Composite Behaviour of Steel Frames with Precast Concrete Infill Panels

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

COMPOSITE BEHAVIOUR OF STEEL FRAMES WITH PRECAST CONCRETE INFILL PANELS

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ABSTRACT
This paper presents preliminary experimental and numerical results of an investigation into the composite behaviour of a steel frame with a precast concrete infill panel (S-PCP) subject to a lateral load. The steel-concrete connections consist of two plates connected with two bolts which are loaded in shear only. The connections are designed for a failure mechanism consisting of ovalisation in the bolt holes due to bearing of the bolts to avoid brittle failure. Experimental pull-out and shear tests on individual frame-panel connections were performed to establish their stiffness and failure load. A full scale experiment was performed on a one-storey one-bay 3 by 3m infilled frame structure horizontally loaded at the top. With the known characteristics of the frame-panel joints from the experiments on individual connections, a numerical analysis was performed on the infilled frame structure taking non-linear behaviour of the structural components into account. The finite element model yields reasonably accurate results and indicates a connection failure sequence similar to experimental failure.

1 INTRODUCTION

1.1 Classification of Structure

When subjected to horizontal loading, 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 done on the structural behaviour of high-rise steel frames with masonry and cast-in-place concrete infill [1]. Infill panels used to be considered as non-structural elements in design practice thereby conservatively neglecting their significant structural benefits. It has been shown [2, 3] that ignoring the infilling may not be conservative but can critically cause certain elements in the lower parts of the structure to be overloaded.

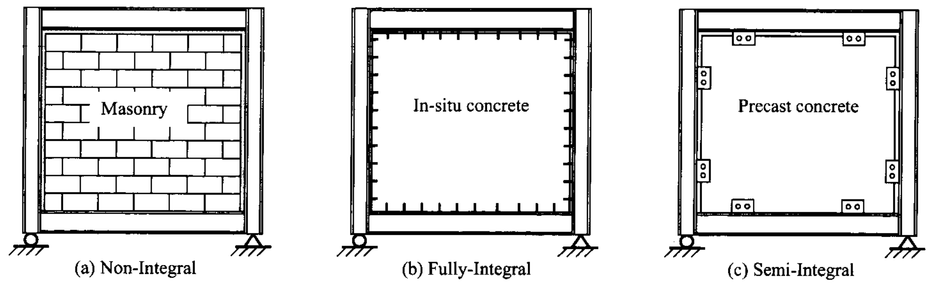


Fig. 1: Classification of infilled steel frames

Infilled frames have generally been classified as non-integral or fully-integral infilled frames (Figures 1a and 1b). When connections such as strong bonding or shear connectors at the structural interface between the frame and infill panel 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 [4, 5]. Providing strong bonding or shear connectors at the interface significantly improves performance of infilled frames in terms of load resistance [6, 7, 8]. Such infilled frames are called fully-integral infilled frames and generally have higher lateral stiffness and strength than non-integral infilled frames. Several theories [8, 9, 10] have been developed for the structural interaction between frame and infill.

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, see (Figure 1c). The contribution of precast concrete 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. This research project is mainly concerned with the setting up of a simple finite element model to investigate the influence of the number and location of the connections on the composite behaviour of semi-integral infilled frames subject to lateral loading. Precast concrete panels with openings for windows would allow them to be placed in the façade where they could contribute significantly to the lateral stiffness of the structure. 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. The project aims to give a better understanding of the complex behaviour of this structural system and ultimately provide design rules for the structural performance in resisting lateral loading.

2 STRUCTURAL BEHAVIOUR

2.1 Non-Integral Infilled Frames

Experimental investigations on non-integral infilled frames under racking load have shown [2, 9, 6] that poor interaction between the frame and infill due to the absence of connectors or bonding yields friction only at the structural interface. As the infill panel takes a large portion of lateral load at its loaded corner, the effects of the infill panel are similar to the action of a 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. Separation between the frame and infill is due to the difference in the deformed shapes and to tensile forces induced at the interface. Consequently, friction-slip at the interface becomes a governing factor in a non-integral infilled frame. Separation at the structural interface in addition to irregularities and unevenness produce considerable variations in strength and stiffness making the structural behaviour difficult to predict.

2.2 Fully-Integral Infilled Frames

When a continuous connection is provided by means of strong bonding or shear connectors at the structural interface, 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 higher lateral stiffness and strength than non-integral infilled frames. They maintain their strength up to large deflections before final collapse of the structure.

2.3 Semi-Integral Infilled Frames

The discrete interface connection for semi-integral frames as shown in Figure 2 may consist of a steel plate precast in a pocket at the edge of the concrete panel. The connection is located on the center line of the elements thereby avoiding eccentricities. It is assumed that this simple 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 semi-integral infilled frames will cause stress concentrations in the concrete panels which will influence the strength of the structure. The formation of stress paths, i.e. equivalent bracing pattern within the panel, will contribute to the stiffness of the structure.

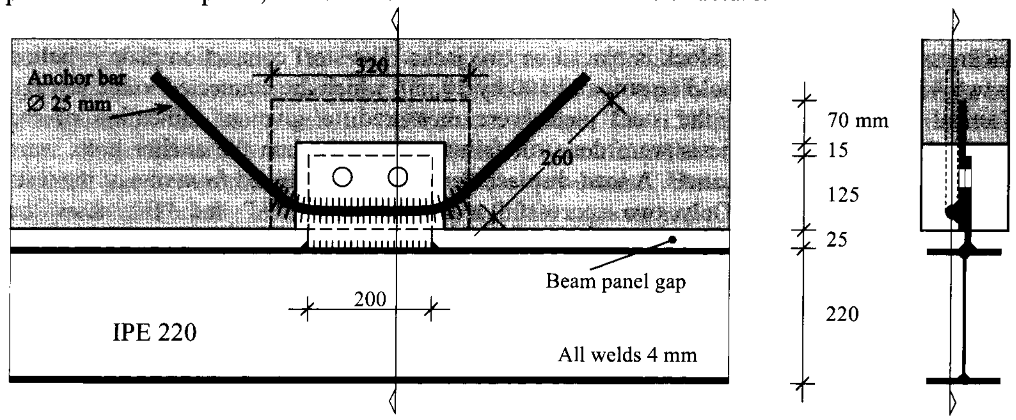


Fig. 2: Steel beam to concrete panel connection

2.4 Preliminary Numerical Analysis

An earlier numerical study [11] 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 lateral load was limited to linear elastic analysis. Commonly used dimensions from the concrete prefabrication industry in The Netherlands were adopted to build a simple model (3600 mm span and 3600 mm height) of the structure. The infill panel had a thickness of 200 mm. HE220A sections were used for columns and IPE200 for beams. Plane stress elements representing the concrete panel were pin-connected to the frame members at discrete locations. The beams were hinge connected to the columns. 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 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 may be able to achieve similar improvements in 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-column joints.

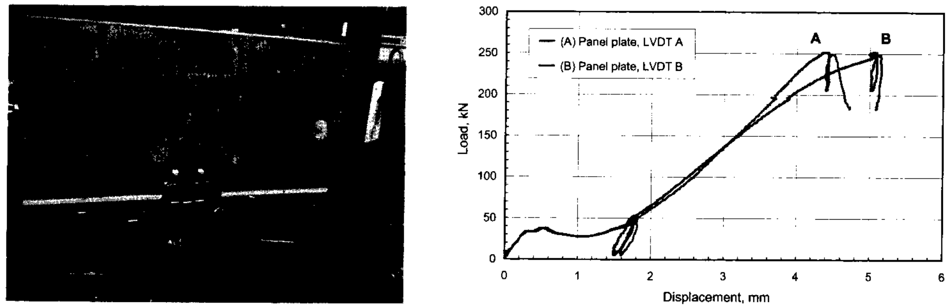
3 EXPERIMENTS

3.1 Materials

The interface connection as shown in Figure 2 was designed for a failure mechanism consisting of ovalisation of the bolt holes due to bolts loaded in bearing. It comprises two 10 mm thick steel plates which are connected together with two 10.9M24 bolts. A plate of 320 by 210 mm is welded to a single 25 mm diameter U-shaped reinforcing bar (Feb500) that acts as an anchor bolt. This is cast in place across a 230 by 140 mm slot in the panel at mid-section in the 150 mm thick concrete. The total length of the anchor bolt in the concrete is 520 mm. The second plate of 200 by 150 mm is welded to the steel beam. Standard tensile tests on the plate material yielded for the plates cast in concrete $f_y = 247$ N/mm^2 ($f_t = 408$ N/mm^2) and for the welded plates to the beams $f_y = 294$ N/mm^2 ($f_t = 432$ N/mm^2). Two such connections were tested in shear and for tension pull-out till failure. Standard cube tests of 150×150×150 mm concrete resulted in a compressive strength of 44 N/mm^2. The infill panel is reinforced on both sides with ∅8 mm at 150 mm in both directions.

3.2 Pull-out Tests

The test set-up as shown in Figure 3a comprises a concrete panel of 1400 by 600 mm with a thickness of 150 mm. This block is placed on two jacks. The steel connection 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 of the steel plate were measured to give an indication of the ovalisation. Two pull-out tests were done till failure occurred when the anchor bolts were pulled out of the concrete panels. A load-displacement diagram obtained from one of the tests is shown in Figure 3b. Only two successful readings, curves "A" and "B", show the deformations in the steel plates due to ovalisation and strain in the steel strips.

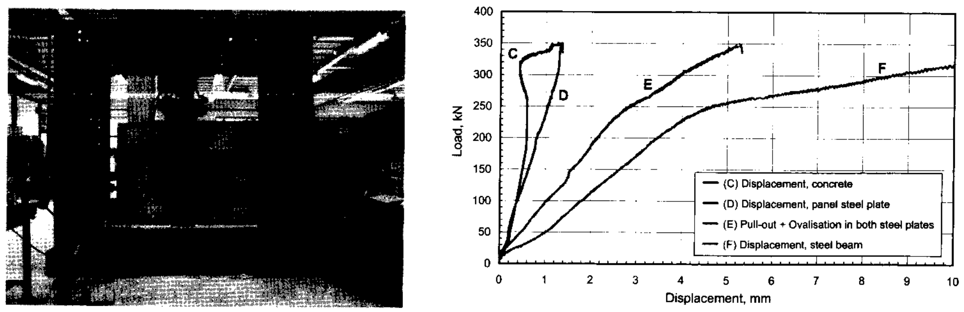


(a) Test set-up (b) Test data

Fig. 3: Pull-out test

3.3 Shear Tests

Two connections were tested in shear in a single experiment. Figure 4a shows the experimental set-up. The two concrete panels measuring 1400 by 600 were connected to a steel beam. Axial compression was applied to the beam to load the connections in shear. The concrete panels were fully supported on the short sides. Vertical displacements were measured on the beam, the connecting plates and the concrete panels.



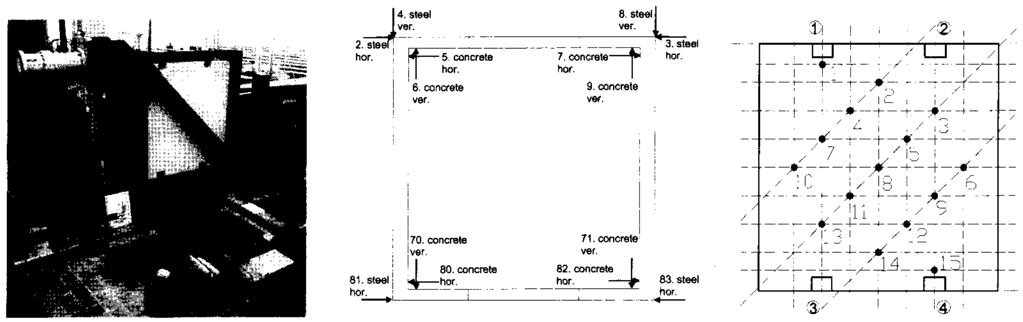
(a) Test set-up (b) Test data

Fig. 4: Shear test

The shear test continued until the limit of several displacement devices was reached. Figure 4b gives a load-displacement diagram. Curve "E" indicates ovalisation in both connection plates in addition to displacement of the panel steel plate in the concrete which caused cracks at mid thickness. The difference between curves "C" and "D" gives an indication of the "anchor pull-out". It was observed that large shear deformations occurred in the steel plates welded to the beam as shown by curve "F".

3.4 Full – Scale Test

The test rig shown in Figure 5a consists of a vertical and a diagonal loop comprising HE300B sections causing the test rig members to be loaded in tension or compression only. The 3000 by 3000 mm infilled frame was built up from IPE220 sections for the beams and HE300B sections for the columns. The beam-column connection consisted of a 10 mm thick header plate welded all around to the beam. This plate was connected with one bolt at mid height on either side of the beam web to the column flange. The 150 mm thick infill panel was connected to the steel beams at four locations, 900 mm from the column center lines, leaving a 25 mm gap between steel and concrete all the way around. The specimen was slotted in the loops and horizontally loaded at the top by a 2 MN capacity jack. Vertical reactions occurred at bottom left and right whilst the horizontal resistance was supplied at the bottom right hand corner. The position of the specimen in relation to the ground was measured by LVDT's (Linear Variable Displacement Transducers) as shown in Figure 5b. Rosettes were placed on a regular grid as indicated in Figure 5c, to measure strains at specific locations on the prefabricated concrete panel.



(a) Test rig (b) LVDT locations (c) Rosette locations

Fig. 5: Full-scale test set-up with instrumentation

The load application at the upper beam was displacement controlled at 0.5 mm per minute. Before testing the infilled frame structure, the bare frame without the infill panel was tested up to a lateral load of 20 kN. It allows the rotational stiffness of the beam-column connections to be determined.

In the initial stages of the test up until 75 kN the structure displayed a rather flexible behaviour. It is thought that in this settling-in stage not all four connections were participating in resisting the applied horizontal loading on the framed structure. It might well have been that the concrete panel was initially only loaded along a "compression diagonal". Beyond 75 kN the lateral stiffness is larger and constant until 240 kN when tension failure occurred at the bottom left steel-concrete connection. The load was further increased to 276 kN where the connection at top-right failed, also in tension. With only a compression diagonal the frame load reached 257 kN when steel concrete contact occurred. It was then decided to take the load off the structure.

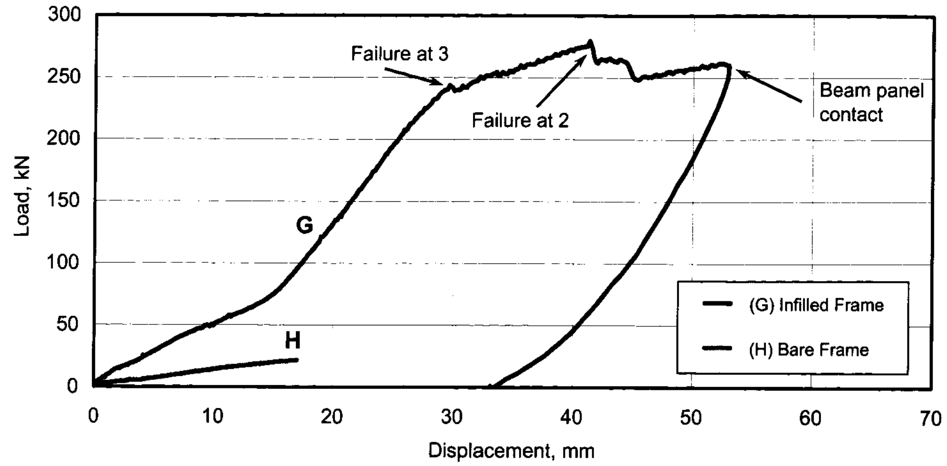


Fig. 6: Load versus deflection, full-scale test infilled frame

4 FINITE ELEMENT MODEL

4.1 Model Set-up

A simple finite element model was developed for simulating the full-scale test. The model consists of a frame, a concrete infill panel, and connections represented by springs between frame and panel. The finite element model set-up is shown in Figure 7.

Beam elements (BEAM3, [12]) are used for the frame members. The sectional properties of the elements fit the sections used experimentally, i.e. HE300B for the columns and IPE220 for the beams. The connection between column and beam is modelled by a rigid offset to take the column depth into account. A rotational spring models the flexibility of this connection. The rotational spring stiffness was calibrated to the experimental test results of the bare frame, see curve "H" in Figure 6.

The concrete infill panel was modelled by plane stress elements (PLANE82, [12]), with a thickness of 150 mm and E is 31000 N/mm^2. The panel steel plate and the frame steel plate are modelled as steel plates with a thickness of 10 mm and E is 210000 N/mm^2.

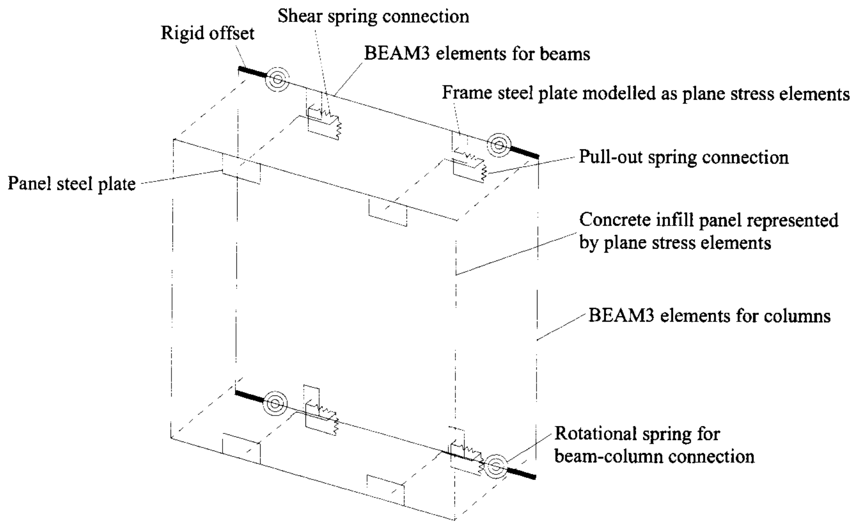


Fig. 7: Finite element model set-up (off-set infill panel for improved clarity)

The connections between frame and infill panel, via the frame steel plate and panel steel plate, are modelled by shear springs and pull-out springs. The horizontal shear and vertical pull-out spring stiffnesses and strengths were derived from the individual experimental test results of the connections.

4.2 Rotational Springs

The rotational springs connecting the beams to the columns as shown in Figure 7 were calibrated by comparing load-displacement behaviour of the finite element model without infill panel to that of a test of a bare frame, see curve "H" in Figure 6. The rotational spring stiffness was found to be 2.5322*10^9 Nmm/rad.

4.3 Pull-out Spring

The pull-out spring connection in Figure 7 simulates the tension and compression behaviour of the actual connection. For derivation of the spring stiffness and ultimate strength, data from the pull-out test in Figure 3 is used. It is assumed here that the deformations are mainly due to ovalisation in the connection plates. This allows the finite element spring to have the same behaviour as displayed by curve "A" or "B". A small correction in linear stiffness is made because of ovalisation and tensile strain in the thicker loading strips. The elastic pull-out stiffness of the connection then is 68400 N/mm. The onset of plastic ovalisation of the bolt-holes is used for limiting the compressive strength to 282 kN. Anchor pull-out limits the tension strength to 250 kN and post-failure strength reduction for the element model is included but less aggressively to avoid numerical instabilities. This behaviour in tension and compression is represented by curve "J" in Figure 8a for a single connection. In the finite element model these characteristics of the connection with two bolts are modelled by two identical springs at the bolt locations.

4.4 Shear Spring

The shear spring connection in Figure 7 simulates the shear behaviour of the actual connection. Curve "E" from Figure 4 shows ovalisation in the steel plates in addition to anchor pull-out. The elastic shear spring only needs to model ovalisation and must be reduced by the anchor pull-out stiffness which is represented by the difference between curves "C" and "D". The elastic shear stiffness of one connection thus is 56800 N/mm. This remains constant up to a shear load of 122 kN where the bolt holes start to yield as shown by curve "K" in Figure 8 for two connections. The second leg of curve "F" from Figure 4 gives an indication of yielding at all bolt holes including strain hardening in the steel plates. The shear

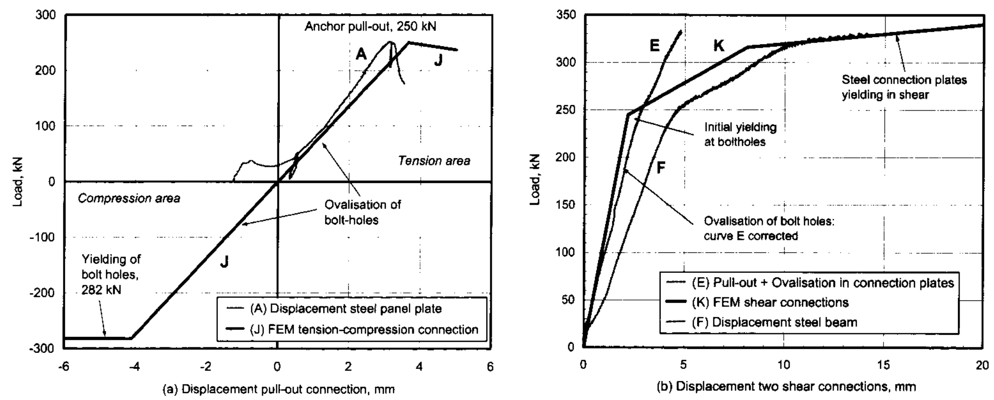


Fig. 8: Load-displacement characteristics of connections

stiffness here is 6300 N/mm for one connection and remains constant up to a shear load of 158 kN. No distinction is made between left or right movement of the connection. In the finite element model the shear characteristics of a connection with two bolts are modelled by two additional identical springs at the bolt locations.

4.5 Finite Element Model Infilled Frame Structure

The behaviour of the anchor pull-out from the concrete in tension and shear is taken to be modelled by the nodal connections between the plane stress elements of the concrete infill and panel steel plates. Then, using the set-up from Figure 7 and the finite element model as shown in Figure 9, the full elastic-plastic load-displacement curve "L" can be determined and compared to the experimentally obtained curve "G", omitting the settling in phase.

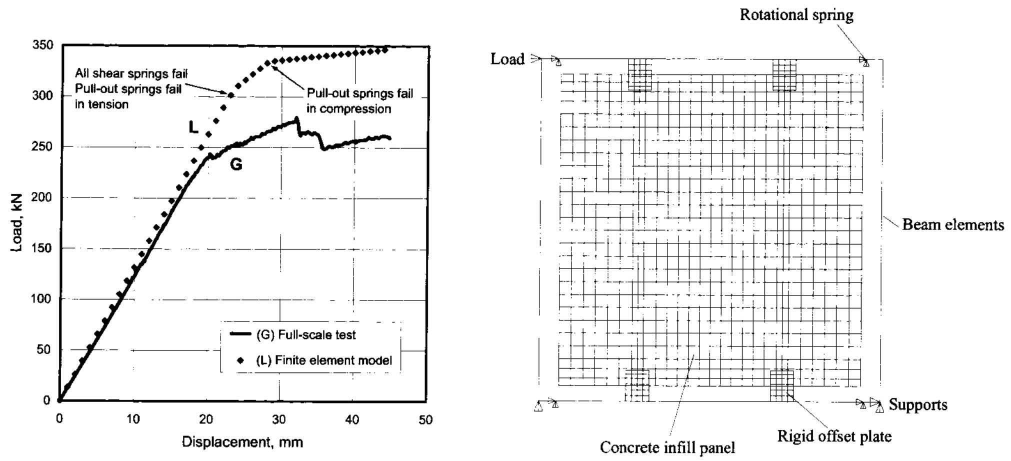


Fig. 9: Total finite element model versus experiment.

It is shown in Figure 9 that a simple finite element model is able to give a reasonably accurate value for the elastic stiffness of a steel frame with a precast concrete infill panel. The numerically derived plastic strength (333 kN) is 22 % higher than the experimental value (272 kN). Initial failure occurs simultaneously at all four connections in shear. Almost at the same time the maximum strength of the pull-out anchors at locations 2 and 3 (see Figure 5c) is reached. Ovalisation failure due to compression in the connection plates at locations 1 and 4 initiates the final plastic stage.

4.6 Adjustment Pull-out Spring

In order to separate the failure sequence, the tension strength of the connections at locations 2 and 3 is reduced by 22 %. These connections are at the ends of the tensile "diagonal" in the concrete panel. During full scale testing anchor pull-out was first observed near location 3 as shown in Figure 6.

Figure 10 shows the numerical results with the adjusted pull-out springs. The experimental behaviour is now modelled with a reasonable accuracy up to a horizontal displacement of 30 mm. This is where the pull-out springs fail in compression.

5 DISCUSSION

The full-scale experimental test shows that the semi-integral infilled steel framed structure with two discrete connections on the top and bottom beams each is able to resist a lateral load

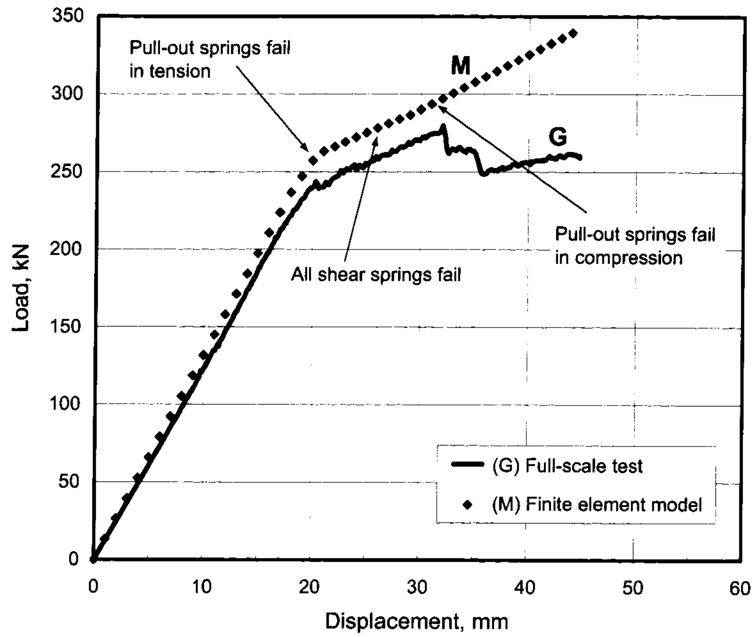


Fig. 10: Infilled frame, FEModel with adjusted tension spring

of over 250 kN. The locations and the number of connections were based on previous research [11].

A simple finite element model was developed by calibrating connection springs for rotation, shear and pull-out to individual series of experiments. The model yields reasonably accurate results for the elastic behaviour and a significant part of the plastic behaviour. It also indicates a connection failure sequence which is similar to experimental failure.

The simple finite element model makes it already possible to systematically investigate the influence of connections on the overall behaviour of steel frames with discretely connected precast concrete infill panels subject to lateral loading.

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