EXPERIMENTAL INVESTIGATION OF THE SHEAR RESISTANCE OF STEEL FRAMES WITH PRECAST CONCRETE INFILL PANELS

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ABSTRACT: At the Technische Universiteit Eindhoven a research program on composite construction is underway aiming at the development of design rules for steel frames with discretely connected precast concrete infill panels subject to in-plane horizontal loading. This paper presents experimental and finite element results of an investigation into their lateral stiffness and strength. A discrete connection between steel frame and concrete panel consist of one or two achor bars welded to a partially cast-in steel plate which is fastened with two bolts to a gusset plate welded to a frame member. The bolts in the connection are loaded in shear only. Two variations on this type of connection were tested experimentally. To avoid brittle failure, the connections are designed for a failure mechanism consisting of ovalisation in the bolt holes due to bearing of the bolts. Experimental pull-out and shear tests on individual frame-panel connections were performed to establish their stiffness and failure load. Two full scale experiments were done on one-storey one-bay 3 by 3m infilled frame structures which were horizontally loaded at the top. With the known characteristics of the frame-panel connections from the experiments on individual connections, finite element analyses were performed on the infilled frame structures taking non-linear behaviour of the structural components into account. The finite element model yields good results for the lateral stiffness and lower and upper bounds for strength.

Keywords: Composite construction, full scale experimental testing, semi-integral infilled frames, precast concrete infill panels, steel frame to concrete panel connections, high-rise structures

1. INTRODUCTION

1.1 Classification of Structure

When subjected to an in-plane horizontal or racking load, the infill in steel framed structures will cause different types of composite behaviour depending on the material used and the way it is attached to the skeletal structure. Since the early fifties research has been carried out on the structural behaviour of steel frames with masonry infill (Thomas [1], Polyakov [2], Benjamin and Williams [3], Holmes [4]) and concrete infill or an isotropic infill material with a brittle type of tensile failure and a plastic type of compressive failure similar to medium strength concrete (Holmes [4], Stafford Smith [5,6]). The infill used to be considered as a non-structural element in design practice thereby conservatively neglecting its significant structural benefits. It was shown (Stafford Smith [5]) that ignoring the infilling may not be conservative but can cause certain elements in the lower parts of the structure to be overloaded.

Infilled frames have generally been classified as non-integral or fully-integral (Figures 1a and 1b). When connections such as strong bonding or shear connectors at the structural interface between the frame and infill are absent as for example with brick infill, the structure is called a non-integral infilled frame. In general, the stiffness and strength of non-integral infilled frames are dependent on the characteristics of separation and slip of the infill panel at the interface. Providing strong bonding or shear connectors at the interface significantly improves the performance of infilled frames. Such frames are called fully-integral infilled frames and generally have larger lateral stiffness and strength than non-integral infilled frames. When precast concrete infill panels are connected to steel frames at discrete locations, interaction at the structural interface is neither complete nor absent. A structure comprising a steel frame with an intermittently connected precast concrete panel can be considered as semi-integral (Figure 1c). The contribution of precast concrete

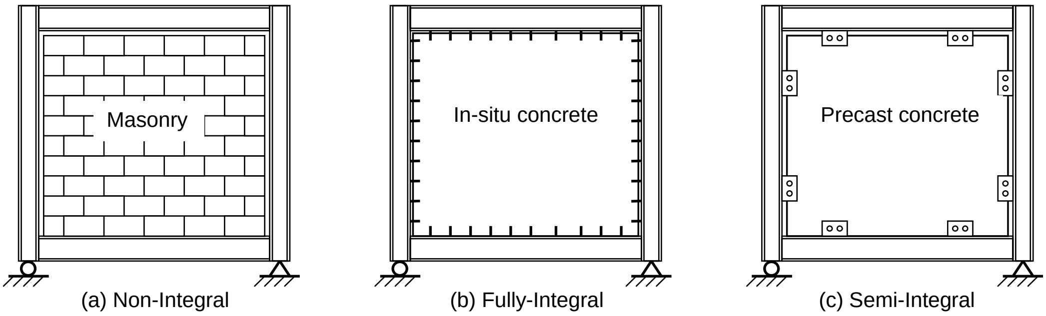


Figure 1. Classification of Infilled Steel Frames

infill panels to the lateral stiffness and strength of steel and concrete frames could be significant if the optimum quality, quantity and location of the discrete interface connections can be determined. Precast concrete panels with openings for windows would allow them to be placed in the building perimeter where they can contribute significantly to the lateral stiffness of the structure.

A research program at the Technische Universiteit Eindhoven is aimed at investigating the composite behaviour of semi-integral infilled frames subject to in-plane lateral loading. Recently the investigation has been focussed on the structural behaviour of discrete connections between the steel frame and precast concrete infill panel in addition to their influence on the lateral structural behaviour of an infilled steel frame. This publication describes the experimental testing of a frame panel connection with two different anchor configurations. It also presents results of two full scale experimental tests on 3 by 3 m one story high and one bay wide infilled frame structures subject to horizontal loading. A simple finite element model was developed to obtain the lateral stiffness and strength of these frames.

The investigation into the composite behaviour of precast reinforced concrete façades as infill panels in steel frames with discrete interface connections represents a new area of research in infilled frames. It aims at giving a better understanding of the complex behaviour of this structural system and ultimately at providing design rules for the structural performance in resisting racking loads.

2. STRUCTURAL BEHAVIOUR

2.1 Non-Integral Infilled Frames

Experimental investigations on non-integral infilled frames under racking load have shown (Stafford Smith [5-7], Barua and Mallick [8], Liauw and Kwan [9], Liauw and Lo [10], Ng'andu et al. [11]) that poor interaction between the frame and infill due to the absence of connectors or bonding causes friction only at the structural interface. As the infill panel takes a large portion of the lateral load at its loaded corners, the effects of the infill panel are similar to the action of a single diagonal strut bracing the frame. This analogy is justified by the phenomenon of slip and separation at the interface between the frame and the infill due to the difference in the deformed shapes of the surrounding steel frame and the concrete infill panel. Consequently, friction-slip at the interface becomes a governing factor in a non-integral infilled frame. The separation at the structural interface in addition to irregularities and unevenness produce considerable variations in strength and stiffness (Dawe and Seah [12]).

2.2 Fully-Integral Infilled Frames

When a continuous connection is provided by means of strong bonding or shear connectors at the structural interface between frame and infill panel, the separation at the interface will be restricted. Friction-slip, which is dependent on normal stress, will not play an important role in fully-integral infilled frames. In addition, the provision of shear connectors overcomes the problem of initial gap (lack of fit) at the interface. Consequently, fully-integral infilled frames in general have larger lateral stiffness and strength than non-integral infilled frames (Mallick and Garg [13]). They maintain their strength up to large deflections before final collapse of the structure.

2.3 Semi-Integral Infilled Frames

The idea of semi-integral infilled steel frames was earlier considered (Liauw and Kwan [14]) in a plastic theory of integral infilled frames with continuous connections along the beams and columns, where a 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 and tension forces.

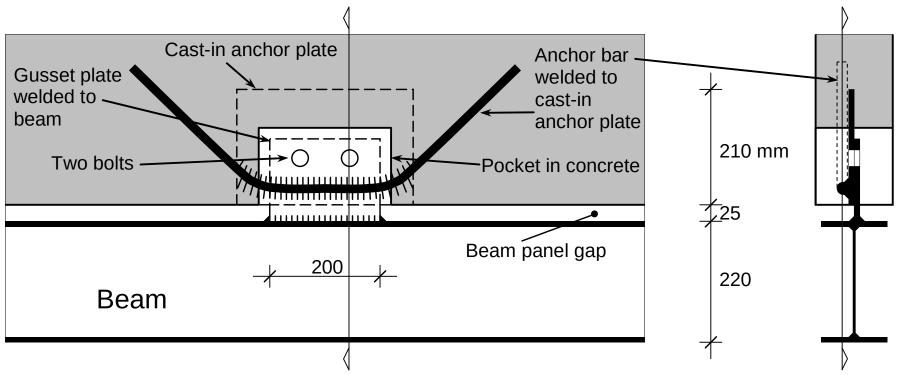


Figure 2. Typical Steel Beam to Concrete Panel Connection

The design of a discrete steel frame to concrete panel connection can take various forms (Teeuwen et al. [15,16]). The type of discrete interface connection for semi-integral frames used in this investigation is as shown in Figure 2. It consists of an anchor bolt, 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 center line of the structural elements thereby keeping eccentricities 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.

2.4 Preliminary Finite Element Analysis

An earlier finite element study (Tang et al. [17]) of the influence of discrete interface connections on the structural behaviour of a square steel frame with a precast reinforced concrete infill panel subject to racking load was limited to linear elastic analysis of 3.6 by 3.6 m one bay one story high frames. The infill panel had a thickness of 200 mm. HE220A sections were used for the columns and IPE200 for the beams. Plane stress elements representing the concrete panel were pin-connected to the frame members at discrete locations. Beam elements used for beams and columns were hinge connected at the corners. Numerical linear elastic analyses were performed to study the behaviour of the infilled frame subject to a horizontal point load at the top. Ten different

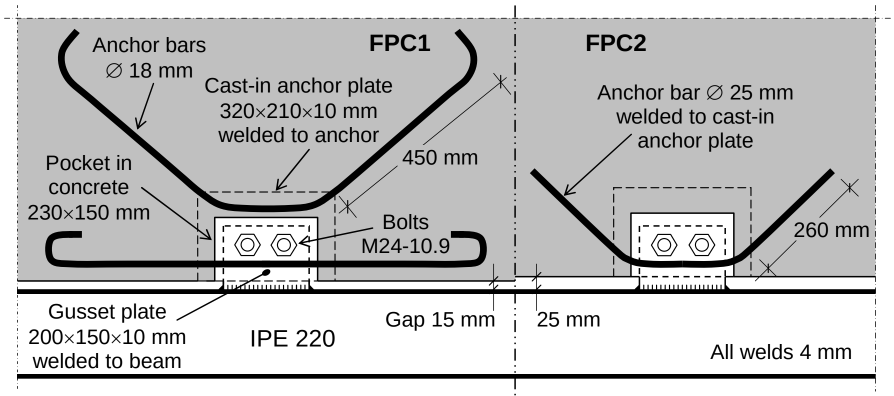


Figure 3. Steel Beam to Precast Concrete Panel Connections

patterns of discrete interface connections were investigated whereby the number and the locations of the connections were varied. It was observed that stress patterns in the panels of semi- and fully-integral infilled frames show extensive similarities. It was concluded that semi-integral frames are able to achieve similar structural performance as fully-integral frames; 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.

3. EXPERIMENTAL TESTING

3.1 Frame-Panel Connection

Frame-panel connections, FPC1 and FPC2 as shown in Figure 3, consist of one or two anchor bars welded to an anchor plate which is precast in a pocket at the edge of the concrete panel. A concrete panel measures 1400 mm by 600 mm with a thickness of 150 mm. The reinforcing bars are placed in two directions at both sides, see Table 1. The cast-in anchor plate is bolted to a gusset plate which is welded to the beam. It is assumed that this connection transfers normal and shear forces only. The connections were designed to fail in bearing, i.e. ovalisation of the bolt holes. Specific data for the materials are given in Table 1. The compressive strength of the concrete was obtained from standard cube tests of 150×150×150 mm. The equivalent characteristic cylinder strengths are given in brackets. The 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: pull-out tests perpendicular to the edge of the panel and shear tests parallel to the edge of the panel.

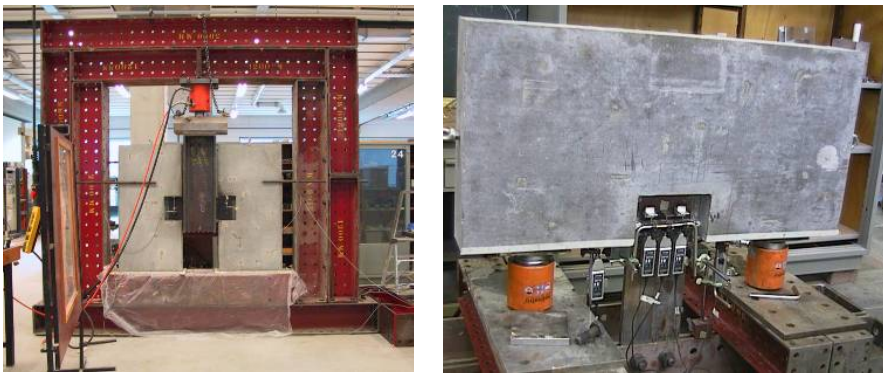
3.1.1 Connection in shear

The shear connection tests were done in pairs as shown in Figure 4a. An axial force is applied to the vertical steel member connected to two concrete panels which are fully supported on their short side.

Table 1. Measured Material Properties of Frame Panel Connections

	Bolts	Anchor bars FeB500		Anchor plates 320×210×10 mm		Gusset plates 200×150×10 mm		Concrete panels 1400×600×150 mm	
	M24	∅ mm	f_u N/mm^2	f_y N/mm^2	f_u N/mm^2	f_y N/mm^2	f_u N/mm^2	Reinforcing bars	f_{ck} N/mm^2
FPC1	10.9	25	500*	529	579	529	579	∅10 @ 200	47 (37)**
FPC2	10.9	18	500*	247	408	294	432	∅8 @ 150	44 (34)**

* assumed, ** approximate cylinder strength in brackets



(a) Shear Test (b) Tension Test

Figure 4. Test Set-Ups

Six frame-panel connections of type FPC1 and two connections of type FPC2 were tested in shear. Vertical displacements were measured on the steel section, the bolts, the connection plates and the concrete panels. The curves in Figure 5 indicate load-displacement measurements for FPC1. The force in kN indicates the load taken by a single connection. The displacements are obtained from the above measurements and represent the sum of anchor plate movement and ovalisation in the connection plates. 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 337kN. The load-deflection curve of the shear test for FPC2 in Figure 6 displays bi-linear behaviour up to a maximum resistance of 350kN. At this load the anchor plate split the concrete. The shear strengths and average shear stiffnesses of the connections are given in Table 2.

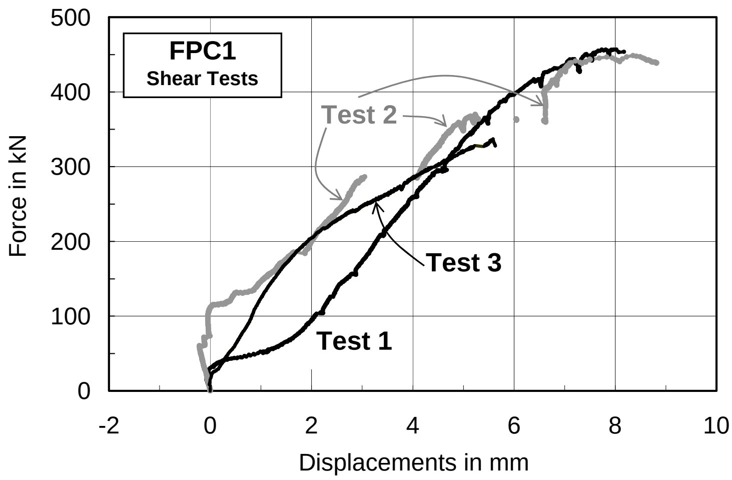


Figure 5. Shear Tests of Frame-Panel Connection 1

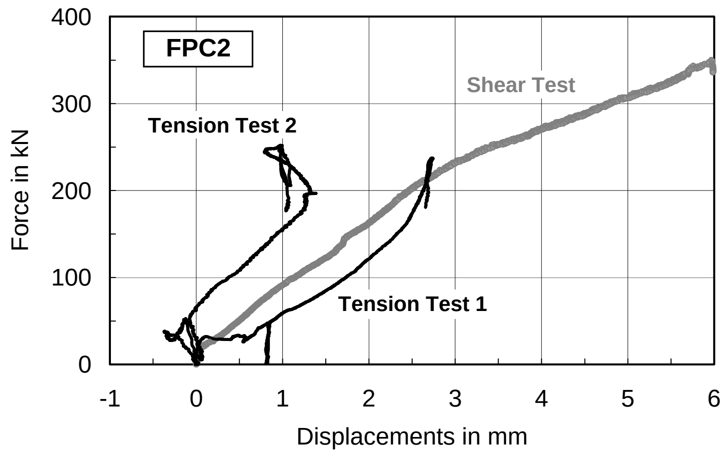


Figure 6. Shear and Tension Tests of Frame-Panel Connection 2

3.1.2 Connection in tension

The test set-up shown in Figure 4b comprises a single concrete panel that is placed on two jacks. The cast-in anchor plate is bolted to two 450 mm long steel holding strips of 100 by 20 mm which are connected to the test rig. Vertical displacements were measured on the the anchor plates, the bolts and the concrete panels. Four connections of type FPC1 were tested and their force-displacement curves are shown in Figure 7. The displacements are obtained from the above measurements and represent the sum of anchor plate movement and ovalisation in the anchor plates. Connection FPC2 was tested twice. Its tension behaviour is shown in Figure 6. The modes of failure for all tension tests were identical: anchor pull-out. The diagrams allow the strength and stiffness characteristics of the pull-out connections to be determined. They are given in Table 2.

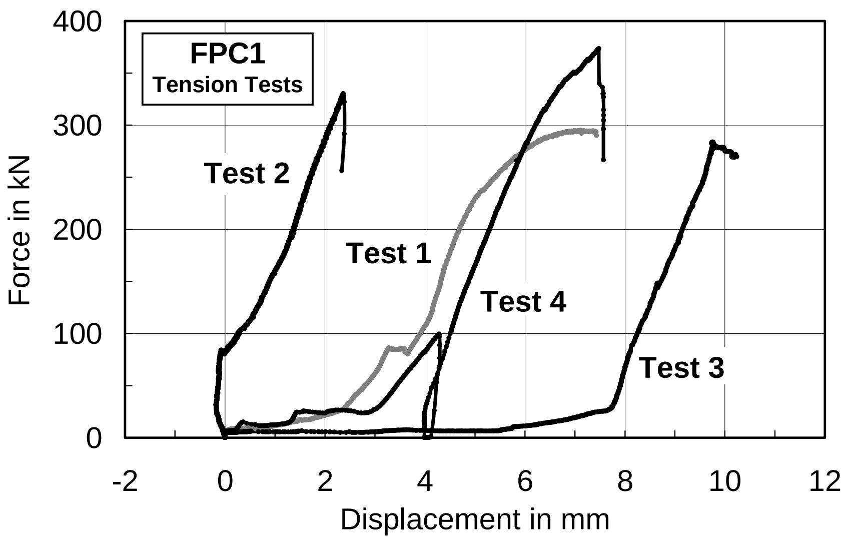


Figure 7. Tension Tests of Frame-Panel Connection 1

Table 2. Connection Properties obtained from FPC Tests

Test		Shear		Tension	
		Stiffness K_s in kN/mm	Strength $F_{s;u}$ in kN	Stiffness K_p in kN/mm	Strength $F_{p;u}$ in kN
FPC1	1	**86.2**	**457**	**112.9**	**295**
	2	88.2	448	103.6	330
	3	54.9	337*	121.7	284
	4	-	-	118.4	374
	Avg	(76.4)	(453)	(114.2)	(321)
FPC2	1	**56.8**	**350**	**86.1**	**237**
	2	-	-	91.4	252
	Avg	(56.8)	(350)	(88.8)	(245)

* test halted before ultimate load

3.2 Full Scale Frame-Panel Tests

The full scale test rig shown in Figure 8 consists of a vertical and a diagonal loop comprising HE300B sections causing the test rig members to be subject to tensile or compressive forces only. The 3.0 by 3.0 m infilled frames as shown in Figure 9 are built up with IPE220 sections for the beams and HE300B sections for the columns. The beam-to-column connections consist of 10 mm thick header plates welded all around to the beam. The header plates are connected to the column flange with one bolt M24 10.9 at mid height on each side of the beam web. The infill panel is connected to the steel beams at four locations, 875 mm from the column center lines with a 15 mm gap between steel and concrete all the way around for Test A with FPC1 type connections. A 25 mm gap was used for Test B with FPC2 type connections, see Figure 3. The reinforcement of the full scale concrete panels is the same as was used for the testing of the two frame-to-panel connections.

Figure 8. Full-Scale Test Set-Up

The specimens were horizontally loaded at the top by a 2 MN capacity jack. The load application at the upper beam was displacement controlled at 0.5 mm per minute. 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 yields a lateral frame stiffness of 1.28 kN/mm^2 and a rotational stiffness of the beam-to-column connections of 2.53×10^3 kNm/rad.

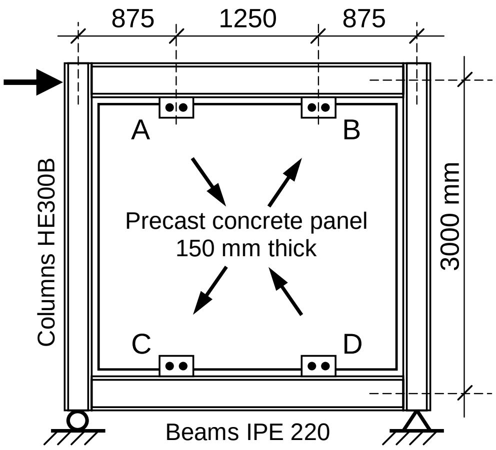


Figure 9. Steel Frame with Precast Concrete Infill Panel

The load-displacement curves in Figure 10 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 is 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 ovalisation of the bolt holes had taken place at several locations. Beyond 75 kN the lateral stiffness of infilled frame B is 12.5 kN/mm until 241 kN when pull-out failure occurred at the lower left frame-panel connection, point C in Figure 9. The load was further increased to 276 kN when pull-out failure occurred at the upper connection on the right at point B. 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.

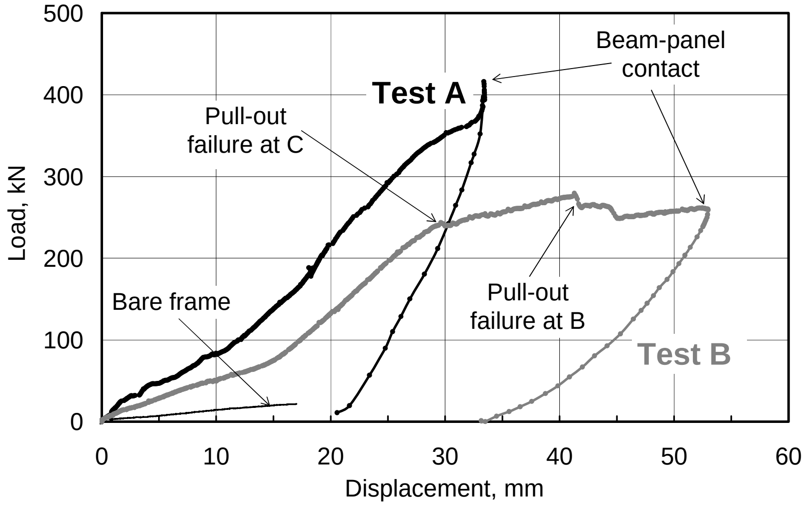


Figure 10. Experimental Results of Full-Scale Tests

4. FINITE ELEMENT ANALYSES

4.1 Finite Element Model

A finite element model has been developed for simulating the two full-scale tests. The model consists of a steel frame, a concrete infill panel and connections represented by springs connecting the gusset plates of the beams to the panel. The finite element model is shown in Figures 11 and 12.

4.1.1 Steel frame

Beam elements (BEAM3, [18]) are used for the frame members. The sectional properties of the beams and columns fit the sections used in the tests and are based on nominal dimensions. The

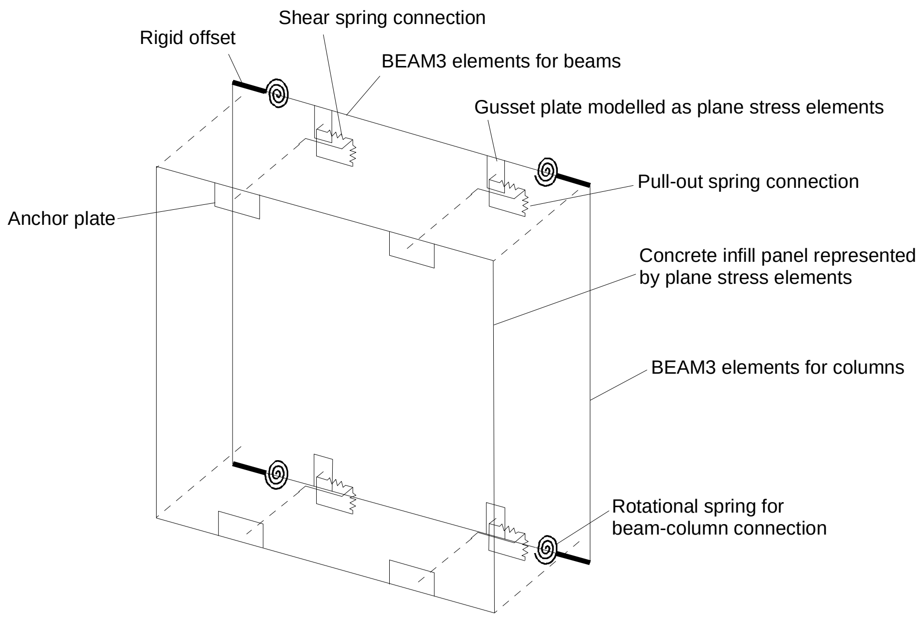


Figure 11. Finite Element Model Set-Up (Offset Infill Panel for Clarity)

connection between column and beam is modelled by a rigid offset (BEAM3 with a very high stifness) to take the column depth into account and a rotational spring (COMBIN14, [18]) which models its stiffness.

4.1.2 Precast concrete infill panel with anchor plate

The precast concrete infill panel as shown in Figure 12a was modelled by plane stress elements (PLANE82, [18]), with a thickness of 150 mm. For Test A, Young's modulus is E_A = 29.2 kN/mm^2 and for Test B E_B = 31.0 kN/mm^2. The plane stress elements of the concrete panel are connected to the plane stress elements of the steel anchor plates which share the same orthogonal grid. There is no overlapping of elements. The finite elements of the anchor plates (PLANE82, [18]) are modeled as steel plates with a thickness of 10 mm and E_s = 210 kN/mm^2. This way of modelling minimizes stress concentrations in the materials.

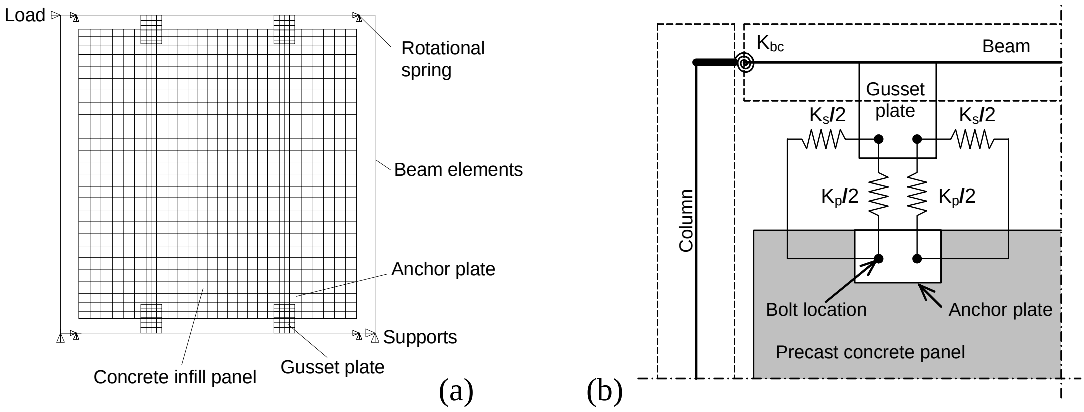


Figure 12. Finite Element Model of Frame-Panel-Connection

4.1.3 Frame-to-panel connections

The gusset plates, welded to the beams, are also modeled as steel plates (PLANE82, [18]) with a thickness of 10 mm and E_s = 210 kN/mm^2. The connection between an achor plate and a gusset plate is modelled by four springs using element COMBIN39 [18], see Figure 12b. They represent the anchor behaviour in the concrete and ovalisation in the steel connection plates: two in the shear direction and two in the tension direction. The results of Test 1 on shear and tension for FPC1 and FPC2 were used in the full scale frame-panel simulations; see bold type values in Table 2. The tensile strengths were reduced due to interaction with the shear load on the connections (Saari et al. [19], McMakin et al. [20]). Two tension-shear interaction curves were employed in an attempt to establish upper and lower bounds for the lateral load capacities of the infilled frames. Linear and

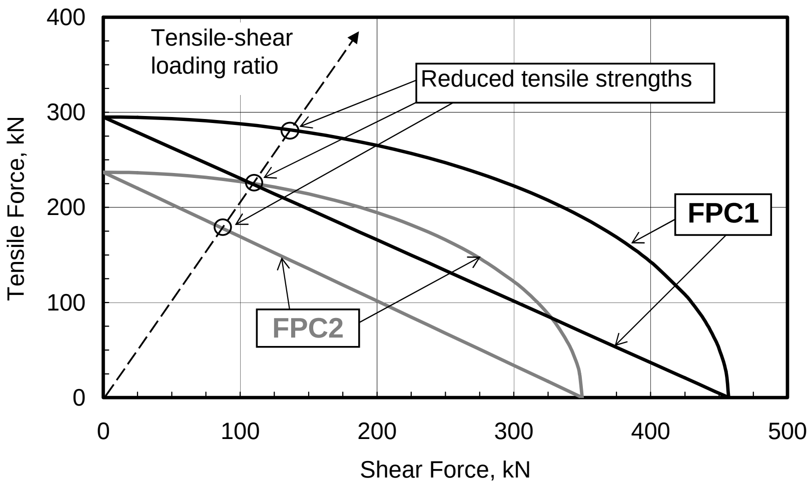


Figure 13. Tensile Strength Reduction of Connections due to Load Interaction

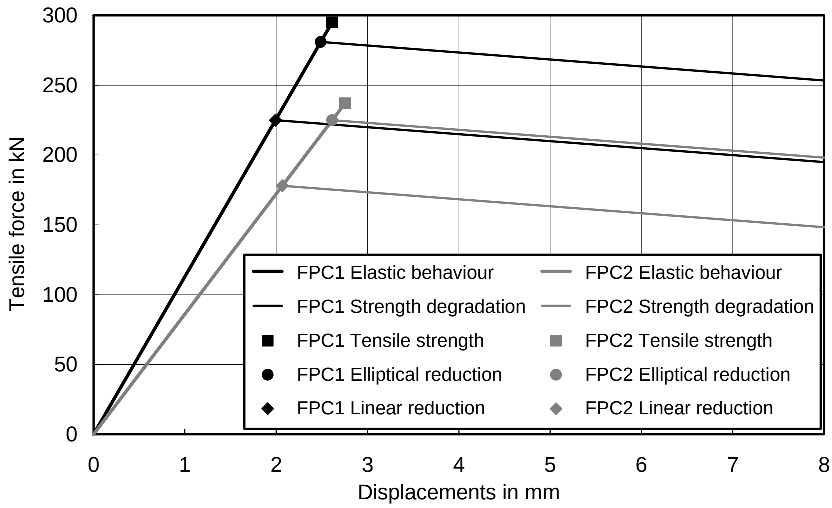


Figure 14. Tensile Characteristics of FPC1 and FPC2 Applied in Simulations

elliptical interaction curves are displayed in Figure 13. The applied tensile-shear load ratio as a result of the locations of the discrete connections clearly indicates that the connections will fail mainly in tension.

The full FPC1 tensile strength from Test 1 of 295 kN was linearly and elliptically reduced to $F_{p;u;1;lin}$ = 225 kN and $F_{p;u;1;ellip}$ = 281 kN resp. Analoguously for Test 1 of FPC2, the reduced values for its 237 kN full tensile strength then become $F_{p;u;2;lin}$ = 178 kN and $F_{p;u;2;ellip}$ = 225 kN. Bi-linear curves for tension, compression and shear behaviour were assumed to be sufficiently accurate. The elastic behaviour in tension is followed by a gradual linear reduction in load due to anchor pull-out: $K_{p;pl}$ = - 5 kN/mm. The tension behaviour of the connections used in the simulations is shown in Figure 14. For each connection the tensile stiffness in the elastic region is taken to be identical to the stiffness in compression. The compression behaviour is assumed to be elastic perfectly plastic. The shear failure load of the bolts in connection FPC1 was taken as the compressive strength of that connection, i.e. 457kN. The calculated onset of plate bearing by the bolts in connection FPC2 was used for limiting its compressive strength to 282kN. The behaviour of the connections in shear is also assumed to be elastic perfectly plastic. The shear strength for FPC1 is 457kN and 350kN for FPC2.

4.2 Simulations of Full-Scale Tests

Two finite element simulations were carried out for Test A and for Test B. One analysis was performed with an elliptically reduced tensile strength for the connection and one with a linearly reduced tensile strength. The load-displacement curve of the simulations are shown in Figure 15 for Test A and Test B in separate diagrams where they are compared to the experimentally obtained curves, omitting the settling-in phase. The diagrams clearly show that in the plastic regions, the experimentally obtained load-deflection curves fall between the results from the two finite element simulations. The use of an elliptically reduced tensile capacity of the connection yields numerically obtained strengths at first yield and ultimate strengths that are larger than the experimental loads. The employment of a linear tensile strength reduction of the connection leads to lower strengths than experimentally obtained for most of the plastic region. The diagrams also show that the finite element model is able to give reasonably accurate values for the elastic stiffness of a steel frame which is discretely connected to a precast concrete infill panel. The differences between numerically and experimentally obtained lateral stiffness are 6%. Table 3 shows numerically and experimentally obtained structural properties of the infilled frames in addition to their comparison.

At first yield one of the two pull-out springs in each connection to the tension diagonal fails. Differential rotation between gusset and anchor plates causes one spring to fail just before the other.

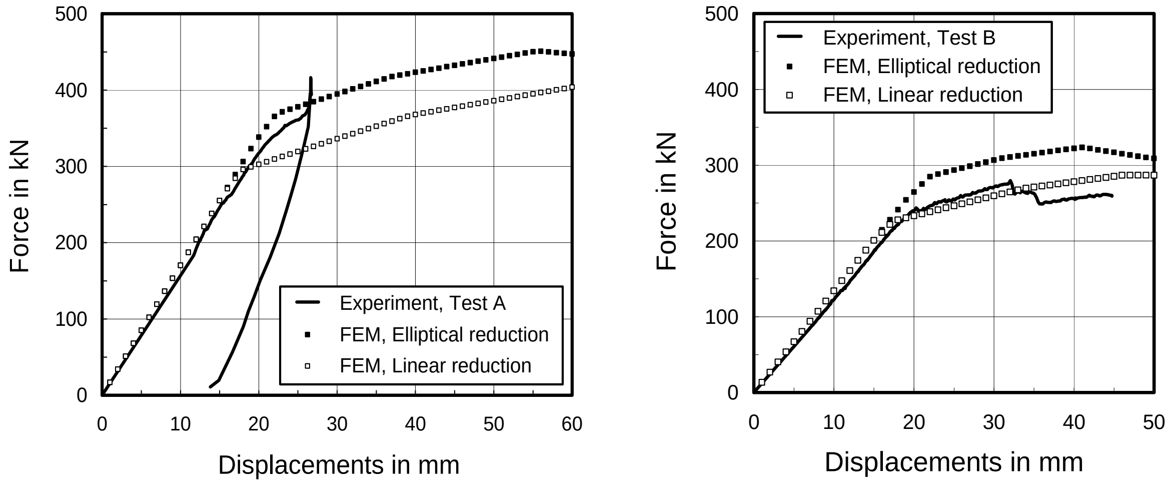


Figure 15. Experimental and Numerical Results of Full-Scale Tests

The strengths at yield as given in Table 3 are the loads for the condition where all four pull-out springs of the tension diagonal connections (locations B and C in Figure 9) have failed in tension. The ultimate strength is reached when all four “tension springs” at the connections of the compression diagonal (locations A and D in Figure 9) have failed in compression. The behaviour of all eight shear springs at the connections remains elastically linear. The simulations of the plastic behaviour, with maximum errors of 17% for strength, were less accurate than for the elastic region.

Table 3. Properties of Steel Frames with Precast Concrete Infill Panels

	Yielding level kN			**Ultimate strength** kN			**Lateral stiffness** kN/mm		
	Experiment	Numerical analysis	% difference	Experiment	Numerical analysis	% difference	Experiment	Numerical analysis	% difference
Test A	345	366* 293** (311)	+6 -15 (-10)	-	451* 407** (416)	-	15.9	16.9 (16.9)	+6 (+6)
Test B	241	283* 226** (240)	+17 -6 (-1)	27 6	323* 287** (287)	+17 +4 (+4)	12.5	13.3 (14.5)	+6 (+16)

* Elliptical reduction tensile strength of connection, ** Linear reduction tensile strength of connection
(-) Using averaged connection properties with linear reduction of tensile strength

For an additional simulation it is suggested that the input properties for the connections be adjusted to take the spread of the experimental results for all connection tests into account. This would lead to a 9% increase in tensile strength for FPC1 to 321 kN and a 3% increase for FPC2 to 245 kN. The average values of the other properties are shown in Table 2. Employing linear reductions for the tensile strengths yields $F_{p;u;1;lin;avg}$ = 239 kN and $F_{p;u;2;lin;avg}$ = 183 kN. The load deflection diagram of the simulations is shown in Figure 16 where they are compared to the experimental curves. The results for the simulation are further shown in brackets in Table 3. They show improved results for strength at yield level as the errors with a linear tensile strength reduction are now -10% and -1% resp. The results for the elastic lateral stiffness do not improve.

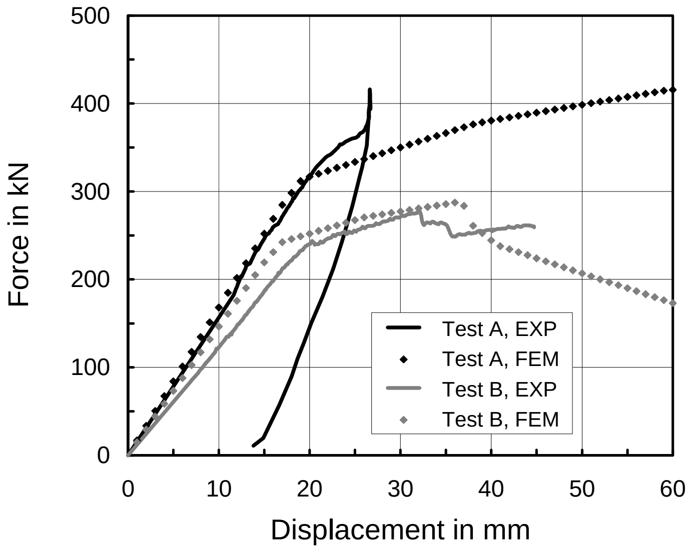


Figure 16. Experimental Tests Compared to Numerical Simulations
Using Averaged Experimental Properties for Connections

5. CONCLUSIONS

This article presents experimental and numerical studies on the behaviour of two full scale one-bay-one-story steel frames with pre-cast concrete infill panels subjected to in-plane horizontal loading. The concrete panels are discretely connected to the top and bottom beams of the steel frames at four locations. Two different types of frame-panel connections were tested experimentally. This allowed characteristic properties for stiffness and strength of the connections to be represented by bi-linear springs in finite element analyses of the infilled frames. The interaction between shear and tensile capacities of the two connections was taken into account.

Finite element simulations with linearly and elliptically reduced tensile strength of the connections were performed for each full-scale infilled frame structure taking into account non-linear material properties. The numerical models yield good results for the elastic behaviour and reasonable results for the plastic region.

It has been shown that simple finite element models make it possible to systematically investigate the influence of connections on the overall behaviour of steel frames with discretely connected precast concrete infill panels subject to in-plane lateral loading.

The following can be concluded from this investigation:

(i) The full-scale experimental tests show that a precast concrete panel in a semi-integral infilled steel framed structure with discrete frame-panel connections at the top and bottom beams can significantly improve the lateral stiffness of bare steel frames.

(ii) The observed lateral stiffness of the two infilled frame structures after the settling-in phase, are roughly 12 and 10 times the bare frame stiffness.

(iii) The lateral stiffness and ultimate strength of the two infilled structures were governed by the discrete frame-panel connections.

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