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Analysis of steel frames with precast concrete infill panels

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Summary

This paper presents experimental and numerical analyses of a new type of hybrid lateral load resisting structure. This structure consists of a steel frame with a discretely connected precast concrete infill panel with a window opening. The discrete connections are formed by structural bolts on the column and beam in every corner of the steel frame, confining the precast concrete panel within the steel frame. With the finite element program DIANA, the response of 5 full-scale experiments on one-storey, one-bay, 3 x 3 m infilled frame structures, having different window opening geometries, was simulated. The finite element simulations were performed taking into account non-linear material characteristics and geometrical non-linearity. A comparison between the full-scale experiments and simulations shows that the finite element models enable simulating the elastic and plastic behaviour of the hybrid lateral load resisting infilled frame.

Keywords: infilled frame; steel; precast concrete; structural stability; high-rise buildings.

1. Introduction

Construction time is a cost-crucial influencing factor. Reducing construction time means saving money, both directly and indirectly for example due to reduced hindrance to the surrounding of the building site. Reduction of construction time can be achieved by many means, for example, the use of prefabricated elements, time efficient joint construction or smarter construction procedures.

Meeting these requirements, a hybrid lateral load resisting structure has been designed for the construction of tall buildings at Eindhoven University of Technology. It is an integrated building system consisting of infilled steel frames with discretely connected precast concrete infill panels, enabling the assembly of tall buildings direct from truck. Besides the stiffening and strengthening effect of the infill panels on the frame structure, economical benefits may be realised in saving costs on materials and labour, and in reducing construction times. Design rules are needed to facilitate the application of this hybrid lateral load resisting structure for the construction of (tall) buildings.

To provide insight into the behaviour of this structure, full-scale experiments on one-storey one-bay infilled frame structures were carried out. To be able to evaluate more thoroughly stresses and deformations in the structure, the experiments were supplemented by finite element analyses.

2. Structure

The one-storey, one-bay, 3 by 3 m infilled frame structure subject to experimental and numerical analysis consists of a simply connected steel frame, constructed of HE180M sections for columns and beams. The precast concrete panels, provided with a window opening, are 200 mm thick. To investigate the effect of the size and position of a window opening, five different opening geometries were experimentally tested.

A discrete steel-concrete connection has been developed, enabling steel frames and precast concrete panels to act compositely when subject to lateral loading. This connection is realized by structural bolts on the column and beam in every corner of the frame, confining the infill panel within the



frame (Fig. 1). When the infilled frame is loaded laterally, the lateral load is transferred from the frame to the panel through the bolts which are loaded in compression only. To introduce the forces into the infill panel, angle members are cast at every corner of the panel. To prevent high stress concentrations in the angle members directly under the compression bolts, high-strength steel caps are applied there. The structures are designed to fail by a bolt failure mechanism. The preferred failure mode is shearing of the bolt through the nut. Failure of the bolts will not directly result in failure of the structure, as force transmission will still occur in the loaded corners of the frame by contact pressure between frame and panel (fail safe concept). Moreover, bolts can easily be replaced while steel structure and concrete panel remain undamaged.

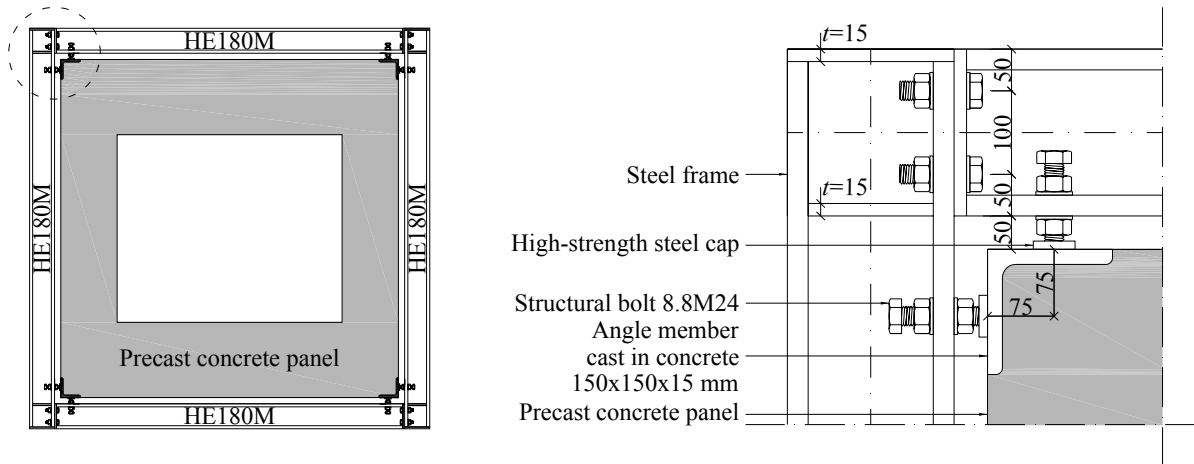


Fig. 1: Infilled steel frame with discretely connected precast concrete panel

3. Full-scale experiments

3.1 Test setup

A unique test rig was used to perform full-scale tests on the one-storey, one-bay infilled frames (Fig. 2). This test rig is composed of two rigid triangular frames, constructed of HE300B members. These two triangular frames are linked through rigid steel members at their corners. A specimen can be positioned between the two triangular frames and is supported on two different supports there. At the side of the jack, the lower corner of the specimen is fixed in vertical direction to the test rig by four steel M30 rods. This support is intended to act as a roller support with a restrained displacement in vertical direction. At the opposite lower corner, the specimen is supported in a steel block which restrains the specimen from both horizontal and vertical displacement. This support is supposed to act as a pin support. A specimen can be loaded laterally by a jack that is coupled to the top corners of the triangular frames by stiff steel plates, acting at height of the centre of the top beam. This jack has a stroke of 200 mm and is able to provide a maximum load of 2 MN.

To record the behaviour of a specimen under lateral loading, several measurements were made during the tests. A scheme of the arrangement of the instrumentation is shown in Fig. 3. The global position of the specimen in relation to the ground was measured with Linear Variable Differential Transformers (LVDTs) and digital clock gages (DCGs) at the four corners of both frame ($\Delta 1$ to $\Delta 8$) and panel ($\Delta 9$ to $\Delta 16$). Deformations of the panel were measured across both diagonals ($\delta 13$ and $\delta 14$) with Cable-Extension Position Transducers (PTs). To find the strain distribution in the precast concrete panel, strain gauge rosettes were placed on specific locations (Rosettes A to K). These locations are mainly situated in the compression zones, since due to cracking of the concrete in the tension zones, rosettes might be damaged there and become unable to provide measurements. Therefore, measurements in the tension zones of the panel were made with 6 LVDTs (δa to δf) over a distance of 300 mm, in order to determine the average strain and initiation of cracks in these zones. Deformations of the discrete interface connections were measured at one side of the specimens. To this end, LVDTs were coupled at bolt height between the panel and the outer flanges, measuring the deformation of the entire connection ($\delta 1$ to $\delta 8$). Four LVDTs were applied to measure the displacement of the discrete connection bolts in the loaded corners with respect to the opposite flange ($\delta 9$ to $\delta 12$). With these LVDTs the anticipated bolt failure behaviour could be recorded.



Fig. 2: Test rig

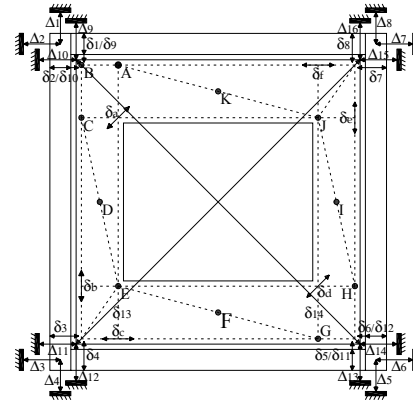


Fig. 3: Measurement scheme

3.2 Test specimens

Five precast reinforced concrete panels ($l \times h \times t = 2700 \times 2700 \times 200 \text{ mm}^3$) having different sizes of window opening (Fig. 4) were cast in a precast concrete factory. A self-compacting concrete was applied of concrete grade C45/55. The concrete mixture comprises aggregates (sand (0 - 6 mm) and gravel (4 - 16 mm)), limestone meal, Portland cement CEM I 52,5 R which develops a high early strength that is needed for a one-day casting cycle, super plasticizer, and water (watercement ratio = 0.45). The panels were all reinforced with longitudinal reinforcement $\text{Ø}25$ and stirrup reinforcement $\text{Ø}8$ with a concrete cover of 15 mm. Angle members ($150 \times 150 \times 15$) in S235 are cast in every corner of the panel. Wedge reinforcement is provided in the corners to prevent concrete tensile splitting there. All applied reinforcement was FeB500.

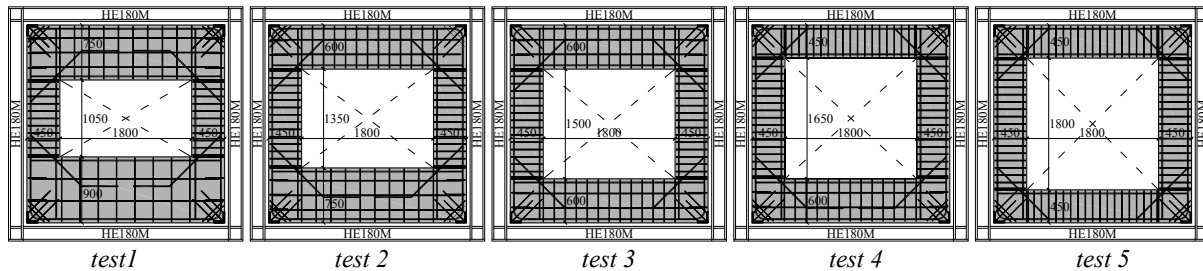


Fig. 4: Geometric properties test specimens

3.3 Testing procedures

In order to quantify the contribution of an infill panel to the stiffness of its confining frame structure, the stiffness of the bare frame structure without the infill has to be known. For that reason, the bare frame was tested each time before placing the precast concrete infill panel within the frame. After the bare frame was tested, it was positioned over the concrete panel. Then, the discrete connection bolts were placed and tightened up to a specified torque of 275 Nm, to provide identical boundary conditions as good as possible for all tests. The testing procedure of the infilled frames involved a preliminary preload up to 50 kN to close up initial gaps and contact tolerances between the specimen and the test rig. After the unloading, the infilled frames were loaded up again to failure. For both bare frame and infilled frame, the load was applied displacement controlled at 1 mm/min.

3.4 Experimental results

The most relevant results to describe the major full-scale behaviour characteristics and to verify the numerical model are briefly summarized in this section. In Fig. 5, the load-deflection response of the 5 tested infilled frame structures and 1 representative bare frame is shown. The typical infilled frame behaviour is characterised by a relative high initial stiffness, resulting from the tightening and thus prestressing of the steel-concrete connection in combination with uncracked panel behaviour. Next, the lateral stiffness decreases due to crack initiation, and can be considered linear till around 500 kN, followed by a non-linear branch and finally failure.



For test numbers 1 to 4, failure occurred by shearing of the steel-concrete connection bolts through the nuts by stripping of the threads of the bolts (Fig. 6). All failures were accompanied by a loud bump and at the same time a drop in load. After this load drop, it could be observed that the structure still was able to support some lateral load, as the load started to increase again. At that moment it was decided to end the tests.

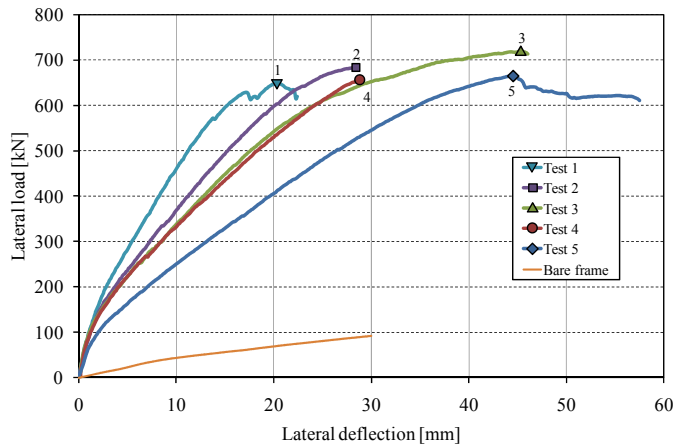


Fig. 5: Load-deflection response infilled frames

Table 1: Test results infilled frames

Test No.	Strength [kN]	Stiffness [kN/mm]	Stiffness factor (α)
1	650	46.1	10.0
2	684	36.9	7.9
3	719	33.9	6.9
4	656	33.2	7.7
5	664	25.1	5.5

For test 5, failure took place at the infill panel (Fig. 7) by crushing of a concrete strut with spalling and yielding of the wedge reinforcement in the two tension corners of the panel. As no obvious load drop was observed, it was decided to end the test after a lateral deformation was measured of 60 mm. At this moment the structure was still able to support a lateral load of 600 kN.

Table 1 presents the results in numerical values. Terms used to describe the behaviour of the infilled frame structures are the ultimate strength (F_u), being the maximum reached load level and the secant stiffness (k_{sec}). This secant stiffness is determined by taking the load, corresponding to a lateral deflection of 10 mm which is 1/300 of the height of the structure. This is the recommended serviceability limit state for the horizontal deflection of a storey in a multi-storey building according to Eurocode 3 [1]. In the last column of the table, a comparison is made between the stiffness of the infilled frame structure and its bare frame by means of a stiffness factor $\alpha = k_{infilled_frame} / k_{bare_frame}$.

It can be seen in Table 1 that the observed lateral stiffness of the infilled frames ranges between 5.5 respectively 10.0 times the bare frame stiffness, depending on the size of window opening. Besides, all specimen types were able to support a lateral load of 650 kN or more.



Fig. 6: Failure of discrete steel-concrete connection by bolt shear through nut

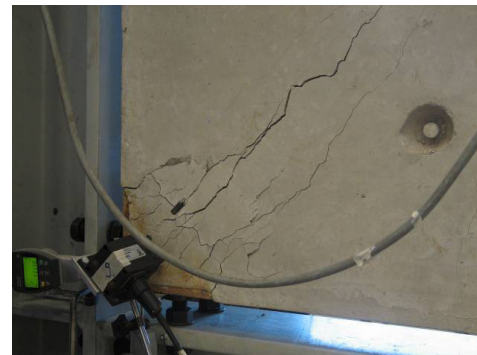


Fig. 7: Concrete crushing with spalling and yielding of wedge reinforcement

4. Finite element modelling

The foregoing experimental research is supplemented by finite element analyses. The finite element package used is DIANA, release 9.2 [2]. The aim of the numerical research is to develop and validate a finite element model that simulates the experimental infilled frame behaviour, taking into account non-linear material characteristics and geometrical non-linearity. The validated numerical model can then be used to carry out parametric studies to investigate other configurations of the hybrid lateral load resisting infilled frame.

4.1 Model design

A two-dimensional finite element model was created. The model can be divided in three groups, being panel, frame and discrete steel-concrete connections. For each group, the applied element types and material properties are discussed below.

4.1.1 Panel

The concrete panel is modelled with eight-node isoparametric plane stress elements (CQ16M, [2]) with a thickness of 200 mm. The longitudinal reinforcement and stirrups are modelled with reinforcement bars, embedded in the plane stress elements [2]. The technique of embedding allows the lines of the reinforcement to deviate from the lines of the mesh. The embedded reinforcements do not have degrees of freedom of their own. The reinforcement strains are computed from the displacement field of the plane stress elements, implying perfect bond between the reinforcement and the surrounding concrete.

4.1.2 Frame

The three-node, two-dimensional class-III beam element CL9BE [2] is used to model the frame members. These elements are based on the so-called Mindlin–Reissner theory which does take shear deformation into account. The sectional properties of the beam elements correspond to the sections used experimentally (HE180M). The beam-to-column connection is modelled by a rigid offset to take the column depth into account, and a two-node rotational spring element (SP2RO, [2]) representing the stiffness of this connection.

4.1.3 Discrete steel-concrete connection

The discrete steel-concrete connections are represented by two-node translation spring elements (SP2TR, [2]), and are only able to support axial compressive forces. An initial force in the springs is applied of 100 kN, representing the pretension resulting from the torque controlled tightening of the bolts. The magnitude of this prestress load was experimentally found.

4.2 Material properties

4.2.1 Panel

Prior to cracking, concrete can be modelled sufficiently accurately as isotropic, linear elastic [3]. The initial Young's modulus E_c was determined by performing standard material tests with concrete prisms ($100 \times 100 \times 500$ mm³), and can be found in Table 2. The Poisson's ratio of concrete under uni-axial compressive stress ranges from about 0.15 to 0.22, with a representative value of 0.19 or 0.20 [4]. For this study, a Poisson's ratio is adopted of $\nu = 0.2$. Standard material tests were performed with concrete 150 mm cubes to find the actual tensile strength f_{ct} and compressive strength f_c (Table 2).

The non-linear concrete material model combines the Mohr-Coulomb plasticity model for the compressive regime with a smeared cracking model for the tensile regime. For the behaviour of the concrete in compression the Mohr-Coulomb yield surface limits the elastic state of stress. The DIANA software evaluates the yield surface using the current state of stress, the angle of internal friction ϕ and the cohesion c . As per recommendation of the DIANA software manual [2], the angle of internal friction of the concrete can be approximated to be $\phi = 30^\circ$. The cohesion c can then be calculated as follows:

$$c = f_c(1 - \sin \phi) / 2 \cos \phi \quad (1)$$



For the smeared crack approach, a multi-directional fixed crack model is applied, in which typically the direction of the normal to the crack is fixed upon initiation of the crack. A linear stress cut-off criterion is applied, which means that a crack arises if the major principal stress exceeds the minimum of f_{ct} and $f_{ct} (1 + \sigma_{lateral}/f_c)$, with $\sigma_{lateral}$ being the lateral principal stress. Besides, a linear tension softening based on fracture energy G_f is adopted according to the CEB-FIP Model code [5] (for G_f , see Table 2).

Table 2: Material properties concrete

Test	E_c [N/mm ²]	f_c [N/mm ²]	f_{ct} [N/mm ²]	G_f [Nm/m ²]
1	3.54E+04	62.2	3.9	117
2	3.67E+04	64.4	3.9	120
3	3.66E+04	70.6	4.2	127
4	3.70E+04	75.6	4.4	133
5	3.72E+04	66.0	3.9	122

The Mode-I fracture energy is released in an element if the tensile strength is exceeded and the deformations localize in the element. With this approach the results obtained with the analysis are objective with regard to mesh refinement. Due to cracking of concrete, the shear stiffness is reduced, generally known as shear retention. A constant shear retention factor $\beta = 0.2$ is used, which is a commonly adopted value [3].

For the embedded reinforcement bars, a Young's modulus of $E_s = 2.0E+05$ N/mm² is assumed. The stress-strain curve of the reinforcement bars is assumed to be elastic-perfectly plastic, with yielding according to the Von-Mises criterion, with yield stress $\sigma_y = 560$ N/mm².

4.2.2 Frame

Steel with the elastic material properties Young's modulus $E_s = 2.1E+05$ N/mm² and Poisson's ratio $\nu = 0.3$, in combination with the Von Mises plasticity model ($\sigma_y = 235$ N/mm²) is used for the frame. The rotational springs representing the beam-to-column connections were calibrated on the results of the full-scale experiments with the bare frames.

4.2.3 Discrete steel-concrete connection

The discrete steel-concrete connections are modelled with non-linear spring elements, with no tension capacity. Input is a stiffness diagram (Fig. 8) which was obtained from preceding investigations into the structural behaviour of the steel-concrete connection [6].

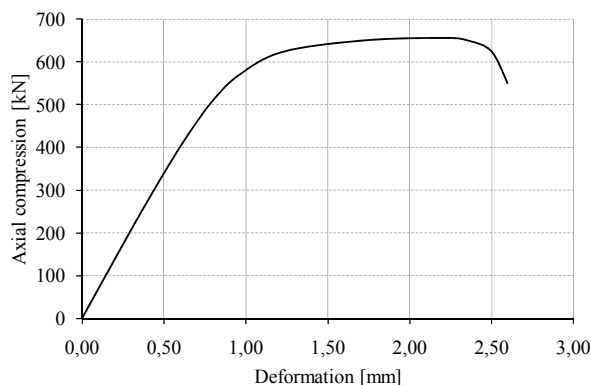


Fig. 8: Load-deformation curve translation springs

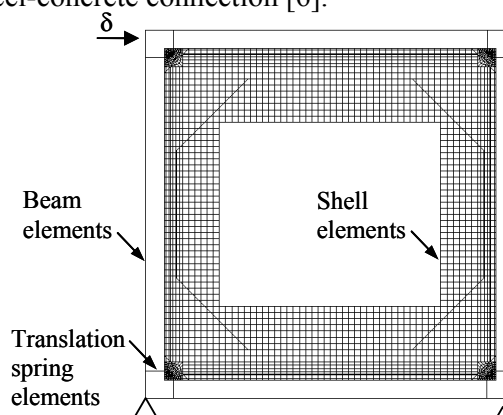


Fig. 9: Total finite element model

4.3 Boundary conditions and loading

In Fig. 9 the total finite element model (specimen 3) is shown. Support conditions matching the test-setup were used. The loads applied to the infilled frame include initial prestressing of the translation springs, representing the tightening of the steel-concrete connection (LC1), followed by horizontal loading at the left upper corner up to failure (LC2). A displacement controlled procedure was applied to impose the load up to failure. A regular Newton-Raphson iteration procedure was used to find the solution.

5. Results and discussion

The simulated and experimental global responses of all specimens are presented in Fig. 10. Comparison shows that the load-deflection behaviour obtained from the FE-analysis is quite similar to the experimental results. The initial higher stiffness is present, followed by the approximately linear branch and finally failure. The simulated failure modes of specimens 1 to 4 are connection failure, which are identical to the experimental failure.

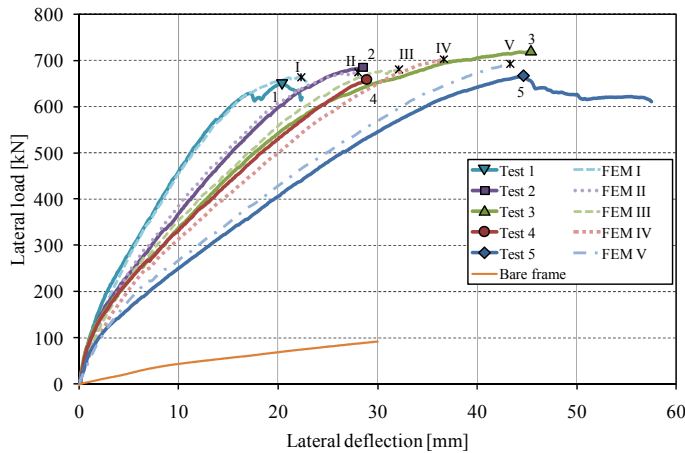


Fig. 10: Load-deflection response infilled frames

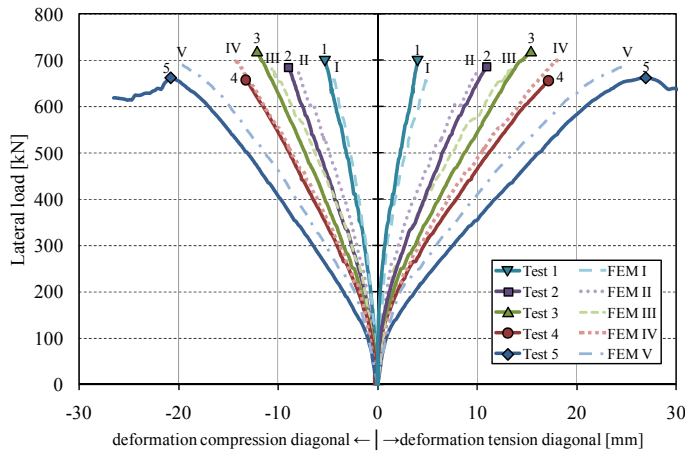


Fig. 11: Deformation panel diagonals

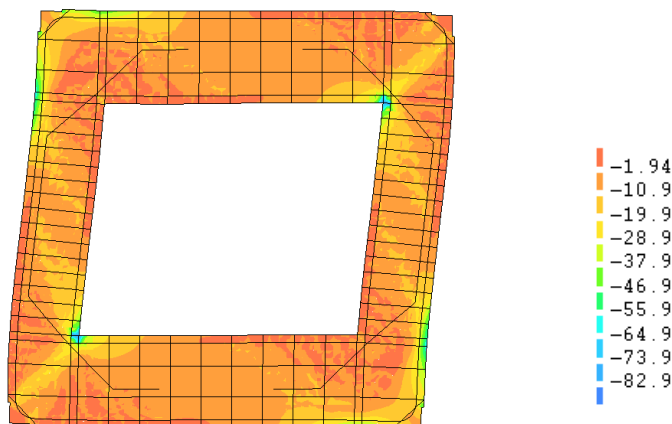


Fig. 12: Deformed panel with major principal stresses (specimen 3)

For test 5 the infill panel failed first with considerable deformations concentrated in open cracks, and yielding of the wedge reinforcement. Although the finite element simulation indicates panel failure with yielding of the wedge reinforcement, the smeared crack model is not suitable to adequately describe the post-peak behaviour. To get a more accurate post-peak prediction, the concrete model should be supplied with a discrete crack model. In this combined approach, the smeared crack model is used to model the distributed cracking and concrete crushing, while the discrete cracks are inserted into the areas, where major cracking can be expected based on the experimental observations. However, panel failure will be prevented in the design since the steel-concrete connection will be designed to govern the strength of the structure. Therefore, the smeared crack approach is satisfactory for design purposes.

Besides the global behaviour, also the predicted and observed local behaviour of the infill panel and steel-concrete connection was compared. Fig. 11 shows the predicted and observed panel deformation for all specimens, measured over the compression diagonal respectively the tension diagonal (Fig. 3: δ_{13} and δ_{14}). Comparison shows that behaviour obtained from the FE-analysis is quite similar to the experimental results.

To provide a better understanding of the stress distribution in the infill panel, the deformed shape of the panel of specimen 3, together with a contour plot of the major principal stresses at ultimate load is shown in Fig. 12. The figure shows that particularly two of the window corners are subject to large compressive stresses. Also in the area near the loaded corners of the panel large compressive stresses are present.

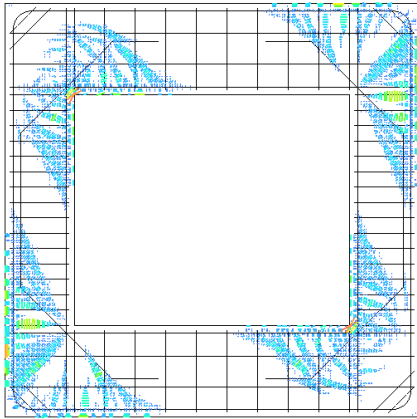


Fig. 13: Smeared crack pattern versus experimental crack pattern (specimen 3)

In Fig. 13 the smeared crack pattern of the concrete panel of specimen 3 at ultimate load is presented, together with the experimentally found crack pattern. It can be concluded that the numerical crack pattern matches the experimental observation qualitatively well. Also for the other four specimens, the FE-analyses showed that results of the local deformations and crack patterns matched well with the experimental observations.

The above presented comparisons between experimental observations and numerical predictions of both the global and local behaviour of the hybrid lateral load resisting infilled frame, indicate that the finite element model used in this study is adequate, and corresponding results are reliable. The numerical investigation will, therefore, be further extended to investigate other configurations of the hybrid lateral load resisting infilled frame.

6. Conclusions

The experiments show that discretely connected precast concrete panels with window openings can significantly improve the performance of steel frames. The observed lateral stiffness of the infilled frames ranges between 5.5 respectively 10.0 times the bare frame stiffness. All infilled frame structures were able to support a lateral load of 650 kN or more. For four panel geometries, the discrete connections were governing by design the strength of the structure while for the test with the largest panel opening the infill panel failed first.

Finite element simulations were performed taking into account non-linear material properties and geometrical non-linearity. A comparison between the full-scale experiments and simulations shows that the FE model presented in this paper is able to predict the lateral load versus deflection relationship of the hybrid lateral load resisting infilled frame, and the ultimate lateral load carrying capacity for all failure mechanisms. Therefore, the FE-study will be extended to study the infilled frame performance by varying different parameters.

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