Racking shear behaviour of steel frames with discretely connected precast concrete infill panels

Lieven, R.

Award date: 2011
RACKING SHEAR BEHAVIOUR OF STEEL FRAMES WITH DISCRETELY CONNECTED PRECAST CONCRETE INFILL PANELS

BY
R. LIEVEN
RACKING SHEAR BEHAVIOUR OF STEEL FRAMES WITH DISCRETELY CONNECTED PRECAST CONCRETE INFILL PANELS

BY

R. LIEVEN

A THESIS SUBMITTED TO THE DEPARTMENT OF ARCHITECTURE, BUILDING AND PLANNING AT EINDHOVEN UNIVERSITY OF TECHNOLOGY IN PARTIAL FULFILMENT OF THE REQUIREMENTS FOR THE DEGREE OF MASTER OF SCIENCE

THESIS SUPERVISOR COMMITTEE:
DR. IR. J.C.D. HOENDERKAMP
PROF. IR. H.H. SNIJDER
DR. IR. H. HOFMEYER

JUNE 2011
EINDHOVEN, THE NETHERLANDS
Acknowledgments

Several people have supported and helped me during different phases of my Master thesis. Therefore I owe them a word of gratitude.

First of all I would like to thank J.C.D. Hoenderkamp for offering me this research project and his enthusiastic guidance. I also would like to thank H.H. Snijder and H. Hofmeyer for their critical guidance and supervision during my research.

Furthermore I want to thank the complete staff of the Pieter van Musschenbroek laboratory for their professional help and pleasant time during my work in the laboratory. Without the help of H.M. Lamers, M.P.F. Canters, T.J. van de Loo, G.A.H. Maas and C.P.W.M van Asten, I would not have had appropriate test specimens for the experimental investigation. To perform these experimental tests I have had valuable help from H.L.M. Wijen, who I owe many thanks. I also want to thank J.J.P van den Oever, M.A.C.M. Ceelen and C.F.P. Naninck for their occasional help.

Additionally I would like to thank my colleague students and friends for their help and enjoyable time during my study, especially J. Courage and R.C. Spoorenberg for their help when I got stuck in the finite element program and J.A.H. Dekkers for his contribution concerning my research topic.

Finally I would like to thank my family for their support and contribution during my entire study.

Robbert

Eindhoven, June 2011
Summary

This research project is aimed at the study of a semi-integral infilled frame. A semi-integral infilled frame consists of a panel which is connected to a frame by four frame panel connections, figure 1. A frame panel connection consists of five anchor bars, embedded in the concrete panel and welded to an anchor plate. The anchor plate is welded perpendicular to another anchor plate which is fastened with two bolts to a gusset plate, welded to a steel section, figure 2. The bolts are loaded in shear only. The frame is made of four steel sections and the infill is a prefabricated reinforced concrete panel, which provides the stability of the structure since the steel sections are pin connected.

This type of structure can be used as structural system for tall buildings. The prefabricated infill panel will not only stiffen the structural behavior of the building structure, but will also offer the possibility to develop a completely integrated tall building system which improves the construction process. The structural elements are manufactured under controlled conditions, which results in high quality elements. This is important to strength, stiffness and tolerances of the elements. The panels arrive at the building site complete with window-frames, insulation, façade and even cable ducts and plumbing.

Although the contribution of the infills to the lateral stiffness and stability of frame structures has been recognized for a long time, the infills are still considered non-structural and their effects are usually neglected in structural analysis and design. The main reasons are the lack of knowledge about the composite behavior of infilled frames and practical methods for estimating their stiffness and strength. Equations for preliminary design are lacking. Generally it is assumed that ignoring the influence of the infill will lead to a conservative or safe design of the structure. This may be correct for low-rise structures but it has been shown that these assumptions for high-rise structures are not correct and even may overload the lower parts of the structural frame. Therefore research into semi-integral infilled frames, especially the frame panel connection, was started at Eindhoven University of Technology. The frame panel connection is the governing part when it comes to strength and stability of semi-integral infilled frames. In this research a new type of frame panel connections, FPC3, will be applied.
By performing full scale experimental tests, insight into the racking shear behaviour of the infilled frame with the new frame panel connection (FPC3) will be obtained. A test specimen consists of a steel frame, a reinforced concrete panel and FPC3. Together these elements form a single-story, single-bay three by three meter infilled frame structure. The load deflection behaviours of the infilled frames with FPC3 are compared with the results of earlier investigated semi-integral infilled frames.

In addition to the experimental tests, a finite element model has been created to analyze the racking shear behaviour of the infilled frames. The frame panel connections are modeled by longitudinal springs. The spring characteristics are gained from previously carried out connection tests. The finite element model has been calibrated and checked for accuracy with data from the experimental full scale tests. This the finite element model can than be used for further parametric studies. By performing separate connection tests, the frame panel connection characteristics can be adapted, in the finite element model, to determine the influence on a full scale model.

Compared to the previous investigated infilled frames with FPC1, FPC2 and FPCL, the infilled frame with FPC3 shows significantly improved behaviour in terms of strength and stiffness, figure 3. The increase of strength and stiffness of the infilled frame is caused by the improved frame panel connection and by relocation of the frame panel connections to the frame’s corners.

The results from the finite element analysis with the spring characteristics of FPC3, correspond to the racking shear behaviour found during the full scale experimental tests, figure 4. Both load-deflection graphs show great similarity in terms of change of stiffness. Also the ultimate load is corresponding with a high accuracy.

![Figure 3 - Racking shear behaviour of infilled frames.](image1)

![Figure 4 - Finite element results.](image2)
Nomenclature

**Latin symbols**

- **A**: Cross sectional area
  
- **A<sub>cb</sub>**: Cross sectional area of bolt
  
- **A<sub>b;s</sub>**: Cross section of shear plane of bolt
  
- **A<sub>o</sub>**: Original Cross sectional area
  
- **A<sub>s</sub>**: Tensile stress area of anchor bars
  
- **a**: Weld throat
  
- **b<sub>p</sub>**: Effective beam width, centre to centre of the bar spacing
  
- **C<sub>C</sub>**: Compression spring stiffness
  
- **C<sub>E</sub>**: Extension spring stiffness
  
- **C<sub>L</sub>**: Longitudinal spring stiffness
  
- **C<sub>O</sub>**: Spring stiffness due to bolt hole ovalisation
  
- **C<sub>p</sub>**: Extension-compression spring stiffness
  
- **C<sub>R</sub>**: Spring stiffness due to bolt rotation
  
- **C<sub>S</sub>**: Shear spring stiffness
  
- **C<sub>SL</sub>**: Spring stiffness due to anchor bar slip
  
- **C<sub>T</sub>**: Torsional spring stiffness
  
- **c**: Spacing cover of anchor bars
  
- **c<sub>d</sub>**: Concrete cover
  
- **d**: Nominal bolt diameter
  
- **d<sub>b</sub>**: Anchor bar diameter
  
- **d<sub>o</sub>**: Bolt hole diameter
  
- **E<sub>c</sub>**: Young’s modulus of concrete
  
- **E<sub>FPC1</sub>**: Young’s modulus of steel, frame panel connection 1
  
- **E<sub>FPC2</sub>**: Young’s modulus of steel, frame panel connection 2
  
- **E<sub>s</sub>**: Young’s modulus of steel
  
- **e<sub>1</sub>**: End distance from the center to the bolt hole to the adjacent end of the plate, measured in the direction of the load transfer
  
- **e<sub>2</sub>**: End distance from the center to the bolt hole to the adjacent end of the plate, measured at right angles to the direction of the load transfer
  
- **F**: Force
  
- **F<sub>a</sub>**: Axial force
  
- **F<sub>bfr</sub>**: Lateral force at bare frame
  
- **F<sub>b;Rd</sub>**: Bearing resistance per bolt
  
- **F<sub>dia;com</sub>**: Force in the compression diagonal of concrete panel
  
- **F<sub>dia;ten</sub>**: Force in the tension diagonal of concrete panel
  
- **F<sub>el</sub>**: Elastic strength
$F_{FFPC1}$ : Ultimate frame panel connection 1 strength [N]

$F_{FFPC2}$ : Ultimate frame panel connection 2 strength [N]

$F_{lf}$ : Lateral force at infilled frame [N]

$F_{p,u}$ : Ultimate pull-out strength [N]

$F_{S,u}$ : Ultimate shear strength [N]

$F_{ten}$ : Ultimate tension strength [N]

$F_{V,Rd}$ : Shear resistance per bolt [N]

$F_{w,Rd}$ : Weld resistance per unit length [N]

$f_{bd}$ : Ultimate adhesive strength [N/mm$^2$]

$f_{ck}$ : Compressive strength of concrete [N/mm$^2$]

$f_s$ : Design value of the tensile strength of an anchor bar [N/mm$^2$]

$f_{w,d}$ : Design shear strength of the weld [N/mm$^2$]

$f_u$ : Ultimate tensile strength [N/mm$^2$]

$f_{ub}$ : Ultimate tensile strength of the bolt material [N/mm$^2$]

$f_y$ : Yield stress [N/mm$^2$]

$H$ : Height [mm]

$h$ : Section height [mm]

$I_b$ : Moment of inertia beam section [mm$^4$]

$I_c$ : Moment of inertia column section [mm$^4$]

$I_y$ : Moment of inertia y-axis [mm$^4$]

$k_{ini}$ : Initial structural stiffness [N/mm]

$k_{tan}$ : Tangent structural stiffness [N/mm]

$k_1$ : Edge distance factor [-]

$L$ : Length [mm]

$l_{bd}$ : Design value of the anchor bar length [mm]

$l_{0,min}$ : Minimal anchor bar length [mm]

$l_{0,rad}$ : Necessary basic anchor bar length [mm]

$L_c$ : Length original cross section area [mm]

$l_d$ : Required embedment length [mm]

$l_{eff}$ : Effective length of fillet weld [mm]

$L_o$ : Original gauge length [mm]

$L_t$ : Total length tensile coupon [mm]

$l_{w,min}$ : Minimum weld length [mm]

$N_{di}$ : Design value of the theoretical tension force [N]

$M$ : Moment [Nmm]

$p_1$ : Spacing between centers of fasteners in an outer line in direction of load [mm]

$p_2$ : Spacing measured perpendicular to the load transfer direction between adjacent lines of fasteners [mm]

$t$ : Thickness [mm]

$u$ : Displacement [mm]

$W_y$ : Moment of resistance [mm$^3$]

$w$ : Width [mm]
Greek symbols

\( \alpha_b \) : Bearing factor  
\( \alpha_s \) : Shear plane through bolt part factor  
\( \alpha_2 \) : Effect of minimum concrete cover coefficient  
\( \beta_w \) : Correlation factor for fillet wells  
\( \gamma M2 \) : Partial factor for resistance of cross-sections in tension to fracture  
\( \varepsilon \) : Strain  
\( \nu_c \) : Poisson’s ratio of concrete  
\( \nu_s \) : Poisson’s ratio of steel  
\( \sigma \) : Stress  
\( \sigma_{sd} \) : Value of the anchor bar stress  
\( \sigma_1 \) : Stress parallel to force  
\( \sigma_\perp \) : Stress perpendicular to force  
\( \tau_1 \) : Shear stress parallel to force  
\( \tau_\perp \) : Shear stress perpendicular to force

Symbols:

Nomenclature

\[ \text{N/mm}^2 \]
## Contents

1 INTRODUCTION .................................................................................................................. 1

1.1 INFILLED STEEL FRAMES .......................................................................................... 1

1.2 MOTIVATION AND RELEVANCE ............................................................................. 1

1.3 PROBLEM STATEMENT ............................................................................................ 2

1.4 OBJECTIVES ............................................................................................................... 2

1.5 FINAL RESULTS ......................................................................................................... 2

1.6 PROCEDURE OF RESEARCH .................................................................................... 3

1.7 REPORT OUTLINE ....................................................................................................... 4

2 LITERATURE REVIEW .................................................................................................... 5

2.1 INFILLED FRAMES ..................................................................................................... 5

2.1.1 NON-INTEGRAL INFILLED FRAMES ....................................................................... 5

2.1.2 SEMI-INTEGRAL INFILLED FRAMES ...................................................................... 5

2.1.3 FULLY-INTEGRAL INFILLED FRAMES ..................................................................... 7

2.2 GENERAL BEHAVIOUR .............................................................................................. 7

2.3 SEMI-INTEGRAL FRAME PANEL CONNECTIONS ........................................................ 10

2.3.1 FRAME PANEL CONNECTION 1 ............................................................................. 10

2.3.2 FRAME PANEL CONNECTION 2 ............................................................................. 11

2.3.3 FRAME PANEL CONNECTION 3 ............................................................................. 12

2.3.4 FRAME PANEL CONNECTION L ............................................................................ 13

2.4 FINITE ELEMENT ANALYSIS ..................................................................................... 14

2.4.1 STEEL FRAME ........................................................................................................ 14

2.4.2 BEAM-COLUMN CONNECTION ........................................................................... 15

2.4.3 CONCRETE PANEL ................................................................................................ 16

2.4.4 FRAME PANEL CONNECTION ............................................................................... 16

2.4.5 VERIFICATION OF FINITE ELEMENT MODELS ..................................................... 20

3 EXPERIMENTAL INVESTIGATION .................................................................................. 23

3.1 TEST SPECIMENS ....................................................................................................... 23

3.1.1 STEEL FRAME ....................................................................................................... 23

3.1.2 CONCRETE PANEL ............................................................................................... 24

3.1.3 FRAME PANEL CONNECTION ............................................................................... 24

3.2 TEST RIG .................................................................................................................... 25

3.3 TESTING PROCEDURE .............................................................................................. 25

3.4 MEASURING ARRANGEMENT .................................................................................. 27

3.5 OBSERVATIONS AND RESULTS ................................................................................. 30

3.5.1 SPECIMEN A ......................................................................................................... 30

3.5.2 SPECIMEN B ......................................................................................................... 34
4  **FINITE ELEMENT INVESTIGATION** .................................................................41
   4.1  **FINITE ELEMENT MODELING** ...............................................................41
   4.2  **STEEL FRAME** .........................................................................................41
      4.2.1  **ELEMENTS** ..................................................................................41
      4.2.2  **GEOMETRY** ..................................................................................42
      4.2.3  **MATERIAL PROPERTIES** ..............................................................42
      4.2.4  **TORSIONAL SPRING CHARACTERISTICS** ....................................42
   4.3  **CONCRETE PANEL** ................................................................................42
      4.3.1  **ELEMENTS** ..................................................................................42
      4.3.2  **GEOMETRY** ..................................................................................42
      4.3.3  **MATERIAL PROPERTIES** ..............................................................42
   4.4  **FRAME PANEL CONNECTION** ..............................................................43
      4.4.1  **ELEMENTS** ..................................................................................43
      4.4.2  **GEOMETRY** ..................................................................................43
      4.4.3  **LONGITUDINAL SPRING CHARACTERISTICS** .............................44
   4.5  **BOUNDARY CONDITIONS AND SOLUTION PROCEDURE** ..................49
   4.6  **RESULTS FINITE ELEMENT ANALYSIS** ...............................................50
   4.7  **CHAPTER CONCLUSIONS** ....................................................................51

5  **DISCUSSION** ..................................................................................................55
   5.1  **RESULTS EXPERIMENTAL TESTS** .....................................................55
   5.2  **RESULTS FINITE ELEMENT ANALYSIS** .............................................56

6  **CONCLUSIONS AND RECOMMENDATIONS** ..............................................59
   6.1  **CONCLUSIONS** ....................................................................................59
   6.2  **RECOMMENDATIONS** ...........................................................................59

REFERENCES............................................................................................................61
# Contents

APPENDIX A  **DRAWINGS**

<table>
<thead>
<tr>
<th>Appendix</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>INFILLED FRAME</td>
<td>64</td>
</tr>
<tr>
<td>A2</td>
<td>BARE STEEL FRAME</td>
<td>65</td>
</tr>
<tr>
<td>A3</td>
<td>DETAILS STEEL FRAME</td>
<td>66</td>
</tr>
<tr>
<td>A4</td>
<td>REINFORCEMENT CONCRETE PANEL</td>
<td>67</td>
</tr>
<tr>
<td>A5</td>
<td>STEEL PARTS FOR FRAME PANEL CONNECTION</td>
<td>68</td>
</tr>
<tr>
<td>A6</td>
<td>FRAME PANEL CONNECTION</td>
<td>69</td>
</tr>
</tbody>
</table>

APPENDIX B  **PHOTOS**

<table>
<thead>
<tr>
<th>Appendix</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>REINFORCEMENT PANEL A</td>
<td>72</td>
</tr>
<tr>
<td>B2</td>
<td>TEST SPECIMEN A1</td>
<td>73</td>
</tr>
<tr>
<td>B3</td>
<td>TEST SPECIMEN A2 BEFORE TESTING</td>
<td>74</td>
</tr>
<tr>
<td>B4</td>
<td>TEST SPECIMEN A2 AFTER TESTING</td>
<td>76</td>
</tr>
<tr>
<td>B5</td>
<td>TEST SPECIMEN A3 AFTER TESTING</td>
<td>79</td>
</tr>
<tr>
<td>B6</td>
<td>REINFORCEMENT PANEL B</td>
<td>82</td>
</tr>
<tr>
<td>B7</td>
<td>TEST SPECIMEN B1</td>
<td>83</td>
</tr>
<tr>
<td>B8</td>
<td>TEST SPECIMEN B2 BEFORE TESTING</td>
<td>84</td>
</tr>
<tr>
<td>B9</td>
<td>TEST SPECIMEN B2 AFTER TESTING</td>
<td>86</td>
</tr>
<tr>
<td>B10</td>
<td>TEST SPECIMEN B3 AFTER TESTING</td>
<td>88</td>
</tr>
</tbody>
</table>

APPENDIX C  **MATERIAL PROPERTIES**

<table>
<thead>
<tr>
<th>Appendix</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>MATERIAL PROPERTIES CONCRETE</td>
<td>92</td>
</tr>
<tr>
<td>C2</td>
<td>MATERIAL PROPERTIES STEEL FPC</td>
<td>94</td>
</tr>
</tbody>
</table>

APPENDIX D  **CALCULATIONS**

<table>
<thead>
<tr>
<th>Appendix</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>D1</td>
<td>BEAM-COLUMN CONNECTION STEEL FRAME</td>
<td>98</td>
</tr>
<tr>
<td>D2</td>
<td>DESIGN CALCULATIONS FRAME PANEL CONNECTION 3</td>
<td>100</td>
</tr>
</tbody>
</table>

APPENDIX E  **OVALISATION BOLT HOLES**

<table>
<thead>
<tr>
<th>Appendix</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>E1</td>
<td>OVALISATION BOLT HOLES ANCHOR PLATE TWO</td>
<td>108</td>
</tr>
<tr>
<td>E2</td>
<td>OVALISATION BOLT HOLES GUSSET PLATE</td>
<td>110</td>
</tr>
</tbody>
</table>

APPENDIX F  **FINITE ELEMENT MODEL**

<table>
<thead>
<tr>
<th>Appendix</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>F1</td>
<td>CONSTRUCTION FINITE ELEMENT MODEL</td>
<td>114</td>
</tr>
<tr>
<td>F2</td>
<td>INPUT FILE BARE FRAME</td>
<td>116</td>
</tr>
<tr>
<td>F3</td>
<td>INPUT FILE INFILLED FRAME</td>
<td>123</td>
</tr>
</tbody>
</table>

APPENDIX G  **ENLARGED FIGURES**

<table>
<thead>
<tr>
<th>Appendix</th>
<th>Title</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>G1</td>
<td>ENLARGED FIGURES CHAPTER 3</td>
<td>138</td>
</tr>
<tr>
<td>G2</td>
<td>ENLARGED FIGURES CHAPTER 4</td>
<td>139</td>
</tr>
<tr>
<td>G3</td>
<td>ENLARGED FIGURES CHAPTER 5</td>
<td>143</td>
</tr>
</tbody>
</table>
1 Introduction

1.1 Infilled steel frames
An infilled frame consists of four pin connected steel sections (two columns and two beams) and a panel. The panel provides the stability of this skeletal steel frame and is made of reinforced concrete or masonry. The integration of the panel in the steel frame can be classified into three types. Those classifications are based on the connection between the frame and the infill panel. Infilled frames have been classified as non-integral, semi-integral and fully-integral, figure 1.1.

In a non-integral infilled frame, the frame and the infill are not physically connected with each other. The infill consists usually of masonry, figure 1.1a. A semi-integral infilled frame consists of a precast concrete panel discretely connected to the steel frame, figure 1.1b and 1.1c. In contrast to the non-integral infilled frame, the infill in a fully integral infilled frame has a continuous connection to the steel frame. This connection is obtained by casting concrete in situ, figure 1.1d.

This research project is aimed at the study of a semi-integral infill frame with corner connections. This type of infill frame has not been investigated before. There are no equations for preliminary design of the semi-integral frame. For this reason research was started at Eindhoven University of Technology.

1.2 Motivation and relevance
Prefabricated concrete elements are widely used in the Dutch construction industry, especially for low-rise structures. Therefore it could be useful to develop design equations for steel frames, infilled with precast concrete panels, for the use in high-rise structures. The prefabricated infill panels will not only stiffen the structural behavior of the steel frame but also offer the possibility to develop a completely integrated tall building system, which improves the construction process and even can lead to a reusable product.

The structural elements are manufactured under controlled conditions, which results in high quality elements. This is important to strength, stiffness and tolerances of the elements. The panels arrive at the building site complete with, window frames, insulation, façade and even cable ducts and plumbing.

Figure 1.1 - Steel frames with infill panels: a) Non-integral, b & c) Semi-integral, d) Fully-integral.
These elements can be assembled directly from a truck to safe space on the construction site. The elements can be removed as easily as they were placed at the end of the building life. Reuse of the infill panels is then possible.

1.3 Problem statement
Research on steel frames with discretely connected precast concrete infill panels was started at Eindhoven University of Technology to formulate design equations for strength, stiffness and deformation capacity. During this research two types of discrete frame panel connections, FPC1 and FPC2, have been developed and tested individually [2, 5] and also applied in full scale tests [5, 14], figure 1.2. The characteristics of both discrete connections were also used in a finite element model to estimate the behaviour of the full scale experimental tests [5, 14].

With the use of a finite element model, parametric studies can be carried out to develop design equations for infilled steel structures.

As a result of this research the location of the frame panel connections has been changed, figure 1.1c. Through the change of location a new frame panel connection, FPC3, has been developed between the concrete and steel frame [4], figure 1.2c. Through experimental testing the characteristics, of this frame panel connection, have been determined [3]. However, the behaviour of this connection in a full scale test is still unknown. There is also no finite element model available to estimate the actual system behaviour related to the new frame panel connection in a full scale test. It is assumed that the new frame panel connection will improve the stiffness of the steel frame compared to the connections used in prior full scale tests [5, 14].

1.4 Objectives
The objectives of this research project are:

- To study the racking shear behaviour, in terms of strength, stiffness and deformation capacity, of a steel frame infilled with a precast concrete panel discretely connected to the beams at the frame corners by performing full scale experimental tests on two infilled frames.
- To estimate the influence of the new frame panel connection on the racking shear behaviour of the infilled frame compared to prior investigated frame panel connections in semi-integral infilled frames.
- To develop a nonlinear finite element model to estimate the racking shear behaviour of the full scale experimental tests.

1.5 Final results
By performing two full scale experimental tests, insight into the racking shear behaviour of the infilled frame with FPC3 will be obtained. The data from these experimental tests will be compared with earlier carried out investigations [14, 15].

The characteristics of the frame panel connections in the finite element model, represented by longitudinal springs, are gained from previously carried out connection tests [4]. The results from the full scale experimental tests will also be used for the calibration of the nonlinear finite element model. The nonlinear finite element model can then be used for further parametric studies and offers the possibility to investigate the influence of different frame panel connections on infilled frames without carrying out new experimental full scale tests. Only connection tests have to be performed to gain new spring characteristics for the nonlinear finite element model.
1.6 Procedure of research

Before experimental and numerical research will begin, a literature study will be started to gain an understanding of the behaviour of previously investigated infilled frames. The experimental tests are subdivided into three tests, in which the following characteristics will be determined:

- The stiffness of the bare steel frame
- The material properties from the applied steel of FPC3 and the used concrete for the infill
- The behaviour of the infilled frame in terms of strength, stiffness and deformation capacity (two infilled frames will be tested, each infilled frame will be tested twice)

After completion of the experimental tests a nonlinear finite element model will be developed. The frame panel connections will be represented by using longitudinal spring. The spring characteristics are gained from previously carried out connection tests [4]. The reliability of the nonlinear finite element model will be checked with the results from the experimental full scale tests. After calibration the nonlinear finite element model should be ready for parametric studies. A flowchart of the procedure of research is given in figure 1.3.

Figure 1.3 - Flowchart of the research method.
1.7 Report outline

This report consists of six chapters. In the current introductory chapter, the topic and motivation of the research have been introduced. Chapter two consists of a literature review of research done in the past into infilled frames. Chapter three presents the experimental investigation of the full scale tests. The full scale tests are simulated with the use of a finite element model, which is described in chapter four. The results of these investigations are discussed in chapter five. Finally the conclusions and recommendations are presented in chapter six.

Figures marked with an asterisk are presented enlarged in appendix G.
2 Literature review

2.1 Infilled frames

The increasing demand towards taller buildings requires more efficient and innovative structural systems than the used systems these days. The use of steel rigid frames as lateral load resisting structures in tall buildings is too expensive because of the increasing costs of labour required for fabrication of moment resisting connections. Braced or trussed frames do not have those expensive connections but have bracing diagonals that obstruct eventually needed openings in the frame.

A more efficient and innovative structural system, is the infilled frame. The relatively expensive moment connections, that provide the stability of a rigid frame, can be omitted by the use of infills. A pin connection between the column and beam will now suffice, because the infill will act like a bracing strut. Vertical loads acting on the structure are supported by the steel frame structure only, while horizontal loads are transferred to the foundation by the composite action between the infill and frame. The benefit of an infill compared to a strut is the possibility to add door or window openings in the structural frame [7, 16].

2.1.1 Non-integral infilled frames

In the fifties research began on the structural behaviour of non-integral infilled frames by subjecting the frame to a racking load. The infill usually consisted of masonry. Tests have shown that, despite the low and variable tensile strength of masonry, the infilled frame is considerably stronger and stiffer, when subjected to lateral forces, compared to a bare frame [17]. Although the contribution of the infill has been recognized, there are still several problems that may occur. Sudden failure of the structure can appear, with little warning, in terms of cracks in the masonry. The difference in the deformed shapes of the infill and the frame causes poor interaction between the frame and infill. Only at the loaded corners of the infill interaction will remain between the frame and infill. The infill takes a large portion of the lateral load at the loaded corners and therefore acts like a single diagonal compressive bracing strut. The infill can therefore be considerate as an equivalent strut, figure 2.1a [6, 7].

In recent research on non-integral infilled frames, calcium silicate elements were used as infill material [11]. Several tests with and without gaps between the infill and frame and with different sections for the steel beams and columns were carried out. The lateral stiffness of a non-integral infill frame depends on the amount of margin between the infill panel and the frame structure. Compared to the other types of infill frames its lateral stiffness is small, figure 2.2a.

2.1.2 Semi-integral infilled frames

A semi-integral frame consists of a precast concrete panel discretely connected to the steel frame. Interaction at the interface is neither complete nor absent. In the eighties a similar idea about semi-integral frames was considered [8]. However, the infill panel was not discretely connected to the frame but finite shear strength at the infill frame interface was taken into account.
In the late nineties the first discretely connected infill panels were experimentally tested at Eindhoven University of Technology. The design of the frame panel connection has changed since the beginning of the research on semi-integral infilled frames, figure 2.3 [4, 5, 9, 15].

During the investigation of the first frame panel connection (FPC1, figure 2.3a), ten different patterns of the frame panel connections were considered. The conclusions are that frame connections located at the beams of the frame are more efficient than located at the columns of the frame. Also the lateral stiffness of the frame will increase when the frame connections are located closer to the frame corners. However, due to the size of the frame panel connection it was not possible to locate the connection near the frame corners. This investigation makes clear that the contribution of a discretely connected precast infill panel to the lateral stiffness and strength of a frame could be significant, when the frame panel connections are located according to an optimum pattern on the frame, figure 2.2a [15].

The first used frame panel connection, in a full scale test, seemed to be to conservatively designed. An attempt to improve the frame panel connection resulted in FPC2, figure 2.3b. This connection did not come up to expectations [14]. The frame panel connection was designed to fail by ovalisation of the bolt holes due to bearing of the bolts, but during the full scale test the connection failed by anchor pull out. The lateral strength of the infill structure with FPC2 is a deterioration compared to FPC1, figure 2.2b.

As a result of these tests a new connection, FPC3, has been developed, figure 2.3c [4]. This connection is based on a different load configuration than FPC1 and FPC2. Therefore the location of FPC3, at the steel frame is changed, figure 2.1b & 2.1c. The new frame panel connection, located at the frame corners, will only be subjected to tension or compression forces, corresponding with the compression and tension diagonals in the concrete panel [3, 4]. This is an optimum location for the frame panel connections [15].
Parallel to the research on frame panel connections 1, 2 and 3, a fourth discrete frame panel connection, FPCL, has been investigated in a full scale test [9], figure 2.3d. This connection is also located at the frame’s corners, which leads to an absence of shear forces in the connection. This connection can only transfer compression forces from frame to panel and vice versa due to limitations of the frame panel connection. Due to this restriction the lateral stiffness of the infilled frame with FPCL is comparable to the stiffness of the infilled frame with FPC1, despite the unfavourable positions of FPC1 in the frame, figure 2.2b.

2.1.3 Fully-integral infilled frames

Studies on the elastic behaviour of single-story, single-bay infilled frames was started in the early seventies. In contrast to the non-integral frame, the infill of a fully integral frame has a continuous connection to the structural frame, figure 2.1d. This can be obtained by bonding between cast in situ concrete and the steel frame or providing steel connections along the entire interface. Through the shear connectors the infill and frame remain fixed which means that there will not only occur a compression diagonal, but also a tension diagonal, figure 2.1d. This improves the performance of the infilled frame in terms of load resistance and lateral stiffness compared to the two other types of infilled frames, figure 2.2a.

2.2 General behaviour

Although the contribution of the infills to the lateral stiffness and stability of frame structures has been recognized for a long time, the infills are still considered nonstructural and their effects are usually neglected in the structural analysis and design. The main reasons are the lack of knowledge about the composite behavior of infilled frames and practical methods for estimating their stiffness and strength. Generally it is assumed that ignoring the influence of the infill will
lead to a conservative or safe design of the structure. This may be right for low-rise structures but it has been shown that these assumptions concerning high-rise structures are not correct and even may overload the lower parts of the structural frame [12].

The deformation of an infilled frame subjected to lateral loading is different to that of a rigid frame without an infill. Due to lateral loading the beam-column connections of a rigid frame will rotate and the beams and columns display double curvature. This is called a racking shear configuration, figure 2.4b. The infilled frame on the other hand will tend to behave as a flexural element, figure 2.5b.

To clarify the difference in behaviour of the above mentioned structures, it is assumed that the cross sectional area of the compressive bracing strut, or equivalent strut (which represents the infill panel), can vary between zero and infinity. The cross sectional area is zero for the rigid frame and infinite for the infilled frame. It is assumed that the actual behaviour of an infilled frame lies between those configurations.

The lateral deflection of a rigid frame subjected to horizontal loading is shown in figure 2.4b. The deflected shape of the structure indicates a racking shear configuration. The overturning moment ($F_a L$) is a resisting couple, set up by the axial column force ($F_a$) multiplied by the moment arm ($L$) and depends on the ability of the beams to carry shear across the structure between the columns. Slender beams will keep the axial forces in the columns relatively small, figure 2.4c. When the beams have almost zero stiffness the lateral load will be completely resisted by individual bending moments in the columns, figure 2.4d.

A frame with an infinite cross sectional area for the diagonal deforms like a flexural cantilever, figure 2.5b. The strut will carry the shear across the frame and will maximize the axial forces ($F_a$) in the columns, figure 2.5c. Thereby the overturning moment will increase compared to the rigid frame and the bending moments in the columns will become smaller, figure 2.5d.

Figure 2.4 - Rigid frame: a) Loading, b) Deflection, c) Axial column force, d) Bending moment of columns.
The behaviour of an infilled frame will lie between the above mentioned cases. The bending moments in the columns of an infilled frame are smaller than the bending moments in the columns of a rigid frame. Thus ignoring the infill will lead to a safe and conservative design. However, the overturning moment of the infilled frame is larger than of the rigid frame. Ignoring the infill will cause a larger overturning moment and thus higher axial forces in the columns, which will lead to an unsafe design of the columns [12]. In figure 2.6 the gray infill indicates the difference between both frames for deformation, axial forces and bending moments. The actual behaviour of an infilled frame is located between the above discussed examples in the gray colored area of figure 2.6.

Figure 2.5 - Infilled frame: a) Loading, b) Deflection, c) Axial column force, d) Bending moment of columns.

Figure 2.6 - Frame differences: a) Deflection, b) Axial column force, c) Bending moment of columns.
2.3 Semi-integral frame panel connections

This research project is aimed at the study of a semi-integral infill frame and therefore only the frame panel connections used in semi-integral infill frames will be considered. Also the frame panel connection, applied in a semi-integral infill frame, is the governing part for the strength and lateral stiffness of the structure.

A logical method for connecting a precast concrete panel to a steel structure is by the use of a cast in steel plate. This type of connection has been investigated extensively [13]. Anchor bars are welded to a cast in steel plate, which will secure the steel plate in the concrete. The cast in steel plate can than be connected to a steel structure. This type of ‘connecting’ is applied at FPC1 and FPC2, figure 2.3a & 2.3b.

The frame panel connections are bolted to the steel structure because welding has a negative influence on the strength of the connection. The head of the welding causes cracks in the concrete panel which will loosen the anchor bars. These cracks will increase when the steel plate is subjected to a compression load. The steel plate will act like a knife and split the concrete. The ‘bigger’ cracks lead to slip and deformation of the anchor bars in the concrete, which will weaken the connection.

2.3.1 Frame panel connection 1

The design of the first frame panel connection, FPC1, applied in a semi-integral frame, is mainly based on Eurocode 3, even though this code does not contain any design rules about frame panel connections between concrete and steel [2].

The frame panel connection consists of two anchor bars Ø 18 mm welded on a 10 mm thick anchor plate, figure 2.3a. The anchor plate and anchor bars are cast in concrete. The cast in anchor plate is connected with two bolts to a gusset plate which is welded on the steel beam of the infill structure. The frame panel connection can not be placed near the frame corners due to the length of the anchor bars.

FPC1 has been designed to fail in bearing, meaning that ovalisation of the bolt holes in the anchor plates should occur. Several connection tests have been carried out to establish the tension and shear capacity of FPC1 [5]. Dimensions and material properties of the test specimens are given in table 2.1.

The calculated bearing resistance is 110 kN per bolt, therefore the design value for the anchor bars should be at least 220 kN. From experimental tests appears that the actual average shear and tensile strengths are 453 and 321 kN respectively, table 2.2 and figure 2.7 & 2.8 [5]. The desired failure mode, namely bearing of the bolts, occurred.

<table>
<thead>
<tr>
<th>Table 2.1 - Material properties of FPC1 &amp; FPC2.</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Bolts</strong></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>FPC1</td>
</tr>
<tr>
<td>FPC2</td>
</tr>
</tbody>
</table>

* assumed, ** approximate cylinder strength in parentheses
2.3.2 Frame panel connection 2

The second frame panel connection, FPC2, is a modified design based on FPC1, figure 2.3b. It has been assumed, and later confirmed by experimental tests, that FPC1 has been over dimensioned due to the shortcoming of Eurocode 3 [2].

FPC1 has been designed on a maximum shear force and a maximum tensile force of 220 kN each. It is not taken into account that the shear and tensile forces at the connection will occur simultaneously due to the angle of the tension or compression diagonal in the concrete panel, figure 2.1b. The maximum load of the tension or compression diagonal can not be higher than the calculated bearing resistant of 220 kN [2]. This results in a tensile and shear force at the frame panel connection of 202 and 86 kN respectively, figure 2.9. The maximum calculated tensile and shear forces are respectively 9% and 61% higher than the actual occurring forces. This shows that the frame panel connection has been over dimensioned.

The two anchor bars, used at FPC1, have been replaced by one shorter anchor bar, despite the minor overcapacity of the anchor bars at FPC1, figure 2.3a & 2.3b. The reason for this is to reduce the dimensions of FPC2, so that it can be placed closer to the frame corners.

From experimental tests on FPC2 it appeared that the actual shear resistance is larger than the calculated shear resistant, table 2.2 and figure 2.7. Also the desired failure mechanism, ovalisation of the bolt holes, occurred. It is assumed that the influence of the cast in gusset plate, where the anchor bars are welded on, partly causes the greater resistance. The influence of the gusset plate is not taken into account during the design of FPC2. Furthermore the steel quality of the gusset plates and the concrete strength are higher than the used values for the calculation [2].
In contradiction to the shear test, the tensile test failed in an unexpected way. Instead of bearing of the bolts the concrete failed by cracking. Bearing of the bolts was inhibited by the location of the anchor bar, figure 2.10. The location of the anchor bar is too close to the bolts which makes ovalisation of the bolt holes impossible. However, the connection failed at the calculated load, table 2.2 and figure 2.11.

### 2.3.3 Frame panel connection 3

FPC1 and FPC2 failed mainly in tension by anchor pull out, instead of bearing of the bolts, during the experimental full scale tests [14]. The tension and compression forces on the frame panel connections are 2.5 times larger than the shear forces.

A new frame panel connection, FPC3, was designed, based on only tension and compression forces, table 2.3 for material properties. To avoid shear forces the location of the frame panel connection changed to the frame corners, figure 2.1c. At this optimum location, according to previous investigations [15], only compression or tension forces will occur.

The design of FPC3 is different compared to the two previously used frame panel connections. At FPC3 the anchor bars are located perpendicular to the tension and compression forces and are therefore expected to be more effective. As a result of this optimum location the anchor bars can decrease in size, compared to the previous connections, which will increase the bond strength as well as the concrete cover on the anchor bars [3].

Another improvement of the frame panel connection is that there are no steel plates cast in the concrete panel. Instead of a cast in steel plate an anchor plate, anchor plate one, perpendicular to the concrete surface, connects the anchor bars with another anchor plate, anchor plate two, which is bolted to the gusset plate at the steel frame, figure 2.1c.

FPC3 is also designed on ovalisation of the bolt holes due to bearing of the bolts. The strength of the other parts from this connection are based on the bearing resistances of the bolt holes to

<table>
<thead>
<tr>
<th>Test</th>
<th>Shear</th>
<th>Tension</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Stiffness $C_s$ [kN/mm]</td>
<td>Strength $F_{p,u}$ [kN]</td>
</tr>
<tr>
<td>FPC1-1</td>
<td>86.2</td>
<td>457</td>
</tr>
<tr>
<td>FPC1-2</td>
<td>88.2</td>
<td>448</td>
</tr>
<tr>
<td>FPC1-3</td>
<td>54.9</td>
<td>337*</td>
</tr>
<tr>
<td>FPC1-4</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>FPC1 Avg.</td>
<td>(76.4)</td>
<td>(453)</td>
</tr>
<tr>
<td>FPC2-1</td>
<td>56.8</td>
<td>350</td>
</tr>
<tr>
<td>FPC2-2</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>FPC2 Avg.</td>
<td>(56.8)</td>
<td>(350)</td>
</tr>
<tr>
<td>FPC3-1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>FPC3-2</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>FPC3-3</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>FPC3-4</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>FPC3 Avg.</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

* test halted before ultimate load, ** No reliable results available to determine initial stiffness.
assure that this failure mechanism will occur. The calculated bearing resistance of FPC3 is 394 kN [3, 4].

Four specimens of FPC3 have been experimentally tested, table 2.2. Every specimen failed due to anchor bar pull out instead of the expected ovalisation of the bolt holes, figure 2.13. Also the ultimate tensile strength has been lower than the calculated strength, table 2.2 and figure 2.12. Despite the unexpected failure mechanism small ovalisation of the bolt holes occurred, figure 2.14. The ovalisation varies between 2.4 and 4.0 mm, depending on the specimen. In the second experimental test the structural bolts were fastened tighter than the bolts in the other tests. This caused friction between the two steel plates and influenced the magnitude of ovalisation of the bolt holes compared to the other tensile tests [4].

2.3.4 Frame panel connection L
This type of connection connects the precast concrete panel with the steel frame by structural bolts, instead of the earlier described cast in frame panel connections, figure 2.3d. The location of the structural bolts in the steel frame is comparable to the location of FPC3. The columns are now connected to the concrete panel as well.

The structural bolts are only able to transfer compression loads between the frame and panel. Therefore only the bolts in the compressed corners are active when the infilled steel frame is subjected to a lateral load. A compression diagonal will occur, just as happens in a non-integral infilled frame. Steel angles have been cast in in the concrete panel to avoid high stresses in the concrete at the position of the structural bolts. These high stresses will result in local cracking of the concrete, which can lead to loosening of the concrete panel.
The benefit of coupling the precast concrete panel and the steel frame by structural bolts is that tolerances of the panel, or frame, can be adopted in both horizontal and vertical direction by adjusting the bolts. Where the tolerance at a structure provided with FPC 1, 2 & 3 only can be adopted by reaming of the bolt holes. This frame panel connection also consists of fewer parts than the other frame panel connections, figure 2.3. For material properties, table 2.3.

The anticipated failure mode for this type of connection is a bolt failure mechanism consisting of bolt shear through the nut. In case of bolt failure, the bolts can easily be replaced while the concrete panel and steel frame remain undamaged [16].

FPCL was only applied in full scale experimental tests, not in separate compression tests [9]. However, characteristics of the structural bolts were tested in another research project [16]. From experimental tests it became clear that the strength and stiffness of FPCL mostly depends on the strength and stiffness of the flanges of the steel sections in the frame.

2.4 Finite element analysis

Numerical investigations of the semi-integral infilled frames with the frame panel connections, FPC1, FPC2 and FPCL have been carried out in different finite element programs [5, 9, 14]. There has also been carried out a numerical investigation to determine the optimal location of the frame panel connections [15]. This investigation does only represent the influence of the location of the frame panel connection.

The numerical model of FPC2 and the model to define the optimal location of the frame panel connection are modeled with Ansys 6.1 and Ansys 5.5 respectively. For the numerical investigation of the FPCL, ESA Prima win was used. A more detailed finite element model was created in Diana 9.2 for an investigation of comparable frame panel connections but with a different infill panel [16]. Due to the corresponding frame panel connection, this model will be described instead of the ESA Prima win model.

2.4.1 Steel frame

The finite element models of the investigated semi-integral steel frames with FPC1 and FPC2 have been modeled with Beam 3 elements [5, 14]. These elements have also been used in the investigation for determining the optimum location of the frame panel connection [15]. A Beam3 element is an uniaxial element with, tension, compression and bending capabilities. The element has three degrees of freedom at each node, namely translation in the nodal x and y direction and rotation about the nodal z-axis. This element does not have plastic, creep or swelling capabilities. Shear deformation is also neglected [1]. The columns and beams have been divided into five elements each.

| Table 2.3 - Material properties of FPC3 & FPCL. |
|-----------------|--------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|-----------------|
| **Bolts**       | **Anchor bars** | **Anchor plate 2** | **Angle section** | **Concrete panel** |
| FeB500          | 389x389x15 mm  | 150x150x18 mm    |                  |                  |
| \(\varnothing\) [mm] | \(f_u\) [N/mm²] | \(f_y\) [N/mm²] | \(f_y\) [N/mm²] | \(f_y\) [N/mm²] | \(R_{ck}\) [N/mm²] | \#8-100         | \#6-150         |
| FPC3            | M24 10.9       | 16 \(\ast\)    | 266             | 516             | -               | -               | 58 \((45)\) **   |
|                 | M27 10.9       | -               | -               | -               | 235 \*(\ast\)  | 340-470         | 44 \((34)\) **   |

\(\ast\) assumed, \(\ast\ast\) approximate cylinder strength in parentheses
For the semi-integral infilled frame with FPCL, class III beam elements, type CL9BE, have been used [16]. This beam element takes shear deformation into account in contradiction to the used beam elements in the Ansys models.

The used beam elements do not represent the physical dimensions of the beam and column sections. The beam and column geometry are numerically taken into account. This leads to a connection problem between the beam and column. When the beam and column depths are not taken into account the location of the connection is not at the actual position at the frame compared to the frame used in the experimental investigation. Therefore the beam-column connection is modeled by placing a rigid offset, to take the column’s depth into account. The rigid offset has also been modeled with BEAM3 elements but now with a very high stiffness to simulate the actual stiffness of the column at the point of the semi rigid connection, figure 2.15.

2.4.2 Beam-column connection
The bolted beam to column connections are neither rigid nor hinged. The semi rigid joints are modeled by torsional spring elements and are located at the end of rigid offsets that are connected to the columns at the neutral axis of the beams, figure 2.15.

It has been proved that modeling of the bare steel frame with the use of rigid offsets at the columns leads to a realistic load-displacement behaviour that approaches the load-displacement behaviour of the experimental test with a high accuracy [11, 14, 16]. The applied torsional spring stiffness ($C_T$) in the beam-column connection can be determined by the use of NEN-EN 1993-1-8 article 6.3. A linear elastic spring stiffness has been used in the numerical models.

The calculated spring stiffness for the finite element model, with FPC2, leads to a load-displacement behaviour that does not corresponds the actual load-displacement witch was found during the experimental full scale test [14]. The used spring stiffness in the finite element model has therefore been adjusted. Instead of a calculated spring stiffness, according to NEN-EN 1993-1-9 article 6.3, of $5.680 \times 10^6$ kNmm/rad, a spring stiffness of $2.5322 \times 10^6$ kNmm/rad has been applied. This stiffness leads to a load-displacement behaviour that describes the load-displacement behaviour of the full scale test.

![Figure 2.15 - Steel frame finite element model: a) Steel frame, b) Beam-column connection.](image-url)
For the finite element models of the infilled frames with FPC1 and FPC2, created in Ansys, a Combin14 element has been used to represent the spring stiffness of the semi rigid joints. This element has longitudinal or torsional capability in 1D, 2D or 3D applications [1]. For the connections between the columns and beams only the torsional capability has been used. The torsional spring damper option is a purely rotational element with three degrees of freedom at each node, rotations about the nodal x, y and z-axes. No bending or axial loads have been considered.

The applied spring element in the finite element model created in Diana, is a two node torsional spring element, type SP2RO.

2.4.3 Concrete panel
Numerical investigations with FPCI and FPC2 have been carried out. The differences between the two finite element models are the characteristics of the frame panel connection and the infill panel. Both panels have a thickness of 150 mm. The Young’s modulus are, \( E_{FPCI} = 29.2 \text{ kN/mm}^2 \) and \( E_{FPC2} = 31.0 \text{ kN/mm}^2 \).

The reinforced precast concrete panels, used in this investigation, were modeled with PLANE82 elements. This eight node element has two degrees of freedom at each node namely, translations in the nodal x and y directions. The element has plasticity, creep, swelling, stress stiffening, large deflection, and large strain capabilities [1].

In the numerical investigation for the determination of the optimum location of the frame panel connections, PLANE 42 elements have been used. This element is a four node element and has two degrees of freedom at each node [1]. This element has also plasticity, creep, swelling, stress stiffening, large deflection, and large strain capabilities. However this element provides less accurate results for mixed automatic meshes compared to the PLANE82 element.

Despite the anchor pull out failure mechanism, which occurred during the experimental tests, cracking of the concrete is not taken into account in the finite element model. Also the influence of the reinforcement bars is neglected.

In the finite element model, which is developed during the investigation of the infilled frame with FPCL, cracking of the concrete has been taken into account. Cracks will occur at the location of the frame panel connection. These cracks have influence on the stiffness of the reinforced concrete panel and therefore influence on the lateral stiffness of the infilled frame. The concrete panel is modeled with eight node isoparametric plane stress elements, type CQ16M, with a thickness of 200 mm [16]. Different Young modules have been used, depending on the Young’s modulus which was found per panel due to standard tests on concrete prisms. The influence of the reinforcement bars has been taken into account by modeling the longitudinal reinforcement and stirrups with reinforcement bars which are embedded in the plane stress elements. The applied Young’s modulus \( (E_s) \) for the reinforcement bars is 200 kN/mm\(^2\).

2.4.4 Frame panel connection
Frame panel connection 1 & 2
The connection of the reinforced concrete panel to the steel frame was modeled by using spring elements. The spring elements have been connected to a gusset steel plate at the steel frame and a cast in steel plate in the reinforced concrete panel, figure 2.16. Both the gusset and cast in steel plates were modeled with PLANE82 elements and have a thickness of 10 mm and the Young’s modulus \( (E_s) \) is 210 kN/mm\(^2\).
Four spring elements will connect the gusset and cast in steel plate together, figure 2.16. Two springs represent the shear stiffness \((C_s)\) of the frame panel connection and two springs represent the pull out stiffness \((C_p)\). The spring stiffness’s have been experimentally determined in shear and pull out tests [2]. These stiffness’s are taking the behaviour of the anchor bars in the concrete panel and ovalisation of the bolt holes in the steel plates into account.

The springs were modeled with COMBIN39 elements. This unidirectional element with nonlinear generalized load deflection capability can be used in any analysis. It has longitudinal or torsional capability in 1D, 2D or 3D applications. This element has a large displacement capability for which there can be two or three degrees of freedom at each node [1].

The determined values of the experimental shear and pull out tests have been used in the numerical full scale frame panel simulations, however the tensile strength had to be reduced due to interaction with the shear load on the connections [10].

The reduction of the tensile strength of FPC1 and FPC2 has been accomplished by the use of linear and elliptical interaction curves to establish upper and lower bounds for the lateral load capabilities of the infilled frames, figure 2.17 (based on average experimental gained values, table 2.2). The determined tensile strength for FPC 1 is 321 kN. The linearly and elliptically reductions are 305 kN and 243 kN respectively. For FPC 2 the determined tensile strength is 245 kN, the linearly reduction is 185 kN and the elliptically reduction is 233 kN.

The applied tension behaviour in the numerical simulations is shown in figure 2.18. Those bilinear shear and compression/tension curves are assumed to be accurate enough to approach the actual behaviour of the connections. The elastic tension behaviour is followed by a gradual linear reduction in load due to anchor bar pull out.

The compression stiffness is taken to be identical to the tensile stiffness, in the elastic region, for each connection.

The shear strength of the connections has not been reduced because the connections will mainly fail in pull out instead of shear, due to the steep angle of 60° between the upper and lower connections. The applied shear strength for FPC 1 & 2 is 453 kN and 350 kN respectively. The behaviour is also assumed to be elastic perfectly plastic.
Figure 2.17 – Linear and elliptical reduction.

Figure 2.18 – Tensile characteristics of FPC 1 & 2.

Frame panel connection 3

FPC3 has only been experimentally investigated for its tensile characteristics and has not been applied in a full scale test. Consequently, only a finite element model that represents the behaviour of the frame panel connection under tensile load has been created [4]. This finite element model is created in Diana and will be used for further investigation at FPC3 and can eventually be used in a full scale numerical model.

Since there occurred three phenomena in the experimental test, that influence the stiffness of FPC3, namely ovalisation of the bolt holes, rotation of the bolts and slip of the anchor bars, it is possible to implement those phenomena in longitudinal springs in two possibilities [4].

The first approach is to add reciprocal values of the phenomena, bolt hole ovalisation ($C_0$) and bolt rotation ($C_R$) to one spring stiffness and anchor bar slip into another spring stiffness ($C_{SL}$), figure 2.19a.

The initial stiffness due to ovalisation of the bolt holes ($C_{O, ini}$) is determined at 296 kN/mm and the tangent stiffness ($C_{O,tan}$) is 41 kN/mm, figure 2.20a. The initial stiffness of the frame panel connection due to rotation of the bolts ($C_{R, ini}$) is 118 kN/mm, figure 2.21. Slip of the anchor bars will only occur at the frame panel connections subjected to tensile forces, which occur at the infilled frame’s tension diagonal. The stiffness along the tension diagonal is described with an initial and tangent stiffness until pull out occurs, figure 2.20b. The initial anchorage slip stiffness ($C_{SL,ini}$) is 790 kN/mm and the tangent stiffness ($C_{SL,tan1}$) is 188 kN/mm. After the yield plateau the stiffness ($C_{SL,tan2}$) is decreasing with 74 kN/mm.

Secondly it is possible to combine the above mentioned spring stiffnesses to one overall spring stiffness ($C_l$) by taking their reciprocal values [4], figure 2.19b. Note that the spring stiffness of the frame panel connections subjected to compression forces only, exists of the spring stiffness that represents ovalisation of the bolt holes and bolt rotation, since anchor bar slip will not occur. The spring stiffness is therefore the added reciprocal values of bolt hole ovalisation ($C_0$) and bolt rotation ($C_R$). The frame panel connections subjected to tensile forces will exists of all three mentioned phenomena.

The gusset- and anchor plates are created with triangular plane stress six nodes quadrie elements, CT12M. The gusset plate is connected to the beam element through a rigid offset. This rigid offset represents the beam depth, just like the rigid offset which connects the beam to the column.
represents the column depth. The springs that represent anchorage slip, in the first model, are connected to anchor plate via a rigid edge.

This rigid edge represents the steel plate where the anchor bars are welded to, figure 2.19a. The springs are also connected to a rigid offset in the second model. This time the rigid edge is connected to the prefabricated concrete panel and represents the steel anchor plate which distributes the force at the concrete panel, figure 2.19b.

**Frame panel connection L**

FPCL has been represented by two node nonlinear longitudinal spring elements, type SP2TR [16]. The experimental model contains four bolts per corner of the concrete panel, two vertical and two horizontal. In the finite element model these four springs have been represented by two longitudinal springs ($C_L$), one vertical and one horizontal, figure 2.22.

The springs are only capable to support axial compressive forces. The spring stiffness is determined by connection tests which were carried out on the steel sections, used at the location of the connection and the structural bolts. The springs represent, shortening of the bolts, bending of the steel section’s flanges and also compression of the steel section’s flanges. There was also an initial force applied in the springs which embodies the prestress in the bolts which is caused by the torque controlled tightening of the structural bolts. Torque controlled tightening will ensure that the concrete panel remains at its position.

Figure 2.20 - Stiffnesses FPC3: a) Ovalisation bolt holes ($C_O$), b) Anchor bar slip ($C_{SL}$).
2.4.5 Verification of finite element models

From the four mentioned finite element models only the models with FPC1 & FPC2 will be compared with results gained from experimental investigations, since there are no full scale finite element models present with FPC3 and FPCL.

Both finite element analyses have been performed twice, once with elliptical and once with linear reduced tensile strengths [5]. All four results have been compared to the experimental analysis of the infilled frames, figure 2.23.

It can be seen that the finite element analyses describe the elastically behaviour of the infilled frames with an acceptable accuracy. Differences between experimentally and numerically obtained lateral stiffness are 6%, table 2.4. At the plastic region, the numerical obtained load deflection curves deviate from the experimental results with a maximum error of 17%, table 2.4.

There has also been carried out an additional finite element simulation with average input properties from the tensile and shear tests from FPC1 & FPC2, table 2.2. Using linear reduction for the tensile strengths yields in the strengths, \( F_{FPC1} = 239 \text{kN} \) and \( F_{FPC2} = 183 \text{kN} \). Applying these reduced tensile strengths in the finite element models results in reduction of the error at yield level, table 2.4 values in parentheses. The elastic laterall stiffness's do not improve.
Table 2.4 - Infilled frame properties.

<table>
<thead>
<tr>
<th>Test</th>
<th>Yielding level [kN]</th>
<th>Ultimate strength [kN]</th>
<th>Lateral stiffness [kN/mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Experiment</td>
<td>Numerical analysis</td>
<td>Difference %</td>
</tr>
<tr>
<td>FPC1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>345</td>
<td>293** (-15)</td>
<td>(-10)</td>
</tr>
<tr>
<td>FPC2</td>
<td></td>
<td>226** (-6)</td>
<td>(-1)</td>
</tr>
</tbody>
</table>

* elliptical reduction tensile strength of connection, ** linear reduction tensile strength of connection, (-) using averaged connection properties with linear reduction of tensile strength

Figure 2.23 - Numerical and experimental results of full scale tests: a) FPC1, b) FPC2.
3 Experimental investigation

3.1 Test specimens
Two full scale test specimens, specimen A & B, have been tested three times each, table 3.1. Per test specimen first the bare frame has been tested to determine its contribution to the stiffness of the infilled frame. The second test consisted of the infilled frame and at the third test the same infilled frame has been tested in opposite direction, to estimate its capacity after failure in the first tested direction. A test specimen consists of three main elements, a steel frame, a reinforced concrete panel and frame panel connection 3. Together these elements form a single story, single bay three by three meter infilled frame structure, figure 3.1 and appendix A.

3.1.1 Steel frame
Four HE180M steel sections (S235), two beams and two columns, have been used to construct one steel frame. These steel sections have been used in an earlier experimental research project [16], and have only been subjected to elastic deformations. However, some flanges of these sections were plastically deformed at locations of the used frame panel connections, figure 3.2. It is expected that these deformations will not have an adverse influence on the experiments, since the frame panel connections, used in this investigation, are located at mid flange of the steel beams.

Both beams are connected to the columns by eight M24 10.9 bolts, four at each end of the beam. The bolts are torque controlled tightened up to a specified torque of 400 Nm, to obtain an identical stiffness of each frame. A small adjustment had to be carried out to the beams, since a different type of frame panel connection will be investigated. Triangular gusset plates are welded to the beams near the frame corners for connection to the concrete panel, figure 3.3, appendix A2 and A3.

Between the flanges of both columns steel plates have been welded, near the beam sections, to stiffen the beam-column connection, figure 3.2. For connection with the test rig a 50 mm thick steel plate has been welded at the lower end of the left column. The right column does not have this steel base plate because at this position the test specimen has been connected in a different way, section 3.2.

Figure 3.1 - Infilled frame.  Figure 3.2 - Beam-column connection.
Table 3.1 – Test overview.

<table>
<thead>
<tr>
<th>Test</th>
<th>Note</th>
<th>Test</th>
<th>Note</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>Bare frame A</td>
<td>B1</td>
<td>Bare frame B</td>
</tr>
<tr>
<td>A2</td>
<td>Infilled frame A (reverse direction)</td>
<td>B2</td>
<td>Infilled frame B (reverse direction)</td>
</tr>
<tr>
<td>A3</td>
<td>Infilled frame A</td>
<td>B3</td>
<td>Infilled frame B</td>
</tr>
</tbody>
</table>

### 3.1.2 Concrete panel
C35/45 concrete is used to cast a 2760 x 2760 mm² panel with a thickness of 200 mm and a overall concrete coverage of 25 mm. The panel is reinforced with two Ø8-150 Feb500 meshes, one at each side of the panel. Along the panel edges, reinforcement hooks Ø8-150 Feb500 are placed, appendix A4, B1 & B6 for drawings and photos. The compressive strength of the concrete is determined at the same day a full scale experiment is carried out. These results are presented in appendix C1, just as the determination of the Young’s modulus of the concrete.

### 3.1.3 Frame panel connection
A frame panel connection consists of two steel plates and five reinforcement bars, figure 3.4a, appendices A5 and A6. The frame panel connections are connected to the beam sections near the frame corners through a gusset plate. Distance x in figure 3.4b is set to avoid contact, under compression loads, between anchor plate two and the weld of the gusset plate and contact between the gusset plate and the weld of anchor plate two. Contact between those two parts could occur when ovalisation of the bolt holes becomes extensive.

Anchor plate one is made of cold formed steel (S235JRC+C), which has the benefit that the rolling skin is absent and therefore no time consuming preparations are needed before the welding process can begin, since anchor plate two and five reinforcement bars, Ø16 Feb500, have to be welded to anchor plate one. The gusset plates are also made of cold formed steel. This type of steel is used for fabrication of the gusset plates since it has a higher tensile strength, compared to hot rolled steel which is used to fabricate anchor plate two. This will force the bolt holes in anchor plate two to deform before the bolt holes in the gusset plate will deform. The steel grade of both applied types of steel are determined from tension tests, table 3.2 and appendix C2.

During concrete casting of the panel, the frame panel connections are held in place by the steel frame which also supports the form boards. The frame panel connection is bolted to the gusset plates of the HE180M beams with two M24 10.9 bolts. Before each experiment these bolts will be torque controlled tightened up to a specified torque of 200 Nm, to obtain similar conditions at
Table 3.2 - Material properties of FPC3.

<p>| | | | | | | | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Anchors</td>
<td>Bars</td>
<td>Plate 2</td>
<td>Plate 1</td>
<td>Plate 1</td>
<td>Plate 1</td>
<td>Plate 1</td>
</tr>
<tr>
<td></td>
<td>FeB500</td>
<td>275x10</td>
<td>275x10</td>
<td>389x15</td>
<td>389x15</td>
<td>275x15</td>
<td>275x15</td>
</tr>
<tr>
<td>FPC3</td>
<td>M24 10.9</td>
<td>16</td>
<td>500*</td>
<td>314</td>
<td>515</td>
<td>470</td>
<td>564</td>
</tr>
</tbody>
</table>

* assumed

every frame panel connection. The gap between the concrete panel and the steel frame is, for a practical reason, 18 mm. This is the thickness of the form boards. An 18 mm gap offers enough space for ovalisation of the bolt holes. A small gap leads also to a stiffer behaviour of the infilled frame, because the frame panel connection is now located closer to the frame corners. In previous investigations [14, 15] the gap was wider to avoid contact between the beam sections and the concrete panel. Due to the relocated frame panel connection it is expected that the bending deformations of the steel beam sections, during this experiment, will be almost zero and therefore a wide gap would also be unnecessary.

3.2 Test rig

The full scale tests will be carried out in a specially designed test rig for experiments on infilled frames, figure 3.5. Two triangular frames are transversely linked through short steel sections at the test rig corners, figure 3.5a & 3.5b. The frames are assembled with HE300B sections. At the top left corner of the test rig a hydraulic jack is mounted, with a capacity of 2 MN and a stroke of 200 mm.

A test specimen will be positioned between the two triangular frames. In figure 3.5 the test specimen (only the steel frame is shown for clarity) is indicated with a blue color.

At the lower left corner of the test rig the test specimen will be supported only in vertical direction to simulate a roller support, figure 3.5c. Four steel M30 threaded rods are used to transfer vertical loads to the test rig and permit rotation and horizontal translation of the test specimen.

At the lower right corner of the test rig, the specimen is supported in a steel block or saddle, figure 3.5d. This support acts like a hinged connection and only allows the test specimen to rotate. Lateral and vertical translation is restricted due to the saddle and the loading direction of the hydraulic jack. The test specimen can only be loaded in one direction, due to restrictions of the connection between the hydraulic jack and the test specimen and the test rig supports.

3.3 Testing procedure

At the first test, on both specimens A & B, the stiffness of the bare frame will be determined. The bare frame will be preloaded up to 20 kN, to close initial gaps between the test rig and the specimen. After unloading the test specimen it will be loaded up to 80 kN. At this load the bare steel frame will only deform elastically to avoid any damage which could affect the behaviour during the full scale tests [16].

After the bare steel frame is tested, the concrete panel will be placed. Connecting of the concrete panel to the steel frame takes place in a horizontal position at floor level. At this position the weight of the concrete panel does not influence the initial prestress in the bolts of the frame panel connection, so that the prestress will be equal in every bolt of every frame panel connection.

R. Lieven
At the second test, of specimens A & B, the initial gaps between the test rig and specimen will be closed by a preload of 50 kN. After unloading of the test specimen the test can begin and the infilled frame will be loaded up to failure.

For the third test, the test specimen has to be turned around its vertical axis of symmetry. Otherwise it is not possible to test the specimen, due to limitations of the test rig. Since the steel columns of the test specimen are specially designed to fit in one way at the test rig, only the concrete panel and the beam sections will be turned around their vertical axis of symmetry. The beams have to turn around too, so that the connection between the panel and beams is comparable to the second test, due to antisymmetry of the frame panel connection, figure 3.4b.

The horizontal load on the test specimen is applied under controlled displacement, during all tests. The stroke of the hydraulic jack is controlled at 1 mm/min.
3.4 Measuring arrangement

The behaviour of the test specimens will be recorded by linear variable differential transformers (LVDT), strain gauges (s.g.), wire gauges (w.g.) and digital clock gauges (DCG), during the experiments, figure 3.6 & 3.7.

Two measuring arrangements are presented because the test specimen will be tested in two directions. Several measurement units will therefore be located at a different corner of the concrete panel when the concrete panel is turned around its vertical axis of symmetry. The measuring arrangement for the tests A1, A2, B1 and B2 are presented in figure 3.6. For the tests in opposite direction, test A3 and B3, the measuring arrangement is presented in figure 3.7. In those figures at each measurement instrument a plus and minus is placed, which indicates the direction of the displacement. The ranges of the applied measuring instruments are given in table 3.3.

Deformation of the steel frame will be recorded by digital clock gauges m-00 to m-04 and A1-14, (blue). These are fixed to standalone measuring frames and will measure in plane deformations of the steel frame both vertically and horizontally.

The displacement of the concrete panel in relation to the displacement of the steel frame is recorded by four LVDT's, A1-06 to A1-09, (red). At the concrete panel corners where tension loads will occur, anchorage slip of the frame panel connection will be recorded by one LVDT at each side of the concrete panel, A1-10 to A1-13, (magenta).

Deformation of the concrete panel, caused by compression and tension forces, is recorded by two wire gauges at each side of the panel, A1-02 to A1-04, (green).

On every beam and column two strain gauges are placed, at the middle of the flanges, so that the normal forces and moments in these sections can be determined, s.g. 7-00, 7-16, 8-09 to 8-11, 8-16 and 8-17, (orange). Furthermore 10 rosette gauges are applied at the two lower frame panel connections to determine stresses which are transferred from the diagonals of the concrete panel to the steel frames corners, (yellow).

The lateral load on the test specimen, generated by the hydraulic jack, will be recorded by a 2 MN load cell, (L.C.).

Table 3.3 – Applied measuring instruments.

<table>
<thead>
<tr>
<th>Instrument</th>
<th>Manufacturer</th>
<th>Model</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>DCG m-00, m-01 &amp; m-03</td>
<td>Mitutoyo</td>
<td>ID-F150</td>
<td>50 mm</td>
</tr>
<tr>
<td>DCG m-02 &amp; m-04</td>
<td>Mitutoyo</td>
<td>ID-C150B</td>
<td>50 mm</td>
</tr>
<tr>
<td>w.g. A1-02 &amp; A1-03</td>
<td>Celesco</td>
<td>PTIA-2-UP-10K-M6</td>
<td>25 mm</td>
</tr>
<tr>
<td>w.g. A1-04 &amp; A1-05</td>
<td>Celesco</td>
<td>PTIA-5-FR-10K-M6</td>
<td>50 mm</td>
</tr>
<tr>
<td>LVDT A1-06 to A1-13</td>
<td>Solartron</td>
<td>AX/5/S</td>
<td>10 mm</td>
</tr>
<tr>
<td>w.g. A1-14</td>
<td>ae sensors</td>
<td>WS10-250-R10K-L10-SBO-D8-SD4</td>
<td>125 mm</td>
</tr>
<tr>
<td>s.g. 7-00 &amp; s.g. 7-16</td>
<td>Tokyo Sokki Kenkyuyo Co</td>
<td>PFL-30-11</td>
<td>30 mm</td>
</tr>
<tr>
<td>s.g. 8-08 &amp; s.g. 8-09</td>
<td>Tokyo Sokki Kenkyuyo Co</td>
<td>PFL-30-11</td>
<td>30 mm</td>
</tr>
<tr>
<td>s.g. 8-10 &amp; s.g. 8-11</td>
<td>Tokyo Sokki Kenkyuyo Co</td>
<td>PFL-30-11</td>
<td>30 mm</td>
</tr>
<tr>
<td>s.g. 8-16 &amp; s.g. 8-17</td>
<td>Tokyo Sokki Kenkyuyo Co</td>
<td>PFL-30-11</td>
<td>30 mm</td>
</tr>
<tr>
<td>701 to 715</td>
<td>Tokyo Sokki Kenkyuyo Co</td>
<td>FRA-10-11</td>
<td>10 mm</td>
</tr>
<tr>
<td>717 to 723</td>
<td>Tokyo Sokki Kenkyuyo Co</td>
<td>FRA-10-11</td>
<td>10 mm</td>
</tr>
<tr>
<td>800 to 807</td>
<td>Tokyo Sokki Kenkyuyo Co</td>
<td>FRA-10-11</td>
<td>10 mm</td>
</tr>
</tbody>
</table>
Figure 3.6 – Measuring arrangement tests A1, A2, B1 & B2.
Figure 3.7 – Measuring arrangement tests A3 & B3.
3.5 Observations and results

The experimental test observations and experimental test results will be presented, per specimen, in the following sections.

3.5.1 Specimen A

Bare frame

The stiffness of the bare frame, during test A1, increases until a lateral load of approximately 15 kN, figure 3.8. Until this load initial gaps between the test specimen and test rig are closed. Thereafter two linear branches are determined, the initial stiffness and tangent stiffness, figure 3.9. The initial stiffness \( (k_{ni;A1}) \) is 4.1 kN/mm and the tangent stiffness \( (k_{tan;A1}) \) is 2.3 kN/mm.

Infilled frame

Figure 3.10 represents load deflection curves of tests A2 & A3. Corrections have been made for rigid body displacement and rotation, which occur due to sliding of the test specimen and deformation of the test rig. The displacement measured at the upper right corner (m-02) is rectified by subtracting the displacement due to sliding of the test specimen into the right support (m-03) and by subtracting the lengthening of the threaded rods at the left support (m-04) which causes rotation of the test specimen around its right support.

The behaviour of specimen A, during test A2, can be considered linear up to around 584 kN with an initial stiffness \( (k_{ni;A2}) \) of 34.8 kN/mm, figure 3.11. At this load the first crack in the tension diagonal of the concrete panel occurred near the ends of the frame panel connection’s anchor bars, figure 3.10 & appendix B4 (photos). After a load drop of 22 kN the lateral load increases to 585 kN before the second crack, at the opposite corner, in the tension diagonal occurs, figure 3.10 & appendix B4. This crack is also located at the end of the anchor bars and caused a load drop of 21 kN. From this point the lateral load increases to 644 kN with an accompanying deflection of 28.9 mm. After reaching the ultimate strength of the infilled frame, the frame panel connection in corner four failed in an unexpected way. Anchor plate two and the gusset plate deformed out of their plane, figure 3.12. This results in a decreasing lateral load and increasing lateral displacements of the infilled frame.

During test A3, the load deflection behaviour of specimen A is similar to the behaviour during test A2. However, the initial stiffness \( (k_{ni;A3}) \) is now 23.7 % smaller at 23.2 kN/mm, due to cracks in the concrete panel and increased deformations in the frame panel connections as a result of test A2, figure 3.11. Cracks in the tension diagonal at the ends of the anchor bars of the

---

Figure 3.8 – Deflection of bare frame A.  
Figure 3.9 – Stiffness of bare frame A.
frame panel connections occur at 483 kN and 532 kN, figure 3.10 & appendix B5 (photos). The load drops are 24 and 17 kN respectively. Similarly to the first experiment, out of plane deformation of the frame panel connection at corner three occurred. The ultimate load at the specimen during test A3 is 601 kN. This is a reduction of 6.7% compared to test A2. The deflection of the frame at the ultimate load is 40.6 mm which is an increase of 40.4% compared to test A2.

Pull out of the anchor bars during tests A2 & A3 is negligible small, figure 3.15. These graphs are representing average values obtained from two LVDT’s per frame panel connection, figure 3.6 & 3.7 (magenta). To illustrate the necessity of the average value, corner three of specimen A, test A2, is taken as example. The difference in pull out at each side of the panel can be seen in figure 3.13 LVDT A1-11 indicates that there occurs a negative pull out, which indicates a slight rotation of anchor plate one due to the asymmetric load, figure 3.14. During test A2 the frame panel connection in corner four was subjected to compression forces which resulted in plastic out of plane deformation of anchor plate two and the gusset plate, figure 3.12. This causes different pull out behaviour at test A3 compared to the undeformed frame panel connections, figure 3.13 grey line.

The concrete panel deformed significantly over its tension diagonal due to cracking near the ends frame panel connection’s anchor bars, figure 3.16. It can be seen that before cracking of the concrete panel the deformation is almost zero. Due to cracking of the concrete panel forces in the compression diagonal increase. This results in out of plane deformation of the frame panel connection in corner four. It can be seen that there is almost no shortening of the compression diagonal during test A2 due to this out of plane deformation occurs.
The compression diagonal at test A3 is shortened significantly compared to the compression diagonal during test A2. The reason for this shortening is that the compression diagonal in test A3 was the tension diagonal in test A2, which extended significantly. At test A3 the cracks in the concrete panel are compressed and the compression diagonal of the concrete panel shortens. However, the shortening of the compression diagonal in test A3 is not as much as the lengthening was of the same diagonal in test A2, figure 3.16 blue line extension of 8 mm and grey line shortening of 3.5 mm.

In figure 3.17a the displacement of the concrete panel in relation to the steel frame is presented for test A2. During this test LVDT A1-07 and A1-09 were incorrectly placed. LVDT A1-09 is replaced during the test. LVDT A1-07 could not be replaced due to its unattainable position (during the experiment). It can be seen that the results of LVDT A1-09 are similar to the results of LVDT A1-08, only the displacements are shifted negatively, due to the replacement. The results indicate that the steel sections of the frame are coming closer towards the corners of the concrete panel at the compression diagonal. At the corners in the tension diagonal the steel sections are turning away from the concrete panel.

During test A3, LVDT A1-06 fell off the test specimen and was replaced during the test, figure 3.17b. The graph from A1-06 is therefore shifted positively. At the end of the experiment LVDT A1-08 & A1-09 fell also of the test specimen due to out of plane displacement of the concrete panel caused by out of plane deformation of the frame panel connection in corner three. The results indicate the same sort of displacement of the concrete panel as during test A2.

Figure 3.13 - Anchor bar pull out corner 3 test A2.
Figure 3.14 - Rotation anchor plate 1.
Figure 3.15 - Average pull out of anchor bars.
Figure 3.16 - Deformation concrete panels.
The normal forces and bending moments in the steel sections of specimen A are presented in figure 3.18 & 3.19 for test A2 and figure 3.20 & 3.21 for test A3. The normal forces and moments are calculated by the determined strains in the steel sections which are measured with two strain gauges per section, figure 3.6 & 3.7 (orange). Strain due to normal force plus strain due to a moment is determined, figure 3.22a (principle). The average value of these strains is the strain caused by the normal force in the section, figure 3.22b. The strain due to normal force can be subtracted from the total strains to gain the strains caused by the bending moment on the section, figure 3.22c. For conversion from strain to normal force and moment a Young's modulus ($E_s$) of 210000 N/mm$^2$, a sectional area ($A$) of 11325 mm$^2$ and a moment of resistance ($W_y$) of 748300 mm$^3$ are used. These are nominal values belonging to HE180M sections.
After test A2, the ovalisation of the bolt holes is measured, table 3.4 and appendix E. The amount of ovalisation of the bolt holes in the anchor plates is less than expected, due to the unexpected failure mechanism of the frame panel connection in corner four. At this frame panel connection the largest ovalisation occurred, figure 3.12b. At the other three frame panel connections ovalisation, with half the magnitude of that of corner four, is measured. As expected, the ovalisation in the gusset plates is small compared to the ovalisation of the bolt holes in the anchor plates, table 3.4.

After test A3, the same order of magnitude of ovalisation is found compared to test A2, table 3.4 and appendix E. (Note, the frame panel connections in the tension diagonal at test A2 turn into the frame panel connections in the compression diagonal of test A3, figure 3.6 & 3.7. For example: corner one at test A2 should be compared with corner two at test A3)

### 3.5.2 Specimen B

**Adjusted frame panel connection**

The frame panel connections, used at specimen B, are slightly adapted compared to the frame panel connections used at specimen A. Adjustments had to be made to avoid out of plane deformation of anchor plate two. To resist the out of plane deformation of anchor plate two a steel plate (stiffener) is welded perpendicular to anchor plate one and anchor plate two, figure 3.23. Due to this stiffener a larger degree of ovalisation of the bolt holes is expected, since out of plane deformation is prevented. The stiffener is located at the position of the middle rosette gauge at anchor plate two. Therefore the strain gauges 704, 705, 706, 720, 721 and 722 are left out at tests B2 & B3.

Due to the adapted frame panel connection a higher ultimate strength and a larger lateral deflection of the test specimen is expected. The lateral deflection is recorded by two digital clock gauges (m-01 & m-02). Both have a reach of 50 mm. This reach will not be enough to measure

### Table 3.4 - Ovalisation bolt holes after tests A2 & A3 (average per corner).

<table>
<thead>
<tr>
<th>Corner</th>
<th>Anchor plate 2</th>
<th>Gusset plate</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test A2</td>
<td>Test A3</td>
</tr>
<tr>
<td>1</td>
<td>1.7</td>
<td>1.9</td>
</tr>
<tr>
<td>2</td>
<td>1.4</td>
<td>1.7</td>
</tr>
<tr>
<td>3</td>
<td>1.4</td>
<td>3.0</td>
</tr>
<tr>
<td>4</td>
<td>2.8</td>
<td>1.2</td>
</tr>
</tbody>
</table>
the lateral deflection during tests B2 & B3. DGC m-02 is therefore replaced by a wire gauge (A1-14) with a reach of 125 mm, figure 3.6 & 3.7. DCG m-01 will be replaced during the experiment when it becomes out of reach. This measurement unit is accessible during the experiments and will only be used as backup measurement in case DCG m-02 or w.g. A1-14 will breakdown.

**Bare frame**

Initial gaps between the bare frame and test rig are closed until a lateral load of approximately 15 kN during test B1, figure 3.24. After this load two linear branches are determined, figure 3.25. The initial stiffness ($k_{ini,B1}$) is 4.5 kN/mm and the tangent stiffness ($k_{tan,B1}$) is 2.2 kN/mm.

**Infilled frame**

The load deflection curves of specimen B at tests B2 and B3 are also corrected for rigid body displacement and rigid body rotation as mentioned in section 3.5.1, figure 3.26.

During test B2, specimen B has a linear behaviour up to 536 kN with an initial stiffness ($k_{ini,B2}$) of 40.7 kN/mm, figure 3.27. The linear branch ends when the first crack at the concrete panel occurred, appendix B9 (photos). The accompanying load drop is 23 kN. Hereafter the load is increasing to 596 kN before the next crack in the concrete panel occurs. This time the load drop is 24 kN. After the second crack the load increases non linearly to 762 kN, with an accompanying deflection of 35.3 mm. At this ultimate load a yield plateau occurs. The yield point elongation ends when the load drops to 725 kN. This load drop is caused by rotation and shear deformation of the bolts in the compressed corners, figure 3.28. Rotation of the bolts occurred due to local deformation of anchor plate two. The test was stopped when contact occurred between anchor plate one and the weld of the gusset plate at corner four.
The load deflection behaviour of specimen B, during test B3, is similar to that of test B2. The initial stiffness \( K_{\text{init,B3}} \) is 21.3 kN/m, which is a decrease of 47.7%, compared to specimen B2. Cracks in the tension diagonal of the concrete panel occurred at 533 kN and 587 kN, figure 3.27 & appendix B10 (photos). The load drop after the first and second crack is 29 kN and 25 kN respectively. The ultimate strength is 690 kN, at a deflection of 53.2 mm. Compared to test B2 the strength has decreased 9.4% and the accompanying deformation increased 50.7%. There does no yield plateau occur, during test B3, but shear deformation of the bolts and local deformation of anchor plate two occurs at the compressed corners, figure 3.29. The test was stopped after contact between anchor plate one and the weld of the gusset plate at corner three.

The pull out behaviour of the anchor bars during test B2 is similar to the pull out behaviour of test B3, figure 3.30. Till a lateral load of 640 kN, at specimen B2 & B3, the pull out behaviour is even quite comparable to the pull out behaviour of the anchor bars at test A2 & A3. After 640 kN the frame panel connection’s anchor bars, in the upper corner of the tension diagonal, start to slip significantly compared to the anchor bars in the lower corner of the tension diagonal. It is assumed that this difference is caused by the weight of the concrete panel.

The difference of pull out behaviour of the anchor bars between specimens A & B can be clarified by the fact that the ultimate load, and therefore the lateral deflection, during tests on specimen B are larger than on specimen A.

The lengthening and shortening behaviour of the tension and compression diagonals, during tests B2 & B3 are similar to tests A2 & A3, figure 3.16 & 3.31. However, the amount of lengthening and shortening differs. At test B2 the compression diagonal did almost not shorten, while the tension diagonal lengthened significantly. Shortening in the compression diagonal is absorbed by
ovalisation of the bolt holes, local deformation of the anchor plates and shear deformation of the bolts. Due to extension of the tension diagonal at test B2, the compression diagonal at test B3 shortens considerably compared to the compression diagonal at test B2, figure 3.31. Measurement unit A1-05 fell of the test specimen at the end of the experiment, which results in the incomplete graph.

The displacement behaviour of the concrete panel in relation to the steel frame for test B2 is analogous to the tests A2 & A3, figure 3.17 & 3.32a. LVDT A1-06, A1-08 and A1-09 became out of reach during the experiment. These results indicate that the steel sections of the frame are coming closer towards the corners of the concrete panel at the compression diagonal. At the corners in the tension diagonal the steel sections are turning away from the concrete panel.

During test B3, the measurement units A1-06, A1-08 and A1-09 had to be replaced several times during the experiment, figure 3.32b (number per part of graph). The graphs are therefore shifted positively for A1-06 and negatively for A1-08 and A1-09. The measurement units hat to be replaced due to the large deformed specimen, caused by test B2, which results in even larger deformations at test B3. These results indicate the same sort of displacement as during the test on specimen B2.

The normal force and moment per steel sections of specimen B are presented in figure 3.33 & 3.34 for test B2 and figure 3.35 & 3.36 for test B3. The normal forces and moments are calculated as explained in section 3.5.1. The behaviour of normal forces and moments that are determined for the sections in test B2 & B3 are similar to those determined for test A2 & A3.

![Figure 3.30 - Average pull out of anchor bars.](image1)

![Figure 3.31 - Deformation concrete panels.](image2)

![Figure 3.32 - Displacement concrete panel in relation to steel frame: a) Test B2, b) Test B3.](image3)
After test B2, the ovalisation of the bolt holes is measured, table 3.5 and appendix E. The amount of ovalisation of the bolt holes in the anchor plates is larger compared to the ovalisation of the bolt holes at tests A2 & A3. This can be ascribed to the applied stiffeners at the frame panel connections, which prevented out of plane deformation of anchor plate two.

The ovalisation of the bolt holes in the gusset plates also increased, compared to the tests A2 & A3, table 3.5. After test B3 the order of magnitude of ovalisation is considerable larger compared to test B2, table 3.5 and appendix E. The difference is between 10% and 40%.

### 3.6 Comparison test results

When comparing the experimental test results from both bare frames, it can be seen that their load deflection behaviour is similar, table 3.6. However, the elastic strength and tangent stiffness of bare frame B, test B1, are reached earlier, figure 3.37.

When comparing the experimental full scale tests, it becomes clear that the initial stiffnesses for test specimens A2 & B2 and for tests A3 & B3 are quite similar, table 3.6 & figure 3.38. At all four experimental tests closure of the gaps between the test specimen and test rig occurs till a lateral load of approximately 225 kN.

The elastic strength of the four experimental tests is also comparable. Larger differences are found for the ultimate strength, since at test A2 & A3 premature failure of the frame panel connections occurred. However, the beginning of the plastic phase, after cracking of the concrete panel’s tension diagonal, is similar to those of test B2 & B3.
Table 3.5 - Ovalisation bolt holes after tests B2 & B3 (average per corner).

<table>
<thead>
<tr>
<th>Corner</th>
<th>Anchor plate 2</th>
<th>Gusset plate</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Test B2</td>
<td>Test B3</td>
</tr>
<tr>
<td>1</td>
<td>5.3</td>
<td>2.7</td>
</tr>
<tr>
<td>2</td>
<td>0.9</td>
<td>6.6</td>
</tr>
<tr>
<td>3</td>
<td>1.4</td>
<td>6.4</td>
</tr>
<tr>
<td>4</td>
<td>5.9</td>
<td>3.4</td>
</tr>
</tbody>
</table>

Table 3.6 - Results overview of the experimental tests.

<table>
<thead>
<tr>
<th>Test</th>
<th>Elastic strength [kN]</th>
<th>Ultimate strength [kN]</th>
<th>Initial stiffness [kN/mm]</th>
<th>Tangent stiffness [kN/mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>40 (100 %)</td>
<td>-</td>
<td>4.1 (91 %)</td>
<td>2.3 (100 %)</td>
</tr>
<tr>
<td>B1</td>
<td>29 (73 %)</td>
<td>-</td>
<td>4.5 (100 %)</td>
<td>2.2 (96 %)</td>
</tr>
<tr>
<td>A2</td>
<td>584 (100 %)</td>
<td>644 (85 %)</td>
<td>34.8 (86 %)</td>
<td>-</td>
</tr>
<tr>
<td>B2</td>
<td>536 (92 %)</td>
<td>762 (100 %)</td>
<td>40.7 (100 %)</td>
<td>-</td>
</tr>
<tr>
<td>A3</td>
<td>483 (83 %)</td>
<td>601 (79 %)</td>
<td>23.7 (58 %)</td>
<td>-</td>
</tr>
<tr>
<td>B3</td>
<td>533 (91 %)</td>
<td>690 (91 %)</td>
<td>21.3 (52 %)</td>
<td>-</td>
</tr>
</tbody>
</table>

Figure 3.37 - Deflections of bare frames.  
Figure 3.38 - Deflection of infilled frames.

3.7 Chapter conclusions

Six experimental tests have been carried out to provide insight into the racking shear behaviour of an infilled frame structure with FPC3 and to provide data for a nonlinear finite element model. The following conclusions are drawn:

- The preload of 50 kN at the full scale test specimen is not enough to close initial gaps and slack in the specimen, since the linear branch starts at 225 kN for the undamaged test specimens at test A2 & B2. It could be stated that preloading is unnecessary, because the load deflection behaviour does not change when the test specimens are loaded again after unloading.
- After closure of initial gaps, between test specimen and test rig, and closure of slack in the specimen’s connections a linear behaviour occurs until the first concrete cracks.
- After cracking of the concrete panel in the tension diagonal at tests A2 & A3 the frame panel connections in the compression diagonal transfer the lateral load to the supports of the test specimen, which results in relatively quick failure of the frame panel connections (out of plane deformation of anchor plate two) without much ovalisation of the bolt holes.
- The bolts in the modified frame panel connection, at tests B2 & B3, are the weakest link since they deform plastically and anchor plate two remains straight.
- The improved frame panel connection results in a different failure mechanism of the infilled frame and a higher ultimate load.
- The mean load drop after a crack in the tension diagonal of the concrete panel is 23 kN, with a maximum deviance of 6 kN.
- Pull out of the anchor bars does not occur till the ultimate load is reached. Instead the concrete panel cracks at the end of the anchor bars in the tension diagonal (extension of tension diagonal).
- The improved frame panel connections do not have a significantly influence on the initial stiffness's of the infilled frames.
4 Finite element investigation

4.1 Finite element modeling
Two finite element models have been created to analyze the racking shear behaviour of the infilled frames that have been experimentally tested, appendix F1. The first model is a representation of the experimentally tested bare frames A1 and B1, figure 4.1a & appendix F2. With the results of the first model the contribution of the bare frame's stiffness to the total stiffness of the infilled frame will be determined. The second finite element model represents the experimentally tested infilled frames A2 and B2, figure 4.1b & appendix F3. For both finite element models characteristics of experimental tests A(1, 2) or B(1, 2) can be set, such that the desired experimental test will be simulated. The construction of the finite element models is represented in the following sections: steel frame, concrete panel and frame panel connection. In these sections the applied elements, element geometry and material characteristics are discussed.

4.2 Steel frame

4.2.1 Elements
BEAM3 elements have been used to model the beams and columns of the steel frame. This two node uniaxial element has tension, compression and bending capabilities [1]. A node has three degrees of freedom, translation in the nodal x and y direction and rotation about the nodal z-axis.

COMBIN39 elements are used to model the torsional springs that represent the bolted connections between the beams and columns of the steel frame. The applied torsional option is a purely rotational element with three degrees of freedom at each node [1]. In the finite element model only the rotations about the nodal z-axis are admitted. Translation in x and y directions will be restricted by constrains.

Figure 4.1 - Finite element model: a) Bare frame, b) Infilled frame.
4.2.2 Geometry
For the beams and columns nominal values of HE180M sections are used: sectional area (A) 11325 mm$^2$, sectional height (h) 200 mm and moment of inertia (I_y) 7483000 mm$^4$. The beams and columns are divided in thirty elements each. This number of elements is sufficient enough to provide a reliable display of moments. The bolted connection between the columns and beams is modeled as described in section 2.4.2. The applied nominal values of the frame’s geometry are represented in appendix A2.

4.2.3 Material properties
Material properties that are used for the steel sections and the torsional springs are: Young’s modulus ($E_5$) 210000 N/mm$^2$ and Poisson’s ratio ($\nu_5$) 0.3. These characteristics are acquired from earlier investigations [16].

4.2.4 Torsional spring characteristics
The torsional spring characteristics ($C_T$) that represent the bolted connection between the beams and columns of the bare steel frames are chosen such that the load deflection behaviour of the finite element model will describe the experimental obtained load-deflection curves, figure 4.2. This results in the values as shown in table 4.1 and figure 4.3. Depending on the desired simulation, test A2 or B2, the spring characteristic can be set in the finite element model.

4.3 Concrete panel

4.3.1 Elements
PLANE183 elements are used to model the concrete panel. This higher order 2D element has 8 nodes at quadric shaped elements and six nodes at triangular shaped elements [1]. The key option for plane stress, with a constant thickness input, is enabled. The element has two degrees of freedom at each node. These are translation in the nodal x and y direction.

4.3.2 Geometry
Quadric and triangular elements are used to model the concrete panel, figure 4.4. The triangular elements are applied where the concrete panel connects to anchor plate two. This will smoothen the transition between the mesh of the concrete panel and the mesh of anchor plate two. The dimensions of the concrete panel are according to nominal values, appendix A4.

4.3.3 Material properties
The material properties of the applied concrete have been experimentally determined, appendix C1. In the finite element model, two Young’s moduli are applied for the concrete panel. One for test specimen A2 ($E_{C;A2}$), which is 32131 N/mm$^2$ and one for experimental test B2 ($E_{C;B2}$) which is 31376 N/mm$^2$. The Poisson’s ratio ($\nu_C$) for both options is 0.2.

<table>
<thead>
<tr>
<th>Test</th>
<th>Stiffness</th>
<th>Moment</th>
<th>Angular rotation</th>
<th>Experimentally obtained stiffness</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>[Nnm]</td>
<td>[rad]</td>
<td>($C_T$) [Nmm/rad]</td>
</tr>
<tr>
<td>A1</td>
<td>$k_{int;A1}$</td>
<td>2.789·10$^7$</td>
<td>1.420·10$^{-3}$</td>
<td>1.964·10$^{10}$</td>
</tr>
<tr>
<td></td>
<td>$k_{tan;A1}$</td>
<td>6.058·10$^7$</td>
<td>6.800·10$^{-3}$</td>
<td>8.909·10$^9$</td>
</tr>
<tr>
<td>B1</td>
<td>$k_{int;B1}$</td>
<td>2.034·10$^7$</td>
<td>1.000·10$^{-3}$</td>
<td>2.034·10$^{10}$</td>
</tr>
<tr>
<td></td>
<td>$k_{tan;B1}$</td>
<td>5.827·10$^7$</td>
<td>7.850·10$^{-3}$</td>
<td>7.423·10$^9$</td>
</tr>
</tbody>
</table>
4.4 Frame panel connection

4.4.1 Elements
The steel plates of FPC3 are modeled with PLANE183 plane stress elements. These elements are used to model the offset, the gusset plate and anchor plate two, figure 4.4.

COMBIN39 elements are used to model springs that represent the stiffness of the frame panel connection. The longitudinal option is applied. The element has two degrees of freedom at each node. Bending and torsion are not considered.

4.4.2 Geometry
Triangular elements are used to model the gusset plate and anchor plate two of the frame panel connections, figure 4.4. These elements have a more suitable fit in the triangular shape compared to quadric elements. The gusset plate is connected to an offset, which represents the beams depth, and is modeled with quadric elements. The thickness of the gusset plate’s offset is 15 mm. All other dimensions of the frame panel connection are specified in appendices A5 & A6.

Anchor plate two is directly connected to the concrete panel to prevent deformations between those two components, such as happened during the experimental tests due to slip of the anchor bars. The force transmission from anchor plate two into the concrete panel, due to the different thicknesses of anchor plate two compared to the concrete panel, is not modeled according to

Figure 4.4 - Mesh finite element model: a) Simplification infilled frame, b) Frame panel connection.
reality. This has however no significant influence on the overall results of the finite element analysis.

The connection between the gusset plate and anchor plate two is modeled with four springs, figure 4.4b. These springs represent the stiffnesses due to bolt hole ovalisation, slip of the anchor bars and bolt rotation. These phenomena are not specifically represented in the finite element model. In plane deformation of the gusset plate and anchor plate two is considered by the modeled plates and is therefore not implemented in the spring characteristics. Two springs are modeled per bolt location and attached to the PLANE183 elements. The longitudinal spring \( C_p \), represents bolt hole ovalisation, slip of the anchor bars and bolt rotation. Perpendicular to this spring another longitudinal spring \( C_s \) is modeled, which will prevent the concrete panel from rigid body rotation. This spring represents bolt hole ovalisation and bolt rotation, since slip of the anchor bars does not occur in that direction.

4.4.3 Longitudinal spring characteristics
The longitudinal springs in the finite element model will be determined in the following sections, spring stiffness due to bolt hole ovalisation in anchor plate two, spring stiffness due to bolt hole ovalisation in the gusset plate, spring stiffness due to anchor bar slip and spring stiffness due to bolt rotation. Experimental results from the connection tests [4] and material properties of the applied steel and concrete in the full scale tests will be used to obtain these spring characteristics.

Spring stiffness due to bolt hole ovalisation in anchor plate two
The spring stiffness due to the ovalisation behaviour of the bolt holes in anchor plate two \( C_{O;ap2} \) has been gained from connection tests [4], figure 2.20a. However, the applied anchor plates in the full scale tests were made of a higher steel grade compared to the applied anchor plates in the connection tests, figure 4.5a. Therefore the ovalisation behaviour must be adapted. The ratio between the yield stress of the applied steel of anchor plate two in the connection tests \( f_{y;ap2;ct} \) and the yield stress of the steel in the full scale test \( f_{y;ap2;rs} \) is used to increase the original elastic strength of the bolt hole ovalisation \( F_{e;ap2;ct} \) to an assumed elastic strength of anchor plate two \( F_{e;ap2;fs} \), equation 4.1 & figure 4.6a.

\[
F_{e;ap2;fs} = \frac{f_{y;ap2;fs}}{f_{y;ap2;ct}} \cdot F_{e;ap2;ct} = \frac{314}{266} \cdot 290 = 342 \text{ kN} \quad (4.1)
\]

It is assumed that the plastic behaviour of the adjusted spring characteristic is comparable with the plastic behaviour of the original spring characteristic. This assumption is based on corresponding plastic phases of both tensile tests, figure 4.5a. Since there is no measured ultimate load available of the bolt hole ovalisation in anchor plate two, the tangent stiffness of anchor plate two will end at an assumed load of 540 kN, figure 4.6a. This load is theoretical shear resistance of two bolts \( F_{v;Rd} \), equation 4.2.

\[
F_{v;Rd} = 2 \cdot \alpha_v \cdot f_{ub} \cdot A_{cb} \cdot 10^{-3} = 2 \cdot 0.6 \cdot 1000 \cdot 452 \cdot 10^{-3} = 540 \text{ kN} \quad (4.2)
\]

Spring stiffness due to bolt hole ovalisation in the gusset plate
The stiffness due to bolt hole ovalisation in the gusset plate \( C_{O;gp} \) is not measured. It is assumed that the stiffness of the gusset plate increases by 33% compared to the stiffness of anchor plate two, since the thickness of the gusset plate is 50% thicker. It is also assumed that the elastic strength of the gusset plate \( F_{e;gp} \) will increase proportional compared to the elastic strength of
Finite element investigation

anchor plate two \( (F_{el;ap2;fs}) \), as the elastic stress of the gusset plate \( (f_{y;gp}) \) increases compared to the elastic stress of anchor plate two \( (f_{y;ap2;fs}) \), equation 4.3, figure 4.5b & 4.6a.

\[
F_{el;gp} = f_{y;gp} : F_{el;ap2;fs} = \frac{470}{314} \cdot 342 = 512 \text{kN} \quad (4.3)
\]

The plastic behaviour of the ovalisation in the gusset plate is omitted, since this behaviour is not known and the elastic strength of 512 kN is within 5% of the theoretical shear resistance load of the bolts, which is 540 kN, that can be transferred through the frame panel connection.

**Total spring stiffness due to ovalisation**

Adding the stiffnesses due to bolt hole ovalisation in anchor plate two \( (C_{O;ap2}) \) to the gusset plate’s ovalisation stiffness \( (C_{O;gp}) \), results in the total bolt hole ovalisation stiffness \( (C_{O}) \), figure 4.6b. The sum of reciprocal values is taken, since both springs are in series, equation 4.4.

\[
C_{O} = \left( \frac{1}{C_{O;ap2}} + \frac{1}{C_{O;gp}} \right)^{-1} \quad (4.4)
\]

**Spring stiffness due to anchor bar slip**

The spring stiffness due to slip of the anchor bars \( (C_{O1}) \) is determined from the connection tests [4], figure 2.20b. The concrete panels applied in the connection tests, differs compared to the panels used in the full scale tests in terms of strength and stiffness. Therefore the curves of

![Figure 4.5 - Tensile tests: a) Connection test and full scale test, b) Gusset plate and anchor plate 2.](image)

![Figure 4.6* - Bolt hole ovalisation: a) Adjusted stiffnesses, b) Connection stiffness \( (C_{O}) \).](image)
the bar slip behaviour are adapted. This is done by using an equation which has been used for
determination of the anchor bar length, equation 4.5 & appendix D2. It expresses the allowable
tensile force as a function of the concrete panel dimensions and strength.

\[
\frac{I_d}{d_b} = \frac{1.85 \cdot \left( \frac{F_{ten}}{A_b} \right)}{\left( \frac{c + \left( \frac{b_w}{d_b} \right)}{d_b} \right) \sqrt{f_{ck}}}
\]

(4.5)

With this equation the ultimate tensile force \( F_{ten} \) can be calculated, that can be resisted by the
anchor bars. This tensile force is known for the anchor bars from the connection tests \( F_{ten;ct} \).
The equation can also be used for determination of the tensile force in the full scale model
\( F_{ten;fs} \). Since the anchor bar length \( l_d \) and diameter \( d_b \) are the same in both, the connection
test and the full scale tests, it follows that:

\[
\frac{1.85 \cdot \left( \frac{F_{ten;ct}}{A_{b;ct}} \right)}{\left( \frac{c_{ct} + \left( \frac{b_w}{d_{b;ct}} \right)}{d_{b;ct}} \right) \sqrt{f_{ck;ct}}} = \frac{1.85 \cdot \left( \frac{F_{ten;fs}}{A_{b;fs}} \right)}{\left( \frac{c_{fs} + \left( \frac{b_w}{d_{b;fs}} \right)}{d_{b;fs}} \right) \sqrt{f_{ck;fs}}}
\]

Substituting of the equation:

\[
\frac{1.85 \cdot \left( \frac{350 \cdot 10^3}{1005} \right)}{\left( \frac{80 + \left( \frac{\%}{16} \right)}{16} \right) \sqrt{58}} = \frac{1.85 \cdot \left( \frac{F_{ten;fs}}{1005} \right)}{\left( \frac{92 + \left( \frac{\%}{16} \right)}{16} \right) \sqrt{74}} \Rightarrow F_{ten;fs} = 441 \text{ kN}
\]

(4.6)

(4.7)

This tension force \( F_{ten;fs} \) is an increase of 26% compared to the maximum force of the bar slip
in the connection tests, which is 350 kN. Therefore the forces at the anchor bar slip curve should
be increased with 26%, figure 4.7. The influence of the different Young's moduli of the
connection tests, \( E_{c;ct} = 28953 \text{ N/mm}^2 \) [4], compared to the full scale tests, \( E_{c;fs} = 31253
\text{ N/mm}^2 \), has been ignored, since the 7% difference is small.

To confirm the assumed increase of strength of the anchor bars, the calculated force in the
tension diagonal \( F_{ten;fs} \) can be checked with use of the experimental full scale test results
(results from experimental test B2 will be used, since premature failure of the frame panel
connections at test A2 occurred). At a lateral load of 762 kN, which is the maximum lateral load
on test specimen B2, the anchor bars start to slip significantly, figure 3.26 & 3.30. When the
maximum lateral load on the infilled frame \( F_{lr} \) has been reached, the deflection is 35 mm. At a
deflection of 35 mm, the frame's resistance \( F_{br} \) is also known, figure 3.24. The maximum force
that can be transferred through the compression diagonal \( F_{dia;com} \) depends on the shear capacity
of both bolts \( F_{v;rad} \) in the frame panel connection, equation 4.2. With these three know values
the force in the tension diagonal \( F_{dia;ten} \) of the concrete panel can be derived, equation 4.8.

\[
F_{dia;ten} = \left( F_{lr} - F_{ef} - \frac{F_{dia;com}}{\sqrt{2}} \right) \sqrt{2} = \left( 762 - 80 - \frac{540}{\sqrt{2}} \right) \cdot \sqrt{2} = 424 \text{ kN}
\]

(4.8)
The converted tensile force of the connection tests \(F_{ten;rs}\), 441 kN, is within a 5% margin of the tensile force determined from the full scale test results \(F_{dia;ten}\), figure 4.7. Therefore the converted curves from the connection test results are reasonable to use.

*Spring stiffness due to bolt rotation*

Due to asymmetric forces in the frame panel connections the gusset plate and anchor plate two will bend which leads to rotation of the bolt and causes extension of the frame panel connection figure 4.9a. From the connection tests it is known what the amount of bolt rotation is, figure 2.21b.

Since the gusset plate used in the full scale test is thinner compared to the gusset plate used in the connection tests, the bending resistance will be lower which leads to a larger bolt rotation and a decrease of stiffness. A thinner plate leads also to a smaller moment arm and thus a smaller moment which decreases bolt rotation and increases the stiffness, figure 4.9b. However this increase does not compensate the decrease due to a smaller bending resistance.

The stiffness due to rotation of the bolts also depends on the amount of slack between the edge of the bolt holes and the bolt shaft. At the gusset plate in the full scale tests, the amount of slack is larger compared to the bolt slack of the gusset plate in the connection tests, which is zero, figure 4.9c & 4.9e. Slack causes less resistance to bolt rotation, therefore the stiffness decreases.

Bolt rotation is also influenced due to the steel grades of the gusset plate and anchor plate two, because the bolt rotation is affected by the amount of bolt hole ovalisation. Due to the asymmetric forces the bolt hole shows different amounts of ovalisation at the front and rear side of the plates. Since the steel grade of the applied steel in the full scale test is higher compared to the steel plates in the connection tests, the amount of ovalisation will be less which results in less rotation and a stiffer behaviour of the frame panel connections.

Since there are several unknown factors that influence the bolt rotation stiffness \(C_R\), it is assumed that the stiffness due to bolt rotation from the connection tests is similar to that of the full scale tests. The thinner gusset plate and the higher steel grades will increase the stiffness and the bolt slack will decrease the stiffness. However, the load on the frame panel connection during the full scale tests is higher compared to the connection tests, therefore the load-bolt-rotation curves have been nonlinearly extended till the bolt failure load, figure 4.8. The bolt rotation stiffness is represented by a bilinear line.

**Figure 4.7** – Bar slip stiffness \(C_{SL}\).

**Figure 4.8** – Bolt rotation stiffness \(C_R\).
Total spring stiffness for finite element model

The spring stiffnesses due to, bolt hole ovalisation \( (C_o) \), anchor bar slip \( (C_{SL}) \) and bolt rotation \( (C_R) \) are combined to a extension spring \( (C_E) \), figure 4.10. The spring characteristics of bolt hole ovalisation \( (C_o) \) and bolt rotation \( (C_R) \) are combined to a compression spring \( (C_C) \), since anchor bar slip will not occur at compression loads. The compression spring characteristic is also used as shear spring \( (C_S) \), which will prevent the concrete panel for rigid body rotation. Adding spring stiffnesses is carried out by taking the reciprocal values, equation 4.9 & 4.10.

\[
C_E = \left( \frac{1}{C_o} + \frac{1}{C_{SL}} + \frac{1}{C_R} \right)^{-1}
\]

\[
C_C = \left( \frac{1}{C_o} + \frac{1}{C_R} \right)^{-1}
\]

These equations can be rewritten such that only the displacements \( (u) \) per spring characteristic can be added. The spring stiffness is defined by:

\[
C = \frac{F}{u}
\]

Substituting equation 4.11 in, for example, equation 4.9 results in:

\[
\frac{1}{F} = \frac{1}{u_E} + \frac{1}{u_o} + \frac{1}{u_{SL}} + \frac{1}{u_R}
\]

When the load is set equal for every change of stiffness at every spring characteristic, equation 4.12 can be simplified to:

\[
u_E = u_o + u_{SL} + u_R
\]

For equation 4.10 the result is:

\[
u_C = u_o + u_R
\]

The displacements per load, where a spring stiffness changes, have been added according to this reasoning, table 4.2. The displacements can be found in the figures 4.6, 4.7, 4.8 & 4.10.
Table 4.2 – Added spring characteristics.

<table>
<thead>
<tr>
<th>Load (kN)</th>
<th>Displacement per stiffness (C₀) [mm]</th>
<th>Displacement per stiffness (Cₛ) [mm]</th>
<th>Displacement per stiffness (Cᵣ) [mm]</th>
<th>Displacement per stiffness (Cₐ) [mm]</th>
<th>Total displacement per stiffness (Cₜ) [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>299</td>
<td>1.68</td>
<td>0.38</td>
<td>2.55</td>
<td>4.61</td>
<td>-</td>
</tr>
<tr>
<td>342</td>
<td>1.92</td>
<td>0.61</td>
<td>2.92</td>
<td>5.45</td>
<td>4.84</td>
</tr>
<tr>
<td>411</td>
<td>4.57</td>
<td>1.13</td>
<td>4.34</td>
<td>10.04</td>
<td>-</td>
</tr>
<tr>
<td>411</td>
<td>4.57</td>
<td>1.58</td>
<td>4.34</td>
<td>10.49</td>
<td>-</td>
</tr>
<tr>
<td>540</td>
<td>7.22</td>
<td>-</td>
<td>5.75</td>
<td>-</td>
<td>12.97</td>
</tr>
<tr>
<td>340</td>
<td>4.57</td>
<td>3.00</td>
<td>4.34</td>
<td>11.91</td>
<td>-</td>
</tr>
</tbody>
</table>

When the frame panel connection is subjected to compression forces, only the stiffness due to bolt hole ovalisation and bolt rotation should be taken into account and if the frame panel connection is subjected to tension forces, the combined stiffness due to bolt hole ovalisation, anchor bar slip and bolt rotation are used. These two spring characteristics are combined into one overall spring characteristic \( Cₚ \), figure 4.11. This combination of spring characteristics is applied in the finite element model. The decreasing stiffness of the spring characteristic after anchor bar failure is adjusted to avoid solution errors during the finite element analysis. The slope of the decreasing line is decreased by increasing the force at the end of the graph.

4.5 Boundary conditions and solution procedure

Two vertical supports at the lower corners, of the finite element model, and one horizontal support at the lower right corner, are applied to represent the supports of the full scale experiments, figure 4.4.

The lateral load of the experimental test will be represented by a prescribed horizontal displacement. The solution procedure is displacement controlled, since the load displacement tangent is expected to become horizontal, figure 3.26. In this case a force controlled solution procedure would result in a solution failure. In experimental test B2, a maximum lateral displacement of 45 mm has been measured, figure 3.26. The behaviour of the test specimen during the first 6 mm will not be described by the finite element model. Therefore the predefined displacement is set to 40 mm. This displacement will be released in 100 steps in the finite element analysis, during the solution procedure.

The solution convergence is set to 0.5% for force and displacement. When the residual forces and displacements are within this tolerance the next substep can be entered.

Figure 4.10* – Total spring stiffnesses FPC3.

Figure 4.11* – Overall spring stiffness FPC3 \( Cₚ \).
Geometric nonlinearity is taken into account. Material non-linearity is only taken into account for the applied longitudinal and torsional springs, since their characteristics are fully described in the finite element model, in contrary to the applied steel and concrete characteristics.

### 4.6 Results finite element analysis

The material properties of experimental test B2 are set for the finite element analysis, since the premature failure of experimental test A2 can not be described by the finite element model. When comparing the load deflection results of the finite element analysis with the experimental full scale results it is clear that both initial stiffnesses are similar, figure 4.12a. The stiffness of the finite element model ($k_{\text{init,FEM}}$) of 35.8 kN/mm is about 5% lower than the average experimental full scale model stiffness ($k_{\text{init,EXP}}$) of 37.7 kN/mm. The ultimate load of the finite element analysis is, with 761 kN, nearly equal to the ultimate load of experimental test B2, which is 762 kN. The failure sequence of the applied springs, which represent the frame panel connection during the finite element analysis, is indicated in figure 4.12b. When a critical point on the failure sequence is reached a character is given. This character refers to the corresponding stress distribution in the concrete panel, figure 4.15.

The normal forces in the steel sections of the frame, obtained from the finite element analysis, are comparable with the experimental test results, figure 4.12. However, the moments in the steel sections are not similar to the moments found in the experimental tests, figure 4.13. This difference is due to imperfections of the test specimen and the different location of the strain gauges compared to the perfect geometry of the finite element model.

![Figure 4.12 - Load deflection results FEM: a) Comparison load deflection, b) Spring failure per step.](image)

![Figure 4.13 - Normal forces sections FEM.](image)

![Figure 4.14 - Moments sections FEM.](image)
The corresponding normal forces indicate that the lateral load in the test specimen is similarly transferred by the specimen to its supports, as the simulated load in the finite element analysis is transferred to its supports. Therefore it can be assumed that the stress distribution in the concrete panel's diagonals, gained from the finite element analyses, are corresponding to the stress distribution in the concrete panel's diagonals during the experimental tests, figure 4.15.

It can be seen that the stresses in the compression diagonal cover a larger area compared to the stresses in the tension diagonal during the entire simulation, figure 4.15. Plastic ovalisation of the bolt holes occurs first at the corners one and four, due to the higher stresses in the compression diagonal, figure 4.12b & 4.15. Shortly after plastic ovalisation, at the compressed corners, plastic ovalisation of the bolt holes at corners two and three occurs. After plastic ovalisation of the bolt holes the stiffness of the infilled frame reduces, figure 4.12b.

The first anchor bar pull out occurs at corner three. This is immediately followed by anchor bar failure at the same corner. As a result the lateral capacity of the infilled frame decreases, figure 4.12b. The stresses in the tension diagonal become significantly lesser compared to the stresses in the compression diagonal when the anchor bar capacity ends at corner three, figure 4.12b & 4.15. At corner two only pull out of the anchor bars occur, figure 4.12b. Failure of the anchor bars in corner two does not occur since the force in the tension diagonal decreases after failure of the frame panel connections in corner three. The amount of needed stresses for failure of the anchor bars is therefore not reached.

From this point on the compression diagonal starts to transfer a significantly larger lateral load to its supports, which results in failure of the bolts in the compressed frame panel connections at the corners one and four, figure 4.12b.

4.7 Chapter Conclusions

The following conclusions can be drawn:

- The load deflection behaviour of the finite element model describes the load deflection behaviour of the experimental test specimen B2.
- The normal forces in the steel sections of the finite element model are comparable with the results of experimental test B2.
- The stresses in the diagonals of the concrete panel indicate that the compression diagonal transfers a larger force to the infilled frame's supports compared to the tension diagonal.
Figure 4.15 – Stress distribution concrete panel: a) Plastic ovalisation corner 4, b) Plastic ovalisation corner 1, c) Plastic ovalisation corner 3.
Figure 4.15 – Stress distribution concrete panel: d) Plastic ovalisation corner 2, e) Anchor bar pull out 3, f) Anchor bar failure corner 3.
Figure 4.15 – Stress distribution concrete panel: g) End of anchor bar capacity corner 3, h) Anchor bar pull out 2, i) Bolt failure corner 1 and 4.
Discussion

5.1 Results experimental tests

In this section the load deflection behaviour of the infilled frames will be compared to the load deflection behaviour of previously investigated infilled frames [9, 14, 15].

The average elastic strength of the infilled frames with FPC3 is significantly higher compared to the infilled frames provided with FPC1, FPC2 and FPCL, figure 5.1 & table 5.1. For comparing the ultimate strength it can be seen that the infilled frame with the improved frame panel connections (test B2), has a significantly higher strength than the other infilled frames.

When considering the initial stiffnesses of the infilled frames, it can be seen that the average initial stiffness of the infilled frames with FPC3 is between two and three times higher compared to the infilled frames with the other frame panel connections, figure 5.2 & table 5.1.

The deformation capacities of the infilled frames with FPC3 can not be compared with the results from the previously investigated infilled frames, since the previous experimental tests were halted, due to safety reasons, before a proper yield plateau could occur.

The increase of strength and stiffness of the infilled frame with FPC3 is partly caused by the increased strength and stiffness of its frame panel connections, table 5.2. Also the relocation of the frame panel connection to the frame’s corners has an influence, since the relocation causes a relative smaller force in the diagonals and less bending in the frame’s beam sections. It can, however, not be stated how much the influence of one change is, compared to the other change, to the overall behaviour of the infilled frame.

Table 5.1 – Results overview of infilled frames with different frame panel connections.

<table>
<thead>
<tr>
<th>Infilled frame with connection</th>
<th>Initial stiffness [kN/mm]</th>
<th>Elastic strength [kN]</th>
<th>Ultimate strength [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>FPC1</td>
<td>15.9 (39%)</td>
<td>345 (59%)</td>
<td>-</td>
</tr>
<tr>
<td>FPC2</td>
<td>12.5 (31%)</td>
<td>241 (41%)</td>
<td>276 (36%)</td>
</tr>
<tr>
<td>FPCL</td>
<td>20.1 (49%)</td>
<td>332 (57%)</td>
<td>503 (66%)</td>
</tr>
<tr>
<td>FPC3 (Test A2)</td>
<td>34.8 (86%)</td>
<td>584 (100%)</td>
<td>644 (85%)</td>
</tr>
<tr>
<td>FPC3 (Test B2)</td>
<td>40.7 (100%)</td>
<td>536 (92%)</td>
<td>762 (100%)</td>
</tr>
</tbody>
</table>

* The ultimate strength could not be determined since the concrete panel made contact with the steel frame and the test was stopped

Table 5.2 – Tensile stiffness and strength of frame panel connections.

<table>
<thead>
<tr>
<th>Connection</th>
<th>Stiffness [kN/mm]</th>
<th>Ultimate strength [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>FPC1</td>
<td>114 (79%)</td>
<td>305* (70%)</td>
</tr>
<tr>
<td>FPC2</td>
<td>89 (61%)</td>
<td>185** (42%)</td>
</tr>
<tr>
<td>FPC3</td>
<td>145*** (100%)</td>
<td>441*** (100%)</td>
</tr>
</tbody>
</table>

* Elliptical reduction, ** Linear reduction, *** adjusted to full scale test characteristics (section 4.4)
5.2 Results finite element analysis

The initial stiffness of the finite element model lies between the initial stiffnesses of the experimental tests A2 and B2, table 5.3. When taking the average initial stiffness of the experimental tested infilled frames, \( k_{\text{ini,EXP}} \) is 37.7 kN/mm, the difference is about 5% compared to the initial stiffness of the finite element model, \( k_{\text{ini,FEM}} \) is 35.8 kN/mm. The initial stiffness of the experimental tested infilled frames partly depends on bonding of the anchor bars in the concrete panel. This bonding differs per frame panel connection due to differences in concrete compaction and differences in the location of stirrups. Therefore the initial stiffnesses of the infilled frames can differ per experimental test and thus differ compared to the finite element results in which these phenomena are not taken into account.

The elastic strength of the finite element model is 10% less compared to the elastic strength of the results from experimental test A2 and 2% less compared to the results from experimental test B2, table 5.3. The elastic strength is taken where at the experimental tests concrete cracking occurs and at the finite element results plastic ovalisation of the bolt holes occurs, since concrete cracking is not modeled, figure 5.3. The differences in elastic strengths can be clarified by the fact that cracking of the concrete near the end of the anchor bars is not taken into account at the finite element analysis, but plastic ovalisation of the bolt holes is. The bolt hole ovalisation behaviour in the steel anchor plates is more precisely to predict compared to the behaviour of cracking in the concrete panel. Therefore, the range wherein concrete cracking could occur is larger than the range where plastic ovalisation of the bolt holes occurs. This could clarify the difference of elastic strength between the experimental test result and the finite element model.

Cracking of the concrete, at the experimental tests, occurs at more or less the same lateral load as plastic ovalisation of the bolts occurs in the finite element model. Both phenomena occur during a similar lateral load because shortening of the compression diagonal, due to ovalisation of the bolt holes, leads to lengthening of the tension diagonal. Lengthening of the tension diagonal

Table 5.3 - Results overview of infilled frames with different frame panel connections.

<table>
<thead>
<tr>
<th></th>
<th>Initial stiffness [kN/mm]</th>
<th>Elastic strength [kN]</th>
<th>Ultimate strength [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Finite element analysis</td>
<td>35.8 (88 %)</td>
<td>528 (90 %)</td>
<td>761 (100 %)</td>
</tr>
<tr>
<td>Experimental test A2</td>
<td>34.8 (86 %)</td>
<td>584 (100 %)</td>
<td>644 (85 %)</td>
</tr>
<tr>
<td>Experimental test B2</td>
<td>40.7 (100 %)</td>
<td>536 (92 %)</td>
<td>762 (100 %)</td>
</tr>
</tbody>
</table>

Figure 5.1* - Load-deflection of infilled frames.

Figure 5.2* - Stiffnesses of infilled frames.
leads to settling of the reinforcement meshes in the concrete panel. This results in cracking of the concrete near the end of the anchor bars, since at this location the tension forces are not longer taken by the anchor bars.

The ultimate strength of the finite element analysis can only be compared to the results from experimental test B2, since experimental test A2 failed prematurely. It can be seen that the ultimate strength of the finite element model equals the ultimate strength of the results from experimental test B2, table 5.3 & figure 5.3. However, the ultimate strengths are not reached at the same lateral deformation, since the initial stiffness of the experimental tested infilled frame differs compared to the initial stiffness of the finite element model. The ultimate strength is reached when anchor bar pull out occurs at the frame panel connections subjected to tension forces, figure 5.3.

After the ultimate load of the experimentally tested infilled frame is reached, a small yield plateau occurs till the bolts in the frame panel connection, at the compressed corners, deform plastically. Plastic bolt deformation occurs since a significant part of the lateral load is transferred by the compression diagonal to the frame’s supports after failure of the frame panel connections, located in the tension diagonal. Deformation of the bolts is not modeled in the finite element model, but bolt failure is by taking a bolt shear capacity of 540 kN into account. Since the bolt shear deformation behaviour is not described by the finite element model the behaviour before bolt failure is not known. The absent of bolt shear deformation at the finite element model clarifies the different behaviour, compared to the experimental tests, after the ultimate strength has been reached.
6 Conclusions and recommendations

6.1 Conclusions

Compared to previously investigated infilled frames, the infilled frame with FPC3 shows improved strength and stiffness characteristics. The increase of strength and stiffness of the infilled frame is caused by the improved frame panel connection and by relocation of the frame panel connections to the frame's corners.

The deformation capacities of the experimentally tested infilled frames can not be compared to the results of the previous investigated infilled frames, since these experimental tests were halted due to unexpected occurrences.

At experimental test B2, a small yield plateau occurs after reaching the ultimate load. Cracking of the concrete, near the end of the anchor bars, in the tension diagonal has little effect on the deformation capacity, since the compression diagonal remains intact. The ultimate strength and the stiffness of the infilled frame however, reduce due to cracking of the concrete.

In tests A2 & A3, ovalisation of the bolt holes did occur, but the main failure mechanism of the frame panel connection is buckling of the gusset plate and anchor plate two. At the improved frame panel connections of tests B2 & B3, where plate buckling was prevented, ovalisation of the bolt holes prevails as failure mechanism and a small yield plateau in the load deflection behaviour of the infilled frame occurs. However the amount of bolt hole ovalisation is limited, which eventually leads to pull out of the anchor bars and bolt shear deformation.

The finite element model does describe the racking shear behaviour of the experimental test specimen in terms of strength and stiffness. The failure sequence of the frame panel connections at the finite element model and the experimental tests is also comparable till the ultimate strength has been reached. After this ultimate strength the phenomena bolt failure did not correspond. Therefore it can be concluded that the spring characteristics, which represent the frame panel connections, are correctly adapted from the connection tests. The bolt shear capacity is, however, incorrectly assumed.

6.2 Recommendations

During the research presented in this thesis, several complications emerged due to mismatches between the connection tests and the full scale tests. To avoid these complications in future research into semi-integral infilled frames and to improve the strength of FPC3, the following recommendations are given:

- Every phenomenon that influences the strength and stiffness of FPC3, such as bolt hole ovalisation, bolt rotation, bolt deformation and slip of the anchor bars should be measured individually, under tension, compression and shear loading. Only then reliable spring characteristics can be obtained for the finite element model.
- Test specimens, for connection tests and full scale tests, should be fabricated on the same day and the used materials should be from the same batch, so that the material characteristics of the applied steel and concrete are similar for all test specimens and no conversion of the material characteristics is needed.

- Test specimens, for connection tests and full scale tests, should also be tested on the same day, to be sure that the concrete characteristics are comparable.

- Also the geometrical properties of the connection and full scale test specimens should be identical, such as concrete slab thickness, reinforcement bar diameter, amount of reinforcement, thickness of steel plates, bolt hole diameters and bolt dimensions. This will also avoid conversion from test results to match connection test results with the full scale test results.

- The load configuration on the frame panel connection during a connection test should be comparable with the load configuration on the frame panel connection during a full scale test, figure 6.1.

![Figure 6.1 - Differences between load configurations on the frame panel connection: a) Load configuration during a connection test, b) Load configuration during a full scale test.](image-url)
References


Appendix A  Drawings
A1  Infilled frame
A2  Bare steel frame

Gusset plate
A3 Details steel frame

Side view beam HE180M (left and right side)

Beam section (HE180M)
A4 Reinforcement concrete panel

Infill panel

Section AA

Section BB
A5  Steel parts for frame panel connection

Anchor plate 2
- t: 10 mm
- Steel: S235
- Number: 8

Gusset plate
- t: 15 mm
- Steel: S235
- Number: 8

Anchor plate 1
- t: 15 mm
- Steel: S235
- Number: 8

Reinforcement bar
- ø: 16 mm
- Steel: FEB500
- Number: 40
A6 Frame panel connection

Side view anchor plates 1 & 2

Front view anchor plates 1 & 2

Side view entire FPC

Top view anchor plates 1
Appendix B Photos
B1  Reinforcement panel A

Figure B1.1 – Reinforcement panel A.

Figure B1.2 – FPC3 at corner 2.

Figure B1.3 – FPC3 at corner 1.

Figure B1.3 – FPC3 at corner 4.

Figure B1.5 – FPC3 at corner 3.
Appendix B

B2 Test specimen A1

Figure B2.1 – Bare frame A (test A1).

Figure B2.2 – Mitutoyo corner 1.

Figure B2.3 – Mitutoyo corner 2.

Figure B2.4 – Mitutoyo corner 4.

Figure B2.5 – Mitutoyo corner 3.
B3  Test specimen A2 before testing

Figure B3.1 – Infilled frame A before testing (test A2, front view).

Figure B3.2 – Corner 1 front view.

Figure B3.3 – Corner 1 back view.

Figure B3.4 – Corner 2 front view.

Figure B3.5 – Corner 2 back view.
B4 Test specimen A2 after testing

Figure B4.1 – Infilled frame A after testing (test A2, front view).

Figure B4.2 – Corner 2 front view. Figure B4.3 – Corner 2 back view.

Figure B4.4 – Corner 3 front view. Figure B4.5 – Corner 3 back view.
Figure B4.6 - Corner 1.

Figure B4.7 - Corner 2.

Figure B4.8 - Corner 3.

Figure B4.9 - Corner 4.

Figure B4.10 - Anchor plate 2 corner 4.

Figure B4.11 - Gusset plate corner 4.

Figure B4.12 - Corner 1 anchor plate 2.

Figure B4.13 - Corner 2 anchor plate 2.
Figure B4.14 – Corner 3 anchor plate 2.

Figure B4.15 – Corner 4 anchor plate 2.

Figure B4.16 – Corner 1 gusset plate.

Figure B4.17 – Corner 2 gusset plate.

Figure B4.18 – Corner 3 gusset plate.

Figure B4.19 – Corner 4 gusset plate.
B5  Test specimen A3 after testing

Figure B5.1 – Infilled frame A after testing (test A3, front view).

Figure B5.2 – Corner 1 front view.  
 Figure B5.3 – Corner 1 back view.

Figure B5.4 – Corner 4 front view.  
 Figure B5.5 – Corner 4 back view.
Figure B5.6 – Corner 1.

Figure B5.7 – Corner 2.

Figure B5.8 – Corner 3.

Figure B5.9 – Corner 4.

Figure B4.10 – Anchor plate 2 corner 4.

Figure B4.11 – Anchor plate 2 corner 3.

Figure B5.12 – Corner 1 anchor plate 2.

Figure B5.13 – Corner 2 anchor plate 2.
B6  Reinforcement panel B

Figure B6.1 - Reinforcement panel B.

Figure B6.2 - FPC3 at corner 2.

Figure B6.3 - FPC3 at corner 1.

Figure B6.3 - FPC3 at corner 4.

Figure B6.5 - FPC3 at corner 3.
B7  Test specimen B1

Figure B7.1 - Bare frame B (test B1, front view).

Figure B7.2 - Mitutoyo corner 1.

Figure B7.3 - Mitutoyo corner 2.

Figure B7.4 - Mitutoyo corner 4.

Figure B7.5 - Mitutoyo corner 3.
B8  Test specimen B2 before testing

Figure B8.1 – Infilled frame B before testing (test B2, front view).

Figure B8.2 – Corner 1 front view.  Figure B8.3 – Corner 1 back view.

Figure B8.4 – Corner 2 front view.  Figure B8.5 – Corner 2 back view.
B9 Test specimen B2 after testing

Figure B9.1 - Infilled frame A2 after testing (no picture available of specimen B2 after testing).

Figure B9.2 - Corner 2 front view.

Figure B9.3 - Corner 2 back view.

Figure B9.4 - Corner 3 front view.

Figure B9.5 - Corner 3 back view.
Appendix B

Figure B9.8 - Corner 1 anchor plate 2.

Figure B9.9 - Corner 2 anchor plate 2.

Figure B9.10 - Corner 3 anchor plate 2.

Figure B9.11 - Corner 4 anchor plate 2.

Figure B9.12 - Corner 1 gusset plate.

Figure B9.13 - Corner 2 gusset plate.

Figure B9.14 - Corner 3 gusset plate.

Figure B9.15 - Corner 4 gusset plate.
B10 Test specimen B3 after testing

Figure B10.1 – Infilled frame B3 after testing (test B3, front view).

Figure B10.2 – Corner 1 front view.

Figure B10.3 – Corner 1 back view.

Figure B10.4 – Corner 4 front view.

Figure B10.5 – Corner 4 back view.
Figure B10.6 - Corner 1 anchor plate 2.

Figure B10.7 - Corner 2 anchor plate 2.

Figure B10.8 - Corner 3 anchor plate 2.

Figure B10.9 - Corner 4 anchor plate 2.

Figure B10.10 - Corner 1 gusset plate.

Figure B10.11 - Corner 2 gusset plate.

Figure B10.12 - Corner 3 gusset plate.

Figure B10.13 - Corner 4 gusset plate.
Appendix C  Material properties
C1 Material properties concrete

Compression strength test
The ordered concrete quality is C35/45. To obtain the strength of the concrete during the full scale experiments nine concrete cubes have been tested, table C1.1. A loading speed of 10 kN/sec is applied.

Table C1.1 - Concrete compression strength.

<table>
<thead>
<tr>
<th>Test specimen</th>
<th>Production date</th>
<th>Test date</th>
<th>Age (days)</th>
<th>Width (mm)</th>
<th>Depth (mm)</th>
<th>Area (mm²)</th>
<th>$f_u$ (kN)</th>
<th>$f'_{ck}$ (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A2-1</td>
<td>22-07-2010</td>
<td>09-09-2010</td>
<td>49</td>
<td>150.3</td>
<td>149.5</td>
<td>22470</td>
<td>1436.6</td>
<td>64</td>
</tr>
<tr>
<td>A2-2</td>
<td>22-07-2010</td>
<td>09-09-2010</td>
<td>49</td>
<td>150.0</td>
<td>149.8</td>
<td>22470</td>
<td>1487.2</td>
<td>66</td>
</tr>
<tr>
<td>A2-3</td>
<td>22-07-2010</td>
<td>09-09-2010</td>
<td>49</td>
<td>153.4</td>
<td>149.7</td>
<td>22964</td>
<td>1440.0</td>
<td>63</td>
</tr>
<tr>
<td>A3-1</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>A3-2</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>A3-3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Average: 64

<table>
<thead>
<tr>
<th>Test specimen</th>
<th>Production date</th>
<th>Test date</th>
<th>Age (days)</th>
<th>Width (mm)</th>
<th>Depth (mm)</th>
<th>Area (mm²)</th>
<th>$f_u$ (kN)</th>
<th>$f'_{ck}$ (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B2-1</td>
<td>22-07-2010</td>
<td>10-11-2010</td>
<td>111</td>
<td>153.2</td>
<td>149.6</td>
<td>22919</td>
<td>1606.5</td>
<td>70</td>
</tr>
<tr>
<td>B2-2</td>
<td>22-07-2010</td>
<td>10-11-2010</td>
<td>111</td>
<td>152.5</td>
<td>149.5</td>
<td>22799</td>
<td>1743.9</td>
<td>77</td>
</tr>
<tr>
<td>B2-3</td>
<td>22-07-2010</td>
<td>10-11-2010</td>
<td>111</td>
<td>149.8</td>
<td>149.9</td>
<td>22455</td>
<td>1657.5</td>
<td>74</td>
</tr>
</tbody>
</table>

Average: 74

<table>
<thead>
<tr>
<th>Test specimen</th>
<th>Production date</th>
<th>Test date</th>
<th>Age (days)</th>
<th>Width (mm)</th>
<th>Depth (mm)</th>
<th>Area (mm²)</th>
<th>$f_u$ (kN)</th>
<th>$f'_{ck}$ (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B3-1</td>
<td>22-07-2010</td>
<td>11-11-2010</td>
<td>112</td>
<td>153.1</td>
<td>149.7</td>
<td>22915</td>
<td>1684.8</td>
<td>74</td>
</tr>
<tr>
<td>B3-2</td>
<td>22-07-2010</td>
<td>11-11-2010</td>
<td>112</td>
<td>153.5</td>
<td>149.7</td>
<td>22989</td>
<td>1578.3</td>
<td>69</td>
</tr>
<tr>
<td>B3-3</td>
<td>22-07-2010</td>
<td>11-11-2010</td>
<td>112</td>
<td>150.3</td>
<td>149.2</td>
<td>22410</td>
<td>1661.2</td>
<td>74</td>
</tr>
</tbody>
</table>

Average: 72

Prism test
Three concrete prisms have been tested twice to determine the elasticity modulus of the concrete during the full scale tests, table C1.2 and figure C1.1 & C1.2. The height of all prisms is 500 mm. The gauge length is 200 mm. These tests have been performed at the same day as the full scale tests A2 & B2 were performed.

Table C1.2 - Prism test.

<table>
<thead>
<tr>
<th>Test specimen</th>
<th>Production date</th>
<th>Test date</th>
<th>Age (days)</th>
<th>Average width (mm)</th>
<th>Average depth (mm)</th>
<th>Area (mm²)</th>
<th>Young's modulus (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A2-1</td>
<td>22-07-2010</td>
<td>09-09-2010</td>
<td>49</td>
<td>100.8</td>
<td>100.2</td>
<td>10100</td>
<td>33133</td>
</tr>
<tr>
<td>A2-2</td>
<td>22-07-2010</td>
<td>09-09-2010</td>
<td>49</td>
<td>101.6</td>
<td>100.3</td>
<td>10190</td>
<td>32370</td>
</tr>
<tr>
<td>A2-3</td>
<td>22-07-2010</td>
<td>09-09-2010</td>
<td>49</td>
<td>101.2</td>
<td>100.2</td>
<td>10140</td>
<td>30891</td>
</tr>
</tbody>
</table>

Average: 32131

<table>
<thead>
<tr>
<th>Test specimen</th>
<th>Production date</th>
<th>Test date</th>
<th>Age (days)</th>
<th>Average width (mm)</th>
<th>Average depth (mm)</th>
<th>Area (mm²)</th>
<th>Young's modulus (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B2-1</td>
<td>22-07-2010</td>
<td>11-11-2010</td>
<td>112</td>
<td>100.8</td>
<td>100.2</td>
<td>10100</td>
<td>32127</td>
</tr>
<tr>
<td>B2-2</td>
<td>22-07-2010</td>
<td>11-11-2010</td>
<td>112</td>
<td>101.6</td>
<td>100.3</td>
<td>10190</td>
<td>31702</td>
</tr>
<tr>
<td>B2-3</td>
<td>22-07-2010</td>
<td>11-11-2010</td>
<td>112</td>
<td>101.2</td>
<td>100.2</td>
<td>10140</td>
<td>30300</td>
</tr>
</tbody>
</table>

Average: 31376
Figure C1.1 - Stress strain curve concrete prisms Panel A.

Figure C1.2 - Stress strain curve concrete prisms Panel B.
C2  Material properties steel FPC

Tensile strength test

Six tensile strength tests are performed to determine the quality of the applied steel for the frame panel connections. For the gusset plate’s cold formed steel (S235JRC+C) is used. Three coupons are made, coupon 1, 2 and 3, table C2.1 and figure C2.1. This steel quality is also used for fabrication of anchor plate one. The other three coupons are samples from steel which is used for fabrication of anchor plate two, coupon four, five and six, table C2.1 and figure C2.1. For anchor plate two hot rolled steel (S235JR) is used. All tensile strength tests are performed according to NEN-EN 10002-1.

Table C2.1 - Dimensions coupons.

<table>
<thead>
<tr>
<th>Coupon number</th>
<th>Position</th>
<th>w [mm]</th>
<th>t [mm]</th>
<th>A₀ [mm²]</th>
<th>Lₜ [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (C-F)</td>
<td>A</td>
<td>13.09</td>
<td>14.96</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>13.09</td>
<td>14.95</td>
<td>195.79</td>
<td>200.6</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>13.10</td>
<td>14.95</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2 (C-F)</td>
<td>A</td>
<td>13.13</td>
<td>14.95</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>13.08</td>
<td>14.95</td>
<td>195.74</td>
<td>200.7</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>13.06</td>
<td>14.96</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3 (C-F)</td>
<td>A</td>
<td>13.06</td>
<td>10.18</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>13.12</td>
<td>15.00</td>
<td>196.94</td>
<td>200.6</td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>13.12</td>
<td>15.04</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4 (H-R)</td>
<td>A</td>
<td>13.05</td>
<td>10.23</td>
<td>133.06</td>
<td>197.4</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>13.07</td>
<td>10.18</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>13.05</td>
<td>10.17</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5 (H-R)</td>
<td>A</td>
<td>13.06</td>
<td>10.26</td>
<td>133.58</td>
<td>197.4</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>13.06</td>
<td>10.23</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>13.06</td>
<td>10.19</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6 (H-R)</td>
<td>A</td>
<td>13.07</td>
<td>10.19</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>13.06</td>
<td>10.26</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>13.07</td>
<td>10.19</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

(C-F) = Cold formed  (H-R) = Hot rolled

All tests are performed with a strain rate of 0.6 mm/min. in the elastic area and a strain rate of 3.6 mm/min. in the plastic area. Test results are given in table C2.2 and figures C2.2 & C2.3.

Table C2.2 - Tensile strength test results (true stress).

<table>
<thead>
<tr>
<th>Coupon (C-F)</th>
<th>Yield stress 0.2% [N/mm²]</th>
<th>Tensile-strength [N/mm²]</th>
<th>Coupon (H-R)</th>
<th>Yield stress [N/mm²]</th>
<th>Tensile-strength [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>480.5</td>
<td>577.3</td>
<td>4</td>
<td>313.0</td>
<td>515.7</td>
</tr>
<tr>
<td>2</td>
<td>456.3</td>
<td>545.6</td>
<td>5</td>
<td>313.5</td>
<td>514.5</td>
</tr>
<tr>
<td>3</td>
<td>471.9</td>
<td>569.4</td>
<td>6</td>
<td>314.9</td>
<td>515.1</td>
</tr>
<tr>
<td>Average</td>
<td>469.9</td>
<td>564.1</td>
<td>Average</td>
<td>313.8</td>
<td>515.1</td>
</tr>
</tbody>
</table>
Figure C2.2 – True stress true strain curve, cold formed steel.

Figure C2.3 – True stress true strain curve, hot rolled steel.
Appendix D  Calculations
D1 Beam-column connection steel frame

Shear resistance bolts
Four M24 10.9 bolts per connection are used. The bolts have to resist shear forces (through thread) only, therefore the shear capacity \( F_{v,Rd} \) is according to Eurocode 3:

\[
F_{v,Rd} = \frac{\alpha_v \cdot f_{ub} \cdot A_{b,s}}{\gamma_{M2}}
\]

\( \alpha_v = 0.5 \) for bolt class 10.9 and shear plane through the threaded portion

\( f_{ub} = 1000 \text{ N/mm}^2 \)

\( A_{b,s} = 352 \text{ mm}^2 \)

\( \gamma_{M2} = 1.25 \)

\[
F_{v,Rd} = \frac{0.5 \cdot 1000 \cdot 352}{1,25} = 140 \text{ kN} \quad \text{(per bolt)}
\]

\( 140 \cdot 4 = 560 \text{ kN} \quad \text{(per connection)} \)

A shear force of 560 kN at the bolts is equal to a lateral force of 1120 kN at the frame. Since the expected lateral force on the frame is 600 kN, due to the strength of FPC3, the bolts will sustain.

Bearing resistance steel plate
The bolts are located at the head plate of the beam and the flanges of the column. Since the head plate is thinner than the flanges of the column the bearing resistance \( F_{b,Rd} \) of the head plate will be calculated:

\[
F_{b,Rd} = \frac{k_1 \cdot \alpha_b \cdot f_v \cdot d \cdot t}{\gamma_{M2}}
\]

\( k_1 = \text{smallest value of: } \frac{2.8 \cdot \frac{e_l}{d_0} - 1.7}{2.8 \cdot \frac{43}{26} - 1.7} = 3.26 \quad \text{or} \quad 2.5 \)

\( \alpha_b = \text{smallest value of: } \frac{50}{3 \cdot 26} = 0.64, \quad \frac{1000}{360} = 2.78 \quad \text{or} \quad 1.0 \)

\( f_v = 360 \text{ N/mm}^2 \)

\( d = 24 \text{ mm} \)

\( t = 15 \text{ mm} \)

\( \gamma_{M2} = 1.25 \)

\[
F_{b,Rd} = \frac{2.5 \cdot 0.64 \cdot 360 \cdot 24 \cdot 15}{1.25} = 166 \text{ kN} \quad \text{(per bolt)}
\]

\( 166 \cdot 4 = 664 \text{ kN} \quad \text{(per connection)} \)
The force of 664 kN is larger as the bolt shear force, therefore the bearing resistance will sustain.

**Weld head plate**

When the following equations are both satisfied, the weld will be sufficient according to Eurocode 3:

\[
\sqrt{\sigma_1^2 + 3(\tau_1^2 + \tau_2^2)} \leq \frac{f_u}{\beta_w \cdot \gamma_{N2}} \quad \text{and} \quad \sigma_1 \leq \frac{0.9 \cdot f_u}{\gamma_{N2}}
\]

![Figure D1.1 - Forces on weld.](image)

Magenta: half force of diagonal.

Blue: dissolved magenta diagonal force

Black: dissolved blue horizontal force

\[L = 138 \text{ mm}\]
\[a = 9 \text{ mm}\]

Results:

\[
\sqrt{100^2 + 3(100^2 + 143^2)} \leq \frac{360}{0.8 \cdot 1.25} \quad \text{and} \quad 100 \leq \frac{0.9 \cdot 360}{1.25}
\]

\[
318 \leq 360 \quad \text{and} \quad 100 \leq 259
\]

The weld throat of 9 mm will be sufficient, since it can resist at lateral force at the frame of 707 kN.
D2 Design calculations frame panel connection 3

The connection has been designed based on a failure mode of bearing in the bolt holes. This appendix describes the design calculations to determine the bearing resistance of the steel anchor plate and is copied, with permission from the author, from previous research [4]. The shear resistance of the bolts and the strength of the fillet weld (between steel gusset plate and steel beam) are designed to maintain bearing in the bolt holes as the governing failure mode. In order to assure the connection will fail due to bearing in the bolt holes, the value for the tensile strength \( f_u \) of the steel plate is determined from tension tests of five steel specimens according to NEN-EN 10020-1. The average yield strength \( f_y \) of the steel anchor plate is 314 N/mm\(^2\) and an average tensile strength \( f_u \) is 515 N/mm\(^2\). These values are used for further calculation purposes.

Figure D2.1 - Simplified force distribution of an infilled frame.

**Bearing resistance of the steel plates**

The bearing resistance of the steel plates will be an indicative value to design other components of the connection. The calculations are made according to Eurocode 3. The bearing resistance of the steel plates can be determined as followed:

\[
F_{b,nu} = \frac{k_b \cdot k_t \cdot f_u \cdot d \cdot t}{f_y} \cdot 10^{-3}
\]

- \( F_{b,nu} \): the design bearing resistance per bolt
- \( k_b \): a factor to take edge distances into account
- \( k_t \): the bearing factor

The factors \( k_b \) and \( k_t \) are dependent variables of the edge distance and spacing of the bolt. The minimal edge distance \( e_e \) and \( e_o \) of the bolt is \( \geq 1.2 \cdot d_o \), and the minimal spacing between the bolts \( (p_e \) and \( p_o \) is \( \geq 2.4 \cdot d_o \). In order to keep bearing of the bolt holes decisive, the edge distance will be adjusted to get a bearing factor of 1.0. Values for edge bolts:

\[
k_b = \frac{2.8 \cdot e_e}{d_o} - 1.7 \quad \text{or} \quad 2.5
\]

\[
k_t = \frac{2.8 \cdot e_o}{3 \cdot d_o} \quad \text{or} \quad 1.0
\]

\( e_e = 78.0 \text{ mm} \)

\( e_o = 78.0 \text{ mm} \)

\( f_u = 515 \text{ N/mm}\(^2\) \)

\( d = 24 \text{ mm} \)

\( t = 10 \text{ mm} \)
The bearing resistance of the connection along the main force direction is 494.4 kN (2x 247.2) and can be resolved into a horizontal and vertical force to design the anchor configuration as well as the fillet weld between the steel gusset plate and the frame member, 494.4 kN should be decisive.

**Shear resistance of the bolts**

Each bolt must be able to resist a maximum force of 247.2 kN/bolt. The previous connections contained 2M24 10.9 bolts. The design value of the shear resistance of a single bolt will be calculated according to Eurocode 3.

$$F_{v,Rd} = \frac{2.5 \cdot 1.0 \cdot 515 \cdot 24 \cdot 10^{-3}}{1.25} = 247.2 \text{ kN/bolt}$$

If the grade of the bolts is reduced to 8.8, the maximum shear resistance of the bolt will be:

$$F_{v,Rd} = \frac{0.6 \cdot 1000 \cdot 452}{1.25} \cdot 10^{-3} = 216.69 \text{ kN/bolt (Total = 433.38 kN)}$$

$\text{Table D2.1 - Classification of bolts.}$

<table>
<thead>
<tr>
<th>Classification</th>
<th>4.6</th>
<th>8.8</th>
<th>10.9</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f_{pu}$ [N/mm$^2$]</td>
<td>240</td>
<td>640</td>
<td>900</td>
</tr>
<tr>
<td>$f_{ub}$ [N/mm$^2$]</td>
<td>400</td>
<td>800</td>
<td>1000</td>
</tr>
</tbody>
</table>

Two bolts M24 10.9 are able to resist the maximum load before bearing in the bolt holes occurs.
Weld of the steel gusset plate to beam

According to the Eurocode 3 (Part 1-8, article 4.5.3.2), the design resistance of the fillet weld will be sufficient if the following equations are both satisfied:

\[
[s^2 + 3 \cdot (r_s + r_u)]^{0.5} \leq \frac{f_u}{\beta_h \cdot \gamma_h} \quad \text{and} \quad \sigma \leq 0.9 \cdot \frac{f_u}{\gamma_h}
\]

- \(\beta_h\) correlation factor for fillet welds
- \(f_u\) nominal ultimate tensile strength of the weaker part joined
- \(\gamma_h\) partial safety factor for welded joints

\[
\begin{align*}
\beta_h &= 0.8 \\
f_u &= 360 \text{ N/mm}^2 \\
\gamma_h &= 1.25
\end{align*}
\]

Figure D2.4 - Stresses at the weld between steel frame and gusset plate.

The stresses at section 'A' with a weld throat of 9 mm are:

As a result of \(F_{b,\text{Rd}} \cdot \sin(\theta)\) \(\Rightarrow \sigma_z = \frac{2 \cdot F_{b,\text{Rd}} \cdot \sqrt{2}}{4 \cdot a \cdot l_{eff}} = \frac{2 \cdot 197.2 \cdot \sqrt{2}}{4 \cdot 9 \cdot 275} = 39.8 \text{ N/mm}^2\)

As a result of \(F_{b,\text{Rd}} \cdot \cos(\theta) \cdot e\) \(\Rightarrow \sigma_z = \frac{2 \cdot 12 \cdot M_{\text{keq}}}{a \cdot l_{eff}} = \frac{2 \cdot 12 \cdot 42.5 \cdot 10^6}{9 \cdot 275^2} = 132.5 \text{ N/mm}^2\)

As a result of \(F_{b,\text{Rd}} \cdot \cos(\theta)\) \(\Rightarrow \tau_{ji} = 0 \text{ N/mm}^2\)

The weld check is as follows:

(1) \[\left[39.8 + 132.5 \right]^2 + 3 \cdot (139.4 + 132.5)^2 \leq \frac{360}{0.8 \cdot 1.25} \Rightarrow 345 \leq 360 \quad [\text{N/mm}^2]\]

(2) \[127.3 \leq \frac{0.9 \cdot 360}{1.25} \Rightarrow 172 \leq 259 \quad [\text{N/mm}^2]\]

The two unity checks as stated before are: (1): 0.96 and (2): 0.66 respectively. Therefore 9 mm welds with an effective length of 275 mm will resist the maximum bearing load of the bolt holes.
Appendix D

Steel anchor bars

According to the Eurocode 2, the minimally required steel anchor bars can be determined by using the theoretical tension force $N_{a}$:

$$N_{a} = 2 \cdot F_{b,Rd}$$

$N_{a}$ design value of the theoretical tension force  
$F_{b,Rd}$ bearing resistance of connection

$$N_{a} = 494.4 \text{ kN}$$

$$F_{b,Rd} = 247.2 \text{ kN}$$

$$A_{s} = \frac{N_{a}}{f_{s}} \text{ required section of area of the anchor bar}$$

$A_{s} = \frac{494.4 \cdot 10^{3}}{500} = 989 \text{ mm}^{2}$

$$f_{s} \text{ design value of the tensile strength of the anchor bar} f_{s} = 435 \text{ N/mm}^{2}$$

$$50 \Omega 16 \text{ (A}_{s} = 1005 \text{ mm}^{2})$$

$$N_{a} = 1005 \cdot 500 \cdot 10^{3} = 502 \text{ kN}$$

Anchor embedment length

According to Eurocode 2, the anchor embedment length of the anchor bars has to be at least:

$$l_{b} = a_{1} \cdot a_{2} \cdot a_{3} \cdot a_{4} \cdot a_{5} \cdot l_{b,min}$$

$l_{b}$ design value of the anchorage length
$a_{1}, a_{2}, a_{3}$ t/m as coefficients equal to 1
$a_{4}$ coefficient for the effect of minimum concrete cover  
$a_{5} = 1 - 0.15 \cdot (c_{d} - 3 \cdot 16)/16 \geq 0.7$

$c_{d}$ concrete cover
$l_{b,min}$ the minimal anchorage length

$$l_{b,min} = \max(0.3 \cdot l_{b,reqd}, 10 \cdot f_{b} R_{d}, 100 \text{ mm})$$

$$l_{b,reqd} = \frac{\sigma_{ad}}{4} \cdot f_{b} R_{d}$$

$\sigma_{ad}$ value of the anchor stress
$f_{b} R_{d}$ ultimate adhesive strength

$$\sigma_{ad} = 500 \text{ N/mm}^{2}$$

$$f_{b} R_{d} = 3.37 \text{ N/mm}^{2}$$

$$\sigma_{ad} = 500 \text{ N/mm}^{2}$$

$$f_{b} R_{d} = 3.37 \text{ N/mm}^{2}$$

$$l_{b,reqd} = \frac{16 \cdot 500}{4 \cdot 3.37} = 593 \text{ mm}$$

$$l_{b,min} = 0.7 \cdot 593 = 415 \text{ mm}$$

The minimum anchorage length to transfer the force to resist the maximum bearing strength of the bolt holes has to be 415 mm. An additional check of splitting of the concrete is necessary.

According to analytical research by Wang the embedment length of the reinforcement bars to prevent splitting of the concrete has to be:

$$l_{b} = \frac{1.85 \cdot f_{y,a}}{c \cdot (\frac{f_{c}^{'}}{d_{b}})} \cdot \sqrt{f_{c}}$$

$l_{b}$ required embedment length
$d_{b}$ anchor diameter
$c$ spacing or cover dimension of anchor bars
$f_{y,a}$ yielding stress of reinforcement steel ($394.4 \cdot 10^{3}/1005$)
$f_{c}^{'}, f_{c}^{'}, f_{c}$ compressive strength of concrete C35/45 (assumed)

$$d_{b} = 16 \text{ mm}$$

$$f_{y,a} = 494.4 \text{ N/mm}^{2}$$

$$f_{c}^{'}, f_{c}^{'}, f_{c} = 72 \text{ N/mm}^{2}$$

$$f_{c} = 494.4 \text{ N/mm}^{2}$$

R. Lieven
The anchors will have an embedment length of 420 mm according to configuration of EC2.

**Weld of the anchor bars**
The anchor bars will be welded to a steel plate using a fillet weld and will lead the forces from the concrete infill panel into the steel connection. Each anchor has to transfer a maximum force of 98.9 kN (494.4/5) through the weld into the steel plate.

![Figure D2.5 - Overview anchor position and effective length welds.](image)

\[
f_{w,d} = \frac{500}{\sqrt[3]{0.8 \cdot 1.25}} = 288.7 \text{ N/mm}^2
\]

\[
F_{w,Rd} = 288.7 \cdot 6 = 1732.1 \text{ N/mm}
\]

\[
l_{eff} = 2 \cdot \pi \cdot r_w = 2 \cdot \pi \left( 8 + \frac{1}{2} \cdot \frac{6}{3} \right) = 63.6 \text{ mm}
\]

\[
F_{w,Rd} = 1732.1 \cdot 63.6 \cdot 10^{-3} = 110.2 \text{ kN}
\]

\[
\frac{98.9}{110.2} = 0.9 \leq 1
\]

**The weld of the steel T-section**
The steel plate with the welded anchors cast in the concrete infill panel will be welded to a second steel gusset plate, mentioned in the previous section. A nominal weld throat of 5 mm will be used to distribute the forces from the anchors to the steel gusset plate.

The average maximum force on the anchor bar is 1140.2 kN as stated in the previous section. This will result in a necessary weld length between the two steel plates using the following formulas:

\[
F_{w,Rd} = f_{w,d} \cdot a
\]

\[
f_{w,d} = \frac{f_u}{\sqrt{3} \cdot \beta_w \cdot \gamma_{M2}}
\]
Figure D2.6 - Weld length of steel plate.

\[ f_{w,d} = \frac{314}{\sqrt{3 \cdot 0.8 \cdot 1.25}} = 181.3 \text{ N/mm}^2 \]

\[ F_{w,Rd} = 181.3 \cdot 5 = 906.4 \text{ N/mm} \]

\[ l_{\text{min}} = \frac{5 \cdot 98.9 \cdot 10^3}{2 \cdot 906.4} = \frac{494.4 \cdot 10^3}{2 \cdot 906.4} = 273 \text{ mm} \]

The fillet weld at each side of the plate has to be at least 273 mm. The weld length is 398 mm per side.
Appendix E  Ovalisation bolt holes
E1 Ovalisation bolt holes anchor plate two

Figure E1.1 – Bolt hole location Anchor plate 2 (backside view).

Table E1.1 – Ovalisation bolt holes anchor plate 2 test A2.

<table>
<thead>
<tr>
<th>Corner</th>
<th>Bolt hole</th>
<th>Back side [mm]</th>
<th>Front side [mm]</th>
<th>Average [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Left</td>
<td>1.78</td>
<td>1.60</td>
<td>1.7</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>1.88</td>
<td>1.66</td>
<td>1.8</td>
</tr>
<tr>
<td>2</td>
<td>Left</td>
<td>1.07</td>
<td>0.92</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>1.91</td>
<td>1.74</td>
<td>1.8</td>
</tr>
<tr>
<td>3</td>
<td>Left</td>
<td>1.93</td>
<td>0.97</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>1.31</td>
<td>1.38</td>
<td>1.4</td>
</tr>
<tr>
<td>4</td>
<td>Left</td>
<td>3.20</td>
<td>2.28</td>
<td>2.7</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>3.39</td>
<td>2.35</td>
<td>2.9</td>
</tr>
</tbody>
</table>

Table E1.2 – Ovalisation bolt holes anchor plate 2 test A3.

<table>
<thead>
<tr>
<th>Corner</th>
<th>Bolt hole</th>
<th>Back side [mm]</th>
<th>Front side [mm]</th>
<th>Average [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Left</td>
<td>2.13</td>
<td>1.27</td>
<td>1.7</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>2.46</td>
<td>1.53</td>
<td>2.0</td>
</tr>
<tr>
<td>2</td>
<td>Left</td>
<td>2.33</td>
<td>2.02</td>
<td>2.2</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>1.38</td>
<td>0.98</td>
<td>1.2</td>
</tr>
<tr>
<td>3</td>
<td>Left</td>
<td>3.28</td>
<td>3.04</td>
<td>3.2</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>3.66</td>
<td>2.18</td>
<td>2.9</td>
</tr>
<tr>
<td>4</td>
<td>Left</td>
<td>2.36</td>
<td>0.32</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>-0.08</td>
<td>2.33</td>
<td>1.1</td>
</tr>
</tbody>
</table>
Table E1.3 – Ovalisation bolt holes anchor plate 2 test B2.

<table>
<thead>
<tr>
<th>Corner</th>
<th>Bolt hole</th>
<th>Back side [mm]</th>
<th>Front side [mm]</th>
<th>Average [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Left</td>
<td>6.05</td>
<td>4.38</td>
<td>5.2</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>6.33</td>
<td>4.52</td>
<td>5.4</td>
</tr>
<tr>
<td>2</td>
<td>Left</td>
<td>0.98</td>
<td>0.82</td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>0.92</td>
<td>0.80</td>
<td>0.9</td>
</tr>
<tr>
<td>3</td>
<td>Left</td>
<td>1.76</td>
<td>1.55</td>
<td>1.7</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>1.14</td>
<td>0.96</td>
<td>1.1</td>
</tr>
<tr>
<td>4</td>
<td>Left</td>
<td>6.17</td>
<td>5.28</td>
<td>6.2</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>6.27</td>
<td>4.82</td>
<td>5.6</td>
</tr>
</tbody>
</table>

Table E1.4 – Ovalisation bolt holes anchor plate 2 test B3.

<table>
<thead>
<tr>
<th>Corner</th>
<th>Bolt hole</th>
<th>Back side [mm]</th>
<th>Front side [mm]</th>
<th>Average [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Left</td>
<td>2.87</td>
<td>3.70</td>
<td>3.29</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>1.70</td>
<td>2.41</td>
<td>2.06</td>
</tr>
<tr>
<td>2</td>
<td>Left</td>
<td>6.07</td>
<td>5.55</td>
<td>5.81</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>7.96</td>
<td>6.77</td>
<td>7.37</td>
</tr>
<tr>
<td>3</td>
<td>Left</td>
<td>7.17</td>
<td>6.17</td>
<td>6.67</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>7.04</td>
<td>5.32</td>
<td>6.18</td>
</tr>
<tr>
<td>4</td>
<td>Left</td>
<td>2.61</td>
<td>3.68</td>
<td>3.15</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>3.33</td>
<td>3.84</td>
<td>3.59</td>
</tr>
</tbody>
</table>
E2  Ovalisation bolt holes gusset plate

Figure E2.1 – Bolt hole location gusset plate (backside view). a) Corner 2 & 3, b) Corner 1 & 4

Table E2.1 – Ovalisation bolt holes gusset plate test A2.

<table>
<thead>
<tr>
<th>Corner</th>
<th>Bolt hole</th>
<th>Back side [mm]</th>
<th>Front side [mm]</th>
<th>Average [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Left</td>
<td>0.30</td>
<td>0.59</td>
<td>0.5</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>0.31</td>
<td>0.66</td>
<td>0.5</td>
</tr>
<tr>
<td>2</td>
<td>Left</td>
<td>0.19</td>
<td>0.45</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>0.13</td>
<td>0.52</td>
<td>0.3</td>
</tr>
<tr>
<td>3</td>
<td>Left</td>
<td>0.24</td>
<td>0.43</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>0.02</td>
<td>0.24</td>
<td>0.1</td>
</tr>
<tr>
<td>4</td>
<td>Left</td>
<td>0.19</td>
<td>0.98</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>0.34</td>
<td>0.83</td>
<td>0.6</td>
</tr>
</tbody>
</table>

Table E2.2 – Ovalisation bolt holes gusset plate test A3.

<table>
<thead>
<tr>
<th>Corner</th>
<th>Bolt hole</th>
<th>Back side [mm]</th>
<th>Front side [mm]</th>
<th>Average [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Left</td>
<td>0.18</td>
<td>0.14</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>0.11</td>
<td>0.09</td>
<td>0.1</td>
</tr>
<tr>
<td>2</td>
<td>Left</td>
<td>0.14</td>
<td>0.51</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>0.28</td>
<td>0.60</td>
<td>0.4</td>
</tr>
<tr>
<td>3</td>
<td>Left</td>
<td>0.21</td>
<td>0.97</td>
<td>0.6</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>0.33</td>
<td>0.89</td>
<td>0.6</td>
</tr>
<tr>
<td>4</td>
<td>Left</td>
<td>0.24</td>
<td>0.11</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>0.19</td>
<td>0.12</td>
<td>0.2</td>
</tr>
</tbody>
</table>
### Table E2.3 – Ovalisation bolt holes gusset plate test B2.

<table>
<thead>
<tr>
<th>Corner</th>
<th>Bolt hole</th>
<th>Back side [mm]</th>
<th>Front side [mm]</th>
<th>Average [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Left</td>
<td>0.71</td>
<td>1.81</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>0.37</td>
<td>1.18</td>
<td>0.8</td>
</tr>
<tr>
<td>2</td>
<td>Left</td>
<td>0.03</td>
<td>0.16</td>
<td>0.1</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>0.08</td>
<td>0.03</td>
<td>0.1</td>
</tr>
<tr>
<td>3</td>
<td>Left</td>
<td>0.11</td>
<td>0.53</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>0.12</td>
<td>0.16</td>
<td>0.1</td>
</tr>
<tr>
<td>4</td>
<td>Left</td>
<td>0.59</td>
<td>1.92</td>
<td>1.3</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>0.53</td>
<td>1.58</td>
<td>1.1</td>
</tr>
</tbody>
</table>

### Table E2.4 – Ovalisation bolt holes gusset plate test B3.

<table>
<thead>
<tr>
<th>Corner</th>
<th>Bolt hole</th>
<th>Back side [mm]</th>
<th>Front side [mm]</th>
<th>Average [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Left</td>
<td>0.18</td>
<td>0.45</td>
<td>0.32</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>-0.02</td>
<td>0.68</td>
<td>0.33</td>
</tr>
<tr>
<td>2</td>
<td>Left</td>
<td>0.52</td>
<td>1.46</td>
<td>0.99</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>0.46</td>
<td>2.85</td>
<td>1.66</td>
</tr>
<tr>
<td>3</td>
<td>Left</td>
<td>0.44</td>
<td>1.41</td>
<td>0.93</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>0.46</td>
<td>3.58</td>
<td>2.02</td>
</tr>
<tr>
<td>4</td>
<td>Left</td>
<td>0.13</td>
<td>1.01</td>
<td>0.57</td>
</tr>
<tr>
<td></td>
<td>Right</td>
<td>0.14</td>
<td>0.37</td>
<td>0.26</td>
</tr>
</tbody>
</table>
Appendix F  Finite element model
Construction finite element model

Parameters

Two parametric finite element models are created. The first model represents the bare steel frame, appendix F2. The second model represents the infilled frame, appendix F3. Both models are constructed in the same way, except that the first model does not contain a concrete panel, figure F1.1. Several parameters are added in the input files of the finite element models, figure F1.2, (all four corners are identically constructed. For clarity, this is not drawn). These variables can be changed to analyze different geometries such as, height and width of the frame, dimensions of the steel sections and concrete panel, size of the frame panel connections and its position compared to the steel frame. There are also several locations named such as, upper beam, panel middle and corner 1. These location indicators are also mentioned in the input files for a better insight into the location of the Keypoints, areas, lines and nodes at the finite element model.

Post processing

During the finite element analysis displacements and deformations at several locations of the specimen will be recorded, figure F1.3. These recording locations are an equivalent of the measurement units m-00 till m-04/A1-14, s.g.7-00, s.g.7-16, s.g.8-08 till s.g.8-11, s.g.8-16 and s.g.8-17 at the experimental test specimens, chapter 3. Furthermore all longitudinal springs, \( C_p \) and \( C_s \), are monitored during the solution procedure to get an insight at the spring behaviour during lateral loading of the test specimen. After the solution procedure the results will be summarized in tables and written to a text file. For the bare frame model the text file is named BARE_FRAME.OUTPUT and for the infilled frame model INFILLED_FRAME.OUTPUT.

---

Figure F1.1 – Keypoints of finite element model.
Variables:

- height: height frame
- width: width frame
- section height: height section
- gap: gap between panel and frame
- gusl: length gusset plate
- ancl: length anchor plate 2

Automated:

offset = offset gusset plate = \frac{1}{2} \text{section height}.

Dimensions concrete panel are determined by the variable dimensions above.

Figure F1.2 – Parameters of finite element model.

Figure F1.3 – Measurement arrangement finite element model.
F2  Input file bare frame

FINISH
/CLEAR

/FILENAM,BARE_FRAME

!DATA AFTER SOLUTION PROCEDURE WILL BE EXPORTED TO FILE: BARE_FRAME.OUTPUT
!SET THE CORRECT TORSIONAL SPRING characteristic (R,6) TO GET RESULTS FROM THE DESIRED FRAME
!(BF-A = BARE FRAME A OR BF-B = BARE FRAME B)
!MODELING BY R.LIEVEN

/CONFIG,NRES,10000
/PREP7
!MAXIMUM NUMBER ALLOWED AT RESULTS FILE
!ENTERS THE MODEL CREATION PREPROCESSOR

!GEOMETRY STEEL FRAME
!--------------------------------------
height=3000
width=3000

!GEOMETRY STEEL SECTIONS
!--------------------------------------
secheight=200
elco=30
elbe=30
elof=1
elgap=1

!GEOMETRY GUSSET PLATE
!--------------------------------------
gap=20
gusl=270
elgus=4

!PRESCRIBED DISPLACEMENT
!--------------------------------------
disp=30
nds=100

!ELEMENTS
!--------------------------------------
ET,1,BEAM3
KEYOPT,1,6,0
KEYOPT,1,9,0
KEYOPT,1,10,0

ET,2,PLANE183
KEYOPT,2,1,0
KEYOPT,2,3,3

ET,3,PLANE183
KEYOPT,3,1,1
KEYOPT,3,3,3

ET,4,COMBIN39
KEYOPT,4,1,0
KEYOPT,4,2,0
KEYOPT,4,3,6

!MATERIAL CONSTANTS
!--------------------------------------
MP,EX,1,210000
MP,NUXY,1,0.3

!REAL CONSTANTS
!--------------------------------------
R,1,11325,74830000,200
R,2,11325,74830000,200
Appendix F

TORSIONAL SPRING STIFFNESSES BEAM-COLUMN CONNECTION

| R, 6, 0.00142, 27886125, 0.00680, 60581550 | SPRING (CT) ADJUSTED BF-A |
| R, 6, 0.00274, 27886125, 0.00888, 60581550 | SPRING (CT) THEORETICAL BF-A |
| R, 6, 0.000887, 27886125, 0.001040, 60581550 | SPRING (CT) FMN BF-A |
| R, 6, 0.000100, 20338050, 0.00785, 58271625 | SPRING (CT) ADJUSTED BF-B |
| R, 6, 0.00197, 20338050, 0.00977, 58271625 | SPRING (CT) THEORETICAL BF-B |
| R, 6, 0.000647, 20338050, 0.001854, 58271625 | SPRING (CT) FMN BF-B |

STEEL FRAME

K, 1, 0, height, 0
K, 2, width, height, 0
K, 3, 0, 0, 0
K, 4, width, 0, 0
K, 5, secheight/2, height, 0
K, 6, secheight/2, height, 0
K, 7, width-secheight/2, height, 0
K, 8, width-secheight/2, height, 0
K, 9, secheight/2, 0, 0
K, 10, secheight/2, 0, 0
K, 11, width-secheight/2, 0, 0
K, 12, width-secheight/2, 0, 0
K, 13, secheight/2+gap, height, 0
K, 14, secheight/2+gap+gus1, height, 0
K, 15, width-secheight/2+gap-gus1, height, 0
K, 16, width-secheight/2+gap, height, 0
K, 17, secheight/2+gap, 0, 0
K, 18, secheight/2+gap+gus1, 0, 0
K, 19, width-secheight/2+gap-gus1, 0, 0
K, 20, width-secheight/2+gap, 0, 0

GUSSET PLATE

K, 21, secheight/2+gap, height-secheight/2, 0
K, 22, secheight/2+gap+gus1, height-secheight/2, 0
K, 23, width-secheight/2+gap-gus1, height-secheight/2, 0
K, 24, width-secheight/2+gap, height-secheight/2, 0
K, 25, width-secheight/2+gap, height-secheight/2-gus1, 0
K, 26, width-secheight/2+gap-gus1, height-secheight/2, 0
K, 27, secheight/2+gap, secheight/2, 0
K, 28, secheight/2+gap, secheight/2+gus1, 0
K, 29, secheight/2+gap+gus1, secheight/2, 0
K, 30, width-secheight/2+gap, secheight/2, 0
K, 31, width-secheight/2+gap+gus1, secheight/2, 0
K, 32, width-secheight/2+gap, secheight/2+gus1, 0

COLUMNS

LSTR, 1.3
LSTR, 2.4

BEAMS

LSTR, 14.15
LSTR, 18.19
LSTR, 1.5
LSTR, 7.2
Eindhoven University of Technology

!------------------------------------------------------------------------------------------------
!

<table>
<thead>
<tr>
<th>AMOUNT OF ELEMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>LSIZE, 1, height/elco</td>
</tr>
<tr>
<td>LSIZE, 2, height/elco</td>
</tr>
<tr>
<td>LSIZE, 3, (width-secheight-gap-gap-gusl-gusl)/elbe</td>
</tr>
<tr>
<td>LSIZE, 4, (width-secheight-gap-gap-gusl-gusl)/elbe</td>
</tr>
<tr>
<td>LSIZE, 5, (secheight/2)/elof</td>
</tr>
<tr>
<td>LSIZE, 6, (secheight/2)/elof</td>
</tr>
<tr>
<td>LSIZE, 7, (secheight/2)/elof</td>
</tr>
<tr>
<td>LSIZE, 8, (secheight/2)/elof</td>
</tr>
<tr>
<td>LSIZE, 9, gap/elgap</td>
</tr>
<tr>
<td>LSIZE, 10, gap/elgap</td>
</tr>
<tr>
<td>LSIZE, 11, gap/elgap</td>
</tr>
<tr>
<td>LSIZE, 12, gap/elgap</td>
</tr>
<tr>
<td>LSIZE, 13, gusl/elgus</td>
</tr>
<tr>
<td>LSIZE, 14, gusl/elgus</td>
</tr>
<tr>
<td>LSIZE, 15, gusl/elgus</td>
</tr>
<tr>
<td>LSIZE, 16, gusl/elgus</td>
</tr>
</tbody>
</table>

!------------------------------------------------------------------------------------------------
!

<table>
<thead>
<tr>
<th>GUSSET PLATE</th>
</tr>
</thead>
<tbody>
<tr>
<td>A, 13, 14, 22, 21</td>
</tr>
<tr>
<td>A, 15, 16, 24, 26</td>
</tr>
<tr>
<td>A, 17, 27, 29, 18</td>
</tr>
<tr>
<td>A, 19, 31, 30, 20</td>
</tr>
<tr>
<td>A, 21, 22, 23</td>
</tr>
<tr>
<td>A, 24, 25, 26</td>
</tr>
<tr>
<td>A, 27, 28, 29</td>
</tr>
<tr>
<td>A, 30, 31, 32</td>
</tr>
</tbody>
</table>

!------------------------------------------------------------------------------------------------
!

<table>
<thead>
<tr>
<th>AMOUNT OF ELEMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>AESIZE, 1, gusl/elgus</td>
</tr>
<tr>
<td>AESIZE, 2, gusl/elgus</td>
</tr>
<tr>
<td>AESIZE, 3, gusl/elgus</td>
</tr>
<tr>
<td>AESIZE, 4, gusl/elgus</td>
</tr>
<tr>
<td>AESIZE, 5, gusl/elgus</td>
</tr>
<tr>
<td>AESIZE, 6, gusl/elgus</td>
</tr>
<tr>
<td>AESIZE, 7, gusl/elgus</td>
</tr>
<tr>
<td>AESIZE, 8, gusl/elgus</td>
</tr>
</tbody>
</table>

!------------------------------------------------------------------------------------------------
!

<table>
<thead>
<tr>
<th>COLUMN</th>
</tr>
</thead>
<tbody>
<tr>
<td>MAT. 1</td>
</tr>
<tr>
<td>REAL. 1</td>
</tr>
<tr>
<td>TYPE, 1</td>
</tr>
<tr>
<td>LMESS, 1, 2</td>
</tr>
</tbody>
</table>

!------------------------------------------------------------------------------------------------
!

<table>
<thead>
<tr>
<th>BEAM, GAP &amp; GUSSET PLATE</th>
</tr>
</thead>
<tbody>
<tr>
<td>MAT. 1</td>
</tr>
<tr>
<td>REAL. 2</td>
</tr>
</tbody>
</table>
Appendix F

TYPE, 1
LMESH, 3, 4
LMESH, 9, 16

RIGID OFFSET BEAM-COLUMN

MAT, 1
REAL, 3
TYPE, 1
LMESH, 5, 8

OFFSET GUSSET PLATE

MAT, 1
REAL, 4
TYPE, 2
AMESH, 1, 4

GUSSET PLATE

MAT, 1
REAL, 5
TYPE, 3
AMESH, 5, 8

BEAM-COLUMN (KEYPOINTS 5/6 8/7 9/10 12/11)

SELTOL, 0.0001
NSEL, S, LOC, X, secheight/2
NSEL, R, LOC, Y, height
*GET, node1, NODE, 0, NUM, MIN
*GET, node2, NODE, 0, NUM, MAX
NSEL, ALL

NSEL, S, LOC, X, width-secheight/2
NSEL, R, LOC, Y, height
*GET, node3, NODE, 0, NUM, MIN
*GET, node4, NODE, 0, NUM, MAX
NSEL, ALL

NSEL, S, LOC, X, secheight/2
NSEL, R, LOC, Y, 0
*GET, node5, NODE, 0, NUM, MIN
*GET, node6, NODE, 0, NUM, MAX
NSEL, ALL

NSEL, S, LOC, X, width-secheight/2
NSEL, R, LOC, Y, 0
*GET, node7, NODE, 0, NUM, MIN
*GET, node8, NODE, 0, NUM, MAX
NSEL, ALL

CONSTRANTS ON FRAME

node9=NODE(0, height, 0) !CORNER 1
node10=NODE(width, height, 0) !CORNER 2
node11=NODE(0, 0, 0) !CORNER 3
node12=NODE(width, 0, 0) !CORNER 4

TORSIONAL SPRING BEAM-COLUMN

MAT, 1
REAL, 6
TYPE, 4
E, node1, node2
E, node3, node4
E, node3, node6
E, node7, node8
Eindhoven University of Technology

! BEAM-COLUMN

CP, 1, UX, node1, node2
CP, 2, UY, node1, node2
CP, 3, UX, node3, node4
CP, 4, UY, node3, node4
CP, 5, UX, node5, node6
CP, 6, UY, node5, node6
CP, 7, UX, node7, node8
CP, 8, UY, node7, node8

! SUPPORTS

D, node11, UY
D, node12, UY
D, node12, UX

!/ SOLU

*DO, var, 1, nds
D, node9, UX, (disp/nds)*var
SOLVE
*ENDDO

! REACTION FORCES & DISPLACEMENT CORNER 3

*DIM, tabout_m00, TABLE, nds, 2, 1
*DO, var, 1, nds
SET, var, LAST
*GET, Displacement_m00, NODE, node9, U, X
*GET, Force_m00, NODE, node9, RF, FX
*VFILL, tabout_m00(var, 1), DATA, Force_m00
*VFILL, tabout_m00(var, 2), DATA, Displacement_m00
*ENDDO

! DISPLACEMENT CORNER 2

*DIM, tabout_m02, TABLE, nds, 2, 1
*DO, var, 1, nds
SET, var, LAST
*GET, Displacement_m02, NODE, node10, U, X
*GET, Force_m02, NODE, node9, RF, FX
*VFILL, tabout_m02(var, 1), DATA, Force_m02
*VFILL, tabout_m02(var, 2), DATA, Displacement_m02
*ENDDO

! DISPLACEMENT CORNER 1

*DIM, tabout_m01, TABLE, nds, 2, 1
*DO, var, 1, nds
SET, var, LAST
*GET, Displacement_m01, NODE, node9, U, X
*GET, Force_m01, NODE, node9, RF, FX
*VFILL, tabout_m01(var, 1), DATA, Force_m01
*VFILL, tabout_m01(var, 2), DATA, Displacement_m01
*ENDDO

! SOLUTION PROCEDURE

ANTYPE, STATIC
NLGEOM, ON
AUTOTS, ON
NSUBST, nds, 1000, 1
OUTRES, ALL, ALL
LNSRCH, AUTO
CNVTOL, F, ,0.0005, ,1
CNVTOL, U, ,0.0005, ,1

/SOLU

ANTYPE, STATIC
NLGEOM, ON
AUTOTS, ON
NSUBST, nds, 1000, 1
OUTRES, ALL, ALL
LNSRCH, AUTO
CNVTOL, F, ,0.0005, ,1
CNVTOL, U, ,0.0005, ,1

*DO, var, 1, nds
D, node9, UX, (disp/nds)*var
SOLVE
*ENDDO

! POSTPROCESS

ETABLE, F_SEC, SMISC, 7
ETABLE, M_SEC, SMISC, 12

! DISPLACEMENT CORNER 3

*DIM, tabout_m03, TABLE, nds, 2, 1
*DO, var, 1, nds
SET, var, LAST
*GET, Displacement_m03, NODE, node10, U, X
*GET, Force_m03, NODE, node9, RF, FX
*VFILL, tabout_m03(var, 1), DATA, Force_m03
*VFILL, tabout_m03(var, 2), DATA, Displacement_m03
*ENDDO

! STATIC ANALYSIS

NON-LINEAR GEOMETRY ANALYSIS
AUTO TIME STEPPING NOT WITH ARC-LENGTH
NUMBER OF SUBSTEPS, NUMBER, MAX, MIN
SAVE RESULTS OF ALL ITERATIONS
LINE SEARCH USED ONLY WITH NEWTON-RAPHSON

*VFILL, tabout_m01(v a r,1),DATA,Force_m01
*VFILL, tabout_m01(v ar,2),DATA,Displacement_m01

*VFILL, tabout_m02(v ar,1),DATA,Force_m02
*VFILL, tabout_m02(v ar,2),DATA,Displacement_m02

*VFILL, tabout_m03(v ar,1),DATA,Force_m03
*VFILL, tabout_m03(v ar,2),DATA,Displacement_m03

! LATERAL LOAD (L.C.)

! LATERAL LOAD (L.C.)
*DO, var, 1, nds
SET, var, LAST
*GET, Displacement_m00, NODE, node11, U, X
*GET, Force_m00, NODE, node11, RF, FX
*VFILL, tabout_m00(var,1), DATA, Force_m00
*VFILL, tabout_m00(var,2), DATA, Displacement_m00
*ENDO

*DIM, tabout_m04, TABLE, nds, 2, 1
*DO, var, 1, nds
SET, var, LAST
*GET, Displacement_m04, NODE, node11, U, Y
*GET, Force_m04, NODE, node11, RF, FY
*VFILL, tabout_m04(var,1), DATA, Displacement_m04
*VFILL, tabout_m04(var,2), DATA, Force_m04
*ENDO

!----------------------------------------------------------------------------------------------------
!REACTION FORCE CORNER 4
!----------------------------------------------------------------------------------------------------
*DIM, tabout_m03, TABLE, nds, 2, 1
*DO, var, 1, nds
SET, var, LAST
*GET, Force_m03x, NODE, node12, RF, FX
*GET, Force_m03y, NODE, node12, RF, FY
*VFILL, tabout_m03(var,1), DATA, Force_m03x
*VFILL, tabout_m03(var,2), DATA, Force_m03y
*ENDO

!NORMAL FORCES AND MOMENTS IN COLUMNS AND BEAMS
!----------------------------------------------------------------------------------------------------
*DIM, tabout_sg0809, TABLE, nds, 2, 1
*DO, var, 1, nds
SET, var, LAST
*GET, Force_sg0809, ELEM, (elco/2), SMISC, 7
*GET, Moment_sg0809, ELEM, (elco/2), SMISC, 12
*VFILL, tabout_sg0809(var,1), DATA, Force_sg0809
*VFILL, tabout_sg0809(var,2), DATA, Moment_sg0809
*ENDO

*DIM, tabout_sg1011, TABLE, nds, 2, 1
*DO, var, 1, nds
SET, var, LAST
*GET, Force_sg1011, ELEM, (elco*2)+(elbe/2), SMISC, 7
*GET, Moment_sg1011, ELEM, (elco*2)+(elbe/2), SMISC, 12
*VFILL, tabout_sg1011(var,1), DATA, Force_sg1011
*VFILL, tabout_sg1011(var,2), DATA, Moment_sg1011
*ENDO

*DIM, tabout_sg1617, TABLE, nds, 2, 1
*DO, var, 1, nds
SET, var, LAST
*GET, Force_sg1617, ELEM, (elco*1.5), SMISC, 7
*GET, Moment_sg1617, ELEM, (elco*1.5), SMISC, 12
*VFILL, tabout_sg1617(var,1), DATA, Force_sg1617
*VFILL, tabout_sg1617(var,2), DATA, Moment_sg1617
*ENDO

*DIM, tabout_sg0016, TABLE, nds, 2, 1
*DO, var, 1, nds
SET, var, LAST
*GET, Force_sg0016, ELEM, (elco*2)+(elbe*1.5), SMISC, 7
*GET, Moment_sg0016, ELEM, (elco*2)+(elbe*1.5), SMISC, 12
*VFILL, tabout_sg0016(var,1), DATA, Force_sg0016
*VFILL, tabout_sg0016(var,2), DATA, Moment_sg0016
*ENDO

!----------------------------------------------------------------------------------------------------
!DISPLACEMENT CORNER 1
!----------------------------------------------------------------------------------------------------
*WRITE, ('DISPLACEMENT CORNER 1')
*WRITE, ('Force x [N]', 'Displacement x (m-01) [mm]')
*WRITE, tabout_m01(1,1), tabout_m01(1,2)
(E18.10, ',E18.10)
!DISPLACEMENT CORNER 2

*WRITE,
  (' ')
*WRITE,
  (' ')
*WRITE,
  ('DISPLACEMENT CORNER 2')
*WRITE,
  ('  ', 'Force x [N]', ', 'Displacement x (m-02 / A1-14) [mm]')
*WRITE, tabout_m02(1,1), tabout_m02(1,2)
  (E18.10, ', E18.10, )

!REACTION FORCES & DISPLACEMENT CORNER 3

*WRITE,
  (' ')
*WRITE,
  (' ')
*WRITE,
  ('REACTION FORCES CORNER 3')
*WRITE,
  ('  ', 'Reaction force x [N]', ', 'Displacement x (m-00) [mm]')
  ', 'Reaction force y [N]', ', 'Displacement y (m-04) [N]'
*WRITE, tabout_m00(1,1), tabout_m00(1,2), tabout_m04(1,1), tabout_m04(1,2)
  (E18.10, ', E18.10, ', E18.10, ', E18.10, )

!REACTION FORCES CORNER 4

*WRITE,
  (' ')
*WRITE,
  (' ')
*WRITE,
  ('REACTION FORCES CORNER 4')
*WRITE,
  ('  ', 'Force x [N]', ', 'Force y [N]')
*WRITE, tabout_m03(1,1), tabout_m03(1,2)
  (E18.10, ', E18.10, )

!NORMAL FORCES AND MOMENTS IN COLUMNS AND BEAMS

*WRITE,
  (' ')
*WRITE,
  (' ')
*WRITE,
  ('NORMAL FORCES AND MOMENTS IN COLUMNS AND BEAMS')
*WRITE,
  ('  ', 'Left column', ', 'Upper beam', ', 'Right column', ', 'Lower beam')
*WRITE,
  ('  ', 'Force [N]', ', 'Moment [Nmm]', ', 'Force [N]', ', 'Moment [Nmm]', , 'Force [N]', ', 'Moment [Nmm]'
  ', 'Force [N]', ', 'Moment [Nmm]'
*WRITE, tabout_sg0809(1,1), tabout_sg0809(1,2), tabout_sg1011(1,1), tabout_sg1011(1,2), tabout_sg1617(1,1), tabout_sg1617(1,2), tabout_sg0016(1,1), tabout_sg0016(1,2)
*CFCLOSE

!OUTPUT GRAPHS

!SHOW, PNG, 0
PNGR, COMP, 0
PNGR, COLOR, 2
/GFILE, 2400,
/AXLAB, X, Displacement_m02 [mm]
/AXLAB, Y, Force_m02 [N]
*VPLOT, tabout_m02(1,1), tabout_m02(1,2)
/SHOW, CLOSE

FINISH
F3  Input file infilled frame

FINISH
/CLEAR

/FIENAM, INFILLED_FRAME

!DATA AFTER SOLUTION PROCEDURE WILL BE EXPORTED TO FILE: INFILLED_FRAME.OUTPUT. THERE OCCURES ONE
!WARNING [Coefficient ratio exceeds 1.0e8 - Check results]. DUE TO LARGE DISPLACEMENTS OF THE
!COMBIN39 ELEMENTS. THIS WARNING HAS NO INFLUENCE ON THE RESULTS AND CAN BE IGNORED.
!GET RESULTS FROM THE DESIRED INFILLED FRAME (IF-A = INFILLED FRAME A OR IF-B = INFILLED FRAME B)
!MODELING BY R.LIEVEN

/CONFIG,NRES,10000  !MAXIMUM NUMBER ALLOWED AT RESULTS FILE
/PSYMB,NDIR,1      !SHOWS NOTE ROTATION

!------------------------------------------------------------------------------------------------

!GEOMETRY STEEL FRAME
!------------------------------------------------------------------------------------------------
height=3000          !HEIGHT FRAME
width=3000           !WIDTH FRAME
secheight=200        !HEIGHT STEEL SECTIONS
elco=30             !NUMBER OF ELEMENTS COLUMNS
elbe=30             !NUMBER OF ELEMENTS BEAMS
elof=1              !NUMBER OF ELEMENTS OFFSETS
elgap=1             !NUMBER OF ELEMENTS GAP

!------------------------------------------------------------------------------------------------

!GEOMETRY GUSSET PLATE
!------------------------------------------------------------------------------------------------
gap=20               !GAP BETWEEN GUSSET PLATE AND COLUMN
gus1=270            !LENGTH GUSSET PLATE & ITS OFFSET
ancl=277            !LENGTH ANCHORPLATE 2
elgus=4             !NUMBER OF ELEMENTS GUSSET PLATE

!------------------------------------------------------------------------------------------------

!GEOMETRY CONCRETE PANEL
!------------------------------------------------------------------------------------------------
elpan=26             !NUMBER OF ELEMENTS CONCRETE PANEL

!------------------------------------------------------------------------------------------------

!PRESCRIBED DISPLACEMENT
!------------------------------------------------------------------------------------------------
disp=40              !DISPLACEMENT [mm]
nds=100             !NUMBER OF DISPLACEMENT STEPS

!------------------------------------------------------------------------------------------------

!ELEMENTS
!------------------------------------------------------------------------------------------------
ET,1,BEAM3           !ELEMENTS FOR COLUMNS & BEAMS
KEYOPT,1,6,0
KEYOPT,1,9,0

ET,2,PLANE183        !ELEMENTS FOR GUSSET PLATE OFFSET
KEYOPT,2,1,0
KEYOPT,2,3,3

ET,3,PLANE183        !ELEMENTS FOR GUSSET PLATE OFFSET
KEYOPT,3,1,1
KEYOPT,3,3,3

ET,4,COMBIN39        !SPRING BETWEEN COLUMNS & BEAMS
KEYOPT,4,1,0
KEYOPT,4,2,0
KEYOPT,4,3,6

ET,5,COMBIN39        !SPRING BETWEEN AP2 & GP X-DIRECTION
KEYOPT,5,3,1

ET,6,COMBIN39        !SPRING BETWEEN AP2 & GP Y-DIRECTION
KEYOPT,6,3,2

!------------------------------------------------------------------------------------------------

R. Lieven
```
!MATERIAL CONSTANTS
!YLNG'S MODULUS STEEL
MP, EX, 1, 210000
MP, NUXY, 1, 0.3

MP, EX, 2, 32127
MP, NUXY, 2, 0.2

MP, EX, 2, 31376
MP, NUXY, 2, 0.2

!REAL CONSTANTS

R, 1, 11325, 74830000, 200
R, 2, 11325, 74830000, 200
R, 3, 11325, 7483000000, 200
R, 4, 15
R, 5, 15
R, 6, 10
R, 7, 200
R, 8, 200

selsp=4000

!STARTING NUMBER OF FPC3 SPRING ELEMENTS

!TORSIONAL SPRING STIFFNESSES

R, 9, 0.00142, 27886125, 0.00680, 60581550
R, 9, 0.00100, 20338050, 0.00785, 58271625

!LONGITUDINAL SPRING STIFFNESSES

R, 10, -15, -540000, 12.97, -540000, 12.97, -540000, 12.97
R, 10, 0.0, 0.0, 5.45, 5.45, 342000, 12
R, 10, 0.0, 0.0, 5.45, 5.45, 342000, 12
R, 10, 0.0, 0.0, 5.45, 5.45, 342000, 12
R, 10, 0.0, 0.0, 5.45, 5.45, 342000, 12

!STEEL FRAME

K, 1, 0, height, 0
K, 2, width, height, 0
K, 3, 0, 0
K, 4, width, 0
K, 5, secheight/2, height, 0
K, 6, secheight/2, height, 0
K, 7, width-secheight/2, height, 0
K, 8, width-secheight/2, height, 0
K, 9, secheight/2, 0, 0
K, 10, secheight/2, 0, 0
K, 11, width-secheight/2, 0, 0
K, 12, width-secheight/2, 0, 0
K, 13, secheight/2+gap, height, 0
K, 14, secheight/2+gap, height, 0
K, 14, secheight/2+gap, height, 0
K, 15, width-secheight/2+gap, height, 0
K, 16, width-secheight/2+gap, height, 0
K, 17, secheight/2+gap, 0, 0
K, 18, secheight/2+gap, 0, 0
K, 19, width-secheight/2+gap, 0, 0
K, 20, width-secheight/2+gap, 0, 0

K, 21, secheight/2+gap, height-secheight/2, 0
K, 22, secheight/2+gap, height-secheight/2, 0
K, 23, secheight/2+gap, height-secheight/2, 0

!GUSSET PLATE

K, 21, secheight/2+gap, height-secheight/2, 0
K, 22, secheight/2+gap, height-secheight/2, 0
K, 23, secheight/2+gap, height-secheight/2, 0
```

Appendix F

!BOLT HOLES GUSSET PLATE

K,24,,width-secheight/2-gap, height-secheight/2,0
K,25,,width-secheight/2-gap, height-secheight/2-gusl,0
K,27,,secheight/2+gap,secheight/2,0
K,28,,secheight/2+gap,secheight/2+gusl,0
K,29,,secheight/2+gap+gusl,secheight/2,0
K,30,,width-secheight/2-gap, secheight/2,0
K,31,,width-secheight/2-gap-gusl,secheight/2,0
K,32,,width-secheight/2-gap,secheight/2+gusl,0

K,33,,secheight/2+gap+55,height-secheight/2-137,0
K,34,,secheight/2+gap+115,height-secheight/2-77,0
K,35,,width-secheight/2-gap-115,height-secheight/2-77,0
K,36,,width-secheight/2-gap-55 , height-secheight /2-137,0
K,37,,secheight/2+gap+55,secheight/2+137,0
K,38,,secheight/2+gap+115,secheight/2+77,0
K,39,,width-secheight/2-gap-115, secheight/2+77,0
K,40,,width-secheight/2-gap-55 ,secheight/2+137,0

!ANCHOR PLATE 2

K,41,,secheight/2+gap, height-secheight/2-20,0
K,42,,secheight/2+gap+ancl, height-secheight/2-20,0
K,43,,secheight/2+gap, height-secheight/2-ancl-20,0
K,44,,width-secheight/2-gap, height-secheight/2-20,0
K,45,,width-secheight/2-gap, height-secheight/2-ancl-20,0
K,46,,width-secheight/2-gap+ancl, height-secheight/2-20,0
K,47,,secheight/2+gap, secheight/2+20,0
K,48,,secheight/2+gap, secheight/2+ancl+20,0
K,49,,width-secheight/2-gap-ancl, secheight/2+ancl+20,0
K,50,,width-secheight/2-gap,secheight/2+20,0
K,51,,width-secheight/2-gap+ancl, secheight/2+20,0
K,52,,width-secheight/2-gap,secheight/2+ancl+20,0

!BOLT HOLES ANCHORPLATE 2

K,53,,secheight/2+gap+55,height-secheight/2-137,0
K,54,,secheight/2+gap+115,height-secheight/2-77,0
K,55,,width-secheight/2-gap-115,height-secheight/2-77,0
K,56,,width-secheight/2-gap-55 ,height-secheight /2-137,0
K,57,,secheight/2+gap+55,secheight/2+137,0
K,58,,secheight/2+gap+115,secheight/2+77,0
K,59,,width-secheight/2-gap-115,secheight/2+77,0
K,60,,width-secheight/2-gap-55 ,secheight/2+137,0

!CONCRETE PANEL

K,61,,secheight/2+gap+ancl, height-secheight/2-ancl-20,0
K,62,,width-secheight/2-gap+ancl, height-secheight/2-ancl-20,0
K,63,,secheight/2+gap+ancl, secheight/2+ancl+20,0
K,64,,width-secheight/2-gap+ancl, secheight/2+ancl+20,0

!KEYPOINTS FOR COORDINATE SYSTEM UNDER 45 DEGREES

K,96,4000,1000,0
K,97,3500,500,0
K,98,4500,500,0
K,99,4000,0,0

R. Lieven
| !COLUMNS |
| LSTR, 1, 3 |
| LSTR, 2, 4 |
| !BEAMS |
| LSTR, 14, 15 |
| LSTR, 18, 19 |
| LSTR, 1, 5 |
| LSTR, 7, 2 |
| LSTR, 3, 9 |
| LSTR, 11, 4 |
| LSTR, 6, 13 |
| LSTR, 16, 8 |
| LSTR, 10, 17 |
| LSTR, 20, 12 |
| LSTR, 13, 14 |
| LSTR, 15, 16 |
| LSTR, 17, 18 |
| LSTR, 19, 20 |

| !AMOUNT OF ELEMENTS |
| LSIZE, 1, height/elco |
| LSIZE, 2, height/elco |
| LSIZE, 3, (width-seceight-gap-gap-gus1-gus1)/elbe |
| LSIZE, 4, (width-seceheight-gap-gap-gus1-gus1)/elbe |
| LSIZE, 5, (seceheight/2)/elof |
| LSIZE, 6, (seceheight/2)/elof |
| LSIZE, 7, (seceheight/2)/elof |
| LSIZE, 8, (seceheight/2)/elof |
| LSIZE, 9, gap/elgap |
| LSIZE, 10, gap/elgap |
| LSIZE, 11, gap/elgap |
| LSIZE, 12, gap/elgap |
| LSIZE, 13, gus1/elgus |
| LSIZE, 14, gus1/elgus |
| LSIZE, 15, gus1/elgus |
| LSIZE, 16, gus1/elgus |

| !GUSSET PLATE |
| A, 13, 14, 22, 21 |
| A, 15, 16, 24, 26 |
| A, 17, 27, 29, 18 |
| A, 19, 31, 30, 20 |
| A, 21, 22, 34, 33, 23 |
| A, 22, 23, 33, 34 |
| A, 24, 25, 36, 35, 26 |
| A, 25, 26, 35, 36 |
| A, 27, 28, 37, 38, 29 |
| A, 28, 29, 38, 37 |
| A, 30, 31, 39, 40, 32 |
| A, 31, 32, 40, 39 |
| AESIZE, 1, gus1/elgus |
| AESIZE, 2, gus1/elgus |
| AESIZE, 3, gus1/elgus |
| AESIZE, 4, gus1/elgus |
| AESIZE, 5, gus1/elgus |
| AESIZE, 6, gus1/elgus |
| AESIZE, 7, gus1/elgus |
| AESIZE, 8, gus1/elgus |
| AESIZE, 9, gus1/elgus |

126
Appendix F

ANCHORPLATE 2

A, 41, 42, 54, 53, 43
A, 42, 43, 53, 54
A, 44, 45, 56, 55, 46
A, 45, 46, 55, 56
A, 47, 48, 57, 58, 49
A, 48, 49, 58, 57
A, 50, 51, 59, 60, 52
A, 51, 52, 60, 59

CONCRETE PANEL

A, 42, 61, 43
A, 45, 62, 46
A, 48, 63, 49
A, 51, 64, 52
A, 43, 61, 63, 48
A, 42, 46, 62, 61
A, 45, 52, 64, 62
A, 49, 63, 64, 51
A, 61, 62, 64, 63

AMOUNT OF ELEMENTS

<table>
<thead>
<tr>
<th>ELEMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>AESIZE, 21, ancl/elgus</td>
</tr>
<tr>
<td>AESIZE, 22, ancl/elgus</td>
</tr>
<tr>
<td>AESIZE, 23, ancl/elgus</td>
</tr>
<tr>
<td>AESIZE, 24, ancl/elgus</td>
</tr>
</tbody>
</table>

COLUMN

MAT, 1
REAL, 1
TYPE, 1
LMESH, 1, 2

BEAM, GAP & GUSSET PLATE

MAT, 1
REAL, 2
TYPE, 1
LMESH, 3, 4
LMESH, 9, 16

RIGID OFFSET BEAM-COLUMN

MAT, 1
REAL, 3
TYPE,1
AMESH,5,8

! OFFSET GUSSET PLATE
MAT,1
REAL,4
TYPE,2
AMESH,1,4

! GUSSET PLATE
MAT,1
REAL,5
TYPE,3
AMESH,5,12

! ANCHORPLATE 2
MAT,1
REAL,6
TYPE,3
AMESH,13,20

! CONCRETE PANEL
MAT,2
REAL,7
TYPE,3
AMESH,21,24
MAT,2
REAL,8
TYPE,2
AMESH,25,29

! BEAM-COLUMN (KEYPOINTS 5/6 8/7 9/10 12/11)
SELTOL,0.0001
NSEL,S,LOC,X,secheight/2 ! CORNER 1 (KEYPOINT 5/6)
NSEL,R,LOC,Y,height
*GET,node1,NODE,O,NUM,MIN
*GET,node2,NODE,O,NUM,MAX
NSEL,ALL
NSEL,S,LOC,X,width-secheight/2 ! CORNER 2 (KEYPOINT 8/7)
NSEL,R,LOC,Y,height
*GET,node3,NODE,O,NUM,MIN
*GET,node4,NODE,O,NUM,MAX
NSEL,ALL
NSEL,S,LOC,X,secheight/2 ! CORNER 3 (KEYPOINT 9/10)
NSEL,R,LOC,Y,0
*GET,node5,NODE,O,NUM,MIN
*GET,node6,NODE,O,NUM,MAX
NSEL,ALL
NSEL,S,LOC,X,width-secheight/2 ! CORNER 4 (KEYPOINT 12/11)
NSEL,R,LOC,Y,0
*GET,node7,NODE,O,NUM,MIN
*GET,node8,NODE,O,NUM,MAX
NSEL,ALL

! CONSTRAINTS ON FRAME
node9=NODE(O,height,0) ! CORNER 1
node10=NODE(width,height,0) ! CORNER 2
node11=NODE(0,0,0) ! CORNER 3
node12=NODE(width,0,0) ! CORNER 4

! ANCHOR PLATE 2 - GUSSET PLATE (KEYPOINTS 33/53 34/54 35/55 36/56 37/57 38/58 39/59 40/60)

128
Appendix F

NSEL, S, LOC, X, secheight/2 + gap + 55
NSEL, R, LOC, Y, height-secheight/2 - 137
*GET, node20, NODE, 0, NUM, MIN
*GET, node21, NODE, 0, NUM, MAX
NSEL, ALL

NSEL, S, LOC, X, secheight/2 + gap + 115
NSEL, R, LOC, Y, height-secheight/2 - 77
*GET, node22, NODE, 0, NUM, MIN
*GET, node23, NODE, 0, NUM, MAX
NSEL, ALL

NSEL, S, LOC, X, width-secheight/2 - gap - 115
NSEL, R, LOC, Y, height-secheight/2 - 77
*GET, node24, NODE, 0, NUM, MIN
*GET, node25, NODE, 0, NUM, MAX
NSEL, ALL

NSEL, S, LOC, X, width-secheight/2 - gap - 55
NSEL, R, LOC, Y, height-secheight/2 - 137
*GET, node26, NODE, 0, NUM, MIN
*GET, node27, NODE, 0, NUM, MAX
NSEL, ALL

NSEL, S, LOC, X, secheight/2 + gap + 55
NSEL, R, LOC, Y, secheight/2 - 137
*GET, node28, NODE, 0, NUM, MIN
*GET, node29, NODE, 0, NUM, MAX
NSEL, ALL

NSEL, S, LOC, X, secheight/2 + gap + 115
NSEL, R, LOC, Y, secheight/2 - 77
*GET, node30, NODE, 0, NUM, MIN
*GET, node31, NODE, 0, NUM, MAX
NSEL, ALL

NSEL, S, LOC, X, width-secheight/2 - gap - 115
NSEL, R, LOC, Y, secheight/2 - 77
*GET, node32, NODE, 0, NUM, MIN
*GET, node33, NODE, 0, NUM, MAX
NSEL, ALL

NSEL, S, LOC, X, width-secheight/2 - gap - 55
NSEL, R, LOC, Y, secheight/2 - 137
*GET, node34, NODE, 0, NUM, MIN
*GET, node35, NODE, 0, NUM, MAX
NSEL, ALL

!---------------------------------------------------------------------------------------------
! CORNER 1 (KEYPOINT 33/53)

!CORNER 1 (KEYPOINT 34/54)

![CORNER 2 (KEYPOINT 35/55)]

![CORNER 2 (KEYPOINT 36/56)]

![CORNER 3 (KEYPOINT 37/57)]

![CORNER 3 (KEYPOINT 38/58)]

![CORNER 4 (KEYPOINT 39/59)]

![CORNER 4 (KEYPOINT 40/60)]

!---------------------------------------------------------------------------------------------

! CORNER 1 (KEYPOINT 33/53)

! CORNER 1 (KEYPOINT 34/54)

! CORNER 2 (KEYPOINT 35/55)

! CORNER 2 (KEYPOINT 36/56)

! CORNER 3 (KEYPOINT 37/57)

! CORNER 3 (KEYPOINT 38/58)

! CORNER 4 (KEYPOINT 39/59)

! CORNER 4 (KEYPOINT 40/60)

!---------------------------------------------------------------------------------------------

!creates and activates local coordinate system (45 degrees)

!creates and activates local coordinate system (45 degrees)

!activates default coordinate system

!mesh springs

R. Lieven 129
TORSIONAL SPRING BEAM-COLUMN

MAT, 1
REAL, 9
TYPE, 4
E, node1, node2
E, node3, node4
E, node5, node6
E, node7, node8

TRANSLATION SPRING COMPRESSION DIAGONAL [CORNER 1]

NUMSTR, ELEM, selsp
MAT, 1
REAL, 11
TYPE, 6
E, node20, node21
E, node22, node23
MAT, 1
REAL, 12
TYPE, 5
E, node20, node21
E, node22, node23

TRANSLATION SPRING TENSION DIAGONAL [CORNER 2]

MAT, 1
REAL, 13
TYPE, 6
E, node24, node25
E, node26, node27
MAT, 1
REAL, 10
TYPE, 5
E, node24, node25
E, node26, node27

TRANSLATION SPRING TENSION DIAGONAL [CORNER 3]

MAT, 1
REAL, 13
TYPE, 6
E, node28, node29
E, node30, node31
MAT, 1
REAL, 10
TYPE, 5
E, node28, node29
E, node30, node31

TRANSLATION SPRING COMPRESSION DIAGONAL [CORNER 4]

MAT, 1
REAL, 11
TYPE, 6
E, node32, node33
E, node34, node35
MAT, 1
REAL, 12
TYPE, 5
E, node32, node33
E, node34, node35
Appendix F

---

**BEAM-COLUMN**

<table>
<thead>
<tr>
<th>Constraint</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>CP,1, UX, node1, node2</td>
<td>CORNER 1</td>
</tr>
<tr>
<td>CP,2, UX, node3, node4</td>
<td>CORNER 2</td>
</tr>
<tr>
<td>CP,3, UX, node5, node6</td>
<td>CORNER 3</td>
</tr>
<tr>
<td>CP,4, UX, node7, node8</td>
<td>CORNER 4</td>
</tr>
</tbody>
</table>

---

**SUPPORTS**

- D, node1, UY
- D, node12, UY
- D, node12, UX

---

**SOLUTION PROCEDURE**

/SOLU
ANTYPE,STATIC
NLGEOM,ON
NROPT,MODI
AUTOTS,ON
NSUBST,nds,1000,1
OUTRES,ALL,ALL
LNSrch,AUTO
CVTOL,F,0.005,1
CVTOL,U,0.005,1

*DO, var, 1, nds
D, node9, UX, (disp/nds)*var
SOLVE
*ENDDO

---

**DISPLACEMENT CORNER 1**

*DIM, tabout_m01, TABLE, nds, 2, 1
*DO, var, 1, nds
SET, var, LAST
*GET, Displacement_m01, NODE, node9, U, X
*GET, Force_m01, NODE, node9, RF, FX
*VFILL, tabout_m01(var, 1), DATA, Force_m01
*VFILL, tabout_m01(var, 2), DATA, Displacement_m01
*ENDDO

---

**DISPLACEMENT CORNER 2**

*DIM, tabout_m02, TABLE, nds, 2, 1
*DO, var, 1, nds
SET, var, LAST
*GET, Displacement_m02, NODE, node10, U, X
*GET, Force_m02, NODE, node9, RF, FX
*VFILL, tabout_m02(var, 1), DATA, Force_m02
*VFILL, tabout_m02(var, 2), DATA, Displacement_m02
*ENDDO

---

**REACTION FORCES & DISPLACEMENT CORNER 3**

*DIM, tabout_m00, TABLE, nds, 2, 1
*DO, var, 1, nds

---

R. Lieven
Eindhoven University of Technology

*GET, Displacement_m00, NODE, node11, U, X

*GET, Force_m00, NODE, node11, RF, FX

VFILL, tabout_m00(var,1), DATA, Force_m00
VFILL, tabout_m00(var,2), DATA, Displacement_m00

ENDDO

*GET, Displacement_m04, NODE, node11, U, Y

*GET, Force_m04, NODE, node11, RF, FY

VFILL, tabout_m04(var,1), DATA, Force_m04
VFILL, tabout_m04(var,2), DATA, Displacement_m04

ENDDO

!------------------------------------------------------------------------------------------------
!
!
!
!REACTION FORCE CORNER
!
!
!

*DIM, tabout_m03, TABLE, nds, 2, 1

*DO, var, 1, nds

SET, var, LAST

*GET, Force_m03X, NODE, node12, RF, FY

*GET, Force_m03Y, NODE, node12, RF, FY

VFILL, tabout_m03(var,1), DATA, Force_m03x
VFILL, tabout_m03(var,2), DATA, Force_m03y

ENDDO

!------------------------------------------------------------------------------------------------
!
!
!
!NORMAL FORCES AND MOMENTS IN COLUMNS AND BEAMS
!
!
!

*DIM, tabout_sg0809, TABLE, nds, 2, 1

*DO, var, 1, nds

SET, var, LAST

*GET, Force_sg0809, ELEM, (elco/2), SMISC, 7

*GET, Moment_sg0809, ELEM, (elco/2), SMISC, 12

VFILL, tabout_sg0809(var,1), DATA, Force_sg0809
VFILL, tabout_sg0809(var,2), DATA, Moment_sg0809

ENDDO

*DIM, tabout_sg1011, TABLE, nds, 2, 1

*DO, var, 1, nds

SET, var, LAST

*GET, Force_sg1011, ELEM, (elco*2)+(elbe/2)), SMISC, 7

*GET, Moment_sg1011, ELEM, (elco*2)+(elbe/2)), SMISC, 12

VFILL, tabout_sg1011(var,1), DATA, Force_sg1011
VFILL, tabout_sg1011(var,2), DATA, Moment_sg1011

ENDDO

*DIM, tabout_sg1617, TABLE, nds, 2, 1

*DO, var, 1, nds

SET, var, LAST

*GET, Force_sg1617, ELEM, (elco*1.5), SMISC, 7

*GET, Moment_sg1617, ELEM, (elco*1.5), SMISC, 12

VFILL, tabout_sg1617(var,1), DATA, Force_sg1617
VFILL, tabout_sg1617(var,2), DATA, Moment_sg1617

ENDDO

*DIM, tabout_sg0016, TABLE, nds, 2, 1

*DO, var, 1, nds

SET, var, LAST

*GET, Force_sg0016, ELEM, (elco*2)+(elbe*1.5)), SMISC, 7

*GET, Moment_sg0016, ELEM, (elco*2)+(elbe*1.5)), SMISC, 12

VFILL, tabout_sg0016(var,1), DATA, Force_sg0016
VFILL, tabout_sg0016(var,2), DATA, Moment_sg0016

ENDDO

!------------------------------------------------------------------------------------------------
!
!
!
!SPRING FPC3 CORNER 1 KEYPOINT 33/53
!
!
!

*DIM, tabout_s33y, TABLE, nds, 2, 1

*DO, var, 1, nds

SET, var, LAST

*GET, Force_s33y, ELEM, selsp+0, SMISC, 1

*GET, Displacement_s33y, ELEM, selsp+0, NMISC, 1

VFILL, tabout_s33y(var,1), DATA, Force_s33y(var,1)
VFILL, tabout_s33y(var,2), DATA, Displacement_s33y

ENDDO

*DIM, tabout_s33x, TABLE, nds, 2, 1

*DO, var, 1, nds

SET, var, LAST

*GET, Force_s33x, ELEM, selsp+2, SMISC, 1

ENDDO

!------------------------------------------------------------------------------------------------

132
*GET,Displacement_s33x,ELEM,selsp+2,NMISC,1 !DISPLACEMENT LOCAL X
*VFILL,tabout_s33x(var,1),DATA,Force_s33x
*VFILL,tabout_s33x(var,2),DATA,Displacement_s33x
*ENDDO

!SPRING FPC3 CORNER 1 KEYPOINT 34/54

*DIM,tabout_s34y,TABLE,nds,2,1
*DO, var, 1, nds
SET, var, LAST
*GET,Force_s34y,ELEM,selsp+1,SMISC,1 !FORCE LOCAL Y
*GET,Displacement_s34y,ELEM,selsp+1,NMISC,1 !DISPLACEMENT LOCAL Y
*VFILL,tabout_s34y(var,1),DATA,Force_s34y
*VFILL,tabout_s34y(var,2),DATA,Displacement_s34y
*ENDDO

*DIM,tabout_s34x,TABLE,nds,2,1
*DO, var, 1, nds
SET, var, LAST
*GET,Force_s34x,ELEM,selsp+3,SMISC,1 !FORCE LOCAL X
*GET,Displacement_s34x,ELEM,selsp+3,NMISC,1 !DISPLACEMENT LOCAL X
*VFILL,tabout_s34x(var,1),DATA,Force_s34x
*VFILL,tabout_s34x(var,2),DATA,Displacement_s34x
*ENDDO

!SPRING FPC3 CORNER 2 KEYPOINT 35/55

*DIM,tabout_s35y,TABLE,nds,2,1
*DO, var, 1, nds
SET, var, LAST
*GET,Force_s35y,ELEM,selsp+4,SMISC,1 !FORCE LOCAL Y
*GET,Displacement_s35y,ELEM,selsp+4,NMISC,1 !DISPLACEMENT LOCAL Y
*VFILL,tabout_s35y(var,1),DATA,Force_s35y
*VFILL,tabout_s35y(var,2),DATA,Displacement_s35y
*ENDDO

*DIM,tabout_s35x,TABLE,nds,2,1
*DO, var, 1, nds
SET, var, LAST
*GET,Force_s35x,ELEM,selsp+6,SMISC,1 !FORCE LOCAL X
*GET,Displacement_s35x,ELEM,selsp+6,NMISC,1 !DISPLACEMENT LOCAL X
*VFILL,tabout_s35x(var,1),DATA,Force_s35x
*VFILL,tabout_s35x(var,2),DATA,Displacement_s35x
*ENDDO

!SPRING FPC3 CORNER 3 KEYPOINT 36/56

*DIM,tabout_s36y,TABLE,nds,2,1
*DO, var, 1, nds
SET, var, LAST
*GET,Force_s36y,ELEM,selsp+5,SMISC,1 !FORCE LOCAL Y
*GET,Displacement_s36y,ELEM,selsp+5,NMISC,1 !DISPLACEMENT LOCAL Y
*VFILL,tabout_s36y(var,1),DATA,Force_s36y
*VFILL,tabout_s36y(var,2),DATA,Displacement_s36y
*ENDDO

*DIM,tabout_s36x,TABLE,nds,2,1
*DO, var, 1, nds
SET, var, LAST
*GET,Force_s36x,ELEM,selsp+7,SMISC,1 !FORCE LOCAL X
*GET,Displacement_s36x,ELEM,selsp+7,NMISC,1 !DISPLACEMENT LOCAL X
*VFILL,tabout_s36x(var,1),DATA,Force_s36x
*VFILL,tabout_s36x(var,2),DATA,Displacement_s36x
*ENDDO

!SPRING FPC3 CORNER 3 KEYPOINT 37/57

*DIM,tabout_s37y,TABLE,nds,2,1
*DO, var, 1, nds
SET, var, LAST
*GET,Force_s37y,ELEM,selsp+6,SMISC,1 !FORCE LOCAL Y
*GET,Displacement_s37y,ELEM,selsp+6,NMISC,1 !DISPLACEMENT LOCAL Y
*VFILL,tabout_s37y(var,1),DATA,Force_s37y
*VFILL,tabout_s37y(var,2),DATA,Displacement_s37y
*ENDDO

*DIM,tabout_s37x,TABLE,nds,2,1
*DO, var, 1, nds
SET, var, LAST

R. Lieven

133
*GET, Force_s37x, ELEM, selsp+10, SMISC, 1
*GET, Displacement_s37x, ELEM, selsp+10, NMISC, 1
*VFILL, tabout_s37x(var,1), DATA, Force_s37x
*VFILL, tabout_s37x(var,2), DATA, Displacement_s37x
*ENDDO

/* SPRING FPC3 CORNER 3 KEYPOINT 38/58 */

*DIM, tabout_s38y, TABLE, nds, 2, 1
*DO, var, 1, nds
  SET, var, LAST
  *GET, Force_s38y, ELEM, selsp+9, SMISC, 1
  *GET, Displacement_s38y, ELEM, selsp+9, NMISC, 1
  *VFILL, tabout_s38y(var,1), DATA, Force_s38y
  *VFILL, tabout_s38y(var,2), DATA, Displacement_s38y
  *ENDDO

*DIM, tabout_s38x, TABLE, nds, 2, 1
*DO, var, 1, nds
  SET, var, LAST
  *GET, Force_s38x, ELEM, selsp+11, SMISC, 1
  *GET, Displacement_s38x, ELEM, selsp+11, NMISC, 1
  *VFILL, tabout_s38x(var,1), DATA, Force_s38x
  *VFILL, tabout_s38x(var,2), DATA, Displacement_s38x
  *ENDDO

/* SPRING FPC3 CORNER 4 KEYPOINT 39/59 */

*DIM, tabout_s39y, TABLE, nds, 2, 1
*DO, var, 1, nds
  SET, var, LAST
  *GET, Force_s39y, ELEM, selsp+12, SMISC, 1
  *GET, Displacement_s39y, ELEM, selsp+12, NMISC, 1
  *VFILL, tabout_s39y(var,1), DATA, Force_s39y
  *VFILL, tabout_s39y(var,2), DATA, Displacement_s39y
  *ENDDO

*DIM, tabout_s39x, TABLE, nds, 2, 1
*DO, var, 1, nds
  SET, var, LAST
  *GET, Force_s39x, ELEM, selsp+14, SMISC, 1
  *GET, Displacement_s39x, ELEM, selsp+14, NMISC, 1
  *VFILL, tabout_s39x(var,1), DATA, Force_s39x
  *VFILL, tabout_s39x(var,2), DATA, Displacement_s39x
  *ENDDO

/* SPRING FPC3 CORNER 4 KEYPOINT 40/60 */

*DIM, tabout_s40y, TABLE, nds, 2, 1
*DO, var, 1, nds
  SET, var, LAST
  *GET, Force_s40y, ELEM, selsp+13, SMISC, 1
  *GET, Displacement_s40y, ELEM, selsp+13, NMISC, 1
  *VFILL, tabout_s40y(var,1), DATA, Force_s40y
  *VFILL, tabout_s40y(var,2), DATA, Displacement_s40y
  *ENDDO

*DIM, tabout_s40x, TABLE, nds, 2, 1
*DO, var, 1, nds
  SET, var, LAST
  *GET, Force_s40x, ELEM, selsp+15, SMISC, 1
  *GET, Displacement_s40x, ELEM, selsp+15, NMISC, 1
  *VFILL, tabout_s40x(var,1), DATA, Force_s40x
  *VFILL, tabout_s40x(var,2), DATA, Displacement_s40x
  *ENDDO

! /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /\ /|
Appendix F

!------------------------------------------------------------------------------------------------
!DISPLACEMENT CORNER 2
!------------------------------------------------------------------------------------------------
*WRITE,
(' ')
*WRITE,
(' ')
*WRITE,
('DISPLACEMENT CORNER 2')
*WRITE,
(' ', 'Lateral load [N]', 'Displacement x (m-02 / A1-14) [mm]')
*WRITE, tabout_m02(1,1), tabout_m02(1,2)
(E18.10, ' ', E18.10)

!------------------------------------------------------------------------------------------------
!REACTION FORCES & DISPLACEMENT CORNER 3
!------------------------------------------------------------------------------------------------
*WRITE,
(' ')
*WRITE,
(' ')
*WRITE,
('REACTION FORCE & DISPLACEMENT CORNER 3')
*WRITE,
(' ', 'Reaction force x [N]', 'Displacement x (m-00) [mm]', 'Reaction force y [N]', 'Displacement y (m-04) [N]')
*WRITE, tabout_m00(1,1), tabout_m00(1,2), tabout_m04(1,1), tabout_m04(1,2)

!------------------------------------------------------------------------------------------------
!REACTION FORCES CORNER 4
!------------------------------------------------------------------------------------------------
*WRITE,
(' ')
*WRITE,
(' ')
*WRITE,
('REACTION FORCES CORNER 4')
*WRITE,
(' ', 'Reaction force x [N]', 'Reaction force y [N]')
*WRITE, tabout_m03(1,1), tabout_m03(1,2)

!------------------------------------------------------------------------------------------------
!NORMAL FORCES AND MOMENTS IN COLUMNS AND BEAMS
!------------------------------------------------------------------------------------------------
*WRITE,
(' ')
*WRITE,
(' ')
*WRITE,
('NORMAL FORCES AND MOMENTS IN COLUMNS AND BEAMS')
*WRITE,
(' ', 'Left column', 'Right column', 'Lower beam')
*WRITE,
(' ', 'Force [N]', 'Moment [Nmm]', 'Force [N]', 'Moment [Nmm]', 'Force [N]', 'Moment [Nmm]', 'Force [N]', 'Moment [Nmm]')
*WRITE, tabout_sg0009(1,1), tabout_sg0009(1,2), tabout_sg1011(1,1), tabout_sg1011(1,2), tabout_sg1617(1,1), tabout_sg1617(1,2), tabout_sg0016(1,1), tabout_sg0016(1,2)

!------------------------------------------------------------------------------------------------
!SPRING STIFFNESS FPC3 CORNER 1 KEYPOINT 33/53 & 34/54
!------------------------------------------------------------------------------------------------
*WRITE,
(' ')
*WRITE,
(' ')
*WRITE,
('SPRING CHARACTERISTICS CORNER 1')
*WRITE,
(' ', 'KEYPOINT 33/53 y', 'KEYPOINT 33/53 x', 'KEYPOINT 34/54 y', 'KEYPOINT 34/54 x')
*WRITE,
(' ', 'Force y [N]', 'Displacement y [N]', 'Force x [N]', 'Displacement x [N]')
*WRITE, tabout_s33y(1,1), tabout_s33y(1,2), tabout_s33x(1,1), tabout_s33x(1,2), tabout_s34y(1,1), tabout_s34y(1,2), tabout_s34x(1,1), tabout_s34x(1,2)
Appendix G  Enlarged figures
G1  Enlarged figures chapter 3

Figure G1.1 – Enlargement of figure 3.11.

Figure G1.2 – Enlargement of figure 3.27.
Appendix G

G2 Enlarged figures chapter 4

![Enlarged figure 4.6a](image1)

Figure G2.1 – Enlargement of figure 4.6a.

![Enlarged figure 4.6b](image2)

Figure G2.2 – Enlargement of figure 4.6b.
Figure G2.3 – Enlargement of figure 4.7.

Figure G2.4 – Enlargement of figure 4.8.
Figure G2.5 – Enlargement of figure 4.10.

Figure G2.6 – Enlargement of figure 4.11.
Figure G2.7 – Enlargement of figure 4.12a.

Figure G2.8 – Enlargement of figure 4.12b.
G3   Enlarged figures chapter 5

![Graph showing lateral load vs lateral deflection](image)

Figure G2.1 – Enlargement of figure 5.1.

![Graph showing lateral load vs lateral deflection](image)

Figure G2.2 – Enlargement of figure 5.2.
The aim of this research project is to gain an understanding of the racking shear behaviour of a semi-integral infilled frame, with a new type of frame panel connection which discretely connects the precast concrete panel to the steel frame. The research is carried out by performing experimental full scale tests. The test results are compared with previous researches into semi-integral infilled frames and are used to calibrate a finite element model, which has been created to simulate the racking shear behaviour of the semi-integral infilled frame. The finite element model can be used for further parametric studies.