conditions, relationships between the deflection ratio $DR$ and the maximum direct bending strains ($\varepsilon_{b,\text{max}}$) and maximum diagonal strains ($\varepsilon_{d,\text{max}}$) were derived. The derivation of these relationships made use of the beam deflection theory of Timoshenko (1957). Following Burland et al. (1977), who slightly refined the initial relationships to a more general form, the relationships for a central point load can be given as

$$DR = \left\{ \frac{L}{12t} + \frac{3IE}{2tLHG} \right\} \varepsilon_{b,\text{max}}$$ (2.16)

$$DR = \left\{ 1 + \frac{HL^2G}{18IE} \right\} \varepsilon_{d,\text{max}}$$ (2.17)

where $E$ is the Young’s modulus, $G$ is the shear modulus, $L$ is the length, $H$ is the height of the building, $I$ is the second moment of area of the equivalent beam and $t$ is the distance to the neutral axis.

In the sagging mode Burland and Wroth (1974) argued that the neutral axis is at $t = H/2$ while the neutral axis of the hogging mode is at the lower edge of the beam ($t = H$). This recommendation is based on observations of building performance and experiments of model brick walls. For buildings in hogging, it was shown that at the top of the wall continuous cracks are free to develop and propagate downwards to the foundation. This mechanism results in a lowering of the neutral axis. However, for buildings in sagging continuous cracks were not observed due to a restraining effect of the foundation and the position of the neutral axis can be taken to remain in the middle of the building. Figure 2.10 compares the differences between these assumptions in the hogging and sagging mode. The generally lower $\Delta / (L\varepsilon_{\text{crit}})$ value in hogging indicates that the building is more susceptible to hogging than sagging deformations. Although this framework is widely applied in practice, it assumes that structures in hogging
2.2 Response of buildings to tunnelling-induced settlements

are cracked and therefore does not represent the true condition of an intact structure. For this reason, this recommendation can result in overly conservative damage assessments. Figure 2.10 indicates that for low $L/H$ values the diagonal tensile strain becomes critical whereas bending becomes critical after a certain $L/H$ threshold is exceeded. This $L/H$ boundary is at about 0.7 and 1.1 for the sagging and hogging modes respectively. In reality, it is likely that both types of deformation occur simultaneously (Burland and Wroth, 1974).

Another finding of the work of Burland and Wroth (1974) is that the onset of visible cracking is not sensitive to different load scenarios (e.g. uniformly distributed load). Equations 2.16 and 2.17 can now be used to estimate the maximum allowable deformations of a building (i.e. $DR$) by setting $\varepsilon_{b,\text{max}}$ and $\varepsilon_{d,\text{max}}$ equal to a tolerable limiting tensile strain $\varepsilon_{\text{lim}}$. One can follow that the maximum allowable $DR$ of a building depends on various factors including the ratio of the material parameters $E/G$, the ratio of the building dimensions $L/H$ and assumptions regarding the location of the neutral axis (Burland et al., 2004).

For masonry structures, a global $E/G$ value of 2.6 which is equal to a Poisson’s ratio of $\nu = 0.3$ is widely used (Mair et al., 1996) and case histories from load bearing masonry showed a good agreement with the obtained results (Burland and Wroth, 1974). However, Burland and Wroth (1974) also conducted studies on anisotropic beams using different $E/G$ values of 12.5 and 0.5 which describe framed buildings and load bearing masonry respectively. The fictitious change of $E/G$ is a simplified approach to account for bending and shear stiffness differences of a variety of structures. Specifically, buildings with extensive openings (i.e.

---

**Fig. 2.9** Representation of an actual building through a simple beam model (after Burland and Wroth, 1974; Farrell, 2010).
windows and doors) are characterised by a relatively low shear stiffness, which can be captured in a simplified way by an $E/G$ value that typically exceeds 2.6. Burland and Wroth (1974) showed that relatively flexible buildings in shear, with an assumed $E/G$ of 12.5, withstand greater values of $\Delta/L$ than isotropic beams with an $E/G$ ratio of 2.6. Structures with an $E/G$ lower than 2.6 (e.g. 0.5 for load bearing masonry) are even more vulnerable in terms of $\Delta/L$.

At this stage, it is worth emphasising that global structural parameters such as the $E/G$ ratio are difficult to estimate for real buildings. For instance, Melis and Rodriguez Ortiz (2001) reported $E/G$ values of 2.3 to 2.6 for masonry. However, for steel structures or flexible structures with large spans, Melis and Rodriguez Ortiz (2001) recommend to use $E/G$ values of 12 or even as high as 15. These suggestions of Melis and Rodriguez Ortiz (2001) concur with the recommendations of Mair et al. (1996) (i.e. $E/G$ of 2.6 for masonry and 12.5 for framed buildings on shallow foundations). Devriendt (2003) also addressed issues with assigning global $E/G$ ratios to three-storey load bearing brick structures on thick concrete strip footings. To overcome the uncertainty of determining $E/G$, a finite element analysis was performed by Devriendt (2003) to gain a better understanding of the behaviour of these structures in shear and bending. Based on this analysis $E/G$ values between 4.5 and 6.5 were found to be reasonable. An appropriate estimation of global $E/G$ ratios is in particular difficult when building openings are present. Numerical results of Son and Cording (2007) highlight this by showing that $E/G$ ratios of masonry façades with window openings between 10-20% are between 12 to 23. Similar findings were reported by Cook (1994) who back-calculated the response of masonry façades and stated $E/G$ values as high as 30.
2.2 Response of buildings to tunnelling-induced settlements

**Horizontal building strains:** Although the work of Burland and Wroth (1974) and Burland et al. (1977) provides an important framework to calculate strains within a building it mainly deals with vertical deformation components. However, tunnelling can result in horizontal ground movements (Section 2.1) and horizontal strains can be transferred from the soil to the building. Thus, Boscardin and Cording (1989) accounted for horizontal strains. The resultant building bending strain $\varepsilon_{br}$ can be written as

$$\varepsilon_{br} = \varepsilon_{b,\text{max}} + \varepsilon_{h}$$  \hspace{1cm} (2.18)

where $\varepsilon_{b,\text{max}}$ is the maximum bending strain obtained through Equation 2.16 and $\varepsilon_{h}$ is the horizontal strain transferred to the building which can be estimated through

$$\varepsilon_{h} = \frac{\Delta L}{L}$$  \hspace{1cm} (2.19)

where $\Delta L$ is the change of the building length. The calculation of the resultant diagonal tensile strain, $\varepsilon_{dr}$, through superimposing of the diagonal strain and the horizontal strain, $\varepsilon_{h}$, is more complex. Following Netzel (2009), the strain relationships to derive principal strains according to Timoshenko and Gere (1971) can be used to determine $\varepsilon_{dr}$ which results in

$$\varepsilon_{dr} = \frac{\varepsilon_{ht}}{2} + \sqrt{\left(\frac{\varepsilon_{ht}}{2}\right)^2 + \varepsilon_{d,\text{max}}^2}.$$  \hspace{1cm} (2.20)

To date there has been little agreement on how to consider horizontal strains when predicting tunnelling-induced building strains. Boscardin and Cording (1989) assumed that the horizontal ground strains are equally transferred to the structure. This concept has been challenged by Geddes (1991) who experimentally demonstrated that the strain transferred to the structure can be significantly different than the horizontal ground strains (Geddes, 1977). This finding is consistent with various case studies (Burland et al., 2004; Dimmock and Mair, 2008; Farrell et al., 2011; Mair, 2013; Viggiani and Standing, 2001) and centrifuge tests (Farrell and Mair, 2011). It is therefore likely that the superposition of the horizontal ground strains using the equations above substantially overestimates the transfer of horizontal ground strains to buildings on continuous foundations. By contrast, Elshafie (2008) and Goh and Mair (2011b) demonstrated that considerable horizontal strains can be transferred to structures on individual footings.

**Relevant building dimensions:** As mentioned above, the $L/H$ ratio is an important parameter influencing the estimation of the maximum strains of the building. Mair et al. (1996) suggested to partition the building into the sagging and hogging region, as depicted in Figure
Making use of the characteristics of the predicted greenfield settlement trough, the deflection ratios in hogging ($\Delta_{hog}/L_{hog}$) and sagging ($\Delta_{sag}/L_{sag}$) can be measured or predicted. The length of the building reduces to the building length in the hogging and sagging zone ($L_{hog}$ and $L_{sag}$). Further, a cut off of the building length is applied when the building length exceeds practical limits of the trough width (i.e. 2.5$i$, Mair et al., 1996). The building height stays unchanged and generally is the height of the building without taking the roof into account but considering the foundation depth (Burland et al., 2004).

Although these guidelines of partitioning the building to estimate the building strains individually for the hogging part and sagging part are widely recognised, Netzel (2005) showed that this approach can result in underestimation of the building damage. Firstly, Netzel (2005) found that the cut off of the building after a trough width of 2.5$i$ or settlements below 1mm (Netzel, 2005) can lead to considerable underestimation of tensile strains. Netzel (2009) thus argued that for buildings exceeding the cut off length the entire building length should be considered. Furthermore, Netzel (2009) found that the partitioning approach might lead to significant underprediction of building damage if the difference between the tilt of the separate building parts and the entire buildings is greater than 15%. Although this additional recommendation regarding the tilt is an improvement, tilt values exceeding 15% seem to be unrealistic.

Another limitation of this partitioning of a building at the theoretical greenfield inflection point ($i_{GF}$ in Figure 2.11) is that the interaction between the building and the soil potentially...
2.2 Response of buildings to tunnelling-induced settlements

Fig. 2.12 Damage category diagrams.

modifies the position of the inflection point. This is schematically depicted with \( i_{St} \) in Figure 2.11 and likely results in uncertain predictions of the building response.

**Interaction diagrams:** Boscardin and Cording (1989) presented simplified damage category charts, which can be used to estimate the potential damage through relating angular distortion values to maximum horizontal strain values as shown in Figure 2.12a. While the Boscardin and Cording (1989) damage criteria are based on the angular distortion, Burland (1995) proposed a similar damage chart which is based on the deflection ratio (Figure 2.12b).

**State of strain concept**

Son and Cording (2005) updated the criteria of Boscardin and Cording (1989) to a more general state, which is independent from \( L/H \), \( E/G \) and the position of the neutral axis. This concept is based on the state of strain (SoS) in a building unit, which is given by:

\[
\varepsilon_p = \varepsilon_h \cos \theta_{max}^2 + \beta \sin \theta_{max} \cos \theta_{max}
\]  

(2.21)

where \( \varepsilon_p \) is the maximum principal tensile strain, \( \theta_{max} \) the direction of the crack which is equal to the angle of the plane where \( \varepsilon_p \) acts and is measured from the vertical plane. This direction of crack formation can be calculated through:

\[
\tan(2\theta_{max}) = \frac{\beta}{\varepsilon_h}
\]  

(2.22)

where \( \beta \) is the angular distortion and \( \varepsilon_h \) the horizontal strain.
Son and Cording (2005) recommended to define building units based on intermediate walls, columns, different building properties (e.g. geometry, stiffness) or different gradients of soil displacements. These assumptions might result in uncertainties. Moreover, to accurately estimate $\beta$ is also controversial because it requires tilt measurements (Burland et al., 2004). Specifically, throughout the design stage assumptions about the magnitude of rigid body rotation are problematic.

Figure 2.13 illustrates the adjusted damage criteria of Son and Cording (2005) and indicates different values of $\theta_{\text{max}}$. When solely horizontal strains are imposed on the building, $\theta_{\text{max}}$ is zero and the resulting plane of the crack formation is vertical. In contrast, $\theta_{\text{max}}$ is 45° if the building strains are only a result of the angular distortion.

Section 8.1 provides further detail about these methods to estimate building strains caused by tunnelling-induced soil displacements. The following section provides a framework to assess potential building damage due to tunnel excavation that is often applied for major urban tunnelling projects.

### 2.2.3 Three-stage building risk assessment

Mair et al. (1996) outlined a widely accepted risk assessment approach which was, for example, applied for the Jubilee Line Extension (JLE), Channel Tunnel Rail Link (CRTL, now HS1) for Crossrail and High Speed Two (HS2) project. Using this approach, the assessment of potential building damage is undertaken in three stages (Figure 2.14).
2.2 Response of buildings to tunnelling-induced settlements

2.2.3.1 Preliminary assessment

In the preliminary assessment, a greenfield settlement prediction is carried out using the Gaussian assumption (Section 2.1.1). Contour lines of settlements are often plotted along the proposed tunnel alignment and buildings are assessed based on two defined thresholds: a maximum rotation of \( 1/500 \) (\( \theta \) in Figure 2.14) and settlements of 10 mm. Buildings experiencing less distortions than these thresholds are considered not to be at risk and thus will not be further assessed. The other buildings qualify to the second stage assessment.

This first stage simplifies the interaction problem by considering only greenfield displacements. The assessment thresholds are based on experience of previous case studies and are also equal to the lower boundaries of the second damage category ('slight') of Rankin (1988) (Table 2.6). Any soil–structure interaction is neglected. For this reason, this preliminary assessment is often extensively conservative but enables an effective screening of a large number of affected buildings.

Fig. 2.14 Damage assessment framework according to Mair et al. (1996) (after Franzius, 2003).
2.2.3.2 Second stage assessment

The second stage applies the LTSM (Section 2.2.2.2). The tensile strains induced in the building are determined by modelling the building as an elastic beam and imposing the greenfield ground deformations of stage one on the fully flexible structure. The corresponding damage category can be found by either using Table 2.7 or the corresponding damage assessment diagrams (see Figure 2.12). Buildings classified as damage category 3 or above require further assessment.

Though this second stage of the building evaluation is more detailed, the greenfield settlements are imposed on the structure and any interaction between the soil and the structure is neglected. Clearly, this is a simplification because field data (e.g. Frischmann et al., 1994, Dimmock and Mair, 2008, Farrell et al., 2011) and centrifuge tests (Farrell, 2010; Taylor and Grant, 1998; Taylor and Yip, 2001) demonstrated that surface structures modify the ground response and different structural deformations compared to the greenfield settlements have been observed. Consequently, this second stage assessment is still conservative. Because of this, Mair et al. (1996) pointed out that the damage level obtained at this second stage is a ‘possible’ level of damage and the real building damage would often be below this level.

2.2.3.3 Detailed evaluation

In stage three of this damage assessment method details of the building, the tunnel construction and the soil–structure interaction are considered. The detailed evaluation is generally very complex, time consuming and costly and is often performed in form of numerical analysis. Assets still experiencing unjustifiable degree of damage (category 3 or above) need protective measures.

This detailed evaluation frequently leads to damage categories below the second stage assessment. A significant difference between the third and second stage is that soil–structure interaction effects are usually taken into account. The next section introduces a methodology to account for the potential interaction between a building and the ground which could extend the second stage of this risk assessment framework.

2.2.4 Soil–structure interaction

So far the damage assessment methods discussed are based on imposing greenfield settlements on buildings. Although this assumption is reasonable for a first assessment, it is likely that the presence of a building in the proximity of the tunnel excavation influences the ground and building response. While the greenfield settlements can be predicted with a certain degree of
2.2 Response of buildings to tunnelling-induced settlements

Table 2.8 Example variables influencing the soil–structure interaction (after Boone, 2008).

<table>
<thead>
<tr>
<th>Ground Response</th>
<th>Building Response</th>
</tr>
</thead>
<tbody>
<tr>
<td>Face pressure</td>
<td>Age and condition of structure (critical strain)</td>
</tr>
<tr>
<td>Annular gap grouting</td>
<td>Proximity to tunnel (displacement mode, magnitude)</td>
</tr>
<tr>
<td>TBM attitude control</td>
<td>Height and length of building relative to displacement patterns</td>
</tr>
<tr>
<td>Lining deflection</td>
<td>Superstructure (bearing wall, frame, floors, connections, etc.)</td>
</tr>
<tr>
<td>Soil properties</td>
<td>Openings in buildings (bending stiffness, shear stiffness)</td>
</tr>
<tr>
<td>Groundwater conditions</td>
<td>Foundation type(s)</td>
</tr>
<tr>
<td>Consolidation response</td>
<td>Material types and finishes (brick, block, plaster, stucco, etc.)</td>
</tr>
<tr>
<td>Depth and diameter of tunnel</td>
<td>Building weight</td>
</tr>
<tr>
<td>Workmanship</td>
<td>Interface between the foundations and the soil</td>
</tr>
</tbody>
</table>

confidence, much uncertainty still exists about the mechanisms involved in the soil–structure interaction subject to tunnel construction. Following Boone (2008), some example parameters influencing this interaction problem can be subdivided into ground response parameters and building response parameters as shown in Table 2.8. A key parameter affecting the soil–structure interaction is the building stiffness (Elshafie, 2008; Franzius et al., 2006; Goh and Mair, 2011a; Potts and Addenbrooke, 1997). This finding led to the so-called relative stiffness methods (RSM).

2.2.5 Building stiffness - the relative stiffness methods

To investigate the role of the structure’s axial and bending stiffness on the ground movements due to tunnelling, Potts and Addenbrooke (1997) carried out an extensive plain-strain parametric study using the finite element method (FEM). The focus of their analysis was placed on studying the influence of the building’s width, its axial and bending stiffness, the tunnel depth and the position of the building with respect to the tunnel axis. While the soil was modelled using a non-linear elastic perfectly plastic constitutive model in order to simulate London clay, the surface structure was modelled as a weightless, linear-elastic beam positioned transverse to the tunnel heading. A rough interface between the structure and the soil was modelled which did not allow for a separation of the structure from the soil. Figure 2.15 illustrates the geometry of the plane-strain model of Potts and Addenbrooke (1997) where $B$ is the building width, $z$ the tunnel depth, $D$ the tunnel diameter and $e$ the eccentricity.

Potts and Addenbrooke (1997) considered the building stiffness when estimating tunnelling-induced ground movements in urban areas. Therefore, the authors defined rela-
Relative stiffness parameters to relate the building’s axial and bending stiffness to the soil stiffness which can be expressed as:

\[
\rho^* = \frac{EI}{E_s \left(\frac{L}{2}\right)^4} \tag{2.23}
\]

\[
\alpha^* = \frac{EA}{E_s \left(\frac{L}{2}\right)} \tag{2.24}
\]

where \(\rho^*\) and \(\alpha^*\) are the relative bending stiffness and relative axial stiffness. Note that \(B\) (Figure 2.15) was used for the building length \(L\) in the original work of Potts and Addenbrooke (1997). The building’s bending stiffness \(EI\) and axial stiffness \(EA\) are calculated per meter length parallel to the tunnel heading. According to Potts and Addenbrooke (1997), the soil stiffness \(E_s\) is the secant stiffness at 0.01% of axial strain and at half of the tunnel depth \((z_t/2)\) and obtained through triaxial compression tests on soil samples derived at this depth. Equation 2.23 is not dimensionless and results in \(m^{-1}\) units for a plane-strain situation.

Additionally, Potts and Addenbrooke (1997) proposed modification factors, \(M\), which relate building deformations to greenfield \((GF)\) deformations. The modification factors are individually defined for the hogging and sagging region and can be expressed in terms of \(DR\) and \(\varepsilon_h\) as written below:
2.2 Response of buildings to tunnelling-induced settlements

\[ M_{DR_{hog}} = \frac{DR_{hog,Str}}{DR_{hog,GF}}, \quad M_{DR_{sag}} = \frac{DR_{sag,Str}}{DR_{sag,GF}} \]  

\[ M_{\epsilon_{ht}} = \frac{\epsilon_{ht,Str}}{\epsilon_{ht,GF}}, \quad M_{\epsilon_{hc}} = \frac{\epsilon_{hc,Str}}{\epsilon_{hc,GF}} \]  

where \( M_{DR_{hog}} \) and \( M_{DR_{sag}} \) are the modification factors for \( DR \) in hogging and sagging respectively (Figure 2.11). \( M_{\epsilon_{ht}} \) describes the modification factors for \( \epsilon_{ht} \) in hogging (tension) while \( M_{\epsilon_{hc}} \) describes the modification factor for \( \epsilon_{hc} \) in sagging (compression). A modification factor close to zero represents buildings which behave very stiff while modification factors of 1 characterise buildings with a fully flexible response and displacements equal to greenfield conditions.

Franzius (2003) expanded the study of Potts and Addenbrooke (1997) by investigating additional building widths \( B \), different building weights, different interfaces between the beam and the soil and different tunnel depths. Moreover, Franzius et al. (2004) investigated influences due to the 3D tunnelling sequence. This study is summarised in Franzius et al. (2004) and Franzius et al. (2006) and led to the following modified relative building stiffness expressions:

\[ \rho_{mod}^* = \frac{EI}{E_z L^2 B} \]  

\[ \alpha_{mod}^* = \frac{EA}{E_z LB} \]  

where \( z_t \) is the tunnel depth and \( B \) the building dimension parallel to the tunnel heading. These modified relative building stiffness expressions consider the influence of the tunnel depth and the dimension of the building out of plane. Furthermore, the modified relative bending stiffness relationship \( \rho_{mod}^* \) is dimensionless and thus applicable for 2D and 3D analysis.

Making use of the relative stiffness relationships and the modification factors, a design approach was proposed by Potts and Addenbrooke (1997) and Franzius et al. (2006) to forecast building deformations. Figure 2.16 illustrates the design curves identified by Franzius et al. (2006). The design curves allow to obtain modification factors after the relative bending and axial stiffness values were determined. Subsequently, \( DR \) and \( \epsilon_h \) in hogging and sagging can be predicted using the Equations 2.25 and 2.26. These building deformation parameters are then used to perform the LTSM or to apply the damage category diagrams (Figure 2.12b).
benefit of using this design approach proposed by Potts and Addenbrooke (1997) and adjusted by Franzius et al. (2006) is that soil–structure interaction effects are taken into account and thus this approach might be used to update the second stage of the building damage assessment method (Section 2.2.3). Specifically, this methodology often results in less conservative estimates than adopting the procedures based on the greenfield distortions.

More recently, Goh and Mair (2011a) and Goh and Mair (2011c) proposed new relative stiffness expressions, which are based on findings of centrifuge experiments, field data and numerical analysis. By using a partitioning approach of estimating the relative bending stiffness in hogging and sagging, a new design envelope was found which reduces the scatter of the previous works of Potts and Addenbrooke (1997) and Franzius et al. (2006) (Mair, 2013). Figure 2.17a illustrates this design chart and shows the good fit of various field data. The proposed relative bending stiffness in hogging and sagging is given by:

\[
\rho_{sag} = \frac{EI}{E_s L_{sag}^3}
\]  

(2.29)
2.2 Response of buildings to tunnelling-induced settlements

(a) Deflection ratio (Mair, 2013).

(b) Horizontal strain (Goh and Mair, 2011c)

Fig. 2.17 Design curves for modification factors.

\[ \rho_{hog} = \frac{EI}{EsL_{hog}^3} \]  

(2.30)

where \( L_{sag} \) and \( L_{hog} \) are the length of the building transverse to the tunnel heading in the sagging and hogging region respectively. This approach accounts for different response of buildings in hogging and sagging which is widely found when analysing field data. Additionally, this partitioning approach is based on the greenfield inflection point \( i_{GF} \). The modified horizontal building strains can either be found by applying Equation 2.28 and using the design chart (Figure 2.17b) identified by Goh and Mair (2011c) or can be neglected according to Mair (2013).

A considerably different methodology was suggested by Son and Cording (2005) who focused on the shear stiffness of the building when estimating a relative stiffness which can be expressed as:

\[ \rho_{SC} = \frac{E_s L^2}{GHB^w} \]  

(2.31)

where \( E_s \) is the soil stiffness in the foundation influence zone and \( b_w \) the thickness of the façade. Figure 2.18a shows that the related design chart is based on a reduction of \( \beta \). Additionally, this framework focuses on building units (similar to the SoS), accounts for building damage and the effect of structural components perpendicular to the façade.

Son and Cording (2005) recommended to apply the approach proposed by Boscardin and Cording (1989) to consider a potential reduction of the horizontal strains due to the presence of a building. As can be seen from the related design chart (Figure 2.18b), the Boscardin and
Cording (1989) formulation for the axial stiffness can be written as:

$$\alpha_{SC} = \frac{E_gA}{E_sHS} \quad (2.32)$$

where $E_g$ is the Young’s modulus of the foundation (i.e. grade beam in Boscardin and Cording (1989)), $A$ the cross-section of the foundation, $H$ the depth of the foundation and $S$ the spacing between the footings.

Although these RSMs are encouraging to better predict building response to tunnelling, a frequently discussed limitation is the accuracy of predicting the building stiffness $EI$ and $EA$ which directly affects the damage assessment (Giardina et al., 2017; Haji et al., 2018).

### 2.2.5.1 Estimating the global building stiffness

Estimating the stiffness of a structure is by far not a straightforward process due to uncertainties about the interaction of various building components (e.g. foundations, walls, floors and roof), the material properties, the history of damage and construction details (Giardina et al., 2017; Melis and Rodriguez Ortiz, 2001).

Potts and Addenbrooke (1997) proposed to calculate the building stiffness by considering the floor of each storey and neglecting foundations, columns and walls of the building. It was assumed that the neutral axis of the building is at mid-height and the parallel axis theorem was applied to define the stiffness of each concrete slab against bending about its neutral axis. The axial ($EA$) and bending ($EI$) stiffness of the structure can be calculated as:

$$EA = E_c \sum_{i=1}^{n+1} (A_{slab,i}) \quad (2.33)$$

$$EI = E_c \sum_{i=1}^{n+1} (I_{slab,i} + A_{slab,i}d_i^2) \quad (2.34)$$

where $E_c$ is the elastic Young’s modulus of concrete, $A$ the cross section, $n$ the number of storeys, $I$ the second moment of area and $d$ the distance of the neutral axis of each concrete slab to the neutral axis of the entire structure. Potts and Addenbrooke (1997) pointed out that this approach of calculating the building stiffness may results in an overestimation of the ’real’ building stiffness because the parallel axis theorem applies only in a rigidly framed structure where each element contributes to the entire building stiffness. For this reason, they suggested the following alternative:

$$EI = E_c \sum_{i=1}^{n+1} (I_{slab,i}) \quad (2.35)$$
2.2 Response of buildings to tunnelling-induced settlements

(a) Angular distortion (Son and Cording, 2005).

(b) Horizontal strain (after Son and Cording, 2005).

Fig. 2.18 Design curves for modification factors focusing on shear distortions (Boscardin and Cording, 1989; Son and Cording, 2005).
which is based on Meyerhof (1953) who also proposed building stiffness estimates for multi-
storey building frames with different building typologies. A similar approach as shown in
Equation 2.35 was also applied by Lambe (1973).

Melis and Rodriguez Ortiz (2001) suggested another procedure of calculating the build-
ing’s axial and bending stiffness which considers the contribution of each individual element
of the structure (e.g. floors, walls and basement):

\[ EI = \sum (EI)_{floors} + \sum (EI)_{walls} + \sum (EI)_{basement} \]  

(2.36)

The \( EI \) per metre run is usually determined by obtaining \( EI \) of each structural typology
with the building’s width, \( B \). This so-called spacing factor approach to estimate the plane-
strain building stiffness is further addressed in Section 4.5.1.2. Additionally, Melis and Ro-
driguez Ortiz (2001) suggested the following:

Bending stiffness estimation of the floors:

• The neutral axis of a building can be assumed to be close to the mid-height of a structure.

• Each floor contributes to the entire building stiffness; thus, the parallel axis theorem has
to be applied for every floor.

Bending stiffness estimation of the walls:

• Load bearing walls and concrete framed structures with infill of common-bricks have to
be considered when deriving the bending stiffness of the walls (\( EI_{walls} \)). Walls of con-
crete framed structures with air bricks or hollow bricks and columns of framed structures
are negligible.

• In certain cases, internal walls considerably contribute to the overall stiffness of a build-
ing and thus might be considered.

• Openings notably reduce the bending stiffness of the wall which has to be taken into
account. The proposed approach of Melis and Rodriguez Ortiz (2001) to account for
building openings is presented below.

Bending stiffness estimation of the basement:

• If the basement is hinged to the walls and floors of the building (superstructure), the
bending stiffness of the basement (basement walls, continuous footings or the raft foun-
dation) has to be estimated assuming the neutral axis at mid-height of the basement.
2.2 Response of buildings to tunnelling-induced settlements

Table 2.9 Bending stiffness reduction factors due to façade openings and $L/H$ ratio (Melis and Rodríguez Ortiz, 2001).

<table>
<thead>
<tr>
<th>Type of wall</th>
<th>Length $&lt; H$</th>
<th>Length $&gt; 2H$</th>
</tr>
</thead>
<tbody>
<tr>
<td>No openings</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Openings from 0% to 15%</td>
<td>0.70</td>
<td>0.90</td>
</tr>
<tr>
<td>Openings from 15% to 25%</td>
<td>0.40</td>
<td>0.60</td>
</tr>
<tr>
<td>Openings from 25% to 40%</td>
<td>0.10</td>
<td>0.40</td>
</tr>
</tbody>
</table>

- By contrast, if the basement is rigidly connected to the superstructure it is assumed that the neutral axis is at mid-height of the entire structure.

**Considering façade openings**

Based on data of various case histories, Melis and Rodríguez Ortiz (2001) proposed reduction factors to reduce the bending stiffness of walls when openings such as doors or windows are present. Table 2.9 summarises these reduction factors which are a function of the aspect ratio ($L/H$). Son and Cording (2007) carried out numerical investigations to study the change of the bending stiffness of masonry walls due to window openings and reported a reduction of the overall bending stiffness of as high as 26% when openings of 30% are present. More striking, the shear stiffness reduced up to 77% (Son and Cording, 2007). Pickhaver et al. (2010) proposed an alternative framework to account for façade openings that also considers the $L/H$ ratio.

The recommendations of Melis and Rodríguez Ortiz (2001) provide an adequate framework to estimate $EI$ but engineering judgement is required to assess the building stiffness of a structure. For instance, as the transfer of shear between the individual elements of a building depends strongly on the quality of the connections between individual elements such as floors and walls, it is not always reasonable to assume that every component of the building fully contributes to the overall building stiffness. Specifically, when buildings consist of load bearing masonry with timber slabs, the slab to wall connections are generally not fully rigid and shear transfer between the wall and the slab is restrained (Farrell et al., 2011). Hence, in such cases it is recommend to calculate the global bending stiffness by taking the neutral axis of each component itself.

Mair and Taylor (2001) estimated the per metre building stiffness of masonry structures and walls as $EI = E b H^3$, where $b = 1$ and represents the unit building width. This procedure also adopted the spacing factor approach. However, Dimmock and Mair (2008) pointed out that the bending stiffness of a building is probably different in the hogging and sagging zones and thus suggested to reduce the overall bending stiffness of a building deforming in a hogging mode by considering only the contribution of the foundations. In sagging, the neutral axis can

49
Literature review

remain at mid-height of the structure because buildings in sagging are less prone to damage (Burland and Wroth, 1974). This empirical approach of estimating the bending stiffness in hogging by considering solely the foundations can result in significantly different $EI$ values in hogging and sagging. This is in particular evident for buildings with very shallow foundations and $EI_{\text{sag}}/EI_{\text{hog}}$ values as high as 600 may be observed (Dimmock and Mair, 2008). Further frameworks to estimate the overall stiffness of frame structures are reported elsewhere (Goh and Mair, 2014; Haji et al., 2018; Netzel, 2009).

Overall, the estimation of the global stiffness of a building is affected by various factors such as the building type, the building material used, the position of the building with respect to the tunnel axis, the existing damage of the building and the contribution of the building components out of plane-strain. Furthermore, the discussed methods of calculating the building stiffness assume a perpendicular location of the building to the tunnel axis. In reality, most buildings are located in a skewed configuration to the tunnel and it is likely that such structures are even subject to torsional strains (Melis and Rodriguez Ortiz, 2001). Several authors argue that such scenarios require a detailed investigation using 3D finite element analysis (Melis and Rodriguez Ortiz, 2001; Rampello et al., 2012). The overall uncertainty in the estimation of the overall plane-strain bending stiffness of a structure may result in a significant scatter when predicting the response of buildings to tunnelling-induced settlements (Giardina et al., 2017) which indicates the need for refinement of estimating $EI$.

2.2.5.2 The dominant mode of building deformations

Tunnelling-induced settlement damage to pre-existing buildings is a result of bending and shear deformations, which typically occur simultaneously. However, as outlined in Section 2.2.5, available assessment methods (e.g. Franzius et al., 2006; Goh and Mair, 2011a; Potts and Addenbrooke, 1997; Son and Cording, 2005) tend to focus on the mode of deformation (i.e. shear or bending) that is assumed to govern the onset of building damage. For this reason, in these RSMs the relative stiffness is calculated by either the bending (Franzius et al., 2006; Goh and Mair, 2011a; Potts and Addenbrooke, 1997) or shear stiffness (Son and Cording, 2005) of the structure. This inconsistency can also be highlighted by the different deformation parameters (i.e. $DR$ or $\beta$) used in these methods. While research showed that the governing contribution (i.e. shear or bending) is a function of $L/H$ (Burland and Wroth, 1974; Pickhaver et al., 2010) and the façade opening area (Melis and Rodriguez Ortiz, 2001; Pickhaver et al., 2010) the importance of bending or shear deformations is still not agreed on. Currently, a relative stiffness method that accounts for both bending and shear response is missing.
2.2.6 The influence of the building weight

Franzius et al. (2004) conducted a parametric study on building weight effects and thus provided notable insight into the effects of the building load on this tunnel–soil–structure interaction problem. This previous work demonstrated that building load changes the stress state of the soil surrounding a tunnel. This change of the mean effective stresses controls the soil stiffness, changes the $\sigma_h' / \sigma_v'$ ratio, is more dominant in the region close to the surface and decreases with depth. For these reasons, the tunnelling-induced ground deformations may alter. These ground deformations propagate through the soil body, with changed mechanical properties due to the building load, and finally interact with the building close to the soil surface.

An increase of the soil stiffness directly below the building affects the horizontal ground and building deformations (Franzius et al., 2004). Firstly, an increase of the soil stiffness results in a reduction of the horizontal ground movements. Secondly, the higher soil stiffness enables a significantly greater transfer of these horizontal ground movements to the foundations which results in an increase of the horizontal strain transferred to the building (though considerably smaller than the greenfield). Similar findings of an increase of the building distortions with increasing soil stiffness were also reported by Netzel (2009) and Son (2015). However, Franzius et al. (2004) concluded that for realistic building load to stiffness combinations the influence of the building weight tends to cause a negligible increase of the related modification factors.

Augarde et al. (2004) is critical of some of the conclusions of Franzius et al. (2004) and argues that the stiffness of masonry structures might considerably drop due to progressive damage caused by tunnelling effects. Due to this reduced building stiffness the building weight effects become considerably more important. Liu (1997), Liu et al. (2001) and Pickhaver et al. (2010) reported that the building load is the governing factor of the magnitude of the settlements and the building stiffness is responsible for the shape of the settlement trough. The increase of the magnitude of tunnelling-induced settlements caused by increasing building load was also observed in several centrifuge tests (Farrell, 2010; Taylor and Yip, 2001) and in the field (Bilotta et al., 2017; Farrell et al., 2011).

The redistribution of the building weight caused by tunnelling-induced ground displacements also plays a vital role in this tunnel–soil–structure interaction. Farrell (2010) and Farrell et al. (2011) showed that rigid buildings tend to redistribute their load which potentially causes localised embedment of the building into the soil. This embedment can result in severe tilt of a building and considerable differential soil displacements close to the edge of the building which potentially affects buried services (Farrell et al., 2011).
More recently, Giardina et al. (2015a) demonstrated that the interaction between the soil and the tunnelling-induced soil displacements notably depends on the building’s self-weight. Consequently, Giardina et al. (2015a) proposed to normalise the relative stiffness expression defined by Goh and Mair (2011a) by a dimensionless indicator of the building weight, \( n_s \), which is equivalent to the number of storeys. This refinement of the Goh and Mair (2011a) relative stiffness formulation showed the potential to narrow the design envelopes identified by Goh and Mair (2011a), though further field data is required to confirm this finding.

### 2.2.7 Centrifuge tests quantifying the soil–structure interaction

Centrifuge model testing demonstrated significant potential to simulate realistic ground displacements above tunnels (e.g. Franza, 2017; Grant and Taylor, 2000; Mair, 1979; Marshall et al., 2012; Potts, 1976) through replicating prototype self-weight stresses in the soil model. Previous centrifuge modelling studies on the building response to tunnelling-induced settlements are informative (Caporaletti et al., 2005; Farrell, 2010; Farrell and Mair, 2010, 2011; Taylor and Grant, 1998; Taylor and Yip, 2001), but employ rather simple building models. Furthermore, initial centrifuge modelling studies on the effect of surfaces structures on tunnelling-induced ground movements (Caporaletti et al., 2005; Taylor and Grant, 1998; Taylor and Yip, 2001) were limited by the precision of the employed image-based deformation measurement techniques. As a consequence, the reported conclusions, including the impact of the building stiffness (Caporaletti et al., 2005; Taylor and Grant, 1998; Taylor and Yip, 2001), the building weight (Taylor and Yip, 2001) and the building eccentricity relative to the tunnel centreline (Taylor and Yip, 2001) on the tunnelling-induced ground displacements, were derived at unrealistically high tunnel volume loss (e.g. 10% in Taylor and Yip, 2001).

Recent centrifuge tests on simplified model structures (aluminium, micro-concrete and masonry beams, Figure 2.19) above tunnels in sand (Farrell, 2010; Farrell and Mair, 2010, 2011) identified a loss of contact between a rigid structure with zero eccentricity and the soil (Farrell and Mair, 2010, 2011), a significant reduction of the horizontal displacements transferred to the building (Farrell and Mair, 2010, 2011) and the depth of constraint of horizontal soil displacements beneath structures (Farrell and Mair, 2010). Moreover, Farrell (2010) confirmed the crucial role of the building stiffness in this tunnel–soil–structure interaction system which was a significant contribution to the RSM according to Goh and Mair (2011a). These recent findings were made at more realistic tunnel volume losses. So far, however, the impact of more representative building models, with, for example, realistic foundations, partition walls and façade openings on soil and building displacements caused by tunnelling works has not
2.2 Response of buildings to tunnelling-induced settlements

![Masonry structure](image)

**Fig. 2.19** Masonry structure studied in Farrell (2010) (after Farrell and Mair (2011)).

been investigated. Furthermore, the vital role of the building position relative to the tunnel has not been closely examined.

### 2.2.8 Summary

This section provided the background of assessing potential damage of buildings caused by tunnelling-induced ground movements. Centrifuge tests that studied the interaction between surface structures and tunnelling-induced settlements were summarised, and their limitations were addressed. Additionally, numerous computational studies exist, which would be unrealistic to cover in detail in this review, but throughout this thesis it will be referred to these previous works when relevant. Within this summary, the LTSM and the RSMs with focus on their limitations are discussed.

#### 2.2.8.1 Discussion of the limiting tensile strain method

The LTSM is widely accepted but is based on various simplifications, which are as follows:

- Within the LTSM no attempt is made to account for interactions between a building and the soil. Consequently, the LTSM is often overly conservative.

- A full transfer of horizontal ground movements to the building is generally assumed within the LTSM framework.

- The LTSM assumes that the building is subject to uniform strain out of plane (plane-strain analysis) which may result in different building deformations and damage when structural stiffness differences out of plane exist.

- Structural details such as façade openings or different foundation layouts cannot be represented with fictitious \( E/G \) values. Consequently, strain localisations cannot be captured within the LTSM.
Literature review

- The LTSM models buildings as a linear-elastic, weightless beam. Thus non-linear mechanical properties of typical buildings are neglected.

- Building load and its redistribution is ignored within the LTSM.

- The condition of a building before tunnel construction is not considered in the LTSM. Pre-existing damage might considerably reduce the vulnerability of buildings to ground movements.

- The LTSM is limited to transverse displacement profiles. Skewed building-to-tunnel positions and longitudinal soil displacements are ignored.

Though these limitations exist, the LTSM in conjunction with the three-stage assessment framework has proven to be a practical tool when assessing the risk of building damage due to tunnelling-induced ground movements. However, a general consensus exists that the LTSM often results in overly conservative predictions.

2.2.8.2 Discussion of the relative stiffness methods

The vital role of the building stiffness in this tunnel–soil–structure interaction problem is widely accepted. For this reason, various relative stiffness formulations were proposed (Franzius et al., 2006; Giardina et al., 2015a; Goh and Mair, 2011a; Potts and Addenbrooke, 1997; Son and Cording, 2005) to account for this interaction. Although these relative stiffness relationships are a valuable contribution to consider soil–structure interaction effects, recent research showed that the relative stiffness methods available result in inconsistent predictions (DeJong et al., 2016; Giardina et al., 2017). These observations may are caused by some of the inherent limitations of the RSM which are as follows:

- The numerical work of Potts and Addenbrooke (1997) and Franzius et al. (2006) is based on a simplified building model. The building is modelled by using linear-elastic material properties. Thus, important mechanisms such as cracking of masonry are not accounted for. Building features such as the building layout, foundations and façade openings are also neglected.

- Another limitation related to the building model of Potts and Addenbrooke (1997) and Franzius et al. (2006) is that uniform building stiffness out of the plane is assumed. Also in the 3D analyses of Franzius et al. (2006), the same building model with uniform building stiffness out of plane is used. Clearly, this assumption does not realistically model the nature of real structures.
2.3 Performance of masonry to tunnelling-induced settlements

- The main building input in the suggested relative building stiffness relationships is the global stiffness of the building. It has been shown that the estimation of this building bending or axial stiffness is a challenging task and a significant amount of uncertainty is included. Specifically, to account for window openings and different building layouts when estimating the overall building stiffness per metre run is challenging.

- To date, there is little agreement on the governing mode of building deformation. While some methods focus on bending distortions, others solely consider shear distortions. A relative stiffness method that accounts for both bending and shear stiffness of a building is still missing.

- The available relative stiffness expressions reflect that the building-to-tunnel positions is affecting the building response. For instance, Potts and Addenbrooke (1997) and Franzius et al. (2006) provide different design lines for different building eccentricities while Goh and Mair (2011a) provide a unique design envelope for all building-to-tunnel positions but suggest to partition a structure at the greenfield inflection point. This indicates that much uncertainty still exists on the impact of the position of the structure relative to the tunnel.

2.3 Performance of masonry to tunnelling-induced settlements

Masonry structures are prone to extensive damage caused by small differential settlements (Burd et al., 2000). For this reason, this section focuses on the response of masonry buildings to tunnelling. First, a background of the mechanical properties of masonry is given, after which recent research on the performance of masonry structures to tunnelling-induced ground settlement is reviewed.

2.3.1 Mechanics of masonry

Masonry is a composite material of units and mortar joints (Figure 2.20). A huge number of possible combinations of different types of units, e.g., clay bricks and stone blocks, and mortars, e.g., lime mortar or Portland cement mortar, exist. Throughout this work the term masonry refers to a composite material of clay bricks and mortar. Although this is a considerable reduction, a large scatter between the observed material properties within brickwork exists. Clearly, the variability of the strength of the units and the mortar and different types of
arrangement of the units are reasons for this scatter. Rots et al. (1997) summarised additional factors influencing the mechanical properties of masonry which are: moisture condition of the unit, finishing, joint width, suction rate of the unit, dimension of the unit (ratio between the joint thickness and unit weight), cracks and stresses within the unit, quality of workmanship and finishing of joints. For entire buildings, the construction sequence of different walls, a possible variability of the used bricks and mortars and existing damage may also affect its behaviour (Al Heib, 2012).

Brickwork is characterised by inhomogeneous material behaviour due to its two components brick and mortar (Lourenco, 1996). Hilsdorf (1969) demonstrated that the source of masonry failure is the difference between the elastic properties of the units and the mortar. The most striking material property of masonry is the high compressive strength in the direction perpendicular to the bed joints. This material property of masonry explains the application of masonry in load-bearing walls. Another important mechanical property of masonry is its low tensile strength which under tensile loading results in cracking.

Figure 2.21 illustrates the behaviour of masonry in compression, tension and shear. The material properties $f_c$ and $f_t$ denote the compressive and tensile strength respectively. It can be seen from Figure 2.21 that stress increase leads to a considerable increase of the displacement which is attributed to quasi-brittle materials such as masonry. This softening behaviour is the result of a growth of internal cracks which is the general failure mechanism of heterogeneous materials in tension and compression. To describe this inelastic behaviour of masonry, the term fracture energy ($G_f$ in tension and $G_c$ in compression) is introduced which is the area below a stress-displacement curve (Figure 2.21a and 2.21b). These material properties describe the post-peak behaviour of masonry.

Another salient feature of masonry is shear failure (Figure 2.21c). Under shear loading, slip within the interface of the unit and the mortar can occur and is denoted as mode II failure. The related fracture energy is the so-called mode II fracture energy $G_{II}^f$. By contrast, a loss of bond between the unit and mortar due to tensile failure is termed mode I failure with the
2.3 Performance of masonry to tunnelling-induced settlements

corresponding mode I fracture energy $G_{I}^{f}$. This indicates that mortar beds are often weak zones of masonry. Specifically, the bond between the mortar and the unit is a distinct weakness.

Based on this review of the main properties of masonry, it is evident that masonry has to be modelled using a non-linear material which captures these complex material characteristics. In particular when cracking initiates, the material behaviour of masonry alters which cannot be described by linear-elastic materials.

2.3.2 The mode of deformation

While the emphasis of Section 2.2.5.2 was placed on bending or shear deformations, the following highlights the effect of the building position relative to the tunnel on either hogging are sagging deformations.

As discussed in Section 2.2, Burland and Wroth (1974) accounted for the different behaviour of masonry in hogging and sagging by lowering the position of the neutral axis of the beam model to the lower fibre while the neutral axis is at mid-height for the sagging mode. This approach is explained by a significant lateral restraint of tensile strains provided by the foundation and soil in sagging. This restraint results in predominantly compressive strains within the masonry façade. However, masonry walls in hogging offer little resistance to tensile strains because of its inability to withstand tensile strains in the upper part of the wall and thus the same amount of angular strain leads to damage. In other words, masonry buildings in hogging are more vulnerable to cracking which may lead to an extensive loss of stiffness. This stiffness reduction in hogging results in a more flexible response of the masonry building to tunnelling-induced ground movements. However, for masonry buildings in sagging it is likely that the same building behaves as rigid or semi-flexible.

Cases studies show that real masonry walls indicate the same trend of being more vulnerable in hogging (Dimmock and Mair, 2008). Numerical studies of masonry structures which accounted for the complex material properties of masonry also revealed the different performance of masonry in hogging and sagging. Liu (1997) explained this behaviour by arching effects within masonry façades. For masonry walls in sagging, arching was observed between windows (Figure 2.22), which results in a different transfer of building loads to the ground and thus a different soil response. The pressure below each base of the stress arch increases considerably and thus leads to a flattened soil profile which can be seen in Figure 2.22. Because of this, the building experiences fewer differential settlements and damage.

However, in the hogging mode, where horizontal tensile strains are predominant, masonry façades cannot form these arches. As a consequence, masonry walls in hogging experience
Literature review

Uniaxial compressive behaviour of masonry.

Uniaxial tensile behaviour of masonry.

Behaviour of masonry under shear.

Fig. 2.21 Typical stress-displacement curves of masonry (after Lourenço, 1996).
2.3 Performance of masonry to tunnelling-induced settlements

significantly higher differential settlements and thus are more vulnerable to cracking. Friction table tests performed by Cox (1980) also showed these stress arches within brickwork when subject to sagging. However, in the hogging mode severe tensile strains were observed close to the top of the wall (Cox, 1980).

2.3.3 Damage patterns

When analysing the performance of masonry to tunnelling, crack patterns within a masonry building are typically discussed. A review of data from published case studies, numerical investigations and 1g physical tests found that generally the same trends were observed. Following the previous discussion, it is obvious that differences between the crack pattern in sagging and hogging exist, as shown in Figure 2.23. Cracks, which are caused by ground movement, often concentrate in areas of maximum distortion or in structurally weak zones such as window and door openings (BRE, 1995).

Giardina et al. (2012) investigated the behaviour of masonry subject to hogging by performing a 1/10th scaled test on a wall with openings. With respect to the observed crack pattern, it was reported that an initial crack arose at the top of the façade at a window corner. This fits well to the theory that masonry in hogging cannot withstand tensile strains at the top of the wall. With increasing ground movement, further cracks between the corners of openings evolved. This caused a stress redistribution within the façade and further cracks emerged. Giardina et al. (2012) also reported that the cracks subdivide the masonry façade into rigid body blocks which deformed according to the ground settlements and thus triggered further damage. In this previous experiment the majority of cracks were observed within the mortar joints.
Laefer et al. (2011) performed a similar physical experiment which studied the performance of a 1/10th scaled historic masonry building subject to deep excavation-induced settlements. Interestingly, the reported damage patterns are notably different to the one observed by Giardina et al. (2012). The main difference was that cracks emerged from the bottom and subsequently propagated to the top. Although both experiments lack roof or floor elements, the initial crack evolution is different. Beside this difference, Laefer et al. (2011) also observed that most of the cracks appeared at the interface between the brick units and the mortar. Furthermore, the crack localisation was diagonal between the corners of the openings. Laefer et al. (2011) compared the crack pattern of this experiment with cracks monitored at case studies and found good agreement. However, the authors reported that the major difference of the experimental results was that cracks were distributed over a larger area of the masonry wall with respect to field data. Laefer et al. (2011) explains this by a missing roof on top of the masonry model which might confine the crack pattern. In contrast, the reported case studies generally showed cracks at the building corner closest to the excavation and around window openings (Laefer et al., 2011).

As pointed out above, cracks mainly develop in weak zones of the building such as at corners of windows and door openings. Giardina et al. (2015c) conducted a numerical parametric study on the influence of wall openings by varying the amount of opening from 0% to 10% and 30%. The results of this investigation showed that an increase of the amount of openings results in strain concentrations which result in localisation of cracks between the corners of the openings (Figure 2.24). Similar results were reported by Yiu et al. (2017). This shows that the performance of masonry subject to ground movements caused by tunnelling is notably affected by façade openings. Thus, guidelines proposed by Melis and Rodriguez Ortiz (2001),
2.3 Performance of masonry to tunnelling-induced settlements

Fig. 2.24 Maximum principal strain distribution of masonry walls with different amount of window openings (Giardina et al., 2015c). The small ‘openings’ are imperfections such as missing bricks.

Son and Cording (2007) or Pickhaver et al. (2010) to account for openings when estimating the building stiffness are worthwhile.

2.3.4 Summary

Masonry structures are susceptible to settlement damage caused by tunnel construction. Important characteristics of masonry related to tunnelling-induced settlements are:

- a high compressive strength,
- a low tensile strength,
- a distinct post-peak behaviour (i.e. softening after cracks occur).

The performance of masonry structures subject to tunnelling-induced settlements is highly dependent on the position relative to the tunnel. Damage in the form of cracking is generally observed in the hogging region and results in a significant loss of the stiffness of the structure. By contrast, masonry structures in the sagging region withstand deformations considerably better. Experiments, field data and numerical studies showed that masonry cracking is located within weak zones of the structure such as the corners of window or door openings.

It was highlighted that the response of masonry to tunnelling-induced displacements is complex because both the soil and the structure are characterised by highly non-linear material properties and the structural behaviour is influenced by building features. For this reason, it is essential to capture this complex masonry behaviour when experimentally modelling this tunnel–soil–structure interaction.
Chapter 3

Experimental method and equipment

This chapter discusses the centrifuge model testing adopted in this research. First, a brief introduction into the principles of centrifuge modelling is presented, after which common errors and limitations of centrifuge modelling are addressed. Then the experimental setup including the centrifuge strong box, the soil type, the model tunnel and the tunnel volume and pressure control system are described. Subsequently, details of the model preparation, the data acquisition and the instruments used to monitor soil and building behaviour are reported. Finally, the testing procedure and an overview of the centrifuge test series is presented. Due to the novelty of the small-scale building models used within this work, the following chapter will solely focus on the 3D printed building models. Chapter 5 will discuss specific experimental challenges observed throughout this research.

3.1 Introduction to centrifuge modelling

Centrifuge model testing has been demonstrated to be of invaluable use for studying geotechnical engineering problems. The advantage of geotechnical centrifuge tests compared to 1g experiments is that correct self-weight stress-strain behaviour of the soil and structure can be captured. Because of this, realistic soil and structural behaviour, which both are often highly non-linear and dependent on current stress state, can be replicated in a centrifuge model test.

Due to the artificial gravitational field within the centrifuge it is possible to generate stress profiles that vary in depth equivalent to what is observed in prototype scale. This is achieved by preparing a small scale model (e.g. scaled down by the factor \( N \)) and rotating it about the centrifuge axis until the centrifuge model experiences \( N \) times the Earth’s gravity, \( g \). As a consequence of the increased gravitational field, the self-weight soil stresses in the small-scale
Experimental method and equipment

Table 3.1 Scaling laws relevant for this research (after Kutter, 1992).

<table>
<thead>
<tr>
<th>Quantity</th>
<th>Units</th>
<th>Scaling law (model/prototype)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>m</td>
<td>1/N</td>
</tr>
<tr>
<td>Area</td>
<td>m²</td>
<td>1/N²</td>
</tr>
<tr>
<td>Volume</td>
<td>m³</td>
<td>1/N³</td>
</tr>
<tr>
<td>Mass</td>
<td>kg</td>
<td>1/N³</td>
</tr>
<tr>
<td>Acceleration, Gravity</td>
<td>m/s²</td>
<td>N</td>
</tr>
<tr>
<td>Force</td>
<td>N</td>
<td>1/N²</td>
</tr>
<tr>
<td>Stress</td>
<td>N/m²</td>
<td>1</td>
</tr>
<tr>
<td>Strain</td>
<td></td>
<td>1</td>
</tr>
<tr>
<td>Bending Stiffness</td>
<td>Nm²</td>
<td>1/N⁴</td>
</tr>
<tr>
<td>Axial Stiffness</td>
<td>N</td>
<td>1/N²</td>
</tr>
</tbody>
</table>

model substantially increase with depth and resemble full-scale soil stress profiles. Table 3.1 summarises scaling laws to convert measured small-scale model values to prototype scale.

Numerous researchers applied centrifuge modelling to investigate tunnel stability, tunnelling-induced ground movements and tunnelling effects on existing structures including tunnels, pipelines and buildings on shallow and piled foundations. Previous centrifuge model tests on building response to tunnelling were primarily concerned with the influence of the building stiffness on the response of structures to tunnelling-induced settlements (Caporaletti et al., 2005; Farrell, 2010; Taylor and Grant, 1998; Taylor and Yip, 2001). These experiments provided useful data to get further insights into the interaction between the tunnel excavation and nearby structures.

3.2 Errors and limitations in centrifuge modelling

When carrying out centrifuge model tests, it is of importance to be aware of inherent errors and limitations of this technique. The sections below point out the main errors of centrifuge modelling and relate them to the experimental setup adopted in this research. An extensive discussion of errors in centrifuge modelling is provided by Taylor (1995a).

3.2.1 Variation in gravity field

The ground stresses observed in the field increase uniformly with depth as a result of the Earth’s gravity field. However, in centrifuge modelling the centrifugal gravity field depends on the distance from the centre of rotation and varies within the soil model. Thus, in a centrifuge model the soil stresses will be slightly non-uniform with regions of over-stress and under-stress compared to the vertical prototype stress profile. It was estimated that the maximum
over and under-stress values within the adopted centrifuge model, shown in Figure 3.1, are about 1.22%. Differences of the gravitational field occur also in lateral direction and increase with distance from the centreline of the centrifuge model. The maximum error of the radial gravity field, which is located at the bottom edges of the centrifuge model, was estimated to be about 0.46%. These errors are considered to be relatively small and thus of negligible effect on the observed mechanisms.

### 3.2.2 Influence of the Earth’s gravity field

After swing-up of the centrifuge package, the model container is in a horizontal position and in this research project approximately 75 times gravity act in this horizontal plane (which is the vertical direction with respect to the tested model or the corresponding prototype). However, the Earth’s gravity field (1g) acts perpendicular to this horizontal plane and thus the resulting acceleration field acting on the centrifuge model is slightly out of the horizontal plane. It was calculated that the angle of this effective acceleration field is 0.76° to the horizontal plane.

### 3.2.3 Particle size effects

A common criticism of centrifuge modelling is that the soil particle size is not scaled according to the scale factor of the main components of the centrifuge model (i.e. tunnel and structures for this study). Clearly, scaling sand by a factor of 1:75 would result in particle sizes similar to clay and significantly change the stress-strain behaviour compared to the prototype soil. For this reason, the same soil as the prototype is used in centrifuge model testing and considered to be a continuum which is an assumption often made in soil mechanics (Madabhushi, 2014). However, this continuum assumption has limitations, which are specifically relevant when modelling soil–structure interaction mechanisms, and under certain conditions can result in erroneous results. To avoid these biased results, which are often called particle size effects, research demonstrated that the ratio between the smallest dimension of a tested structure and the average particle size ($d_{50}$) has to exceed certain limits. Fuglsang and Ovesen (1988) recommended that this critical ratio should not be below 30. Bolton et al. (1993) investigated particle size effects when performing in flight cone penetration tests in sand and stated that cone to particle size ratios greater than 20 lead to unbiased results. For the conducted centrifuge model tests, the foundation width ($b_f = 10.7$ mm) represents the smallest model dimension in contact with the soil. The used soil is a dry silica sand (Section 3.3.3) and is characterised by a $d_{50}$ value of 0.140 mm. Thus, the ratio between the dimension of the foundations and the
grain size is 10.7/0.140 which results in about 76. This value is significantly higher than the literature thresholds (see above).

Another important aspect of particle size effects is related to shear mechanisms within the soil model. Kutter et al. (1994) performed centrifuge tests to exploit the mechanisms involved when cavities collapse in sand. This previous work found that the volume of the generated crater ($V_{cr}$) normalised by the volume of the cavity ($V_{ci}$) is a function of the ratio between the diameter of the cavity ($D_{ci}$) and the average grain particle size ($d_{50}$). For $D_{ci}/d_{50}$ values between 30 to 1000, particle size effects were observed which decrease when the $D_{ci}/d_{50}$ ratio is greater than 350 (Kutter et al., 1994). Marshall (2009) updated this finding by accounting for the volumetric behaviour of sands and stated that particle size effects can be neglected for $D_{ci}/d_{50}$ ratios above 500. However, Marshall et al. (2009) also pointed out that lower $D_{ci}/d_{50}$ thresholds are reasonable when pre-collapse mechanisms are studied. For this research the cavity is replaced by the model tunnel with a diameter of 82 mm. Thus, the ratio between the tunnel diameter and $d_{50}$ is about 585 and above the threshold defined by Marshall (2009). Therefore, it is assumed that particle size effects will not considerably influence the results of this centrifuge model testing program.

3.2.4 Boundary conditions

When conducting plane-strain centrifuge experiments and applying imaged-based displacement techniques such as digital image correlation (DIC), it is of importance to be aware of possible friction effects between the soil and the Poly(methyl methacrylate) (PMMA) window. These boundary effects are caused by the frictional forces between the sand grains adjacent to the PMMA window and might affect the DIC results, which are measures of soil displacements within the PMMA plane. Due to the relatively low hardness of the PMMA window and the high gravitational field acting on the soil model, it can be assumed that sand grains will scrape into the PMMA which will constrain soil movements. This may result in unreliable measurements. Previous research was carried out by Marshall (2009) to quantify these boundary effects. Marshall et al. (2009) compared surface soil displacements measured with DIC with data from lasers and linear variable differential transformers (LVDTs) obtained at distance from the PMMA window; he found that the DIC data is about 10% lower than the vertical soil displacements close to the centre of the soil model. Clearly, this frictional effect increases with distance from the soil surface due to increasing overburden pressure causing higher friction between the PMMA and the sand grains. Farrell (2010) and Elshafie et al. (2013) reported boundary effects of similar magnitude. Although these boundary conditions
exist, the results of Marshall (2009) and Farrell (2010) showed that the physical mechanisms investigated are not significantly affected by these boundary effects. DIC results of realistic ground movements in fair agreement with field data were reported by numerous researchers.

Specific modelling limitations and boundary effects which were observed in this work are addressed in Chapter 5.

3.3 Experimental setup and equipment

This section focuses on the used experimental setup and equipment. The adopted strong box, soil model, model tunnel and model dimensions are based on those used by Farrell (2010). Marshall et al. (2012) demonstrated that this experimental setup realistically reproduces greenfield displacements for tunnels in sand. Farrell (2010) recently employed this experimental setup to study the response of aluminium, micro-concrete and masonry beams to tunnelling subsidence (see also Farrell and Mair (2011) and Giardina et al. (2015a)). Due to the similar tunnelling scenario modelled, these experiments provide a unique database to compare with the research on more representative building models described herein.

3.3.1 Model geometry and instrumentation

Figure 3.1 provides the basic dimensions of the centrifuge model, which replicates a 75 times larger tunnelling project with a cover-to-tunnel diameter ratio, $C/D_t$, of 1.35. While the soil conditions were kept constant, the building eccentricity, $e$, the building length, $L$, and the façade openings, $O$, varied between tests. Additionally, results of a greenfield test performed by Farrell (2010), which replicated the identical tunnelling prototype, are presented.

A relatively shallow tunnel was modelled because building damage decreases with tunnel depth (Son, 2015; Vu et al., 2015). The entire centrifuge model tests were performed at an angular velocity, $\omega$, of 131.9 revolutions per minute. This results in a gravitational field of 75.5$g$ at tunnel axis depth, 72.5$g$ at soil surface and 71.6$g$ at mid-height of the building models. For simplicity, it is referred to as a g-level of 75 within this dissertation. In prototype scale, the model setup resembles a shallow urban tunnelling scenario with a tunnel diameter of 6.15 m and a cover of 8.32 m. The modelled tunnelling scenario is comparable with a railway tunnel project such as the Crossrail project with running tunnels of 6.2 m inner diameter (Mason and Hansraj, 2005). As mentioned above, a low cover-to-diameter ratio was intentionally chosen to account for the increased risk of building damage as the tunnel depth decreases. The buildings were not embedded into the soil (Figure 3.1b). Although this configuration represents
an extreme scenario, it allows a direct comparison with previous experiments carried out by Farrell (2010). Additionally, a rough soil-structure interface (Section 3.3.4) was replicated to prevent slippage between the foundation base and the soil surface.

Figure 3.2 illustrates the final centrifuge package before testing. A main component of the package is the so-called ‘strong box’, which contains the soil model. The front boundary of the strong box is a PMMA window while the base, side walls and the back-wall are made out of steel. Three digital cameras (Canon PowerShot G10) were installed in front of the PMMA window to track ground and structure displacements using digital image correlation (DIC) and the software GeoPIV (White et al., 2003). Section 3.4 details this monitoring technique. Two 24 V<sub>DC</sub> LED light beams were located in front of the PMMA window to provide adequate lightning for the image acquisition. In addition, laser displacement sensors (Baumer Electric OADM 1216430/S35A) and Solartron linear variable differential transformers (LVDTs) monitored surface soil settlements (Figure 3.1c). The used lasers have a measurement distance between 16-26 mm, a resolution between 0.002-0.005 mm and a response time < 900 µs.

The surface settlement instruments (i.e. lasers and LVDTs) enable a comparison to the GeoPIV results and thus provide an adequate degree of redundancy. Furthermore, boundary effects, caused by friction between the PMMA and the soil, as mentioned in Section 3.2.4,
3.3 Experimental setup and equipment

can be quantified by comparing the data of the lasers and LVDTs with the GeoPIV results. This is further addressed in Section 5.3.1. Micro-Electro Mechanical Systems (MEMS) based accelerometers, supplied by ANALOG DEVICES, were installed on the building models to monitor rotation of the building model in the PMMA plane and orthogonal to the PMMA plane. The range of these MEMS varied between $\pm 1.7g$, $\pm 18g$ and $\pm 35g$ depending on the location of the MEMS within the centrifuge gravitational field. The used instruments were calibrated before and after each test.

3.3.2 Model tunnel and tunnel excavation simulation

The tunnelling process is modelled by reducing the tunnel diameter to schematically simulate ground loss (Farrell, 2010; Jacobsz, 2002; Loganathan et al., 2000; Mair et al., 1984; Marshall et al., 2012; Potts, 1976; Taylor and Grant, 1998; Taylor and Yip, 2001; Vorster, 2005). Figure 3.3 illustrates the model tunnel, which consists of a 1 mm thick Latex membrane with a diameter of 70 mm and a brass mandrill with an outer diameter of 60 mm. The membrane is sealed to the circular end pieces of the brass mandrill (diameter of 80 mm) using a wire, and the cavity between the brass cylinder and the membrane is filled with water until a tunnel diameter of 82 mm is obtained. To restrain the position of the model tunnel, the tunnel is fixed to the front and back walls with brass fitting rings. To attempt to ensure uniform settlement
Experimental method and equipment

Fig. 3.3 Model tunnel: (a) cross-section through tunnel centreline, (b) cross-section of model tunnel and (c) image of model tunnel.

along the length of the tunnel, including at the front and back of the soil box, the end of the model tunnel was set within a recess of the PMMA window (Figure 3.3a).

The tunnel diameter is reduced by withdrawing water from the tunnel using a tunnel control system (Figure 3.4). This system was first reported by Jacobsz (2002) and consists of a standpipe, a solenoid valve, a linear actuator that moves a piston of a water-filled sealed cylinder, a pore water pressure transducer (PPT) and 4 mm outer diameter copper pipes to connect these individual parts to the model tunnel. The tasks of the tunnel control system are twofold. Firstly, during the acceleration of the centrifuge the standpipe is connected via the solenoid valve to the model tunnel. The constant water head of the standpipe results in a tunnel pressure that balances the vertical soil stresses at the tunnel axis to minimize soil displacements as
3.3 Experimental setup and equipment

Fig. 3.4 Tunnel control system.

the centrifuge accelerates. Section 5.2 discusses the implications of this technique. Secondly, after reaching 75g, the solenoid valve is closed and the volume of the tunnel is controlled by a piston. Calibration procedures determined that a piston movement of 2.5 mm was required to obtain a tunnel volume loss of 1.0%. During the centrifuge test, the linear actuator was remotely controlled and the piston movement was monitored using a potentiometer to track the tunnel volume loss. With respect to a real tunnelling operation, this modelling technique replicates ground movements caused by shield loss, tail void loss and lining deflection loss. Ground movements caused by face loss are neglected; consequently, this technique is more feasible to simulate closed face tunnelling.

The used tunnelling technique is not able to keep the equilibrium between the earth pressure and the internal tunnel pressure, and therefore should not be applied to study stress-strain conditions close to the tunnel (König, 1998). However, the technique realistically replicates tunnelling-induced soil movements at soil surface and with distance from the tunnel (König, 1998) as was shown by various researchers (Farrell, 2010; Jacobsz, 2002; Loganathan et al., 2000; Marshall et al., 2012; Vorster, 2005).
Experimental method and equipment

Table 3.2 Leighton Buzzard fraction E silica sand properties (Tan, 1990)

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$d_{10}$ grain size (mm)</td>
<td>0.095</td>
</tr>
<tr>
<td>$d_{50}$ grain size (mm)</td>
<td>0.14</td>
</tr>
<tr>
<td>$d_{60}$ grain size (mm)</td>
<td>0.15</td>
</tr>
<tr>
<td>minimum voids ratio, $e_{\text{min}}$</td>
<td>0.65</td>
</tr>
<tr>
<td>maximum voids ratio, $e_{\text{max}}$</td>
<td>1.01</td>
</tr>
<tr>
<td>specific gravity, $G_s$</td>
<td>2.67</td>
</tr>
<tr>
<td>critical state friction angle, $\phi_{\text{crit}} (^\circ)$</td>
<td>32</td>
</tr>
</tbody>
</table>

### 3.3.3 Soil model and preparation

For all centrifuge tests, dry Leighton Buzzard Fraction E silica sand (Table 3.2) was poured to a nominal relative density, $I_D$, of 90% using an automatic sand pourer (Madabhushi et al., 2006). A sand pouring setup similar to Zhao et al. (2006) with a multiple sieve arrangement of two sieves with an aperture size of 0.85 mm and a nozzle with a diameter of 4.9 mm was used. Additionally, a fixed drop height of 650 mm and a pass every 15 mm was chosen. This procedure resulted in an average $I_D$ of 90.6% ($\pm 3\%$) and an average sand density of about 1,605 kg/m$^3$. To obtain a uniform soil density throughout the model, and in particular surrounding the model tunnel, the sand was poured with the model lying on the PMMA window, shown in Figure 3.5, and the model tunnel positioned in the recess of the PMMA window. A paper sleeve was placed around the tunnel to avoid bulging of the Latex lining due to gravitational forces. This sleeve was incrementally lifted following the sand pouring progress. This technique was applied by various researchers (e.g. Vorster, 2005; Marshall, 2009; Farrell, 2010; Williamson, 2014) and has the advantage that a soil model with homogeneous properties around the tunnel can be poured. Figure 3.5 shows that a plywood was placed parallel to the bottom of the strong box in order to achieve a levelled soil surface and a defined model height. Additionally, metal sheets were mounted along the borders of the sand pouring area to avoid that sand flows from the centrifuge package boundaries into the soil model. After the sand pouring was finished, the backplate was mounted on the strong box and the entire package was raised into a vertical position. Then the plywood was lifted and surface structures and measuring devices were placed. The employed sand type, the very dense soil models and the pouring technique employed are in line with Farrell (2010); Jacobsz (2002); Marshall et al. (2012) and Vorster (2005). The reason for using a consistent very dense soil model is that the degree of building damage increases with soil relative density (Netzel, 2009; Son, 2015).
3.3 Experimental setup and equipment

Model tunnel protected with paper sleeve
Plywood (pouring shield)
PMMA
Metal sheets (barriers)

Fig. 3.5 Sand pouring arrangement.

3.3.4 Building model

This section provides a brief introduction of the complex surface structures used within this research, while Chapter 4 will solely focus on the design, the 3D printing (3DP) procedure and the material properties of the building model. The building models were 3D printed to create structural models with representative building layout and characteristics, as shown in Figure 3.6. The main advantage of employing 3DP was that building features such as partitioning walls, strip footings and window openings could be replicated at a scale of 1:75. Additionally, the 3D printed material exhibits brittle material behaviour and thus is capable of cracking.

An additional surcharge in the form of dead load bars (Figure 3.6a) was placed on top of the structural model to obtain a soil–structure stress of 100 kPa beneath the footings perpendicular
Experimental method and equipment

To the tunnel. This value of 100 kPa represents an upper value for a two-storey Georgian house including the load of a roof, floors, outer and inner walls, foundations and the furniture. The mass of the dead load bars was calculated by assuming that the load of the front and back façades, the footings perpendicular to the tunnel and the partitioning walls is transferred to the footings perpendicular to the tunnel. By contrast, the load of the end walls and the foundations parallel to the tunnel is assumed to be fully transferred to the foundations parallel to the tunnel. To capture the building displacements with DIC, the front façade, which was in contact with the PMMA window, was coloured with an artificial texture as can be seen in Figure 3.6a.

Figure 3.6 indicates that a rough soil-structure interface, which is typical for historic masonry buildings founded on brickwork, was replicated by printing an uneven foundation base. The geometrical details of this rough interface are shown in Figure 3.7a. To further assess the surface roughness of the interface, a Taylor Hobson Form Talysurf i120 profilometer was used to measure the roughness of the foundation base (Figure 3.7b). The normalised roughness parameter, $R_n$, defined by Uesugi and Kishida (1986) as the ratio between the maximum surface asperities, $R_{max}$, measured at a gauge length of $d_{50}$, and $d_{50}$ was subsequently derived. For the foundation base, an average $R_{max}$ of 65 µm was calculated which results in $R_n = 463 \cdot 10^{-3}$. The obtained $R_n$ value is significantly above the critical surface roughness ($R_n = 75 \cdot 10^{-3}$) identified by Uesugi and Kishida (1986). At the critical surface roughness, the interface shear resistance is equal to the internal soil shear strength and shear failure develops within the soil body. This behaviour is typical for a rough interface for which sand to foundation sliding (i.e. slippage) is prevented which, as observed in the centrifuge tests (Section 6.4.2).

3.4 Digital image correlation

Image-based displacement monitoring provides the main measurement technique of this research. More specifically, digital image correlation (DIC) using the GeoPIV software (White et al., 2003) was employed to track soil and building movements. The main principle of PIV is to measure displacement fields from images of digital cameras. To capture the entire soil movements caused by the tunnel excavation, the centrifuge package was equipped with three Canon Powershot G10 digital cameras, each with a 14.7 megapixel sensor as shown in Figure 3.2. The middle camera was mounted in portrait orientation while the two other cameras took images in landscape orientation. The fields of view of adjacent cameras overlapped in order to obtain an overall coordinate system. In addition, PTFE blocks were used to support the lenses during the tests and to limit lens distortion.
Control markers painted on the PMMA window were used to calibrate the soil movements from image-space (in pixels) to object-space (in millimetres). This calibration accounts for rigid body movements, slightly altered camera positions and lens distortions occurring during the experiment. To obtain the centre of these control markers a technique of multiple-threshold centroiding was used. Based on White et al. (2003) this procedure results in an accuracy of 0.1 pixel when estimating the centre of the control markers. The entire error of the PIV is slightly higher because of additional errors when tracking the soil movements but should be in close agreement with the results of the control marker centroiding (White et al., 2003).

Photogrammetric targets (e.g. mylar card) with dots in a regular grid were used to calibrate the control markers. The photogrammetric targets were placed flush against the PMMA window and images were taken. Based on these images, the relative position of the control markers within the photogrammetric target can be found. This process is challenging because the entire control markers do not fit within the dots of the mylar card. Thus, the position of the mylar card has to be changed various times but at least two overlapping control markers have to be provided between adjacent images to compute an overall coordinate system for each camera. Finally, the coordinate systems of the three cameras can be stitched together and the overall
coordinate system in object space is defined. More detailed information about this procedure is provided in Take (2003).

After determining the coordinates of the control markers in object-space, the soil movement data can be computed by tracking soil patches from one picture to the other. Soil and structure patches of 32 x 32 pixels were used throughout this research and provided reliable outcome. Each patch is defined by a matrix of brightness intensity. The PIV process basically searches for the same brightness values between a pair of two images and thus calculates the displacement of a soil patch between these two images (Figure 3.8a). This tracking between two images is carried out by using a correlation function which results in a correlation surface across the defined search patch of a single patch. The peak of this correlation surface gives the displacement vector in pixel accuracy, shown in Figure 3.8b. A further increase in accuracy is provided by fitting a curve to the peak using a bicubic interpolation (Figure 3.8c). The peak of this curve defines the final displacement of the soil patch in sub-pixel precision. Further details of this correlation procedure are provided by White et al. (2003).

3.5 Data acquisition

The experimental data was acquired using the data acquisition software DASYLab. To collect the data of the instruments, each instrument is connected to a junction box, which converts the analogue signal to a digital signal and passes the data to an on-board computer on the beam centrifuge. The junction box generally supplies the instruments with excitation voltage (5 V or 10 V) and, where necessary, amplifies the output signal of the instrument with a gain value of 1, 10, or 1000 to reduce the effect of electrical noise. However, the lasers need an alternative power supply of 12 V and therefore a bespoke power box was mounted on the centrifuge package and connected to a 12 V power supply on the beam of the centrifuge. During the centrifuge tests the DASYLab software is remotely accessed from the control room. This remote access allows one to real-time monitor the signal of the instruments during the test. In addition, the data acquisition setup can be adjusted throughout the test. The data of the entire instruments was acquired at 100 Hz.

To control the three digital cameras throughout the experiments, the cameras were connected to a centrifuge computer and the software PSRemote (Breeze Systems) was used. A time-lapse setup was created and adjusted to the tunnel excavation simulation procedure in order to acquire images at defined stages of the experiment. The acquired images were saved to the centrifuge computer.
3.5 Data acquisition

(a) Soil patches and search zone.

(b) Correlation function in pixel accuracy.

(c) Subpixel-accuracy using cubic fit.

Fig. 3.8 Tracking of soil movements using PIV (after Take, 2003).
3.6 Testing procedure

Prior to running the centrifuge test, the 3D printed structure was placed on top of the soil model and the lasers and LVDTs were installed. Throughout this process, the solenoid valve (Figure 3.4) remained closed. After the centrifuge package was loaded onto the centrifuge, the instruments were connected to the data acquisition system and tested, the PMMA window was carefully cleaned and a standard testing procedure was followed:

- Spin-up was carried out in 10g increments up to 70g, plus a final 5g increment. At about 6g the solenoid valve was opened and the pressure in the tunnel was controlled by the water head in the standpipe (Figure 3.4). At this g-level the vertical soil stresses at tunnel depth approximately match the initial water pressure within the tunnel, which is the result of stretching the tunnel membrane to a diameter of 82 mm. After reaching a spin-up increment, the centrifuge acceleration was paused and images were captured. At 75g the solenoid valve was closed to connect the model tunnel to the piston (Figure 3.4).

- The tunnel excavation process was then simulated. Digital photos were taken at every 0.1% of tunnel volume loss and instrument readings were taken at 100 Hz. Tunnel excavation simulation proceeded until the volume of the tunnel was reduced by 26% of its initial volume or the building collapsed.

- The centrifuge was spun down and final images were acquired.

3.7 Overview of centrifuge test series

Throughout this research 14 centrifuge tests were carried out. Tests 1-4 focused on the design of a low cost laser scanner device to measure any potential change in the soil surface settlement trough along the length of the tunnel. The remaining tests were concerned with the interaction of complex 3D printed building models and tunnelling-induced ground movements. Some issues occurred throughout the testing programme, which were mainly related to initial uncertainties related to the 3DP procedure and building design. In two cases, these issues caused building collapse during the spin-up of the centrifuge.

Figure 3.9 provides an overview of the centrifuge model tests reported in this thesis that focus on tunnelling effects on complex building models. As mentioned above, the aim of this testing program was to investigate the influence of building characteristics on this tunnel–soil–structure interaction problem. Because of this, the tunnelling conditions (i.e. tunnel dimen-
3.8 Summary

This chapter has presented the experimental techniques and equipment used to study the response of surface structures to tunnelling subsidence. The modelled tunnelling scenario represents a shallow urban tunnelling site in dense dry sand and the affected building models are two-storey structures of different building geometry and features. The tunnel volume loss was simulated by extracting water from a tunnel model to reduce the tunnel diameter in a controlled manner. Details of the used soil type and the soil model preparation are provided while the building models are briefly introduced. The following chapter discusses the complex 3D printed building models in further detail. Digital cameras enabled to monitor the soil and building displacements throughout the centrifuge model tests using DIC. An overview of the performed centrifuge test series is presented in Table 3.3 and depicted in Figure 3.9.

Table 3.3 Details of the test series.

<table>
<thead>
<tr>
<th>Test</th>
<th>Model scale</th>
<th>Dimensionless groups</th>
<th>Test Model scale</th>
<th>Dimensionless groups</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>e (mm)</td>
<td>L (mm)</td>
<td>H (mm)</td>
<td>O (%)</td>
</tr>
<tr>
<td>A</td>
<td>0</td>
<td>200</td>
<td>90</td>
<td>20</td>
</tr>
<tr>
<td>B</td>
<td>160</td>
<td>200</td>
<td>90</td>
<td>20</td>
</tr>
<tr>
<td>C</td>
<td>100</td>
<td>200</td>
<td>90</td>
<td>20</td>
</tr>
<tr>
<td>D</td>
<td>160</td>
<td>200</td>
<td>90</td>
<td>40</td>
</tr>
<tr>
<td>E</td>
<td>130</td>
<td>260</td>
<td>90</td>
<td>20</td>
</tr>
<tr>
<td>F</td>
<td>130</td>
<td>260</td>
<td>90</td>
<td>40</td>
</tr>
<tr>
<td>G</td>
<td>130</td>
<td>260</td>
<td>90</td>
<td>40</td>
</tr>
</tbody>
</table>
Fig. 3.9 Centrifuge test series with varying building length, $L$, building eccentricity, $e$, façade openings, $O$, and building layout, $G$. 

---

(a) $A$: $e/L=0$, $L/H=2.2$, $O=20\%$, $G=3D$  
(b) $B$: $e/L=0.8$, $L/H=2.2$, $O=20\%$, $G=3D$  
(c) $C$: $e/L=0.5$, $L/H=2.2$, $O=20\%$, $G=3D$  
(d) $D$: $e/L=0.8$, $L/H=2.2$, $O=40\%$, $G=3D$  
(e) $E$: $e/L=0.5$, $L/H=2.9$, $O=20\%$, $G=3D$  
(f) $F$: $e/L=0.5$, $L/H=2.9$, $O=40\%$, $G=3D$  
(g) $G$: $e/L=0.5$, $L/H=2.9$, $O=40\%$, $G=2D$
Chapter 5 focuses on boundary effects observed throughout this research, while the results of the conducted centrifuge model tests are presented in Chapters 6 and 7.
Chapter 4

3D printed building models

Centrifuge modelling has proven useful for studying soil–structure interaction effects. However, due to space limitations within geotechnical centrifuges, large scale factors are necessary to study typical soil–structure interaction problems. Replicating every detail of a prototype at this small scale is not possible. This is particularly true for masonry buildings which are made of bricks and mortar, which typically have anisotropic and inhomogeneous material properties, and which inherently contain numerous imperfections and uncertainties. Nevertheless, 3D printing (3DP) does allow rapid creation of detailed building models which capture important building characteristics (e.g. layout, openings, foundations, etc.) that affect the overall structural performance.

In this chapter, the use of powder based 3DP to replicate masonry structures for centrifuge modelling is discussed. Four-point-bending tests were carried out on 3D printed beams to determine the mechanical properties of the 3D printed material including elastic modulus, strength and strain to failure. Results of previous research reveal a variation in material properties with position and orientation of the 3D printed object in the print bed which is caused by the printing procedure. After restricting the position of the model in the print bed, repeatable material properties with lower stiffness and higher strength than typical masonry were observed. However, building layout and window opening percentage could be adjusted to create building models with overall bending and axial stiffness representative of field data. These improved 3D printed scale models were subsequently used in centrifuge tests exploring the response of surface structures to tunnelling-induced settlements. Results of these experiments, discussed in the Chapters 6 and 7, replicate typical masonry building damage in the form of cracking, and indicate that powder based 3D printed masonry structures are useful in modelling soil–structure interaction problems in centrifuge models.
4.1 Background

In the last 20 years, centrifuge modelling research has been performed to study the response of building models to tunnelling works (Caporaletti et al., 2005; Farrell, 2010; Taylor and Grant, 1998; Taylor and Yip, 2001). Although these investigations disentangled important mechanisms governing this soil–structure interaction problem, much of the research has been limited by simple structural models. This limitation is mainly caused by the necessity to employ large scale factors, often between 1:20 and 1:100, when studying soil–structure interaction phenomena in a geotechnical centrifuge (Knappett et al., 2011).

Recent developments in rapid prototyping (RP) technologies have opened the door to an array of applications in civil engineering research (DeJong and Vibert, 2012; Feng et al., 2015; Liang et al., 2014, 2015). Specifically, detailed small-scale models that replicate structurally important features of the prototype can be fabricated. This is crucial when studying the response of surface structures to tunnelling-induced ground movements.

Appropriate mechanical properties of the material used to replicate a prototype structure are vital to realistically model the deformation and strength behaviour of structures in contact with the soil (Knappett et al., 2011). Current guidelines to assess building response to tunnel excavation are based on relations between the soil and structure stiffness. Consequently, the main objective of this research is to realistically model the global stiffness of surface structures subject to tunnelling-induced ground movement. In the light of the latest generations of tunnel boring machines, which often induce very small soil displacements and distortions of nearby structures, the initial building stiffness is a key parameter governing this tunnell–soil–structure interaction problem. Nevertheless, as pointed out by Knappett et al. (2011) the scaling of strength properties is crucial when studying collapse mechanisms of soil–structure interaction phenomena. This is particularly relevant for historic masonry structures where cracking could occur at small differential displacements.

This chapter discusses the 3DP of 2-storey surface structures on shallow foundations at a scale of 1:75, which are subsequently exposed to tunnelling-induced ground displacements at 75g. The aim of these 3D printed structures is to model realistic building characteristics including front, rear, end and intermediate walls, window openings and strip foundations while realistically modelling the axial, $E_A$, and bending stiffness, $E_I$, of the building. Therefore, the used approach carefully balances the Young’s modulus, the moment of inertia and the area of the reduced scale model. In the following, the used 3DP technique is first introduced, after which the preparation of the 3D printed building models for the centrifuge model tests is described. Then, the material properties of the 3D printed material are discussed and ini-
4.2 3D printing technique

The 3D printed surface structures were printed on a Z Corporation Zprinter350 using Visijet PXL Core powder and VisiJet PXL Clear binder. The used 3D printer is owned by the Department of Architecture at the University of Cambridge. This rapid prototyping technology can be classified as a Three-Dimensional Printing (3DP) technique using a cementitious powder (Feng et al., 2015). Figure 4.1 depicts the main steps involved in the 3DP process while Figure 4.2 provides an overview of the 3D printer and visualises the fabrication of the 3D printed building models.

Before printing, shown in Figure 4.1a, the feed bin contains the used powder. When printing starts, the so-called roller, which is mounted with the print head (i.e. nozzle) on the gantry, feeds the build pin with powder. In a first step, a base layer is created on which the 3D solid will subsequently be created. The nozzle prints droplets of the binder fluid at predefined positions of the powder bed, as can be seen in Figure 4.1b. The binder triggers a reaction with the powder, which locally solidifies predefined parts of the powder. This procedure is repeated for every layer until the entire 3D object is fabricated. After a layer is finished, the feed bin is raised while the build bin is lowered by the same distance. From Figure 4.1c it is clear that the 3D printed object is surrounded by loose powder. To extract the 3D solid from the build bin, a vacuum hose is used to hoover the remaining loose powder (Figures 4.2a and 4.2c). This has to be carried out with care because the strength of the 3D printed material develops with time and parts are prone to break. The remaining powder is removed in the fine powder removal chamber, shown in Figure 4.2d, using the so-called air wand (Figure 4.2a).

4.2.1 3D printing process

Within this research, the following 3DP process was adopted:

1. The 3D printed building and model and specimen were first created using computer aided design (CAD) software (i.e. AutoCAD 2015).

2. The CAD model was then saved as a standard triangulation language (STL) file, which converts the solid sections of the CAD model into numerous thin digital layers. For the entire print jobs a layer thickness of 0.0889 mm was used.
Fig. 4.1 Overview of the 3DP procedure: (a) before printing, (b) throughout printing process and (c) finalised print job (after Feng et al., 2015).
4.2 3D printing technique

Fig. 4.2 Overview of 3D printer and main steps of creating the 3D printed building models.
3D printed building models

3. The printing process was started by transmitting the STL file to the 3D printer and subsequently the 3D printer prints each layer atop another, as shown in Figure 4.2b.

4. After the printing process was finished and sufficient initial curing time was allowed, the printed components were removed from the powder bed (Figure 4.2c).

5. An air nozzle was used to remove the remaining powder from the printed parts (Figure 4.2d).

For this research the 3D printed building models had to be printed in parts; the process is described below. Each part of the building model required about 7 hours of printing. Additional time was necessary for initial curing (about 3 hours for the 3D printed objects created herein) and removal of the powder (about 1.5 hours). Depending on the different building configurations used, the time to 3D print an entire structural model was between 3 to 4 days.

4.2.2 Coordinate system

Previous research (Asadi-Eydivand et al., 2016; Farzadi et al., 2015; Feng et al., 2015; Gharai et al., 2013) identified that the printing direction affects the material properties of the 3D printed material. To study these orthotropic material characteristics the coordinate system defined by Feng et al. (2015) and depicted in Figure 4.3 is adopted. The $X$ axis is the direction in which the nozzle moves when it it drops binder in the build chamber. The $Y$ axis is perpendicular to $X$ and both the $X$ and $Y$ axis define the nozzle path for one layer of the 3D printed object. The $Z$ axis is in the vertical direction perpendicular to the layers of the 3D printed structure and the feed and build bins (Figure 4.1) move in the $Z$ direction.

4.2.3 Material composition

The used powder and binder was the Visijet PXL Core powder and the Visijet PXL Clear binder supplied by 3D Systems. Based on safety data sheets (3D Systems, 2013) the main component (80-90%) of the powder is calcium sulphate hemihydrate ($\text{CaSO}_4 \frac{1}{2}\text{H}_2\text{O}$), which is also called plaster of Paris (Butscher et al., 2011). The remaining components are not specified but the previous generation of powder (Zp150; Z Corporation, 2009) consisted of vinyl polymer (<20%) and carbohydrate (<10%). Data sheets for the binder indicate that the binder is a mixture of primarily water and a humectant (0-1% 2-pyrrolidone, $\text{C}_4\text{H}_7\text{NO}$). Properties of the binder are very similar to water as was identified by Asadi-Eydivand et al. (2016). From the components of the powder and the binder it can be followed that the binder
dissolves the calcium sulphate cements and the polymer to form a solid structure while the carbohydrate acts as a filler. The main binder/powder setting reaction can be written as

\[
CaSO_4 \frac{1}{2}H_2O + \frac{1}{2}H_2O \rightarrow CaSO_42H_2O
\]  

where the calcium sulphate hemihydrate reacts with water to form gypsum (CaSO$_4$ 2H$_2$O, calcium sulphate dihydrate). The polymer reaction remains a company secret of 3D Systems. However, an X-ray diffraction phase analysis performed by Asadi-Eydivand et al. (2016) identified that the zp150 powder before and after printing consisted of CaSO$_4 \frac{1}{2}$H$_2$O and CaSO$_4$ 2H$_2$O, respectively. This suggests that the main reaction causing the solid 3D printed object can be attributed to the hydration of calcium sulphate hemihydrate leading to the crystallization of gypsum.

### 4.2.4 Microstructure effects

The 3D printed material is characterised by a distinct orthotropic behaviour that is related to the orientation of the structure in the print bed (Asadi-Eydivand et al., 2016; Chan, 2012; Farzadi et al., 2015; Feng et al., 2015; Gharai et al., 2013). This observation can be related to the previously described printing procedure that results in a characteristic layered microstructure of the 3D printed objects. Figure 4.4 shows a part of the building models 3D printed for this research and employed to point out the 3D printed microstructure. The following observations were made:
In XZ plane distinct layers can be identified by the many parallel lines which represent the vertical layers of the 3DP procedure.

In XY plane further layers are visible which is caused by the nozzle pattern printing distinct strips in X direction (Figure 4.3). Interestingly, these patterns were not observed at all XY planes which is likely related to the position of the XY plane with respect to the layers in Z direction. Specifically, XY surfaces facing towards the nozzle were very smooth.

The YZ plane was not characterised by a layered structure and had a similar roughness than the XZ surface.

These observations imply that the print orientation has a significant effect on the 3D printed material properties. Chan (2012, 2013) studied the mechanical properties of specimen printed with the equivalent printer used for this work and a previous generation of powder and binder. This previous work shows that the 3D printed material is weakest when loaded in the XY plane. Equivalent findings were reported by Feng et al. (2015) when testing 3D printed specimen in tension and bending. They related their observations to the lower strength between layers (in Z direction) compared to strips (in X direction). For the 3D printed material, Chan
4.3 Building models and specimen

(2013) reported a bending-to-shear-stiffness ratio, $E/G$, of 2.9 which is in fair agreement with typically adopted values for masonry (e.g. 2.6 following Burland and Wroth, 1974). Moreover, Chan (2012, 2013) identified certain areas within the print bed that result in consistent material properties of the 3D printed material. These findings of previous researchers were considered when designing the building models.

**4.3 Building models and specimen**

To make use of the lower interlayer bond strength pointed out above, the building model was printed so that the façade walls are perpendicular to the XY plane of the 3D printer, as shown in Figure 4.5. The dashed line surrounding the building model indicates the area of consistent material properties identified by Chan (2012, 2013). Due to the size of the print space (250 x 200 x 150 mm) and the required orientation in the print bed, the printing models were printed in two or three parts for the structures with $L = 200$ mm and 260 mm, respectively, and subsequently glued together using Araldite standard (Figure 4.6). In every print job, specimens were also fabricated as shown in Figure 4.5 and subsequently tested to derive the 3D printed material properties.

Figure 4.6 indicates the main steps carried out to finalise the building models after 3DP, which included: (i) connecting the individually printed building parts with Araldite standard glue, (ii) colouring the front façade of the building window with a so-called speckle pattern which enabled to track building displacements with DIC, (iii) attaching brass dead load bars to the top of the front and rear façades of the building model to replicate a vertical stress of
3D printed building models

(a) Connecting 3D printed building parts and creating speckle pattern. (b) Attaching dead load bars, black window sheets and MEMS accelerometers.

Fig. 4.6 Building model preparation after 3D printing.

100 kPa beneath the strip footings of the front and rear façades, (iv) attaching a black sheet of paper to the back of the front façade and (v) installing MEMS accelerometers on the structure models. From Figure 4.6a it can be seen that g-clamps were used while connecting the individually printed building parts. Furthermore, Figure 4.6a indicates that the speckle pattern is a result of splattering standard enamel colour paint using a toothbrush until a sufficiently irregular texture was obtained. The black window sheets, shown in Figure 4.6b, were attached along the back of the front façade to obtain black windows which was found advantageous when subsequently applying GeoPIV to track building displacements.

A complete building model is shown in Figure 4.7a, while Appendix A provides the detailed building dimensions. Figure 4.7 shows that the 3DP procedure enabled to obtain complex building models with realistic building layout (e.g. front, rear, end and intermediate walls) and building features such as strip foundations, window openings and a rough-soil structure interface by printing an uneven foundation base. As stated above, replicating these structural details at 1/75\textsuperscript{th} scale was the primary aim of adopting the 3DP technique. The dead load bars and the speckle pattern can also be seen in Figure 4.7a. Figure 4.7b depicts a typical cross-section of the building models with a 3D building layout indicating the strip footings, a solid cross-section of the front and rear façades (i.e. no windows) and the view of the intermediate wall. From this cross-section it can be seen that the intermediate walls were not in contact with the soil surface.

For test G an isolated façade was modelled which was identical to the front façade of test F. To achieve a vertical stress of 100 kPa beneath the isolated façade of test G, the dead load bar application had to be modified compared to the tests with a 3D building layout (i.e. front and
4.4 Mechanical tests

Fig. 4.7 Building model: (a) structural details and (b) cross-section (dimensions in mm).

rear façades). Figure 4.8 indicates that an extension of the back steel wall of the strong box was designed to rest the rear end of the dead load bars. The surcharges were placed on top of a foam layer to allow movements of the building model during spin-up and tunnel excavation. Moreover, a pin was attached to the steel wall extension and boreholes of significantly greater diameter compared to the pins were drilled close to the rear end of the dead load beams. The dead load bars were placed on the steel support so that the pins were inserted in the oversized holes. This prevented these bars from sliding too much during spin-up and tunnel excavation. Also the connection of the front end of the dead load bars to the building model was modified by placing a foam layer between the top of the façade and each beam (Figure 4.8b). This design ensured that the dead load bars remained in full contact with the façade while not constraining the building movements. Furthermore, throughout the entire experiment the dead load bars ensured that the front face of the façade remained in contact with the PMMA window plane (see Section 5.3.2).

4.4 Mechanical tests

Four-point-bending tests were conducted to determine the material properties of the 3D printed material including the elastic modulus, strength and ultimate strain to failure. Therefore, as mentioned above, specimens were printed in every print job. In the following, the specimens, the test procedure and the obtained results are described.
3D printed building models

(a) Support system of dead load bars. (b) Connection of dead load bars to façade.

Fig. 4.8 Application of dead load bars for test G.

Fig. 4.9 3D printed specimen. Dimensions in mm.

4.4.1 Specimen

The material properties were determined using 20 mm x 4 mm x 125 mm specimens, as shown in Figure 4.9. In every print job carried out for the 3D printed building models, two specimens were printed. Consequently, four and six specimens were available for the building models with $L = 200$ mm and $L = 260$ mm, respectively. The isolated façade of test G was printed in two print jobs and therefore four specimen were created for test G.

4.4.2 Test procedure

The four-point-bending test followed the test procedure described in ASTM D 790M-86 II, Procedure A (ASTM Standards, 1986). This specific test method was chosen because the entire portion of the specimen between the load spans experiences the maximum stress, which is essential when studying brittle materials for which weak parts are directly related to crack initiation and material strength. The Instron Frame 2 of the Structures research group of the Department of Engineering was used for the entire material tests. Figure 4.10 shows the test setup. The specimens were tested flatwise with a support span of 99 mm and a load span of
33 mm as depicted in Figure 4.10a. The position of the specimen in the material tests was defined so that the load was applied in the XY plane to test the weak interlayer bond of the 3D printed material. A cross-head motion of 4.5 mm/min was applied and a laser extensometer supplied by Electronic Instrument Research was used to monitor the mid-span deflection of the samples (Figure 4.10b). To avoid any influence of the laser measurements due to reflection of steel parts, the surfaces were painted black or black tape was used as can be seen in Figure 4.10b. The applied load was monitored with a 1 kN load cell. At 4 N, which is in good agreement with the measured average peak load (i.e. 4.32 N), this setup results in a load reading that is accurate to 0.01 N. Significant noise was not observed in the load cell and laser extensometer data. After a load threshold of 2 N was reached three unloading and loading cycles between 2 N and 1 N were performed using the identical cross-head motion rate of 4.5 mm/min. The test data was acquired using a sampling frequency of 10 Hz.
4.4.3 Results and analysis

According to ASTM D 790M-86 II, Procedure A (ASTM Standards, 1986) the mechanical properties of the 3D printed material were derived. In addition, the density of the 3D printed material was estimated by weighing each specimen and determining the volume of each specimen by measuring each dimension at three different positions along the Z direction of the specimen (Figure 4.9).

The peak loads, \( P \), from the four-point bending tests were used to derive the flexural strength, \( f_t \), of the 3D printed material, which can be expressed as

\[
f_t = \frac{P \cdot L}{b \cdot d^2}
\]  

(4.2)

where \( L \) is the support span in mm, \( b \) the width of the specimen in mm and \( d \) the depth (height) of the specimen in mm. The 3D printed material is more vulnerable in tension than compression. Consequently, the flexural strength represents the highest tensile stress experienced by the extreme fibre of the bent specimen. In contrast to a tension test, only the extreme fibre experiences the maximum stress value. Hence, the flexural strength is commonly greater than the tensile strength of the same material. The Young’s modulus of the 3D printed material, \( E_{3DP} \), was derived by

\[
E_{3DP} = \frac{0.21 \cdot L^3 \cdot m}{b \cdot d^3}
\]  

(4.3)

where \( m \) is the slope of the tangent to the initial straight-line portion of the load-deflection curve in N/mm. This was generally found to be between a load of 0.5 N and 1.5 N but for certain specimens manual adjustment was carried out. The midspan deflection, \( D \), in mm, which was obtained with the laser measurement device was used to obtain the ultimate strain to failure, \( \varepsilon_{ult} \), of the 3D printed material, and can be written as

\[
\varepsilon_{ult} = \frac{4.36 \cdot D \cdot d}{L^2}.
\]  

(4.4)

Figure 4.11 shows the stress-strain curves for the 3D printed samples of the entire centrifuge test series while Table 4.1 presents the corresponding material properties. It is evident from Figure 4.11 that the 3D printed material exhibits a softening behaviour typical of brittle materials. This implies that the 3D printed material can be used to experimentally model cracking damage. Figure 4.10b shows a specimen during the flexural test after visible cracking had occurred. The crack initiated at the bottom of the specimen and then propagated vertically.
through the whole section. The location of the cracks varied but was generally between the central supports.

The stress-strain curves of the samples associated with a single centrifuge test showed similar initial response (Figure 4.11). This indicates a consistent Young’s modulus value. Differences were observed for the tests C, D and E, which is further discussed below. By contrast, the samples typically fractured at considerably different strain values (apart from test F). This is likely related to 3DP defects causing a weaker bond between layers of the 3D printed material.

Notable differences in the 3D printed material properties between different centrifuge tests are evident in the stress-strain curves shown in Figure 4.11. This is particularly true for the tests C, D and E for which the lowest Young’s moduli and flexural strength values were derived (Table 4.1). A substantial increase of these mechanical properties of the 3D printed material is apparent for the tests F and G. Before printing these tests the 3D printer was supplied with new powder and binder. It is therefore likely that either the powder or the binder deteriorated throughout the 3DP period of this research (February 2015 to January 2016).

The determined material properties of the 3D printed material with respect to the centrifuge test series are summarised in Table 4.1. Additionally, a wide range of typical material properties of masonry, which were historically obtained by a variety of different methods, are presented for reference. Consequently, these data provide reference points to qualitatively compare the material properties of the 3D printed material and masonry. The 3D printed material has a lower density than masonry while the Young’s modulus values are in the range of historic masonry. The derived flexural strength of the 3D printed material and the ultimate strain to failure are higher than typical values for masonry. As a result of the greater $\epsilon_{ult}$, cracking is expected to initiate at greater building distortions than in real structures. However, the brittle material behaviour enables the study of cracking patterns induced in the building models.

In addition to the material tests summarised in Table 4.1, an investigation into the ageing of the 3D printed material was carried out. Four-point-bending tests were performed at 2, 3, 5, 7, 10, 20, 35, 65 and 100 days after 3D printing. A significant change of the material properties with time could not be observed.

To replicate typical global building stiffness values observed in the field, the building layout and façade openings were carefully adjusted. Before discussing the global stiffness values of the building models, the next section presents an investigation into the influence of different curing temperatures on the 3D printed material properties.
Fig. 4.11 Stress-strain curve of the 3D printed material.
Table 4.1 3D printed material properties compared to typical masonry properties from Giardina et al. (2015c). The variability in the material properties is measured by the standard deviation (SD).

<table>
<thead>
<tr>
<th>Test</th>
<th>Specimen</th>
<th>Density (kg/m³)</th>
<th>Flexural strength (MPa)</th>
<th>Young's modulus (MPa)</th>
<th>Ultimate strain (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>value</td>
<td>mean SD</td>
<td>value mean SD</td>
<td>value mean SD</td>
</tr>
<tr>
<td>A</td>
<td>a</td>
<td>1.294</td>
<td>1.293 1.076</td>
<td>1.502 1.362 0.100</td>
<td>913.3 893.1 49.1</td>
</tr>
<tr>
<td></td>
<td>b</td>
<td>1.293</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>c</td>
<td>1.293</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>d</td>
<td>1.292</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>1.278</td>
<td>3.273 0.738</td>
<td>1.330 1.311 0.073</td>
<td>815.5 800.6 50.5</td>
</tr>
<tr>
<td></td>
<td>b</td>
<td>1.275</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>c</td>
<td>1.279</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>d</td>
<td>1.283</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>1.281</td>
<td>2.61 24.81</td>
<td>1.238 1.130 0.161</td>
<td>798.7 727.4 154.5</td>
</tr>
<tr>
<td></td>
<td>a</td>
<td>1.281</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>b</td>
<td>1.284</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>c</td>
<td>1.246</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>d</td>
<td>1.235</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>D</td>
<td>1.258</td>
<td>1.272 10.87</td>
<td>0.875 0.934 0.051</td>
<td>460.2 515.9 39.6</td>
</tr>
<tr>
<td></td>
<td>a</td>
<td>1.270</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>b</td>
<td>1.283</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>c</td>
<td>1.286</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>d</td>
<td>1.278</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>E</td>
<td>1.318</td>
<td>1.280 34.35</td>
<td>1.311 1.139 0.216</td>
<td>847.0 689.9 194.8</td>
</tr>
<tr>
<td></td>
<td>a</td>
<td>1.314</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>b</td>
<td>1.314</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>c</td>
<td>1.286</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>d</td>
<td>1.284</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>f</td>
<td>1.244</td>
<td>0.921</td>
<td>921</td>
<td>494.4 0.379</td>
</tr>
<tr>
<td></td>
<td>e</td>
<td>1.236</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>F</td>
<td>1.245</td>
<td>1.247 9.036</td>
<td>1.721 1.702 0.219</td>
<td>1.123 1.039 145.2</td>
</tr>
<tr>
<td></td>
<td>a</td>
<td>1.252</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>b</td>
<td>1.234</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>c</td>
<td>1.239</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>d</td>
<td>1.258</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>G</td>
<td>1.219</td>
<td>1.210 14.34</td>
<td>0.905 1.221 0.229</td>
<td>649.8 737.2 67.5</td>
</tr>
<tr>
<td></td>
<td>a</td>
<td>1.224</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>b</td>
<td>1.208</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>c</td>
<td>1.191</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>d</td>
<td>1.191</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Average</td>
<td>1.263</td>
<td>30.29 1.277</td>
<td>0.288 783.5</td>
<td>196.9 0.293 0.060</td>
</tr>
<tr>
<td></td>
<td>Masonry</td>
<td>1.900</td>
<td>0.1 - 0.9</td>
<td>1.000 - 9.000</td>
<td>0.038 - 0.06*</td>
</tr>
</tbody>
</table>

*Strain at onset of cracking for brick walls (Burland and Wroth, 1974).
4.4.4 Effect of different curing temperature

The mechanical properties of the 3D printed material revealed that the 3D printed material is stronger and less stiff than typical masonry. The influence of the temperature during curing of the 3D printed objects was investigated to provide further guidance for future research aiming to replicate more realistic masonry properties. Therefore, specimens identical to the ones shown in Figure 4.9 were printed and a separate testing programme was carried out. Twelve specimens were printed and exposed to the following curing temperatures: (a) room temperature, (b) 70°C and (c) 110°C. After 24 hours of curing, the specimens were stored in a desiccator. Seven days after the 3DP they were tested in four-point-bending using the test procedure described above.

Table 4.2 presents the results of this curing temperature investigation on the 3D printed mechanical properties. An increase of the curing temperature to 70°C results in stronger, stiffer and more brittle material properties than curing at room temperature. By contrast a curing at 110°C reduced the strength and stiffness properties compared to curing at room temperature or at 70°C. This is likely due to evaporation of the binder that reduces the calcium sulphate hemihydrate reaction (Equation 4.1). The data in Table 4.2 suggests that a curing at 70°C could result in 3D printed materials that are more similar to masonry. Further experimental investigations are recommended to optimize the properties of the 3D printed material.

4.5 Global building stiffness

This section details the estimation of the axial stiffness, $EA$, and the bending stiffness, $EI$, of the different building configurations. To obtain $EI$, first the the position of the neutral axis of the different building models was determined, after which $EI$ was estimated by considering different approaches of reducing $EI$ due to façade openings. The subsequent estimation of $EA$ also takes façade openings into account.
4.5 Global building stiffness

4.5.1 Bending stiffness

4.5.1.1 Neutral axis of centrifuge model buildings

To calculate the theoretical position of the neutral axis, it is assumed that the position of the neutral axis is equivalent to the centroidal axis. A building model with 20% of openings and \( L = 200 \text{ mm} \), as shown in Figure 4.12a, is discussed in detail, followed by an overview of results for all building models.

To obtain the overall position of the neutral axis, the position of the neutral axis for the cross-sections A-A and B-B, shown in Figure 4.12, are estimated by subdividing the cross-sections into rectangles as shown in Figures 4.12b and 4.12c. The cross-sections of the rear façade are identical and thus not shown in Figure 4.12. The centroid of a single cross-section can be estimated by

\[
\bar{z} = \frac{\sum_{i=1}^{n} z_i \cdot A_i}{\sum_{i=1}^{n} A_i}
\]  

(4.5)

where \( z_i \) is the distance from the centroid of each rectangle to the reference axis (e.g. base of foundation) and \( A_i \) the area of a rectangle. For the solid cross-section \( \bar{z}_{AA} \) is 37.5 mm while \( \bar{z}_{BB} \) is 30.2 mm. The overall position of the neutral axis is determined by

\[
\bar{z}_{tot} = \frac{L_{AA} \cdot \bar{z}_{AA} + L_{BB} \cdot \bar{z}_{BB}}{L}
\]  

(4.6)

where \( L_{AA} \) and \( L_{BB} \) are the lengths of the building with each type of cross-section (A-A and B-B) and \( L \) is the entire building length. The obtained overall neutral axis for the reference scenario is 34.6 mm and slightly below the mid-height of the building (i.e. 43.8 mm).

The identical procedure was also employed for the remaining building configurations. The obtained overall neutral axis for the short structure, \( L = 200 \text{ mm} \), with 40% openings is 29.9 mm. For the ‘long’ building models with \( L = 260 \text{ mm} \), the position of the neutral axis changes due to different amount and length of the cross-sections A-A and B-B. Table 4.3 summarises the position of the neutral axis for the building configurations of the performed centrifuge test series.

4.5.1.2 Plane-strain relative building stiffness measures

To obtain the relative building stiffness per running metre, the stiffness of the wall and foundation were reduced to per meter values in the plane perpendicular to bending. A so-called spacing factor \( s \) for the wall and foundation of the building was therefore introduced. This approach follows the procedure outlined in Farrell (2010) for buildings with shallow strip
3D printed building models

(a) Building model with 20% of openings and a length of 200mm.

(b) Cross-sections through solid section (A-A).

c) Cross-sections through opening section (B-B).

Fig. 4.12 Reference scenario to estimate neutral axis of building models. Cross-sections show only the front façade. (1) relates to the strip footing while (2) to (4) are parts of the façade wall. Dimensions in mm.

Table 4.3 Position of neutral axis for the centrifuge tests.

<table>
<thead>
<tr>
<th>Test</th>
<th>Openings (%)</th>
<th>Length (mm)</th>
<th>Position Neutral Axis (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>20</td>
<td>200</td>
<td>34.6</td>
</tr>
<tr>
<td>B</td>
<td>20</td>
<td>200</td>
<td>34.6</td>
</tr>
<tr>
<td>C</td>
<td>20</td>
<td>200</td>
<td>34.6</td>
</tr>
<tr>
<td>D</td>
<td>40</td>
<td>200</td>
<td>29.9</td>
</tr>
<tr>
<td>E</td>
<td>20</td>
<td>260</td>
<td>34.5</td>
</tr>
<tr>
<td>F</td>
<td>40</td>
<td>260</td>
<td>29.7</td>
</tr>
<tr>
<td>G</td>
<td>40</td>
<td>260</td>
<td>29.7</td>
</tr>
</tbody>
</table>
foundations. For example, Equation 4.7 gives the spacing factor for the façade wall by dividing the sum of the width of the walls perpendicular to the plane of bending \( b_{\text{w,tot}} = 2 \cdot 4 \text{ mm} \) by the total width of the building \( B = 100 \text{ mm} \).

\[
sw = \frac{b_{\text{w,tot}}}{B}
\]  

Equation 4.7

Shear transfer between the walls and the foundation was assumed due to the rigid connection between the foundation and the walls in the 3D printed building models. Therefore, an overall neutral axis of the surface structure (see above) was employed. In the following, three previous approaches to estimate the building bending stiffness, which mainly differ in the concept of taking window openings into account, are presented. The calculations are carried out in the prototype space to subsequently compare the obtained building stiffness values with field data and previous centrifuge tests. The used scale factor (i.e. \( N = 71.6 \)) considers the location of the building model in the centrifugal acceleration field. Figure 4.13 illustrates the dimensions of cross-section A-A (Figure 4.12b) in prototype scale and shows the front and back façades. A Young’s modulus of 727.4 MPa, which is the average Young’s modulus of the 3D printed material of the test C, is applied for the reference calculation. For the other tests, the corresponding Young’s moduli and building dimensions are used.

4.5.1.3 Approach by Melis and Rodriguez Ortiz (2001)

The overall building bending stiffness based on the framework developed by Melis and Rodriguez Ortiz (2001) can be estimated by

\[
EI_{\text{MRO}} = \sum EI_{\text{floors}} + \sum EI_{\text{walls}} + \sum EI_{\text{basement}}.
\]  

Equation 4.8
Floors are not taken into account in the 3D printed building model. As a consequence, Equation 4.8 reduces to the contributions of the walls and the basement (i.e. foundation).

**Shallow Foundation:** 0.716 m high and 0.766 m wide footings at a spacing of 0.21 m/m, distance to the theoretical neutral axis of 2.12 m (29.6 mm · N · 10⁻³) and Young’s modulus of 727.4·10³ kN/m²

\[ s_f = 2 · 0.766 / 7.161 = 0.21 \text{m/m} \]

\[ I_f = (1.0 · 0.716^3 / 12 + 1.0 · 0.716 · 2.12^2) · 0.21 = 0.694 \text{m}^4 / \text{m} \]

\[ EI_f = 727.4 · 10^3 · 0.694 = 5.05 · 10^5 \text{ kNm}^2 / \text{m} \]

**Facade walls:** 0.286 m thick and 5.55 m high walls with a spacing of 0.08 m/m, 20% window openings, distance to neutral axis of 1.016 m and Young’s modulus of 727.4·10³ kN/m²

\[ s_w = 2 · 0.286 / 7.161 = 0.08 \text{m/m} \]

\[ \alpha_{red} = 0.6 \text{ opening reduction factor for 20% window openings and L>2H) } \]

\[ I_w = (1.0 · 5.55^3 / 12 + 1.0 · 5.55 · 1.016^2) · 0.08 = 1.60 \text{m}^4 / \text{m} \]

\[ EI_w = 727.4 · 10^3 · 1.60 · 0.6 = 6.97 · 10^5 \text{ kNm}^2 / \text{m} \]

**Building bending stiffness (test C):**

\[ EI_{MRO} = 5.05 · 10^5 + 6.97 · 10^5 = 1.20 · 10^6 \text{ kNm}^2 / \text{m} \]

The EI estimation following Melis and Rodriguez Ortiz (2001) was carried out for all building configurations. For every building model the average Young’s modulus, obtained from the four-point bending tests on the 3D printed material samples, was applied. The spacing factor procedure was employed for the entire set of buildings with a 3D building geometry (i.e. front, rear, partitioning and side walls) while for the isolated façade test (i.e. test G) the spacing factor (Equation 4.7) was obtained by changing \( B \) to the width of the foundation (i.e. \( B = 10.7 \text{mm} \)). According to Melis and Rodriguez Ortiz (2001) façade opening reduction factors of 0.60 and 0.15 were applied to \( EI_{walls} \) for buildings with 20% and 40% of window openings. Table 4.4 summarises the obtained bending stiffness values of the entire centrifuge test series.

**4.5.1.4 Approach by Son and Cording (2005, 2007)**

The RSM proposed by Son and Cording (2005) relates the soil stiffness to the elastic shear modulus of the building, \( G_{eq} \). Son and Cording (2005) report that \( G_{eq} \) can be determined by
4.5 Global building stiffness

Table 4.4 Global building bending stiffness following Melis and Rodriguez Ortiz (2001).

<table>
<thead>
<tr>
<th>Test</th>
<th>O (%)</th>
<th>L (mm)</th>
<th>$E_{3DP}$ (MPa)</th>
<th>$EI_{MRO}$ (kNm²/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>20</td>
<td>200</td>
<td>893.1</td>
<td>$1.5 \times 10^6$</td>
</tr>
<tr>
<td>B</td>
<td>20</td>
<td>200</td>
<td>800.6</td>
<td>$1.3 \times 10^6$</td>
</tr>
<tr>
<td>C</td>
<td>20</td>
<td>200</td>
<td>727.4</td>
<td>$1.2 \times 10^6$</td>
</tr>
<tr>
<td>D</td>
<td>40</td>
<td>200</td>
<td>515.9</td>
<td>$4.0 \times 10^5$</td>
</tr>
<tr>
<td>E</td>
<td>20</td>
<td>260</td>
<td>689.9</td>
<td>$1.1 \times 10^6$</td>
</tr>
<tr>
<td>F</td>
<td>40</td>
<td>260</td>
<td>1039.2</td>
<td>$8.1 \times 10^5$</td>
</tr>
<tr>
<td>G</td>
<td>40</td>
<td>260</td>
<td>737.2</td>
<td>$2.7 \times 10^6$</td>
</tr>
</tbody>
</table>

numerical analysis. In this research, $G_{eq}$ is obtained by using $G_{3DP} = E_{3DP}/(2(1 + \nu))$. A Poisson’s ratio, $\nu$, of 0.3 was used, which is identical to the average $\nu$ derived by Feng et al. (2015) for specimens loaded in XY plane. The $G_{3DP}$ results for every building configuration of the centrifuge test series are listed in Table 4.5.

Son and Cording (2007) reported a reduction of $G_{eq}$ between -33% to -77% when opening percentage increases from 0% to 30%. The range of their results is a consequence of investigating different ratios between joint shear stiffness and joint normal stiffness to account for the anisotropy of masonry. For this work, their mean reduction factor of -55% was adopted and a linear relation between shear stiffness reduction and window opening percentage was assumed to obtain shear stiffness reduction factors for 20% and 40% of window openings. These assumptions resulted in reduction factors of 0.63 and 0.27 for 20% and 40% openings respectively. The reduced shear stiffness values of the building models, $G_{red}$, are also presented in Table 4.5.

Identical to the approach of Melis and Rodriguez Ortiz (2001), the reduction due to openings was applied only to the façade walls. For the foundation, the unreduced $G_{3DP}$ was used. The equivalent shear stiffness, $GA_{eq}$, can therefore be written as

$$GA_{eq} = G_{3DP}h_f b_f + G_{red} h_w b_w$$  \hfill (4.9)

where $h_f$ is the height of the strip footings, $b_f$ the width of the strip footings, $h_w$ the height of the façade wall and $b_w$ the width of the façade wall (Figure 4.13). For the tests with a 3D building layout (i.e. tests A to F), $b_f$ and $b_w$ are the sum of the front and rear façade widths. Table 4.5 summarises the obtained results of $GA_{eq}$.

For the equivalent bending stiffness, $E_{eq}$, Son and Cording (2007) reported a significantly smaller reduction due to window openings compared to $G_{eq}$. Their parametric study showed that $E_{eq}$ reduced between -20% to -26% when the opening percentage increased from 0% to
3D printed building models

Table 4.5 Global building shear stiffness following Son and Cording (2005).

<table>
<thead>
<tr>
<th>Test</th>
<th>$E_{3DP}$ (MPa)</th>
<th>$G_{3DP}$ (MPa)</th>
<th>$\alpha_{red}$</th>
<th>$G_{red}$ (MPa)</th>
<th>$GA_{eq}$ (kN)</th>
<th>$EI_{eq}$ (kNm²/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>893.1</td>
<td>343.5</td>
<td>0.63</td>
<td>217.6</td>
<td>$1.1 \cdot 10^6$</td>
<td>$1.8 \cdot 10^6$</td>
</tr>
<tr>
<td>B</td>
<td>800.6</td>
<td>307.9</td>
<td>0.63</td>
<td>195.0</td>
<td>$9.6 \cdot 10^5$</td>
<td>$1.6 \cdot 10^6$</td>
</tr>
<tr>
<td>C</td>
<td>727.4</td>
<td>279.8</td>
<td>0.63</td>
<td>177.2</td>
<td>$8.7 \cdot 10^5$</td>
<td>$1.5 \cdot 10^6$</td>
</tr>
<tr>
<td>D</td>
<td>515.9</td>
<td>198.4</td>
<td>0.27</td>
<td>52.9</td>
<td>$3.9 \cdot 10^5$</td>
<td>$9.5 \cdot 10^5$</td>
</tr>
<tr>
<td>E</td>
<td>689.9</td>
<td>265.4</td>
<td>0.63</td>
<td>168.1</td>
<td>$8.3 \cdot 10^5$</td>
<td>$1.4 \cdot 10^6$</td>
</tr>
<tr>
<td>F</td>
<td>1039.2</td>
<td>399.7</td>
<td>0.27</td>
<td>106.6</td>
<td>$7.8 \cdot 10^5$</td>
<td>$1.9 \cdot 10^6$</td>
</tr>
<tr>
<td>G</td>
<td>737.2</td>
<td>283.5</td>
<td>0.27</td>
<td>75.6</td>
<td>$2.8 \cdot 10^5$</td>
<td>$6.4 \cdot 10^6$</td>
</tr>
</tbody>
</table>

30%. Taking the same assumptions as for $G_{red}$ (i.e. mean reduction of -23% and a linear relation between stiffness reduction and window opening percentage), the $EI_{walls}$ reductions due to 20% and 40% window openings are -15.3% and -30.7% respectively. This results in window opening reduction factors of 0.85 and 0.69 for 20% and 40% of openings. The global $EI_{eq}$ values, shown in Table 4.5, were also derived by performing the identical procedure as discussed in Section 4.5.1.3, but using the $EI$ reduction factors of Son and Cording (2007) instead of Melis and Rodriguez Ortiz (2001).

4.5.1.5 Approach by Pickhaver et al. (2010)

Pickhaver et al. (2010) proposed an equivalent beam model to replicate a building as a mesh of equivalent elastic Timoshenko beams which is often called ‘strip method’. Although their strip method was aimed to simplify the input of 3D finite element analyses, their approach takes a reduction of the building stiffness due to window openings and the aspect ratio, $L/H$, into account. It is therefore of interest to evaluate how this approach compares to the work of Melis and Rodriguez Ortiz (2001) and Son and Cording (2007).

The strip method subdivides the building into horizontal strips to consider window openings when estimating the global building bending stiffness. Subsequently, Pickhaver et al. (2010) proposed an effective height for the window strips by relating the surface area of a strip, $a_j$, to the building length. This conceptual method can be seen in Figure 4.14 and the effective height can be estimated by

$$h_{eff,j} = \frac{a_j}{L}. \quad (4.10)$$

The height of the strips without windows remains constant. For their investigated façade, without a foundation, Pickhaver et al. (2010) assumed the centroid at mid-height for a smooth soil–structure interface while for a rough base the centroid is coincident with the soil surface.
Within this work the theoretical neutral axis locations (Section 4.5.1.1) was considered. The second moment of area of an individual strip, \( j \), according to Pickhaver et al. (2010) is given by

\[
I_j = \left( \frac{t_j (h_{\text{eff},j})^3}{12} + t_j h_{\text{eff},j} (b_j)^2 \right) s_f,j
\]

(4.11)

where \( t \) is the width of the foundation or the wall, \( h_{\text{eff}} \) the effective height of the strip (see above), \( b_j \) the distance from the mid-point of the strip to the neutral axis of the building and \( s_f \) is the spacing factor for the foundation or the wall. As mentioned before the second moment of area is calculated per running metre (i.e. plane-strain value for \( t = 1 \) m/m) and the spacing factor is used similar to above. The overall second moment of area of the building model or the façade is the sum of the individual strips, which can be written as

\[
I^* = \sum_{j=1}^{n} I_j.
\]

(4.12)

Pickhaver et al. (2010) found that their conceptual model overestimates the second moment of area when the aspect ratio (i.e. \( L/H \)) is below \((L/H)_{\text{crit}} = 3\). For \( L/H \) values below 3 the second moment of area should be reduced by a factor \( k \), which is defined as

\[
k = \frac{(L/H)}{(L/H)_{\text{crit}}}.
\]

(4.13)

To be consistent with the dimensionless group \( L/H \) (see Chapter 3), the overall height of the building model, \( H = 90 \) mm, including the elevation of the side walls (Figure 4.12a) was applied when deriving \( k \). Table 4.6 shows the obtained \( k \) values for the different building
3D printed building models

Table 4.6 Global building bending stiffness following Pickhaver et al. (2010).

<table>
<thead>
<tr>
<th>Test</th>
<th>$I_{A-A}$ (m$^4$/m)</th>
<th>$L/H$ ()</th>
<th>$k$ ()</th>
<th>$\alpha_{red}$ ()</th>
<th>$k \cdot \alpha_{red}$ ()</th>
<th>$I_{sm}$ (m$^4$/m)</th>
<th>$EI_{sm}$ (kNm$^2$/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>2.26</td>
<td>2.22</td>
<td>0.74</td>
<td>0.89</td>
<td>0.66</td>
<td>1.50</td>
<td>1.3 · 10$^6$</td>
</tr>
<tr>
<td>B</td>
<td>2.26</td>
<td>2.22</td>
<td>0.74</td>
<td>0.89</td>
<td>0.66</td>
<td>1.50</td>
<td>1.2 · 10$^6$</td>
</tr>
<tr>
<td>C</td>
<td>2.26</td>
<td>2.22</td>
<td>0.74</td>
<td>0.89</td>
<td>0.66</td>
<td>1.50</td>
<td>1.1 · 10$^6$</td>
</tr>
<tr>
<td>D</td>
<td>2.26</td>
<td>2.22</td>
<td>0.74</td>
<td>0.74</td>
<td>0.55</td>
<td>1.24</td>
<td>6.4 · 10$^5$</td>
</tr>
<tr>
<td>E</td>
<td>2.26</td>
<td>2.89</td>
<td>0.96</td>
<td>0.89</td>
<td>0.86</td>
<td>1.94</td>
<td>1.3 · 10$^6$</td>
</tr>
<tr>
<td>F</td>
<td>2.26</td>
<td>2.89</td>
<td>0.96</td>
<td>0.73</td>
<td>0.70</td>
<td>1.59</td>
<td>1.6 · 10$^6$</td>
</tr>
<tr>
<td>G (2D)</td>
<td>10.58</td>
<td>2.89</td>
<td>0.96</td>
<td>0.73</td>
<td>0.70</td>
<td>7.42</td>
<td>5.5 · 10$^6$</td>
</tr>
</tbody>
</table>

Table 4.7 Adopted opening reduction factors.

<table>
<thead>
<tr>
<th>Literature</th>
<th>O: 20%</th>
<th>O: 40%</th>
<th>Applicability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Melis and Rodriguez Ortiz (2001)</td>
<td>0.60</td>
<td>0.15</td>
<td>walls, $L &gt; 2H$</td>
</tr>
<tr>
<td>Son and Cording (2007)</td>
<td>0.85</td>
<td>0.69</td>
<td>walls, mean values, $\alpha_{red} \propto$ Open.</td>
</tr>
<tr>
<td>Pickhaver et al. (2010)</td>
<td>0.89</td>
<td>0.73</td>
<td>back-calculated, $k = 1$</td>
</tr>
</tbody>
</table>

models. The final second moment of area according to the strip method, $I_{sm}$ is given by

$$I_{sm} = I^*k.$$  \hspace{1cm} (4.14)

To obtain the global bending stiffness of the building models, the average Young’s modulus derived from the material tests on 3D printed samples are used for each centrifuge test (Table 4.4). Table 4.6 presents the global building stiffness values calculated according to the Pickhaver et al. (2010) strip method with the neutral axis at the centroid of the building. Additionally, the second moment of area neglecting openings, $I_{A-A}$, reduction factors due to window openings, $\alpha_{red}$, and due to the aspect ratio, $k$, and the reduced second moment of area, $I_{sm}$, are presented in Table 4.6.

Table 4.7 summarises the opening reduction factors adopted for the building models of this research. As indicated in Table 4.7, these values were obtained by certain assumptions and might change with different building features such as the aspect ratio. Furthermore, the Melis and Rodriguez Ortiz (2001) and Son and Cording (2005) factors were used to reduce the stiffness of the building walls while the Pickhaver et al. (2010) reduction factors were back-calculated based on their discussed strip method considering both the strip foundation and the building walls.

As indicated in Table 4.7, the reduction factors are applied for certain structural elements (i.e. walls). To estimate the reduction of the entire building bending stiffness, the global build-
### 4.5 Global building stiffness

**Table 4.8** Adopted opening reduction factors.

<table>
<thead>
<tr>
<th>Test</th>
<th>$EI_{A-A}$ (kNm²/m)</th>
<th>$EI_{MRO}$ (kNm²/m)</th>
<th>$EI_{eq}$ (kNm²/m)</th>
<th>$EI_{sm}$ (kNm²/m)</th>
<th>$\alpha_{MRO}$</th>
<th>$\alpha_{SC}$</th>
<th>$\alpha_{sm}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>2.0·10⁶</td>
<td>1.5·10⁶</td>
<td>1.8·10⁶</td>
<td>1.3·10⁶</td>
<td>0.73</td>
<td>0.90</td>
<td>0.66</td>
</tr>
<tr>
<td>B</td>
<td>1.8·10⁶</td>
<td>1.3·10⁶</td>
<td>1.6·10⁶</td>
<td>1.2·10⁶</td>
<td>0.73</td>
<td>0.90</td>
<td>0.66</td>
</tr>
<tr>
<td>C</td>
<td>1.7·10⁶</td>
<td>1.2·10⁶</td>
<td>1.5·10⁶</td>
<td>1.1·10⁶</td>
<td>0.73</td>
<td>0.90</td>
<td>0.66</td>
</tr>
<tr>
<td>D</td>
<td>1.2·10⁶</td>
<td>4.0·10⁵</td>
<td>9.5·10⁵</td>
<td>6.4·10⁵</td>
<td>0.35</td>
<td>0.82</td>
<td>0.55</td>
</tr>
<tr>
<td>E</td>
<td>1.6·10⁶</td>
<td>1.1·10⁶</td>
<td>1.4·10⁶</td>
<td>1.3·10⁶</td>
<td>0.73</td>
<td>0.90</td>
<td>0.86</td>
</tr>
<tr>
<td>F</td>
<td>2.4·10⁶</td>
<td>8.1·10⁵</td>
<td>1.9·10⁶</td>
<td>1.6·10⁵</td>
<td>0.34</td>
<td>0.82</td>
<td>0.70</td>
</tr>
<tr>
<td>G</td>
<td>7.8·10⁶</td>
<td>2.7·10⁵</td>
<td>6.4·10⁵</td>
<td>5.5·10⁵</td>
<td>0.34</td>
<td>0.82</td>
<td>0.70</td>
</tr>
</tbody>
</table>

**Fig. 4.15** Comparison of $EI$ values after applying window opening reduction.

The bending stiffness without openings, $EI_{A-A}$, was determined and subsequently compared to the $EI$ values derived above. This overall $EI$ reduction for the different building configurations is summarised in Table 4.8 and visualised in Figure 4.15. It is clear from Figure 4.15 that the approach proposed by Son and Cording (2007) resulted in the greatest $EI$ values. For 20% of openings and $L/H = 2.2$, the strip method (Pickhaver et al., 2010) caused the greatest decrease of $EI$, while the procedure according to Melis and Rodriguez Ortiz (2001) showed the greatest decrease for 20% opening area and $L/H = 2.9$ (test E) and for buildings with 40% opening area (tests D, F and G). Specifically, for buildings with 40% opening area, the Melis and Rodriguez Ortiz (2001) procedure resulted in a notable greater reduction than the other methods. Overall, the three procedures are in fair agreement when considering the logarithmic scale of the associated design charts.

However, a comparison of the estimates of the overall bending stiffness for the different building configurations reveals that the overall bending stiffness of the isolated façade case
3D printed building models

(test G) is notably greater than for the other tests (Figure 4.15). This can be attributed to the current concept of estimating the global building stiffness per metre run using a spacing factor to account for the building geometry out of plane-strain. For the isolated building test, the spacing factor is estimated by dividing the width of a structural component (i.e. wall or foundation) by the foundation width. This approach was applied in the field when assessing the overall stiffness of masonry walls (Mair and Taylor, 2001) and in centrifuge model tests for simplified masonry beams (Farrell, 2010). The measured structural performance of these cases suggests that the spacing factor approach is also adequate for an isolated façade but the considered building stock had a significantly great aspect ratio (i.e. $L/H$). In other words, the masonry walls and beam models were very short. A comparison between the performance of the tests G and F will enable to evaluate the applicability of this spacing factor methodology for different building layouts which potentially is a current limitation to accurately estimate the global building stiffness in bending.

4.5.2 Axial stiffness

While the section above discussed the estimation of the building bending stiffness, this section is concerned with the determination of the axial building stiffness, $EA$. Reduction factors to reduce $EA$ due to façade openings are less established compared to $EI$. This is likely due to the high contribution of building foundations to the global $EA$ of structures and the often very rigid response of buildings to horizontal ground movements (Burland et al., 2004).

This study estimates the $EA$ values for the different building configurations with and without considering window openings. Pickhaver et al. (2010) suggested a procedure that accounts for window openings when estimating an equivalent façade area, $A^*$, to derive the shear stiffness of a façade. This framework is followed herein.

In this approach, the building is subdivided into vertical strips based on the location of the windows, as shown in Figure 4.16. $A^*$ is then determined by

$$A^* = \frac{\sum_{j=1}^{n} A_j L_j}{L}$$  \hspace{1cm} (4.15)

where $A_j$ is the cross-sectional area of the strip parallel to the tunnel, $L_j$ the length of the strip transverse to the tunnel and $L$ the entire building length. A total cross-sectional area per metre run for the entire building, $A_{tot}$, is determine by

$$A_{tot} = \frac{2A^*}{B}$$  \hspace{1cm} (4.16)
4.5 Global building stiffness

Fig. 4.16 Equivalent area considering façade openings (after Pickhaver et al., 2010).

Table 4.9 Global axial building stiffness.

<table>
<thead>
<tr>
<th>Test</th>
<th>$E_A$ (kN/m)</th>
<th>$E_{A_{eq}}$ (kN/m)</th>
<th>$\lambda$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>$5.3 \cdot 10^5$</td>
<td>$4.49 \cdot 10^5$</td>
<td>0.84</td>
</tr>
<tr>
<td>B</td>
<td>$4.8 \cdot 10^5$</td>
<td>$4.03 \cdot 10^5$</td>
<td>0.84</td>
</tr>
<tr>
<td>C</td>
<td>$4.3 \cdot 10^5$</td>
<td>$3.66 \cdot 10^5$</td>
<td>0.84</td>
</tr>
<tr>
<td>D</td>
<td>$3.1 \cdot 10^5$</td>
<td>$2.04 \cdot 10^5$</td>
<td>0.66</td>
</tr>
<tr>
<td>E</td>
<td>$4.1 \cdot 10^5$</td>
<td>$3.45 \cdot 10^5$</td>
<td>0.84</td>
</tr>
<tr>
<td>F</td>
<td>$6.2 \cdot 10^5$</td>
<td>$4.06 \cdot 10^5$</td>
<td>0.65</td>
</tr>
<tr>
<td>G</td>
<td>$4.1 \cdot 10^6$</td>
<td>$1.35 \cdot 10^6$</td>
<td>0.65</td>
</tr>
</tbody>
</table>

where the factor 2 accounts for the rear and front façade area and $B$ is the building width parallel to the tunnel. For the isolated façade test (i.e. test G) Equation 4.16 reduces to $A_{total} = A^\ast / B$ where $B = b_f$. Table 4.9 lists the global axial building stiffness considering a solid cross-section, $E_A$, and the $E_{A_{eq}}$ taking façade openings into account. The reduction of $EA$ due to the window openings is expressed with the factor $\lambda$. For the buildings with 20% and 40% openings $EA$ reduced by approximately 16% ($\lambda = 0.84$) and 35% ($\lambda = 0.65$), respectively.

A reduction of the lateral strain due to the axial building stiffness is also proposed by Boscardin and Cording (1989) and Son and Cording (2005) as introduced in Section 2.2.5. To obtain the axial building stiffness, Boscardin and Cording (1989) suggested to consider solely the cross-section of the foundation. A spacing factor, $S$, which is the distance between transverse strip footings, is already defined in their relative axial building stiffness formulation (Equation 2.32) and thus the 3D building layout can be neglected when estimating $EA$ according to Boscardin and Cording (1989). Therefore, $EA_{BC}$ according to Boscardin and Cording (1989) is computed by multiplying the $E_{3DP}$ with the cross-section of a single strip footing, $A_f$. Table 4.10 summarises the results of estimating $EA_{BC}$ after Boscardin and Cord-
### Table 4.10 Global axial building stiffness according to Boscardin and Cording (1989).

<table>
<thead>
<tr>
<th>Test</th>
<th>$E_{3DP}$ (kPa)</th>
<th>$A_f$ (m²)</th>
<th>$EA_{BC}$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>8.93 · $10^5$</td>
<td>0.55</td>
<td>4.90 · $10^5$</td>
</tr>
<tr>
<td>B</td>
<td>8.00 · $10^5$</td>
<td>0.55</td>
<td>4.39 · $10^5$</td>
</tr>
<tr>
<td>C</td>
<td>7.27 · $10^5$</td>
<td>0.55</td>
<td>3.99 · $10^5$</td>
</tr>
<tr>
<td>D</td>
<td>5.16 · $10^5$</td>
<td>0.55</td>
<td>2.83 · $10^5$</td>
</tr>
<tr>
<td>E</td>
<td>6.90 · $10^5$</td>
<td>0.55</td>
<td>3.79 · $10^5$</td>
</tr>
<tr>
<td>F</td>
<td>1.04 · $10^6$</td>
<td>0.55</td>
<td>5.70 · $10^5$</td>
</tr>
<tr>
<td>G</td>
<td>7.37 · $10^5$</td>
<td>0.55</td>
<td>4.04 · $10^5$</td>
</tr>
</tbody>
</table>

ing (1989). A comparison to the global axial building stiffness values according to Pickhaver et al. (2010) indicates that the approach of Boscardin and Cording (1989) results in slightly greater $EA$ values for the building with 3D building layout (i.e. tests A to F). By contrast, the $EA$ value for the isolated façade test (i.e. test G) is substantially greater when using the Pickhaver et al. (2010) framework. This can be attributed to the procedure of estimating the $EA$ per metre run (Equation 4.16).

### 4.5.3 Comparison to field data and previous research

Figure 4.17 compares the overall building stiffness values of the 3D printed building models with reported field data and previous research. While for Figure 4.17a the approach of Mair and Taylor (2001) was applied to derive global stiffness values of the 3D building models, the $EI$ data of the building models shown in Figure 4.17b were obtained by applying the approach outlined by Melis and Rodriguez Ortiz (2001) (see Section 4.5.1.3) and the $EA$ values take façade openings according to Pickhaver et al. (2010) into account (see Section 4.5.2).

Figure 4.17a shows that the building stiffness of the model buildings are close to the range of case histories (solid black markers) if the procedure of Mair and Taylor (2001) to estimate $EA$ and $EI$ is applied. This historical approach, which was used for the Jubilee line project, neglects stiffness differences out-of-plane and a stiffness reduction due to façade openings. In other words, the building is simplified as a solid infinite beam. Consequently, spacing factors and opening reduction factors were not applied when calculating the global stiffness values of the 3D printed building models in Figure 4.17a. Dimmock and Mair (2008) considered a stiffness reduction due to openings compared to Mair and Taylor (2001) but spacing factors were not applied. On the other hand, the stiffness values of Building 106 and Building 107 were determined by applying spacing factors, but for $EA$ a stiffness reduction due to openings was neglected (Farrell et al., 2011). Both buildings (Building 106 and 107) are characterised
4.5 Global building stiffness

(a) Historical estimate: \(^1\)EA and EI values of the 3D printed building models were estimated by the identical approach used by Mair and Taylor (2001) for the Jubilee line (i.e. neglecting spacing factors and openings).

(b) Accounting for building details: \(^1\)EA and EI values of the 3D printed building models were determined by using spacing factors (Section 4.5.1.3) and reduced due to façade openings (Section 4.5.2). \(^1\)EA and EI values neglect the contribution of floor slabs.

**Fig. 4.17** Global building stiffness values of the centrifuge model buildings in prototype scale compared to field data and previous research.
3D printed building models

by reinforced concrete slabs, which significantly contribute to the global $EA$ values while the contribution to $EI$ is negligible (compare Figure 4.17a with Figure 4.17b). Therefore, the agreement between Building 106 and 107 with the Jubilee line data in Figure 4.17a is purely a coincidence, because the overall $EA$ stiffness of Building 106 and 107 is dominated by floor slabs, which were neglected in the stiffness estimates of the Jubilee line buildings. Floor slabs were not replicated in the 3D printed building models because floors of historic masonry buildings are often made from timber, which have a significantly lower stiffness contribution than concrete diaphragms. For this reason, the contribution of floor slabs is typically neglected for historic masonry structures.

The $EI$ values of the 3D printed building models reported in Figure 4.17b consider building stiffness differences out-of-plane by applying spacing factors and take façade openings into account. Applying this more realistic estimate of the global building stiffness results in a notable difference between the $EA$ values of the 3D printed building models and the historical estimates of the case studies. Figure 4.17b indicates that the global stiffness values of the 3D printed building models are in fair agreement with Building 106 and 107 when neglecting the contribution of floor slabs. Adding spacing factors for the Jubilee line data would shift these data points towards the origin (i.e. to the left in Figure 4.17). The data shown in Figure 4.17b highlights that the estimate of the overall building stiffness depends on the followed procedure. However, minor variations in estimating $EI$ and $EA$ are of negligible impact on the assessment of building response to tunnelling subsidence because the related design charts of the RSMs are in logarithmic scale.

Table 4.11 summarises the $EA$ and $EI$ values of the 3D printed building models presented in Figure 4.17b and used throughout this dissertation. Although the material properties of the 3D printed material showed some variability, Figure 4.17b and Table 4.11 indicate that the global building stiffness of identical building models (i.e. tests A, B and C) are in fair agreement. An increase of the window openings reduced the $EI$ values of the tests D and F while greater building stiffness values were obtained for the isolated façade test (i.e. test G). This provides confidence that the 3D printed building models represent realistic scenarios to gain further insight into the effects of building features on this tunnelling–soil–structure interaction problem.

4.6 Summary

This chapter has discussed the novel application of 3DP to replicate surface structures subject to tunnelling-induced ground displacements. The adopted 3DP technique was introduced in-
Table 4.11 $EA$ and $EI$ of centrifuge model buildings.

<table>
<thead>
<tr>
<th>Test</th>
<th>$EA$ (kN/m)</th>
<th>$EI$ (kNm$^2$/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>$4.49 \cdot 10^5$</td>
<td>$1.48 \cdot 10^6$</td>
</tr>
<tr>
<td>B</td>
<td>$4.03 \cdot 10^5$</td>
<td>$1.32 \cdot 10^6$</td>
</tr>
<tr>
<td>C</td>
<td>$3.66 \cdot 10^5$</td>
<td>$1.20 \cdot 10^6$</td>
</tr>
<tr>
<td>D</td>
<td>$2.04 \cdot 10^5$</td>
<td>$4.05 \cdot 10^5$</td>
</tr>
<tr>
<td>E</td>
<td>$3.45 \cdot 10^5$</td>
<td>$1.14 \cdot 10^6$</td>
</tr>
<tr>
<td>F</td>
<td>$4.06 \cdot 10^5$</td>
<td>$8.10 \cdot 10^5$</td>
</tr>
<tr>
<td>G</td>
<td>$1.35 \cdot 10^6$</td>
<td>$2.69 \cdot 10^6$</td>
</tr>
</tbody>
</table>

Including a description of the applied 3DP process, the microstructure and the composition of the 3D printed material. Subsequently, the location and orientation of the 3D printed structures within the print bed and the preparation of the small-scale models after 3DP is discussed. Details about applying the dead load bars for the isolated façade test (i.e. test F) were then presented. The mechanical properties of the 3D printed material were derived by four-point-bending tests. From this testing programme it was observed that the 3D printed material exhibits brittle material properties similar to that of masonry and hence can be employed to study cracking damage. However, the mechanical properties of the 3D printed material revealed that the stiffness of the 3D printed material is comparable to historic masonry but the 3D printed material is notably stronger than masonry. A preliminary investigation into different curing temperatures suggests that strength properties in better agreement with masonry could be achieved in the future.

Based on the data from the 3D printed material testing, a careful balancing of the building layout and the façade openings resulted in building models with axial and bending stiffness in the range of typical case studies. A comparison between different methods of estimating the overall bending stiffness of the building variations, which particularly focused on the stiffness reduction due to façade openings, revealed that the method by Son and Cording (2007) resulted in little reduction. By contrast, the reduction factors proposed by Melis and Rodriguez Ortiz (2001) caused the greatest reduction for buildings with 40% of opening area. Overall, the different methods resulted in broadly consistent estimates when considering the logarithmic scale of the associated design charts.
Chapter 5

Centrifuge modelling effects and boundary conditions

This chapter discusses centrifuge modelling challenges when exploring the response of surface structures to tunnelling in sand. As outlined in Chapter 3, the model tunnel employed consists of an interior brass cylinder surrounded by water inside a sealed flexible latex lining. Prior to extracting water to simulate tunnelling volume loss, soil displacements obtained during centrifuge acceleration indicate that this flexible tunnel can lead to differential settlements during spin-up. This chapter addresses this interaction between the tunnel, the soil and the building during the spin-up and discusses boundary effects observed during the experiments.

5.1 Background

In centrifuge testing, it is essential to evaluate modelling limitations, and potential undesired effects that occurred due to modelling techniques that schematically simulate geotechnical processes (e.g. tunnel excavation). The adopted tunnelling technique (Section 3.3.2) aims to minimise the impact of the model tunnel on the initial stress conditions of the soil surrounding the tunnel. However, as was pointed out by König (2012), the stress conditions in the tunnel cannot replicate the theoretical earth pressure at rest. As a consequence, this tunnelling simulation method adequately replicates ground movements away from the tunnel but is less suitable to observe stress changes surrounding the tunnel. Specifically, the employed artificial tunnelling simulation technique can lead to ground displacements during the acceleration of the centrifuge (Vorster, 2005). This chapter quantifies potential differential surface displacements during centrifuge acceleration and the impact of these non-uniform displacements on the structure models. Furthermore, this chapter is concerned with the reliability of the image-
Centrifuge modelling effects and boundary conditions

Fig. 5.1 Centrifuge tests A, B, E and F used to address spin-up phenomena.

Based deformation measurement technique. Friction effects between the soil model and the PMMA window are quantified to evaluate their influence on soil deformation measures obtained by using GeoPIV. Finally, a potential rotation of the building model out of the PMMA window plane is assessed by making use of MEMS accelerometer data.

5.2 Spin-up phenomena

During spin-up, the soil model theoretically experiences uniform one-dimensional compression caused by an increase in the self-weight of the soil. It is common practice to take the final state of the spin-up as the initial condition of the subsequent tunnel excavation simulation. However, an investigation of the spin-up phase of the conducted centrifuge tests revealed an interaction between the soil, the tunnel and the building. A number of issues were identified which are discussed below. Focus is primarily placed on the tests A, B, E and F, which are depicted in Figure 5.1, because these tests present the entire range of parameters studied in the centrifuge test series. Results for all tests are presented when discussing the impact of these spin-up phenomena on the building models. Further details of the centrifuge tests are provided in Section 3.7.
5.2 Spin-up phenomena

![Pressure graph](image)

Fig. 5.2 Pressure in the tunnel pressure and volume loss control system during spin-up.

5.2.1 Tunnel pressure control

The performance of the model tunnel during spin-up of the centrifuge tests is connected to the pressure within the tunnel pressure system (Figure 3.4). Figure 5.2 presents the development of this pressure as monitored by the pore pressure transducer (PPT, Figure 5.2). The pressure in Figure 5.2 is related to the water head in the standpipe and it is not the tunnel pressure. It can be seen from Figure 5.2 that the pressure increased with spin-up duration as g-level increases. The data shows close agreement with the expected pressure at $75g$. The differences between the tests are likely to be related to a minor variation of the height of the PPT ($h_{PPT}$, Figure 3.4) between the centrifuge tests. Nevertheless, the data indicates that the tunnel pressure control system performed successfully, and stress imbalances between the model tunnel and the surrounding soil during spin-up were minimized.

5.2.2 Impact of tunnel excavation simulation technique

During spin-up the tunnel control system (Figure 3.4) balances the tunnel pressure, $\sigma_t$, with the vertical soil stresses, $\sigma_v$, at mid-height of the tunnel to minimize soil displacements surrounding the tunnel. However, due to density differences between the sand and the water, a stress imbalance arises with vertical distance from the tunnel axis, as illustrated in Figure 5.3. This imbalance is larger in the horizontal direction; the horizontal soil stresses at rest, $\sigma_h$, which were approximated by using the assumption of Jaky (1944), are significantly smaller than the tunnel pressure (Figure 5.3). Additionally, the self-weight of the structure affects the stress conditions in the soil.
The stress imbalance during spin-up did induce movements of the ground and the flexible tunnel lining. Figure 5.4 shows the ground displacements surrounding the model tunnel after reaching 75g. The stress imbalance at the tunnel crown reduced the vertical soil displacements directly above the tunnel (Figure 5.4, left). In addition, the flexible tunnel lining moved horizontally outwards at the tunnel springlines (Figure 5.4, right) due to the lower horizontal soil stresses compared to the tunnel pressure. These deformations indicate that the tunnel ovalised during spin-up (Vorster, 2005). A comparison between Figures 5.4a and 5.4b-d depicts the influence of the building model on the spin-up behaviour. In test A, the building model was placed symmetrically above the tunnel and symmetric ground displacements were observed (Figure 5.4a). On the contrary, the structures in tests B, E and F represent eccentric building-to-tunnel scenarios, and were positioned to the right of the model tunnel; the structures triggered higher vertical soil displacements above the right tunnel shoulder (Figures 5.4b-d, left). Consequently, the tunnel lining was notably more constrained at the right hand side of the tunnel which increased movements at the left tunnel shoulder (Figure 5.4b-d, right).

5.2.3 Near surface soil and structure vertical displacements

The tunnel–soil–structure interaction during spin-up discussed above resulted in non-uniform surface soil settlements. Figure 5.5a and the left hand side of Figure 5.5b show that the settlements in the regions left and right of the tunnel were smaller than above the tunnel. This observation can be explained by an increase of the soil stiffness next to the tunnel springline, which is caused by the tunnel ovalisation. This finding is also evident for structures placed in the sagging/hogging transition zone (tests E and F), as shown in Figures 5.5c and 5.5d. The identified mechanism is in line with Vorster (2005) who used a similar experimental setup. By contrast, the right hand side of Figures 5.5b, 5.5c and 5.5d indicate that the spin-up soil settlements in this region are dominated by the building weight.
Fig. 5.4 Vertical (left) and horizontal (right) soil displacements (in mm) adjacent to the model tunnel for: (a) tests A, (b) test B, (c) test E and (d) test F (settlements are positive, left horizontal displacements are negative while right horizontal displacements are positive, displacement vectors are times 20).
Fig. 5.5 Spin-up surface soil and base structure movements.
5.2 Spin-up phenomena

Figure 5.5 also shows that the vertical displacements of the base of the building models exceed the underlying soil settlements. Similar observations were reported by Farrell (2010). This discrepancy may be due to modelling imperfections, the applied image based deformation measurement technique and boundary effects. Although great care was taken during the model preparation, the soil surface cannot be made perfectly level and the building model cannot be placed perfectly flush with the underlying soil surface. As a consequence, some levelling of the soil or embedment of the building models might have caused the slightly higher building displacements observed during spin-up. This effect might have been amplified by the GeoPIV analysis in which the displacements of the structure and the soil cannot be measured directly at the soil–structure interface. Both the soil and the structure displacements were analysed at a certain distance (i.e. 4 mm) from the interface. Another possible explanation might be that the building was not completely flush with the Perspex plane throughout the spin-up phase, and the sand immediately next to the Perspex window might have experienced boundary effects.

5.2.4 Impact on building models

Widely applied methods to estimate the risk of building damage caused by tunnel excavation make use of foundation movement parameters (e.g. Burland, 1995). Within this section, the structure response is evaluated using the horizontal building strains, \( \varepsilon_h \), the deflection ratio, \( DR \), and the slope, \( s \). The \( DR \) and \( s \) can be written as

\[
DR = \frac{\Delta_{max}}{L} \quad \text{(5.1)}
\]

and

\[
s = \frac{S_{V,A} - S_{V,B}}{L} \quad \text{(5.2)}
\]

Figure 5.6 schematically illustrates \( DR \) and \( s \). For the spin-up, the \( DR \) was derived by smoothing the GeoPIV data with a 5th order polynomial, which was adopted to determine the maximum deflection, \( DR_{max} \). This curve fitting of the vertical displacement data obtained by GeoPIV was carried out to smooth the displacement data to prevent overestimation of \( DR \). Note that in Ritter et al. (2017b) the \( DR \) was determined by using a higher order polynomial, which explains the negligible differences for the tests A and B. Distinct hogging and sagging deformations for a single building were not observed during the spin-up; thus, an average spin-up \( DR \) was estimated. \( \varepsilon_h \) is the average horizontal strain at the base of the structure and was determined from the slope of a linear function fitted to the horizontal building displacements.
Figure 5.7 depicts displacements at the base of the structure during centrifuge acceleration. The related building deformation parameters introduced above are also shown in Figure 5.7. It is evident from this data that significant deflection ratios were induced in the building models throughout the spin-up, though they were less than the deflection ratios measured throughout the tunnel excavation stage (see Chapter 7). Surprisingly, the average base horizontal building strains, estimated over the entire building length, were found to be consistently higher during the spin-up compared to the tunnel excavation phase (see Chapter 7). This result might be explained by the lateral component of the centrifugal acceleration, which tends to drag the building model away from the centre of the centrifuge model as g-levels increase. It is also important to point out that during the tunnel excavation phase negligible horizontal strains were induced to the buildings; this confirms previous research (Farrell and Mair, 2011) and field data (Burland et al., 2004). Also notable slope values caused by the spin-up were observed but rigid body tilting has little effect on damage, and is thus of lower importance for damage prediction.

After these building movement parameters are estimated, the tensile strains induced in the building models can be computed following Burland and Wroth (1974) and Boscardin and Cording (1989). The building tensile strains due to the spin-up displacements were estimated to be in general more than an order of magnitude smaller than the strain to failure of the 3D printed material (i.e. 0.29%, Table 4.1). However, for test G, which is particularly flexible to ground movements due to its high window percentage, building length and layout, building tensile strains of 0.13% were calculated. The induced building tensile strains during spin-up represent the condition of the building models before the tunnel excavation. In the real buildings, the physical condition of buildings are often affected by historic differential ground movements caused by, for instance, the building’s self-weight, adjacent construction or temperature cycles and notably affect the sensitivity of structures subject to tunnelling works (Clarke and Laefer, 2014; Devriendt et al., 2013).
5.2 Spin-up phenomena

(a) $A$: $e/L=0$, $L/H=2.2$, $O=20\%$, $G=3D$

(b) $B$: $e/L=0.8$, $L/H=2.2$, $O=20\%$, $G=3D$

(c) $C$: $e/L=0.5$, $L/H=2.2$, $O=20\%$, $G=3D$

(d) $D$: $e/L=0.8$, $L/H=2.2$, $O=40\%$, $G=3D$
Centrifuge modelling effects and boundary conditions

Fig. 5.7 Structure response after spin-up. Deflection ratio ($DR_{su}$), slope ($s_{su}$) and average horizontal base strain ($\varepsilon_{h,su}$, tensile strains are positive) are presented at 75g.
The above quantification of the spin-up phase tensile strains shows that the building material was within the elastic region of the 3D printed material. Overall, no building damage (i.e. cracking) was observed during the centrifuge acceleration. Thus, the soil and structure displacements at 75\(g\) are valid reference conditions for the subsequent modelling of tunnel excavation. All results reported in the following chapters are relative to the reference displacements after centrifugal acceleration.

In theory, strains induced during spin-up should be taken into account when investigating cracking in the structure. However, assessing the effect of the spin-up phase on the building cracking is difficult because the spin-up induced strains cannot be directly superimposed with the tunnelling-induced building strains due their different nature. Due to the fact that the 3D printed material is considerably stronger than masonry (Chapter 3), cracking damage is expected at significantly greater volume losses than in the field. For these reasons, only a qualitative analysis of cracking damage is provided in Chapter 7. Nevertheless, the strains induced during centrifugal acceleration can be compared with a real building’s condition prior to tunnelling, which in reality is even more difficult to assess than the influence of the spin-up phase. The centrifugal acceleration phase can, however, significantly affect building models with strength properties in better agreement with masonry. Therefore, there is a need to account for spin-up effects when studying tunnelling-induced settlement damage of buildings in a geotechnical centrifuge.

5.3 Boundary effects

As discussed in Chapter 3, boundary conditions can affect centrifuge model testing. Within this section, boundary effects of the used centrifuge model testing setup and equipment are addressed with respect to the reliability of the image-based deformation measurement technique (i.e. GeoPIV).

5.3.1 Friction effects between the PMMA window and the soil model

Various researchers have observed boundary effects caused by friction between the PMMA window and the sand model (e.g. Elshafie et al., 2013; Marshall et al., 2012). Figure 5.8 gives a measure of these boundary conditions by comparing the readings of the LVDTs and lasers with the obtained GeoPIV data for test B. The position of the instruments within the centrifuge model is illustrated in Figure 3.1c. Overall, the ratio between the LVDT/laser readings and the GeoPIV data \(\frac{S_{v,LVDTs/lasers}}{S_{v,GeoPIV}}\) was about 1.12 for test B. This value is indicative,
but varied slightly between the different tests. Figure 5.8 indicates that the difference between GeoPIV displacements and LVDT/laser measurements increases with volume loss. Similar results were reported by Marshall et al. (2009), Farrell (2010) and Elshafie et al. (2013). Marshall et al. (2009) found that the shape of the settlement profiles is not significantly affected by the boundary friction. These findings confirm the reliability of the image-based deformation measurement technique and indicate similar boundary effects to previous research.

5.3.2 Out of PMMA window plane movement of building model

This section focuses on building rotation from the PMMA window. Potential sources of this rotation are the friction effects discussed above and effects of the tunnel excavation technique resulting in non-uniform settlement along the length of the tunnel. This limitation caused by the tunnel simulation technique might be caused by the connection of the tunnel lining (i.e. Latex membrane) to the end pieces of the model tunnel (Figure 3.3), and in combination with the friction effects might cause increased soil movements closer to the centre of the strong box (Elshafie et al., 2013; Farrell, 2010; Marshall, 2009).

A movement of the structure out of the PMMA window plane might impact the DIC deformation measurements because the DIC is calibrated within the PMMA window plane. Consequently, it is of main importance to assure that the building model stays in contact with the PMMA window to minimise the error of GeoPIV results. However, artificially pushing the
Fig. 5.9 Position of micro-electro mechanical systems (MEMS) accelerometers on top building models. MEMS not to scale.

building models towards the PMMA window could result in additional friction between the structure and the window and may restrict movement of the building. Because of this, the centrifuge model was designed so that after swing-up the 1g component of the Earth’s gravity field was acting in favour and supports the building model against the window. It was therefore decided that an additional support system to keep the building models within the PMMA window plane was not required. Only for test G was a different setup necessary because of the 2D building layout as discussed in Chapter 4. Nevertheless, MEMS accelerometers were mounted on top of the building model to monitor a potential out of plane movement of the structures. Figure 5.9 provides an overview of the position of the MEMS on the building models as well as with respect to the tunnel centreline.
Figure 5.9 shows that either two (tests A-F) or three (test G) MEMS were used to evaluate the rotation of the building model out of the PMMA window plane. For the building models with 3D printed building layout these instruments were attached to the longitudinal building walls while for test G (2D building layout) three 'longitudinal’ MEMS were mounted on top of the building façade. For all tests, common superglue was used to install the MEMS. In the following, the MEMS are referred to as left, centre (only for test G) and right according to their position in Figure 5.9.

Figure 5.10 illustrates the rotation of the building models out of the PMMA window plane as \( V_{l,t} \) develops. While the entire data of the MEMS placed on the right and centre of the building models showed a negligible rotation, the left MEMS of test C reveals a notable rotation of the structure from the PMMA window. However, for realistic tunnel volume loss values (i.e. < 2.0%), the measured rotation was very small. The considerable rotation at greater \( V_{l,t} \) can be explained by cracking that separated the building into two rigid blocks, and the building block closer to the tunnel rotated out of plane (Section 7.5.3). By contrast the right building experienced negligible rotation as is evident in Figure 5.10b. The data of the left MEMS of test F at \( V_{l,t} > 8.0\% \), shown in Figure 5.10a, which indicates that the building rotated into the PMMA window, can be related to building collapse. The right hand side MEMS of test E was lost during the final model preparation and no reliable data was acquired (Figure 5.10b).

To quantify a potential GeoPIV measurement error caused by the building rotation, it was estimated that a rotation out of plane of \( 1^\circ \) causes a maximum error of 0.014 mm for the vertical displacements, which is close to the precision limit of GeoPIV measurements. For the conducted test series, the measured rotations were significantly lower in the \( V_{l,t} \) region of interest (i.e. < 4.0%), as shown in Figure 5.10. Hence the MEMS data demonstrate that the building movement from the PMMA window plane had a minor effect.

Moreover, the MEMS data indicate that the non-plane-strain behaviour of the building models had a minor effect on the displacements of the front and back façade wall. As can be seen from Figure 3.1b, the building model does not extend to the full width of the strong box, and distinct strip footings were replicated. These features were intentional, as real buildings are not infinite and solid in the direction parallel to the tunnel axis. The desire to simulate a completely plane-strain building was not the objective, because it was not the aim to simulate an unrealistic solid building. While the tunnelling-induced soil displacements between the strip footings could not be measured with the used experimental setup, the data in Figure 5.10 shows a minor rotation of the building models from the PMMA window. From this data it can be deduced that the front and back façades experienced similar ground displacements, and
5.3 Boundary effects

Fig. 5.10 Rotation of building models out of PMMA plane along tunnel volume loss.
the realistic non-plane-strain building layout had a minor influence on the schematic tunnel excavation simulation.

5.4 Conclusions

This chapter identified and quantified experimental challenges when modelling the response of surface structures to tunnelling in sand. Non-uniform ground displacements during centrifuge acceleration were monitored as a result of the adopted tunnel excavation simulation technique and the position of the building model relative to the tunnel. A quantification of the structure response during spin-up highlighted that notable building distortions can be observed as the g-level increases. The deflection ratio prior to tunnel excavation may be significant, and should theoretically not be ignored when evaluating the volume loss at which cracking occurs in centrifuge tests. In addition, the horizontal strain induced by spin-up can be larger than those induced by tunnel excavation. These findings underline the need to consider the spin-up phase when studying the mechanisms of tunnelling on realistic surface structures, and provide a base for further centrifuge modelling research dealing with the effects of tunnel construction on buildings. In addition, these findings highlight the potential of existing strain prior to tunnelling under real buildings, though these strains are extremely difficult to predict in reality. Finally, boundary effects that might influence the image-based deformation measurements were also quantified and showed negligible impact.
Chapter 6

The influence of a surface structure on tunnelling-induced subsidence

In urban tunnelling projects, surface buildings interact with tunnelling-induced ground movements. Understanding this interaction is crucial when predicting the behaviour of buildings above tunnelling works. However, much uncertainty still exists about the impact of structures on tunnelling subsidence and thus current design practice is widely based on empirical methods that neglect this soil–structure interaction. To refine current modelling assumptions and reduce uncertainty, more detailed knowledge of the influence of buildings on tunnelling-induced ground displacements is needed. This chapter investigates the interaction mechanisms between surface structures and tunnelling-induced ground movements. Although numerous studies have considered this soil–structure interaction problem, previous experiments have neglected important building characteristics, and field data inherently contains numerous uncertainties related to the soil, the structure and the tunnelling procedure. Consequently, interpretation of results and validation of computational models can be problematic. The following therefore focuses on the effect of building characteristics on tunnelling-induced soil displacements.

6.1 Background

Buildings above tunnel excavation are subject to ground movements that may induce unacceptable building damage. For greenfield tunnelling, where no buildings are present, there is a good understanding of the associated ground displacements. Nearby structures, however, interact with tunnelling-induced ground displacements and thus add complexity. Much un-
The influence of a surface structure on tunnelling-induced subsidence

certainty still exists on the impact of surface structures on ground displacements caused by underground excavation.

Ground movements associated with tunnelling works are most commonly predicted using empirical methods (Mair and Taylor, 1997). Typically, settlements transverse to the tunnel are described by a Gaussian curve (Peck, 1969; Schmidt, 1969). Applying this method requires empirical assumptions about the magnitude of ground loss, $V_l$, and the width of the settlement profile, which is frequently characterised by the trough width parameter, $K$. For tunnels in drained conditions (i.e. sand and gravel), numerous authors (Celestino et al., 2000; Jacobsz, 2002; Marshall et al., 2012; Vorster, 2005) noted considerable differences in observed settlements compared to the assumed Gaussian distribution. This can be explained by the chimney-like deformation mechanism frequently observed in drained soil conditions, which becomes prevalent when tunnelling-induced deformations increase (Marshall et al., 2012). Vorster et al. (2005) and Marshall et al. (2012) showed that modified Gaussian curves provide a better fit to transverse settlement troughs above tunnels in dense sand.

A significant limitation of this empirical method, as described above, is that it neglects any interaction between ground movements associated with tunnelling and the built environment. On the contrary, field data (e.g. Breth and Chambosse, 1974; Farrell et al., 2011; Frischmann et al., 1994; Viggiani and Standing, 2001), numerical studies (Amorosi et al., 2014; Franzius et al., 2006; Giardina et al., 2015a; Goh, 2011; Melis and Rodriguez Ortiz, 2001; Potts and Addenbrooke, 1997; Son and Cording, 2007; Yiu et al., 2017) and experimental investigations (Al Heib et al., 2013; Caporaletti et al., 2005; Farrell and Mair, 2010, 2011; Shahin et al., 2011; Taylor and Grant, 1998; Taylor and Yip, 2001) showed that surface structures notably affect the ground movements beneath them. A quantification of the variation of the main input parameters of the empirical method caused by nearby buildings is, however, still lacking, and can lead to uncertainty when assessing tunnelling-induced subsidence in the urban environment.

The purpose of this chapter is to investigate the effect of the building characteristics (e.g. building-to-tunnel position, building layout, building length and façade openings) on the tunnelling-induced ground displacements. This involves an evaluation of the effect of these building features on the vertical and horizontal ground response, the settlement trough shape and the development of shear and volumetric strains in the soil.
6.2 Tunnel stability

Before focusing on the distinct effects of building features on the soil response, the tunnel pressure during the tunnel excavations is first reviewed. Figure 6.1 compares the normalised tunnel pressure, $\sigma_{t,n}$, at tunnel axis and tunnel crown versus tunnel volume loss. This data was obtained by the tunnel PPT and the tunnel potentiometer readings during the tunnel excavation stage (Figure 3.4). The upper and lower bound solutions from Atkinson and Potts (1977) were determined using the parameters $C/D_t=1.34$, $\phi_{max}=44^\circ$ and $\gamma_s=15.75$ kN/m$^3$.

From Figure 6.1 a sudden drop of the tunnel pressure is evident as the schematic tunnel excavation started. After a tunnel volume loss of approximately 0.5% this decrease becomes considerably non-linear and the tunnel pressure slightly increases after approximately 4.0% of tunnel volume loss. The fair match between the tunnel pressure profiles of the different tests and the similar results reported by Farrell (2010) provides confidence in the performance of the tunnel excavation simulation.

For this reason, a direct comparison between the different tests to reveal building effects on the soil response is feasible. This enables to unlock new information regarding the impact of the building-to-tunnel position, the building length, the façade opening area and the building layout on this tunnel–soil–structure interaction problem, which is discussed below.
The influence of a surface structure on tunnelling-induced subsidence

6.3 Effect of building-to-tunnel position

The following sections discuss tests A, B and C and indicate how identical building models placed at different building-to-tunnel positions modified the tunnelling-induced ground displacements beneath them. Emphasis is placed on investigating the structure’s effect on the main input parameters of the empirical method to predict ground displacements subject to tunnelling works, such as the volume loss, the maximum vertical displacements above the tunnel centreline and the trough width. Mechanisms controlling the interaction between the buildings and the tunnelling-induced ground displacements are addressed. The term surface soil displacements describes near-surface soil displacements (i.e. \( z/z_t < 0.05 \), where \( z_t \) is the tunnel depth and \( z \) the vertical distance from the soil surface to the analysed soil depth) throughout this work, and are the closest possible measures near the soil surface using GeoPIV.

6.3.1 Vertical soil response

Empirical methods primarily focus on the vertical settlement profiles caused by the tunnel excavation procedure. Thus, alteration of the vertical soil response due to adjacent buildings is first explored.

Figure 6.2 illustrates that for each of the tests A, B and C the maximum vertical ground movements occurred close to the tunnel crown. With distance from the tunnel, the vertical soil displacements reduced. A clear impact of the building models on the soil displacements above the tunnel excavation is apparent in Figure 6.2. In particular, for the structures positioned eccentric to the tunnel (tests B and C), a widening of the vertical displacement contours in the region beneath the buildings was observed (Figure 6.2c and 6.2d). The symmetrically located structure (test A) had less impact on the tunnelling-induced deformation field (Figure 6.2b).

For the soil–structure tests, the maximum vertical ground displacement (\( S_v,\text{max} \)) reduced on average by about 25% compared to the greenfield test (Figure 6.2). These differences can be explained by the modified stress conditions in the soil body caused by the building weight (Franzius et al., 2004). Ground movements induced by tunnelling works depend on the soil stiffness, which is proportional to the mean effective soil stress. The higher soil stiffness in the soil–structure tests reduced the soil movements towards the tunnel. In addition, the lateral stress ratio (\( \sigma_h'/\sigma_v' \)) decreases due to the applied building load. Both these effects caused a reduced deformation field surrounding the tunnel excavation for the magnitude of tunnel volume loss (e.g. \( V_{lt} = 2.0\% \)). In granular material (e.g. sand), the change in confining stress due to the building surcharge directly influences the volumetric behaviour of the soil body (as will be further discussed in Section 6.3.4.4). An increase in the confining stress reduces the
6.3 Effect of building-to-tunnel position

Fig. 6.2 Building position effects on vertical displacement contours at $V_{lt} = 2.0\%$. 

The influence of a surface structure on tunnelling-induced subsidence

Fig. 6.3 Effect of building eccentricity (constant \( L \) and \( O \)) on soil volume loss, \( V_{l,s} \), versus tunnel volume loss, \( V_{l,t} \).

magnitude of dilation. This is particularly relevant close to the soil surface, where the change of the stress conditions due to the building load is significant. The effect of this complex interaction on the volume of the settlement troughs is discussed next.

6.3.1.1 Ground loss

As mentioned in Chapter 3, a large range of tunnel volume losses were simulated within a single experiment. The relationship between the monitored surface ground loss, \( V_{l,s} \), and the modelled tunnel volume loss, \( V_{l,t} \), is given in Figure 6.3a. Typical for tunnels in drained conditions, \( V_{l,s} \) is different from \( V_{l,t} \) which is caused by dilation due to shearing of the soil above the tunnel (Hansmire and Cording, 1985; Marshall et al., 2012). The experimentally obtained data generally agrees with the relationship proposed by Marshall et al. (2012) for tunnelling in the same ground conditions (Figure 6.3a). Figure 6.3a shows that the difference between the surface and tunnel volume loss increases with tunnel volume loss, indicating increased dilation.

Figure 6.3a indicates that the building position influenced the ground loss. While the values of \( V_{l,s} \) below buildings positioned eccentrically to the tunnel (tests B and C) were in fair agreement with the greenfield \( V_{l,s} \), an identical building located symmetrically to the tunnel (test A) resulted in notably lower \( V_{l,s} \) values as \( V_{l,t} \) exceeded 1.5%. This observation can be related to two sources: (1) the embedment at the building ends caused earlier local shear failure in the soil, causing increased dilation; (2) the loss of contact between the soil surface and the central region of the structure (Farrell and Mair, 2010, 2011) causing reduced confinement of the soil along the building centre, which also may have increased dilation effects. By contrast, the eccentric buildings reduced the development of shear strains above the right-hand tunnel
6.3 Effect of building-to-tunnel position

shoulder which resulted in a considerable amount of contraction beneath the buildings in tests B and C. The volumetric soil behaviour observed in the centrifuge tests is further discussed in Section 6.3.4.4. Nevertheless, at volume losses frequently reported in case histories ($V_{l,s} < 1.5\%$) nearby structures showed only a minor impact on $V_{l,s}$.

The effect of building weight contributes to ground loss results. As reported by Franzius et al. (2004), the building load increases the mean effective stresses in the soil body, which then leads to greater soil stiffness and strength. For this reason, the ground movements surrounding the tunnel reduced compared to the greenfield due to building weight effects, as was discussed above. This observation explains the lower $V_{l,s}$ values at half of the tunnel depth (Figure 6.3b). The pivotal role of the building weight in this tunnel–soil–structure interaction problem was also pointed out by Giardina et al. (2015a) and Bilotta et al. (2017).

The volume loss values shown in Figure 6.3 were derived by numerical integration of vertical soil displacement profiles that were obtained by image-based measurements. Hence, boundary effects caused by the PMMA window and the non-plane strain building model might affect the derived ground loss values. Consequently, a variation in ground loss with distance from the PMMA plane could theoretically be obtained. The experimental setup did not allow to measure deformations of the entire surface of the soil model. For this reason, the influence of the buildings on the ground loss could only be quantified in the PMMA plane.

The results below are given for a tunnel volume loss of 2.0%, which equals a greenfield soil surface volume loss of about 1.6% (Figure 6.3a). This volume loss is similar to the theoretically derived maximum surface volume loss (approximately 1.5%) for tunnels in sand with equal $C/D_t$ ratio and tunnel diameter (Vu et al., 2016). Also a frequently applied upper bound design value to assess potential building damage caused by shotcrete lining (SCL) tunnels is a surface volume loss of 1.5%.

6.3.1.2 Vertical surface soil displacements

Figure 6.4 relates the vertical surface soil displacements of the building tests to the greenfield surface settlements for tunnel volume losses of 0.5%, 1.0%, 2.0% and 4.0%. Deviations from the typical greenfield trough due to the soil–structure interaction are clearly visible. Although the modifications of the greenfield ground displacements were observed at each volume loss step, the alterations become more distinct as volume loss increases.

Structures located asymmetric to the tunnel increased the vertical settlements beneath the entire extent of the building (Figure 6.4b and 6.4c), whereas test A caused greater vertical soil displacement only beneath the building corners (Figure 6.4a). This finding for test A is related to a loss of contact between the centre of the building and the soil surface, resulting in building
Fig. 6.4 Building position effects on vertical surface soil displacement profiles at $V_{l,t} = 0.5\%$, 1.0\%, 2.0\% and 4.0\%. Greenfield (GF) profiles (Farrell, 2010) are plotted as reference.
6.3 Effect of building-to-tunnel position

Fig. 6.5 Effect of building eccentricity on vertical surface soil displacement profiles at $V_{l,t} = 2.0\%$ for tests A, B and C. Greenfield (GF) surface soil settlement profiles (Farrell, 2010) are plotted as reference.

Load redistribution to the building corners where the structure embedded into the soil surface. Similar results were reported by Farrell and Mair (2010, 2011) and Potts and Addenbrooke (1997) for rigid structures with zero eccentricity.

Test C also showed an embedment of the left building corner into the soil surface, which was triggered by a rigid body rotation of the building model towards the tunnel. For test A, the significant reduction of the vertical soil displacements directly above the tunnel can partly be explained by the reduction of the confining soil stress below the surface at this location, which results in a higher degree of dilation. Another possible reason for the observed lower vertical soil settlements above the tunnel centreline is the initial increase in soil stiffness and strength due to the building weight, which reduced the soil displacement field induced by the tunnel excavation, as discussed above. Reduced vertical surface settlements above the tunnel centreline due to adjacent buildings were also noticed in centrifuge modelling studies (Caporaletti et al., 2005; Taylor and Yip, 2001) and computational models (Franzius, 2003; Potts and Addenbrooke, 1997).

For tests A and C, a significant change of the rate of the vertical surface settlements was found in the regions where the building corners embedded (Figure 6.5a and 6.5c). More flexible neighbouring structures constructed in these regions or buried services might suffer severe damage. A complete recovery of the greenfield settlement curve, as reported by Potts and Addenbrooke (1997), was not observed due to the previously mentioned differences in the tunnelling-induced displacement field caused by the initially higher soil stiffness in the building tests. However, beyond approximately 100 mm from the edge of the structure (i.e. half of building length) the measured settlements were often in reasonable agreement with the greenfield profiles.
The influence of a surface structure on tunnelling-induced subsidence

Fig. 6.6 Effect of building eccentricity (constant $L$ and $O$) on the ratio between maximum surface soil settlements, $S_{v,max}$, and maximum greenfield surface soil settlements, $S_{v,max}^{GF}$, versus tunnel volume loss, $V_{lt}$.

Increased vertical surface displacements compared to the greenfield test were measured when the structure was placed so that one building corner was coincident with the tunnel centreline (Figure 6.5c). As previously mentioned, this effect was caused by a rigid rotation of the structure towards the tunnel, which results in embedment of the left corner of the building. Greater surface soil settlements caused by structures in a similar location relative to the tunnel were observed in the field (Bilotta et al., 2017; Farrell et al., 2011), in centrifuge experiments (Taylor and Grant, 1998; Taylor and Yip, 2001) and in numerical studies (Amorosi et al., 2014; Liu et al., 2001).

Figure 6.6 quantifies the change in maximum soil settlements caused by the presence of the building models by normalising them to the greenfield maximum vertical displacements. While noise affects the image-based measurements at low tunnel volume loss, a clear trend was observed above a tunnel volume loss of about 1.0%. Between 1.0% and 2.0% of tunnel volume loss, the ratio between the vertical surface settlements above the tunnel centreline and the corresponding greenfield settlements was about 0.8, 0.9 and 1.1 for tests A, B and C. These findings confirm that the location of the structure has a direct impact on the maximum surface soil settlement caused by tunnel excavation.

6.3.2 Surface trough width

After exploring the volume and maximum vertical displacements of the surface settlement troughs, this section focuses on alterations of the surface trough shape due to the soil–structure interaction. Vorster (2005) and Marshall et al. (2012) showed that modified Gaussian curves can provide a better fit to transverse soil settlements profiles above tunnels in sand compared to the more widely used Gaussian curves. This finding was expected due to the additional degree
of freedom of the modified Gaussian curve. Although employing a modified Gaussian curve adds complexity, its better fit is more adequate to investigate the variations of the settlement trough shape when tunnelling in sand.

Historically, the width of a Gaussian curve is analysed by the horizontal distance, \( i \), from the tunnel centreline to the point of inflection of the vertical settlement profile. The widely used trough width parameter, \( K \), is derived by normalising \( i \) with \( (z_t - z) \). As pointed out in Chapter 2, Marshall et al. (2012) suggested a framework to quantify the trough width of modified Gaussian curves. The authors introduced the parameter \( x^* \), which is the offset from the tunnel centreline where the vertical displacement of the modified Gaussian curve is 0.606 of \( S_{v,max} \). This approach is similar to obtaining the point of inflection for a Gaussian curve because at the point of inflection \( S_v = 0.606S_{v,max} \) also holds. \( K^* \) is then determined by dividing \( x^* \) by \( (z_t - z) \). Although, the trough width \( K^* \) is only qualitatively comparable to \( K \), \( K^* \) enables a valuable approach to evaluate the width of settlement profiles.

Modified Gaussian curves were fit to the data to determine the trough width parameters \( K^* \). Asymmetric settlement profiles were observed for the tests with structures located asymmetric to the tunnel centreline (tests B and C), as can be seen in Figure 6.2. Thus, to quantify the immediate effect of the surface structures on the trough shape, the curve fitting was carried out for the right hand side of the settlement troughs of tests B and C. Figure 6.7 presents the obtained \( K^* \) values along tunnel volume loss. The settlement troughs for the soil–structure configurations were wider than the greenfield conditions, which is particularly evident for \( V_{l,t} \) greater than 1.5%. A range of widely observed greenfield trough width parameters, \( K \), for tunnels in sand is given for reference (Mair and Taylor, 1997).

The structure with zero eccentricity (test A) caused the highest trough width parameter while the buildings placed at an offset from the tunnel resulted in a minor widening. A reduction of \( K^* \) as \( V_{l,t} \) increases was observed for each test, and indicates the typical trough narrowing (i.e. chimney effect) for tunnels in sand when ground displacements increase.

The noted widening mechanism caused by the surface structures can be related to the building weight. As previously highlighted, the building load alters the stress regime in the soil prior to tunnel excavation compared to the greenfield configuration. The greater confining stresses can be compared with a higher soil cover above the tunnel, and tunnel scenarios with higher \( C/D_t \) ratio result in wider soil settlement profiles than shallower tunnels (Marshall et al., 2012). Wider surface settlement troughs due to nearby buildings were also observed in computational studies (Amorosi et al., 2014; Franzius et al., 2004; Liu et al., 2001; Potts and Addenbrooke, 1997) and in experimental investigations (Caporaletti et al., 2005; Farrell, 2010; Taylor and Yip, 2001). Previous case histories also indicated trough widening due
The influence of a surface structure on tunnelling-induced subsidence

Fig. 6.7 Building position effects on surface trough width parameter $K^*$ versus tunnel volume loss, $V_{t,t}$. Greenfield (GF) data (Farrell and Mair, 2010) is given as reference. Literature values for $K$ in sand are indicated $\dagger$ (Mair and Taylor, 1997).

to soil–structure interaction, though the displacement data was generally obtained through monitoring points located on the building façades (e.g. Frischmann et al., 1994; Lu et al., 2001; Viggiani and Standing, 2001). An implication of the observed widening of tunnelling-induced vertical settlement troughs at surface level due to adjacent buildings is that building damage assessments applying typical trough width parameters derived in greenfield conditions might lead to conservative predictions.

6.3.3 Horizontal surface soil displacements

The focus of the chapter thus far has been the vertical soil response, but the effect of buildings on horizontal ground movements is also important for the assessment of surface structures.

Figure 6.8 shows that the horizontal surface ground movements considerably deviate from the greenfield displacements due to a nearby building. In particular, the maximum horizontal displacements beneath the building models are significantly reduced by the shear transfer between the soil and the structure. Similar to the effects on the vertical displacement profiles, these modifications were observed for different magnitude of tunnel volume loss.

The eccentricities of the building models play a vital role on the horizontal ground displacements caused by tunnel excavation. Buildings placed close to the tunnel (i.e. tests A and C) significantly restrained the surface horizontal ground displacements over the entire tunnelling-induced displacement field (Figures 6.9a and 6.9c). When the structure was placed
Fig. 6.8 Building position effects on horizontal surface soil displacement profiles at \( V_{lt} = 0.5\%, 1.0\%, 2.0\% \) and 4.0\%. Greenfield (GF) profiles (Farrell, 2010) are plotted as reference.
The influence of a surface structure on tunnelling-induced subsidence

in the hogging region (test B), the structure did not affect the left hand side of the tunnelling-induced displacement field (Figure 6.9b), but the displacements beneath the structure were significantly affected and became relatively uniform.

It is notable that the horizontal soil movements beneath the structure in test A are non-uniform, changing significantly near the centre of the structure, which can be explained by a loss of contact between the soil and the structure (Figure 6.8a). This soil–structure gap (Farrell and Mair, 2010, 2011; Giardina et al., 2014) developed at a $V_{lt}$ of approximately 1.4% (Section 7.3.3) and enabled free horizontal soil movement (similar to the greenfield case) in the zone close to the tunnel centreline. The entire length of the buildings placed asymmetrically to the tunnel (tests B and C) remained in full contact with the soil surface; the horizontal ground displacement component was restrained, and relatively uniform, over the entire building extent (Figure 6.9b and 6.9c).

In all tests, the horizontal displacements were relatively uniform beneath the structures, apart from where the soil–structure gap formed in Test A. This finding can be explained by friction in the soil–structure boundary; the structure dragged the soil beneath it. Consequently, greater horizontal ground movements than measured in the greenfield case were observed towards the right-hand side of the buildings B and C (Figures 6.9b and 6.9c). Overall, these results point out that significantly smaller horizontal strains were transferred to the buildings than assuming the structures are following the greenfield movements. Case studies identified similar findings (e.g. Burland et al., 2004; Farrell et al., 2011; Viggiani and Standing, 2001).

Horizontal displacements at the building foundation level are often determined by assuming that the resultants of the vertical and horizontal displacements point towards a single point. For undrained soil conditions, this so-called ‘point-sink’ is often assumed to be at the tunnel axis (Attewell, 1978; O’Reilly and New, 1982) or at a point $0.175z_t/0.325$ below the tunnel
6.3 Effect of building-to-tunnel position

axis (Taylor, 1995b). Further, modifications to estimate the horizontal building displacements from the vertical displacements were provided by Attewell and Yeates (1984) and New and Bowers (1994). However, Farrell (2010) pointed out that existing methods such as the ribbon-sink method (New and Bowers, 1994) provided a poor fit to centrifuge test data in sand and showed that the focal point of the resultant displacement vectors is a function of the $C/D_t$, $V_{l,t}$ and $x$.

Figure 6.10 illustrates the impact of a nearby structure on the displacement vector direction at $V_{l,t} = 2.0\%$. The dashed lines are extrapolated from the measured vertical and horizontal displacements in order to assess whether a clear focal point exists for the displacement vectors. Emphasis is placed on the soil level just beneath the building. The erroneous displacement vectors occurring at great values of $|x|$ are most likely due to both very small displacements and noise in the GeoPIV data.

From Figure 6.10b it is apparent that the greenfield displacement vectors in the region of approximately $-1.5D_t/2 < x < 1.5D_t/2$ point towards a distinct focal point marginally below the tunnel crown. For lower $V_{l,t}$, the depth of the focal point was greater and gradually decreased as $V_{l,t}$ developed. Outside this region, a specific focal point was not observed, but the resultants are notably more vertical. This observation implies that the horizontal displacements significantly reduce as $|x|$ increases.

The influence of the soil–structure interaction on the displacement vector orientation is clearly visible from Figure 6.10. For test A, the displacement vectors beneath the building are directed to points that are significantly lower compared to the greenfield equivalents. Specifically, in the regions of the left and right building corners this restraining effect of the horizontal soil displacements is significant. The displacement vectors close to the centre of the building and directly above the tunnel indicate a focal point, which is lower than the greenfield one. In this region, the building lost contact to the soil and horizontal displacements evolved, though smaller than in the greenfield case.

For the asymmetric building-to-tunnel scenarios (tests B and C), the horizontal displacements beneath the structure are notably affected by the presence of the buildings (Figures 6.10c and 6.10d). For both tests the displacement vectors beneath the building are directed at much steeper angles than in the greenfield, which indicates much greater vertical displacement components than horizontal ones. Clearly defined point-sinks could not be observed for the asymmetric building positions. Moreover, from the data shown in Figure 6.10 it can be followed that existing methods characterising greenfield soil displacements (e.g. point-sink or ribbon-sink) are not suitable to describe soil-structure interaction phenomena.
The influence of a surface structure on tunnelling-induced subsidence

Fig. 6.10 Effect of building eccentricity on surface displacement vectors at a tunnel volume loss of 2.0%. Greenfield (GF) data (Farrell, 2010) is given as reference.
6.3 Effect of building-to-tunnel position

6.3.4 Subsurface soil displacements

In addition to the discussed effects on the surface ground level, the building impact on tunnelling-induced ground displacements below the soil surface is of key importance to assess potential damage on buried infrastructures.

6.3.4.1 Vertical subsurface soil response

Figure 6.11 compares surface soil settlement troughs ($z/z_t = 0.03$) to subsurface soil settlement profiles ($z/z_t = 0.13$ and $0.26$). Greenfield vertical soil movement profiles (Farrell, 2010) are presented for reference. To visualize surface and subsurface troughs in a single plot, the vertical displacements for the subsurface profiles are shifted uniformly by $0.2$ mm and $0.4$ mm, as illustrated in Figure 6.11. A significant reduction of the soil–structure effects with depth can be observed in Figure 6.11. Particularly, embedment effects of building corners diminished with depth, and at a depth of $z/z_t = 0.26$, only minor alterations can be seen.

6.3.4.2 Horizontal subsurface soil response

Vertical profiles of horizontal soil movements at a horizontal distance, $x$, of $70$ mm from the tunnel centreline are presented in Figure 6.12 to explore the soil–structure interaction zone influencing the horizontal ground displacements. At $x = 70$ mm contact between the building foundations and the soil surface was maintained for all the building experiments. Within the top soil horizons ($z/z_t < 0.13$) reduced horizontal soil displacements were measured for all tests; this indicates a significant restraint of the horizontal ground movements due to the interaction effect. Between a depth of $z/z_t = 0.13$ and $0.26$ the horizontal displacements were in reasonable agreement with the greenfield ground movements. This observation is in line with the subsurface vertical displacements discussed in the section above, and similar observations were reported by Standing (2001) and Farrell and Mair (2011). However, structures that showed embedment of building corners (i.e. tests A and C) may increase the constraining depth compared to test B where embedding was not observed. The overall slightly lower horizontal ground displacements obtained in the tests with the building models are again a result of the building weight, causing a change in the stress regime in the soil body and are consistent with Franzius et al. (2004).

6.3.4.3 Subsurface trough width

Variations in trough width with depth play an important role when predicting tunnelling-induced settlement effects on buried infrastructures such as piles or pipelines. Marshall et al.
The influence of a surface structure on tunnelling-induced subsidence

Fig. 6.11 Effect of building position on soil settlements at different depths ($z/z_t = 0.03, 0.13$ and $0.26$) at $V_{lt} = 2.0\%$. Greenfield data (Farrell, 2010) is given as reference. For visualisation the settlements at a depth of $z/z_t = 0.13$ and $z/z_t = 0.26$ are increased by a vertical offset of 0.2 mm and 0.4 mm, respectively.
6.3 Effect of building-to-tunnel position

Fig. 6.12 Effect of building eccentricity on vertical profiles of horizontal soil displacements at \( x = 70 \) mm and \( V_{l,t} = 2.0\% \). Greenfield data (Farrell, 2010) is given as reference.

(2012) identified that the trough width for tunnels in sand varies with depth, tunnel volume loss and \( C/D_t \). To study the impact of the building models on the shape of the subsurface vertical displacement profiles, the Marshall et al. (2012) framework is adopted, similar to the investigation at surface level. Figure 6.13 presents the obtained \( K^* \) values along \( z \) with published trends of \( K \) with depth. Wider troughs were generally observed when building models were present, and the symmetric structure tended to cause the widest trough. The currently available methods to predict the trough width for a greenfield case generally showed poor agreement with the greenfield experimental data as well as for the soil–structure tests. For \( V_{l,t} = 0.5\% \) (Figure 6.13a), the obtained \( K^* \) values for tests A and B indicate a minor widening of the trough with depth, similar to methods 1 and 3, while the greenfield test and test C showed no widening with depth, as in method 2.

For \( V_{l,t} = 2.0\% \) (Figure 6.13b), the soil–structure tests indicate that the trough widths differ near the surface and then became rather similar with increasing depth, although \( K^* \) remained greater than measured for the greenfield configuration. Generally, the trough width decreased or remained relatively uniform with depth for all tests, indicating a trend most similar to method 2 (Figure 6.13b). These observations highlight that in drained soil conditions \( K^* \) varies considerably with \( V_{l,t} \) and that the previously proposed methods are less appropriate as \( V_{l,t} \) increases.

6.3.4.4 Shear and volumetric strains

Figure 6.14 illustrates the developed engineering shear strains, \( \gamma_{xz} \), at a tunnel volume loss of 2.0\% which can be written as

\[
\gamma_{xz} = \sqrt{(\varepsilon_{xx} - \varepsilon_{zz})^2 + (2\varepsilon_{xz})^2}
\]  

(6.1)
The influence of a surface structure on tunnelling-induced subsidence

Fig. 6.13 Effect of building position on trough width parameter \(K^*\) versus soil depth for (a) \(V_{l,t} = 0.5\%\) and (b) \(V_{l,t} = 2.0\%\). Other published methods are: (1) Mair et al. (1993) with parameters for sand \((K_s = 0.35, \delta_i/\delta_z = 0.26)\) derived by Jacobsz (2002), (2) a constant \(K = 0.35\) for tunnels in sand (Mair and Taylor, 1997; O’Reilly and New, 1982) and (3) Moh et al. (1996) for silty sand \((m = 0.4)\).

where \(\varepsilon_{xx}\) and \(\varepsilon_{zz}\) are the principal strain components in the \(x\) and \(z\) planes respectively and \(\varepsilon_{xz}\) the shear strain. Greenfield data (Farrell, 2010) is given for reference. For all experiments, the main shear strains developed from the tunnel shoulders and formed a bulb above the tunnel crown, which correlates well with the collapse mechanism for an upper bound solution suggested by Atkinson et al. (1975).

It is apparent from Figure 6.14 that the presence and location of surface structures affect the activation of shear strains within the soil. The magnitude of shear strains obtained in the soil–structure configurations is lower compared to the greenfield test. This was to be expected because the building acted as a surcharge and thus increased the soil confinement, which governs the amount of shear strains. Equal findings were reported by Farrell and Mair (2010) when comparing the shear strain development above tunnels with different \(C/D_t\). The smaller shear strains observed in the soil–structure tests (Figures 6.14b, 6.14c and 6.14d) caused less dilation in the soil than the greenfield case, as can be seen from Figure 6.15. This finding partly explains the rather equal volume of the surface settlement troughs (Figure 6.3a) although the ground movements in close proximity to the tunnel were lower in the tests with the buildings.

When the structure is placed symmetrically to the tunnel (Figure 6.14b) pronounced shear strains can be observed below the left and right edges of the building. This localised activation of shear zones directly beneath the building corners explains the embedment of the structure at its corners, which was previously mentioned. However, less volumetric contraction, shown in Figure 6.15b, was observed for test A compared to the other tests with a nearby building. This result explains the observed lower soil settlements for test A and the smaller \(V_{l,s}\) values.
6.3 Effect of building-to-tunnel position

(a) Greenfield (Farrell, 2010).

(b) Test A: $e/L=0$, $L/H=2.2$, $O=20\%$, $G=3D$.

(c) Test B: $e/L=0.8$, $L/H=2.2$, $O=20\%$, $G=3D$.

(d) Test C: $e/L=0.5$, $L/H=2.2$, $O=20\%$, $G=3D$.

Fig. 6.14 Building position effects on engineering shear strain, $\gamma_{xz}$, at $V_{lt} = 2.0\%$. 

153
The influence of a surface structure on tunnelling-induced subsidence

(a) Greenfield (Farrell, 2010).

(b) Test A: $e/L=0$, $L/H=2.2$, $O=20\%$, $G=3D$.

(c) Test B: $e/L=0.8$, $L/H=2.2$, $O=20\%$, $G=3D$.

(d) Test C: $e/L=0.5$, $L/H=2.2$, $O=20\%$, $G=3D$.

Fig. 6.15 Building position effects on volumetric strain, $\varepsilon_{\text{vol}}$, at $V_{l,t} = 4.0\%$. ($\varepsilon_{\text{vol}} < 0$ indicates dilation)
6.4 Effect of building characteristics

Thus far, this chapter was concerned with the effect of different building-to-tunnel positions on the soil response to tunnel excavation. This section discusses the impact of different building dimensions (e.g. length, layout and façade opening area) on this tunnel–soil–structure boundary problem. The subsurface soil behaviour was less affected by the different building features. For this reason, the following sections primarily focus on the surface soil displacements.

6.4.1 Influence of building characteristics on the vertical soil response

The vertical soil displacements of the tests D, E, F and G at $V_{lt}$ of 0.5%, 1.0%, 2.0% and 4.0% are shown in Figure 6.16. As expected, the different building configurations altered the typical greenfield settlement profiles, and modifications increased as $V_{lt}$ developed. For the tests F and G, the onset of building damage was observed at $V_{lt} > 2.0\%$, which had a significant
The influence of a surface structure on tunnelling-induced subsidence

Figure 6.16 Building characteristic effects on vertical surface soil displacement profiles at \( V_{l,t} = 0.5\% , 1.0\% , 2.0\% \) and 4.0\%. Greenfield (GF) profiles (Farrell, 2010) are plotted as reference.

impact on the measured soil response. Chapter 7 and specifically Section 7.5.3 address these effects in more detail.

Figure 6.17 shows more in detail the vertical soil response directly beneath the structures at \( V_{l,t} = 2.0\% \). The corresponding vertical soil displacements of test B and C are replotted to enable a better discussion of the effects of building features. A comparison between the tests B (Figure 6.17a) and D (Figure 6.17c) reveals that an increase of the amount of openings from 20\% to 40\% of the area of the building façade slightly reduced the modification of the greenfield soil movements underneath the structure D. This is likely caused by the greater opening percentage, which increased flexibility. This trend was also observed for the longer structures (compare test E with test F in Figure 6.17). Structure F also caused substantially greater soil settlements beneath the left-hand building edge, demonstrating that higher flexibility in the hogging region caused increased embedment above the tunnel. By contrast, for structure E, which had 20\% openings but was otherwise similar to test F, the reduced curvature of the vertical soil settlement profile beneath the building indicates a less flexible response, which then decreased embedment above the tunnel (Figure 6.17d). These findings confirm that the amount of window area changes a building’s flexibility, which then affects soil displacement.
6.4 Effect of building characteristics

Fig. 6.17 Building characteristic effects on the vertical surface soil displacement profiles at $V_{lt} = 2.0\%$. Greenfield (GF) surface soil settlement profiles (Farrell, 2010) are plotted as reference.
The influence of a surface structure on tunnelling-induced subsidence

**Fig. 6.18** Effect of building characteristics on the ratio between maximum surface soil settlements, \( S_{v,\text{max}} \), and maximum greenfield surface soil settlements, \( S_{v,\text{GF, max}} \), versus tunnel volume loss, \( V_{l,t} \).

The building length also directly affected the modification of the vertical soil displacements. An increase of the building length towards the right-hand side (compare test E with test C) caused a soil displacement beneath test E that is less flat than observed for test C. This observation confirms that the length of the structure also plays a vital role in this soil–structure interaction problem. From Figure 6.17d it can be seen that the distinct hogging and sagging deformations, typical for a greenfield settlement trough, are less defined due to the building’s effect on the soil displacements.

A notable effect of the building layout can be observed when comparing Figure 6.17e with Figure 6.17f. For the isolated façade configuration (test G), the measured soil displacements nearly matched the greenfield profile. More specifically, a marginal increase of the vertical soil displacements can be seen in the hogging region while the soil displacements closer to the tunnel (i.e. sagging region) were in good agreement with the greenfield (Figure 6.17f). On the contrary, the structure in test F with front, end, rear and intermediate walls showed a distinct modification of the sagging region, and a significant embedment of the left building corner. This suggests that differences in the building dimensions out of plane-strain potentially have a substantial effect on the soil response.

Figure 6.18 relates the maximum vertical soil displacement of the soil–structure interaction tests with different building characteristics to the corresponding maximum vertical greenfield displacements. For the structures located in the hogging region (tests B and D), an increase of the window area had a minor effect on the \( S_{v,\text{max}}/S_{v,\text{GF, max}} \). However, an increase of the opening percentage led to a substantial increase for the longer structures (compare test E and test F). An increase of the building length towards the tunnel (tests E, F and G) also significantly increased the vertical soil displacements.
6.4 Effect of building characteristics

Figure 6.19 compares the surface soil volume loss, $V_{ls}$, to the corresponding tunnel volume loss, $V_{lt}$. Similar results as observed for the tests A, B and C (Figure 6.3) were observed. However, for the tests E and F the $V_{ls}$ values were slightly exceeding the greenfield equivalents (Figure 6.19a). This can be attributed to volumetric contraction beneath the entire building length, and implies that both the building load and the building length affects $V_{ls}$. At $z/z_t = 0.5$ this influence of the building length could not be observed (Figure 6.19b). For practically relevant volume losses (i.e. $V_{ls} < 1.5\%$) rather similar values of $V_{ls}$ were observed for all tests.

6.4.1.1 Ground loss

Figure 6.20 shows the relationships between building length, area of façade openings and building layout on $K_s^*$. For the shorter buildings ($L/H = 2.2$), an increase of the window opening from 20% to 40% had little effect on the trough width $K^*$ because test D performed rather rigidly despite having 40% of openings. However, for the longer buildings ($L/H = 2.9$), doubling the window area significantly narrowed the settlement trough. In this case, the increased flexibility caused the settlement profile to deviate less from the greenfield equivalent. The isolated façade configuration (test G) performed nearly identical. Clearly, and as expected, the building position, length and amount of window openings need to be considered in combination. The observed widening of the settlement profiles due to an adjacent struc-
The influence of a surface structure on tunnelling-induced subsidence

*\( K_s^* = 0.25 - 0.45 \) (sand)†

\( z/z_t = 0 \)

0.5 1 1.5 2 2.5 3 3.5 4

0.2 0.4 0.6 0.8

Fig. 6.20 Effect of building characteristics on surface trough width parameter \( K^* \) versus tunnel volume loss, \( V_{t,t} \). Greenfield (GF) data (Farrell and Mair, 2010) is given as reference. Literature values for \( K \) in sand are indicated †(Mair and Taylor, 1997).

... is consistent with field data (e.g. Frischmann et al., 1994; Lu et al., 2001; Viggiani and Standing, 2001), although these investigations were based on façade monitoring points.

Although \( x^* \) and \( K^* \) provide a valuable approach to investigate the settlement trough width, these parameters are less suitable to estimate hogging and sagging modes of displacement profiles, which are defined by the inflection point of the associated settlement trough. The partitioning between hogging and sagging is particularly important for building damage assessment, for which Mair et al. (1996) proposed to separate buildings at the inflection point. Therefore, Figure 6.21 provides the variation of the inflection point, \( i_{mG,s} \), of the modified Gaussian curves fitted to the surface soil settlements for the entire series of centrifuge model tests. As expected, the data in Figure 6.21 indicates similar trough width trends as shown in Figures 6.7 and 6.20, but it emphasises that the soil–structure interaction has a significant impact on the position of the inflection point. This outcome implies that treating a building separately either side of the assumed greenfield inflection point might result in substantial uncertainty when predicting building performance to tunnelling subsidence.

6.4.2 Influence of building characteristics on the horizontal soil response

The effect of building characteristics on the horizontal soil response is explored next. Figure 6.22 presents the horizontal surface ground movements that complement the vertical ground movement profiles given in Figure 6.16. In all the tests with different building characteristics, the building models substantially restricted the horizontal ground movements beneath them.
6.4 Effect of building characteristics

In particular, the maximum horizontal displacements beneath the building models are significantly reduced by the shear transfer between the soil and the structure. Similar to the effects on the vertical displacement profiles, these modifications were observed for different magnitude of tunnel volume loss.

However, for test G, the horizontal displacements beneath the structure substantially increased as $V_{lt}$ developed (Figure 6.22d). Specifically, at $V_{lt} = 4.0\%$ the horizontal soil displacements considerably increased at $x \leq 100\ mm$, which can be attributed to building damage (Section 7.5.3). Similar, the notable change in the horizontal displacements beneath the structure of test F, evident in Figure 6.23e, can be related to the onset of building damage. Section 7.5.3 focuses on building damage.

Figure 6.23 plots the horizontal soil displacements beneath the building models at $V_{lt} = 2.0\%$. The data of the tests B and C are replotted to stress effects of the building variations. Although all the tests significantly restricted the horizontal soil displacements beneath them, it can be seen that the window opening area influenced the horizontal soil response. For all tests with 40% of window openings ($O = 40\%$), the profiles of horizontal soil displacements beneath the structures are marginally less uniform than compared to the tests with 20% openings (compare tests B and D or tests E and F). This implies that the structures with 40% openings are axially slightly more flexible, which reduces the restraining effect of the involved soil–structure interaction. The isolated façade test showed an even less uniform horizontal displacement profile (Figure 6.23f).
The influence of a surface structure on tunnelling-induced subsidence

Fig. 6.22 Effect of building characteristics on horizontal surface soil displacement profiles at $V_{l,t} = 0.5\%, 1.0\%, 2.0\%$ and $4.0\%$. Greenfield (GF) profiles (Farrell, 2010) are plotted as reference.
6.4 Effect of building characteristics

![Graphs showing building characteristic effects on horizontal surface soil displacement profiles at $V_{lt} = 2.0\%$. Greenfield (GF) surface soil settlement profiles (Farrell, 2010) are plotted as reference.]

**Fig. 6.23** Building characteristic effects on the horizontal surface soil displacement profiles at $V_{lt} = 2.0\%$. Greenfield (GF) surface soil settlement profiles (Farrell, 2010) are plotted as reference.
The influence of a surface structure on tunnelling-induced subsidence

The restraining effect of the buildings on the horizontal ground movements is further pointed out by plotting the horizontal soil displacements with depth at an offset from the tunnel centreline of \( x = 70 \) mm, as was done in Figure 6.24. For all the soil–structure interaction tests, buildings were present at that \( x \) position. Close to the soil surface, the restraining effect of the soil–structure interaction is clearly visible in Figure 6.24 for all the tests with different building features. However, at a soil depth between \( z/z_t = 0.13 \) and 0.26, the horizontal ground movements were in fair agreement with the greenfield displacements (Farrell, 2010; Standing, 2001). These findings are the results of the rough soil–structure interface and indicate that the restraining effect of the building on the horizontal soil movements is a rather shallow mechanism; slippage between the foundation and the soil surface was not observed. At \( z/z_t \geq 0.26 \), the slightly lower horizontal ground displacements measured in the soil–structure tests compared with the greenfield case can be attributed to the greater soil stiffness, as mentioned earlier. The variation of horizontal soil displacements with depth was unaffected by the differing building features.

**6.5 Conclusions**

Experimental data of a centrifuge test series on building response to a shallow tunnel excavation in dry, dense sand have been presented. Building features (e.g. a rough soil–structure interface, strip footings, intermediate walls and façade openings) were replicated by 3D print-
6.5 Conclusions

ing the building models in order to assess the influence of building characteristics including the building-to-tunnel position, the building layout, the façade opening area and the building length on the soil response. Results of the centrifuge experiments illustrated a significant deviation of the ground displacements due to adjacent buildings, from which the following conclusions can be drawn:

• Building weight causes a change of the stress regime in the soil body, which affects the tunnelling-induced ground movements (Bilotta et al., 2017; Franzius et al., 2004). The higher stress state and thus soil stiffness reduced the ground movements caused by the tunnel excavation. For the soil–structure tests, therefore, lower tunnelling-induced soil movements were measured compared to the greenfield case.

• A structure in close proximity to the tunnel excavation changes the volumetric behaviour of the soil above a tunnel and the tunnelling-induced displacement field. Similar building weight effects were also identified previously (Bilotta et al., 2017; Franzius et al., 2004; Giardina et al., 2015a). Consequently, the observed surface and subsurface volume loss values were affected by nearby structures. Long structures caused volumetric soil contraction beneath them and the surface volume loss values were greater than those of the greenfield. Structures placed symmetric to the tunnel reduced the surface volume loss while window openings showed a minor influence.

• Variations in the building location relative to the tunnel directly influenced the maximum surface settlements which confirms data in the literature (Amorosi et al., 2014; Bilotta et al., 2017; Liu et al., 2001; Potts and Addenbrooke, 1997; Taylor and Grant, 1998; Taylor and Yip, 2001). Buildings with zero eccentricity caused less surface settlement above the tunnel while vertical soil surface displacements exceeding the greenfield values were observed when the tunnel excavation passed directly underneath a building corner. Similar results were reported by Burd et al. (2000), Liu et al. (2001) and Bilotta et al. (2017). This phenomena reduced for the stiffer (i.e. $O = 20\%$) longer building tested.

• Building characteristics had a significant effect on the vertical soil response just beneath them. An increase of the façade opening area from 20% to 40% notably reduced the modification from the greenfield vertical ground movements. The increase of the opening percentage increased the building’s flexibility which then affected the soil response. Increasing the building length even further reduced the alteration of the greenfield profile, and for an isolated façade configuration rather similar surface soil displacements
than in the greenfield case were observed. This implies that the building stiffness plays a key role in this interaction problem and building characteristics such as the façade openings and the building dimensions are also important factors to consider.

• The presence of buildings widened the vertical soil settlement trough compared to the greenfield case. The widest trough was obtained for a structure with zero eccentricity and the trough became narrower as the distance between the building and the tunnel increased. An increase of the flexibility of the building, caused by a greater building length and/or a greater amount of window openings, reduced this widening effect. Field data of building displacement profiles above tunnels also confirm these findings (Frischmann et al., 1994; Lu et al., 2001; Viggiani and Standing, 2001).

• The absolute and differential horizontal ground displacements just beneath the surface structures were significantly restrained by the buildings. However, the horizontal greenfield soil movements were recovered at a soil depth between $z/z_t = 0.13$ and 0.26, which is in fair agreement with Standing (2001) and Farrell (2010). The building location influenced the magnitude of the horizontal ground displacements. Structures in asymmetric positions tended to have relatively uniform horizontal ground movements beneath them, with an approximate magnitude equal to the average horizontal ground displacements observed for the greenfield scenario. In the symmetric case, the structure reduced the horizontal displacement component throughout the building extent. Changes in the building length had little effect on the reduction of the horizontal ground movements. The restriction of the horizontal ground movements, however, is a function of the window opening area and/or the building dimensions parallel to the tunnel. Slippage between the soil and the structure was not observed.

• Localised soil–structure phenomena, such as building embedment and restraining of horizontal ground movements were constricted to the top soil horizons. It was found that these phenomena diminish between a depth of $z/z_t = 0.13$ and 0.26. Standing (2001) and Farrell and Mair (2010) reported similar findings. Building positions that notably activate shear strains beneath their corners tend to marginally increase the influence depth.

• The surface structures and their position relative to the tunnel have a significant effect on the development of shear bands in the soil body above the tunnel (Bilotta et al., 2017; Liu et al., 2001; Shahin et al., 2011) while the building characteristics had a minor influence. For all soil–structure cases, lower shear strains were observed than
in the greenfield scenario, which is related to the building weight effect (see above). The building position can lead to a serious concentration of shear strains close to the soil surface when a building corner is coincident with a potential shear band. This scenario can result in large vertical surface displacements, rotation and embedment of the building.
Chapter 7

Building response to tunnelling-induced subsidence

In this chapter the results of a series of centrifuge modelling tests investigating the response of complex surface structures to tunnelling-induced ground displacements are presented. The centrifuge tests investigated in Chapter 6 are again considered, but the focus is on the effect of building characteristics on the structural response rather than the effect of the building characteristics on the soil movements. Again, the building characteristics considered include different building-to-tunnel position, building length, façade opening percentage and building layout. Previous experimental data on the influence of these building features is scarce. Vertical and horizontal structural distortion was examined by typical building deformation parameters, after which the vital role of the building features in this tunnel–soil–structure interaction problem is addressed. Investigating the influence of these structural details is conducted at a global building scale as well as at a more local scale. Building damage in terms of cracking is then explored. Subsequently, a discussion of the suitability of treating a structure separately at either side of the greenfield inflection point is provided, after which the role of building characteristics on the governing mode of building deformation (i.e. shear or bending) is explored.

7.1 Background

An extensive literature review on the response of buildings to tunnelling-induced ground movements was provided in Chapter 2. This background section re-emphasis essential previous research that relates directly to the findings in this chapter. Various researchers (Franzius et al., 2006; Goh and Mair, 2011a; Potts and Addenbrooke, 1997; Son and Cording, 2005)
Building response to tunnelling-induced subsidence

identified the critical role played by the building stiffness, which enabled the formulation of design methodologies (i.e. relative stiffness methods) that consider soil–structure interaction effects. However, in these previous studies surface structures were often simplified as elastic beams and building characteristics (e.g. façade openings or non-linear material behaviour) were not considered. Consequently, uncertainty still exists about the impact of building features on this interaction problem. Further, the extent to which horizontal ground strains are transferred to buildings is still debated. As a consequence, predictions based on these relative stiffness methods (RSM) might differ, and in certain scenarios can lead to underestimation of building risk to tunnel excavation (Camós et al., 2014; De Jong et al., 2016; Giardina et al., 2017). While Chapter 8 provides a direct comparison between predictions of these RSMs and the experimentally obtained results, this chapter discusses the effects of building features on the building response to tunnelling-induced subsidence.

Widely accepted procedures to predict the response of surface structures to tunnelling-induced ground movements assume that a building located within both the hogging and sagging region of the settlement trough can be subdivided into its sagging and hogging parts, which are then analysed separately. Specifically, the LTSM (Boscardin and Cording, 1989; Burland et al., 1977; Mair et al., 1996) and the more recent approach by Goh and Mair (2011a) separately analyse the building parts either side of the greenfield point of inflection. Data of various cases studies (Frischmann et al., 1994; Mair, 2013; Viggiani and Standing, 2001) and centrifuge tests (Farrell, 2010; Ritter et al., 2017a), however, showed that buildings are generally not responding fully flexibly to tunnelling-induced settlements and thus considerably deviate from greenfield displacement profiles. For this reason, the soil–structure interaction notably alters the position of the inflection point compared to the greenfield case. This indicates that the widely accepted framework of assessing the hogging and sagging part of a building individually has limited accuracy for less flexible surface structures. Consequently, prediction methods applying this partitioning approach might underestimate potential building damage as was reported by Netzel (2009). This chapter evaluates the performance of this partitioning procedure.

A further major uncertainty, addressed in this chapter, is the governing mode of building deformation (i.e. shear or bending). Existing methods of assessing potential building damage due to tunnelling operation (Burland and Wroth, 1974; Franzius et al., 2006; Goh and Mair, 2011a; Potts and Addenbrooke, 1997) focus on the critical mode of deformation, which depends on the building geometry, i.e. length to height ratio \( L/H \) (Burland and Wroth, 1974; Pickhaver et al., 2010), and the presence of façade openings (Melis and Rodriguez Ortiz, 2001; Pickhaver et al., 2010; Son and Cording, 2007). For masonry structures, with a frequently used
Young’s modulus to shear modulus ratio, $E/G$, of 2.6 (Mair et al., 1996), structures exceeding an $L/H$ ratio of unity are reported to be more vulnerable to bending than to shear deformations (Burland and Wroth, 1974). By contrast, Boscardin and Cording (1989) related the angular distortion, which is a measure of shear deformations, in combination with the horizontal strain to building damage. This approach was followed by Son and Cording (2007), who showed that shear distortions control building damage of structures with a $L/H$ ratio of 3-10. Additionally, these authors reported $E/G$ values between 12 to 23 for masonry walls, which is in fair agreement with field observations reported by Cook (1994). This controversy about the critical mode of distortion leads to significant amount of uncertainty when assessing building response to tunnelling-induced settlements. For this reason, this part of the dissertation sheds new light on the importance of taking shear and bending distortions into account when evaluating building response to tunnelling.

### 7.2 Volume loss

As was pointed out in the previous chapter, the drained sand was observed to change volume within the soil body when the tunnelling procedure induced shear strains. Because of this, the volume of the settlement troughs vary with depth as was also observed in the field (e.g. Hansmire and Cording, 1985). The presence of existing structures (i.e. building models) caused further alteration of the volume of the vertical settlement profiles, as was shown in Figure 6.3a.

Building damage assessment is generally performed at the depth of the building foundation, and predefined volume loss limits are employed to assess tunnelling impacts on the built environment. These volume loss values are often estimated based on experience, associated with greenfield conditions and assumed to develop at the foundation level. Recent research by Vu et al. (2016) provides guidance on determining TBM-induced volume loss accounting for different $C/D_t$ and ground types. Based on their theoretical work, at the soil surface a volume loss value of approximately 1.5% might be applied as a conservative design value for the tunnelling scenario simulated in the centrifuge model tests (i.e. $C/D_t = 1.35$ and $D = 6.15$ m in prototype scale). For the greenfield test (Farrell, 2010), 2.0% of volume loss at the tunnel ($V_{l,t}$) caused a surface volume loss ($V_{l,s}$) of approximately 1.7%, which is close to the proposed design value of 1.5%. Therefore, in this chapter the building response is primarily discussed at $V_{l,t} = 2.0\%$, though other results are also presented.

To quantify the alteration of soil displacements due to nearby structures, modification factors for the deflection ratio, $M^{DR}$, and the horizontal strain, $M^\varepsilon_h$, are often adopted (Potts and
This requires relating building distortions to the theoretical greenfield distortions. In the design stage of a tunnelling project, this often involves deriving modification factors based on the relative building stiffness. Following this, the obtained modification factors are multiplied with the theoretical greenfield distortions. This procedure assumes that the building would experience the theoretical greenfield volume loss, but nearby structures might alter the soil volume loss at the surface. Consequently, modification factors should account for volume loss differences caused by the soil-building interaction. Because of this, the modification factors presented in this thesis were obtained at identical tunnel volume loss values. However, it should be again emphasised that, due to soil dilation, these tunnel volume losses represent smaller surface volume losses.

In practice tunnel volume loss values are difficult to determine, while surface volume loss is typically measured using monitoring transects across the settlement trough. This is particularly true for greenfield sections where no buildings are present.

### 7.3 Vertical building response

This section is concerned with the vertical building displacements measured with GeoPIV. As pointed out above, the vertical building response to the tunnel excavation is addressed with respect to $V_{l,t}$. The results are presented in model scale to account for the experimental nature of this research.

#### 7.3.1 Vertical building displacements

Contours of vertical building displacements at $V_{l,t} = 2.0\%$ are presented in Figure 7.1. The contours are plotted on the undeformed shape of the building models. Note that due to the different positions of the building relative to the tunnel centreline, a different scale for the contours was used for test A with $e/L = 0$ (Figure 7.1a) whereas the remaining tests are plotted using an identical scale. Window areas are indicated by white rectangles. The following paragraphs describe the main observations from these contour plots.

For test A the largest vertical displacements were measured close to the centre of the building while the remaining tests showed the largest vertical building movements at the left building edge. This mechanism is clearly related to the building position relative to the tunnel and the largest vertical displacement occurred in the building sections closest to the tunnel centreline.
7.3 Vertical building response

Fig. 7.1 Structure vertical displacement contours at $V_{tj} = 2.0\%$. 

(a) $e/L=0$, $L/H=2.2$, $O=20\%$, $G=3D$. 

(b) $e/L=0.8$, $L/H=2.2$, $O=20\%$, $G=3D$. 

(c) $e/L=0.5$, $L/H=2.2$, $O=20\%$, $G=3D$. 

(d) $e/L=0.8$, $L/H=2.2$, $O=40\%$, $G=3D$. 

(e) $e/L=0.5$, $L/H=2.9$, $O=20\%$, $G=3D$. 

(f) $e/L=0.5$, $L/H=2.9$, $O=40\%$, $G=3D$. 

(g) $e/L=0.5$, $L/H=2.9$, $O=40\%$, $G=2D$. 

Fig. 7.1 Structure vertical displacement contours at $V_{tj} = 2.0\%$. 

173
A gradual decrease of the vertical building displacements with distance from the tunnel centreline is apparent in Figure 7.1. The buildings with $e/L = 0.8$ (tests B and D) and the long buildings (tests E, F and G), experienced very small vertical displacements on the right side of the building. This behaviour is again related to the building-to-tunnel position and the extent of the settlement trough, which leads to almost negligible vertical soil displacements at $x > 250$ mm and $V_{l/t} = 2.0\%$.

Large vertical displacements were observed when the left building edge was located directly above the tunnel centreline (tests C, E, F and G). From these tests, test E experienced the smallest vertical displacements in this region. This is likely due to the large building length, which reduced the tilt of the building compared to the shorter structure of test C. The building models of the tests F and G had 40% façade openings resulting in a more flexible response than test E. This indicates that both the building length and the window opening percentage influence the vertical building response.

Tests B and D showed similar vertical building response although the building models had 20% and 40% of façade openings. As expected, these building models in the hogging region experienced smaller vertical displacements compared to the eccentric tests closer to the tunnel. Smaller vertical displacements were also observed for Test A.

For a given horizontal building position the vertical displacements remained rather unchanged with height. This observation is evident for the entire test series. From Figure 7.1a it is apparent that the building in test A settled more at its right end compared to its left end. This is surprising because the building is placed symmetric to the tunnel (i.e. $e/L = 0$) and in theory a symmetric building response would be expected. The observed asymmetry is likely due to small natural variations in the soil model. Although great care was taken during the model preparation and the latest techniques (i.e. robotic sand pouring) were applied, these variabilities were unfortunately unavoidable.

### 7.3.2 Vertical soil–structure interaction

The response of buildings to soil movements caused by a tunnel excavation depends on the interaction between the ground and the surface structures, which is the focus of this section. Figure 7.2 presents the vertical displacement profiles measured at the soil surface and the building base at $V_{l/t}$ of 0.5%, 1.0%, 2.0% and 4.0%. In addition, greenfield data from a centrifuge test performed by Farrell (2010) is given. The presence of the structural models clearly resulted in vertical soil displacements that differ from the greenfield case. The differential vertical soil displacements transferred to the building were significantly reduced by
7.3 Vertical building response

the soil–structure interaction. Depending on the location of the building relative to the tunnel, and variations in structural details (length, openings and layout), different mechanisms were identified.

For a structure placed symmetrically to the tunnel (test A) a loss of contact between the centre of the building model and the soil became evident as $V_{l,t}$ developed. This so-called gap formation (Farrell and Mair, 2010) resulted in a redistribution of the building weight and the building corners embedded into the soil (Figure 7.2a). The difference between the structure and soil displacements at the building corners, shown in Figure 7.2a, can be attributed to two reasons related to the image-based deformation measurements: (1) the soil and structure displacements cannot be measured directly at the soil–structure interface and the measured soil movements are expected to be slightly smaller than at the soil surface, and (2) although great care was taken during the model setup the structure cannot be placed perfectly flush with the PMMA window and the sand in contact with the PMMA may have experienced some boundary effects. These modelling limitations were further detailed in Chapter 5.

Figure 7.3 depicts the gap formation mentioned above. This local separation of the building from the soil was only observed for test A, where the structure was placed symmetric to the tunnel centreline. The $V_{l,t}$ when this mechanism became evident was determined to be approximately 1.4%, as indicated in Figure 7.3. The gap formation mechanism is extensively discussed in Farrell and Mair (2010), Farrell (2010) and Marshall (2009) and hence is not further addressed herein.

From Figure 7.2 it can be seen that the entire base of structures placed asymmetrically to the tunnel (tests B to G) remained in contact with the sand. The structures with a building edge directly above the tunnel (tests C, E, F and G) caused increased vertical structure displacements directly above the tunnel centreline compared to the greenfield. This finding can be related to a rigid rotation of the structural models towards the tunnel causing embedment of the left building corner. An increase of $V_{l,t}$ amplifies this observation. For test E with $L = 260$ mm and 20% of openings, this increase of vertical displacements compared to the greenfield equivalent is only apparent at $V_{l,t} > 2.0$. This is likely due to a more rigid response of test E compared to the tests F and G, which had equal length and position but different opening percentage. The behaviour of the building model in test E can be explained by a cantilever analogy, which is caused by a building load redistribution. The rather stiff building in terms of flexural rigidity experiences a considerable building rotation towards the tunnel with a redistribution of the building load to the building centre. Consequently, the foundation pressure at the left and right-hand side of the building reduces compared to the centre of the building. This can be compared with cantilevers on each side of the structure. Thus, the supported mid-
Fig. 7.2 Vertical base structure displacements compared to vertical soil surface displacements at $V_{i,t} = 0.5\%, 1.0\%, 1.5\%$ and $2.0\%$. Greenfield data (GF) is given as reference (Farrell, 2010).
7.3 Vertical building response

![Graph showing separation of building model and soil surface at tunnel centreline (x = 0 mm) with increasing tunnel volume loss for test A.]

length of the rather stiff building reduced the vertical movement of the left end of the building section above the tunnel. Meanwhile, vertical displacements at the building centre in test E were slightly larger than in tests F and G. This can be explained by the above mentioned load redistribution to the centre of the building, which is caused by the increased flexural rigidity of the structure in test E.

Tests B and D, which are predominantly placed in the hogging region of the tunnelling-induced settlement trough, showed a slight decrease of the maximum vertical soil displacements above the tunnel compared to the greenfield conditions. This observation is a result of the increased vertical building and soil displacements underneath the entire building length. For the remaining scenarios placed asymmetrically to the tunnel, these increased vertical soil displacements beneath the entire building extent compared to the greenfield configuration are also apparent.

For all tests, the soil displacements beyond the building edges were affected by the building presence. Beyond the building corners substantial differential vertical surface soil displacements are evident for the tests A, C and G. For the discussed $V_{l,t}$ a total recovery of the greenfield vertical displacement trough was not observed, but the vertical surface soil displacements showed reasonable agreement with the greenfield trough when the distance to the building corner was greater than 100 mm (about half of the building length).

After relating the vertical building base displacements to the underlying soil and greenfield settlement profiles, the following section discusses the vertical building displacements at different $V_{l,t}$ values and building levels. Curve fitting of the vertical building displacements is also addressed.
Vertical building distortions

Measurements of vertical building displacements along its distance from the tunnel centreline at $V_{l,t} = 2.0\%$ and three levels of the building façade (base, neutral axis and top) are shown in Figure 7.4. The base displacements were obtained as close as possible to the soil–structure interface while the neutral axis displacements were obtained at the theoretical position of the neutral axis, considering the real geometry of the building cross-sections (see Chapter 4). The top displacements were measured in the building section above the first floor windows. Due to the façade openings and the modelled strip footings, the determined neutral axes of the building configurations are situated below the middle axis of the buildings and within the ground floor windows.

From Figure 7.4 it can be seen that the vertical displacement profiles at the three levels of investigation match very well. Differences are found to be within the GeoPIV noise. The consistent three settlement profiles are not surprising given the vertical displacement contours presented in Figure 7.1. Also Burland et al. (2004) reported very similar vertical displacement profiles observed at three different monitoring levels for the Ritz Hotel affected by the construction of the Jubilee Line Extension. In the following, the base displacement profiles were used to further investigate the vertical building behaviour.

The base settlement data of the building displacements were plotted along the distance from the tunnel centreline at $V_{l,t}$ of 0.5%, 1.0%, 2.0% and 4.0%, as shown in Figure 7.5. As was expected, the magnitude of vertical building displacements increased with an increase of the induced volume loss. Modified Gaussian curves (Vorster et al., 2005) were fitted to these vertical displacement profiles in order to smooth the GeoPIV data and also to derive a potential point of inflection along the building length. From Figure 7.5 it can be seen that the maximum building displacements are often located closest to the tunnel centreline. However, for test A the maximum vertical displacements were not necessarily directly above the tunnel centreline as is visible from Figure 7.5a. This is particularly evident for a greater magnitude of $V_{l,t}$. As a consequence, the adopted curve fitting procedure did not constrain the position of the maximum vertical displacement about which the modified Gaussian curve is symmetrical.

Figure 7.5h presents the trend of the coefficient of determination, $R^2$, versus $V_{l,t}$ for the fitted modified Gaussian curves. It is evident from Figure 7.5h that the vertical movements of the building tests were in general in good agreement with a modified Gaussian curve when $V_{l,t}$ increases above 0.5%. At lower $V_{l,t}$ the vertical deflections are within the DIC noise and thus affect the curve fitting. For the entire asymmetric building tests, the curve fitting resulted in $R^2$ above 0.97 for $V_{l,t}$ greater than 0.5%. However, it can also be seen from Figure 7.5h that the modified Gaussian curves provided a rather poor fit for test A. This is mainly a result of
7.3 Vertical building response

Fig. 7.4 Vertical structure displacements at base, neutral axis and top of structure at $V_{t,1} = 2.0\%$.
Building response to tunnelling-induced subsidence

Fig. 7.5 Vertical base structure displacements fitted with modified Gaussian curves at $V_{l,t} = 0.5\%, 1.0\%, 2.0\%$ and $4.0\%$. 

(a) A: $e/L=0$, $L/H=2.2$, $O=20\%$, $G=3D$.

(b) B: $e/L=0.8$, $L/H=2.2$, $O=20\%$, $G=3D$.

(c) C: $e/L=0.5$, $L/H=2.2$, $O=20\%$, $G=3D$.

(d) D: $e/L=0.8$, $L/H=2.2$, $O=40\%$, $G=3D$.

(e) E: $e/L=0.5$, $L/H=2.9$, $O=20\%$, $G=3D$.

(f) F: $e/L=0.5$, $L/H=2.9$, $O=40\%$, $G=3D$.

(g) G: $e/L=0.5$, $L/H=2.9$, $O=40\%$, $G=2D$.

(h) Goodness of fit for modified Gaussian curves.
the very rigid behaviour of the building model in test A. The qualitative comparison between the GeoPIV data and the fitted modified Gaussian curves, shown in Figure 7.5a, indicates that the modified Gaussian curves also reasonably smoothed the vertical displacement profiles of test A. As was pointed out above, it is worth to stress that the small building deflections, as observed for $V_{l,t} < 0.5\%$, which are a result of the scope of this research to model realistic structures with semi-rigid flexural response, are close to the limit in precision of the GeoPIV measurements. This results in issues when fitting curves to the displacement data and explains some of the challenges below.

The fitted curves, shown in Figure 7.5, were used to deduce points of inflection of the modified Gaussian curves, $i_{mg}$, after which the deflection ratios, $DR$, and modification factors for the $DR$, $M^{DR}$, were estimated. Figure 7.6 presents the position of the inflection points with respect to the tunnel centreline for the different building tests. Test A is not included in this data because the entire structure deformed in sagging. A substantial scatter in the location of the inflection points is apparent in the plot which is a result of both the low building deflections and the precision of the GeoPIV measurements. Nevertheless, two main trends can be observed: (1) the presence of the buildings increased the trough width ($i_{mg}$) compared to the greenfield conditions and (2) the trough width decreased with $V_{l,t}$. As expected, both trends were also observed for the trough width of the soil surface settlement profiles discussed in Chapter 6. Similar observations of trough widening due to adjacent structures were observed in the field (Frischmann et al., 1994; Lu et al., 2001; Viggiani and Standing, 2001) and in centrifuge model tests (Farrell, 2010; Franza, 2017).

A closer study of the data presented in Figure 7.6 reveals that the building models located in the theoretical hogging regions (tests B and D) showed the greatest $i_{mg, str}$. This is a rather surprising outcome because the soil inflection points of the modified Gaussian troughs fitted to surface soil settlement profiles (Figure 6.21) showed only a minor increase of the trough width. As a consequence, the data of the position of the inflection points shown in Figure 7.6 need to be interpreted with caution. Because of this, Ritter et al. (2017a) used the soil surface inflection points and 5th order polynomials to smooth the vertical displacement data of the structures when estimating DRs and $M^{DR}$ values. Nevertheless, describing settlement profiles with modified Gaussian curves is more common in literature (e.g. Marshall et al., 2012; Vorster, 2005) and thus followed herein. As pointed out above, the small building deflections in the range of GeoPIV measurement scatter caused the curve fitting troubles. A potential reduction of the scatter could be observed when constraining the shape parameter $\alpha$ to a mean value.
Building response to tunnelling-induced subsidence

Figure 7.7 shows the variation of the deflection ratios, \( DR \), versus tunnel volume loss, \( V_{lt} \), in sagging and hogging. The building subdivision into hogging and sagging was carried out using the inflection point associated with the modified Gaussian curves fitted to the vertical base structure displacement profiles (Figure 7.6). Subsequently, for structures spanning the hogging/sagging transition zone the \( DR_s \)s were estimated individually for both the hogging and sagging mode. Note that the observed scatter in \( DR_s \) for the tests B, D, and E are caused by small sagging regions and small measurements of building distortion. As noted above, this procedure is different compared to Ritter et al. (2017a), where 5\(^{th}\) order polynomials were used for curve fitting and the soil surface inflection points were adopted to partition buildings spanning the sagging and hogging regions. Although different procedures were used, the derived results, which are discussed in the next paragraphs, are in fair agreement.

The adopted hogging/sagging subdivision implies that the building-to-tunnel position plays a key role, which can also be observed in Figure 7.7. The building placed at zero eccentricity (test A) caused a positive deflection ratio, \( DR_{sag} \), while an equal building positioned at \( e/L = 0.8 \) (test B) showed a substantial \( DR_{hog} \), and thus is likely to be at greater risk of building damage (Mair et al., 1996). For test C, where the building was spanning the hogging/sagging transition region, an even greater \( DR_{hog} \) compared to test B is apparent in Figure 7.7 as \( V_{lt} \) developed. This indicates that structures positioned in the hogging/sagging transition zone of the tunnelling-induced settlement trough are potentially more vulnerable to building damage.
7.3 Vertical building response

Fig. 7.7 Deflection ratios versus tunnel volume loss.
Building response to tunnelling-induced subsidence

This is contrary to current damage assessment methods, which treat a building separately either side of the theoretical greenfield inflection point (Goh and Mair, 2011a; Mair, 2013; Mair et al., 1996).

As the building length and the window opening percentage increased, greater $DR$ values were measured (Figure 7.7). This is particularly evident for the hogging mode of deflection, and can be explained by a more flexible building response. For test D the increase in window area nearly doubled the $DR_{hog}$ value of test B, in which the building had identical length and eccentricity. Likewise, $DR_{hog}$ substantially increased for the long structure with 40% of openings (test F) compared to the equally long structure in test E with 20% of openings.

With respect to the building length, the magnitude of $DR_{hog}$ doubled or even tripled when the building length increased from $L/H = 2.2$ to $L/H = 2.9$. This can be clearly seen in Figure 7.7 when comparing the $DR_{hog}$ trends of test E to test B and of test F to test D. This impact of the building length is particularly true when the increase of the building length causes a decrease of the eccentricity (i.e. the building moves closer to the tunnel). By contrast, buildings with equal eccentricity but different building length showed $DR_{hog}$ values in better agreement. For instance, the $DR_{hog}$ of test E with $L = 260$ mm and $e/L = 0.5$ increased to a maximum of -0.06 compared to a $DR_{hog}$ of -0.04 for test C with $L = 200$ mm and $e/L = 0.5$.

The greatest $DR_{sag}$ was observed for test G, representing an isolated building façade (i.e. 2D building layout). As $V_{l,t}$ developed above 1.0%, $DR_{sag}$ substantially increased. A comparison between the tests F and G reveals that the isolated façade (test G) showed a more flexible response than test F (Figure 7.7). A potential explanation for this difference might be that the additional building components of test F influenced the building performance. For instance, the foundation below the left side wall may pulled the front façade towards the tunnel and thus reduced $DR_{sag}$ of test F. It may also be that the front and rear façade in test F interacted and the structure was therefore more rigid in sagging. The significantly greater $DR_{sag}$ as $V_{l,t}$ developed can partially also be related to the onset of building cracking, which is further addressed in Section 7.5.3. Furthermore, the curve fitting procedure potentially tends to overestimate $DR_{sag}$. This is particularly evident in Figure 7.5g where the fitted modified Gaussian curve at $V_{l,t} = 4.0\%$ results in more dominant sagging distortions compared to the GeoPIV data. On the contrary, a variation of the building geometry (i.e. comparing test G to test F) showed a minor influence on $DR_{hog}$. However, this similar behaviour, which is particularly evident as $V_{l,t}$ develops, could be misleading due to cracking, which would be expected to influence the hogging region response as well.

In Figure 7.8, the data from Figure 7.7 is compared to the associated greenfield deflection ratios, which were obtained by using the greenfield soil settlement profiles (Farrell, 2010) and
taking the corresponding building-to-tunnel position into account. The obtained modification factors for the deflection ratio in sagging, $M_{DR_{sag}}$, and hogging, $M_{DR_{hog}}$, were generally below unity. This result indicates that the building deflections were lower than the greenfield equivalent. Only for tests F and G at $V_{lt} > 3.0\%$ did $M_{DR}$ values exceed unity.

The modification factors vary with the building features. While minor differences were obtained for $M_{DR_{sag}}$, building characteristics substantially altered $M_{DR_{hog}}$. The following observations were made:

- A change of the building position from $e/L = 0.8$ (test B) to $e/L = 0.5$ (test C) caused an increase of $M_{DR_{hog}}$ (Figure 7.8). This implies that identical buildings located in the hogging/sagging transition region of the greenfield settlement profile are potentially more vulnerable to building damage.

- Increasing the façade opening percentage from 20\% to 40\% caused an increase in $M_{DR_{hog}}$ of 0.1 to 0.3. This finding implies that buildings with a high opening percentage are responding more flexibly to the tunnelling-induced ground displacements, as expected.

- An increase of the building length also caused greater $M_{DR_{hog}}$, and for the long structures of tests F and G with 40\% of window openings hogging deflections similar to the greenfield values were observed (i.e. $M_{DR_{hog}} = 1$). Notably, for the tests F and G high $M_{DR_{hog}}$ values above 0.8 already occurred at $V_{lt}$ of about 1.0\%. The remaining tests showed building distortions lower than about 65\% of the greenfield values for the entire range of surface soil volume loss studied.

- For test G, $M_{DR_{sag}}$ values greater than 0.6 were observed. By contrast, the $M_{DR_{sag}}$ values of test F, which had an identical front façade and building-to-tunnel position, were substantially smaller. A potential explanation for this result is the different building layout. The 3D building layout of test F caused a more rigid sagging response than observed for the isolated façade (test G). This may be attributed to an influence between the front and rear façades, which affects the overall building stiffness and thus results in a stiffer response of test F. However, the hogging response of the tests F and G was almost identical, and thus indicates that the distance between the front and rear façades is potentially too large for any interaction. Another potential explanation is that the left foundation perpendicular to the front and rear façades of test F pulled the left corner of the façades towards the tunnel, which reduced the sagging distortions. Moreover, the used curve fitting procedure tends to overestimate the building deflection in sagging as $V_{lt}$ develops.
Building response to tunnelling-induced subsidence

(see Figure 7.5g at $V_{l,t} = 4.0\%$) and the increasing trend of $M^{DR}_{int}$ for test G can also be related to the onset of building cracking. The $DR_{sag}$ values of the remaining tests were below 50% of the greenfield conditions.

7.4 Horizontal building response

The horizontal response of the buildings to tunnelling-induced ground movements is addressed next. Horizontal building displacements are discussed in model scale. Similar to above, the building response is related to the tunnel volume loss.
7.4 Horizontal building response

![Diagram](image1)

(a) A: $e/L=0$, $L/H=2.2$, $O=20\%$, $G=3D$.  
(b) B: $e/L=0.8$, $L/H=2.2$, $O=20\%$, $G=3D$.  
(c) C: $e/L=0.5$, $L/H=2.2$, $O=20\%$, $G=3D$.  
(d) D: $e/L=0.8$, $L/H=2.2$, $O=40\%$, $G=3D$.  
(e) E: $e/L=0.5$, $L/H=2.9$, $O=20\%$, $G=3D$.  
(f) F: $e/L=0.6$, $L/H=2.9$, $O=20\%$, $G=3D$.  
(g) G: $e/L=0.5$, $L/H=2.9$, $O=40\%$, $G=2D$.  

Fig. 7.9 Structure horizontal displacement contours at $V_{l,t}=2.0\%$. Left horizontal displacements are negative while right horizontal displacements are positive.

7.4.1 Horizontal building displacements

Figure 7.9 illustrates contours of horizontal building displacements at $V_{l,t}=2.0\%$. For test A, shown in Figure 7.9a, a different scale for the contours was used compared to the other tests. This was required because of the symmetric location of the building model of test A, experiencing notable positive and negative horizontal displacements.

The most striking aspect of the data in Figure 7.9 is that the base horizontal displacements are small and rather constant along the entire building length. This finding indicates that the presence of the building notably restricted the transfer of the tunnelling-induced ground movements to the foundations, and implies that horizontal strains at the foundation level are very small. Field data reported by, for example, Burland et al. (2004) and Mair (2013) revealed similar observations.
Building response to tunnelling-induced subsidence

For the structures placed eccentrically to the tunnel, the horizontal displacement contours changed along the building height. This is particularly apparent for the tests B to F, and was generally caused by counter-clockwise rotation, or tilt, of the structure. In particular, in tests C, E and F, the buildings rotated towards the tunnel causing substantially greater horizontal displacements of the top left building corner. For buildings placed so that one building edge is located above the tunnel, a decrease of the building length tends to increase the magnitude of horizontal building movements caused by rotation, as expected. By contrast, the right hand side of test G showed relatively smaller, and more constant, horizontal displacements. This result of test G is related to the increase of sagging deformation on the left side of the structure which effectively decreased the tilt. Further, building cracks were observed close to the right side of the third window from the left in the first floor, as discussed in Section 7.5.3. After cracking occurred, the two halves of the building rotated towards each other, as indicated by the increased sagging deformation noted in Section 7.3.3, again causing a relatively smaller horizontal displacement at the top left corner. For the remaining tests, less notable cracking effects were observed at $V_{lt} = 2.0\%$.

The tests B and D located predominantly in the hogging zone of the settlement trough also showed an increase of the magnitude of horizontal displacements with building height. For both tests, the horizontal displacement contours appear identical, which implies that the varying opening percentage had a minor effect on the horizontal building response, i.e. the rotation. On the contrary, for the structures with $L = 260$ mm (tests E and F) an increase of the window opening percentage from 20% to 40% did cause a minor increase in the horizontal building displacement at the top left corner of the building, indicating that the increased flexibility did slightly increase tilt at the left end.

From this discussion of the horizontal building contours, it becomes clear that horizontal building displacements are particularly a function of the building-to-tunnel position, the building length, the building layout and the onset of building damage. The following sections move on to shed further light on the horizontal building behaviour.

7.4.2 Horizontal soil–structure interaction

Figure 7.10 presents the horizontal surface soil and base structure displacements at a $V_{lt}$ of 0.5\%, 1.0\%, 2.0\% and 4.0\%. Results from a greenfield test (Farrell, 2010) are given for reference. For the entire test series and for different magnitude of $V_{lt}$, the soil–structure interaction caused a significant reduction of the peak and differential horizontal surface soil movements compared to the greenfield condition. In general, the horizontal displacements transferred to
the structure showed a relatively uniform magnitude along the entire extent of the building. However, as $V_{lt}$ increased above 2.0% the structures placed in the hogging/sagging transition region of the greenfield trough experienced non-uniform horizontal displacements, as is particularly evident in the Figures 7.10c and 7.10e to 7.10g. These non-uniform horizontal displacements result in axial building strains, although these strains were a fraction of what would be predicted by assuming that the horizontal greenfield displacement profile is imposed on the buildings.

While the building placed symmetrically to the tunnel (test A) showed reduced base building displacement throughout the entire building length, the asymmetric configurations (tests B - G) tend to cause a greater horizontal building displacement than the greenfield values as $x$ increases. In other words, at greater $x$ also the horizontal displacements of the underlying soil are smaller compared to the structure movements. This is a result of the transfer of the horizontal soil displacements closer to the left building edge which are distributed along the entire building length. As long as the building is fully intact (i.e. no cracks developed), the horizontal displacement profile is rather constant as $x$ develops, which explains the greater horizontal displacements towards the right building end. In other words, the relatively constant horizontal displacements tend to be somewhat of an average for the greenfield values.

Figure 7.10 also shows that the horizontal surface soil displacements tend to be smaller than the structure displacements, indicating that the interaction mechanisms between the soil and the building is rather shallow. The notable discrepancies between the soil and structure horizontal displacements are, however, also a result of the inability to accurately measure displacements adjacent to an interface using GeoPIV. This is particularly true when studying a larger dataset with notable displacements which frequently result in so-called ’wild vectors’ close to the region of the soil–structure contact zone. The restraining effects of the presence of the buildings on the underlying soil are addressed in detail in Chapter 6.

Effects of building cracking on the horizontal building displacements are also apparent in Figure 7.10. A salient increase of the horizontal building displacements at $V_{lt} = 4.0\%$ and approximately $x = 100$ mm can be seen in Figure 7.10g, which is related to cracking of the structure in test G, where cracking propagated through the entire height of the structure at a tunnel volume loss of less than 4% causing the two halves to behave nearly independently. A similar trend is visible for test F in Figure 7.10f, though cracking was not as extensive in this case.
Building response to tunnelling-induced subsidence

Fig. 7.10 Horizontal base structure displacements compared to horizontal soil surface displacements at $V_{t,3} = 0.5\%, 1.0\%, 2.0\%$ and $4.0\%$. Left horizontal displacements are negative while right horizontal displacements are positive.
7.4.3 Horizontal building distortions

Horizontal building distortions, in the form of horizontal building strains, play a vital role in current assessment methods. However, numerous researchers reported that very little horizontal strain is often induced in surface structures (e.g. Burland et al., 2004), and recent recommendations to assess building response to tunnel excavation (Mair, 2013) completely neglect horizontal building strains. Emphasis of this section is placed on quantifying the amount of horizontal strains induced in the building models of the centrifuge test series. A comparison to greenfield deformations, which are often adopted in initial damage assessments, is also provided.

Figure 7.11 illustrates the horizontal soil displacements along the building length for three different building height levels at $V_{lf} = 2.0\%$. The gaps in the data at the location of the neutral axis are related to the window openings. It is clear from these plots that the amount of horizontal displacements notably increases with height, again primarily due to tilt. This is particularly true for the asymmetrically placed buildings. The data of test A, shown in Figure 7.11a, indicates that the right building part experienced hardly any horizontal displacements while the horizontal displacements towards the right (positive) increased with building height and decrease of $x$. The increase with height might be attributed to bending deformations.

While most of the tests showed rather constant horizontal displacements along the building length (and thus very small horizontal strains), for test E the top horizontal displacement profile considerably increased (i.e. becomes more negative) as $x$ decreased (Figure 7.11d). By contrast, the base horizontal displacements indicate a slight decrease (i.e. becomes less negative) with $x$. This result is typical for bending along a neutral axis close to the mid-height of a building. The differential horizontal displacements evident for the top level and $x$ between 100 and 150 might are related to the onset of building damage.

A distinct change of the horizontal displacement profile at approximately $x = 100$ mm is apparent in Figure 7.11g. This sudden increase of the horizontal building displacement for test G can be related to building damage in this location. Particularly, the top and neutral axis levels were severely affected by cracking while the base displacement profile showed a minor impact at the discussed $V_{lf}$. Similar trends are evident in Figure 7.11f, though for a significantly lesser extent.

Horizontal building strains were determined from the slope of a linear curve fitted to the horizontal displacements of the sagging and hogging regions of the building. The horizontal displacements were derived based on the horizontal displacement profiles at neutral axis level (Figure 7.11) because in theory only horizontal extension is apparent at this level and bending deflections are not causing any horizontal strains. This assumption, however, neglects that...
Building response to tunnelling-induced subsidence

Fig. 7.11 Horizontal structure displacements at base, neutral axis and top of structure at $V_{l,t} = 2.0\%$.
Left horizontal displacements are negative while right horizontal displacements are positive.
7.4 Horizontal building response

The soil–structure interaction effects (Serhal et al., 2016) and the different tensile resistance of buildings in hogging and sagging (Burland and Wroth, 1974) might alter the location of the neutral axis. The hogging/sagging partitioning was based on $i_{mG, str}$ (Figure 7.6). Figure 7.12 gives the obtained average horizontal strains in compression, $\varepsilon_{hc}$, and tension, $\varepsilon_{ht}$. Particularly, $\varepsilon_{hc}$ is affected by rather small sagging regions that caused substantial scatter when deriving $\varepsilon_{hc}$.

For test A, which was predominantly placed in the sagging zone, compressive strains were derived while the buildings placed at $e/L = 0.8$ (test B) showed tensile strains. Marginally greater tensile strains were monitored for the building of test D which had the same length and location as the structure in test B but 40% of openings. The greatest tensile strains were however measured for the tests F and G, which is identical to the $DR_{hog}$ results. The increase in building length and façade openings caused substantially greater horizontal building tensile strains. The crack initiation, which for test G is apparent at $V_{lt}$ exceeding 1.0%, explains the increase of $\varepsilon_{ht}$ of test G compared to test F. Nevertheless, at $V_{lt}$ values below 2.0% for all tests the obtained horizontal building strains were in the ‘Category 0’ (negligible) when adopt-
Building response to tunnelling-induced subsidence

The data in Figure 7.12 shows only compressive strains for test A and the structures placed in the hogging/sagging transition region (tests C, E, F and G). For test C considerable compressive strains were measured as $V_{l,t}$ develops. This result is related to a rigid body rotation of the building towards the tunnel which then caused an embedment of the left building corner. Consequently, the horizontal movement of the structure was substantially constrained above the tunnel and substantial compressive horizontal strains were measured.

Figure 7.13 gives the modification factors for the horizontal strain in compression $M_{\varepsilon hc}$ and tension $M_{\varepsilon ht}$. The greenfield strains were obtained at the soil surface following Mair et al. (1996) while the horizontal strains induced in the building models were analysed at the neutral axis position (see above). From Figure 7.13 the following main observations can be made:

- For all tests, the compressive strains were generally below 25% of the corresponding average horizontal greenfield strains. Test C showed the greatest compressive strains due to the significant rotation of the entire building towards to the tunnel combined with
the embedment of the left building corner (see above) while the remaining tests showed negligible horizontal compressive strains induced in the buildings.

- In tension, considerably greater modification factors for the horizontal building strains were observed. While most of the induced horizontal strains were below 30% of the greenfield equivalent, for test F average horizontal building tensile strains of about 40% compared to the greenfield were measured. For test G, with identical building position and opening percentage but an isolated façade, similar horizontal strains were measured until about 1.0% of $V_{t,t}$. As $V_{t,t}$ developed cracking initiated and the horizontal tensile strains substantially increased in the building of test G. These two tests are followed by test D (at lower $V_{t,t}$) which indicates that buildings with larger window openings might be more susceptible to horizontal tensile strains. This is to be expected due to the reduced cross-section directly affecting the axial building stiffness.

- Although the data presented in Figure 7.13 suggests that horizontal building strains cannot be completely neglected, for the entire test series the pre-cracking $M^{\varepsilon_h}$ were lower than 0.5 for $V_{t,t}$ below 1.5%. This observation implies that a maximum of 50% of the theoretical greenfield horizontal strains was transferred to the structures, though significantly smaller percentages were transferred in most cases.

- A significant increase of the $M^{\varepsilon_{ht}}$ as $V_{t,t}$ develops is evident for test G where cracking damage was visible at rather low tunnel volume loss (Section 7.5.3). This result indicates that substantial horizontal building strains can be transferred to buildings which suffer from pre-existing building damage.

### 7.5 Effect of building features on building response and damage

The scope of this section is to provide further understanding of the influence of building details on the global and local building response to tunnelling-induced subsidence. Moreover, building damage in terms of cracking is addressed. First, an alternative framework of analysing building response to ground movements using so-called building deformation parameters to quantify the structural response is introduced.

Son and Cording (2005) subdivided a building adjacent to a deep excavation into building units based on the location of intermediate walls, building columns, different structural properties (e.g. geometry or stiffness) or gradients of ground displacements. Figure 7.14 shows
Building response to tunnelling-induced subsidence

such a building section, including the four corner points of the building bay and schematic building deformation. Based on horizontal, $S_h$, and vertical displacements, $S_v$, of the corner points, the building height, $H$, and the length of the building section, $L$, the response of the buildings to the tunnelling-induced settlements is quantified. Figure 7.14 and the following terms define the used building deformation parameters, originally reported by Son and Cording (2005):

Base horizontal strain:

$$
\varepsilon_{h,\text{base}} = \frac{S_{h,B} - S_{h,A}}{L} \quad (7.1)
$$

Top horizontal strain:

$$
\varepsilon_{h,\text{top}} = \frac{S_{h,C} - S_{h,D}}{L} \quad (7.2)
$$

Slope:

$$
\varsigma = \frac{S_{v,A} - S_{v,B}}{L} \quad (7.3)
$$

Tilt (rigid body rotation):

$$
\theta = \frac{(S_{h,A} - S_{h,D}) + (S_{h,B} - S_{h,C})}{2H} \quad (7.4)
$$

Angular distortion:

$$
\beta = \varsigma - \theta \quad (7.5)
$$

For the tilt, the average rigid body rotation of the left and right hand side of the building section (Equation 7.4) is used throughout this dissertation.

As can be seen in Figure 7.14, the building deformation parameters are a result of the displacements of the points A, B, C and D, which can be either the corner points of the entire structure or a certain section of the building. Global behaviour of a building is estimated
7.5 Effect of building features on building response and damage

Fig. 7.15 Subdivision of building at partition walls into building bays and notation of corner points for a building of 260 mm length. For a building with a length of 200 mm, the building is subdivided into bays 1-3 and corner points 1-8.

by using the displacements of the corner points of the entire structure while a subdivision of the building into sections or bays allows a more detailed study of localised effects related to building damage. Within this section the building models are analysed in two approaches: (1) the global building response and (2) the buildings are subdivided into bays at the position of the partitioning walls. Figure 7.15 depicts the building subdivision into bays for a building configuration with \( L = 260 \) mm and the notation of corner points. For buildings with \( L = 200 \) mm, only three building bays with the corner points 1-8 exist.

To obtain the vertical and horizontal displacements at the corner points, the average displacements of nine GeoPIV patches, surrounding the theoretical position of the corner point, was determined. For buildings with 40% of window openings and certain corner points, the average displacements of a corner point were the result of considering eight GeoPIV patches. Figures 7.16 illustrates the vertical and horizontal displacements along \( V_{l,t} \) for the corner points of Bay 1 of test F. Having discussed the building deformation parameters, the notation of building bays and corner points, the next section addresses the global behaviour of the buildings subject to tunnelling.

7.5.1 Global building response

The global building response is estimated by using the entire extent of the building, essentially providing building deformation parameters over the entire length of the building. Figure 7.17 presents the building deformation parameters of the entire test series. As expected, compressive or tensile top horizontal strains were measured for a building placed in the sagging (test A) or hogging region (test B), respectively. Surprisingly, test C, which spans the hog-
Building response to tunnelling-induced subsidence

Fig. 7.16 Displacements of corner points (CP) of Bay 1 of test F.

ging/sagging transition zone, showed substantial top compressive strains. For test D, similar tensile top horizontal strains were derived as for test B. Long structures placed in the hogging/sagging transition regions (tests E, F and G) showed considerable tensile strains at the top. The greatest tensile strain was observed for test G which implies that a long structure with a significant amount of window openings (i.e. 40%) placed in the hogging and sagging region is likely to be exposed to a significant risk of building damage. The onset of building cracking as \( V_{l,t} \) exceeds 1.0% explains the notable difference between the tests G and F. The increase of the tensile top horizontal strains for test F after 2.5% of \( V_{l,t} \) can also be related to building damage.

For all tests the horizontal strains measured at the base of the structures (Figure 7.17b) were significantly lower than the top horizontal strains. This is likely to be caused by the rough soil–structure interface, which constrained the horizontal displacements at the base of the structures. For the tests with buildings located in the hogging zone (tests B and D) tensile base strains were obtained. This indicates that the strain induced by horizontal soil displacements dominated over the base horizontal strain caused by hogging deformations. For structures placed in the hogging/sagging transition zone, the window percentage caused a considerable difference in the response. Buildings with 20% of openings (tests C and E) were in compression while the base of the tests F and G was in tension. It is likely that the structures with 20% openings responded primarily in bending while the structures with 40% of openings showed mainly shear deformations, but this will be further considered in Section 7.7.

Figures 7.17c and 7.17d indicate that the slope and tilt values are a function of the eccentricity. Notably, the buildings with one edge closest to the tunnel (tests C, E, F and G)
7.5 Effect of building features on building response and damage

Fig. 7.17 Global building deformation parameters (no subdivision into bays): (a) top horizontal strain, (b) base horizontal strain, (c) slope, (d) tilt and (e) angular distortion.

experienced the greatest global slope, followed by the buildings in the hogging region (tests B and D) whereas negligible slope values were estimated for test A.

Similar trends are also evident for the tilt (Figure 7.17d). In all tests but test G, the tilt and the slope are nearly identical. This suggests that the rigid body rotation is significantly greater than the structural deformation (i.e. angular distortion). However, for test G a negligible tilt was measured at low $V_{lt}$ which can be related to the flexible sagging response of test G that reduced the tilt (see Section 7.4.1). The high slope and tilt values observed in tests C, E and F are less important for damage predictions but might cause serviceability problems.

Figure 7.17e presents the angular distortion along $V_{lt}$. This global angular distortion is indicative of the shearing distortion of the building if the strain is constant over the entire building length. Although this assumption is a considerable simplification, the angular distortion relates to structural deformations and thus is of importance when assessing potential building damage. As can be seen from Figure 7.17e, the structure of test G resulted in the greatest angular distortion, followed by test C. Also for tests E and F notable angular distortions (although negative) were measured. The global measure of shear distortion indicates a significant potential for cracking of the buildings placed in the hogging/sagging transition zone (tests C, E, F and G) while negligible angular distortion values were observed for the tests A, B and D. Particularly when taking the notable tensile strains measured at the top of
Building response to tunnelling-induced subsidence

tests E, F and G into account, the location, increased length and percentage of façade openings tends to result in substantial susceptibility to building damage.

7.5.2 Local building response

Localisation effects of building damage are discussed next by subdividing the buildings at their intermediate walls (Figure 7.15). For every building bay, the displacements of the corner points are estimated, and subsequently the deformation parameters derived. Figures 7.18 and 7.19 compare the top horizontal strain and the angular distortion, respectively, for the entire centrifuge test series. These two building deformation parameters, which are related to bending and shear distortions, are of key importance when assessing potential building damage. However, both parameters assume constant deformation over the length of a building bay and thus do not directly quantify bending or shear distortions.

When the structure was placed in the sagging region of the settlement trough (test A), compressive horizontal strains were monitored for the entire building bays, as is evident from Figure 7.18a. By contrast, the angular distortion of Bay 3 remained close to zero and similar magnitude of angular distortion with different sign were measured for Bay 1 and 3 (Figure 7.19a). These results for the angular distortion were to be expected due to the symmetric position of the building model in test A.

Figures 7.18b and 7.19b summarise the $\varepsilon_{h,\text{top}}$ and $\beta$ values for the different bays of test B. For the structure placed in the hogging region, top tensile strains were measured throughout all bays. The greatest $\varepsilon_{h,\text{top}}$ was measured in Bay 2, and indicates potential tension cracking in this region. However, the angular distortion estimated for Bay 2 was close to zero while considerable angular distortion values were calculated for the Bays 1 and 3 (Figure 7.19b). These observations for $\beta$ can be explained by a change of the slope in the different bays (i.e. slope decreases with distance from the tunnel) and a rather constant rigid body rotation (i.e. tilt) for all bays. The considerable amount of angular distortion in Bays 1 and 3 suggests significant shear deformation, in addition to the bending deformation. As a result, the location of potential cracking is rather difficult to predict based on the observed amount of strain and angular distortion.

The building model of test C experienced substantial compressive strains in Bay 1 whereas tensile strains were observed in Bay 2 (Figure 7.18c). In Bay 3 the horizontal top strains remained close to zero for the tunnel volume losses considered. The significant compressive strain observed in Bay 1 is likely due to the embedment of the left building corner into the soil, restraining the horizontal displacement at the left building corner, combined with the
7.5 Effect of building features on building response and damage

Fig. 7.18 Top horizontal strain for building bays. Positive strains indicate tension.
Building response to tunnelling-induced subsidence

![Graphs showing angular distortion for building bays](image)

Fig. 7.19 Angular distortion for building bays.
7.5 Effect of building features on building response and damage

substantial rigid body rotation towards the tunnel. Similar to test B, the angular distortion for Bay 1 and Bay 3 resulted in rather equal magnitude but opposite sign, as expected, while the angular distortion calculated for Bay 2 was close to zero.

Figure 7.18d shows that minor horizontal building strains were transferred to the structure of test D. A comparison with test B (Figure 7.18b) indicates that an increase of the window openings from 20% to 40% for test D but identical building-to-tunnel position and length had a minor impact on the top horizontal building strains. By contrast, the angular distortions of Bays 1 and 3 of test D, shown in Figure 7.19d, nearly doubled compared to test B (Figure 7.19b). This rise in $\beta$ can be attributed to the increased shear flexibility due to the greater opening percentage.

For the long buildings placed at $e/L = 0.5$ (tests E, F and G), the greatest horizontal top tensile strains were measured in Bay 2 (Figures 7.18e, f and g). This is followed by Bay 3. By contrast, $\varepsilon_{h,\text{top}}$ is almost negligible in Bay 1 and Bay 4. To keep a constant scale for the entire test series, the horizontal top tensile strains for Bay 2 of test F and G are not shown after reaching 0.125%. The substantial rise of these tensile strain is clearly related to building cracking.

A significant amount of angular distortion was measured in Bays 1, 2 and 4 of the tests E, F and G, as is visible in Figures 7.19e, f and g. In Bay 3 the lowest angular distortion was generally estimated. Only for Bay 1 of test G, the angular distortion notably dropped after a $V_{lt}$ of approximately 2.0%. Increasing the façade opening percentage and a 2D building layout further increased the angular distortion values. In combination with the significant amount of horizontal tensile strains experienced in the Bays 2 and 3 of the tests with structures of $L = 260$ mm this finding indicates that long structures spanning the inflection point (tests E to G) are the most vulnerable scenario studied. It can therefore be assumed that for these building-to-tunnel configurations building damage in terms of cracking will occur at the lowest tunnel volume loss values as will be explored in the following section.

7.5.3 Building damage

The 3D printed structures exhibit brittle behaviour similar to that of masonry. Building damage in terms of cracking was therefore experienced throughout the centrifuge test series. Within this section, the onset and location of these cracks relative to tunnel and surface soil volume loss is identified. Additionally, cracking damage observed by visual inspection of images that were acquired during the experiments is presented. Microcracking, which is not visible to the naked eye and occurred at lower tunnel volume losses than the visible cracking, was
observational in the data and is also addressed. As noted in Chapter 4, the ultimate strain to failure of the 3D printed material is about an order of magnitude higher compared to brick and mortar structures; thus, cracking damage is expected at relatively high tunnel volume loss.

For all structural models cracking initiated at the top of the buildings. This result suggests that the cracking is related to bending distortions. Because of this, horizontal displacement profiles at top building level, shown in Figure 7.20a, were used to derive the crack onset. From Figure 7.20a it is evident that crack locations can be identified where a sharp gradient of the horizontal displacement profiles is apparent. In addition, a visual inspection of the corresponding images that were acquired during the experiments was conducted, as illustrated in Figure 7.20b. Based on this procedure, the first visible crack for test F (i.e. crack A in Figure 7.20b) emerged at a $V_{lt}$ of approximately 2.6%. As volume loss developed, the crack propagated vertically towards the base of the structure causing the cracks B and C in Figure 7.20b. Finally, crack D developed. Microcracking might occurred at lower volume losses.

From Figure 7.20 it is apparent that the crack location was close to the window corners. This was expected because openings define the weakest cross-sections and result in stress localisation close to the window corners. The predominantly vertical direction of the cracks can be related to the weak interlayer bond between the different layers caused by the 3DP procedure (Chapter 4). As $V_{lt}$ developed, the initial cracks grew and a rigid body motion of the building parts defined by the initial cracks A-C becomes visible (Figure 7.20c). This results in mainly two rigid blocks, and after crack D emerged a minor third rigid block between the cracks A and D evolved. The entire main left block rotates towards the tunnel while the main right block experienced notably smaller rotation and displacements.

The observed crack patterns of the entire test series is visualised in Figure 7.21. Except for test A, where the building remained intact throughout the entire experiment, the remaining tests showed building damage induced by the tunnel excavation. Similar trends of crack onset at the top of the building models and vertical development of the cracks towards the base of the structures is apparent. From Figure 7.21 it is striking that identical locations of crack initiation were observed for structures placed at equal building-to-tunnel position. For instance, the location of the crack onset of tests B and D both placed at $e/L = 0.8$ was identical. Likewise, for the tests F and G, which were both positioned in the hogging and sagging transition zone (i.e. $e/L = 0.5$), the same location of crack initiation was observed. Also the position of the cracking onset of the building model of test C placed at $e/L = 0.5$ was identical. By contrast, the cracking pattern of test E, shown in Figure 7.21e, was different. This is likely caused by the rather rigid performance of the structure in test E which had only 20% of façade openings. Glueing the individually 3D printed building parts together with Araldite standard
7.5 Effect of building features on building response and damage

(a) Crack identification using horizontal displacement profiles at top of structure.

(b) Cracking pattern ($V_{l,t} = 6.0\%$).

(c) Vector plot at $V_{l,t} = 6.0\%$. Displacement vectors are times 10. Positions of GeoPIV calibration markers are depicted.

Fig. 7.20 Cracking of test F.
could theoretically have an effect on the crack location and propagation. However, due to the high viscosity of Araldite standard, the glue did not propagate into the porous 3D printed material. Hence, only a very thin glued section with different material properties than the 3D printed material was obtained. Figure 7.21 shows that all the cracks propagated within the 3D printed material and with a considerable distance to the glued section. The exact location of the first crack may be explained by relating the building damage parameters obtained for the different building bays (Section 7.5.2) to the crack location.

For test B, the first cracking damage occurred in Bay 2 (Figure 7.21a), where the greatest tensile strain was monitored (Figure 7.18b), but the amount of angular distortion was significantly lower in Bay 2 compared to the Bays 1 and 3 (Figure 7.19b). Identical results were observed for the horizontal strains (Figure 7.18d) and angular distortions (Figure 7.19d) measured for the different bays of test D. This result suggests that the cracking of test B and D is related to bending distortion, an observation that is supported by the origination of the first crack at the top of the building façade.

Figure 7.21c illustrates the crack pattern observed for test C, including the first crack which initiated at the top of Bay 2. Similar to test B, the highest tensile strain was determined in Bay 2 (Figure 7.18c). However, the angular distortion values were considerably greater in the Bays 1 and 3 than in Bay 2 (Figure 7.19c). This indicates that identifying the crack location based on the $\varepsilon_{h,\text{top}}$ and $\beta$ is challenging.

The initial crack in test E originated at the top of Bay 3, as marked in Figure 7.21e. As $V_{l,t}$ developed notable horizontal tensile strains were measured at the top of Bays 1 and 3, as can be seen in Figure 7.18e. Considerable angular distortion values were also derived for the Bays 2 and 3 (Figure 7.19d). In combination, this data indicates that cracking might occur in Bay 3 but similar to the crack patterns of the previously discussed tests B, C and D, the crack location could not be directly related to the angular distortion measured for the different building bays.

For test F and G, the cracking initiated in Bay 2. Both tests showed the greatest horizontal tension strains in Bay 2 (Figures 7.18f and 7.18g) which significantly increased after crack onset. At $V_{l,t}$ below 1.0% notable angular distortion values were derived for Bay 2 of tests F and G. These findings imply that for the buildings of tests F and G Bay 2 is most susceptible to building damage. As noted above, the substantial amount of window openings likely caused notable shear distortions at building bay scale which often results in local bending of thin elements (i.e. the upper building level in tests F and G). Similar observations were made by Van Kessel (2012) who compared this local behaviour to the response of a portal frame under shear.
7.5 Effect of building features on building response and damage

Fig. 7.21 Crack initiation and location. Solid (blue) ovals indicate cracks while dashed (red) ovals show potential microcracking. Letters indicate order of crack propagation.
Building response to tunnelling-induced subsidence

Table 7.1 Visible cracking. Tunnel volume loss values refer to crack onset.

<table>
<thead>
<tr>
<th>Test</th>
<th>$V_{t,t}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>no cracks</td>
</tr>
<tr>
<td>B</td>
<td>14.0</td>
</tr>
<tr>
<td>C</td>
<td>8.0</td>
</tr>
<tr>
<td>D</td>
<td>10.4</td>
</tr>
<tr>
<td>E</td>
<td>5.5</td>
</tr>
<tr>
<td>F</td>
<td>2.6</td>
</tr>
<tr>
<td>G</td>
<td>1.9</td>
</tr>
</tbody>
</table>

The $V_{t,t}$ at the onset of visible cracking is summarised in Table 7.1. While the building in test A stayed intact throughout the entire experiment, the remaining tests showed cracking damage. In line with the building damage parameters discussed above, the first crack, at a $V_{t,t}$ of 1.9%, became visible in test G. The structure of test F, which was also spanning the hogging/sagging transition zone and also had 40% of openings, followed with a crack initiation at a $V_{t,t}$ of 2.6%, while the first crack of the structure of test E (i.e. identical position but 20% openings) was observed at a $V_{t,t}$ of 5.5%. Next, the structure of test C, which was also placed in the hogging/sagging transition region but was shorter and had 20% openings, cracked at $V_{t,t}$ of 8.0%. The structures in the hogging regions (tests B and D) cracked at 14% and 10.4% of $V_{t,t}$, respectively. An increase of the façade openings from 20% to 40% caused a considerably earlier cracking, which is consistent with the structures placed in the hogging/sagging transition region.

From the data summarised in Table 7.1 it can be followed that buildings that spanned both the hogging and sagging region with notable window openings (i.e. 40%) were more susceptible to cracking damage while a 3D building layout (i.e. rear, intermediate and end walls) reduced the onset of cracking damage. This finding can be related to the more flexible response of the isolated façade test (test G) in sagging (see Section 7.3.3).

For test F and G visible cracking occurred at values of surface soil volume loss ($V_{s,t}$) of 2.0% and 1.5% which are in fair agreement with design values (e.g. Vu et al., 2016). Micro-cracking, which is evident in some of the data but cannot be identified with the naked eye, might have occurred at even lower volume loss values.

In this section, the global and local building behaviour to tunnelling-induced soil displacements and associated building damage was discussed. The presented experimental data revealed that the building response and cracking damage depends on the building-to-tunnel position and structural details. More specifically:
7.6 Building response for hogging and sagging separation

- Structures that were placed in the hogging/sagging transition regions were more vulnerable to building damage than equal buildings located in either sagging or hogging.

- Increasing the building length and the façade openings resulted in larger horizontal top tensile strains and angular distortion values. Modelling an isolated building façade caused even greater building deformation parameters (e.g. horizontal strain and angular distortion).

- Cracking onset and patterns observed for the different building configurations confirmed the building response, resulting from the analysis of building deformation parameters.

7.6 Building response for hogging and sagging separation

To evaluate current assessment methods that analyse building parts on either side of the greenfield inflection point separately, the building response is quantified for the hogging and sagging part individually. Figure 7.22a illustrates this approach, and indicates that assessment predictions for test B and the hogging part of test E ($E_{hog}$) would give the same result. Likewise, the prediction of the behaviour of the sagging part of test C ($C_{sag}$) would be equal to the prediction for the sagging part of test E ($E_{sag}$). For buildings with 40% of openings, illustrated in Figure 7.22b, the hogging parts of test D and test F ($F_{hog}$) should theoretically result in identical building response. While Mair et al. (1996) reported that building parts exceeding $x = 2.5 \cdot i$ can be neglected, Netzel (2009) showed that this assumption might lead to underestimation of bending strains. Therefore, within this work the entire building length is considered. For the building subdivision into hogging and sagging a theoretical greenfield point of inflection at an offset of 60 mm from the tunnel centreline is assumed (Figure 7.22).

Figure 7.23 compares the damage parameters for test B and $E_{hog}$ as $V_{l,\tau}$ developed. From Figure 7.23 it can be seen that the theoretical hogging part of test E experienced a different response than test B. A considerable greater tensile strain was monitored at the top of test E (Figure 7.23a) while the base horizontal strain is rather similar for both tests analysed (Figure 7.23b). The additional extent of the building towards the tunnel in test E caused a significant increase of the slope as can be seen from Figure 7.23c. Similarly, the rigid body rotation (i.e. tilt) measured for the hogging part of test E notably increased compared to the one of test B (Figure 7.23d). Although there is scatter in the GeoPIV data, Figure 7.23e indicates a greater angular distortion for $E_{hog}$. These observations show that test E is more vulnerable to potential building damage than test B, and that treating the theoretical sagging and hogging part of a building separately can lead to underestimation of building damage.
Building response to tunnelling-induced subsidence

Fig. 7.22 Subdivision of building models spanning hogging and sagging region (tests C, E and F) and comparison to buildings placed in hogging regions (tests B and D).

(a) Buildings with $O = 20\%$.

(b) Buildings with $O = 40\%$. 

210
Comparing the response of the theoretical hogging part of test F with test D is carried out in Figure 7.24. Both tests have 40% façade openings. The trends evident in Figure 7.24 match the observations made for the buildings with 20% of openings (Figure 7.23). Specifically, the substantial increase of the tensile top horizontal strain and the angular distortion for $F_{\text{hog}}$ highlight that the widely accepted approach of subdividing a structure at the theoretical greenfield point of inflection might result in unconservative damage assessments.

The building deformation parameters for the sagging parts of test C ($C_{\text{sag}}$) and E ($E_{\text{sag}}$) are presented in Figure 7.25. Interestingly, the data indicates a similar response for both tests. While the compressive top horizontal strain for $C_{\text{sag}}$ is notably greater than for $E_{\text{sag}}$ (Figure 7.25a), the remaining parameters are in fair agreement. As a consequence, the additional building length of test E had a minor influence on the building part in the sagging region. This finding suggests that a sagging/hogging subdivision might result in satisfactory predictions for the sagging part of a building, which generally is the less critical part due to predominantly compressive strains, though additional data is needed to confirm this observation.

The aim of this section was to investigate the widely accepted framework of individually assessing building parts on either side of the greenfield inflection point. For the scenarios under consideration, the following conclusions can be drawn:

- The partitioning approach led to reasonable results for sagging parts of structures.
**Building response to tunnelling-induced subsidence**

![Graphs showing building deformation parameters for hogging parts.](image)

**Fig. 7.24** Building deformation parameters for the hogging parts of buildings with 40% façade openings (tensile strains are positive).

- Hogging parts showed a significantly different structural response depending on the extension of the structure across the assumed partitioning location. This finding was obtained for building configurations of different window opening percentage. The obtained results suggest that neglecting the sagging part of a building when considering hogging might underestimate building damage.

### 7.7 Effect of building characteristics on shear and bending deformations

This section sheds new light on the relative importance of shear and bending distortions during building response to tunnelling. Specifically, building length and façade opening effects on the governing mode of building deformation (i.e. shear or bending) are explored. To distinguish between the bending and shear deflections caused by tunnelling subsidence, the framework outlined by Cook (1994) was adopted. Figure 7.26 defines the sign convention, the definition of the bending and tilt deflections and the subdivision of buildings at the position of their transverse walls. For each bay the following steps were carried out:
Firstly, the displacement due to tilt was defined as

\[ S_{v,\text{tilt}} = \sin\left(\frac{\omega_1 + \omega_2}{2}\right) L. \]  

(7.6)

Secondly, the bending deflection was derived as

\[ S_{v,\text{bend}} = \frac{\omega}{2} L \]  

(7.7)

where \( \omega \) is in radiant and positive values of \( S_{v,\text{bend}} \) indicate a hogging (i.e. convex) mode of deflection. Thirdly, the total vertical displacement was computed as

\[ S_{v,\text{tot}} = S_{v,A} - S_{v,B}. \]  

(7.8)

Finally, the shear deflection was defined as

\[ S_{v,\text{shear}} = S_{v,\text{tot}} - S_{v,\text{tilt}} - S_{v,\text{bend}}. \]  

(7.9)

Note that this framework assumes constant curvature over a single building bay when estimating the deflection due to bending. Likewise, uniform shear deflection over a building
Building response to tunnelling-induced subsidence

(a) Reference condition for a single building bay and sign convention.

(b) Bending and tilt deflection.

(c) Separation of buildings into bays.

Fig. 7.26 Framework to investigate building response after Cook (1994).

bay is assumed. The displacement due to tilt (Equation 7.6) is again based on the average tilt between the left and right edge of a building bay.

7.7.1 Building length effects

The effect of different building lengths on the governing mode of building deformation is studied by making use of two scenarios, which are illustrated in Figure 7.27. Scenario (a) focuses on building configurations with a constant façade opening percentage of 20% and compares Bay 1 of test B with Bay 2 of test E (Figure 7.27a) as highlighted with the red arrow in Figure 7.27a. Both bays are located at equal position with respect to the tunnel centreline. Following the same principles, scenario (b) compares building configurations with 40% façade openings (tests D and F).

Figure 7.28 presents the impact of the building dimensions on the bending and shear deflections. The analysed bays (i.e. Bay 1 for buildings with $L = 200$ mm and Bay 2 for building with $L = 260$ mm) have an equal location relative to the tunnel (see above). For different amounts of window openings (20% and 40%) an increase of the building length from 200 mm to 260 mm caused greater bending deflections while shear deflections were rather similar. This is particularly true as $V_{t,l}$ increases, and the substantial change of bending and shear deflection in test F indicates cracking initiation at a $V_{t,l}$ of approximately 2.5% (Figure 7.28b and 7.28d). Although $L/H$ increased only from 2.2 to 2.9, an increase of the building length combined
7.7 Effect of building characteristics on shear and bending deformations

Fig. 7.27 Scenarios to study building length effects on shear and bending deformations. Tunnel position and diameter are not to scale.

with the position of the building in hogging and sagging led to a substantially higher risk of building damage.

7.7.2 Building opening effects

To study the effect of different façade opening percentage, two scenarios are chosen as can be seen from Figure 7.29. Figure 7.29a shows structures B and D both with a length of 200 mm and placed in the hogging region of the corresponding greenfield settlement profile but with 20% and 40% openings respectively. Likewise, the buildings of the tests E and F are placed at identical building-to-tunnel position and have equal length but differ in opening percentage (Figure 7.29b). These two scenarios are now used to point out the effect of window opening variations on the shear and bending deformation components.

Figure 7.30 shows the shear and bending deformations along tunnel volume loss for different amount of window openings while the building length and position was kept constant. As indicated in Figure 7.29, data from equal building bays are compared within a subplot. For buildings with identical length and position relative to the tunnelling-induced settlement profile, an increase of window openings from 20% to 40% caused greater shear deflections
Building response to tunnelling-induced subsidence

Fig. 7.28 Influence of increasing $L/H$ on bending and shear deflections.

Fig. 7.29 Scenarios to study building opening effects on shear and bending deformations. Tunnel position and diameter are not to scale.
while the bending components generally remained close to zero. This finding is evident for buildings with $L/H = 2.2$, shown in Figure 7.30a, and $L/H = 2.9$ (Figure 7.30b). Only in Bay 2 of the structures with $L = 260$ mm was a considerable bending contribution measured, as can be seen from Figure 7.30b (i).

The main goal of this section was to experimentally determine the bending and shear deformation components of buildings subject to tunnel excavation. The results have shown how shear and bending deformations vary throughout the length of the buildings. The experimental data presented within this section was used to evaluate the effect of changing building dimensions and façade opening percentage on the bending and shear deformations. The key observations from this section are:

- An increase of the building length led to an increase of bending deflections while shear deflections remained rather equal.

- A larger amount of window openings caused a considerable increase of the shear component but had little effect on bending deformations.

These findings indicate the importance of considering both shear and bending deformations when assessing tunnelling-induced settlement damage on structures.

### 7.8 Summary

This chapter discussed the response of buildings with different characteristics to tunnelling-induced subsidence. From the experimental results, the following conclusions can be drawn:

- The building stiffness plays a key role in the building response to tunnel excavation, and predictions based on greenfield assumptions are in general overly conservative. However, the centrifuge experiments showed that the building-to-tunnel position, the building length and the amount of façade openings are also important factors. In the sagging region, the measured vertical building distortions were generally below 50% of the greenfield equivalent. However, for a long, isolated façade with 40% of openings (test G) 80% of greenfield deflections were observed. Parts of this result might be attributed to the discussed curve fitting.

- The building position and characteristics had a substantial affect on the building distortions in hogging. Structures located in the hogging and sagging regions of the tunnelling-induced settlement profile were more vulnerable (in terms of the $DR$) than
Building response to tunnelling-induced subsidence

Fig. 7.30 Influence of increasing the opening percentage on bending and shear deflections.
buildings of identical length positioned entirely in sagging or hogging. Even greater hogging deflections were observed if the building length increased. Likewise, an increase of the opening percentage resulted in a significant increase of the $M_{DR_{hog}}$. Increasing the façade opening percentage from 20% to 40% doubled $M_{DR_{hog}}$ for building configurations of different length. This reveals the general trend that an increase of the building length and the façade openings leads to more severe structural damage.

- For a tunnel passing beneath the building corner of a long flexible structure with 40% openings, building deflections in hogging equal to the greenfield were obtained, even at tunnel volume losses close to 1.0%. As volume loss developed the $M_{DR_{hog}}$ values further increased and the obtained $M_{DR_{hog}}$ of an isolated façade (test G) and a building model with identical front (and rear) façade but end and intermediate walls (test F) were in good agreement. This implies that long structures with substantial façade openings spanning the hogging/sagging transition region show increased risk of building damage.

- Notably different building displacement profiles were observed at different building levels. At foundation level uniform horizontal displacements were measured, which is a result of the axial building stiffness restraining the transfer of horizontal soil displacements to the foundation. By contrast, at neutral axis level (i.e. slightly below the mid-height of the structure) and top building level gradients were observed within the horizontal displacement profiles. This observation is associated with bending and shear deformations, and implies that horizontal building strains might differ along the building height. Published data from field measurements of the Jubilee Line project confirm this observation (Burland et al., 2004).

- For most of the investigated cases the horizontal strains induced in the structures were negligible, as has been observed in the field (Burland et al., 2004; Mair, 2013), in previous centrifuge experiments (Farrell and Mair, 2010, 2011) and recent numerical research (Yiu et al., 2017). But for long structures with a considerable amount of openings average horizontal building strains at neutral axis of up to 40% compared to the greenfield equivalent were measured. For an isolated building façade with equal length and façade openings similar horizontal building strains were measured at $V_{l,t}$ below 1.0%. As $V_{l,t}$ developed even greater horizontal building strains were observed at neutral axis level which can be related to crack initiation. This indicates that horizontal strains, though often considerably less than predicted using the greenfield, might be appreciable in flexible structures.
Building response to tunnelling-induced subsidence

• Analysing the building response globally and locally revealed that the building response is a function of the building position relative to the tunnel and building characteristics. Substantial horizontal top strains and angular distortion values were obtained for long structures located in the greenfield hogging/sagging transition regions. Building damage in terms of cracking confirmed the building response, resulting from the local building response analysis.

• For the entire test series visible crack onset was observed at the upper building level and the cracks propagated vertically towards the building foundations. The rate of cracking was slow enough that crack progression could be observed within a single building level (e.g. top). For buildings with large openings, the crack onset at the upper building level might be related to local bending of thin elements which is typical when shear deformations are dominant for a building bay. Identical crack locations were often observed for structures placed at equal building-to-tunnel position which highlights the importance of the building position relative to the tunnel.

• The widely accepted approach of partitioning a structure at the theoretical greenfield point of inflection and individually assessing the hogging and sagging buildings parts was explored. Reasonable results for sagging parts of structures were observed. By contrast, hogging parts showed a significantly different structural response depending on the extension of the structure across the assumed partitioning location. The derived results indicated that neglecting the sagging part of a building when considering hogging might underestimate building damage. This implies that building damage assessment frameworks that are based on this partitioning approaches and also account for soil–structure interaction effects (e.g. Goh and Mair, 2011a) could result in unconservative predictions.

• The influence of building features on shear and bending deformations was investigated. Results showed that an increase of the building length caused an increase of bending deflections but had a little impact on shear deflections. Increasing the window opening percentage, however, resulted in a converse response; shear deflections considerably increased while bending deflections remained rather unchanged.
Chapter 8

Evaluation of current damage assessment methods

This chapter evaluates the performance of current procedures to assess building response to tunnelling-induced ground displacements. Firstly, the original formulations of Burland and Wroth (1974), Boscardin and Cording (1989) and Son and Cording (2005) are used to estimate building strains by employing the experimentally obtained building distortions presented in Chapter 7. Secondly, the obtained results are compared to initial damage assessments assuming the building follows the greenfield ground movements, which is expected to be overly conservative. Thirdly, the performance of latest assessment methods that account for the interaction between the soil and the building is explored, after which building damage categories are derived for the different configurations studied.

8.1 Background

Several methods exist to assess the risk of building damage due to ground movements, as was described in Section 2.2. Within this background section, important concepts to assess the potential degree of damage caused by tunnelling-induced soil displacements are re-emphasised. Table 8.1 classifies widely accepted procedures to quantify potential risk of building damage caused by tunnelling subsidence. Since Burland and Wroth (1974), the empirical analytical deep beam method (DBM) was constantly updated and soil structure interaction was gradually implemented. This is addressed in Table 8.1 with the prefix ‘A’ in the ‘No.’ column. Building strains are derived by idealising the building as a linear elastic fully flexible beam that follows the greenfield ground movements. Based on the estimated deflection ratio $DR$ and Timoshenko (1957) deep beam theory, for a central point load the maximum bending strains


### Evaluation of current damage assessment methods

Table 8.1 Damage criteria for tunnelling-induced building damage assessments (adapted from Finno et al., 2005). Assessment procedures based on greenfield soil settlements are noted GF while criteria accounting for soil–structure interaction are labelled SSI.

<table>
<thead>
<tr>
<th>No.</th>
<th>Method</th>
<th>Parameters</th>
<th>Settlement configuration</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>Deep beam model (DBM)</td>
<td>DR</td>
<td>GF</td>
<td>Burland and Wroth (1974); Burland et al. (1977)</td>
</tr>
<tr>
<td>A2</td>
<td>Extended deep beam model (EDBM)</td>
<td>β, εₕₜ</td>
<td>GF</td>
<td>Boscardin and Cording (1989)</td>
</tr>
<tr>
<td>B1</td>
<td>Strain superposition method (SSM)</td>
<td>crack width</td>
<td>GF</td>
<td>Boone (1996)</td>
</tr>
<tr>
<td>A3</td>
<td>Relative stiffness method (RSM), EDBM</td>
<td>DR, εₕₜ</td>
<td>SSI</td>
<td>Potts and Addenbrooke (1997)</td>
</tr>
<tr>
<td>C1</td>
<td>State of Strain (SoS)</td>
<td>β, εₕₜ</td>
<td>GF, SSI</td>
<td>Son and Cording (2005, 2007)</td>
</tr>
<tr>
<td>A4</td>
<td>Laminated beam model (LBM)</td>
<td>DR</td>
<td>GF</td>
<td>Finno et al. (2005)</td>
</tr>
<tr>
<td>A5</td>
<td>RSM, EDBM</td>
<td>DR, εₕₜ</td>
<td>SSI</td>
<td>Franzius et al. (2006)</td>
</tr>
<tr>
<td>A6</td>
<td>RSM, DBM</td>
<td>DR</td>
<td>SSI</td>
<td>Goh and Mair (2011a); Mair (2013)</td>
</tr>
</tbody>
</table>

\[ \varepsilon_{b,\text{max}} \text{ and maximum diagonal strains } \varepsilon_{d,\text{max}} \text{ are expressed as} \]

\[ \varepsilon_{b,\text{max}} = \frac{DR}{\left( \frac{L}{12T} + \frac{3IE}{2TLHG} \right)} \]  

\[ \text{and} \]

\[ \varepsilon_{d,\text{max}} = \frac{DR}{1 + \frac{HL^2G}{18TE}} \]  

Boscardin and Cording (1989) expanded the deep beam model by accounting for horizontal building strains \( \varepsilon_{ht} \) caused by tunnel excavation, which yields to resultant bending strains \( \varepsilon_{br} \) and resultant shear strains \( \varepsilon_{dr} \) which are estimated by

\[ \varepsilon_{br} = \varepsilon_{ht} + \varepsilon_{b,\text{max}} \]
and
\[ \varepsilon_{dr} = \frac{\varepsilon_{ht}}{2} + \sqrt{\left(\frac{\varepsilon_{ht}}{2}\right)^2 + \varepsilon_{d,max}^2}. \] (8.4)

The greater of these two strains is the maximum value of tensile strain \( \varepsilon \), for a given \( DR \) and \( \varepsilon_{ht} \) and is subsequently compared to limiting tensile strain values (Table 2.7) to assess the potential degree of building damage which explains why this concept is called the limiting tensile strain method (LTSM).

The remaining damage criteria with the prefix ‘A’ either consider soil–structure interaction effects, often expressed by a ratio of the relative stiffness between the building and the soil (RSM in Table 8.1), or idealise buildings as laminated beams (LBM in Table 8.1). This LBM considers shear restraint by building walls and bending restraint by building floors (Finno et al., 2005).

Son and Cording (2005) introduced an alternative criterion which is based on the state of strain (SoS) in a distorting building unit. The maximum principal tensile strain \( \varepsilon_p \) (equal to \( \varepsilon_t \), Dalgic et al., 2017) is estimated as a combination of \( \beta \) and \( \varepsilon_{ht} \) and defined as
\[ \varepsilon_p = \varepsilon_{ht} \cos(\theta_{max}^2) + \beta \sin(\theta_{max}) \cos(\theta_{max}) \] (8.5)

where \( \theta_{max} \) characterises the direction of crack formation and the angle between the vertical and the plane on which \( \varepsilon_p \) acts. \( \theta_{max} \) is expressed as
\[ \tan(2\theta_{max}) = \frac{\beta}{\varepsilon_{ht}} \] (8.6)

For initial predictions of building response to tunnelling-induced ground displacements, the change of the greenfield ground slope between adjacent building units may be used to approximate \( \beta \) (Son and Cording, 2005). The \( \varepsilon_{ht} \) is also estimated based on greenfield horizontal displacements at soil surface. Identical to the LTSM, Son and Cording (2005) proposed limiting strain values to derive damage categories based on the magnitude of \( \varepsilon_p \).

More recent research (Franzius et al., 2006; Goh, 2011; Mair, 2013; Potts and Addenbrooke, 1997; Son and Cording, 2005, 2007) gradually updated these initial approaches by accounting for soil–structure interaction effects; thus, notably reducing frequently reported overestimation of damage prediction procedures on the basis of greenfield ground movements. Nevertheless, these so-called relative stiffness methods (RSM) draw on the underlying assumptions of Burland and Wroth (1974) and Boscardin and Cording (1989). Further limitations of these methods are uncertainties in deriving the global stiffness of an affected building. Consequently, various researchers (Camós et al., 2014; DeJong et al., 2016; Giardina et al.,
Evaluation of current damage assessment methods

2017) reported considerable divergences between different prediction methods and monitored building performance.

It can be seen from Table 8.1 that currently three main methods exist to derive building strains due to tunnelling-induced settlements (i.e. groups ‘A’ to ‘C’). Concepts that account for the interaction between a building and the soil primarily build on the frameworks ‘A’ and ‘C’ while the damage criteria of Boone (1996) performs a very detailed analysis considering various details of the ground, the tunnel excavation and the building (Finno et al., 2005). It is essential to quantify potential differences between these classical concepts ‘A’ and ‘B’ to pave the way to more accurately assess the influence of tunnelling works on adjacent assets. In the following the performance of the introduced different approaches to estimate building strains (i.e. LTSM, which is a combination of ‘A1 and ‘A2’, and SoS) caused by tunnelling-induced ground movements is evaluated, after which the performance of the RSMs is discussed.

8.2 Performance of criteria to estimate building strains

So far this chapter recalled widely accepted methods to estimate building strains due to tunnelling-induced ground displacements. Within this section these criteria are adopted to calculate building strains for the conducted series of centrifuge model tests.

8.2.1 Limiting tensile strain method

An analytical assessment of the building response to tunnelling-induced soil displacements is performed by applying the principles of the LTSM to the measured building distortions. Additionally, traditional predictions were made by using the LTSM based on greenfield soil displacements. This assumes that the building is fully flexible and deforms according to the greenfield settlement trough. For the LTSM predictions the results of the greenfield test performed by Farrell (2010) were adopted.

Table 8.2 presents the input data for this analytical assessment. The measured vertical and horizontal deflections (i.e. $DR$ and $\varepsilon_{ht}$) of the buildings, presented in the Sections 7.3.3 and 7.4.3, and the corresponding greenfield values were used to derive the tensile strains. The $\varepsilon_{ht,GF}$ values were determined at surface soil level while $\varepsilon_{ht,Str}$ was estimated at neutral axis level of the buildings (Section 4.5.1.1). The contribution of horizontal compressive strains ($\varepsilon_{hc}$) was neglected throughout this chapter because as demonstrated by Netzel (2009) considering horizontal compressive strains might result in underestimation of building damage. Partitioning of the structures into hogging/sagging parts was conducted by using the inflection
## 8.2 Performance of criteria to estimate building strains

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Building tests</th>
<th>Greenfield</th>
<th>Explanation</th>
</tr>
</thead>
<tbody>
<tr>
<td>DR</td>
<td>DR&lt;sub&gt;Str&lt;/sub&gt;</td>
<td>DR&lt;sub&gt;GF&lt;/sub&gt;</td>
<td>for hogging and sagging, changes with ( V_{ij} )</td>
</tr>
<tr>
<td>( \varepsilon_{ht} )</td>
<td>( \varepsilon_{ht,Str} )</td>
<td>( \varepsilon_{ht,GF} )</td>
<td>for hogging and sagging, changes with ( V_{ij} )</td>
</tr>
<tr>
<td>( L )</td>
<td>( L_{Str} )</td>
<td>( L_{GF} )</td>
<td>for hogging and sagging, changes with ( V_{ij} )</td>
</tr>
<tr>
<td>( H )</td>
<td>90 mm</td>
<td>90 mm</td>
<td>constant</td>
</tr>
<tr>
<td>( I )</td>
<td>( I_{A-A} )</td>
<td>( I_{A-A} )</td>
<td>solid cross-section, changes for different building models</td>
</tr>
<tr>
<td>( t )</td>
<td>( t_{hog} = H - \bar{z}<em>{tot} ) and ( t</em>{sag} = \bar{z}_{tot} )</td>
<td>equal for building tests and greenfield, varies for different building configurations</td>
<td></td>
</tr>
<tr>
<td>( E/G )</td>
<td>7 for ( O = 20% ) and 15 for ( O = 40% )</td>
<td>according to Son and Cording (2007) for ( K_s = K_n^* )</td>
<td></td>
</tr>
</tbody>
</table>

*\( K_s \) and \( K_n \) describe the shear and normal stiffness of brick/mortar joints (Son and Cording, 2007).*

The second moment of area, \( I \), of the structure was determined based on the building’s solid cross-section (\( I_{A-A} \) see Figure 4.12b and Table 4.6). The distance between the neutral axis and the extreme fibre of the beam was estimated based on the theoretical position of the neutral axis of the building models (Section 4.5.1.1). For hogging deformations, the extreme fibre was assumed to be at the top level of the structures while for sagging the extreme fibre was assumed to be at the base of the building models. Window openings were considered using the relationship between \( E/G \) and window openings suggested by Son and Cording (2007). This relationship is based on a structure with \( L/H = 3 \) which is in fair agreement with the studied building configurations (i.e. \( L/H = 2.2 - 2.9 \)).

Figure 8.1 compares the induced building tensile strains with the predictions based on the greenfield settlement configuration. As expected, for all tests the greenfield predictions (GF) were conservative. This is particularly true for short structures (i.e. \( L/H = 2.2 \), tests A to D) that performed rather rigid. An increase of the building length and opening area led to building strains that were in better agreement with the greenfield estimates. This can be attributed to the more flexible building response for the longer structures with 40% openings; for test G the predicted building strains approximately converged with the observed building strains as \( V_{ij} \) developed (Figure 8.1g).
Evaluation of current damage assessment methods

Fig. 8.1 Maximum building tensile strains based on measured building distortions and related greenfield predictions by adopting the limiting tensile strain method (Burland et al., 1977; Burland and Wroth, 1974).
8.2 Performance of criteria to estimate building strains

It can be seen from Figure 8.1 that for all cases but test A, the hogging predictions of $\varepsilon_t$ exceeded the sagging equivalents. Likewise, the $\varepsilon_t$ values based on measured building distortions were greater in hogging than sagging for all tests but test A, which primarily performed in sagging. This indicates that the buildings in an asymmetric building-to-tunnel position performed more flexible in hogging than sagging, and this trend is captured by the greenfield estimates.

The data in Figure 8.1 reveal the impact of specific input parameters in the LTSM equations. For instance, the minor difference between the predicted $\varepsilon_t$ for test B (Figure 8.1b) and test D (Figure 8.1d) can be related to the increase of $E/G$ from 7 to 15 due to the increase in opening area for test D. For this reason, the LTSM predicts lower $\varepsilon_t$ for buildings with larger opening area (compare also test E (Figure 8.1e) with test F (Figure 8.1f)).

The significant impact of the estimate of the second moment of area $I$ per metre run on the calculation of $\varepsilon_t$ is evident when comparing the Figures 8.1f and 8.1g. The substantially greater estimate of $I$ for the isolated façade case (test G) resulted in considerably smaller $\varepsilon_t$ values compared to test F. This trend is in contradiction with the observed building response, as was discussed in Chapter 7; for test G, building damage was observed at lower $V_{lt}$ compared to test F. This finding implies that the method of estimating the plane-strain building stiffness for buildings with realistic layout needs refinement which is further addressed in Chapter 9.

Consistent trends between building features and building damage, as was reported in Chapter 7, are evident in Figure 8.1. For instance, for identical buildings placed at different position relative to the tunnel (tests A, B and C), the greatest $\varepsilon_t$ values were obtained for the building in the hogging/sagging transition region (test C). An increase of the opening percentage from 20% to 40% increases the risk of building damage (compare test B with test D and test E with test F). Increasing the building length leads to even more vulnerable cases (compare tests B and C with test E or test D with test F). As discussed above, the procedure of estimating the plane-strain building stiffness led to building tensile strains notably lower for the isolated façade (test G) than for test F.

To quantify the performance of the LTSM, two parameters are introduced. Firstly, a so-called modification factor of $\varepsilon_t$ ($M^{\varepsilon_t}$) is calculated by relating the building tensile strains based on observed building distortions $\varepsilon_{t,Str}$ to the building tensile strains according to the greenfield deformations $\varepsilon_{t,GF}$. Secondly, the prediction accuracy index (PAI) according to Schuster et al. (2009) is adopted to evaluate the accuracy of predicting the associated building damage category. The PAI subtracts the observed category of damage from the predicted category of damage (see Table 2.7 for damage categories). For instance, when a ’moderate’ damage (category of damage = 3) was predicted but a ’very slight’ damage (category of damage = 1) was
Evaluation of current damage assessment methods

![Graph showing performance of the LTSM as a ratio between building tensile strains based on observed building distortions and greenfield distortions.](image)

**Fig. 8.2** Performance of the LTSM as a ratio between building tensile strains based on observed building distortions and greenfield distortions.

observed, the damage category would be overpredicted by 2, and PAI = 2. Consequently, underprediction of building damage is indicated by a negative PAI.

Figure 8.2 plots $M^{\varepsilon_t}$ along $V_{l,t}$. For all tests, the greenfield estimates result in conservative predictions (i.e. $M^{\varepsilon_t}<1$) for $V_{l,t}<3.25\%$, as was expected. While $M^{\varepsilon_t}$ stayed rather constant or decreased for the structures with $L/H = 2.2$ (tests A–D), a notable increase of $M^{\varepsilon_t}$ can be seen for the longer structures (tests E–G). Specifically, for test G, $M^{\varepsilon_t}$ significantly increased as $V_{l,t}$ developed.

Table 8.3 summarises the $M^{\varepsilon_t}$ values presented in Figure 8.2 at specific $V_{l,t}$ values and tabulates PAI values. While notably different $M^{\varepsilon_t}$ values were observed for the different tests, it can be seen that the LTSM based on greenfield soil displacements is overly conservative. This is particularly true for lower $V_{l,t}$. Overall, an average $M^{\varepsilon_t}$ value of approximately 0.5 can be assumed based on the data presented in Table 8.3. This indicates that the calculated tensile strains based on experimentally obtained building distortions were on average 50\% smaller than the greenfield estimates. Unconservative predictions were only observed after extensive building damage was apparent (i.e. in test G).

Greenfield estimates of the LTSM resulted in damage categories greater or equivalent than obtained based on the measured building distortions, as can be seen from the PAI presented in Table 8.3. Specifically, the LTSM predictions substantially overestimated the response of the structures placed in the sagging region (test A); the damage category was overpredicted by 2 to 4 categories. For the remaining tests, the LTSM overpredicted the damage level by 1 to 2 categories and only for test G the predicted damage categories converged with the measured ones (PAI = 0). On average the LTSM greenfield estimates overpredicted the damage categories between 1 to 2 categories.
8.2 Performance of criteria to estimate building strains

Table 8.3 Assessment of the performance of the limiting tensile strain method. (SD standard deviation, PAI prediction accuracy index)

<table>
<thead>
<tr>
<th>Test</th>
<th>M^6 (= \varepsilon_{t,str}/\varepsilon_{t,GF})</th>
<th>PAI</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(V_{t,1} = 1.0%)</td>
<td>(V_{t,2} = 2.0%)</td>
</tr>
<tr>
<td>A (sag.)</td>
<td>0.19</td>
<td>0.11</td>
</tr>
<tr>
<td>B</td>
<td>0.35</td>
<td>0.33</td>
</tr>
<tr>
<td>C</td>
<td>0.39</td>
<td>0.36</td>
</tr>
<tr>
<td>D</td>
<td>0.47</td>
<td>0.41</td>
</tr>
<tr>
<td>E</td>
<td>0.40</td>
<td>0.53</td>
</tr>
<tr>
<td>F</td>
<td>0.67</td>
<td>0.70</td>
</tr>
<tr>
<td>G</td>
<td>0.49</td>
<td>0.74</td>
</tr>
<tr>
<td>Mean</td>
<td>0.42</td>
<td>0.46</td>
</tr>
<tr>
<td>SD</td>
<td>0.14</td>
<td>0.22</td>
</tr>
</tbody>
</table>

8.2.2 State of strain concept

A detailed study of the building response to tunnelling subsidence has been undertaken using the state of strain theory proposed by Son and Cording (2005). This was done by subdividing the buildings into bays (Figure 7.15) and adopting the horizontal strain at top level \(\varepsilon_{h,\text{top}}\) and the angular distortion \(\beta\) for every building unit. These parameters were derived in Section 7.5.2. Additionally, a greenfield prediction according to the SoS concept was conducted. For this assessment the horizontal building strains and the change of the ground slope between building units were determined based on the greenfield soil displacements at soil surface level as suggested by Son and Cording (2005). The horizontal tensile strain and the change of the ground slope, which is adopted to represent \(\beta\), are used as the input values for the Equations 8.5 and 8.6.

Figure 8.3 plots the derived principal strains \(\varepsilon_p\) based on the measured building distortions in comparison to greenfield predictions for every building bay. Damage levels according to Son and Cording (2005) are illustrated for reference. The boundaries of the 'moderate' and 'severe to very severe' damage levels differ slightly compared to the original ones proposed by Boscardin and Cording (1989) which may be attributed to the formulation of the state of strain concept.

Identical predictions for tests located at equivalent building-to-tunnel position were obtained when applying the SoS concept. This can be seen in Figure 8.3 when comparing the greenfield curves of test B with test D or test E with the tests F and G. The identical SoS predictions for tests at identical building location relative to the tunnel can be attributed to the simplicity of the SoS concept, which in its initial level of investigation (i.e. based on green-
Evaluation of current damage assessment methods

Fig. 8.3 Maximum principal tensile strains based on measured building distortions and related greenfield predictions by adopting the state of strain criteria proposed by Son and Cording (2005).
field soil displacements) neglects mechanical properties of the building and structural details such as the dimensions of the cross-section or window openings.

From Figure 8.3 it can be seen that the maximum \( \varepsilon_p \) estimates were obtained in bays with distinct horizontal tensile strains. This result can be related to the considerable impact of horizontal tensile strains on building damage while horizontal compressive strains were not taken into account when calculating \( \varepsilon_p \), as was indicated in Equations 8.5 and 8.6. Because of this, the building bays with predicted maximum values of \( \varepsilon_p \) differ and are not always the bays closest to the tunnel centreline. This observation is in contrast to Son and Cording (2005), who reported damage concentration in the building bay closest to the source of ground movements but this previous work focused on buildings adjacent to deep excavations where the structure usually responds in hogging. However, for tests F and G extensive building damage was observed and for both tests the maximum \( \varepsilon_p \) values were measured in bay 2, which is identical to the predictions based on \( \varepsilon_{ht} \) and \( \beta \) (Figures 8.3f and 8.3g).

Figure 8.3 indicates that the SoS assessment based on greenfield soil displacements is often overly conservative, as was expected. However, it can also be seen from Figures 8.3f and 8.3g that the measured building strains according to the SoS are sensitive to the onset of building damage. This results in \( \varepsilon_p \) values that were significantly greater than the associated greenfield estimates, and implies that the SoS concept is sensitive to local effects (i.e. strain localisation) which is a result of the focus on individual building units. Consequently, the impact of building damage in terms of cracking is more pronounced in the SoS concept compared to the LTSM, which is further explored in Section 8.2.3.

In order to quantify the performance of the SoS estimates based on greenfield displacements, a modification factor \( (M^{\varepsilon_p}) \) for \( \varepsilon_p \) is introduced which relates \( \varepsilon_p,Str \) to \( \varepsilon_p,GF \). Figure 8.4 plots this \( M^{\varepsilon_p} \) along \( V_{l,t} \) for all tests. At low \( V_{l,t} \) the SoS concept notably overpredicts \( \varepsilon_p \) for all cases except for test G. A good agreement between the predicted \( \varepsilon_p \) and the \( \varepsilon_p \) based on measured building distortion can be seen for test G at \( V_{l,t}<1.6\% \). After this \( V_{l,t} \) cracking occurred and thus the measured \( \varepsilon_p \) significantly increased. This suggests that the post-cracking behaviour could not be captured with the SoS greenfield prediction. The distinct change of \( M^{\varepsilon_p} \) for test F is also related to the onset of building damage, which occurred at approximately \( V_{l,t} = 2.6\% \).

Table 8.4 assesses the performance of the SoS at \( V_{l,t} \) of 1.0%, 2.0% and 4.0%. For \( V_{l,t} \) values of 1.0% and 2.0%, which in most cases can be related to the pre-cracking building behaviour, the observed \( \varepsilon_p \) values were on average below 40% of the estimates. Significantly greater values were observed for test G. As \( V_{l,t} \) developed, \( M^{\varepsilon_p} \) increased which, as mentioned above, is the result of building damage in tests F and G. This trend can also be related to the
Evaluation of current damage assessment methods

*Fig. 8.4* Performance of the SoS as a ratio between building tensile strains based on observed building distortions and greenfield distortions.

*Table 8.4* Assessment of the performance of the state of strain concept. (SD standard deviation, PAI prediction accuracy index)

<table>
<thead>
<tr>
<th>Test</th>
<th>$M_{\varepsilon_P}$ ($= \varepsilon_{p,Str}/\varepsilon_{p,GF}$)</th>
<th>PAI</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$V_{l,t} = 1.0%$</td>
<td>$V_{l,t} = 2.0%$</td>
</tr>
<tr>
<td></td>
<td>$V_{l,t} = 1.0%$</td>
<td>$V_{l,t} = 2.0%$</td>
</tr>
<tr>
<td>A (sag.)</td>
<td>0.12</td>
<td>0.09</td>
</tr>
<tr>
<td>B</td>
<td>0.14</td>
<td>0.10</td>
</tr>
<tr>
<td>C</td>
<td>0.08</td>
<td>0.16</td>
</tr>
<tr>
<td>D</td>
<td>0.18</td>
<td>0.15</td>
</tr>
<tr>
<td>E</td>
<td>0.26</td>
<td>0.38</td>
</tr>
<tr>
<td>F</td>
<td>0.43</td>
<td>0.42</td>
</tr>
<tr>
<td>G</td>
<td>0.92</td>
<td>1.41</td>
</tr>
<tr>
<td>Mean</td>
<td>0.31</td>
<td>0.38</td>
</tr>
<tr>
<td>SD</td>
<td>0.29</td>
<td>0.47</td>
</tr>
</tbody>
</table>

more pronounced non-linear behaviour of the 3D printed material as the tunnelling-induced displacements increase.

The PAI for the SoS concept is also presented in Table 8.4. A notable overestimation of the damage categories is apparent for most of the tests. Specifically, PAI > 2 were obtained for the shorter structures (tests A–D). For the remaining configurations with 3D building layout (tests E and F), PAI values equal or greater than zero were derived while for test G a PAI value of -1 was calculated at $V_{l,t} = 4.0\%$. On average the SoS greenfield predictions of the damage category were approximately 2 categories higher than the observed damage, though substantial deviation is evident from Table 8.4.
8.2 Performance of criteria to estimate building strains

Figure 8.5 compares the building tensile strains based on measured building deformations according to the LTSM and the SoS definitions at $V_{lt} = 2.0\%$. For each test only the maximum LTSM strain (i.e. in hogging or sagging) and maximum SoS strain (i.e. in bay 1, bay 2, bay 3 or bay 4) is plotted. For tests A to F, the strain values derived with the LTSM exceed the strains determined according to the SoS methodology. A different trend was observed for test G; notably greater SoS values were derived compared to the LTSM equivalent. As mentioned above, this result can be related to the ability of the SoS procedure to better capture strain localisation compared to the LTSM. Moreover, the LTSM considers structural details including the mechanical properties and the building cross-section which additionally reduces the calculated tensile strains of test G.

To further explore the performance of the LTSM and the SoS criteria, the ability of these criteria to capture strain localisation is evaluated. Therefore, the derived building strains based on the measured building distortions ($\varepsilon_t$ or $\varepsilon_p$) are compared with the ultimate strain of the 3D printed material (Table 4.1). This concept results in a so-called damage parameter index (DPI, adapted from Schuster et al., 2009) which can be written as

\[
\text{DPI} = \frac{\varepsilon_t}{\varepsilon_{ult}}
\]  

(8.7)
where $\varepsilon_t$ (or $\varepsilon_p$) are the derived tensile strains and $\varepsilon_{ult}$ the ultimate strains of the 3D printed material. Theoretically, at DPI = 1 the building models experience failure and visible cracking should be observed at rather equal $V_{lf}$ values.

Figure 8.6 plots the DPI values for all tests along $V_{lf}$. From Figure 8.6a a rather linear relationship between DPI and $V_{lf}$ is evident. A notable change due to the onset of building damage could not be captured with the LTSM definitions (compare curves for test F and G with associated lines indicating cracking damage in Figure 8.6a). By contrast, the SoS criteria captures strain localisation effects significantly better as is apparent from Figure 8.6b. Specifically, for test G, DP = 1 was approximately obtained where the failure line converges with the visible crack line of test G. Also for test F, the notable change of the ratio between DPI and $V_{lf}$ can be related to the observed building damage. This implies that the SoS criteria significantly better replicates strain localisation effects compared to the LTSM which can be attributed to analysing smaller building sections and using the top horizontal strain as a measure of building distortion.

The increase of building tensile strains with $V_{lf}$ is plotted in Figure 8.7. In addition to the tensile strains based on the measured building distortions, the greenfield estimates are also shown. Similar to above, only the maximum tensile strains based on the measured building distortions and the greenfield equivalents are presented. As expected, for all tests the strains developed with $V_{lf}$. For pre-cracking, the building strains based on measured building distortions were notably greater according to the LTSM than to the SoS criteria. Only for the isolated façade test (test G), where the building stiffness estimate results in significant overestimation of the second moment of area, the LTSM values were lower than the SoS strains. The building strains based on the SoS clearly indicate the crack initiation that was observed for the tests F and G (Figures 8.7f and 8.7g).

Except for the tests A and C, the LTSM and the SoS concept tend to provide greenfield estimates that were in rather good agreement. The considerably greater LTSM predictions for tests A and C can be related to the very high estimates of $DR_{GF}$ and $\varepsilon_{ht,GF}$, respectively. Only for test G, the SoS greenfield assessment was notably greater than the LTSM equivalent, which can be related to the overestimate of the building stiffness of test G.

Overall, the data in Figure 8.7 confirms that estimates based on greenfield displacements, which are often employed in an initial investigation, tend to be overly conservative. To provide more adequate prediction in particular for lower tunnel volume loss values, recent concepts of building damage assessment account for the interaction between a structure and the ground. The section below discusses the accuracy of these methods.
8.2 Performance of criteria to estimate building strains

Fig. 8.6 Damage parameter index (DPI) for limiting tensile strain (LTSM) and state of strain (SoS) criteria.
Fig. 8.7 Comparison between LTSM and SoS predictions and derived building tensile strains versus $V_{lt}$. Solid lines indicate damage categories defined by Boscardin and Cording (1989) while dashed lines show the updated damage category boundaries according to Son and Cording (2005).
8.3 Performance of the relative stiffness methods

Current methods that account for soil–structure interaction when assessing building response to tunnelling-induced subsidence are mainly based on an extensive set of data from computational models but are limited by the lack of validation from case histories or experimental data. Within this section, the performance of the so-called relative stiffness methods (RSM) is evaluated by making use of the obtained centrifuge model test results.

8.3.1 Estimating the relative stiffness

This section will examine the estimation of different relative stiffness formulations including the methods proposed by Potts and Addenbrooke (1997), Son and Cording (2005), Franzius et al. (2006) and Goh and Mair (2011a). These procedures relate the global building stiffness to the soil stiffness to account for soil–structure interaction effects. In order to derive the relative stiffness, the bending and axial stiffness values of the building and the soil stiffness have to be estimated, as was extensively discussed in Section 4.5. The section below is concerned with the estimation of the soil stiffness and accounts for a soil stiffness degradation with induced tunnel volume loss.

8.3.1.1 Soil stiffness

Potts and Addenbrooke (1997) suggested to obtain the soil stiffness by retrieving a specimen at half of the tunnel depth and testing the soil sample in a triaxial compression test to estimate the secant stiffness, $E_{sec}$, at 0.01% axial strain, $\varepsilon_a$. This framework was followed by Franzius et al. (2006) and Goh and Mair (2011a), while Son and Cording (2005) recommended to estimate the soil stiffness in the influence zone of the building foundation. Zhao (2008) performed triaxial compressive tests on Leighton Buzzard Fraction E silica sand, which are adopted to quantify the influence of the tunnel excavation on the soil stiffness degradation in the performed centrifuge test series. The framework below is adapted from Elshafie (2008).

Figure 8.8a shows the $E_{sec}$ to $\varepsilon_a$ relationship for a triaxial test of Zhao (2008) with a relative density, $I_D$, of 70% ($e = 0.74$) and an initial mean effective stress, $p'$, of 100 kPa. The shear stiffness, $G_{sec}$, decay with shear strain, $\gamma$, is shown in Figure 8.8b. To account for the different soil conditions in the centrifuge experiments (i.e. $I_D = 90\%$ ($e = 0.65$) and $\sigma'_v = 90$ kPa at $z_t/2$), Equation 8.8 proposed by Hardin and Richart (1963) and the expressions for vertical stiffness reduction (Equation 8.9) by Lehane and Cosgrove (2000) are employed to describe the $E_{sec}$ to
Evaluation of current damage assessment methods

$\varepsilon_a$ data of Zhao (2008):

$$F(e) = \frac{(2.17 - e)^2}{1 + e} \quad (8.8)$$

$$E'_v = \begin{cases} 
E_v0 & \text{for } \varepsilon \geq \varepsilon_{el} \\
1 + \left( \frac{\varepsilon - \varepsilon_{el}}{\varepsilon_r - \varepsilon_{el}} \right)^n & \text{for } \varepsilon \leq \varepsilon_{el} \\
A_E F(e) \left( \frac{\sigma'_v}{p_{atm}} \right)^{0.5} & \text{for } \varepsilon \leq \varepsilon_{el} 
\end{cases} \quad (8.9)$$

where $F(e)$ is a function related to the voids ratio, $E'_v$ the secant stiffness modulus of the soil in vertical direction, $E_v0$ the very small strain vertical Young’s modulus, $\varepsilon$ the current strain, $\varepsilon_{el}$ the linear elastic limit (i.e. the strain when the stiffness to strain relation becomes non-linear) and $\varepsilon_r$ the strain when $E'_v$ is half of the initial value ($E_v0$), $A_E$ a material constant, $n$ an empirical constant and $p_{atm}$ the atmospheric pressure (100 kPa), as shown in Figure 8.8. To fit the data from Zhao (2008) the parameters $\varepsilon_{el} = 1e-5$, $\varepsilon_r = 4e-4$, $E_v0 = 235$ MPa and $e = 0.74$ were used, which results in $A_E$ and $n$ values of 200 and 0.468 respectively (Figure 8.8a). The dotted line in Figure 8.8a indicates that this procedure fits well the triaxial test data from Zhao (2008). After this calibration, the Lehane and Cosgrove (2000) framework was updated to the soil conditions of the centrifuge tests. The $G_{sec}$ to $\gamma$ data was also curve fitted, as shown with the dashed line in Figure 8.8b. To obtain the $G_{sec}$ to $\gamma$ relation for the soil conditions of the centrifuge tests, a correction factor for $G_{sec}$, according to Franza (2017), was determined by

$$\frac{G_{sec}(\gamma, \sigma'_v = 90 \text{ kPa}, I_D = 90\%)}{G_{sec}(\gamma, \sigma'_v = 100 \text{ kPa}, I_D = 70\%)} = \frac{E_{sec}(\gamma, \sigma'_v = 90 \text{ kPa}, I_D = 90\%)}{E_{sec}(\gamma, \sigma'_v = 100 \text{ kPa}, I_D = 70\%)} = 1.08 \ . \quad (8.10)$$

The obtained factor was subsequently used to derive the $G_{sec}$ to $\gamma$ ratio for the desired soil conditions, as illustrated in Figure 8.8b. Although this correction caused a minor difference in the range of shear strains relevant for typical tunnelling works (Figure 8.8b), the corrected $G_{sec}$ to $\gamma$ relation was used to determine the $E_{sec}$ degradation with tunnel volume loss.

The greenfield centrifuge test conducted by Farrell (2010) was employed to estimate a shear strain to volume loss relationship. Therefore, the average shear strains, $\gamma_{av}$, at half of the tunnel depth were derived by

$$\gamma_{av}(z_t/2) = \frac{1}{5x^*} \int_{-2.5x^*}^{2.5x^*} \gamma_{xz} dx \quad (8.11)$$
8.3 Performance of the relative stiffness methods

![Graphs and diagrams showing the performance of relative stiffness methods.]

(a) Secant soil stiffness degradation.

(b) Secant shear stiffness degradation.

(c) Average shear strain versus tunnel volume loss.

(d) Secant soil stiffness versus tunnel volume loss.

Fig. 8.8 Degradation of soil stiffness with induced tunnel volume loss. Triaxial test data from Zhao (2008).
Evaluation of current damage assessment methods

where \( \gamma_{av} \) was obtained based on \( \gamma_{xz} \) discussed in Chapter 6. Finally, the \( \gamma_{av} \) to \( V_{l,t} \) relation, shown in Figure 8.8b, is adopted to derive the \( E_{sec} \) values for the different induced volume losses. To estimate \( E_{sec} \) from \( G_{sec} \), \( E = 2G(1 + \nu) \), assuming a Poisson’s ratio of 0.25 for sands, was used. Figure 8.8d depicts the derived \( E_{sec} \) to \( V_{l,t} \) degradation.

For the building tests, a constant building load of 100 kPa underneath the transversal strip foundations was simulated. To estimate the effect of this building load on the vertical soil stresses and the soil stiffness, Boussinesq (1885) expression for strip loads was applied. As was pointed out by Burland et al. (1977), the Boussinesq equations provide reasonable estimates of vertical soil stress changes for most ground conditions. The vertical stress due to the strip load, \( \sigma_v \), acting at an arbitrary point, \( P \), can be derived from

\[
\sigma_v(P) = \frac{q}{\pi} \left[ \beta + \sin \beta \cos(\beta + 2\delta) \right]
\]

(8.12)

where \( q \) is the building load per unit area, \( \beta \) the angle between the two lines connecting \( P \) with the endpoints of the foundation and \( \delta \) the angle between the vertical and the line connecting \( P \) with the right hand side endpoint of the foundation. Although the building and foundation layout does not represent a plane-strain condition, applying this assumption is a very conservative estimate of the potential vertical soil stress increase due to the building load. For the different building scenarios modelled, Equation 8.12 results in a maximum vertical stress increase of about 15 kPa at \( z_t/2 \), and a vertical soil stress of 105 kPa. An increase of \( E_{sec} \) with the square-root of \( z \) was considered, which represents an idealised soil model often applied for uniform deposits of cohesionless soils (Gazetas, 1984). According to this model, Equation 8.13 can be used to determine the soil stiffness increase due to the building surcharge

\[
\frac{E_{sec,str}}{E_{sec,GF}} = \sqrt{\frac{\sigma_{v,str}}{\sigma_{v,GF}}}
\]

(8.13)

where \( E_{sec,str} \) is the secant soil stiffness beneath the building at a depth \( z \), \( E_{sec,GF} \) the greenfield soil stiffness at identical soil depth, \( \sigma_{v,str} \) the vertical soil stress including the building surcharge (i.e. 105 kPa at \( z = z_t/2 \)) and \( \sigma_{v,GF} \) the vertical soil stress of the greenfield condition (i.e. 90 kPa at \( z = z_t/2 \)). The building surcharge results in a maximum increase of the soil stiffness of about 8% as indicated with the dashed line in Figure 8.8d. This maximum increase of \( E_{sec} \) compared to the greenfield case was considered negligible and because of this the \( E_{sec,GF} \) to \( V_{l,t} \) relation was employed to estimate the relative stiffness of the centrifuge model buildings (Section 8.3.2).

By contrast to the procedure discussed above, Potts and Addenbrooke (1997) recommended to determine \( E_{sec} \) at \( \varepsilon_a = 0.01\% \). This approach is illustrated in Figure 8.8a and the
8.3 Performance of the relative stiffness methods

Table 8.5 Soil stiffness values used to estimate relative stiffness.

<table>
<thead>
<tr>
<th>Phase</th>
<th>$E_s$ (MPa)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design value (RSM)</td>
<td>175</td>
<td>upper bound at $\varepsilon_a = 0.01%$</td>
</tr>
<tr>
<td></td>
<td>45</td>
<td>lower bound at $V_{l,t} = 2.0%$</td>
</tr>
<tr>
<td>Performance assessment</td>
<td>150 - 24</td>
<td>for $V_{l,t} = 0% - 4%$</td>
</tr>
</tbody>
</table>

derived $E_{sec}$ is about 175 MPa. In comparison to the soil stiffness degradation with $V_{l,t}$, shown in Figure 8.8d, the design value of 175 MPa represents an upper bound and hence results in a conservative assumption of the relative stiffness.

Son and Cording (2005) suggested to derive the soil stiffness within the influence zone affected by the building foundation. However, the authors do not provide guidance on the related strain level. It was therefore decided to use the identical $E_{sec}$ than will be applied for the other RSMs. This estimation was considered to be appropriate because the vertical soil stress beneath the foundations (approximately 100 kPa) is in fair agreement with the vertical soil stress at $z_t/2$ (approximately 90 kPa).

To account for the stiffness degradation with induced volume loss when applying the RSMs, a lower bound design value was also adopted for $E_{sec}$ at $V_{l,t} = 2.0\%$. As shown in Figure 8.8d, this lower bound $E_{sec}$ design value was estimated to be about 45 MPa. Table 8.5 summarises the obtained $E_{sec}$ values for assessing the performance of the centrifuge model buildings and when predicting the building response (i.e. design values).

8.3.2 Relative stiffness

In the following, the relative stiffness expressions of Potts and Addenbrooke (1997), Franzius et al. (2006), Goh and Mair (2011a) and Son and Cording (2005) are determined. Sections 4.5 and 8.3.1.1 described the estimation of the global building stiffness and the soil stiffness respectively. Tables 4.11 and 8.5 summarise the main input parameters to derive the relative stiffness. A degradation of the building stiffness due to induced strains or even cracking was not considered.

8.3.2.1 Relative stiffness expressions with focus on bending deflection

Estimates of the global relative stiffness in bending, $\rho^*$, and axially, $\alpha^*$, according to different literature are illustrated in Figure 8.9. The markers for each test indicate the relative stiffness values at a $V_{l,t}$ of 0.5%, 1.0%, 2.0% and 4.0%. The entire range of $\rho^*$ and $\alpha^*$ values due to the decay of $E_s$ is illustrated by the corresponding lines. Furthermore, the pentagonal and hexag-
Evaluation of current damage assessment methods

Fig. 8.9 Estimation of relative stiffness formulations according to literature.

(a) According to Potts and Addenbrooke (1997).

(b) According to Franzius et al. (2006).

(c) According to Goh and Mair (2011a) - sagging.

(d) According to Goh and Mair (2011a) - hogging.

Fig. 8.9 Estimation of relative stiffness formulations according to literature.

..., and...
8.3 Performance of the relative stiffness methods

result in rather similar $\rho^*$ values whereas the $\alpha_{\text{mod}}^*$ values according to Franzius et al. (2006) were found to be slightly lower than the ones of Potts and Addenbrooke (1997).

The RSM of Goh and Mair (2011a) partitions a structure into sagging and hogging regions and determines the relative bending stiffness for each region individually. This so-called partitioning approach results in two independent relative stiffness values for buildings spanning the greenfield inflection point. Although the soil–structure interaction substantially alters the position of the theoretical greenfield inflection point, $i_{gf}$, as reported in Section 6.4.1.2, in the design stage a structure is generally partitioned at the greenfield inflection point. The data shown in Figures 8.9c and 8.9d were calculated by adopting the $i_{gf}$ at $V_{l,t} = 1.0\%$, which results in an $i_{gf}$ of approximately 4.5 m at prototype scale (i.e. 60 mm at model scale). Based on this assumption, the buildings of the tests B and D are solely in the hogging region. For the relative axial building stiffness, Goh and Mair (2011c) followed the definition of Franzius et al. (2006).

A specific design chart was proposed for every RSM, similar to the relative stiffness formulations shown in Figure 8.9. Because of this, the different relative stiffness values are not directly comparable. Figures 8.9c and 8.9d, however, indicate the crucial role of the partitioned lengths (i.e. $L_{sag}$ and $L_{hog}$) when estimating $\rho_{\text{sag,part}}^*$ and $\rho_{\text{hog,part}}^*$, which are subsequently applied to the identical design chart. For instance for test A, which is predominantly located in sagging, $\rho_{\text{sag,part}}^*$ is substantially greater than $\rho_{\text{hog,part}}^*$. This result is caused by the small $L_{hog}$; thus, the hogging part of the building is assumed to respond more rigid than the sagging part of the same structure. By contrast, the $\rho_{\text{part}}^*$ of the tests C, E, F and G indicate a more flexible building response in hogging than in sagging, which stems from the greater $L_{hog}$ compared to $L_{sag}$. These findings suggest a strong correlation between an accurate assessment of the building response and determining $L_{sag}$ and $L_{hog}$ when applying the partitioning approach of Goh and Mair (2011a). However, soil–structure interaction effects can significantly alter a theoretical greenfield settlement profile, the position of the inflection point and hence $L_{sag}$ and $L_{hog}$.

8.3.2.2 Relative stiffness expression with focus on shear deflection

The RSM of Son and Cording (2005) focuses on the shear stiffness of buildings. Figure 8.10 shows the calculated relative shear and axial stiffness of the centrifuge model buildings. In contrast to the $\rho^*$ values discussed above, their method relates the soil stiffness, $E_s$, to the global building stiffness in shear, $G_{A_{\text{eq}}}$ (Table 4.5). Because of this, the decay of $E_s$ with ground loss leads to a decrease of the relative shear stiffness as volume loss increases. An approximate linear relationship between the axial and shear relative stiffness in logarithmic
Evaluation of current damage assessment methods

scale is evident from the data plotted in Figure 8.10. The $E_s$ decay results in approximately an order of magnitude change of the axial and shear relative stiffness. From Figure 8.10 a notable difference in the axial stiffness of test G compared to the remaining cases is apparent. This difference is a result of the relative axial stiffness formulation defined by Boscardin and Cording (1989) which accounts for the distance between individual strip footings using the spacing, $S$, between strip footings. For test G, which replicated an isolated façade founded on a strip footing, $S = 1$ metre was used; the effect of this assumption is discussed in Section 9.1.

The remaining parts of this section compare the experimentally obtained building distortions in vertical and horizontal direction with predictions from the RSMs. This involves an application of the design charts of the RSMs (Section 2.2) to derive modification factors for building distortions based on the relative stiffness estimates. Measured building distortions, which were presented in Chapter 7, are used for evaluation. Finally, associated building tensile strains are calculated and compared to limiting tensile strain values (Boscardin and Cording, 1989; Son and Cording, 2005).

8.3.3 Relative stiffness methods with focus on bending deflections

This section evaluates the performance of RSMs that focus on bending deflections to predict building response to tunnelling-induced ground displacements.
8.3 Performance of the relative stiffness methods

8.3.3.1 Modification factors for vertical building distortions

Figure 8.11 relates estimates of $M_{DR}^{hog}$ according to Potts and Addenbrooke (1997), Franzius et al. (2006) and Goh and Mair (2011a) to the experimentally obtained $M_{DR}^{hog}$. The design curves for $M_{DR}^{hog}$ with respect to the $e/L$ ratio of the corresponding centrifuge test were adopted to derive the predictions. For the RSM of Goh and Mair (2011a), a mean design envelope, which is the average between the proposed upper and lower design envelopes (Figure 2.17a), was employed for all tests.

The $M_{DR}^{hog}$ estimates using the upper design values of the relative stiffness values (Figure 8.9) are considerably greater than the lower design values (compare left to right of Figure 8.11). This finding was expected because the sensitivity of buildings to tunnelling-induced ground displacements increases with increasing soil stiffness (Netzel, 2009; Son, 2015). Stiffer soil is less affected by the presence of structures and thus tunnelling-induced soil displacements are less altered due to the interaction with an overlying building. Consequently, the building experiences substantial distortions. This implies that the estimate of $E_s$ according to Potts and Addenbrooke (1997) often results in a conservative assessment of the potential associated damage.

What is striking about Figure 8.11 is the significant scatter between the different RSMs. The greatest $M_{DR}^{hog}$ values were derived by applying the approach of Franzius et al. (2006), followed by the Goh and Mair (2011a) prediction. The lowest $M_{DR}^{hog}$ values were obtained when applying the Potts and Addenbrooke (1997) procedure. Previous work of Giardina et al. (2017) resulted in similar conclusions.

For all tests and the entire range of $V_{lt}$ presented, the upper bound $M_{DR}^{hog}$ estimates of Franzius et al. (2006) provided a conservative estimate of the measured modification factors. Even modification factors greater than unity were determined which would imply that the building distorts more than the greenfield equivalent. Moreover, also the lower bound values of Franzius et al. (2006) were overly conservative and greater than unity for all tests. Only for $V_{lt}$ values greater than 3.0%, when notable building damage for the tests F and G occurred, the lower bound predictions are in reasonable agreement with the experimental results.

The Goh and Mair (2011a) estimates of $M_{DR}^{hog}$ were always between the estimates of Potts and Addenbrooke (1997) and Franzius et al. (2006). Upper bound predictions of Goh and Mair (2011a) tend to be conservative for structures located mainly in hogging (tests B and D) while the lower bound predictions provided a reasonable estimate. The upper bound assessments for buildings spanning the greenfield inflection point (tests C and E-G) were in fair agreement at lower $V_{lt}$ values but resulted in unconservative estimates for long buildings with extensive...
Evaluation of current damage assessment methods

(a) Test B: upper bounds.

(b) Test B: lower bounds.

(c) Test C: upper bounds.

(d) Test C: lower bounds.

(e) Test D: upper bounds.

(f) Test D: lower bounds.

(g) Test E: upper bounds.

(h) Test E: lower bounds.
façade openings (tests F and G). Unconservative predictions were observed for structures in the sagging/hogging zone when applying lower bound Goh and Mair (2011a) predictions.

The Potts and Addenbrooke (1997) $M_{DR_{hog}}$ predictions were least conservative as is evident from Figure 8.11. While both the upper and lower design values often resulted in underprediction of the building distortions for buildings placed in the hogging/sagging transition zone, the $M_{DR_{hog}}$ values according to Potts and Addenbrooke (1997) provided reasonable estimates for structures located in the hogging zone (tests B and D).

Figure 8.12 compares the measured modification factors for the deflection ratio in sagging to the calculated predictions according to the three presented variations of the RSMs. Similar to above, upper and lower bound predictions are plotted. Results show that the RSM formulation of Franzius et al. (2006) tends to give the most conservative estimates of $M_{DR_{sag}}$, which is identical to what was observed for $M_{DR_{hog}}$. The Potts and Addenbrooke (1997) and Goh and Mair (2011a) RSM expressions resulted in similar predictions when applying the upper bound design value for the soil stiffness. However, the RSM formulations according to Goh and Mair...
Evaluation of current damage assessment methods

(2011a) gave the least conservative estimates when using the lower bound design values, and \( M^{DR_{sag}} \) values of approximately zero were obtained for all tests with \( e/L > 0 \).

For test A, the measured \( M^{DR_{sag}} \) values were smaller than the upper and lower bound predictions of the three RSMs. This indicates that the RSM predictions for the structure located primarily in sagging is conservative. The measured \( M^{DR_{sag}} \) values of the remaining structures, which predominantly deformed in a hogging mode, were often contained between the estimates. By contrast, the RSM formulations were unconservative for test G, which responded very flexible in sagging and hogging, as was pointed out in Chapter 7.

To further quantify the performance of the original RSM formulations by Potts and Addenbrooke (1997), Franzius et al. (2006) and Goh and Mair (2011a), Figure 8.13 provides a detailed comparison between the measured and predicted \( M^{DR} \) values. Figures 8.13a, 8.13b and 8.13c relate the measured \( M^{DR} \) values of all tests to RSM estimates at \( V_{l,f} = 2.0\% \). For the different tests, the main mode of building deflection (i.e. sagging or hogging) is plotted. In other words, for test A, the \( M^{DR_{sag}} \) values are presented while for the remaining tests the \( M^{DR_{hog}} \) values are shown. Both the upper and lower bound design values are plotted in Figure 8.13. The ratio of the measured to the predicted \( M^{DR} \) values was calculated for every variation of the RSM and for a \( V_{l,f} \) range between 0.5\% to 4.0\% as can be seen in Figure 8.13d. A measured/predicted ratio smaller than unity implies that the predicted \( M^{DR} \) value overpredicts the measured \( M^{DR} \) value, which is a conservative estimate. Statistical parameters such as the mean and standard deviation (SD) are also indicated.

In all cases, the predictions notably diverge from the measurements. This is particularly evident for the predictions according to the RSM expressions of Franzius et al. (2006) where only the prediction for test G is within the 25\% overprediction or underprediction line, as shown in Figure 8.13b. The remaining estimates of \( M^{DR} \) are even more conservative and mean ratios between the measured and predicted \( M^{DR} \) values were as low as 0.44 and 0.48 for the upper and lower bound predictions, respectively. Such low ratios of measured to predicted mean \( M^{DR} \) values were observed for the entire range of \( V_{l,f} \) discussed (Figure 8.13d). This suggests that applying the RSM formulations of Franzius et al. (2006) most likely results in overly conservative estimates which might result in costly mitigation measures.

The mean values for the measured to predicted \( M^{DR} \) values according to the RSM expressions of Potts and Addenbrooke (1997) are substantially higher (Figure 8.13a) than the Franzius et al. (2006) equivalents (Figure 8.13b). While the mean of the upper bound predictions of Potts and Addenbrooke (1997) indicates a conservative prediction, the lower bound estimates generally underpredicted \( M^{DR} \). In particular at \( V_{l,f} < 2.5 \), the upper bound Potts and Addenbrooke (1997) estimates often tend to give a reasonable prediction close to the mea-
8.3 Performance of the relative stiffness methods

(a) Test A: upper bounds.

(b) Test A: lower bounds.

(c) Test C: upper bounds.

(d) Test C: lower bounds.

(e) Test E: upper bounds.

(f) Test E: lower bounds.

(g) Test F: upper bounds.

(h) Test F: lower bounds.
Evaluation of current damage assessment methods

Fig. 8.12 Predicted versus measured $M^{DR_{seg}}$ along $V_{l,t}$. 'P&A', 'Fra.' and 'G&M' indicate the estimates of Potts and Addenbrooke (1997), Franzius et al. (2006) and Goh and Mair (2011c). 'ub' and 'lb' are abbreviations of upper and lower bound.

Measured $M^{DR}$ as is evident from Figure 8.13d. However, the significant SD suggests that upper bound $M^{DR}$ values according to the RSM formulation of Potts and Addenbrooke (1997) may be unconservative. At a $V_{l,t}$ of 2.0% such unconservative estimates were observed for the tests C, F and G (Figure 8.13a), which are tests with buildings located in the sagging/hogging transition zone.

Predictions of $M^{DR}$ according to the RSM formulation defined by Goh and Mair (2011a) resulted in measured over predicted mean $M^{DR}$ values that were between the Franzius et al. (2006) and Potts and Addenbrooke (1997) equivalents (Figure 8.13d). Specifically, the upper bound predictions with a mean measured to predicted $M^{DR}$ value of 0.78 and a SD of 0.35 at $V_{l,t} = 2.0\%$ indicate reasonable estimates of the vertical building distortions (Figure 8.13c). Figure 8.13c shows that unconservative assessments of $M^{DR}$ were only obtained for the tests F and G. It is worth to recall that the Goh and Mair (2011a) predictions were based on assuming a mean design envelope between the proposed upper and lower bound design envelopes according to Goh and Mair (2011a). A potential refinement of the predictions might be achieved by applying the upper bound design envelope which would result in more conservative estimates. Another improvement could be to account for the trough widening due to the soil–structure interaction when estimating the partitioned building lengths (i.e. $L_{sog}$, $L_{hog}$) in order to derive more adequate relative stiffness expressions.

Overall, the data in Figure 8.13 demonstrates that current methods to estimate $M^{DR}$ result in significant variation. Applying soil stiffness assumptions recommended by Potts and Addenbrooke (1997) resulted often in conservative predictions while accounting for soil stiffness degradation (lower bound design values) caused more unconservative estimates. Although further refinements are required, the Goh and Mair (2011a) prediction method tends to give
8.3 Performance of the relative stiffness methods

(a) Potts and Addenbrooke (1997) RSM predictions.

(b) Franzius et al. (2006) RSM predictions.

(c) Goh and Mair (2011a) RSM predictions.

(d) Performance versus $V_{lt}$.

**Fig. 8.13** Performance of relative stiffness methods to assess $M^{DR}$ at $V_{lt} = 2.0\%$ for (a) Potts and Addenbrooke (1997), (b) Franzius et al. (2006) and (c) Goh and Mair (2011a). (d) Mean values of measured to predicted $M^{DR}$ values for all tests and upper and lower bound relative stiffness expressions. The standard deviation is abbreviated with SD.
reasonable predictions when applying the upper bound soil stiffness design value. In particular, for structures located primarily in the greenfield sagging or hogging zones (tests A, B and D) conservative predictions were observed. Long structures that were spanning the greenfield inflection point and with substantial opening percentage (tests F and G) may respond more flexibly than the RSM formulation of Goh and Mair (2011a) predicts.

8.3.3.2 Modification factors for horizontal building distortions

Case studies have shown that the transfer of horizontal ground movements to buildings on continuous footings is significantly restrained due to the axial building stiffness (e.g. Mair, 2013). For the centrifuge tests discussed herein, similar trends were observed (Section 7.4.3) although the horizontal building strains were observed at neutral axis level of the buildings and a rough soil–structure interface was modelled. Only for test G and at \( V_{lt} > 1.5 \) measured modification factors for the horizontal building strains, \( M_{\varepsilon_{ht}} \), increasing 0.5 were obtained. A comparison between measured \( M_{\varepsilon_{ht}} \) values, and predicted \( M_{\varepsilon_{ht}} \) values according the three RSM formulations defined by Potts and Addenbrooke (1997), Franzius et al. (2006) and Goh and Mair (2011c) is provided in the following.

Figure 8.14 plots \( M_{\varepsilon_{ht}} \) predictions and measures along \( V_{lt} \) for all cases. Focus is placed on horizontal tensile strains, which are directly related to potential building damage. However, for the tests A and C the structure experienced only horizontal compressive strains which is indicated with 'sag' in Figure 8.14.

For all cases, the RSM definitions predicted horizontal tensile strains that are significantly lower than the greenfield equivalents (i.e. \( M_{\varepsilon_{ht}} < 1 \)). It is also apparent from Figure 8.14 that the variation between the different RSM is significantly lower than for \( M_{DR} \). Clear trends between the different RSMs were not obtained. As was expected, the lower bound design values for the soil stiffness considerably reduced the \( M_{\varepsilon_{ht}} \) estimates compared to the upper bound equivalents.

While the \( M_{\varepsilon_{ht}} \) predictions based on the recommended upper bound soil stiffness values provided reasonable estimates for most of the cases, the predictions based on the lower bound soil stiffness values are generally unconservative. For the tests F and G, unconservative predictions were also derived when applying the upper bound soil stiffness. This is particularly striking for the isolated façade test (test G) for which the RSM formulations predicted \( M_{\varepsilon_{ht}} \) values lower than 0.15. These low estimates of \( M_{\varepsilon_{ht}} \) can be explained by the overestimation of the axial building stiffness of test G when deriving \( EA \) per metre run. By contrast, for test F with a 3D building layout \( M_{\varepsilon_{ht}} \) estimates greater than 0.2 were derived using the RSM formulations of Franzius et al. (2006) and Goh and Mair (2011c). This comparison shows that the
8.3 Performance of the relative stiffness methods
current procedure of estimating the axial building stiffness is not representative for buildings with different building layout. Guidance to overcome this issue is addressed in Chapter 9.

Figure 8.15 quantifies the performance of the different RSM predictions for $M_{Ed}$. For structures that performed in compression (tests A and C), $M_{Ed}$ (measured) = 0 was used. Due to the small values for the predicted $M_{Ed}$, the ratio between the measured to predicted $M_{Ed}$ resulted in substantially greater values than was obtained for $M_{DR}$. This is particularly true for the lower bound prediction of test G, and explains the substantial increase of the statistical parameters (i.e. mean and standard deviation) compared to the $M_{DR}$ equivalents. Test G was neglected because of the overestimation of $EA$ when plotting the measured over predicted mean $M_{Ed}$ values versus $V_{l,t}$, as shown in Figure 8.15d.
8.3 Performance of the relative stiffness methods

Fig. 8.15 Performance of relative stiffness methods to assess $M_{DR}$ at $V_{lt} = 2.0\%$ for (a) Potts and Addenbrooke (1997), (b) Franzius et al. (2006) and (c) Goh and Mair (2011a). (d) Mean values of measured to predicted $M_{DR}$ values for tests A-F and upper and lower bound relative stiffness expressions are plotted versus $V_{lt}$. The standard deviation is abbreviated with SD.
Evaluation of current damage assessment methods

From Figure 8.15d it is evident that the lower bound design values significantly underestimate $M_{\text{ht}}\varepsilon$ while the upper bound predictions of all RSM formulations resulted in reasonable estimates. In particular, applying the design charts proposed by Franzius et al. (2006) and Goh and Mair (2011c) tend to provide conservative estimates (towards the right of the line of unity) when applying the upper bound values of the soil stiffness. Applying the Goh and Mair (2011c) design envelope and the upper bound relative axial stiffness formulation according to Franzius et al. (2006) resulted in predictions that were in fair agreement with the measured values when neglecting test G. Furthermore, from Figure 8.15d it is apparent that the performance of the different RSMs is rather independent from $V_{l,t}$.

8.3.4 Relative stiffness method with focus on shear deflections

The relative stiffness formulations of Son and Cording (2005) focus on the angular distortion and the equivalent building stiffness in shear which is a notably different approach than the relative stiffness methods discussed in Section 8.3.2.1. Son and Cording (2005) also recommended to account for the reduction in horizontal building strains by applying the framework proposed by Boscardin and Cording (1989). Within this section, the obtained experimental results are related to predictions according to Son and Cording (2005) and Boscardin and Cording (1989).

8.3.4.1 Predictions of the angular distortion

Son and Cording (2005) provide an estimate of the angular distortion that accounts for the interaction between the soil and the structure. Moreover, their work also accounts for a building stiffness decay due to cracking. Therefore, Son and Cording (2005) provided a design chart which specified design lines for different ratios between the change of the ground slope, $\Delta GS$, and the cracking strain of the structure, $\varepsilon_{t,\text{crack}}$ ($\varepsilon_{t}$ in the original work of Son and Cording (2005)). To consider this approach when applying this assessment method to the centrifuge model tests, $\Delta GS/\varepsilon_{t,\text{crack}}$ was estimated using the greenfield data of Farrell (2010) and the average cracking strain of the 3D printed material ($\varepsilon_{\text{ult}}$ in Table 4.1). For the test with 3D building layout (tests A-F) the design lines considering a downdrag force of an adjacent wall (i.e. end walls) are adopted while the design envelopes for no-downdrag force were used for test F. The upper and lower bound design values for the soil stiffness were adopted to estimate the relative stiffness according to Son and Cording (2005), as illustrated in Figure 8.10. Consequently, the predictions of $\beta$ according to Son and Cording (2005) depend on both the decay of $E_s$ and the ratio between $\Delta GS/\varepsilon_{\text{ult}}$. 

256
Figure 8.16 relates the experimentally obtained $\beta$ values to the Son and Cording (2005) predictions. For each test, only the maximum $\beta$ values (i.e. in bay 1, 2, 3 or 4) are presented. The predictions resulted in the greatest $\beta$ values for the building bay closest to the tunnel centreline while the location (i.e. bay) of the maximum measured $\beta$ varied between the different tests. As is evident from Figure 8.16 absolute values of $\beta$ are presented because the state of strain concept is not distinguishing between positive and negative values of $\beta$.

For test A, the lower bound estimates provide a significantly greater estimate of $\beta$ compared to the upper bound. This result can be explained by the significant change of the ground slope, $\Delta GS$, between the centre and side building bays which caused a substantial increase of $\Delta GS/\varepsilon_{ult}$ as $V_{l,t}$ develops. Consequently, the adopted design lines according to Son and Cording (2005) increased from the 'elastic (no crack)' line to nearly the design line ’2 - DF’ for the range of $V_{l,t}$ discussed. By contrast, the upper and lower bound estimates of $\beta$ were nearly identical for the remaining cases B to G. This observation implies that the the change of $E_s$ approximately compensated the variation of $\Delta GS/\varepsilon_{ult}$.

The predictions of $\beta$ notably overestimated the measured $\beta$ values of the tests A and B while the predictions were in good agreement for test C. An increase of the window openings from 20% for test B to 40% for test D caused a minor increase of the predicted $\beta$ values which provided a conservative estimate of the measured $\beta$ value. Also for test E the prediction slightly overestimated $\beta$. On the other hand, the predictions significantly underestimated the response of the tests F and G. For both tests, the measured $\beta$ values were nearly twice the value of the estimate as cracking initiated (at $V_{l,t} = 2.6\%$ and $V_{l,t} = 1.9\%$ for tests F and G, respectively). This implies that the Son and Cording (2005) estimates of $\beta$ for flexible buildings that span the greenfield inflection point are unconservative.

Figure 8.17 provides a further evaluation of the Son and Cording (2005) estimation of $\beta$. The mean values of the ratio of the measured to the predicted $\beta$ values were almost identical for the upper and lower bound estimates at $V_{l,t} = 2.0\%$ as shown in Figure 8.17a. For $V_{l,t}$ values between 0.5% and 4.0% similar trends were observed (Figure 8.17b). The mean values indicate that the Son and Cording (2005) prediction often result in a conservative estimate of $\beta$ which is more pronounced when neglecting test G, as shown in Figure 8.17b.

### 8.3.4.2 Predictions of the horizontal strain

For the prediction of modification factors for the horizontal building strains, the upper and lower bound axial building stiffness values according to Boscardin and Cording (1989) and illustrated in Figure 8.10 are adopted. Figure 8.18 compares the obtained upper and lower bound estimates of $\varepsilon_h$ to the measured $\varepsilon_h$ values. In the same way as for $\beta$, the greatest values
Evaluation of current damage assessment methods

Fig. 8.16 Measured versus predicted angular distortion according to Son and Cording (2005) along tunnel volume loss.
8.3 Performance of the relative stiffness methods

![Graph](image)

(a) Angular distortion at $V_{lf} = 2.0\%$.

(b) Performance versus $V_{lf}$ for angular distortion.

Fig. 8.17 Performance of Son and Cording (2005) to assess (a) $\beta$. (b) Mean values of measured to predicted $\beta$ for upper and lower bound relative stiffness expressions versus $V_{lf}$. The standard deviation is abbreviated with SD.

of $\varepsilon_h$ (positive or negative) are presented. For the tests A and C, compressive strains are plotted because in these tests the building models experienced solely compressive horizontal strains at top building level. The remaining tests experienced tension. For the predictions the greatest tensile strains are presented.

The upper bound estimates of $\varepsilon_h$ were notably greater than the lower bound equivalents (Figure 8.18). For the tests A to E, the predictions based on the upper bound design values resulted in conservative estimates of $\varepsilon_h$ while unconservative estimates were obtained for the tests F and G. Specifically, after cracking occurred the measured $\varepsilon_h$ were significantly greater than the predictions. This finding suggests that the prediction of $\varepsilon_h$ according to Boscardin and Cording (1989) cannot capture strain localisation effects. A potential explanation for this observation is that the horizontal strains were measured at top building level whereas the predictions are average horizontal strains obtained by applying the horizontal displacement profile of the greenfield scenario at soil surface level.

Figure 8.18g shows $\varepsilon_h$ estimates close to zero. These unambiguous underestimate of the horizontal building response of test G can be related to the overestimation of the axial relative stiffness per metre run as is evident from Figure 8.10. It is obvious from this data that using a spacing factor, $S$, of one metre significantly overestimates the axial building stiffness. Moreover, it was found that small differences in the relative axial stiffness estimate of Boscardin

259
Evaluation of current damage assessment methods

Fig. 8.18 Measured versus predicted horizontal strain according to Boscardin and Cording (1989).
8.3 Performance of the relative stiffness methods

and Cording (1989) may result in considerably different predictions of $\varepsilon_h$ because the design line of Boscardin and Cording (1989) uses a linear scale for the relative axial stiffness. By contrast, the RSMs with focus on bending deflections employ a logarithmic scale for the axes representing relative stiffness expressions.

The obtained relative axial stiffness estimates for the tests A to F tend to provide a conservative estimate of the axial building behaviour. For structures that were predominantly located in the greenfield sagging or hogging region (tests A, B and D), the lower bound predictions matched the measured $\varepsilon_h$ values reasonably well. Also, for structures with 20% openings that were spanning the hogging/sagging transition zone (tests C and E) the upper bound estimates provided a conservative measure. An increase of the opening percentage (tests F and G) caused substantially greater horizontal building strains. This impact of the opening percentage on the axial building response is not taken into account in the relative axial stiffness definition of Boscardin and Cording (1989) that focuses solely on the stiffness of the footings.

The performance of the Boscardin and Cording (1989) approach to estimate the horizontal building response is further explored in Figure 8.19. For measured horizontal compressive strains, $\varepsilon_h = 0$ is assumed. A significant difference between the lower and upper bound predictions is apparent in Figure 8.19a. While the lower bound values were in reasonable agreement for most of the tests, the upper bound values often caused a substantial overprediction of $\varepsilon_h$. The axial performance of the building model in test G was significantly underpredicted, as discussed above, which explains the great mean and standard deviation (SD) values in Figure 8.19a. Figure 8.19b plots the measured over predicted mean $\varepsilon_h$ values versus $V_{l,t}$ with and without test G. A notable decrease of the measured over predicted mean $\varepsilon_h$ values is evident when neglecting test G, and the upper bound estimates result in conservative predictions throughout the range of $V_{l,t}$ presented.

### 8.3.5 Comparison of relative stiffness methods

In order to assess the performance of the relative stiffness methods with focus on bending or shear deflections, building strains are computed based on the previously derived modification factors and greenfield soil displacements. For the RSMs with focus on bending deflections, the LTSM equations are employed whereas for the RSM according to Son and Cording (2005) the SoS criteria is adopted. The building strains based on these predictions are then compared to the building strains, which were derived based on measured building deflections (Section 8.2).
Evaluation of current damage assessment methods

Figure 8.19 Performance of Son and Cording (2005) to assess (a) $\varepsilon_{ht}$. (b) Mean values of measured to predicted $\varepsilon_{ht}$ for upper and lower bound relative stiffness expressions are plotted versus $V_{l,t}$. The standard deviation is abbreviated with SD.

Figure 8.20 compares upper bound estimates of building tensile strains to building tensile strains based on measured building distortions. As was expected from the inconsistent predictions of $M^{DR}$ and $M^{Eh}$, the estimates of the different RSMs differ significantly. Overall, the Franzius et al. (2006) definitions tend to give the most conservative estimates while the Potts and Addenbrooke (1997) predictions were often the least conservative ones (except for test A). The assessments according to Goh and Mair (2011a,c) and Son and Cording (2005) were frequently contained between the predictions of Franzius et al. (2006) and Potts and Addenbrooke (1997).

The RSM according to Son and Cording (2005) resulted in predictions that were often in fair agreement with the Goh and Mair (2011a,c) method. However, for test F, the estimate of $\varepsilon_0$ according to Son and Cording (2005) provided a notably lower estimate than the Goh and Mair (2011a,c) prediction, and $\varepsilon_p$ is substantially underestimated after cracking occurred (Figure 8.20f). Although, the RSM defined by Son and Cording (2005) accounts for the onset of building damage and provides different tensile curves for different magnitude of cracking, the method could not predict the significant increase in $\varepsilon_p$ when building damage initiated (Figures 8.20f and 8.20g).

Figure 8.21 compares the lower bound predictions of building strains to the building tensile strains derived from measured building deflections. Similar trends as observed for the
8.3 Performance of the relative stiffness methods

Fig. 8.20 Upper bound building tensile strains based on relative stiffness predictions and experimentally obtained building distortions.
upper bound estimates can be seen from Figure 8.21. Due to the more rigid estimate of the relative building stiffness, the Potts and Addenbrooke (1997) and Goh and Mair (2011a,c) assessments often underpredicted $\varepsilon_t$. This is particularly obvious for structures that span the greenfield inflection point (tests C, E, F and G). The lower bound estimates according to Son and Cording (2005) provided conservative predictions for structures with $L/H = 2.2$ (tests A-D). For structures with $L/H = 2.9$ (tests E-G), the lower bound estimates of Son and Cording (2005) led to unconservative predictions.

To further assess the performance of the RSMs, the predicted building strains $\varepsilon_{t,RSM}$ are related to building strains based on measured building deformations $\varepsilon_{t,Str}$. Figure 8.22 presents this strain ratio for all RSMs and conducted tests. Values smaller than one indicate conservative predictions while values greater than one are related unconservative predictions. Emphasis is placed on the upper bound estimates because the associated soil stiffness values are based on the widely accepted design recommendation according to Potts and Addenbrooke (1997), though often causing conservative predictions, as was previously observed.

In particular for test A, overly conservative estimates were produced by all RSMs (Figure 8.22a). Likewise, for the structures placed in the hogging region of the settlement trough (tests B and D) conservative predictions can be seen from the Figures 8.22b and 8.22d. Specifically, the estimates for test D with 40% of façade openings were more conservative than for test B with 20% of façade openings. This observation implies that the influence of openings was overestimated for the structure placed in the hogging zone (test D). This finding could not be confirmed for longer structures that were located closer to the tunnel (compare test E with test F in Figure 8.22).

It is apparent from Figure 8.22 that for structures, which were spanning the greenfield inflection point (tests C, E, F and G), unconservative estimates were observed when applying the RSM proposed by Potts and Addenbrooke (1997). The Goh and Mair (2011a,c) estimates for this specific building-to-tunnel position were conservative and in good agreement with $\varepsilon_{t,Str}$ for the tests C and E-F while the Franzius et al. (2006) method led to considerable overprediction. All RSMs underpredicted the tensile strains of test G. As discussed before, this can be related to the overestimation of the plane-strain building stiffness of test G.

The RSM according to Son and Cording (2005) predicted $\varepsilon_p$ values that were often significantly conservative. For the short structures (tests A, B, C and D), the estimates were similar or even lower than the assessments of Franzius et al. (2006), as shown in Figure 8.20. For the long structure with 20% openings (test E) an identical trend was observed. From Figure 8.22f it is evident that the Son and Cording (2005) method provided conservative estimates before cracking initiated (i.e. $V_{l,t} < 2.6\%$). As $V_{l,t}$ developed the predictions became notably
8.3 Performance of the relative stiffness methods

Fig. 8.21 Lower bound building tensile strains based on relative stiffness predictions and experimentally obtained building distortions.
Evaluation of current damage assessment methods

Fig. 8.22 Ratio between building strains based on measured building distortions ($\varepsilon_{t,Str}$) and upper bound predictions ($\varepsilon_{t,RSM}$).
8.3 Performance of the relative stiffness methods

Fig. 8.23 Performance of current relative stiffness methods to predict building strains caused by tunnel excavation at $V_{lt} = 2.0\%$. Error bars show the standard deviation. (P & A Potts and Addenbrooke (1997), Fra. Franzius et al. (2006); G & M Goh and Mair (2011a,c); S & C Boscardin and Cording (1989); Son and Cording (2005))

unconservative. This observation can be attribute to the ability of the SoS criteria to account for strain localisation, which had substantial impact on the measured $\varepsilon_p$ as is, for example, evident in Figure 8.21f. By contrast, the estimates according to Son and Cording (2005) were not able to replicate strain localisation effects, which was also observed for the other available RSMs.

Figure 8.23 plots the mean and standard deviations of the the ratio between $\varepsilon_{t,Str}$ and $\varepsilon_{t,RSM}$ for the available RSMs at $V_{lt} = 2.0\%$. In addition, the statistical parameters were also obtained for the data of the tests A-F. This was conducted to account for the uncertainty in the predictions of test G due to the overestimate of the relative stiffness for the isolated façade configuration. Table 8.6 summarises this data and additionally provides the corresponding data at 1.0% and 4.0% of $V_{lt}$. For all available RSMs the mean and standard deviation of the ratio between $\varepsilon_{t,Str}$ and $\varepsilon_{t,RSM}$ increased with $V_{lt}$ (Table 8.6).
### Evaluation of current damage assessment methods

Table 8.6: Assessment of the performance of the relative stiffness methods. Mean and standard deviations (SD) in brackets were obtained for tests A-F. (RSM relative stiffness method, P&A Potts and Addenbrooke (1997), Fra. Franzius et al. (2006); G&M Goh and Mair (2011a,c); S&C Boscardin and Cording (1989); Son and Cording (2005); ub upper bound; lb lower bound)

<table>
<thead>
<tr>
<th>RSM</th>
<th>$V_{1,t}$ (%)</th>
<th>$\varepsilon_{t,Str}/\varepsilon_{t,RSM}$ (%)</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
<th>G</th>
<th>Mean</th>
<th>SD</th>
</tr>
</thead>
<tbody>
<tr>
<td>P&amp;A (ub)</td>
<td>1.0</td>
<td>0.36</td>
<td>0.82</td>
<td>1.33</td>
<td>0.73</td>
<td>0.82</td>
<td>1.30</td>
<td>2.08</td>
<td>1.06</td>
<td>(0.89)</td>
<td>0.56</td>
</tr>
<tr>
<td></td>
<td>2.0</td>
<td>0.21</td>
<td>0.79</td>
<td>1.14</td>
<td>0.65</td>
<td>1.03</td>
<td>1.30</td>
<td>2.86</td>
<td>1.14</td>
<td>(0.85)</td>
<td>0.84</td>
</tr>
<tr>
<td></td>
<td>4.0</td>
<td>0.17</td>
<td>0.72</td>
<td>1.14</td>
<td>0.69</td>
<td>1.12</td>
<td>1.58</td>
<td>4.16</td>
<td>1.37</td>
<td>(0.90)</td>
<td>1.31</td>
</tr>
<tr>
<td>Fra. (ub)</td>
<td>1.0</td>
<td>0.16</td>
<td>0.39</td>
<td>0.50</td>
<td>0.49</td>
<td>0.44</td>
<td>0.53</td>
<td>0.88</td>
<td>0.49</td>
<td>(0.42)</td>
<td>0.21</td>
</tr>
<tr>
<td></td>
<td>2.0</td>
<td>0.10</td>
<td>0.38</td>
<td>0.42</td>
<td>0.43</td>
<td>0.55</td>
<td>0.55</td>
<td>1.20</td>
<td>0.52</td>
<td>(0.41)</td>
<td>0.34</td>
</tr>
<tr>
<td></td>
<td>4.0</td>
<td>0.08</td>
<td>0.35</td>
<td>0.42</td>
<td>0.46</td>
<td>0.60</td>
<td>0.68</td>
<td>1.73</td>
<td>0.62</td>
<td>(0.43)</td>
<td>0.53</td>
</tr>
<tr>
<td>G&amp;M (ub)</td>
<td>1.0</td>
<td>0.41</td>
<td>0.64</td>
<td>0.93</td>
<td>0.67</td>
<td>0.69</td>
<td>0.81</td>
<td>1.59</td>
<td>0.82</td>
<td>(0.69)</td>
<td>0.38</td>
</tr>
<tr>
<td></td>
<td>2.0</td>
<td>0.24</td>
<td>0.62</td>
<td>0.80</td>
<td>0.59</td>
<td>0.87</td>
<td>0.84</td>
<td>2.19</td>
<td>0.88</td>
<td>(0.66)</td>
<td>0.62</td>
</tr>
<tr>
<td></td>
<td>4.0</td>
<td>0.19</td>
<td>0.57</td>
<td>0.80</td>
<td>0.63</td>
<td>0.94</td>
<td>1.04</td>
<td>3.19</td>
<td>1.05</td>
<td>(0.70)</td>
<td>0.98</td>
</tr>
<tr>
<td>S&amp;C (ub)</td>
<td>1.0</td>
<td>0.25</td>
<td>0.23</td>
<td>0.14</td>
<td>0.33</td>
<td>0.38</td>
<td>0.76</td>
<td>2.66</td>
<td>0.68</td>
<td>(0.35)</td>
<td>0.89</td>
</tr>
<tr>
<td></td>
<td>2.0</td>
<td>0.18</td>
<td>0.17</td>
<td>0.26</td>
<td>0.24</td>
<td>0.56</td>
<td>0.75</td>
<td>3.91</td>
<td>0.87</td>
<td>(0.36)</td>
<td>1.36</td>
</tr>
<tr>
<td></td>
<td>4.0</td>
<td>0.10</td>
<td>0.12</td>
<td>0.33</td>
<td>0.27</td>
<td>0.56</td>
<td>2.01</td>
<td>8.14</td>
<td>1.65</td>
<td>(0.56)</td>
<td>2.94</td>
</tr>
</tbody>
</table>

The data for the tests A-G (Figure 8.23 and Table 8.6) highlight that the Franzius et al. (2006) methodology is generally overly conservative while the Potts and Addenbrooke (1997) formulations likely underpredict building strains. A notable variation in the mean values of the tests A-G is apparent in Table 8.6 for the Son and Cording (2005) method, which can be related to deriving $\varepsilon_{t,Str}$ by means of the SoS criteria. However, before cracking substantially influences the results (i.e. at $V_{1,t} = 1.0\%$ and 2.0%) the mean values of the Son and Cording (2005) estimates are in reasonable agreement with the Goh and Mair (2011a,c) predictions, which often provided conservative estimates close to the observed strain values.

Figure 8.23 shows that the mean and standard deviation of the estimates of the RSMs substantially reduced when neglecting test G. For $V_{1,t} = 2.0\%$, the currently available relative stiffness methods tend to provide conservative estimates for all tests except the method proposed by Potts and Addenbrooke (1997). Table 8.6 reveals that the Potts and Addenbrooke (1997) methodology particularly underpredicted the response of structures that were spanning the greenfield inflection point (tests C, E and F).

The relative stiffness formulations of Goh and Mair (2011a,c) provided conservative estimates that were in fair agreement with the building strains based on observed building deflections. Only for test F and $V_{1,t} = 4.0\%$ a minor overprediction was observed (Table 8.6).
8.3 Performance of the relative stiffness methods

This finding demonstrates that the procedure defined by Goh and Mair (2011a,c) provided reasonable predictions.

The prediction accuracy index (PAI) was again adopted to further evaluate the accuracy of the predictions of the RSMs. Although, this approach might seem more inaccurate than the strain ratio discussed above, the classification of the building behaviour in damage categories is often applied by decision makers and thus is crucial to decide whether a more detailed assessment is required or not.

Table 8.7 lists the calculated PAI values of the available relative stiffness methodologies. Except for Potts and Addenbrooke (1997), conservative mean estimates (PAI > 1) of the degree of building damage were obtained for the entire range of $V_{t,t}$ presented. Identical to above, the Goh and Mair (2011a,c) framework tends to provide conservative predictions in fair agreement with the observed damage categories. Neglecting the results of test G confirmed the observed trends but notably increased the mean PAI values while reducing the standard deviation.

Table 8.7 Performance of relative stiffness methods to predict damage categories. Mean and standard deviations (SD) in brackets were obtained for tests A-F. (RSM relative stiffness method, P&A Potts and Addenbrooke (1997), Fra. Franzius et al. (2006); G&M Goh and Mair (2011a,c); S&C Boscardin and Cording (1989); Son and Cording (2005); PAI prediction accuracy index)

<table>
<thead>
<tr>
<th>RSM</th>
<th>$V_{t,t}$ (%)</th>
<th>A</th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
<th>G</th>
<th>Mean</th>
<th>SD</th>
</tr>
</thead>
<tbody>
<tr>
<td>P&amp;A (ub)</td>
<td>1.0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0.4 (0.5)</td>
<td>0.5 (0.5)</td>
</tr>
<tr>
<td></td>
<td>2.0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>-2</td>
<td>0.4 (0.8)</td>
<td>1.5 (1.2)</td>
</tr>
<tr>
<td></td>
<td>4.0</td>
<td>0</td>
<td>0</td>
<td>-1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>-2</td>
<td>-0.1 (0.2)</td>
<td>1.2 (1.0)</td>
</tr>
<tr>
<td>Fra. (ub)</td>
<td>1.0</td>
<td>0</td>
<td>0</td>
<td>2</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>1.6 (1.8)</td>
<td>1.0 (0.8)</td>
</tr>
<tr>
<td></td>
<td>2.0</td>
<td>0</td>
<td>0</td>
<td>2</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>1.6 (1.8)</td>
<td>1.3 (1.2)</td>
</tr>
<tr>
<td></td>
<td>4.0</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>-1</td>
<td>1.0 (1.3)</td>
<td>1.2 (0.8)</td>
</tr>
<tr>
<td>G&amp;M (ub)</td>
<td>1.0</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0.7 (0.8)</td>
<td>0.5 (0.4)</td>
</tr>
<tr>
<td></td>
<td>2.0</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>-2</td>
<td>0</td>
<td>0.6 (1.0)</td>
<td>1.5 (1.1)</td>
</tr>
<tr>
<td></td>
<td>4.0</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>-2</td>
<td>0</td>
<td>0.1 (0.5)</td>
<td>1.2 (0.8)</td>
</tr>
<tr>
<td>S&amp;C (ub)</td>
<td>1.0</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>-2</td>
<td>0.6 (1.0)</td>
<td>1.1 (0.0)</td>
</tr>
<tr>
<td></td>
<td>2.0</td>
<td>0</td>
<td>0</td>
<td>2</td>
<td>2</td>
<td>1</td>
<td>0</td>
<td>-2</td>
<td>1.0 (1.5)</td>
<td>1.5 (0.8)</td>
</tr>
<tr>
<td></td>
<td>4.0</td>
<td>0</td>
<td>0</td>
<td>2</td>
<td>2</td>
<td>1</td>
<td>-1</td>
<td>-2</td>
<td>1.1 (1.7)</td>
<td>2.0 (1.5)</td>
</tr>
</tbody>
</table>
8.4 Summary

Results of a series of centrifuge model tests, discussed in Chapters 6 and 7, were used to evaluate the accuracy of currently available methodologies to assess the potential risk of building damage due to tunnelling-induced ground movements. More specifically, within this chapter:

1. criteria to estimate building strains based on building distortions,
2. initial building damage assessments based on greenfield displacements and
3. relative stiffness methods that account for the interaction between a structure and the ground

were verified. The conclusions from each of these studies will be described separately.

8.4.1 Criteria to estimate building strains

A comparison between building tensile strains derived according to the limiting tensile strain method, LTSM, and the state of strain, SoS, concept, revealed that building tensile strains according to the LTSM were more conservative than the SoS equivalents. However, as building cracking occurred, the SoS concept resulted in notably greater building tensile strains which indicates that the SoS concept captures strain localisation and related building damage significantly better than the LTSM. This finding can be related to the focus on building units within the SoS criteria while the LTSM subdivides a structure based on its location with respect to the greenfield inflection point.

The LTSM relies on a realistic estimate of structural details including effects of window openings, the ratio between $E/G$ and building dimensions. While some of these information is not readily available to the design engineer, the SoS concept was developed for brick walls and brick-infilled frame structures and does not account for different material properties of the structure.

8.4.2 Greenfield predictions

As was expected, predictions based on greenfield soil displacements are often overly conservative. The building strains derived on basis of measured building distortions were on average 50% smaller than the LTSM greenfield predictions. For the SoS criteria, it was found that the measured building distortions resulted in building strains that were about 40% smaller than greenfield estimates of the SoS criteria. However, as $V_{ij}$ developed and substantial building damage was observed the average measured building strains were about 70% of the strains produced by the SoS greenfield assessment.
8.4 Summary

The accuracy of predicting damage categories was also quantified. The LTSM greenfield predictions resulted on average in an overestimation of between 1 to 2 damage categories. Initial estimates of the SoS concept based on greenfield ground movements overestimated the degree of building damage by approximately 2 categories.

8.4.3 Relative stiffness method predictions

Estimates of the different available relative stiffness methods (RSM), based on upper and lower bound soil stiffness estimates, were compared to the centrifuge test results. It was found that soil stiffness estimates according to Potts and Addenbrooke (1997) provided the required conservative relative stiffness assumptions.

The range of predictions between the different RSMs was found to be large. The Potts and Addenbrooke (1997) methodology resulted generally in the lowest $M^{DR}$ while the Franzius et al. (2006) led to considerable overestimations. Similar conclusions were reported by Giaardina et al. (2017). Employing a mean design envelope, which was the average of the upper and lower bound design envelopes defined by Goh and Mair (2011a), resulted in estimates of $M^{DR}$ that generally fell between the Potts and Addenbrooke (1997) and Franzius et al. (2006) predictions. The Goh and Mair (2011a) methodology resulted on average in conservative estimates of $M^{DR}$ that were in fair agreement with the measured $M^{DR}$ values.

A reasonable agreement between predicted and measured $M^{E_{ht}}$ was found for all available RSMs when applying the upper bound estimates and neglecting the predictions of test G. This was necessary because for the isolated façade (test G) the predictions of $M^{E_{ht}}$ were significantly unconservative, which can be attributed to substantial overprediction of the building stiffness. However, the available RSMs underestimated the $M^{E_{ht}}$ values of test F by approximately a factor of 2. This notable underestimate was observed at $V_{ult}$ values as low as 1.0% and implies that currently available RSMs might underestimate the imposed horizontal building strains of flexible structures located in the hogging/sagging transition region.

Although conservative estimates of $\beta$ were generally obtained when applying the RSM according to Son and Cording (2005), underpredictions were evident for a long flexible structure spanning the hogging/sagging transition zone (test F). The predictions of the horizontal strain according to Boscardin and Cording (1989) were conservative as long as building damage was not observed.

Building tensile strains and related damage categories were computed based on the predicted building distortions and compared to building strains derived from centrifuge test results. This evaluation reflected the trends observed for the modification factors (discussed...
Evaluation of current damage assessment methods

above). Overall, the Goh and Mair (2011a,c) framework performed better than the others. Nevertheless, the substantial variation between measured and observed building strains and related damage categories indicates the necessity of refinement, which is addressed in the following chapter.
Chapter 9

Recommendations for practical implementation

Accurately predicting the ground and building response to tunnelling operations is fundamental to successfully deliver an urban tunnelling project. However, current methods tend to provide inconsistent predictions, as was extensively shown in the previous chapter. This could have major implications. For instance, underestimation of structural distortions might place existing structures at risk or an overestimation may result in unnecessary costly mitigation measures. This chapter provides recommendations to improve predictions of building response to tunnelling-induced ground displacements by accounting for the influence of building characteristics including the building layout, the building-to-tunnel position, the façade opening percentage and the building length. As the previous chapter demonstrated, the relative stiffness methodology proposed by Goh and Mair (2011a,c) performed reasonably well. For this reason, the focus is placed on this most recent framework to assess the potential building damage caused by tunnelling operations.

9.1 Accounting for the building layout

Carrying out an extensive evaluation of currently available procedures to account for the interaction between the ground and a building demonstrated that an accurate estimate of the overall building stiffness is crucial to realistically predict structural distortions. However, as was pointed out by Giardina et al. (2017) an accurate determination of the overall building stiffness is a difficult task with various uncertainties involved.

Widely accepted methodologies (e.g. Melis and Rodriguez Ortiz, 2001) translate the stiffness of different structural typologies into an overall building stiffness per metre run, which
Recommendations for practical implementation

provides an essential input to the available relative stiffness formulations. This simplification to a 2D plane-strain problem requires a realistic procedure that accounts for the out-of-plane contribution of individual structural members. Moreover, the relative stiffness formulations relate the equivalent building stiffness to the soil stiffness which necessitates an assessment of the amount of soil participating in this soil–structure interaction (Giardina et al., 2017).

The foundation type can influence the amount of soil involved. For buildings on raft foundations and previously used simple experimental models (e.g. continuous plate models) the entire footprint of the equivalent structure is in contact with the soil, while for buildings placed on strip footings this contact area is significantly reduced.

Recent back-calculations of building response to tunnelling-induced ground movements proposed a spacing factor, $s$, as the ratio between the total width, $b$, of a structural member (e.g. walls, foundations, etc.) perpendicular to bending to the overall building span, $B$ (Farrell et al., 2011; Goh and Mair, 2011a). This approach was also adopted by Giardina et al. (2016) to obtain plane-strain stiffness estimates of the 3D printed building models with 3D building layout (tests A-F) in order to numerically investigate the impact of 2D modelling assumptions. So far, throughout this dissertation, this spacing factor methodology was followed to obtain the plane-strain equivalent building stiffness of the small-scale building models.

However, when applied to an isolated building façade (test G) this spacing factor approach resulted in a significant overestimation of the building stiffness because the building span $B$ equals the foundation width $b_f$, which results in a spacing factor of unity. This considerable overestimate is visualised in Figure 9.1, which shows the estimates of the relative stiffness according to Goh and Mair (2011a) and the observed and predicted modification factors for the deflection ratio in the Goh and Mair (2011a) chart. Both tests show rather similar modification factors in hogging, as can be seen from the observed trends of $M^{{\text{DR}}_{\text{hog}}}$ in Figure 9.1a, while the substantial difference between the calculated $\rho_{\text{hog}}$ can be attributed to the overprediction of the building stiffness for test G. Figure 9.1b also shows the difference in the relative sagging stiffness, $\rho_{\text{sag}}$, between the tests F and G, in this case associated to a different building response ($M^{{\text{DR}}_{\text{sag}}}$) in sagging. This is potentially caused by a downdrag force of the end wall, which is parallel and directly above the tunnel, and likely reduced the sagging response of the structure in test F (Section 7.3.3).

To provide a more realistic assumption of the equivalent building stiffness for structures founded on strip footings a simplified method is proposed to estimate a reasonable spacing factor. First, the contours of vertical stress changes due to the load of the building were estimated based on linear, homogeneous, isotropic elastic theory (Equation 8.12). Figure 9.2 illustrates the derived vertical stress contours for both the front and rear strip. It can be seen
9.1 Accounting for the building layout

Fig. 9.1 Comparison between test F and G in the Goh and Mair (2011a) design chart for the estimation of modification factors based on relative building stiffness.
Fig. 9.2 Contours of vertical stress changes beneath strip footings using linear, homogeneous, isotropic elastic theory (Boussinesq, 1885).

that based on the assumption of simple elasticity the interaction between the two strips is rather small, and only the '0.05q' vertical stress bulbs nearly overlap. This potential interaction at a rather small vertical stress change (i.e. 5 kPa for \( q = 100 \text{ kPa} \)) may occur at a depth greater than half of the tunnel cover \( C \). This may suggests that the front and rear façade are too far apart to significantly interact; this conclusion may be supported by the nearly equal hogging response of both structures. However, the data is not conclusive because the rigidly connected end and intermediate walls might have caused 3D effects (as was observed for the sagging response, particularly since the end wall is located directly above the tunnel).

Yiu et al. (2017) reported minor differences between results of an isolated façade configuration and a complete building with rear, end and intermediate walls. In their computational model, the building dimensions were similar to the 3D printed building models with a building span of 10 m and 1 m wide strip footings. Although their modelled soil conditions were considerably different from the sand used in the centrifuge model test, their results provide confidence that the hypothesis of negligible interaction between the front and rear façades is appropriate.

Figure 9.3a details the vertical stress bulbs beneath a single strip. To estimate a spacing factor it is proposed to define the participating soil width as \( 2b_f \) either side of a strip. This assumption contains the vertical stress changes greater than 10% and results in an overall
participating soil width $B_s$ of $5b_f$ per strip footing, as can be seen from Figure 9.3a. More specifically, the estimate of $B_s$ can be written as

$$B_s = \begin{cases} 
5n_f b_f & \text{for } S \geq 5b_f \\
(n_f - 1)S + 5b_f & \text{for } S < 5b_f 
\end{cases} \tag{9.1}$$

where $n_f$ is the number of strip footings and $S$ is the distance between two strips, as depicted in Figure 9.3b. From Equation 9.1 it can be seen that the minimum participating soil width is $5b_f$. The spacing factor for an individual structural typology (e.g. wall) is defined as

$$s = \frac{\sum_{j=1}^{n} b_j}{B_s} \tag{9.2}$$

where $b$ is the width of the structural typology and $n$ the total number of occurrences of the specific structural member. For most practical applications, $B_s$ is considerably smaller than the tunnel depth, $z_t$, but for very shallow tunnels $B_s$ may also be a function of $z_t$. Moreover, this framework is only applicable for strip footings where $b_f$ is notably smaller than $B$ or a single strip; thus, this procedure is not valid for buildings on raft foundations or individual footings.

As a consequence of this procedure, the spacing factor of an isolated strip footing ($s_f$) is always 0.2 while the spacing factor for walls ($s_w$), which have a significant impact on the bending stiffness, depends on the wall thickness. For the conducted series of centrifuge model tests, this methodology results in $s_f = 0.2$ and $s_w = 0.075$ for a single strip and single façade wall, respectively. Note that this is in fair agreement with the used spacing factor of the buildings of the tests A-F with 3D building layout (i.e. 0.21 and 0.08, Section 4.5.1.2) and indicates that this suggested procedure provides reasonable estimates of the plane-strain stiffness of the buildings with 3D building configuration. Also for the case studies reported by Farrell et al. (2011), the proposed procedure results in fair agreement with the predictions made by Farrell (2010) and the likely stiffness range pointed out by Farrell (2010).

For the isolated façade case (test G), however, a significant overestimation of $EI$ was observed when using $B_s = b_f$. Interestingly, an identical assumption was reported by Mair and Taylor (2001) for two ancient walls affected by the construction of the Jubilee Line. However, both walls were shorter than 1.7 m and thus a very flexible response was predicted despite the use of $B_s = b_f$. Withers (2001) reported the observed response of wall 2 (0.8 m high) which
Recommendations for practical implementation

(a) Potential influence zone of a single strip.  
(b) Applicability of spacing factor approach.  
(For two strip footings $S \approx B$).

Fig. 9.3 Estimation of participating soil width.

was in reasonable agreement with the predictions. To the knowledge of the author, monitoring data for wall 1 has not been reported. Farrell (2010) also applied the assumption of $B_s = b_f$ for tests with strip footings fabricated from aluminium beams or masonry type walls (also reported in Farrell and Mair, 2011). However, these tests were rather limited by an unrealistic building height ($< 0.75$ m in prototype) and building weight (resultant surface pressure $< 15$ kPa).

Based on the outlined approach to estimate the participating soil width, a $B_s$ of $5b_f$ is calculated for test G. This results in a revised global $EI$ and $EA$ value of $2.69 \times 10^5$ kN/m and $5.37 \times 10^5$ kNm$^2$/m (Table 9.1). A comparison to field data, centrifuge tests and to the stiffness estimates the building models of tests A-F is presented in Figure 9.4. Clearly, the revised procedure results in $EA$ and $EI$ values for test G that are in better agreement with the other building models of this research.

Figure 9.5 plots the modification factors for the deflection ratio versus the revised relative stiffness values according to Goh and Mair (2011a) for test G in the Goh and Mair (2011a) design chart. Additionally, the relative stiffness values according to the previous estimate of $EI$, the results of test F and the corresponding predictions are presented. From Figure 9.5 it
Table 9.1 Revised $EA$ and $EI$ values of test G.

<table>
<thead>
<tr>
<th></th>
<th>Previous</th>
<th>Revised</th>
<th>Reduction</th>
</tr>
</thead>
<tbody>
<tr>
<td>$EA$ (kN/m)</td>
<td>$1.35 \cdot 10^6$</td>
<td>$2.69 \cdot 10^5$</td>
<td>80%</td>
</tr>
<tr>
<td>$EI$ (kNm$^2$/m)</td>
<td>$2.69 \cdot 10^6$</td>
<td>$5.37 \cdot 10^5$</td>
<td>80%</td>
</tr>
</tbody>
</table>

Fig. 9.4 Revised $EI$ and $EA$ values for test G and comparison to field data (a) Mair and Taylor (2001), 
(b) Dimmock and Mair (2008), (c) Farrell et al. (2011) and previous centrifuge tests (d) Taylor and Grant (1998), 

It can be seen that the data of test G moves horizontally to the left (from test G ‘previous’ to test G ‘revised’) due to the reduction in $EI$.

It is evident from Figure 9.5a that the revised data of test G is in fair agreement with the observed hogging response of test F. The revised upper and lower bound predictions would also result in reasonable estimates of $M_{DRhog}$. Specifically, when applying the upper bound design envelope (solid line in Figure 9.5a) combined with the lower bound (lb) prediction of the relative stiffness, which was obtained by using the soil stiffness at $V_{l,t} = 2.0\%$, the prediction would agree well with the observations at $V_{l,t} = 2.0\%$. The recommended upper bound predictions of $E_s$ would, as intended, result in a more conservative estimate when adopting the upper bound design envelope.

Figure 9.5b indicates that the sagging response of tests F and G notably differ. As was discussed before, this may be attributed to a downdrag force caused by the left end wall that significantly reduces the sagging deformation of test F. The revised predictions for test G are still underpredicting the observed building distortions. However, reasonable $M_{DRsag}$ values would be predicted when applying both the recommended upper bound relative stiffness and the upper bound design envelope (solid line in Figure 9.5b).
Recommendations for practical implementation

Fig. 9.5 Revised assessment of test G compared to test F in the Goh and Mair (2011a) design chart.
9.2 Accounting for the building-to-tunnel position

The post-crack behaviour is also highlighted in Figure 9.5. It can be seen that the Goh and Mair (2011a) design envelope captured the pre-cracking response in hogging (Figure 9.5a) while the post-cracking data was outside the design envelope. This implies that the predicted modification factors might be unconservative if cracking occurred or the building experienced damage before tunnelling. Further, these results indicate that a potential horizontal shift of the design envelope as a function of the ratio between building strains and strength, similar to the approach of Son and Cording (2005), would enable to better capture the post-cracking behaviour. Similar observations were also observed for the sagging response (Figure 9.5b).

The proposed approach of estimating the building stiffness based on the amount of participating soil provides a new framework to assess the plane-strain building stiffness of buildings on shallow strip footings. Within this section it was exemplified, by focusing on vertical building distortions, that this methodology performed well for the isolated façade test. In the future, this framework should be tested by collecting more detailed field data of building performance to tunnel excavation.

9.2 Accounting for the building-to-tunnel position

The results of the series of centrifuge model tests revealed that structures spanning the theoretical greenfield inflection point are more susceptible to building damage than identical structures located either in sagging or hogging. Previous researchers reported similar observations (Bilotta et al., 2017; Liu et al., 2001; Nghiem et al., 2014). Figure 9.6 highlights this finding by plotting the observed modification factors for the deflection ratio at $V_l \ell = 2.0\%$ in the Goh and Mair (2011a) design chart. From this graph it is evident that the measured $M^{DR}$ values of structures located in the sagging/hogging transition zone (indicated with ‘mix’ in Figure 9.6) are located beneath the upper design envelope (solid line) while all the data points of the buildings in either sagging or hogging fell below a mean design envelope (dotted line).

With respect to practical implementation, the data presented in Figure 9.6 suggest that the upper design envelope may be applicable for structures placed in the hogging/sagging transition zone. By contrast, the mean design envelope (dotted line in Figure 9.6) may be used for structures in the hogging or sagging region, though further data is required to further support this recommendation. The plotted $\rho_{sag}$ or $\rho_{hog}$ values are identical to the lower bound predictions (using $E_s = 45$ MPa) of the relative stiffness according to Goh and Mair (2011a). Hence, a vertical extension of each $\rho_{sag}$ or $\rho_{hog}$ value in Figure 9.6 would result in the lower bound prediction of $M^{DR}$. For all tests, this would provide conservative predictions. The
Recommendations for practical implementation

Fig. 9.6 Observed modification factors for the deflection ratio in the Goh and Mair (2011a) design chart. ($e$ eccentricity; $L$ building length)

widely applied upper bound design values of $\rho_{sag}$ or $\rho_{hog}$, which are based on $E_s = 175$ MPa, are smaller and thus would result in even more conservative estimates of $M^{DR}$.

In order to provide further evidence of this trend, case studies and data from previous centrifuge experiments were explored, as shown in Figure 9.7a. From the available data, it is obvious that most of the available cases dealt with structures located in the hogging/sagging transition zone. While for these scenarios the upper design envelope (solid line) provides a reliable upper bound, the data point of the Neptune West façade, which was located in the hogging region of the westbound (WB) tunnel drive, fell significantly above the mean design envelope (dotted line). This might be attributed to the overall rather flexible response of this case study. Further, it is worth pointing out that the interpretation of the available case studies is far from straightforward because field data inherently contains numerous uncertainties related to the ground, the buildings and the tunnelling process.

Figure 9.7b shows results from a series of centrifuge tests performed by Farrell (2010). It can be seen that the data aligns well with the previously identified trend; structures in the transition region fell below the upper design envelope (solid line) while the single test in solely hogging was even below the lower design envelope. Again, limited scenarios with buildings placed in either hogging or sagging were available to provide conclusive results.

The observed trend and associated design recommendation potentially results in a notable improvement of future predictions. However, building length effects are also essential (Section 9.4) and presented test data showed that further research is required to reduce the significant uncertainty. Specifically, there is an urgent need to collect more detailed building monitoring data in order to provide more conclusive evidence.
9.2 Accounting for the building-to-tunnel position

Fig. 9.7 Field and previous centrifuge test data in the Goh and Mair (2011a) design chart indicating building position effects. (For a single building the modification factor of the maximum deflection mode (i.e. hogging or sagging) is plotted. Buildings with \( \epsilon/L = 0.0 \) and \( L_{\text{hog}} > L_{\text{sag}} \) are considered to be located in the hogging/sagging transition region.)
Recommendations for practical implementation

9.3 Accounting for façade openings

The vital role of façade openings in this tunnel–soil–structure interaction problem was addressed by various researchers using field data (Dimmock and Mair, 2008; Goh and Mair, 2011a) and computation models (Giardina et al., 2015b; Melis and Rodriguez Ortiz, 2001; Pickhaver et al., 2010; Son and Cording, 2007). However, as was pointed out by Dimmock and Mair (2008), accounting for window and door openings when assessing the overall stiffness of a building is far from straightforward.

Figure 9.8 indicates that an increase of window openings from 20% to 40% resulted in a significant increase of the observed modification factors. The different building-to-tunnel positions and building lengths likely caused the variability between the two sets of scenarios (B-D and E-F in Figure 9.8).

To further investigate the impact of façade openings on the overall bending stiffness, $EI$ values were back-calculated by assuming that the measured $M^{DR_{hog}}$ values of the tests B, D, E and F are on the same design envelope of the Goh and Mair (2011a) design envelope. This results in back-calculated $\rho_{hog}$ values, which enables to solve for $EI$ in $\rho_{hog} = \frac{EI}{E_s L_{hog}}$. The decay of $E_s$ due to tunnelling-induced ground displacements was considered when deriving $EI$. Moreover, for the tests E and F the change of $L_{hog}$ with $V_{lt}$ was taken into account whereas for the tests B and D it was assumed that $L_{hog} = L$. This simplification was made because of the scatter when deriving the inflection points for buildings placed in the greenfield hogging region (Section 7.3.3).

The hypothetical $EI$ values were then normalised by the derived Young’s modulus of the corresponding 3D printed building model to account for differences in the 3D printed me-

Fig. 9.8 Increase of modification factors for the deflection ratio with window opening percentage, $O$, in the Goh and Mair (2011a) design chart.
9.3 Accounting for façade openings

Fig. 9.9 Normalised ratio between the global bending stiffness of tests with 40% façade openings and tests with 20% openings. \(^a\)Reduction factor applied to façade wall. \(^b\)For tests with 40% openings the contribution of the wall is neglected.

mechanical properties. Figure 9.9 relates the normalised \(EI\) of test D to test B and test F to test E alongside literature that is widely adopted to reduce \(EI\) due to openings. The literature values were obtained by considering the mechanical properties and openings of the corresponding building models.

Although there is notable scatter in the data related to the tests B and D (due to small building distortions and rather similar \(M_{D\text{hog}}\) values at \(V_{l,t} < 2.5\%\)), the experimental results show that an increase of the façade openings caused a decrease of the overall bending stiffness. This finding is more pronounced for the long structures that were placed in the sagging/hogging transition regions (tests E and F). For the tests B and D with building models of \(L = 200\) mm and located in the greenfield hogging zone, this trend becomes more pronounced as \(V_{l,t}\) develops. This observation might be attributed to the onset of building damage in test D due to occurrence of strain localisation around the extensive openings. By contrast, for test F microcracks may have propagated through the building at significantly lower \(V_{l,t}\) which is a potential explanation of the significant bending stiffness reduction between the tests E and F (Figure 9.9).

For the tests B and D, the Melis and Rodriguez Ortiz (2001) methodology provided a conservative estimate while both the Son and Cording (2007) and Pickhaver et al. (2010) methodologies potentially correspond to rather low \(V_{l,t}\) values. However, as can be seen from Figure 9.9, literature significantly underpredicted the bending stiffness reduction of test F. Only when completely neglecting the contribution of the wall to the global building stiffness (as suggested by Melis and Rodriguez Ortiz (2001) for walls with more than 40% openings), the predictions could nearly be forced to converge with the experimental observations. Note
Recommendations for practical implementation

that also Dimmock and Mair (2008) proposed to estimate the bending stiffness by considering the foundations only, though for structures in hogging position.

The data presented in Figure 9.9 highlights that further research is required to better estimate the effect of façade openings on the response of buildings subject to tunnelling-induced soil displacements. For short structures the reduction factors proposed by Melis and Rodriguez Ortiz (2001) may are applicable while for assets with extensive openings ($O \geq 40\%$), considerable aspect ratios and building-to-tunnel positions that span the greenfield inflection point the global bending stiffness may be estimated based on the contribution of the foundations only.

9.4 Accounting for the building length

Section 7.6 showed that the widely accepted approach of partitioning a structure at the hypothetical greenfield inflection point and assessing either part of the structure separately might underestimate building damage. When applying the Goh and Mair (2011a) methodology it is therefore crucial to reliably assess the length of a structure in hogging and sagging. However, as was shown throughout this work the interaction between a building and the soil caused a considerable widening of the tunnelling-induced settlement trough (Section 6.4.1.2). Based on the eccentricity and length of the building, the soil–structure interaction might reduce $L_{\text{hog}}$ while $L_{\text{sag}}$ increases (or vice versa), which affects the building response.

Farrell (2010) identified a relationship between the trough width and the lowest of $\rho_{\text{hog}}$ or $\rho_{\text{sag}}$ (based on the greenfield settlement trough) using field, experimental and numerical data. Figure 9.10 relates observed trough widening (Section 6.4.1.2) to the design guidance identified by Farrell (2010). It can be seen that the obtained experimental data reasonably align with the design line proposed by Farrell and Mair (2010). However, based on the centrifuge test data the design line potentially overpredicts the magnitude of trough widening. This might be attributed to both the semi-flexible building rigidity and the dense soil model causing a minor change of the trough width compared to the data considered by Farrell (2010).

Figure 9.11 indicates a minor impact of the modified building lengths on the $\rho_{\text{sag}}$ and $\rho_{\text{hog}}$ values. This implies that for the scenarios modelled in the centrifuge tests, the interaction mechanisms had a marginal effect on associated predictions. However, Franza et al. (2017) evaluated an elastic continuum-based analysis method (Franza and DeJong, 2017) through comparison with the experimental data discussed herein, and subsequently performed a wide ranging sensitivity study. A significant finding of this work was a substantial reduction in
9.4 Accounting for the building length

**Fig. 9.10** Design guidance to estimate the change of trough width due to soil–structure interaction mechanisms. Design line according to Farrell (2010). $\rho_{part}$ is the lower of $\rho_{hog}$ or $\rho_{sag}$.

**Fig. 9.11** Comparison between relative stiffness values based on greenfield building lengths (*i.e.* 60 mm) and modified building lengths due to soil–structure interaction (SSI).

scatter in the obtained dataset when considering the building length modification factors, as can be seen when comparing Figure 9.12a with Figure 9.12b.

This section stressed that soil–structure interaction mechanisms can affect the building lengths in hogging or sagging. Considering this alteration when estimating the relative building stiffness potentially results in better assessment of building response to urban tunnelling. The design line proposed by Farrell (2010) or the procedure proposed in Franza et al. (2017) is applicable for an initial assessment, though it is strongly recommended to further test this design recommendation with data collected in future tunnelling projects.
Recommendations for practical implementation

Current methods to assess the interaction between tunnelling-induced soil displacements provide limited guidance on how to account for building characteristics. Figure 9.13 summarises how the findings of this thesis are translated into recommendations to account for building characteristics when using the Goh and Mair (2011a) relative stiffness formulation to assess building response to tunnelling subsidence. The focus of this revised framework is placed on the relative building stiffness in bending and the associated design chart proposed by Goh and Mair (2011a) because, as shown in Chapter 8, this framework performed better than the other RSMs.

The first step of this refined procedure is to account for building characteristics when estimating the overall building stiffness in bending, $EI$. More specifically, the building layout is considered by applying a revised spacing factor approach (Equations 9.1 and 9.2) and guidance on the reduction of $EI$ due to façade openings (i.e., windows and doors) is given. Secondly, it is proposed to determine the building lengths experiencing hogging, $L_{\text{hog}}$, or sagging, $L_{\text{sag}}$, deformations by applying the design recommendation according to Farrell (2010), or alternatively using the procedure proposed in Franza et al. (2017). Thirdly, guidance to account for the building-to-tunnel position is given. For buildings that span the theoretical greenfield inflection point, it is proposed that the upper design envelope identified by Goh and

**Fig. 9.12** Results of an elastic continuum-based two-stage analysis method (Franza and DeJong, 2017) exemplifying the narrowing of the relation between $M^{DR}$ and $\rho$ when considering soil–structure interaction mechanism (adopted from Franza et al., 2017). $\rho$ relative stiffness using building lengths according to greenfield predictions; $\rho'$ relative stiffness considering soil–structure interaction when estimating the building length.

### 9.5 Design recommendations to account for building characteristics

Current methods to assess the interaction between tunnelling-induced soil displacements provide limited guidance on how to account for building characteristics. Figure 9.13 summarises how the findings of this thesis are translated into recommendations to account for building characteristics when using the Goh and Mair (2011a) relative stiffness formulation to assess building response to tunnelling subsidence. The focus of this revised framework is placed on the relative building stiffness in bending and the associated design chart proposed by Goh and Mair (2011a) because, as shown in Chapter 8, this framework performed better than the other RSMs.

The first step of this refined procedure is to account for building characteristics when estimating the overall building stiffness in bending, $EI$. More specifically, the building layout is considered by applying a revised spacing factor approach (Equations 9.1 and 9.2) and guidance on the reduction of $EI$ due to façade openings (i.e., windows and doors) is given. Secondly, it is proposed to determine the building lengths experiencing hogging, $L_{\text{hog}}$, or sagging, $L_{\text{sag}}$, deformations by applying the design recommendation according to Farrell (2010), or alternatively using the procedure proposed in Franza et al. (2017). Thirdly, guidance to account for the building-to-tunnel position is given. For buildings that span the theoretical greenfield inflection point, it is proposed that the upper design envelope identified by Goh and
9.5 Design recommendations to account for building characteristics

Refinements of the design approach according to Goh and Mair (2011a)

\[
\rho_{hag} = \frac{E_l}{E_sL_{hag}^2}, \quad \rho_{sag} = \frac{E_l}{E_sL_{sag}^2}
\]

1) Consider building characteristics when estimating \( EI \)

   a) applying proposed spacing factor approach to account for complex building layouts: Equations 9.1 and 9.2

   b) reduce \( EI \) to account for facade openings:
      - for short structures in sagging or hogging apply the Melis and Rodriguez Ortiz (2001) reduction factors
      - for long structures spanning the greenfield inflection point and openings \( \geq 40\% \) neglect stiffness contributions of walls (Dimmock and Mair, 2008)

2) Estimate \( L_{sag} = L_{hag} \) by considering soil-structure interaction effects

   a) determine the trough width parameter, \( K_{tr} \), by applying the design guidance according to Farrell (2010): Figure 9.10, or use the procedure proposed in Franza et al. (2017)

   b) partition the building into hogging and sagging regions by using \( K_{tr} \) to estimate the inflection point

3) Apply the design chart of Goh and Mair (2011a) and consider the building-to-tunnel position: Figure 9.6

   a) upper design line for buildings spanning the greenfield inflection point

   b) mean design line for buildings predominantly in the greenfield hogging or sagging region

Fig. 9.13 Recommendations to account for building characteristics when applying the Goh and Mair (2011a) framework to estimate building response to tunnelling-induced settlements.
Recommendations for practical implementation

Mair (2011a) should be applied, while a mean design envelope should be used for structures located predominantly in either the sagging or hogging region of the theoretical greenfield settlement profile. Based on these recommendations the modification factor for the deflection ratio, $M^{DR}$, can be estimated and subsequently building strains and related damage categories can be derived. The previous sections showed that these recommendations considerably reduce the uncertainty when applying the RSM according to Goh and Mair (2011a). However, it is strongly recommended to verify the proposed recommendations with further research.

9.6 Summary

Comparisons of the performance of buildings subjected to tunnelling-induced ground displacements (Chapter 7) and predictions (Chapter 8) have indicated a significant scatter between predictions and observations. Moreover, it was shown that specific methodologies have performed better than others (Chapter 8). To improve future predictions, this chapter provided recommendations to account for building characteristics (e.g. building layout, position of building relative to tunnel, façade openings and building length) when assessing potential building damage (Figure 9.13). More specifically, the following practical implementations were identified:

- Estimating the global stiffness of buildings with shallow strip foundations is a function of the amount of soil participating in the tunnel–soil–structure interaction. A revised procedure to derive the plane-strain building stiffness of buildings with complex building layout on strip footings was introduced by accounting for a potential interaction between strip footings based on their distance to each other. Applying this revised procedure resulted in a significant reduction of the overall building stiffness of an isolated façade, which compared favourably with the observed building response.

- The position of the building relative to the tunnel plays a key role in the structural response. For buildings spanning across the theoretical greenfield inflection point, the application of the upper design envelope identified by Goh and Mair (2011a) is recommended. Shorter structures located predominantly in either hogging or sagging were less susceptible to building damage and it is proposed to apply a mean design envelope, between the upper and lower design envelops defined by Goh and Mair (2011a). Further research is strongly suggested to further evaluate this recommendation.

- The experimental data revealed that buildings with extensive façade opening area are more susceptible to building damage. Specifically, the bending stiffness dramatically
reduced for long structures that were spanning across the greenfield inflection point as the opening percentage increased from 20% to 40%. Recommendations to account for a bending stiffness reduction due to façade openings overpredicted the global bending stiffness of these cases. Only when the contribution of the wall stiffness was completely neglected, the prediction was in reasonable agreement with the performance. For this reason, the overall bending stiffness of buildings with extensive openings (e.g. 40%) may be assessed by the contribution of the foundations only. On the contrary, the framework proposed by Melis and Rodriguez Ortiz (2001) provided a reasonable conservative estimate of the global bending stiffness reduction for shorter structures located in the hogging region.

- Centrifuge tests showed that soil–structure interaction mechanisms modify the length of the theoretical greenfield deformation modes (i.e. sagging and hogging). This change of the trough width, caused by a semi-flexible or rigid building, affects the building length in sagging and hogging and might influence predictions based on the relative stiffness formulations according to Goh and Mair (2011a). A design procedure proposed by Farrell (2010) was compared to the obtained experimental data, and it was concluded that this framework may be applied for an initial damage assessment, but more detailed monitoring data is required to provide conclusive recommendations.
Chapter 10

Conclusions

The primary aim of this dissertation, as was stated in Chapter 1, is to acquire experimental data to deepen the fundamental understanding of the effect of building characteristics on the tunnel–soil–structure interaction problem. The adopted methodology to achieve this objective included a series of centrifuge model tests of complex 3D printed surface structures subjected to a tunnel excavation in dense, dry sand. This chapter provides a summary of the main findings, the scientific contributions, the applicability of the obtained results and potential future research.

10.1 Main findings

In Chapter 1, the main objective of this thesis was subdivided into four specific aims. More precisely, it was envisioned to unlock new information on the influence of (1) the relative position of the building to the tunnel, (2) the façade opening area, (3) the building length transverse to the tunnel and (4) the building layout on the soil and the building response. The main findings are summarised below.

10.1.1 Influence of building characteristics on tunnelling subsidence

Results of the influence of building features on soil deformations due to tunnelling in sand have been addressed in Chapter 6, from which the following conclusions are drawn:

- **Building-to-tunnel position**: The position of the building relative to the tunnel has a significant impact on the soil displacements. Buildings placed so that one corner was located directly above the tunnel activated shear bands that propagated to the soil surface...
Conclusions

level. For this building-to-tunnel location, vertical soil settlements exceeding the theoretical greenfield soil displacements, significant rotation and local building embedment into the top soil levels was observed. On the contrary, buildings with zero eccentricity reduced the vertical soil settlements and the associated surface volume loss but embedment of the building corners was also apparent. This localised phenomena can be related to a loss of contact between the centre of the building and the soil surface, and subsequent building weight redistribution (Farrell, 2010). Buildings at different positions caused a widening of the soil settlement profile; the widest soil surface trough was observed for the structure with zero eccentricity and the widening effect reduced with eccentricity. Structures placed asymmetrically to the tunnel tended to have relatively uniform horizontal soil displacement profiles just beneath them, with an approximate magnitude equal to the average greenfield horizontal soil movements measured along the building extent. For this reason, horizontal soil displacements larger than in the greenfield case were measured as the offset to the tunnel increased while a structure symmetric to the tunnel reduced the horizontal soil displacements throughout the entire building extent.

• Façade openings: An increase of the façade opening area caused, as expected, an increase of the building’s flexibility which had a direct impact on the soil response. More specifically, the increase of the opening percentage notably reduced the deviation of the vertical soil displacements from the greenfield profiles, reduced the observed surface trough widening and marginally reduced the restraining effect of the soil–structure interaction on the horizontal soil displacements.

• Building length: Longer structures also caused an increase of the building’s flexibility and thus reduced the impact of a nearby building on the tunnelling-induced displacement profile. However, considerable soil contraction was observed beneath long buildings and the derived surface volume loss values were notably greater than those for the greenfield case. Furthermore, an increase of the building length tended to decrease the local embedment observed for a building placed so that the left building edge is coincident with the tunnel centreline. A variation of the building length had little influence on the constraint of the horizontal ground displacements just beneath the structure.

• Building layout: Different building geometries parallel to the tunnel showed a substantial influence on the tunnelling induced settlements. An isolated façade with the identical façade opening area and length than a structure with front, end, rear and partitioning walls had significantly less impact on the soil displacements. In particular the
vertical soil displacements in the sagging region matched the greenfield case. Furthermore, the horizontal soil displacements in the sagging region were less restrained by the isolated façade configuration compared to the building with 3D layout. This suggests that building geometry differences out of plane-strain have a substantial impact on the tunnel–soil–structure interaction.

In addition to the discussed specific effects of building features on the soil response to a tunnel excavation, further insight into the influence of a building on tunnelling-induced ground movements were observed, which are summarised as follows:

For all building configurations, building weight effects changed the volumetric behaviour of the soil above the tunnel and the associated tunnelling-induced displacement field. Similar building weight effects were also identified previously (Bilotta et al., 2017; Franzius et al., 2004; Giardina et al., 2015a). Consequently, the surface and subsurface volume losses were affected by nearby structures.

Field data from various tunnelling projects typically indicate that the transfer of tunnelling-induced horizontal soil displacements to the structure is significantly smaller than the greenfield equivalents. The experimental data presented in this dissertation demonstrated similar findings. The absolute and differential horizontal ground displacements just beneath the surface structures were significantly restrained by the buildings. However, the horizontal greenfield soil movements were approximately recovered at a soil depth between \( z/z_t = 0.13 \) and \( 0.26 \) which is in fair agreement with the findings of Standing (2001) and Farrell (2010). The different building variations had a minor influence on the restraining depth. Slippage between the foundation base and the soil surface was not observed.

### 10.1.2 Influence of building characteristics on structural behaviour

This section focuses on the effects of building characteristics on the structural behaviour (Chapter 7). The experimental results confirmed that the building stiffness plays a crucial role and greenfield assessments are generally overly conservative. However, it was also identified that building features play a crucial role, as is summarised below:

- **Building-to-tunnel position**: Structures spanning the greenfield hogging/sagging transition zone were more susceptible to building deflections than identical buildings located in either sagging or hogging. Similarly, considerable horizontal strains at top building level and angular distortions were measured for structures located in the hogging/sagging transition zone. Consequently, building damage (in terms of cracking) was
first observed for this building-to-tunnel position. Furthermore, the experimental data quantified that the widely adopted approach of partitioning a structure either side of the theoretical greenfield inflection point may result in unconservative predictions for the hogging deformation mode when neglecting the building part in the theoretical sagging region.

- **Façade openings:** An increase of the opening percentage resulted in a substantial increase of structural distortions (in terms of $DR$). For buildings of identical length and position, an increase of the façade area from 20% to 40% approximately doubled the associated modification factors for the deflection ratio. This suggests that extensive openings notably reduce the building stiffness in bending. The axial stiffness was also reduced when increasing the window opening area but the measured horizontal building strains were considerably lower than for the greenfield. Consequently, the onset of cracking was observed at lower $V_{l,t}$ values when increasing the façade opening area. Furthermore, the experiments confirmed that an increase of the opening percentage considerably increased shear deflections while bending deflections stayed rather constant.

- **Building length:** The experiments have shown that structural distortions are a function of the building length. Long buildings with extensive façade openings that were spanning the hogging-sagging transition region were most susceptible to damage. Thus, for the longer structures cracking was observed at rather low $V_{l,t}$. Increasing the building length made no significant difference to the measured shear deflections while bending deflections increased.

- **Building layout:** The relevance of accounting for the building layout was highlighted by the experimental data. An isolated façade configuration performed significantly more flexible than an equally positioned building model with the identical front and rear façade but end and intermediate walls. In particular, the sagging deformations were notably greater for the isolated façade test while the building layout had little effect on the hogging deformations. The rigidly connected end and partitioning walls influenced the overall building performance. Specifically, the end wall directly above the tunnel caused a downdrag force that reduced the sagging deformations and also resulted in notable embedment of the left building corner of the structure with complex building layout. For the isolated façade configuration, substantial embedment effects were observed after building cracking occurred (i.e at greater $V_{l,t}$). The more flexible sagging response of the isolated façade caused earlier building damage (in terms of $V_{l,t}$).

296
In addition to the findings on the influence of building characteristics on the structural behaviour, the brittle material behaviour of the 3D printed material enabled to examine building damage effects. For the entire series of centrifuge tests, visible cracking was first observed at the top building level which is consistent with the observed increase of horizontal strains with building height. The cracks then propagated vertically towards the soil surface. As mentioned above, cracking was first observed for buildings with greater façade opening area in which shear deflections govern. This suggests that the local bending, which is typical for façades with extensive openings, likely caused the crack initiation.

10.2 Scientific contributions

This research set out to provide the required missing experimental data to obtain further insight into the interaction between a representative surface structure and ground displacements caused by tunnelling. Because of the focus on physical modelling, the main scientific contributions can be related to both new insights into centrifuge modelling and deeper knowledge of the tunnel–soil–structure interaction. Furthermore, the obtained data provides essential benchmark data for available damage assessment methods and numerical modelling. More precisely, this dissertation makes several noteworthy contributions, which can be summarised as follows:

• **3D printed surface structures for centrifuge testing:** Powder-based 3D printed surface structures were developed that enabled to model complex structures affected by tunnelling-induced ground displacements. The mechanical properties of the 3D printed material were derived based on four-point bending tests. The material testing identified that the stiffness of the 3D printed material is comparable to historic masonry but the 3D printed material is considerably stronger than masonry. Overall building stiffness values in the range of previous field data were obtained by carefully balancing the building geometry. Experimental results demonstrated that these building models enabled the controlled investigation of structural details in centrifuge model testing.

• **Identification of spin-up phenomena:** The tunnel–soil–structure interaction during the acceleration phase of the centrifuge was examined throughout this research. It was identified that stress imbalances between the soil surrounding the tunnel and the tunnel resulted in soil displacements that may affect the structural model. Further, the building interacted with the tunnel and caused additional spin-up soil displacements. The
Conclusions

quantification of this spin-up phenomena informs future centrifuge modellers studying similar tunnel–soil–structure interaction mechanisms.

- **Benchmark data for validation:** The obtained experimental data provides the required experimental data on the effect of structural details on the building response to tunnelling subsidence. Therefore, this dissertation offers essential benchmark data to evaluate available damage assessment methods and computational modelling. After validation of numerical modelling studies, the tested model can be expanded to a wider range of scenarios.

- **Evaluation of available damage assessment procedures:** Chapter 8 tested the performance of current methods based on greenfield assumptions and more recent procedures that account for soil–structure interaction. As expected, it was shown that the greenfield estimates are generally overpredicting the potential degree of building damage. Considerable variations between the predictions of the relative stiffness procedures were shown. It was identified that the approach of Franzius et al. (2006) tends to provide the most conservative predictions while the Potts and Addenbrooke (1997) expressions might result in underprediction for structures spanning the greenfield sagging/hogging transition region. The predictions according to Son and Cording (2005) were characterised by a significant variation, and for the buildings with 3D layout overly conservative estimates, similar to Franzius et al. (2006) predictions, were obtained. The Goh and Mair (2011a,c) formulations appear to have performed better than the others.

- **Recommendations to account for building characteristics:** Guidance to account for the effect of structural details when assessing the building response to tunnelling-induced ground displacement is provided in Chapter 9. Firstly, a new framework to estimate the building stiffness of buildings on strip footings is proposed. Secondly, recommendations to account for the building-to-tunnel position when adopting the relative stiffness formulations according to Goh and Mair (2011a) are provided. Thirdly, guidance to account for the overall building stiffness reduction due to façade openings is provided by evaluating available literature. Finally, suggestions to determine the effective hogging and sagging length of structures spanning the greenfield inflection point are proposed.

These research contributions can lead to improvements of future centrifuge testing of structure response to tunnelling-induced ground movements and more informed assessment of building damage caused by tunnel excavation.
10.3 Limitations and applicability of results

As with all experimental work, there is a limit on the different configurations that can be tested in a given time frame. For this reason, the modelled soil type and tunnelling scenario were kept constant throughout this research, while the building configurations varied. However, the tunnelling-induced displacement field depends on various parameters including the tunnel depth, the soil type and the relative soil density. Consequently, modified tunnelling conditions may affect the observed soil behaviour which then potentially alters the measured building response.

A further limitation of the conducted centrifuge experiments is that the 3D effects of tunnel advancement, which might cause adverse torsional response of the building, were not studied. In addition, all the building configurations were located orthogonal to the tunnel axis and a skewed building-to-tunnel position was not investigated. While various researchers showed that buildings orthogonal to the tunnel are more susceptible to tunnelling-induced damage (e.g. Camós and Molins, 2015; Yiu et al., 2017) than equal buildings affected by a skew tunnel further data is required to quantify the involved soil–structure interaction.

The building configurations of this experimental study focused on buildings on shallow foundations located on the soil surface. While this represents an extreme scenario of a shallow foundation, an embedded foundation was not considered. Evaluating a potential effect of a foundation embedment on the tunnel–soil–building interaction mechanisms should be undertaken in the future.

The generalisability of the obtained results is therefore subject to certain limitations (see above) and thus appropriate engineering judgement is required when applying the presented research outcome to real tunnelling projects.

10.4 Future research

The conducted experimental research acquired important data to get new insights into tunnel–soil–structure interaction mechanisms, but also identified areas for future work:

- **Centrifuge model testing**: As discussed above, this research was limited to a certain number of centrifuge tests. First, a natural progression of this work would be to supplement the acquired experimental data by studying a wider range of building variations. This could include a wider range of building-to-tunnel positions, aspect ratios, building layouts and/or façade opening areas. Specifically, an investigation into different distances between strip footing to explore the influence zone of a single strip is strongly
Conclusions

recommended. Second, the centrifuge test programme could be extended to further tunnel depths, skewed tunnels, embedded foundations and/or ground conditions. This would enable to better generalise the obtained findings. Third, the mechanical properties of the 3D printed building models could be further refined to better replicate the strength properties of masonry in order to assess cracking effects. A preliminary investigation into the influence of the curing temperature was provided in Chapter 4. However, spin-up phenomena (assessed in Chapter 5) need to be considered when more vulnerable structures are tested.

• **Towards design:** This work provided recommendations to improve future predictions of building damage caused by tunnelling works. However, more detailed building monitoring data from future tunnelling projects needs to be collected to test and confirm the reliability of these suggestions. Recent work on latest tunnelling projects pointed out that building monitoring sections are often limited by the number of monitoring points which makes it impossible to accurately assess the structural behaviour of an asset (DeJong et al., 2016). Future research should therefore concentrate on acquiring detailed building monitoring data to further improve the understanding of this tunnel–soil–structure interaction problem in order to obtain more reliable design recommendations.

• **Numerical modelling:** This dissertation provides a rich dataset of building response to tunnelling-induced subsidence. Computational modelling studies could be validated with these experimental results. After testing, a wider range of scenarios could be evaluated. This envisioned future work could provide the required amount of data to establish robust tools to assess building response to tunnelling.
References


References


References


303


References


References


References


References


References


References


Appendix A

Building geometry

The following figures define the geometry of the 3D printed building models.
Fig. A.1 Building geometry for tests with $L = 200$ mm: (a) front, (b) side and (c) top view (dimensions in mm).
Fig. A.2 Building geometry for tests with $L = 260$ mm: (a) front, (b) side and (c) top view (dimensions in mm).
Building geometry

Fig. A.3 Facade geometry for tests A-C (dimensions in mm).

Fig. A.4 Facade geometry for tests D (dimensions in mm).
**Fig. A.5** Facade geometry for test E (dimensions in mm).

**Fig. A.6** Facade geometry for tests F and G (dimensions in mm).