structural design

Interface connection of a semi-integral infilled frame

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Joost Dekkers
Eindhoven, June 2011
Summary

A significant improvement in structural performance of steel frame structures with discretely connected precast reinforced concrete infills, subjected to lateral loading, has been recognized. The structural composite behaviour of an infilled frame subjected to an in-plane horizontal or racking load depends on the used materials and the connection between infill and skeletal structure. Infilled frames have generally been classified as non-integral, fully-integral or semi-integral, depending on the type of connection between frame structure and infill panel. Non-integral infilled frames have no structural connection between frame and infill, whereas a fully-integral infilled frame consists of a continuous connection along the interface. The semi-integral infilled frame contains connections at discrete locations. The contribution of precast concrete infill panels to the lateral stiffness and strength of steel frames can be significant and lead to more efficient use of materials if the optimum quality, quantity and location of discrete interface locations can be determined.

The current graduation report investigates an interface connection of a semi-integral infilled frame and is based on previous research at the Eindhoven University of Technology on the structural behaviour of steel frames with discretely connected precast concrete infill panels subjected to in-plane horizontal loading. The aim of the current investigation is to design a discrete connection for steel frames with precast reinforced infill panels where bearing of the bolt holes is the governing failure mode thereby improving the structural performance of the infilled frame. This was done by analysing the designs and experimental results of the two previously connections, FPC1 and FPC2. This resulted in a new connection, FPC3.

Four experimental pull-out tests were performed on the connection to assess its structural behaviour under tension. The structural response was approximated by a multi-linear force-displacement curve, allowing its application in FE-models. Additionally, it was tried to develop a simplified two dimensional finite element model to describe the interaction of the single lap joint and the cast-in anchor bars.

Analysis of the two existing connections, FPC1 and FPC2, lead to the conclusion that changing the position of the interface connection would increase the strength and stiffness of the infilled frame. Cast-in steel plates will initiate concrete splitting in compression and tension, resulting in loss of bond and, finally, pull-out failure and should therefore be avoided. Too short anchorages and large diameter anchor bars decreases bond strength. Smaller diameter anchor bars and larger anchorage length should be used to avoid pull-out failure. The new connection is therefore positioned near the corners of the steel frame, and consists of a triangular T-section with five anchor bars $\phi 16$ mm, parallel to the tension/compression diagonal. The failure load for ovalization of the bolt holes was calculated to be 394 kN, according to the Eurocode 3.

Four experimental tests were carried out in a newly designed test set-up, based on tests on previous two
connections. The average failure load of the connection was 361 kN where unexpected pull-out was the governing failure mode. The connection failed due to loss of bonding between the anchor bars and the surrounding concrete. An average ovalization of 2.9 mm was observed in the bolt holes.

Force-displacement relations for ovalization and slip are determined for additional full-frame numerical research of infilled frames with this type of connection. The spring stiffness, describing ovalization of the bolt holes, is determined for the connections along the compression diagonal. A combined spring stiffness, that describes ovalization of the bolt holes and anchorage slip, is determined for the connections along the tension diagonal. Regarding the experimental test data, the overall stiffness and strength of the connection FPC3 is an improvement on the structural behaviour of FPC1 and FPC2 under tension.

Due to unexpected concrete failure, the simplified two dimensional finite element analyses are divided into two separate analyses for the steel plate and the concrete panel.

The development of a two-dimensional finite element model for describing the ovalization of the bolt holes has only been moderately successful in predicting the structural behaviour of the laboratory tests. Three finite element models have been created for analyses. The First model, using regular plane stress elements, has resulted in a good prediction for the ‘elastic’ part of the force-displacement relation but yielded unacceptable results for the plastic deformations along the bolt hole. The second model, consisting of plane strain elements for describing enclosure of the steel plate due to the bolt and the adjacent loading plate, has resulted in a too stiff elastic behaviour, but predicted the stiffness and strength for plastic behaviour more accurately. Finally, a combination of the two element types has been used for the third model. The result of this analysis showed an overestimation of the stiffness for the elastic phase and an underestimation of the stiffness for the plastic phase and the strength at an ovalization of 3.0 mm.

It can be concluded that a two dimensional finite element model is not suitable to describe the structural behaviour of a single lap joint. Three dimensional effects, such as friction between steel plates, friction between steel plate and bolt cap and washers and out-of-plane bending resulting in clamping forces, have probably considerable influence on the structural behaviour and the ovalization of the bolt holes that cannot be ignored in an analysis.

Finally, a pre-numerical analysis has been performed on the concrete panel with cast-in anchor bars to predict its structural behaviour under a tension load. Using a smeared crack approach, a multi-directional fixed crack model with a linear stress cut-off criterion, for the concrete panel resulted in reasonable results for the stiffness and good results for its strength. For the initial phase until the first cracks appear, stiffness is well predicted. The force where stiffness degradation due to cracking occurs is also well predicted. From that point onwards, slip of the anchor bars becomes more influential resulting in excessively stiff behaviour of the FEM model. The ultimate load and the crack patterns are also predicted with reasonable accuracy.

A smeared crack approach used in FEM modelling for predicting the structural behaviour of the concrete panel resulted in reasonable results for stiffness strength and crack patterns. Implementing interface elements to describe bond-slip effects may yield better results for describing the post-cracking stiffness of the connection.
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1 Introduction

1.1 Historical description

For many years steel frames combined with bracing structures and reinforced concrete cores and/or shear walls have been used in (high-rise) buildings all over the world. The steel frames are used to resist vertical loads while the lateral loads are being transferred by floor planes towards the concrete cores and/or walls. A concrete core is usually located in the centre of the building providing space for elevators and staircases. Due to the relative restriction of its size, the contribution to the overall lateral stiffness and strength of the structural system is limited. Steel skeletal structures are often filled in to create interior divisions or facades. These structures are not designed to contribute to the lateral stiffness of the entire building. A search for more sufficient structural systems has resulted in the recognition of infilled frames in the 1950's\textsuperscript{1,2,3,4}. The infills, on account of their composite in-plane action with the frame, considerably enhance the lateral stiffness and strength of the structure. The contribution of the infill panels to the horizontal stiffness and strength of bare frames has great potential\textsuperscript{5}, but the structure is not very well understood. Therefore, structural engineers in current practice are conservative and neglect the effects of the infill, firstly, due to lack of the knowledge regarding the composite interaction of the infill and the surrounding frame, secondly, due to the lack of practical methods for predicting the stiffness and strength, and thirdly, due to the possibility of future removal of the infill for access, doors and windows\textsuperscript{3,5}.

1.1.1 The infilled frame

An infilled frame consists of three components: the skeletal structure, the infill panel and the interface between the frame and its infill. The frame is the main structure consisting of steel or reinforced concrete columns and beams. Steel columns and beams are commonly connected with hinged joints because this type of connection is generally a low-cost construction method. The infill panel is used to fill up the frame in order to create a facade or partition wall. It can also contribute to the structural behaviour of the building when structurally connected to the frame. This contribution can be qualified for some types of infilled frames, but is usually not taken into account. None of the existing theories have been accepted by any building code at the moment.

The interface between the frame and the infill is important for the total structural behaviour of the infilled frame. It determines the contribution of the infill to the horizontal stiffness and the co-operation of the frame and the infill under lateral loading. The interface can be a physical structural connection, but can also be just the location where the frame and infill meet in compression. The type of interface determines the classification of the infilled frame. Infilled frames have generally been classified as non-integral, semi-integral or fully-integral infilled frames, figure 1-1.
A non-integral infilled frame consists of a skeletal structure combined with an infill without any physical connection, such as improved bonding or shear connectors, between the two structural elements. A typical infill for this type of classification consists of masonry. In general, separation and slip at the interface determines the stiffness and strength of the non-integral infilled frame. Research has resulted in an equivalent diagonal strut analogy introduced by Polyakov\(^2\).

Improving bonding and creating shear connectors at the interface significantly improves the structural performance of infilled frames\(^5\). This is then qualified as a fully-integral infilled frame with continuous interaction e.g. a cast-in-place concrete infill panel with numerous shear connectors at the interface. Separation will be hindered and this introduces normal stresses as well as shear stresses along the interface. Analysis of this type of structure can be performed using the equivalent frame theory, proposed by Liauw\(^3\)\(^\text{[8]}\). If the cast-in-place concrete infill with continuous shear connectors is replaced by a precast concrete infill panel with discrete connections at several locations on the steel frame, the infilled frame can be considered as semi-integral infilled frame. The idea of semi-integral infilled steel frames was earlier considered by Liauw and Kwan\(^6\)\(^\text{[9]}\) in a plastic theory of integral infilled frames with continuous connections along the beams and columns, where finite shear strength at the infill-frame interface was taken into account over specified distances. The limited shear resistance of the partial connections were employed to obtain the collapse load of the infilled frame by considering two failure modes: corner crushing and diagonal crushing. The use of discrete beam-infill connections away from the beam-column joints avoids corner crushing and increased shear forces in the columns. Steel-concrete contact only occurs at the frame-panel connections where the forces from the steel frame are introduced into the concrete via anchor bolts. This way the infill panel functions as a bracing system with compression, tension and shear forces.

1.1.2 Research at Eindhoven University of Technology

At Eindhoven University of Technology a research program on composite construction is underway aiming at the development of design rules for steel frames with precast reinforced concrete infill panels subjected to horizontal loading. Preliminary FEM-analyses have shown similar behaviour of lateral loading on semi-
integral infilled frame and on fully-integral infilled frames\textsuperscript{[5]}. It is also been concluded that interface connections on beams are more efficient than on columns and the lateral stiffness of the structure improves when the connections are located closer to the beam-to-column joints\textsuperscript{[7]}.

Recent projects included 3 by 3 meters steel frames with precast reinforced concrete infill panels discretely connected to the top and bottom beams. Four discrete connections allow the concrete panel to behave similarly to X-bracing in a trussed frame. The projects comprise experimental, numerical and analytical research on the in-plane composite behaviour of infilled steel frames\textsuperscript{[8][9][10]}. Two variations on this type of connection were earlier designed and tested experimentally, see figure 1-2. The connections consist of one or two anchor bars, welded to a steel plate and precast in a pocket at the edge of the concrete panel. The cast-in anchor plate is bolted to a gusset plate that is welded to a frame member. The connection is located on the centre line of the structural elements thereby keeping eccentricities, and therefore out-of-plane buckling, to a minimum. It is assumed that this connection acts as a hinge and is able to transfer normal and shear forces at the structural interface. Due to the gap between the concrete panel and the steel beams and columns, friction will not take place. The specific intermittent connection systems in infilled frames will cause stress concentrations in the concrete panels which will influence the strength of the structure. The formation of stress paths constitutes an equivalent bracing pattern within the panel that will contribute to the stiffness of the structure.

\begin{figure}[h]
\centering
\includegraphics[width=\textwidth]{connections.png}
\caption{Connections by Tang (FPC1) and Van der Cruijssen (FPC2)\textsuperscript{[11]}}
\end{figure}

The connections were designed for the failure mode ‘bearing’, consisting of ovalization of the bolt holes. It allows a ductile failure mode which causes large deformations in the structural element and contains a certain amount of residual strength. Experimental tests were performed on both connections individually. Two full-scale infilled frame tests were performed as well. The individual connection tests consisted of a shear test and a tension test, see figure 1-3. The discrete connections were designed for shear and pull-out independently without taking any interaction into account. The connection is an essential part of the infilled frame that allows composite interaction between the steel frame and the precast concrete infill to be developed.
1.2 Definition of the problem

Two different discrete connections between the steel frame and the precast reinforced infill panel have been designed and tested in the Pieter van Musschenbroek laboratory of the Eindhoven University of Technology. The first connection, FPC1, by R.B. Tang\cite{10} was designed too conservatively. A new design, FPC2, was proposed by Van der Cruijssen\cite{11} and tested in shear, tension and on a full scale infilled frame. The lateral stiffness and strength of the infilled frame did not match the desired performance due early failure of the connection FPC2.

1.3 Objective of the research

The aim of this research project is to design a discrete connection for steel frames with precast reinforced concrete infill panels resulting in bearing of the bolt holes as the governing failure mode with an economical use of anchor bars and additional reinforcement thereby improving the structural performance of the infilled frame.

1.4 Working method

In order to design an optimum discrete connection, previously designed connections will be evaluated on their structural performance. Evolved insights of the failures of these connections will be used to develop several concepts according to the defined objective. The concepts will then be assessed to determine the optimum design to perform experimental and numerical research.

In order to perform experimental tests on the connection to determine the interaction between shear and tension a new test series will be designed. The new design will then be tested in the Pieter van Musschenbroek laboratory.

A finite element model of the connection will be created to perform FEM analyses regarding the experimental tests and thereby calibrated by experimental results. Finite element package DIANA will be used to model the anchor bars cast-in the concrete panel and analyse stress distribution in the steel plate,
concrete and cast-in anchor bars. The numerical analysis will be used to adjust loading and refine the structural components of the connection to improve structural quality. An overview of the research is shown in figure 1-4.

**CONCEPTUAL DESIGN**
Creating a discrete connection for a semi-integral infilled frame with improved structural properties regarding strength and stiffness.
Designing different components of the connection with respect to the failure mode of ovalization of the bolt holes.

**EXPERIMENTAL**
Designing a test set-up in order to perform tests on the connection. Analysing results of obtained data and produce load-deformation relationships for the discrete connections for ongoing numerical research.

**NUMERICAL**
Developing a simplified two dimensional FEM model of the designed connection and calibrating it using test data from experimental research in order to perform parametric studies with different loading angles.

**figure 1-4 - scheme of this research**
2 Previous interface connections

In order to create semi-integral composite interaction between the steel frame and the precast concrete infill panel, two different interface connections were designed at Eindhoven University of Technology. The interface connection of the semi-integral infilled frame consists of two individual components. The first part of the connection is a steel gusset plate welded to the beam element of the steel frame. The other part is a steel anchor plate, precast in the reinforced concrete infill panel with welded anchor bars to transfer forces from the concrete panel to the steel gusset plate. Both plates are connected to one other with two bolts. The connection was designed to resist horizontal loading on the frame until bearing in the bolt holes occurs. The bearing resistance of the bolt holes was a benchmark for the design load of the anchor bolts and other structural components of the connection.

2.1 First interface connection

FPC1 was proposed by R.B. Tang, see figure 2-1. The calculations of the connection were mainly based on design calculations by the Eurocode. Design rules for this type of connection were unavailable at the time of research.

![figure 2-1 - FPC1 by Tang (a) and design loads (b)](image)

Both steel gusset plates, with a thickness of 10 mm each, which were connected with two M24 bolts, resulted in a bearing resistance of the bolt holes of 110 kN per bolt. Because this type of plastic deformation of the steel plates should be decisive for the connection, 220 kN was taken as the design value for the anchor bars. The anchor design consisted of two steel anchor bars Ø18 with a length of approximately 1100 and 1250 mm. Both anchor lengths were reduced by the use of hooks at the end of the anchor.

Experimental tests on the individual frame-panel connections were performed to establish structural...
characteristics such as strength and stiffness. Separate tests were done in two orthogonal directions: tension tests perpendicular to the edge of the panel and shear tests parallel to the edge of the panel. The individual test set-ups are shown in figure 2-2. The connection was also tested in a full scale test to examine the stress distribution of the infilled frame\[7].

The shear test

The shear connection tests were done in pairs as shown in figure 2-2(a). Axial compression was applied to the vertical steel member that was connected to two concrete panels which were fully supported on their short side. Both connections were equally loaded in shear. Vertical displacements were measured on the steel section, the bolts, the connection plates and the concrete panels. The curves in figure 2-3 indicate typical load-displacement measurements for FPC1. The force in kN indicates the load taken by a single connection. The displacements were obtained from the above measurements and represent the sum of anchor plate movement and ovalization in the connection plates. The test was performed three times. The unintended use of higher strength materials for the gusset and anchor plates of connection FPC1 resulted in shear failure in the bolts during the first two tests. The last shear test on FPC1 was stopped at 337 kN. This connection failed in yielding of the steel plates with an average load of 169 kN per bolt which is evidently higher than the calculated failure load for ovalization of the bolt holes. These results were attributed to the higher strength capacities of the steel gusset plates compared to the design calculations.

2.1.1 The shear test

figure 2-2 - individual test set-up's for shear and pull-out
2.1.2 The tension test

The test set-up shown in figure 2-2(b) comprises a single concrete panel that is placed on two jacks. The concrete panel was pushed upwards by two jacks during the experiment. The cast-in anchor plate was bolted to two 450 mm long steel holding strips of 100 by 20 mm which were connected to the test rig. Force and vertical displacements at several locations were recorded during the experiment. The connection failed because of concrete edge failure (pull-out) due to the shear loads on the steel anchor bars in the concrete panel instead of the desired bearing of the bolt holes. The steel plate with welded anchor bars was pulled out of the precast concrete panel due to concrete splitting around the anchor bars, figure 2-4. Four connections of type FPC1 were tested and their force-displacement curves are shown in figure 2-5. The displacements were obtained from the above measurements and represent the sum of anchor plate movement and ovalization in the anchor plates. The average failure load was measured at a force of 160 kN per bolt. This value also exceeds the design load for ovalization of the bolt holes. Ovalization could not take place due to the location of the bolt head and nut near the anchor bar, see figure 2-1(a).
2.1.3 The full scale test

The test rig shown in figure 2-6(a) consists of a vertical and a diagonal loop comprising HE300B sections causing the test rig members to be loaded in tension and compression only. The 3000 by 3000 mm steel frames were built up from IPE220 sections for the beams and HE300B sections for the columns. The beam-column connections consisted of 10 mm thick header plates welded all around to the beam. The header plates were connected to the column flange with one bolt M24 10.9 at mid height on either side of the beam web. The specimen was horizontally loaded at the top by a 2 MN capacity jack. Before testing the infilled frame structure, the bare steel frame without the infill panel was tested up to a lateral load of 21.8 kN. This yielded a lateral frame stiffness of 1.28 kN/mm and a rotational stiffness of the beam-to-column connections of $2.53 \times 10^3$ kNm/rad. The load application at the upper beam was displacement controlled at 0.5 mm per minute. The infill panel was connected to the steel beams at four locations. Due to the dimensions of the anchor configuration the connections were located 875 mm from the column centre.
lines with a 15 mm gap between steel and concrete all the way around, see figure 2-6(b).

As a result of the anchor positions, the compression/tension diagonals between the connections did have an angle of 64 degrees. Therefore the vertical component of the diagonals was 210 percent larger than its horizontal component.

![Diagram](attachment:diagram.png)

*Figure 2-7* - Infilled frame test (a) and load-displacement graph (b)

The load-displacement curves in figure 2-7(b) show that in the initial stages up until 90 kN for Test A and 75 kN for Test B, the structure displayed a rather flexible behaviour. It was taken that in this settling-in stage not all four connections were participating in resisting the applied horizontal loading on the structure. On Test A the horizontal load was increased to 345 kN where the linear elastic behaviour with a lateral stiffness of 15.9 kN/mm changed significantly. At an applied load of about 360 kN it was observed that the steel beams were in contact with the concrete panel. Outside the reach of the LVDT measuring the horizontal deflection of the steel frame, the load was further increased to 421 kN. As this was the first full scale experiment with a newly designed test rig it was decided to halt the test. After dismantling, it was observed that ovalization of the bolt holes had taken place at several locations.

### 2.2 Second Interface connection

Van der Cruijsen designed a revised connection FPC2 based on the configuration by Tang. The calculations made by Tang were based on Eurocode 3, which did not provide design rules for a discrete interface connection of a semi-integral infilled frame. Therefore it was assumed that the connection was over dimensioned. This was also confirmed by the experimental tests. Calculations were compared with the Dutch national codes and new assumptions by Van der Cruijsen were made to optimize the connection because design rules of the type of connection were also unavailable in the Dutch national design codes at that time.
2.2.1 Adaptations by Van der Cruijssen

The design values for the revised anchors were assumed to be a maximum tension load or a maximum shear load, figure 2-8(a). However the load on the connection will be applied at a certain angle. Therefore the maximal bearing resistance in the bolt holes will be increased.

The anchorage length of the anchor bars were reduced according to article 9.6.2 of the Dutch national code, NEN6720, as well as the reduction on cast-in anchors according to article 9.16.3 of the same code. The governing reduction should be used instead of a combination of both reductions. The diameter of the anchor bars were increased to 25 mm instead of 18 mm. Several material properties of the two different connections are shown in table 2-1. Pull-out, shear and full-scale tests were performed at the Pieter van Musschenbroek laboratory using the test set up's given in paragraph 2.1.

| Table 2-1 - measured material properties of earlier frame panel connections |
|-------------------|----------------|----------------|----------------|-----------------|----------------|
| Bolts             | Anchor bars    | Anchor plates  | Gusset plates  | Concrete panels |
|                  | FeB500         | 320×210×10 mm  | 200×150×10 mm  | 1400×600×150 mm |
| Ø [mm]            | $f_m$ [N/mm²]  | $f_k$ [N/mm²]  | $f_b$ [N/mm²]  | $f_c$ [N/mm²]   |
| M24               | 10.9           | 25             | 500 *           | 529             | 579             | 529             | 579             | 10 @ 200        | 47 / (37)**    |
| FPC1              | 10.9           | 18             | 500 *           | 247             | 408             | 294             | 432             | Ø8 @ 150        | 44 / (34)**    |
| FPC2              | 10.9           | 25             | 500 *           | 529             | 579             | 529             | 579             | Ø10 @ 200       | 47 / (37)**    |

* assumed, ** approximate cylinder strength in brackets

2.2.2 The shear test

The same test set-up was used as discussed in paragraph 2.1.1. The load-deflection curve of the shear test for FPC2 in figure 2-9(b) displays bi-linear behaviour up to a maximum resistance of 350 kN. At this load the anchor plate cracked the concrete, ovalization was not governing. The maximum bearing load of 110 kN per bolt hole according to the design calculations was therefore an underestimation.
2.2.3 The tension test

The tension test was performed two times using the test set-up given in paragraph 2.1.2. The average load that caused failure of the connection was determined at 245 kN. The failure was initiated by concrete edge failure (pull-out) similar to the first connection but at a lower force (figure 2-10 and figure 2-11). The results correspond reasonably well to the design calculations.
2.2.4 The full scale test

Contrary to the first full scale test (Test A), a 25 mm gap was used for Test B with FPC2 type connections to avoid beam-panel contact. The reinforcement of the full scale concrete panels was the same as being used for the testing of the two frame-to-panel connections.

The full-scale test by De Rooij\cite{rooij} with FPC2 failed on concrete failure instead of bearing in the bolt holes. Similar crack patterns as the tension test were noticeable during the full-scale test, see figure 2-12. Beyond 75 kN the lateral stiffness of infilled frame B is 12.5 kN/mm until 241 kN when failure occurred at the lower left frame-panel connection, figure 2-12. The load was further increased to 276 kN when pull-out failure occurred at the upper connection on the right. With only a compression diagonal present the infilled frame reached a lateral load capacity of 257 kN when the concrete panel made contact with the steel frame. It was then decided to take the load off the structure.
2.3 Comparison of test results

A result overview of the different tests is presented in table 2-2.

<table>
<thead>
<tr>
<th>Maximum force</th>
<th>Tensile test</th>
<th>Shear test</th>
<th>Full-scale test</th>
<th>Tensile test</th>
<th>Shear test</th>
<th>Full-scale test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 01</td>
<td>295 kN (ce)</td>
<td>457 kN (bsf)</td>
<td>421 kN * (-)</td>
<td>237 kN (ce)</td>
<td>350 kN (ysp+bbh)</td>
<td>276 kN (ce)</td>
</tr>
<tr>
<td>Test 02</td>
<td>330 kN (-)</td>
<td>448 kN (ysp+bbh)</td>
<td>252 kN (ce)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Test 03</td>
<td>284 kN (ce)</td>
<td>337 kN * (ysp+bbh)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Test 04</td>
<td>374 kN (ce)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>321 kN</td>
<td>453 kN</td>
<td>421 kN</td>
<td>245 kN</td>
<td>350 kN</td>
<td>276 kN</td>
</tr>
</tbody>
</table>

(ce) = Concrete edge failure (pull-out); (bbh) = Bearing in the bolt holes; (bsf) = Bolt shear failure; (ysp) = Yielding of the steel plate; * = test halted before ultimate load

Both discrete frame-panel connections were able to transfer normal and shear forces in order to govern the lateral stiffness and the ultimate strength of an infilled structure. However, FPC1 resulted in a larger lateral stiffness and a higher ultimate load in the tests comparing to FPC2, see table 2-2 and figure 2-13. The desired failure mode of ovalization of the bolt holes did not occur though and concrete edge failure (pull-out) was inevitable.

<table>
<thead>
<tr>
<th>Test</th>
<th>Shear Stiffness $K_s$ in kN/mm</th>
<th>Shear Strength $F_s$, in kN</th>
<th>Tension Stiffness $K_t$ in kN/mm</th>
<th>Tension Strength $F_t$, in kN</th>
</tr>
</thead>
<tbody>
<tr>
<td>FPC1</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>86.2</td>
<td>457</td>
<td>112.9</td>
<td>295</td>
</tr>
<tr>
<td>2</td>
<td>88.2</td>
<td>448</td>
<td>103.6</td>
<td>330</td>
</tr>
<tr>
<td>3</td>
<td>54.9</td>
<td>337*</td>
<td>121.7</td>
<td>284</td>
</tr>
<tr>
<td>4</td>
<td>-</td>
<td>-</td>
<td>118.4</td>
<td>374</td>
</tr>
<tr>
<td>Avg</td>
<td>(76.4)</td>
<td>(453)</td>
<td>(134.2)</td>
<td>(321)</td>
</tr>
<tr>
<td>FPC2</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>56.8</td>
<td>350</td>
<td>86.1</td>
<td>237</td>
</tr>
<tr>
<td>2</td>
<td>-</td>
<td>-</td>
<td>91.4</td>
<td>252</td>
</tr>
<tr>
<td>Avg</td>
<td>(56.8)</td>
<td>(350)</td>
<td>(88.8)</td>
<td>(245)</td>
</tr>
</tbody>
</table>

* = test halted before ultimate load

As the infilled frame was loaded with a horizontal force, mainly the anchors of the connection in tension are subjected to bond-slip. The displacements that did occur due to anchorage slip resulted in peak stresses at the 10 mm face of concrete and the steel anchor plate, i.e. the steel plate cuts into the concrete panel causing internal cracks (splitting of concrete) which can initiate concrete failure.

The steel anchor plates according to the FPC1 and FPC2 were entirely surrounded by concrete. Movement of the steel plate, as a result of anchorage slip, caused cutting of the steel plate into the concrete element for connections along the tension diagonal as well as the compression diagonal.

Other major cracks that appeared during the tests were developed from the anchor bar to the side of the infill panel where the concrete cover is minimal.
Besides the displacement of the cast-in steel plate, the concrete cover on the anchor bars was an important factor that allowed concrete edge failure. Both tension tests of FPC1 and FPC2 showed similar crack patterns of this failure mode, see figure 2-4 and figure 2-11. The cracks that occurred during the test of FPC1 were larger and the edge failure was bigger. This might be caused by the configuration of the anchor and the use of a single anchor Ø25 instead of two anchors Ø18.

2.4 Conclusion and recommendations for the new design

2.4.1 Position

The first discrete connection between precast reinforced concrete panel and steel frame designed by R. B. Tang was able to transfer horizontal forces in order to gain lateral stiffness without concrete failure. Due to the dimensions of this connection it was positioned 875 mm from the columns. Ovalization of the bolt holes did occur and plastic deformations of the steel frame were noticeable. The dimensions of the connection are out of proportion and a smaller connection is favoured.

2.4.2 Assumed design forces

The second connection by Van der Cruijssen was situated at the same location as the first design and the design forces on the connection were not correctly adjusted to optimize the anchor design because the connection was loaded under a certain angle which caused tensile forces as well as shear forces. The connection needs to be designed on the interaction of these forces instead of the single force.

2.4.3 Anchor length

The anchor length was reduced according to the NEN 6720, article 9.6.2. Thereupon, the reduced anchor length was reduced again with a reduction for short anchors, according to article 9.16.3. This was a false assessment that caused a too small anchorage length. The basic anchorage length should be used for reduction instead of the reduced anchorage length. Therefore, pull-out was to be expected.
2.4.4 Pull-out failure
The connection failed due to concrete failure. The full-scale test showed similar cracks like the tension test. The tension test caused mainly shear forces where the anchor meets the concrete. Concrete edge failure was the main failure mechanism with this connection. Increasing the minimum edge distance will increase the strength of the connection as well as additional reinforcement to postpone cracking.

2.4.5 Tension diagonal
The position of the connection caused mainly a tensional component of the force on the connection like the tension test caused as shown in figure 2-6. To optimize the interaction between the shear and tension forces and prevent ‘pull-out’ failure, the connection should be positioned as near to the corner as possible.

2.4.6 Cutting of the steel plate into the concrete
The steel plate did cut into the concrete panel like a knife, which caused invisible splitting cracks inside the concrete panel. This was the beginning of concrete failure and should be prevented by distributing the forces more equally with a larger contact area.
3 Design of a new connection FPC3

3.1 Introduction

Tension tests of FPC1 and FPC2 indicate the importance of the 'pull-out' failure of the anchor bars and thus the vertical component along the tension diagonal needs to be decreased. It has been shown that the geometry of the previously tested full-scale structures caused tension and compression forces in the concrete panel to be two times larger than the shear forces\textsuperscript{[10]}. Pull-out failure of the discrete connection was indicative for the maximum horizontal load on the infilled frame. The optimum position of the connections is at the corners of the infilled frame thereby reducing the pull-out force. Adapting the location to the corner is expected to improve the lateral stiffness and increase the failure load of the infilled frame\textsuperscript{[5]}.  

![Figure 3-1 - location of discrete connection](image)

Bearing in the bolt holes must be the failure mechanism, as describes in paragraph 1.1.2, and therefore other structural components must be able to resist stresses due to loading until the steel plate along the bolt holes yields. The diagonal force of the tension or compression diagonal can be resolved into two orthogonal forces, see figure 3-1. The vertical and horizontal components are together the design forces for the connection and this is the same assumption Tang has made to design the first discrete connection. The interaction of these forces is the maximum bearing force on the two bolt holes. Many variables can affect the strength and stiffness of the connection. The next paragraphs will discuss the different variables in order to create a new design for the connection.

3.2 Design properties

A discrete steel frame to concrete panel connector comprises two main components. One consists of a
steel plate connected to the concrete infill panel. This part of the connection includes a welded steel T-section welded to the cast-in anchor bars. The second component is a steel plate welded to the beam of the steel frame. Both components are jointed with bolts. These parts determine the strength and failure mode of the connection. The main structural components considered in the overall research are categorized in table 3-1.

<table>
<thead>
<tr>
<th>Component</th>
<th>Adjustable variables of component</th>
<th>paragraph</th>
</tr>
</thead>
<tbody>
<tr>
<td>Infilled panel</td>
<td>Thickness - Quality - Reinforcement</td>
<td>3.2.1</td>
</tr>
<tr>
<td>Anchor bars</td>
<td>Configuration - Position - Length - Diameter - Quality</td>
<td>3.2.2</td>
</tr>
<tr>
<td>Gusset plates</td>
<td>Dimensions - Thickness - Quality</td>
<td>3.2.3</td>
</tr>
<tr>
<td>Bolts</td>
<td>Diameter - Quality</td>
<td>3.2.4</td>
</tr>
<tr>
<td>Welds</td>
<td>Thickness - Length</td>
<td>3.2.5</td>
</tr>
<tr>
<td>Steel frame</td>
<td>Dimensions - Quality</td>
<td>3.2.6</td>
</tr>
</tbody>
</table>

3.2.1 Precast reinforced concrete infill panel

The precast reinforced concrete panel would be the same as used in the previous tests in order to compare results\textsuperscript{[11]}. Thickness of the panel was set to 150 mm and the concrete quality C35/45. However due to measurement failures the concrete panel had to be 175 mm to fit the needed concrete cover on the reinforcements. Additional reinforcement of the concrete panel consists of a Ø8-100# grid.

3.2.2 Anchor bars

Configuration and dimensions of the anchor bolts will be changed. As stated before, the connections will be situated near the corners of the infill panel. Therefore the shape and dimensions of the anchor bars need to be adapted to the new configuration. The grade of the anchor bars will be nominal reinforcement quality B500B. Anchor bars tested in previous research had the same quality.

3.2.3 Steel gusset plates

The dimensions of the gusset plates will be adapted to the new configuration of the connection if needed. However, the thickness of the gusset plate will be changed in order to keep bearing in the bolt holes decisive as failure mode. The thickness of the steel plates will set at 10 mm to determine the bearing resistance of the bolt holes.

3.2.4 Bolted connection between the gusset plates

The bolts must be able to resist the shear forces to allow ovalization of the bolt holes to occur. Previous connection used 2M24 bolts of 10.9 grade. Increasing the size or number of bolts will increase the bearing resistance of the bolt holes and thereby the design forces. As a result, the anchor bar configuration will be too large. The goal is to determine the failure mode of the connection without ‘anchor pull-out’ and comparing results with previous research. Therefore, two M24 bolts will be used to connect the steel
gusset plates.

3.2.5 **Welds**
Failure of the welds is unacceptable. The theoretical throat and the effective length will be adjusted if needed to meet the requirements of bearing in the bolt holes as the governing failure mode.

3.2.6 **Steel frame**
The steel frame used for experimental research of previous connections contained two HEA300 sections as columns and two IPE220 as beam elements. The elements are connected with two M24 bolts to act as a hinged connection.

3.3 **New connections**
Three concepts are considered for interface connections between the precast reinforced concrete panel and the steel frame. As stated before, the optimum position of the connection for global frame behaviour is near the corners of the infilled frame.

3.3.1 **Consideration of the design concepts**

3.3.1.1 **First design**

![Image of first design](image)

The first concept, figure 3-2, of FPC3 is based on the previous connections designed by Tang, FPC1, and Van der Cruijssen, FPC2. It consists of a single anchor bar welded to a steel gusset plate and cast-in the concrete infill panel. The gusset plate is bolted to another steel plate that is welded to the steel frame. The structural behaviour of this concept can be predicted according to failure of the previous connections.
However the vertical component of the diagonal force is reduced with nearly 50% in comparison to the original position, the possible concrete failure mechanism isn’t avoided with this concept. Weak points of this concept are:

- An anchor bar with a large diameter is needed to transfer shear and tensile forces. The anchor bar needs to be bent 90 degrees with a radius $r \geq 2.5 \phi_c$.
- Due to the dimensions of the anchor bar, the concrete cover is relatively small which can lead to sideward cracks, see figure 2-11.
- Cutting of the steel plate in the concrete will cause splitting cracks, see figure 2-11.

Both crack developments as described above lead to concrete failure and this is not acceptable as failure mode for FPC3. Therefore, these crack developments need to be avoided.

3.3.1.2 Second design

In order to suppress splitting cracks and crack development sideways, the anchor bar can be welded to a UNP section so the concrete will be partially locked up by the steel section, see figure 3-3. Shear tests were performed earlier with this kind of structure used in precast concrete elements$^{[12]}$. Some findings on this connection are:

- The anchor bar can be situated in the centre of the concrete panel. However, concrete cover is not optimal due to the large diameter needed.
- The length of the UNP-section needs to be relatively large as the anchor needs to be welded to the steel section.
- The prefabricated concrete elements are cast horizontally. Therefore, compacting of the concrete in and around the steel section for creating efficient anchor bonding, is relatively difficult.

![Figure 3-3: Second design for the connection](image-url)
3.3.2 Proposed connection FPC3

A new proposal for the design of the connection between steel frame and precast reinforced concrete infill panel consists of a steel anchor plate perpendicular to the diagonal forces to optimize the introduction of the forces by avoiding peak stresses, see figure 3-4. The anchors are parallel to the tension and compression diagonals and are therefore expected to be more effective.

![Figure 3-4 - Proposed connection FPC3](image)

The preference for this concept is based upon the following:

- The anchor bars are aligned with the tension diagonal and therefore more effective in transferring tensile forces. As a result, smaller anchor bars can be used.
- The use of a smaller reinforcing bar diameter will increase the bond strength.\textsuperscript{13}
- Increasing the concrete cover on the reinforcement bar will increase the bond strength.\textsuperscript{13}
- No steel plates are cast-in the concrete panel. Cast-in steel plates will cut into the concrete that causes peak stresses and splitting of the concrete panel in an early stage.\textsuperscript{12}
- The anchors are situated in the centre of the concrete panel.

However, there are some negative consequences to this design for consideration:

- The triangularly shaped steel plate in combination with the position of the connection to the beam causes an asymmetric stress pattern in the plates.
- As a result of the asymmetric stresses in the steel plates, this causes non uniform stresses on the welds which may cause tearing of the weld near the corner.
- The position of the bolts causes an eccentricity in the bolted connection.
3.4 Design calculations for connection

The connection is designed based on a failure mode of bearing in the bolt holes. This paragraph describes the design calculations to determine the bearing resistance of the steel anchor plate. 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. A simplified overview of the reaction forces on the infilled frame subjected to lateral load is shown in figure 3-5.

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 [14]. The results of the tensile tests are listed in Appendix A and paragraph 6.2.1. The average yield strength of the steel anchor plate $f_y = 270$ N/mm$^2$ and an average tensile strength $f_u = 412$ N/mm$^2$. These values are used for further calculation purposes.

\[ F_{b, Rd} = \frac{k_1 \cdot A_b \cdot f_u \cdot d \cdot t}{\gamma_M} \cdot 10^{-3} \]

Calculation

$F_{b, Rd}$ the design bearing resistance per bolt

$k_1$ a factor to take edge distances into account

$k_1 = 2.5$

3.4.1 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 [15]. The bearing resistance of the steel plates can be determined as followed:
The factors $k_1$ and $a_b$ are dependent variables of the edge distance and spacing of the bolt, see figure 3-6. The minimal edge distance ($e_1$ and $e_2$) of the bolt is $\geq 1.2 \cdot d_0$ and the minimal spacing between the bolts ($p_1$ and $p_2$) is $\geq 2.4 \cdot d_0$. 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:

- $k_1$ is the smallest value of: $2.8 \cdot \frac{e_1}{d_0} - 1.7$ or 2.5
- $a_b$ is the smallest value of: $\frac{e_2}{3 \cdot d_0}$ or $\frac{f_u}{f_{ut}}$ or 1.0

Values for edge bolts:

- $f_{ut}$: the ultimate tensile strength of the parent material
- $d$: the nominal bolt diameter
- $t$: thickness of the steel plate

The bearing resistance of a single bolt is:

$$F_{b,br} = \frac{2.5 \cdot 1.0 \cdot 412 \cdot 24 \cdot 10}{1.25 \cdot 10^{-3}} = 197.2 \text{ kN/bolt}$$

The bearing resistance of the connection along the main force direction is 394.4 kN (2x 197.2) 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, 394.4 kN is the governing design load.

### 3.4.2 Block tearing

The bolt holes are situated near the edges of the steel plates which can lead to block tearing if the net section of the steel plate is insufficient to transfer shear and tension stresses. In the case of the triangular steel plates there are three possible combinations of block tearing that must be considered. The net section along the pitch distance is subjected to tension only, see figure 3-7(a). The net section along the edge
distances can be subjected to a combination of tension and shear, as shown figure 3-7(b) and (c).

Only tension (a)  
Combined tension/shear (b)  
Tension and shear (c)  

figure 3-7 - possible failure mechanisms for block tearing

\[ V_{eff,3,Rd} = \frac{f_v \cdot A_{net}}{\gamma_M} + \frac{\tau_{web} \cdot A_{web}}{\gamma_{M2}} \cdot 10^{-3} \]

\( V_{eff,3,Rd} \)  
the design value of the block tearing resistance

\( A_{net} \)  
the net section subjected to tension

\( A_{web} \)  
the net section subjected to shear

As the edge distances were set to 55 mm only the pitch distance is adjusted to keep ovalization of the bolt holes as the governing failure mode. A pitch of 84.8 mm is required to prevent block tearing. The decisive failure mode for block tearing is shown in figure 3-7(b) with a resistance of:

\[ V_{eff,3,Rd} = \frac{412.4 \cdot 1009 + \frac{1}{\sqrt{3}} \cdot 270.2 \cdot 420}{1.25} \cdot 10^{-3} = 398.2 \text{ kN} \]

The other values for the sections of figure 3-7(a) and (c) are respectively 615.0 kN and 412.4 kN

Block tearing of the net section is a governing failure mode for the connection. The position of the bolt holes is specified with preset edge and pitch distances. Elongation of the bolt holes will be the failure mode of the steel anchor plate considering these three effects.

### 3.4.3 Shear resistance of the bolts

Each bolt must be able to resist a maximum force of 197.2 kN/bolt. The previous connections contained 2M24 10.9 bolts. A section of the bolted connection for FPC3 is shown in figure 3-8. The design value of the shear resistance of a single bolt will be calculated according to Eurocode 3[^15].

\[ F_{\tau,Ref} = \frac{\alpha_v \cdot f_{ub} \cdot A}{\gamma_{M2}} \cdot 10^{-3} \]

\( F_{\tau,Ref} \)  
design value of the shear resistance of the bolt

\( \alpha_v \)  
shear plane passes through unthreaded portion of the bolt

\( \alpha_v = 0.6 \)

[^15]: J. A. H. Dekkers
ultimate tensile strength of the bolt material, see table 3-2
f_{ub} = 1000 \text{ N/mm}^2

gross cross section of the bolt
A = 452 \text{ mm}^2

partial safety factor
\gamma_{M2} = 1.25

\[ \begin{align*}
F_{v,Rd} &= \frac{0.6 \cdot 1000 - 452}{1.25} \cdot 10^{-3} = 216.69 \text{ kN/bolt} \\
&\quad \text{(Total = 433.38 kN)}
\end{align*} \]

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

\[ \begin{align*}
F_{v,Rd} &= \frac{0.6 \cdot 800 - 452}{1.25} \cdot 10^{-3} = 173.57 \text{ kN/bolt} \\
&\quad \text{(Total = 347.14 kN)}
\end{align*} \]

Two bolts M24 10.9 are able to resist the maximum load before bearing in the bolt holes occurs.

3.4.4 Weld of the steel gusset plate to beam

The forces introduced by the lateral load on the infilled frame are initiated by the two bolts with a capacity of 394.4 kN. As can be notices from figure 3-9, the centre of the parallel forces is eccentric to the weld which causes non-uniform stresses along the fillet welds.

According to the Eurocode 3 (Part 1-8, article 4.5.3.2)\(^{[18]}\), the design resistance of the fillet weld will be sufficient if the following equations are both satisfied:

\[ \begin{align*}
[\sigma_1^2 + 3 \cdot (r_1^2 + r_2^2)]^{\frac{1}{2}} &\leq \frac{f_{w}}{\beta_{w} \cdot \gamma_{M2}} \quad \text{and} \quad \sigma_2 \leq \frac{0.9 \cdot f_{w}}{\gamma_{M2}}
\end{align*} \]
Dimensions and forces on the connection resulting in stresses on the weld are shown in figure 3-9 and figure 3-10.

$\beta_w$ correlation factor for fillet welds
$f_u$ nominal ultimate tensile strength of the weaker part joined
$\gamma_{M2}$ partial safety factor for welded joints

$\beta = 0.8$
$f_u = 360 \text{ N/mm}^2$
$\gamma_{M2} = 1.25$

External forces on the connection and bolt hole spacing are defined as followed:

\[
F_{b,Rd} = 197.2 \text{ kN}
\]

\[
F_{b,Rd} \cdot \cos (\theta) = 139.4 \text{ kN}
\]

\[
F_{b,Rd} \cdot \sin (\theta) = 139.4 \text{ kN}
\]

$e_1 = 212.4 \text{ mm}$

$e_2 = 92.5 \text{ mm}$

$M_{s,d} = 42.5 \text{ kNm}$

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

As a result of $F_{b,Rd} \cdot \sin (\theta)$

\[
\sigma_y = \frac{2 \cdot F_{b,Rd} \cdot \sqrt{2}}{4 \cdot a \cdot l_{we}} = \frac{2 \cdot 139.4 \cdot 10^3 \cdot \sqrt{2}}{4 \cdot 9 \cdot 275} = 39.8 \text{ N/mm}^2
\]

As a result of $F_{b,Rd} \cdot \sin (\theta) \cdot e$

\[
\sigma_y = \frac{2.12 \cdot M_{s,d}}{a \cdot l_{we}} = \frac{2.12 \cdot 42.5 \cdot 10^3}{9 \cdot 275} = 132.5 \text{ N/mm}^2
\]

As a result of $F_{b,Rd} \cdot \cos (\theta)$

\[
\tau_y = 0 \text{ N/mm}^2
\]

The weld check is as followed:

\[
(1) \quad \left\{ \frac{360}{0.8 \cdot 1.25} \right\} \leq \left\{ \frac{360}{0.8 \cdot 1.25} \right\} \quad \rightarrow \quad 345 \leq 360 \quad \left[ \text{N/mm}^2 \right]
\]

\[
(2) \quad 172.3 \leq \frac{0.9 \cdot 360}{1.25} \quad \rightarrow \quad 172 \leq 259 \quad \left[ \text{N/mm}^2 \right]
\]

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.

3.4.5 Steel anchor bars

The cast-in reinforcement bars welded to the steel T-section must be able to transfer the maximum bearing force from the concrete panel to the steel T-section. Forces can be transferred from concrete to anchor bars due to adhesion, bonding and interlocking of the rib faces which needs a certain slip displacement of the anchor bars. However, if the yield capacity of an anchor bar is exceeded, it loses its bonding strength and large slip displacements occur and thus pull-out failure.

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

\[
N_{d} = 2 \cdot F_{b,Rd}
\]

$N_{d}$ design value of the theoretical tension force
MSc-report - Interface connection of a semi-integral infilled frame

\[ F_{b,\text{res}} = \text{bearing resistance of connection} \]
\[ N = 394.4 \text{ kN} \]
\[ A_s = \frac{N}{f_s} \cdot 10^3 \]
\[ f_s = \text{design value of the tensile strength of the reinforcement} \]
\[ f_s = 435 \text{ N/mm}^2 \]
\[ A_s = \frac{394.4}{435} \cdot 10^3 = 907 \text{ mm}^2 \]

As a result, 5 anchor bars with a diameter of 16 mm are needed to preclude steel failure of the anchor bars. 5\#16, with a section area of, \( A_s = 1005 \text{ mm}^2 \), can resist a maximum theoretical tension force of \( N = 1005 \cdot 435 \cdot 10^{-3} = 437 \text{ kN} \)

### 3.4.6 Anchor embedment length

According to Eurocode 2\[^{116}\] the anchor embedment length of the anchor bars has to be at least:

\[ l_{\text{bd}} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot \alpha_4 \cdot l_{\text{bd,req}} \geq l_{\text{bd,min}} \]

\( l_{\text{bd}} \) design value of the anchorage length

\( \alpha_2 \) t/m \( \alpha_3 \) coefficients equal to 1

\( \alpha_2 \) coefficient for the effect of minimum concrete cover

\( \alpha_2 = 1 - 0.15 \cdot (c_d - 3 \phi) / \phi \geq 0.7 \)

\( \alpha_3 = 1 - 0.15 \cdot (67 - 3 \cdot 16) / 16 = 0.70 \)

\( c_d \) concrete cover

\( l_{\text{bd,req}} \) the necessary basic anchorage length

\( l_{\text{bd,min}} \) the minimal anchorage length, \( l_{\text{bd,min}} > \text{maximum of } (0.3 \cdot l_{\text{bd,req}} ; 10 \cdot \phi ; 100) \text{ mm} \)

\[ l_{\text{bd,min}} = \frac{\phi}{4} \cdot \frac{\sigma_{\text{ad}}}{f_b} \]

\( \phi \) anchor diameter

\( \sigma_{\text{ad}} \) design value of the anchor stress

\( f_b \) ultimate adhesive strength

\( \sigma_{\text{ad}} = 393 \text{ N/mm}^2 \)

\( f_b = 3.37 \text{ N/mm}^2 \)

\[ l_{\text{bd,min}} = \frac{16}{4} \cdot \frac{393}{3.37} = 466 \text{ mm} \]

\[ l_{\text{bd}} = 0.70 \cdot 466 = 326 \text{ mm} \]

The minimum anchorage length to transfer the force to resist the maximum bearing strength of the bolt holes has to be 294 mm. An additional check of splitting of the concrete is necessary.

According to analytical research by Wang\[^{131}\] the embedment length of the reinforcement bars to prevent splitting of the concrete has to be:
The anchors will have an embedment length of 420 mm according to this configuration.

3.4.7 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 78.9 kN (394.4/5) through the weld into the steel plate. The effective weld length around an anchor bar Ø16 is 63.6 mm. A weld with a throat of 6 mm is required to be able to resist a maximum tension load of 79.3 kN and has therefore a unity check of 0.99. The minimum value of the tensile strength of 360 N/mm² for steel S235 is used, and thus a conservative assumption. Dimensions and positions of the welded steel anchor bars are shown in figure 3-11.

\[
\frac{I_d}{d_b} = \frac{1.85 \cdot f_{y,t}}{\left(\frac{c}{b_r}\right)^{0.5}} \cdot \sqrt{f_c}
\]

\[
l_d = \frac{1.85 \cdot 393}{16 \left(\frac{67}{16}\right)^{0.5}} \cdot \sqrt{27} \rightarrow l_d = 413 \text{ mm}
\]

The anchors will have an embedment length of 420 mm according to this configuration.

3.4.7 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 78.9 kN (394.4/5) through the weld into the steel plate. The effective weld length around an anchor bar Ø16 is 63.6 mm. A weld with a throat of 6 mm is required to be able to resist a maximum tension load of 79.3 kN and has therefore a unity check of 0.99. The minimum value of the tensile strength of 360 N/mm² for steel S235 is used, and thus a conservative assumption. Dimensions and positions of the welded steel anchor bars are shown in figure 3-11.

\[
f_{w,t} = \frac{360}{\sqrt{3 \cdot 0.8 \cdot 1.25}} = 207.8 \text{ N/mm}^2
\]

\[
F_{w,R} = 207.85 \cdot 6 = 1247.1 \text{ N/mm}
\]

Figure 3-11 - Overview anchor position and effective length welds
\[ I_{\text{eff}} = 2 \cdot \pi \cdot r^2 = 2 \cdot \pi \cdot \left(8 + \frac{1}{3} \cdot \frac{1}{2} \right) = 63.6 \text{ mm} \]
\[ F_{w,d} = 1247.1 \cdot 63.6 \cdot 10^{-3} = 79.3 \text{ kN} \]

### 3.4.8 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 paragraph and illustrated in figure 3-12. A nominal weld throat of 5 mm will be used to distribute the forces from the anchors to the steel gusset plate.

![Weld length of steel plate](image)

The average maximum force on the anchor bar is 78.9 kN as stated in the previous paragraph. This will result in a necessary weld length between the two steel plates using the following formulas:

\[ F_{w,m} = f_{w,d} \cdot a \]
\[ f_{w,d} = \frac{f_y}{\sqrt{3} \cdot \beta_w \cdot F_{M2}} \]
\[ f_{w,d} = \frac{360}{\sqrt{3} \cdot 0.8 \cdot 1.25} = 207.85 \text{ N/mm}^2 \]
\[ F_{w,m} = 207.85 \cdot 5 = 1039.2 \text{ N/mm}^2 \]
\[ I_{w,m} = \frac{5 \cdot 78.9 \cdot 10^3}{2 \cdot 1039.2} = \frac{394.4 \cdot 10^3}{2 \cdot 1039.2} = 190 \text{ mm} \]

The fillet weld at each side of the plate has to be at least 190 mm. The weld length is 398 mm per side.
3.4.9 **Bending of the steel anchor plate in compression**

The connection along the compression diagonal will be loaded eccentrically which results in bending and compression in the steel anchor plate. Strength and stability checks are performed according to the Eurocode 3[^14]. Figure 3-13 shows the connection under compression and the simplified model to determine the effect of eccentricity on the steel anchor plate.

![Diagram showing bending due to eccentric compressive force without support](image)

**Calculation**

### 3.4.9.1 Section check

The steel section of the anchor plate is checked as unsupported section with a single fixed connection. The following unity check must be satisfied:

\[
\frac{M_{ud}}{M_{u}} \leq 1.0
\]

The origin of the compressive force has an eccentricity of 12.5 mm, which results in a bending moment of

\[
M_{ud} = 394.4 \cdot 0.0125 = 4.93 \text{ kNm}
\]

Due to the bending and compression in the steel section, the reduced design value of the bending resistance in combination with normal forces is:

\[
M_{ud,red} = M_{u,red} \left[ 1 - \left( \frac{N_{ud}}{N_{u,red}} \right)^2 \right] = \frac{8200 \cdot 260}{1.0} \left[ 1 - \left( \frac{394.4 \cdot 10^1}{3280 \cdot 260} \right)^2 \right] = 1.68 \text{ kNm}
\]
As a result, the unity check for the steel section is:

\[
\frac{M_{Ed}}{M_{Ed,\text{ult}}^c} = 2.93 \geq 1.0
\]

Failure of the steel section can be expected along the compression diagonal before ovalization of the bolt holes occurs.

**Stability check**

The critical elastic force is determined by:

\[
N_{Ed} = \frac{\pi^2 \cdot E \cdot I}{l_0^2} = \frac{\pi^2 \cdot 2.1 \cdot 10^3 \cdot 27300}{(2 \cdot 69.0)^2} \cdot 10^{-3} = 2971 \text{ kN}
\]

\[
N_{Ed} = 394.4 \text{ kN}
\]

\[
\frac{N_{Ed}}{N_{cr}} = 0.13 > 0.04
\]

Buckling effects of the steel section cannot be ignored. The steel section subjected to compression and bending has to fulfill:

\[
\frac{N_{Ed}}{N_{cr}} + k_y \frac{M_{Ed,\text{ult}} + \Delta M_{Ed,\text{ult}}}{M_{cr,Y}} \leq 1.0
\]

The design value of the normal force on the steel section is:

\[
N_{Ed} = 394.4 \text{ kN}
\]

The characteristic resistance against the normal force is:

\[
N_{Rk} = 3280 \cdot 260 \cdot 10^{-3} = 852.8 \text{ kN}
\]

The reduction factor, to take the buckling effect of the steel member into account, will than be

\[
X_s = \frac{1}{\Phi + \sqrt{\Phi^2 - \lambda_2^2}} = \frac{1}{0.73 + \sqrt{0.73^2 - 0.54^2}} = 0.82
\]

Where the relative fitness of the steel section can be determined by:

\[
\lambda_2 = \sqrt{\frac{A \cdot f_y}{N_{cr}}} = \sqrt{\frac{3280 \cdot 260}{2971 \cdot 10^{-3}}} = 0.54
\]

And value \(\Phi\):

\[
\Phi = 0.5 \left[1 + \alpha (\lambda_2 - 0.2) + \lambda_2^2 \right] = 0.5 \left[1 + 0.49 (0.54 - 0.2) + 0.54^2 \right] = 0.73
\]

The design value of the bending moment along the y-axis is:

\[
M_{Ed,\text{ult}} = 0
\]

The interaction factor is:

\[
J. A. H. Dekkers
\]
The design value of the bending moment along the z-axis is:

\[ M_{z,Ed} = 4.93 \text{kNm} \]

The characteristic resistance against the bending moment along the z-axis is:

\[ M_{z,Rk} = 8200 \cdot 260 \cdot 10^{-3} = 2.13 \text{kNm} \]

The unity check therefore is determined by:

\[
\frac{394.4}{0.82 \cdot 852.8} \cdot 0.8 \cdot 0.8 \cdot 2.13 = 0.56 + 0.8 \cdot 2.31 = 2.41 \geq 1.0
\]

Strength and stability of the steel anchor plate subjected to an eccentric compression force, resulting in compression and bending in the steel section, are not satisfied. Laterally supporting the steel plate will reduce the eccentric force which causes bending in the steel plate, see figure 3-14.

3.5 Summary

The previous paragraphs discussed the different structural parts of the connection. It can be expected that ovalization of the bolt holes will be decisive for failure of the connection. The designed and calculated dimensions and properties are used to assemble the connection for experimental testing. This will be described in the next chapter.

Along the compression diagonal it is recommended to support the steel anchor plate in order to prevent failure of the steel section. This failure mode will not be discussed further in upcoming chapters. Only the connection subjected to tension is part of this research.
4 Experimental testing of FPC3

4.1 Introduction

This chapter contains an elaboration on the experimental investigation of the interface connection tested at the Pieter van Musschenbroek laboratory to get insight into the structural behaviour of the designed connection. A test set-up was developed to perform tension tests on four specimens, performed in collaboration of Custers and Dekker[18]. The four tests are discussed separately with a description of the test and preliminary conclusions are made for further analysis. Earlier connections designed by Tang and Van der Cruijssen[17] were tested in orthogonal directions as described in paragraph 2.1. The experimental investigation included shear tests and tension tests. The results were used to define spring stiffness's in both directions for a numerical model of an infilled frame. A full-scale test of this frame was performed to calibrate the numerical model. In this case the results of the partial tests will also be used to define spring stiffness' for further analysis of the infilled frame. This is not included in the objectives of this research project. The results are also used to verify the numerical model of the partial test set-up. Images and extended data of the experimental testing can be found in Appendix B.

4.2 Test specimen

The new connection as described in the previous chapters will also be tested in the Pieter van Musschenbroek laboratory to verify the design calculations and to be able to describe the structural behaviour with a numerical model. The test series will include four partial tests where the connection will be loaded with a tensile force, see figure 4-1.

figure 4-1 - test set-up
4.2.1 Anchor plate

The triangular T-section is assembled from a steel plate of 275 x 275 x 10 mm welded to a 389 x 150 x 15 mm steel flange plate. The flange is welded to five anchor bars which are cast in the concrete element. The T-section, shown in figure 4-2, is bolt-connected to a loading plate with a thickness of 20 mm. This connection consists of two M24 (10.9) bolts with washers on both sides because of the asymmetric connection, as described in NEN-EN 1993-1-8; paragraph 3.6.1[15]. The triangular T-section is expected to fail in ovalization of the bolt holes at a load of approximately 394 kN.

![Welded anchor plate](image)

figure 4-2 - welded anchor plate

4.2.2 Concrete element

The concrete element of the tension test which represents the infill panel of an infilled frame is cast at the laboratory with a concrete quality of C35/45. Because of the configuration of the test set-up, the concrete element contains an internal beam with a reinforcement cage to transfer forces to the supports. The thickness of the element had to be adjusted to 175 mm instead of 150 mm because of unforeseen rebar congestion. Reinforcement mats consisting of Ø8-100# are applied to both sides of the precast concrete (wall) element as shown in figure 4-3.
4.2.3 *Connection between concrete and anchor plate*

The connection between the concrete element and the steel anchor plate comprises five cast-in anchor bars ø16 FeB500 with a length of 420 mm, welded to the triangular T-section.

4.3 *Test set-up*

The test set-up shown in figure 4-4 consists of HEB300 steel sections and is based on the tensile test set-up used by Tang and Van der Cruijssen[7]. The precast concrete element, which represents a single corner of the infilled frame, is placed on two hydraulic jacks. The two jacks are pushing the concrete element upwards and create a symmetric tensile force along the interface connection. The steel anchor plate is connected to a 20 mm thick steel loading plate with two bolts M24 10.9. The test set up is shown in figure 4-4. An external frame was built to support the measurement devices consisting of linear variable differential transformers (LVDT) as shown in figure 4-1.
4.3.1 Loading

The applied force on the specimen is measured according to the oil pressure of the hydraulic jacks. The two jacks are fed by a 2 MN hydraulic pressure bench. The load application by the two jacks is displacement controlled to determine the structural behaviour of the connection after failure. A testing speed of the 2 MN hydraulic pressure bench was set to a cycle of 200 minutes/200 mm which corresponds to 1.0 mm/minute. At a force of 100 kN the testing speed was adjusted to 0.5 mm/minute.

4.3.2 Measurements

Three different types of measurements were used to obtain data from the tests for further analysis to calibrate the FE model used for the parametric study of the research. These measurements were constructed to determine anchorage slip, ovalization of the bolt holes and concrete/steel strains on several locations. Displacements at several locations on the test specimen were measured from the outer frame with LVDT transducers.

4.3.2.1 Ovalization of bolt holes

Six LVDT’s are situated near the bolts, figure 4-5. L02, L04 and L06 describe displacements of the steel plate. Subtracting an average displacement from the displacement measured from L03 and L05, this will result in the total elongation of the bolt hole. Results of the first test did not meet the expectations and therefore the measurement configuration was adjusted. This alteration will be discussed at the preliminary results of the first test, see paragraph 4.4.2.

4.3.2.2 Anchorage slip

The design of the longitudinal springs along the tensioned diagonal of the infilled frame also depends on anchorage slip. To be able to measure slip of the anchor bars cast-in the concrete panel, two LVDT’s (L00 and L01) were used. The difference between the top of the anchor plate and the adjacent concrete was measured on the front and back of the panel, see figure 4-5.
4.3.2.3 Strains

Several strain gauges are placed on the concrete element and the steel anchor plate to gain data of the strain state of the material during the test. This data is used to calibrate the FEM model. Strain gauge positions are shown in figure 4-6.

4.4 Test JD1011T01

The first specimen was cast on November 10th 2009 and tested 28 days later on December 8th 2009. Three
cubes were also tested to determine the cubic compression strength of the concrete on the day of the main test. The compressive strength of the concrete cubes was determined according to Appendix A and came to an average of 57.4 N/mm².

4.4.1  Testing

At a force of 100 kN the testing speed was unintentionally set to 400 minutes/200 mm instead of 100 minutes/200 mm. First small cracks on the surface of the concrete panel appeared at the edge perpendicular to the anchor plate. Larger cracks emerged at the end of the anchorage length of the anchor bars at approximately 233 kN together with a reduction of the force. From this point on several crack patterns appeared until failure without any major load fall back. Existing cracks grew larger and wider. The specimen failed at a maximum load of 372 kN, which means a reduction of 6% to the calculated failure load.

4.4.2  Discussion of preliminary results

Ovalization of the bolt holes was predicted to be governing. However the specimen failed otherwise. At failure, the load dropped and the displacement of the concrete element increased contrary to the displacement of the steel connection and therefore anchor pull-out was evident as the anchors were still attached to the anchor plate, figure 4-7(a).

After disassembling the specimen from the test set-up ovalization of the bolt holes was clearly visible, figure 4-7(b). The diameter of the bolt holes at the start of the test was 26 mm and afterwards it measured 29.1 and 29.2 mm. As a result the ovalization was 3.1 and 3.2 mm respectively.

Further inspection of the specimen revealed a larger rotation of the bolts than expected because of the asymmetric loading in the connection. Ovalization of the bolt holes could not be described clearly with the
measurement set-up as the displacement of the bolt at the steel plate could not be determined properly, see figure 4-8(a).

![Diagram](image)

figure 4-8 - cracks during the test T01(a) and the effect of rotation of the bolts (b)

Analysis of the retrieved data from the LVDT's compared to the measured ovalization afterwards, confirms the inaccuracy of the measurements. Therefore several adaptations were made to the measurement configuration of the LVDT’s, see paragraph 4.4.3. The data measured from the different LVDT's to determine ovalization of the bolt holes and slip of the anchor bars is represented in figure 4-9. The presented force is the total force subjected to the test specimen.

![Graphs](image)

figure 4-9 - force-displacement graphs T01 to determine ovalization (a) and slip of anchor bars (b)

### 4.4.3 Adjusted measurements

The LVDT positions for measuring the displacements of the steel plate are placed above the bolts. Strains
above the bolts are assumed to be minimal and are therefore neglected. Differences between the displacement of the bolts and steel plate are measured more directly in order to determine ovalization of the bolt holes. To take the rotation of the bolt into account as well, two LVDT's are placed on the back of the bolts. The determination of the ovalization of the bolt holes will be discussed in paragraph 5.2. The adjusted measurements are shown in figure 4-10 and figure 4-11.

![Figure 4-10 - Adjusted LVDT positions for remaining tests T03, T02 and T04](image)

![Figure 4-11 - Adjusted LVDT positions, section A](image)
4.5 Test JD1011T03

On December 12th 2009 the second specimen was tested with the adjusted measurement configuration. Another three concrete cubes were tested to determine the cubic compression strength. The compressive strength after 30 days came to an average of 57.6 N/mm².

4.5.1 Testing

The testing speed started at 0.5 mm/minute and was set to 0.25 mm/minute at a force of approximately 100 kN. Similar crack patterns were noticeable as in the first test. First small cracks appeared at the base of the concrete element and the first major cracks with a force fall back emerged at a load of 232 kN, see figure 4-12(a). At 355 kN the specimen failed analogously to the first test. The slip measured by LVDT L00 and L01 is shown in figure 4-12(b).

![Diagram of cracks during test T03(a) and force-displacement graph of L00/L01(b)]](https://example.com/diagram.png)

4.5.2 Discussion of preliminary results

The second test failed at a load 11% below the design value for ovalization of the bolt holes. Again, the desired failure mode did not occur as the specimen failed due to anchor pull-out. Ovalization of the bolt holes was noticeable. However, the measured elongation of both bolt holes is dissimilar with an ovalization of 4.0 mm for the left bolt hole and 1.5 mm for the right, figure 4-13. A possible explanation of this occurrence is uneven loading on the bolts. It was also mentioned that the bolts were tightened tighter than the bolts of the first test. As a result one bolt caused friction between the two steel plates during the first part of the test rather than bearing. Force-displacement graphs measured from the lvdt’s are exemplified in figure 4-14. The eccentric anchor pull-out may have indicated an asymmetrical loading on the concrete element. Two more measuring devices are added to the test set-up to determine the displacement on either side of the concrete element.
4.6 Test JD1011T02

Specimens T02 and T04 were tested on December 14th 2009. The last three concrete cubes were tested to determine the cubical compression strength. After 34 days the compressive strength came to an average of 59.7 N/mm².

4.6.1 Testing

The installation of the test specimen was done carefully to avoid the element being out of alignment and the bolts were tightened with care. Thereby, friction between the steel plates was minimized and ovalization of the bolt hole was not hindered.

A testing speed of 1.0 mm/minute was set for the first part of the test and was adjusted at a force of approximately 140 kN to 0.5 mm/minute. Similar crack patterns emerged during the test, see figure 4-14 - force-displacement graphs T03 for bolts (a) and steel plate (b).

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A load drop of 10 kN caused by a major crack as described in previous tests occurred at a load of 243 kN. The specimen failed at 376.6 kN due to anchor pull-out.

![Diagram](image1)

**figure 4-15** - cracks during test T02 (a) and force-displacement graph of LOO/LO1 (b)

### 4.6.2 Discussion of preliminary results

The calculated load for the desired failure mode is 4.7% higher than failure load that did occur. Ovalization of the bolt holes emerged during the test with a maximum of 3.0 mm at the end of the test. Again, the desired failure mode did not occur as the specimen failed due to anchor pull-out.

![Image](image2)

**figure 4-16** - ovalization of the bolt holes (a) and overview specimen after test (b); T02
4.7 Test JD1011T04

The last specimen was also tested on December 14th, see paragraph 4.6.

4.7.1 Testing

Preparations of the last specimen of this test series were similar to the previous test. A testing speed of 1.0 mm/minute was set for the first part of the test and was adjusted at a force of approximately 140 kN to 0.5 mm/minute. Unlike the first three tests no large set back occurred during the test. Two relatively small load drops were noticeable at a force of 240 kN and 255 kN. The large horizontal crack at the end of the anchor bars that was visible in other tests came out smaller in this test at a load of 255 kN, see figure 4-18(a). The test ended at a maximum load of 340.1 kN together with loud thumping from inside the concrete element. Anchor pull-out was the failure mode of the connection.

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4.7.2 Discussion of preliminary results

The maximum load of the last test is below previous values with a difference of 16% to the calculated value for the desired failure mode. Elongation of the bolt holes was measured afterwards with an ovalization of 2.7 and 2.4 mm for the left and right bolt respectively, figure 4-19.

![Figure 4-19 - Ovalization of the bolt holes; T04](image)

Figure 4-19 shows the force-displacement graphs constructed for the displacement of the bolts and the positions above the bolts. Ovalization should be determined by subtracting the displacement of the bolts near the steel plate from the displacement above the bolts. This will be explained in the next chapter about the definition of the spring stiffness' for ovalization of the bolt holes and slip of the anchor bars.

![Figure 4-20 - Force-displacement graphs T04 for bolts (a) and steel plate (b)](image)
4.8 Conclusions from experimental research

It can be concluded that none of the tests did fail in ovalization of the bolt holes at a maximum load of 394 kN as calculated. Instead, they failed on combined anchor pull-out and bolt hole ovalization with maximum loads between 340 and 376 kN. This is an average of 361.1 kN and 9% below the design value. The connection failed due to loss off bonding between anchor bars and concrete as the anchor bars were still connected to the anchor plate and the concrete panel did not break off.

After disassembling the test specimens it was noticed that all steel anchor plate were deformed around the bolt holes and therefore it can be concluded that ovalization of the bolt holes did occur. An average elongation of the bolt hole was determined at 2.85 mm with a minimum and maximum of 1.5 mm and 4.0 mm respectively. Both ultimate values were measured at the end of test T03. The measured values are stated in table 4-1.

The primary cracks were initiated along the end of the embedment length of the anchor bars at approximately, two-third of the average ultimate load, 240 kN. Rotation of the bolt holes caused by the eccentric connection of the steel plates was larger than expected and led to a different measurement configuration to determine the ovalization of the bolt holes.

The tested connection is based on the tension diagonal of the infilled frame. Besides ovalization of the bolt holes, which also occurs along the compression diagonal, slip of the anchor bars also determines the connection’s stiffness.

<table>
<thead>
<tr>
<th>Table 4-1 - measured ovalization of the bolt holes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test</td>
</tr>
<tr>
<td>--------</td>
</tr>
<tr>
<td>T01</td>
</tr>
<tr>
<td>left</td>
</tr>
<tr>
<td>right</td>
</tr>
<tr>
<td>T03</td>
</tr>
<tr>
<td>left</td>
</tr>
<tr>
<td>right</td>
</tr>
<tr>
<td>T02</td>
</tr>
<tr>
<td>left</td>
</tr>
<tr>
<td>right</td>
</tr>
<tr>
<td>T04</td>
</tr>
<tr>
<td>left</td>
</tr>
<tr>
<td>right</td>
</tr>
</tbody>
</table>
5 Simplified spring models of connection

5.1 Introduction

The results of the experimental tests are used to calibrate a FEM model to draw conclusions and make recommendations for the designed connection. This chapter discusses two spring models for simplified FE analyses of a full scale infilled frame. The simplified FE analyses are not a subject of this study.

Force-displacement relations are discussed for representing ovalization of the bolt holes and slip of the anchor bars separately. Ovalization of the bolt holes at the T-shaped steel anchor plate (t=10 mm) is taken into account. Two possible model concepts to implement longitudinal springs into a FEM full-scale model of an infilled frame are shown in figure 5-1. Two combined springs are created to model the behaviour of the connection with two bolts along the tension diagonal. The model of the connection along the compression diagonal consists of two compression springs representing only ovalization of the bolt holes.

![Figure 5-1 - two FEM concepts to represent the connection for a full-scale model](image)

5.2 Ovalization

Ovalization of the bolt holes in the steel anchor plates is not measured directly. As stated before the set-up of the first test did not come off correctly to be able to describe the elongation of the bolt holes due to rotation of the bolts. This rotation was taken into account in the remaining tests as the bolt movement was measured on both sides of the test set-up. As a result the displacement of the bolt can be determined at any position along the length of the bolt. The displacement of the bolt at the steel plate is obtained by using the displacements $\delta_1$ and $\delta_2$ in relation to the position of the steel plate and its distance to the back of the bolt where $\delta_2$ is measured, see figure 5-2.
Ovalization of a bolt hole, regarding only the 10 mm steel anchor plate, can be described by subtracting the displacement of the bolt at the centre of the steel plate, $\delta_{x1}$, from the displacement of the steel plate above the bolt, $\delta_3$. Strains above the bolt holes are minimal and therefore neglected. Simplified calculations result in an approximated strain of $6.2 \times 10^{-4}$ mm/mm at failure load for test T02. The measurement point above the bolt is situated 22 mm above the centre of the bolt hole, causing a maximum extension of 0.01 mm. The ovalization of the bolt holes can be determined by subtracting the graphs, $\delta_3 - \delta_{x1}$, plotted in figure 5-3.

The maximum values for the elongation of the bolt holes in figure 5-3 are compared with the values measured after each test, see table 5-1. The first test is not representative because of the absence of...
measuring points at the back of the bolts and therefore the rotation of the bolts is not described which lead to incorrect values. The second test (T03) cannot be used as well because the difference regarding to the measured ovalization after the test and the ovalization calculated from LVDT measurements is over 47%. The average ovalization of the bolt holes is presented in figure 5-4.

![Figure 5-4 - average ovalization in the bolts holes](image)

<table>
<thead>
<tr>
<th>Test</th>
<th>Left</th>
<th>Right</th>
<th>Measured after test</th>
<th>From LVDT measurements</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>T01</td>
<td>3.1</td>
<td>3.1</td>
<td>3.1</td>
<td>-</td>
<td>- %</td>
</tr>
<tr>
<td>T03</td>
<td>4.0</td>
<td>1.5</td>
<td>2.1</td>
<td>2.2</td>
<td>48%</td>
</tr>
<tr>
<td>T02</td>
<td>3.0</td>
<td>3.0</td>
<td>3.0</td>
<td>2.9</td>
<td>3%</td>
</tr>
<tr>
<td>T04</td>
<td>2.7</td>
<td>2.4</td>
<td>2.5</td>
<td>2.3</td>
<td>7%</td>
</tr>
</tbody>
</table>

*Ovalization of the bolt holes could not be determined properly*

Test T02 and T04 are used to describe the behaviour for ovalization of the two bolt holes as a longitudinal spring. The average ovalization for the two bolt holes is plotted in figure 5-5 along with the modelled behaviour of a proposed longitudinal spring. Anchor pull-out was determined to be indicative as failure mode for the connection and is not taken into account for this case. This will be discussed in the total spring model.

The elastic leg of the graph which represents the serviceability state is determined by the secant stiffness at two-third of the pull-out capacity \(F_v = \frac{3}{2} F_u\). A yield capacity of 240 kN is also in accordance with the initiation of the first cracks in the concrete panel. At a value of 240 kN the secant stiffness for the elastic phase is calculated to be 296 kN/mm. The primary plastic leg contains a stiffness of 117 kN/mm succeeded by a second plastic branch with a stiffness of 41 kN/mm.
5.3 Spring for anchorage slip

Bonding between anchor bars and concrete consists of chemical adhesion, mechanical interlock and friction resistance. As a tension force is applied to the connection, it is causing slip of the anchor bars along the embedment length. The gap between the concrete panel and the steel anchor plate is directly measured at the concrete to steel interface to determine the bond stiffness of the anchor bars. Calculated stiffness due to anchorage slip along the tension diagonal of the infilled frame is taken into account for the overall spring model of the connection. Behaviour of anchorage slip is also described with an elastic phase and a plastic phase until pull-out occurs, which results in a negative slope.
The upper limit of the load is determined by the average of the maximum pull-out capacity obtained from the test series. Averages of four tests are used to determine the elastic secant stiffness at $F_v$ and the plastic secant stiffness at $F_w$, see figure 5-6. The spring stiffness' are determined at 790 kN/mm for the elastic phase and 233, 143, 0 and -74 kN/mm respectively for the plastic phase of anchorage slip and pull-out.

5.4 Total spring behaviour

An infilled frame contains four connections to attach the precast concrete infill panel to the steel frame. Due to lateral loading two connections are loaded with a tension force as the other connections have to transfer a compression force. The connection along the tension diagonal of the infilled frame is subjected to ovalization of the bolt holes and slip of the anchor bars. The overall stiffness of the connection is determined by the ovalization resistance of the steel plate and the bond-slip resistance of the bonding between anchor bars and concrete. Combining the spring stiffness' for bolt hole ovalization and anchorage slip in series yields the overall spring behaviour of the connection, see figure 5-7. To combine both spring systems into one equal spring, containing the elongation of the bolt holes as well as the anchorage slip, the overall stiffness is determined by adding the reciprocal value for the ovalization stiffness to the reciprocal value from the slip stiffness. In that case, the elastic leg can be represented by:

$$K_{el,	ext{total}} = \left( \frac{1}{K_{oval}} + \frac{1}{K_{slip}} \right)^{-1} = \left[ \frac{1}{296} + \frac{1}{790} \right]^{-1} = 215 \text{ kN/mm}$$

At a load of 240 kN the first cracks occur and the total spring stiffness is decreased to respectively 77, 32, 0 and -74 kN/mm where anchor pull-out is the governing failure mode.

![Modelled behaviour of the connection spring for ovalization of two bolt holes and the overall anchorage slip](image-url)
The maximum load is limited to 361.1 kN due to anchor pull-out. The behaviour of the springs along the compression diagonal is not limited by anchor pull-out nor anchorage slip or tensional cracking and can therefore be taken as multi-linear according to figure 5-5, until the maximum design load of 394.4 kN from paragraph 3.4.1 is reached. The overall behaviour of a single spring represented by one bolt connector can be described by the graph from figure 5-8. The figure also contains the calculated spring stiffness for ovalization of FPC2 based on the tension test. It can be concluded that the strength and stiffness of FPC3 is an improvement on FPC2.

![Graph showing modelled behaviour for a combined spring of the discrete connection based on a two bolt connection.](image)
6 Numerical research

6.1 Introduction

The experimental tests on the connection are based on the tensional diagonal of the semi-integral infilled frame. A nearly symmetric force was applied to the specimen to observe global behaviour of the connection as well as to measure anchor pull-out force and ovalization of the bolt holes. However, due to the eccentric connection of the triangular steel T-section to the steel frame, the load will cause a non-symmetric stress path. As a result the anchors are loaded with non-uniform tensile forces. The experimental behaviour has been analyzed by creating a simplified two-dimensional FEM model of the tested connection. The FEM model is compared against experimental results. Due to unexpected concrete failure and an unfinished bearing failure of the bolt holes, the numerical analysis is divided into two separate FEM models. The first model represents the steel lap joint subjected to a contact load by the bolt in order to investigate the structural behaviour of the bearing failure. The second model contains the reinforced concrete panel with cast-in anchor bars subjected to a tension load. In the present study, the DIANA (Version 9.4) finite element package is used to create a finite element model and carry out the analyses.

6.2 Steel anchor plate

A simplified two-dimensional FEM model of the steel anchor plate with two bolt holes connected to a concrete infill panel is created in order to perform structural linear and non-linear analyses. The experimental data for describing the ovalization of the bolt holes is used for comparison.

6.2.1 Steel coupon tests

A total number of five tensile tests have been carried out to obtain the mechanical properties of the material used for experimental testing of the partial test described in chapter 4. An average yield strength of 260 N/mm² was obtained. Each coupon, 20 mm width by 230 mm length, was axially loaded up to rupture, and displacement transducers have been used to measure the deformation of the coupons during the tensile test.

Table 6-2 summarizes the measured material properties and parameters of the steel material; the actual thicknesses of the steel materials were measured from the steel strips before testing. The stress-strain curves of the steel strips are obtained from the measured load-extension curves of the coupons.
### Table 6-1 - Measured Material Properties of Steel Plates in Coupon Tests

<table>
<thead>
<tr>
<th>Test Specimen</th>
<th>Size (b x t)</th>
<th>Elastic Modulus</th>
<th>Yield Strength</th>
<th>Tensile Strength</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>nominal [mm]</td>
<td>actual [mm]</td>
<td>[kN/mm²]</td>
<td>[N/mm²]</td>
</tr>
<tr>
<td>1</td>
<td>20 x 10</td>
<td>19.63 x 9.74</td>
<td>210.4</td>
<td>273.4</td>
</tr>
<tr>
<td>2</td>
<td>20 x 10</td>
<td>20.08 x 9.76</td>
<td>219.4</td>
<td>257.1</td>
</tr>
<tr>
<td>3</td>
<td>20 x 10</td>
<td>20.06 x 9.76</td>
<td>213.0</td>
<td>255.4</td>
</tr>
<tr>
<td>4</td>
<td>20 x 10</td>
<td>20.01 x 9.77</td>
<td>214.9</td>
<td>258.7</td>
</tr>
<tr>
<td>5</td>
<td>20 x 10</td>
<td>20.01 x 9.77</td>
<td>217.4</td>
<td>259.2</td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td></td>
<td></td>
<td><strong>215.0</strong></td>
<td><strong>260.0</strong></td>
</tr>
</tbody>
</table>

#### 6.2.2 FEM Modelling

The first FEM model is based on the steel anchor plate with two bolt holes with a diameter of 26 mm. The forces are introduced by a M24 bolt which results in a non-uniform stress distribution at the contact area during loading. The stress distribution along the steel plate caused by the bolt-to-bolt hole contact will create peak stresses that exceed the yield strength of the material. Figure 6-1 shows a qualitative representation of the Von Mises stresses, research by Chung\(^{19}\), across the width of the steel section in the vicinity of the bolt hole at different states of ovalization. Due to large local deformations, the stress distributions vary significantly in both magnitude and shape during the course of the load application.

![Figure 6-1](image)

**Figure 6-1** - Qualitative representation of the Von Mises stress distribution of bolted connections at different bolt displacements

The lap joint has been simplified in the finite element environment as a two-dimensional model where eccentric behaviour is neglected. The problem requires a large strain elasto-plastic analysis with contact to capture the force initiation and the expansion of the contact area between bolt and bolt hole. A physical non-linear analysis is needed to describe plastic growth in the connection.

The FEM model of the steel anchor plate consists of only one half of the section thereby exploiting structural symmetry. Two-dimensional eight-node iso-parametric elements (CQ16M for plane stress and CQ16E for plane strain) have been employed to model the steel components, namely, the steel anchor plate and the bolt. Contact between the anchor plate and the bolt, has been modelled with contact...
elements (CL6T), so that assumptions on the position and the size of contact area during the analysis are not required. Contact elements are special interface elements to model zones of possible contact between a 'contacter' and a 'target'. DIANA recognizes and effectuates contact between contact elements if a 'contacter' touches the outside of a 'target', see figure 6-2. For line contact elements, i.e., two-dimensional analysis, the element y-axis defines the outside of the 'target' element[20]. For the current analyses, the edge of the bolt operates as 'target' elements and the edge of the bolt hole as 'contacter' elements, figure 6-2.

![Contact and target elements](image)

The steel plate is fixed in space along the upper edge. Structural symmetric boundary conditions have been imposed on the left edge. The bolt is loaded with a displacement of -5.0 mm along the y-axis. The bolt is assumed to be linear-elastic throughout the analysis to avoid convergence difficulties. The finite element mesh has been refined locally along the bolt hole to capture an accurate stress distribution, figure 6-3.

![Finite element mesh overview](image)

The 'true' stress–strain curve, shown in figure 6-4, established from material tensile tests are implemented as material properties for the steel anchor plate in order to model both yielding and stiffness degradation during deformation.
The steel coupon cross-sectional area and length change with the onset of plastic deformation. The cross-section becomes smaller and so stress increases. To take account of the change of the dimensions of the steel coupon the true stress and true strain are defined as

\[ \sigma_{\text{true}} = \sigma_0 (1 + \varepsilon_0), \quad \varepsilon_{\text{true}} = \ln(1 + \varepsilon_0) \]

\( \sigma_{\text{true}} \) = true stress [N/mm²]
\( \sigma_0 \) = engineering stress [N/mm²]
\( \varepsilon_{\text{true}} \) = true strain [mm/mm]
\( \varepsilon_0 \) = engineering strain [mm/mm]

This relationship is only valid up to the ultimate tensile stress and accompanying strain.

A material, geometrical and contact nonlinear analysis has been carried out to obtain the load–extension curves of the bolted connection. The finite element modelling can provide more detailed information on the yield zones of the steel anchor plate and the stress distribution acting on the anchor bars welded to the triangular T-section. Three different models have been used for the simplified two dimensional analysis to approximate the experimental test including at first, only plane stress elements, secondly, only plane strain elements and a combination of both element types as a third analysis, see figure 6-5. Plane stress is a state of stress in which the normal stress and the shear stresses perpendicular the x-y plane are assumed to be zero, whereas plane strain is a state of strain in which the normal strain and the shear strain perpendicular to the x-y plane are assumed to be zero. The plane strain elements are implemented to model a certain enclosure of the steel plate. In all three analyses, the bolt is modelled with plane stress elements.
6.2.3 Solution procedure

The solution procedure for the nonlinear analysis requires the load to be applied in a series of small incremental loading steps to approach the force-displacement curves closely. The first loading step is explicitly defined to close the 1.0 mm gap between the bolt and the bolt hole so the FEM analysis registers contact between target and contacter elements. The step size of the consecutive loading increments is defined automatically by the finite element program. The solution of this highly non-linear problem is obtained through a number of equilibrium iterations. In this case a regular Newton-Raphson solution algorithm is selected to solve the non-linear equation iterations.

Due to large local deformations in the steel anchor plate around the bolt holes, plasticity is considered by including the Von Mises yield criterion, the Koiter’s plastic flow rule together with isotropic (work) hardening hypothesis. The yield criterion determines the stress level at the onset of yielding while the flow rule relates the stress increments to strain increments during plastic deformation. This allows the yield surface to change in size with progressive yielding in the steel anchor plate along the bolt holes. Geometric non-linear behaviour is invoked by using an updated Lagrange formulation. Convergence difficulties were encountered due to the large plastic region near the contact area.

6.2.4 Results

In total, three numerical analyses are presented for the two dimensional finite element models. Force-displacement relations have been compared against experimental data, as described in paragraph 5.2, and shown in figure 6-6. In this case, the force on the connection is multiplied by a factor 2 because only one half of the steel plate is modelled. The graphs are plotted up to an ovalization of 3.1 mm for comparison against experimental results. An average ovalization of 3.1 mm has been determined for the experiments T02 and T04.

The first numerical analysis of the steel plate using plane stress elements has resulted in a good approximation up to 150 kN, after which significant stiffness degradation is visible. A maximum force of 252.0 kN is determined at a displacement of 2.2 mm which differs 27% with respect to the test data. The force on the connection was taken from the reaction force due to the prescribed displacement.
The second finite element analysis performed by DIANA has been constructed with plane strain elements. Force-displacement relation of this analysis is shown in figure 6-6 which follows the experimental curve more closely than the first analysis. An overestimation of 33% in stiffness has been observed for the first ‘elastic’ part of the curve in relation to the experiments. This is caused by the formulation of plane strain elements and therefore neglecting transverse contraction of the steel material, (E/(1-v). The stiffness of the ‘plastic’ leg of the graph has an underestimation of 16%.

At an ovalization of 3.1 mm the numerical analysis resulted in an underestimation of 9% for the applied
force. The initial (linear) stiffness is determined by linear regression over region of 0 to 0.25 mm ovalization. For the residual (plastic) stiffness, the same approach has been used for a region between 1.35 and 3.10 mm ovalization.

A third analysis has been evaluated with use of a combination between plane stress and plane strain elements. Regular plane stress elements have been applied to the triangular steel plate section with the exception of the area in the immediate vicinity of the bolt hole, where plane strain elements has been used. The effect of enclosure due to the clamping force of the bolt cap and washers is then taken into account. Out-of-plane strain is restricted near the bolt hole. The result of this analysis showed distinct features of the first two analyses where the stiffness of the 'elastic' phase has been overestimated. This was to be expected because the local stress distribution along the bolt hole in this phase is equal to the second analysis with plane strain elements. As the Von Mises yield criterion is exceeded the yield zone extends and stress distribution in the plane stress elements becomes more influential which causes the 'plastic' leg to correspond with the first analysis. Strength and stiffness' of the bolted connection for ovalization of the bolt hole from FEM analyses have been compared with the experimental test data. Results are shown in table 6-2.

<table>
<thead>
<tr>
<th>Strength</th>
<th>Test</th>
<th>FEM elements</th>
<th>Ovalization [mm]</th>
<th>Force [kN]</th>
<th>'Elastic' (0-0.25 mm) [kN/mm]</th>
<th>'Plastic' (1.35-3.10 mm) [kN/mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Plane stress</td>
<td>3.1</td>
<td>246 (65%)</td>
<td>508 (104%)</td>
<td>4 (9%)</td>
</tr>
<tr>
<td></td>
<td>02</td>
<td>Plane strain</td>
<td>3.1</td>
<td>342 (91%)</td>
<td>647 (133%)</td>
<td>33 (84%)</td>
</tr>
<tr>
<td></td>
<td>03</td>
<td>Plane strain/stress</td>
<td>3.1</td>
<td>278 (74%)</td>
<td>643 (133%)</td>
<td>6 (15%)</td>
</tr>
</tbody>
</table>

6.2.5 Conclusion

Three simplified two dimensional finite element analyses of the steel lap joint used for FPC3 are carried out. A significant discrepancy between experimental and finite element results for strength and stiffness prediction for ovalization of the bolt hole was observed.

The model containing plane stress elements gives good results for the initial part of the load-displacement relation up to an ovalization of 0.2 mm where material yielding starts to expand and the eccentricity of the applied force causes bolt rotation. The rotation results in out-of-plane bending and clamping torque between the bolt cap and the steel plate, which increases friction, see figure 6-8. This last effect of bolt rotation and friction is not taken into account in a two dimensional analysis and thus will lead to an overestimation of stiffness degradation of the connection. Additionally, to the finite element analyses, forces are only transferred through the contact area instead of a combination of friction and contact, which leads to higher peak stresses.

The second analysis with plane strain elements resulted in an overestimation of the initial stiffness of the connection, whereas the plastic stiffness is fairly accurate in comparison to the first analysis. A difference of
16% is measured for the plastic stiffness and 9% for the strength, at an ovalization of 3.1 mm, with respect to the experimental results.

Finally, the last analysis which makes use of both plane stress and plane strain elements resulted in a combination of the force-displacement relation of previous analyses. Due to the local stress distribution in the initial phase along the plane strain elements, the connection also shows an overestimation of the stiffness at this phase. As the yield area extends, the plane stress elements cause loss of stiffness, see figure 6-8.

![Figure 6-8](image)

A simplified two dimensional finite element analysis for evaluation of lap joint of the interface connection FPC3 designed for a semi-integral infilled frame is not satisfactory. The three dimensional effects of the connection can not be ignored. Taking into account the effect of the washers, the bolt cap, the clamping torque, the friction between bolts and side-plates should be modelled with a three dimensional numerical model.

### 6.3 Cast-in anchor bars

The second part of the FEM model consists of a reinforced concrete element with cast-in anchor bars. The steel anchor bars are subjected to a tensile force. A two dimensional FEM model of the concrete panel will be discussed. The results from the analysis are discussed and several recommendations for further research are suggested.

#### 6.3.1 The structural behaviour of the interface connection

Steel-to-concrete bond is the many-faceted phenomenon which allows longitudinal forces to be transferred from the reinforcement to the surrounding concrete in a reinforced concrete structure. Due to this force transfer, the force in a reinforcing bar changes along its length, as does the force in the concrete embedment. Wherever steel strains differ from concrete strains, a relative displacement between the steel
and the concrete (slip) does occur, but this lack of compliance is also the effect of highly-localized strains in the concrete layer closest to the reinforcement (interface).

The typical slip, steel and concrete strain distributions are shown in figure 6-9. The free end of the bar acts as a crack initiator across the tension zone and here, (point Z) the concrete strain $\varepsilon_c$ reaches its maximum value. When $\varepsilon_c = \varepsilon_{cr}$, where $\varepsilon_{cr}$ is the concrete cracking strain, a transverse crack appears but since cracking is not controlled, the fracture process spreads abruptly across the whole section.

![figure 6.9 - typical slip and strain distributions for a long member in bar pull-out and concrete in tension](image)

Anchorages which are usually long ($l/\phi > 10-20$) exhibit complex failure modes, can be summarized as followed:

- **Pull-out failure** with no partial concrete splitting (not visible splitting cracks): high confinement and/or large concrete cover; shearing-off of the concrete keys, see (i) at figure 6-10(a).
- **Pull-out failure** induced by partial or through splitting (visible splitting cracks); moderate confinement and/or limited concrete cover; shearing-off of the concrete keys accompanied by concrete slip on rib faces, somewhere between (i) and (ii) at figure 6-10(a).
- **Splitting failure** induced by the spalling-off of the cover: no confinement and/or very limited cover; and bar slip on rib faces, see (ii) at figure 6-10(a).

The force-displacement graph of the experimental tests is converted to an average bond stress-slip relation, figure 6-10(b), which agrees with the schematic pull-out failure of figure 6-10(a).
6.3.2 Mesh and elements

The reinforced concrete panel is modelled with eight-node iso-parametric plane stress elements (CQ16M) with embedded reinforcement bars. By embedding the reinforcement bars into the plane stress elements, the lines of the reinforcements do not have to coincide with the edges of the mesh. Embedded reinforcements do not have degrees of freedom of their own. Its strains are directly computed from the displacement field of the plane stress elements with perfect bond between reinforcement and the surrounding concrete. The finite element mesh of the concrete panel, embedded reinforcements and anchor bars are shown in figure 6-11.

6.3.3 Material properties

Concrete and cracking can be modelled sufficiently accurately as isotropic, linear elastic. The initial Young's modulus of concrete $E_c$ is determined by performing standard material tests on three concrete
prisms. Poisson's ratio of concrete under uni-axial compressive stress has a representative value of 0.19 or 0.20. For this numerical study, a Poisson's ratio of $v_c = 0.2$ used. Standard material compressive tests are performed with concrete 150 mm cubes to determine the compressive strength, $f_c$. A summary of the results from the material tests is provided in table 6-3.

<table>
<thead>
<tr>
<th>Specimen no.</th>
<th>Specimen age [days]</th>
<th>$f_c$ [kN]</th>
<th>$f_{ck,cube}$ [N/mm$^2$]</th>
<th>E-modulus $E_c$ [N/mm$^2$]</th>
</tr>
</thead>
<tbody>
<tr>
<td>J.D.1011K01</td>
<td>28</td>
<td>1306.1</td>
<td>58.05</td>
<td>J.D.1011P01 28429</td>
</tr>
<tr>
<td>...K02</td>
<td></td>
<td>1270.8</td>
<td>56.48</td>
<td></td>
</tr>
<tr>
<td>...K03</td>
<td></td>
<td>1296.3</td>
<td>57.61</td>
<td></td>
</tr>
<tr>
<td>...K04</td>
<td>30</td>
<td>1309.7</td>
<td>58.21</td>
<td>J.D.1011P02 27602</td>
</tr>
<tr>
<td>...K05</td>
<td></td>
<td>1287.9</td>
<td>57.24</td>
<td></td>
</tr>
<tr>
<td>...K06</td>
<td></td>
<td>1290.0</td>
<td>57.33</td>
<td></td>
</tr>
<tr>
<td>...K07</td>
<td>34</td>
<td>1344.4</td>
<td>59.75</td>
<td>J.D.1011P03 30828</td>
</tr>
<tr>
<td>...K08</td>
<td></td>
<td>1337.1</td>
<td>59.43</td>
<td></td>
</tr>
<tr>
<td>...K09</td>
<td></td>
<td>1345.9</td>
<td>59.82</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>58.21 28953</td>
</tr>
</tbody>
</table>

The non-linear concrete model used by Teeuwen$^{[24]}$ is applied for the concrete panel of the finite element model. It combines the Drucker-Prager plasticity model for the compressive regime with a smeared cracking model for the tensile regime. For the behaviour of the concrete in compression the Drucker-Prager yield surface limits the elastic state of stress. The TNO DIANA software evaluates the yield surface using the current state of stress, the angle of internal friction $\phi$ and the cohesion $c$. The DIANA software manual$^{[20]}$ recommends that the angle of internal friction of the concrete is approximated to be $\phi = 10^\circ$. The cohesion $c$ can then be calculated as follows:

$$c = f_c \left(1 - \sin \phi \right) \frac{1}{2 \cdot \cos \phi}$$

![Failure surface in two dimensional principal stress space](image)
A multi-directional fixed crack model is applied as a smeared crack approach, in which the direction of the normal to the crack is fixed upon initiation of the crack. A linear stress cut-off criterion is applied, see figure 6-12, which means that a crack arises if the major principal tensile stress exceeds the minimum of $f_{ct}$ and $f_{ct}(1+\sigma_{lateral}/f_d)$, with $\sigma_{lateral}$ being the lateral principal stress and $f_{ct}$ as the tensile strength. In addition, a linear tension softening based on the fracture energy of concrete $G_f$ is adopted according to the CEB-FIP Model Code[23]. The fracture energy is related to the compressive strength and the maximum aggregate size, and has been estimated from:

$$G_f = G_{fr} \left( \frac{f_{cm}}{f_{cm,0}} \right)^{0.7}$$

With:

$$f_{cm} = f_{cm} + \Delta f$$

Where:

- $G_{fr}$ = Base value of fracture energy depending on the maximum aggregate size $d_{max}$.
- For this study, $d_{max} = 16$ mm, resulting in $G_{fr} = 0.030$ Nmm/mm$^2$.
- $f_{cm}$ = Mean value of compressive strength [N/mm$^2$].
- $f_{cm,0} = 10$ N/mm$^2$.
- $f_{ck}$ = Characteristic compressive strength [N/mm$^2$].
- $\Delta f = 8$ N/mm$^2$.

As a result the applied value for the fracture energy of concrete is 0.092 Nmm/mm$^2$. Due to cracking, shear retention occurs which causes a reduction of the shear stiffness. A constant shear retention factor $\beta = 0.2$ is used which is a commonly applied value[25].

For the embedded reinforcement bars, a Young’s modulus of $E_s = 2.0E+05$ N/mm$^2$ is taken. The stress-strain curve of the reinforcement bars is assumed to be elastic-perfectly plastic, with yielding according to the Von Mises criterion, with a yield stress equal to $f_y = 435$ N/mm$^2$.

### 6.3.4 Boundary conditions and loading

Several boundary condition are applied to the concrete finite element model, see figure 6-13. The displacements in y-direction along horizontal planes at both ends of the model are restricted. The model is also fixed in space by applying a displacement restriction at one point for x-direction.

A multi-point-constraint, MPC, is used for the low-end nodes of the anchor bars where the external force is applied. The node defined as the end of the middle anchor bar is defined as master node and the nodes of the four remaining anchor bars are defined as slave nodes.

Finally, the tension load is applied as a displacement to the master node. The MPC results in an equal displacement of the anchor bars for describing the force-displacement relations.
6.3.5 Solution procedure

To perform the analysis, a displacement controlled procedure is applied to impose the load up to failure. A regular Newton-Raphson iteration method is used to find the solution for each load-step.

6.4 Results

Output of the finite element analysis is compared to the experimental test data. The predicted stiffness and strength are plotted together with the force-displacement graphs of the test results as described in paragraphs 4.4 to 4.7. A comparison for the plotted force-displacement curves is shown in figure 6-14.

The pre-numerical analysis performed on the concrete panel with cast-in anchor bars to predict its structural behaviour under a tension load, resulted in acceptable results for the 'elastic' stiffness and good results for its strength. For the initial phase until the first cracks appear, the connection stiffness is well
predicted with a slight overestimation. At a force where stiffness degradation due to concrete cracking occurs is also reasonably predicted. From that point onwards, slip of the anchor bars becomes more influential resulting in excessively stiff behaviour of the FEM model compared to the experimental test results. Finally, the ultimate load on the connection and the crack patterns due to the tension load are also predicted with reasonable accuracy. A comparison of the crack patterns from finite element analysis and a common crack pattern from test T02 is given in figure 6-15.

6.5 Conclusion

A smeared crack approach used in FEM modelling for predicting the structural behaviour of the concrete panel resulted in reasonable results for stiffness and strength and the predicting crack patterns. Implementing interface elements to describe bond-slip effects should probably gain better results for describing the post-cracking stiffness of the connection.
7 Conclusions and discussion

The overall conclusions and discussions are presented in this chapter. A semi-integral infilled frame has shown to be effective and efficient in bracing building structures to resist lateral loads. The composite interaction between the steel frame and precast concrete infill panel determines the strength and stiffness of the structure. The main goal of this research is to analyse the shortcomings of the already designed connections and develop a new interface connection with improved structural properties regarding strength and stiffness. In addition, simplified two dimensional finite element models have been developed for further analyses.

7.1 Conclusions

7.1.1 Analyses of FPC1 and FPC2

First of all, it is noticed that the position of the connections, FPC1 and FPC2, does not lead to an optimal interaction between the concrete infill panel and the surrounding steel frame and therefore the potential lateral stiffness and strength of the infilled frame is not fully exploited. The configurations of FPC1 and FPC2 resulted in a concrete failure for the tension test and the full-scale test (only FPC2). The tension force caused splitting of the concrete panel which led to anchor pull-out. The anchorage length of the anchor bars, bond strength and the concrete edge distance were too small. In addition, the cast-in steel plate was also an initiator of the concrete splitting due to the deformations caused by a lateral force.

7.1.2 Experimental research, FPC3

Four experimental tension tests have been carried out on FPC3 which resulted in a combined failure mode of anchor pull-out, loss of bond, and ovalization of the bolt hole. The ultimate load on the connection varies between 340 and 376 kN with an average 361 kN. The calculations, based on the Eurocode predicted a failure load of 394 kN, 9% overestimation, with ovalization of the bolt holes as failure mode. The anchorage length based on Eurocode 3 and additional research by Wang\textsuperscript{[13]} was not sufficient to transfer the tension force and prevent pull-out failure.

7.1.3 Numerical research

A simplified two dimensional finite element analysis, as described in paragraph 6.2, for evaluation of the lap joint of the interface connection FPC3 yielded only reasonable results in comparison to experimental results. Three dimensional effects such as friction between the steel plates, out-of-plane bending and clamping forces due to the bolt cap on the connection could not be included in the analysis. Acceptable results are only obtained for the initial elastic phase where three dimensional effects are minimal. The pre-numerical analysis that has been performed on the concrete panel with cast-in anchor bars to predict its structural behaviour under a tension load has resulted in acceptable results. Using a smeared
crack approach, a multi-directional fixed crack model with a linear stress cut-off criterion, for the concrete panel resulted in reasonable results for the stiffness and good results for its strength. For the initial phase until the first cracks appear, stiffness is well predicted. The force where stiffness degradation due to cracking occurs is also well predicted. From that point, slip of the anchor bars becomes more influential resulting in too stiff behaviour of the FEM model. The ultimate load and the crack patterns are also predicted with reasonable accurate.

7.2 Discussion

Based on the evaluation of the results from this research there are some topics that need more attention for further analyses. They are subdivided into experimental research and numerical research.

7.2.1 Experimental research

The connection FPC3 has not failed as expected based on the calculations. Configuration of the anchor bars, confinement of wall reinforcements should be taken into account for better bonding properties so ovalization of the bolt holes can fully develop as failure mode. On the other hand, brittle failure did not occur as plastic deformations were noticed, making the connection a favourable design for structural applications. However, additional research is required to optimize the connection and to describe the structural behaviour by means of design rules. Another point of interest is the contribution of the connection along the compression diagonal. Whether an unsupported compression diagonal will fail due to out-of-plane buckling before the tension diagonal fails is not evident. A full-scale test on an infilled frame or a compression test is necessary to be able to make any statements.

7.2.2 Numerical research

Taking into account the effects of the washers, the bolt cap, the clamping torque, the friction between bolts and side-plates requires a three dimensional numerical model. A smeared crack approach used in FEM modelling for predicting the structural behaviour of the concrete panel resulted in reasonable results for stiffness and strength and the predicting crack patterns. Implementing interface elements to describe bond-slip effects should probably gain better results for describing the post-cracking stiffness of the connection.
8 References


[22] Lundgren, K., Three-Dimensional Modelling of Bond in Reinforced Concrete, (1999), Chalmers University of Technology, Göteborg.


Appendices

*Interface connection of a semi-integral infilled frame*
Appendix A - Material tests

Material tests are performed to obtain mechanical properties of the steel plate and the concrete panel. These values are used for FEM-analyses to compare results with the experimental tests in order to fine tune the model for further analyses. Conclusions and recommendations are made for the design of the interface connection of a semi-integral infilled frame.

**Tensile strength steel plate**

The yield strength and the tensile strength are determined using a tensile test in order to confirm the definite dimension of the structural components of the connection. The tensile tests are performed in advance because of the sensitivity of the bearing force for ovalization of the bolt holes. The results can be influential on the ultimate design. The tensile tests are performed according NEN-EN 10002-1 in the Pieter van Musschenbroek laboratory.

The thickness of the steel plate is 10 mm as specified in paragraph 3.4. The width of the specimen shall not exceed 80 mm. The gauge length \( L_0 \) of the test specimen is specified by:

\[ L_0 = k \cdot \sqrt{S_0} \]

with \( k = 5.65 \). Because of the limited size of the specimen, a gauge length of 80 mm is used with an original cross section of \( S_0 \) is 200 mm². As a result, the width of the parallel length \( b \) is 20 mm. Dimensions are shown in table A.1.

<table>
<thead>
<tr>
<th>Testno.</th>
<th>pos.</th>
<th>( t ) [mm]</th>
<th>( b ) [mm]</th>
<th>( L_t ) [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>A</td>
<td>9,74</td>
<td>19,62</td>
<td>232</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>9,74</td>
<td>19,63</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>9,76</td>
<td>19,63</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>A</td>
<td>9,75</td>
<td>20,08</td>
<td>231</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>9,76</td>
<td>20,08</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>9,77</td>
<td>20,08</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>A</td>
<td>9,75</td>
<td>20,06</td>
<td>231</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>9,76</td>
<td>20,06</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>9,78</td>
<td>20,06</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>A</td>
<td>9,79</td>
<td>20,01</td>
<td>231</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>9,77</td>
<td>20,01</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>9,77</td>
<td>20,00</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>A</td>
<td>9,78</td>
<td>20,00</td>
<td>231</td>
</tr>
<tr>
<td></td>
<td>B</td>
<td>9,77</td>
<td>20,01</td>
<td></td>
</tr>
<tr>
<td></td>
<td>C</td>
<td>9,78</td>
<td>20,01</td>
<td></td>
</tr>
</tbody>
</table>

The test set-up according to NEN-EN 1002-1 is used to determine the yield and tensile strength. The force applied on the specimen and the extension of \( L_0 \) is measured to obtain these values. A testing speed of
0.00023 s⁻¹ (1.1 mm/min.) is used for the elastic range of the steel specimen. The upper and lower yield strength can be determined. During the plastic phase a test speed of 0.00065 s⁻¹ (3.1 mm/min.) is set until the ultimate tensile strength is reached. Force-displacement graphs of the tests are shown in figure A.3.
figure A.3 - Force-displacement graphs steel tensile tests
Concrete compressive strength

Compressive strength tests were performed on nine identical specimens on three different days. These specimens [J.D.1011K01 to ...K09] all had the same cubical dimensions, 150 x 150 x 150 mm, see table A.2. The tests were performed at the Pieter van Musschenbroek laboratory at the University of Technology Eindhoven.

<table>
<thead>
<tr>
<th>Table A.2 - Concrete compressive test parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>Concrete quality</td>
</tr>
<tr>
<td>Size specimen [mm]</td>
</tr>
<tr>
<td>A₁ [mm²]</td>
</tr>
<tr>
<td>Testing speed [kN/sec.]</td>
</tr>
</tbody>
</table>

Figure A.5 - Concrete compression test on J.D.1011K02

Table A.3 shows the results of the compressive tests on the cubes.

J.A.H. Dekkers
Concrete prism test

Together with the compressive strength tests there were also three prism tests performed on December 8\(^{th}\). These tests were performed with a 2.5 MN pressure bench in order to determine the E-modulus of the concrete. The test parameters are listed in table A.4. The test results of the compressive tests on the prisms are shown in table A.5.

### table A.3 - Concrete compressive test results

<table>
<thead>
<tr>
<th>Specimen no.</th>
<th>Production date [dd-mm-yyyy]</th>
<th>Test date [dd-mm-yyyy]</th>
<th>Specimen age [days]</th>
<th>$F_0$ [kN]</th>
<th>$f_{ck}$ [N/mm(^2)]</th>
</tr>
</thead>
<tbody>
<tr>
<td>J.D.1011K01</td>
<td>10-11-2009</td>
<td>8-12-2009</td>
<td>28</td>
<td>1306.1</td>
<td>58.05</td>
</tr>
<tr>
<td>...K02</td>
<td></td>
<td></td>
<td></td>
<td>1270.8</td>
<td>56.48</td>
</tr>
<tr>
<td>...K03</td>
<td></td>
<td></td>
<td></td>
<td>1296.3</td>
<td>57.61</td>
</tr>
<tr>
<td>...K04</td>
<td>10-11-2009</td>
<td>10-12-2009</td>
<td>30</td>
<td>1309.7</td>
<td>58.21</td>
</tr>
<tr>
<td>...K05</td>
<td></td>
<td></td>
<td></td>
<td>1287.9</td>
<td>57.24</td>
</tr>
<tr>
<td>...K06</td>
<td></td>
<td></td>
<td></td>
<td>1290.0</td>
<td>57.33</td>
</tr>
<tr>
<td>...K07</td>
<td>10-11-2009</td>
<td>14-12-2009</td>
<td>34</td>
<td>1344.4</td>
<td>59.75</td>
</tr>
<tr>
<td>...K08</td>
<td></td>
<td></td>
<td></td>
<td>1337.1</td>
<td>59.43</td>
</tr>
<tr>
<td>...K09</td>
<td></td>
<td></td>
<td></td>
<td>1345.9</td>
<td>59.82</td>
</tr>
</tbody>
</table>

### table A.4 - Concrete prism test parameters

<table>
<thead>
<tr>
<th>Concrete quality</th>
<th>C35/45</th>
</tr>
</thead>
<tbody>
<tr>
<td>Size specimen [mm]</td>
<td>500x100x100</td>
</tr>
<tr>
<td>Load [kN]</td>
<td>230 kN (40% of $f_{ck}$)</td>
</tr>
<tr>
<td>Testing speed [kN/sec.]</td>
<td>200 mm / 20h</td>
</tr>
<tr>
<td>Production date [dd-mm-yyyy]</td>
<td>10-11-2009</td>
</tr>
<tr>
<td>Test date [dd-mm-yyyy]</td>
<td>08-12-2009</td>
</tr>
</tbody>
</table>

![Concrete prism test (a) and quality of concrete prism (b)](image-url)
### Appendix A.5 - Concrete prism test results

<table>
<thead>
<tr>
<th>Specimen no.</th>
<th>Area [mm]</th>
<th>Young's modulus [N/mm²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>J.D.1011P01</td>
<td>Top 100x100, Bottom 105x100</td>
<td>28429</td>
</tr>
<tr>
<td>J.D.1011P02</td>
<td>Top 105x100, Bottom 97x100</td>
<td>27602</td>
</tr>
<tr>
<td>J.D.1011P03</td>
<td>Top 100x100, Bottom 105x100</td>
<td>30828</td>
</tr>
</tbody>
</table>

*Note: Quality of the specimen J.D.1011P01 is bad*

*Note: Quality of the specimen J.D.1011P02 is bad*

*Note: Quality of the specimen J.D.1011P01 is slightly better. Specimen has preliminary cracks around the edges*

---

**Figure A.7 - Stress-strain graphs concrete prism tests to determine secant Young’s modulus**
Appendix B - Images tests

Test specimen
Test set-up and test JD1011T01
Test JD1011T03
Test JD1011T02
Test JD1011T04
## Appendix C - Notes during tests

**JD1011T01**

<table>
<thead>
<tr>
<th>Test no.</th>
<th>JD1011T01</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test date:</td>
<td>8-12-2009</td>
</tr>
<tr>
<td>Test speed:</td>
<td>1.0 / 2.0 mm/min</td>
</tr>
<tr>
<td>$f_{d,iso}^{ext}$</td>
<td>57.4 N/mm²</td>
</tr>
<tr>
<td>$E$</td>
<td>Not representative</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test set-up:</th>
<th>front</th>
<th>back</th>
</tr>
</thead>
</table>

- Measure configuration front
- Measure configuration back

### Crack patterns during test:

#### front
- 1) ±230 kN
- 2) ±280 kN
- 3) ±320 kN

#### back
- 1) ±230 kN
- 2) ±280 kN
- 3) ±320 kN
- 4) ±340 kN

### Comments:
- The testing speed was set to 1.0 mm/min instead of decelerating the speed to 0.25 mm/min at a force of 110 kN.
- At a force of 150 kN it was noticed that strain gauge 08 had a technical malfunction.
- It was noticed that strain gauge 08 did work again at a force of 320 kN.
- The maximum force was reached at about 372.1 kN. A gap between the concrete element and the steel plate was visible.
- The ovalization of the bolt holes was measured after disassembling the test set-up. The measured values are regarding the front view: 29.1 mm for the left bolt hole and 29.2 mm for the right bolt hole.

---

J.A.H. Dekkers
JD1011T03

Test no.: JD1011T03
Test date: 10-12-2009
Test speed: 1.0 / 0.5 mm/min

Test set-up: front back

Measure configuration front

Measure configuration back

Crack patterns during test: front back

1) 210 kN
2) 230 kN
3) 240 kN
4) 250 kN
5) 260 kN
6) 270 kN
7) 340 kN

Crack development front

Crack development back

Comments:
- In order to measure 'ovalization', the measure configuration has changed according to Fout! Verwijzingsbron niet gevonden. and Fout! Verwijzingsbron niet gevonden.
- The testing speed was set to 0.25 mm/min at a force of 100 kN.
- At a force of 230 kN a force drop down was noticed with some loud cracking.
- A crack occurred through the strain gauge (15-16-17) at 240 kN.
- A force drop at 260 kN with crack development.
- The maximum force was reached at about 355.4 kN. A gap between the concrete element and the steel plate was visible.
- The ovalization of the bolt holes was measured after disassembling the test set-up. The measured values are regarding the front view, 30.0 mm for the left bolt hole and 27.5 mm for the right bolt hole.

J.A.H. Dekkers
**JD1011T02**

<table>
<thead>
<tr>
<th>Test no.:</th>
<th>JD1011T02</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test date:</td>
<td>14-12-2009</td>
</tr>
<tr>
<td>Test speed:</td>
<td>1.0 / 0.5 mm/min</td>
</tr>
<tr>
<td>$f_{\text{nom}}$</td>
<td>59.7 N/mm$^2$</td>
</tr>
<tr>
<td>E=</td>
<td>Not representative</td>
</tr>
</tbody>
</table>

**Test set-up:**

<table>
<thead>
<tr>
<th>front</th>
<th>back</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image1.png" alt="Measure configuration front" /></td>
<td><img src="image2.png" alt="Measure configuration back" /></td>
</tr>
</tbody>
</table>

**Crack patterns during test:**

<table>
<thead>
<tr>
<th>front</th>
<th>back</th>
</tr>
</thead>
<tbody>
<tr>
<td><img src="image3.png" alt="Crack development front" /></td>
<td><img src="image4.png" alt="Crack development back" /></td>
</tr>
</tbody>
</table>

**Comments:**

- Two strain gauges had to be moved from its original position because of the presence of a concrete distance block. This is shown in **Fout! Verwijzingsbron niet gevonden.**
- The testing speed was set to 0.25 mm/min at a force of 130 kN.
- Force drop at 243 kN with several cracks.
- The maximum force was reached at about 376.6 kN. A gap between the concrete element and the steel plate was visible.
- The ovalization of the bolt holes was measured after disassembling the test set-up. The measured values are regarding the front view, 29.0 mm for the left bolt hole and 29.0 mm for the right bolt hole.

---

J.A.H. Dekkers
**JD1011T04**

<table>
<thead>
<tr>
<th>Test no.</th>
<th>JD1011T04</th>
<th>$f'_{d,net}$</th>
<th>59.7 N/mm²</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test date:</td>
<td>14-12-2009</td>
<td>$E_u$</td>
<td>Not representative</td>
</tr>
<tr>
<td>Test speed:</td>
<td>1.0 / 0.5 mm/min</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Test set-up:</th>
<th>front</th>
<th>back</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td><img src="image1.jpg" alt="Measure configuration front" /></td>
<td><img src="image2.jpg" alt="Measure configuration back" /></td>
</tr>
<tr>
<td>Crack patterns during test:</td>
<td>front</td>
<td>back</td>
</tr>
<tr>
<td><img src="image3.jpg" alt="Measure configuration front" /></td>
<td><img src="image4.jpg" alt="Measure configuration back" /></td>
<td></td>
</tr>
</tbody>
</table>

**Comments:**
- The testing speed was set to 0.25 mm/min at a force of 140 kN.
- At a force of 150 kN it was noticed that strain gauge 08 had a technical malfunction.
- A small drop in force was noticed at a force of 240 kN.
- Again a small drop at 255 kN.
- The maximum force was reached at about 340.1 kN. A gap between the concrete element and the steel plate was visible. Loud knocks inside the concrete panel were heard.
- The ovalization of the bolt holes was measured after disassembling the test set-up. The measured values are regarding the front view, 28.7 mm for the left bolt hole and 28.4 mm for the right bolt hole.

J.A.H. Dekkers
Appendix D - Graphs experimental tests

Measurements steel plate

Slip of anchor bars

Displacement bolts

J.A.H. Dekkers
Measurements steel plate

Slip of anchor bars

Displacement bolts
Measurements steel plate

<table>
<thead>
<tr>
<th>Force (kN)</th>
<th>Displacement [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>50</td>
<td>1</td>
</tr>
<tr>
<td>100</td>
<td>2</td>
</tr>
<tr>
<td>150</td>
<td>3</td>
</tr>
<tr>
<td>200</td>
<td>4</td>
</tr>
<tr>
<td>250</td>
<td>5</td>
</tr>
<tr>
<td>300</td>
<td>6</td>
</tr>
<tr>
<td>350</td>
<td>7</td>
</tr>
<tr>
<td>400</td>
<td>8</td>
</tr>
</tbody>
</table>

Slip of anchor bars

<table>
<thead>
<tr>
<th>Force (kN)</th>
<th>Displacement [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
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</tr>
<tr>
<td>50</td>
<td>0</td>
</tr>
<tr>
<td>100</td>
<td>0.5</td>
</tr>
<tr>
<td>150</td>
<td>1</td>
</tr>
<tr>
<td>200</td>
<td>1.5</td>
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<tr>
<td>250</td>
<td>2</td>
</tr>
<tr>
<td>300</td>
<td>2.5</td>
</tr>
<tr>
<td>350</td>
<td>3</td>
</tr>
<tr>
<td>400</td>
<td>3.5</td>
</tr>
</tbody>
</table>

Mitutoyo's (overall displacement)

<table>
<thead>
<tr>
<th>Force (kN)</th>
<th>Displacement [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>-1</td>
</tr>
<tr>
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<td>0</td>
</tr>
<tr>
<td>100</td>
<td>1</td>
</tr>
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<td>150</td>
<td>2</td>
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<tr>
<td>200</td>
<td>3</td>
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<tr>
<td>250</td>
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<td>300</td>
<td>5</td>
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<tr>
<td>350</td>
<td>6</td>
</tr>
<tr>
<td>400</td>
<td>7</td>
</tr>
</tbody>
</table>

J.A.H. Dekkers
Displacement bolts

Measurements steel plate

Slip of anchor bars

Appendices

J.A.H. Dekkers
MSc-report - Interface connection of a semi-integral infilled frame

Appendices

J.A.H. Dekkers
Appendix E - Input finite element models

Steel plate - lap joint - 2D - plane stress

part.fgc

Geometry upper right block (hole)

Geometry upper right block

Geometry copy/rotate block

Geometry entire plate

Geometry surface

Meshing

Materials

J.A.H. Dekkers
**Loading**

PROPERTY LOADS DISPLACEMENT P5 -5 Y

**Constraints**

PROPERTY BOUNDARY CONSTRAINT L38 Y
PROPERTY BOUNDARY CONSTRAINT L40 Y
PROPERTY BOUNDARY CONSTRAINT L42 Y
PROPERTY BOUNDARY CONSTRAINT L44 Y
PROPERTY BOUNDARY CONSTRAINT L46 Y
PROPERTY BOUNDARY CONSTRAINT L49 X
PROPERTY BOUNDARY CONSTRAINT L51 X
PROPERTY BOUNDARY CONSTRAINT L53 X
PROPERTY BOUNDARY CONSTRAINT L55 X
PROPERTY BOUNDARY CONSTRAINT L57 X
PROPERTY BOUNDARY CONSTRAINT HOLE X

---

**Nonlin_Partial.com**

* FILOS
INITIA
* INPUT
* NONLIN
BEGIN TYPE
PHYSIC PLASTI
END TYPE
BEGIN OUTPUT FEMVIEW BINARY FILE="partial" 
DISPLA TOTAL
STRESS TOTAL TRACTI LOCAL INTPNT
STRESS TOTAL CAUCHY
STRESS TOTAL CAUCHY GLOBAL INTPNT
STRESS TOTAL CAUCHY LOCAL INTPNT
STRESS TOTAL CAUCHY VONMIS
STRAIN TOTAL
STRAIN TOTAL GREEN PRINCI INTPNT
STRAIN ELASTI GLOBAL
STRAIN PLASTI GLOBAL INTPNT
STATUS CONTAC
STATUS PLASTI INTPNT
FORCE REACTI
END OUTPUT
BEGIN EXECUT
BEGIN LOAD
STEPS EXPLIC SIZE 0.2
LOADNR=1
END LOAD
BEGIN ITERAT
METHOD NEWTON REGULA
MAXITE=10
BEGIN CONVER
ENERGY
FORCE CONTIN TOLCON=0.0001
DISPLA OFF
END CONVER
END ITERAT
END EXECUT
BEGIN LOAD
BEGIN STEPS
BEGIN AUTOMA
SIZE=0.2
MAXSIZ=0.01
END AUTOMA
END STEPS
LOADNR=1
END LOAD
BEGIN ITERAT
METHOD NEWTON REGULA
MAXITE=50
BEGIN CONVER
ENERGY
FORCE CONTIN TOLCON=0.0001
DISPLA OFF
END CONVER
END ITERAT
END EXECUT
BEGIN EXECUT
BEGIN LOAD
BEGIN STEPS
BEGIN AUTOMA
SIZE=0.6
MAXSIZ=0.005
END AUTOMA
END STEPS
LOADNR=1
END LOAD
BEGIN ITERAT
METHOD NEWTON REGULA
MAXITE=50
BEGIN CONVER
ENERGY
FORCE CONTIN TOLCON=0.0001
DISPLA OFF
END CONVER
END ITERAT
END EXECUT
END

---

**hardia.dat**

HARDIA 265. 0. 275. 0.02 362. 0.045 436. 0.149 2.0.18 517 .. 24

---

J.A.H. Dekkers
Steel plate - lap joint - 2D - plane strain

part.fgc

I  Initiation
femgen PARTIAL
property fe-prog diana struct_2d
yes
ut se uni length millimeter
ut se uni mass kilogram
ut se uni force newton
ut se uni time second
ut se uni temper celsius

I  Geometry upper right block(bolt)
GE PO 0 0
GE PO -12 0
GE PO 0 12
GE PO 12 0
GE LI CI P2 P3 P4
CO SE BOLT AP LI ALL
CO SE OP HOLE
GE LI CI P1 2
CO SE CI HOLE
GE SU RE BOLT HOLE QU8

I  Geometry upper right block
GE PO 13 0
GE PO 38.89 0
GE PO 38.89 38.89
GE PO 0 38.89
GE PO 0 13
GE LI ARC P14 P10 P1
GE LI ST P10 P11
GE LI ST P11 P12
GE LI ST P12 P13
GE LI ST P13 P14
CO SE PLATE AP LI L19 L10 L11 L12 L13
GE SU RE PLATE QU8

I  Geometry copy/rotate block
GE CO S2 RO P1 P10 P14
GE CO S3 RO P1 P14 P15
GE CO S4 RO P1 P16 P19
CO SE EDGE AP LI L9 L14 L18 L22

I  Geometry entire plate
GE PO -38.89 -77.78
GE PO -38.89 -116.67
GE PO 0 -77.78
GE PO 77.78 0
GE PO 77.78 38.89
GE PO 116.67 38.89
GE PO 152.03 74.25
GE PO 116.67 74.25
GE PO 77.78 74.25
GE PO 38.89 74.25
GE PO 0 74.25
GE PO -38.89 74.25
GE PO -42.43 74.25
GE PO -42.43 38.89
GE PO -42.43 -38.89
GE PO -42.43 -77.78
GE PO -42.43 -116.67

I  Geometry surface
GE SU 4POINTS P19 P22 P24 P20 QU8
GE SU 3POINTS P22 P23 P24 QU8
GE SU 3POINTS P20 P24 P21 QU8
GE SU 3POINTS P21 P25 P11 QU8
GE SU 4POINTS P11 P25 P26 P12 QU8
GE SU 3POINTS P26 P25 P27 QU8
GE SU 3POINTS P27 P28 P29 QU8
GE SU 4POINTS P29 P30 P26 P27 QU8
GE SU 4POINTS P30 P31 P12 P26 QU8
GE SU 4POINTS P31 P32 P13 P12 QU8
GE SU 4POINTS P32 P33 P16 P13 QU8
GE SU 4POINTS P33 P34 P35 P16 QU8
GE SU 4POINTS P16 P35 P36 P17 QU8
GE SU 4POINTS P17 P36 P37 P19 QU8
GE SU 4POINTS P19 P37 P38 P22 QU8
GE SU 4POINTS P22 P38 P39 P23 QU8

I  Meshing
ME TY BOLT CL6CT
ME TY EDGE CL6CT
ME DI LI ALL 12
ME DI LI L10 8
ME DI LI L13 8
ME DI LI L17 8
ME DI LI L21 8
ME DI LI L28 16
ME DI LI L29 16
ME DI LI L30 16
ME DI LI L31 16
ME DI LI L35 16
ME DI LI L37 16
ME DI LI L48 2
ME DI LI L50 2
ME DI LI L52 2
ME DI LI L54 2
ME DI LI L56 2
ME DI LI L58 2
ME DI SU S1 28
ME OP AL PA ALL
MESHING GENERATE

I  Materials
PR MA MA1 EL IS Z100000 0.3
PR MA MA2 EL IS Z1000000 0.3
PR MA MA1 STATNONL METALS VMISES WHARDIA "hardia.dat"
PR PH PH1 GEOMETRY PLANSTRS THREGULR 10
PR PH PH2 GEOMETRY PLANSTRS THREGULR 20
PR MA CONT STATNONL CONTACT CONTAC
PR MA TARG STATNONL CONTACT TARGET 1.00 0.001 0.0000001
0.0 0.0
PR AT S1 MA2 PH2
PR AT S2 MA1 PH1
PR AT S3 MA1 PH1
PR AT S4 MA1 PH1
PR AT S5 MA1 PH1
PR AT S6 MA1 PH1
PR AT S7 MA1 PH1
PR AT S8 MA1 PH1
PR AT S9 MA1 PH1

J.A.H. Dekkers
### Appendices

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J.A.H. Dekkers
Concrete element - 2D - plane stress

pu.fgc

I INITIATIVE
femgen PU
property fe-prog diana struct_2d
yes
ut se uni length millimeter
ut se uni mass kilogram
ut se uni force newton
ut se uni time second
ut se uni temper celsius

I Keypoints
GE PO -42.43 74.25
GE PO -42.43 494.25
GE PO 22.57 74.25
GE PO 22.57 494.25
GE PO 87.57 74.25
GE PO 87.57 494.25
GE PO 152.03 74.25
GE PO 152.03 494.25
GE PO 362.03 284.25
GE PO 572.03 494.25
GE PO 807.57 494.25
GE PO 807.57 981.19
GE PO 572.03 981.19
GE PO 152.03 981.19
GE PO 87.57 981.19
GE PO 22.57 981.19
GE PO -42.43 981.19
GE PO -107.43 981.19
GE PO -236.88 981.19
GE PO -656.88 981.19
GE PO -892.43 981.19
GE PO -426.88 981.25
GE PO -236.88 944.25
GE PO -236.88 494.25
GE PO -172.43 74.25
GE PO -172.43 981.19
GE PO -107.43 74.25
GE PO -107.43 494.25

I Concrete elementS
GE SU 4POINTS P1 P2 P3 P4 QU8
GE SU 4POINTS P3 P4 P5 P6 QU8
GE SU 4POINTS P5 P6 P7 P8 QU8
GE SU 3POINTS P7 P8 P9 QU8
GE SU 3POINTS P8 P9 P10 QU8
GE SU 4POINTS P10 P13 P12 P11 QU8
GE SU 4POINTS P14 P13 P10 QU8
GE SU 4POINTS P13 P14 P8 QU8
GE SU 4POINTS P14 P16 P15 P6 QU8
GE SU 4POINTS P17 P16 P4 QU8
GE SU 4POINTS P21 P18 P17 P2 QU8
GE SU 4POINTS P23 P19 P18 P31 QU8
GE SU 4POINTS P23 P20 P19 P29 QU8
GE SU 4POINTS P24 P21 P20 P27 QU8
GE SU 4POINTS P22 P21 P20 P27 QU8
GE SU 3POINTS P25 P24 P27 QU8
GE SU 3POINTS P26 P25 P27 QU8
GE SU 4POINTS P26 P27 P29 P28 QU8

GE SU 4POINTS P28 P29 P31 P30 QU8
GE SU 4POINTS P30 P31 P2 P1 QU8

CO SE OP CONC
CO SE AP ALL
CO SE CL CONC

I Mesh division concrete block1
ME TY ALL QU8 CQ16M
ME DI LI AL 4
ME DI PR L45 36
ME DI PR L39 36
ME OP AL PA S4
ME OP AL PA 55
ME DI LI L11 20
ME DI LI L12 16
ME DI PR L13 28
ME DI LI L14 20
ME DI PR L18 12
ME DI PR L40 12
ME DI PR L38 28
ME DI LI L42 20
ME DI LI L43 20
ME DI LI L44 16
ME OP AL PA 516
ME OP AL PA 517
ME GE

I Materials
PR MA M_C EL IS 29000 0.15
PR MA M_C STATNONL CONCBRIT CRACK LINEAR
FRACTURE TAU2RI1 DRUCKE NONE 3.62 49.9 0.092 0.2 38.297
0.1736 0.1736
PR MA M_R ELASTIC REINFORC BOND 2.0E5
PR MA M_R STATNONL REINFORC VMISES NONE 435
PR PH P_C GE PLANSTRS THREGU1R 175
PR PH P_RBA GE RE BAR 201
PR PH P_RB1 GE RE BAR 628
PR PH P_RB2 GE RE BAR 157
PR PH P_RB3 GE RE BAR 101
PR PH P_RG GE RE BAR 101
PR AT CONC M_C P_C

I Reinforcement
RE SE OP ANCHOR
REI BAR SE RE1 P28 P29
REI BAR SE RE2 P30 P31
REI BAR SE RE3 P1 P2
REI BAR SE RE4 P3 P4
REI BAR SE RE5 P5 P6
REI BAR BAR1 RE1
REI BAR BAR2 RE2
REI BAR BAR3 RE3
REI BAR BAR4 RE4
REI BAR BAR5 RE5
RE SE CL ANCHOR
C20 ANCHOR
PR AT ANCHOR M_R P_RBA

J.A.H. Dekkers
| 106.32 99.25 | 106.43 775.50 | -35.10 99.25 | 782.57 916.92 | -175.53 99.25 | -342.95 215.87 | -413.53 286.38 | 250.43 950.46 | -484.37 357.09 | -555.08 427.80 | -32.41 99.25 | -325.79 498.51 | -173.84 950.46 | -925.86 550.46 | -759.55 950.46 | -191.17 99.25 | -867.43 775.50 | -49.75 99.25 | -867.43 916.92 | 91.67 99.25 | -759.55 950.46 | 187.38 144.96 | -618.12 950.46 | -258.09 215.67 | -476.70 950.46 | 328.80 286.38 | -335.28 950.46 | -399.52 357.09 | -193.86 950.46 | 470.23 427.80 | -52.44 950.46 | 540.94 498.51 | 88.98 950.46 | 611.65 569.22 | 230.40 950.46 | 682.36 639.93 | 371.83 950.46 | 753.07 710.64 | 515.25 950.46 | 782.57 822.56 | 654.67 950.46 |
Nonlinear.com

* FILOS
* INITIA
* INPUT
READ FILE="pu.dat"
* NONLIN
BEGIN EXECUT
BEGIN LOAD
LOADNR=1
STEPS EXPLIC SIZES .005 (200)
END LOAD
BEGIN ITERAT
METHOD NEWTON MODIFI
MAXITIE=200
END ITERAT
END EXECUT

BEGIN OUTPUT
DISPLA
FORCE
STATUS
STATUS CRACK
STRAIN
STRAIN TOTAL GREEN PRINCI
STRAIN CRACK
STRAIN PLASTI
STRESS
STRESS CRACK
STRESS TOTAL CAUCHY PRINCI
END OUTPUT
* END

J.A. H. Dekkers