EXPERIMENTAL INVESTIGATION INTO THE RESPONSE OF STEEL FRAMES INFILLED WITH CALCIUM SILICATE ELEMENT WALLS TO IN-PLANE LATERAL LOADS

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ABSTRACT

This paper describes and presents results of an experimental programme investigating the structural response of steel frames infilled with walls constructed of calcium silicate elements (CASIELs) in thin-layer mortar. This type of wall is increasingly employed in wall construction in Europe. Each of ten 3 *m* by 3 *m* steel infilled frames was subjected to an in-plane monotonic horizontal load at the roof beam level. The variables investigated were the presence of an initial gap below the roof beam, the frame-to-wall stiffness ratio and the influence of a top corner bearing wedge. Measurements included rigid body movements, gaps and slips at frame-wall interfaces as well as at selected joints, and strains at selected sites on the walls. Load-deformation curves show a three stage response prior to cracking. In general, there is an initial stiff stage before a transition stage during which frame-wall separation occurs. This is followed by another stiff linear load-deflection primary stiffness range leading to diagonal tension cracking. When shear cracking along the bed joint below the topmost CASIEL layer occurs, the infilled frames more or less instantly recover their stiffness. An initial top gap results in large deflections during the transition phase and a reduced primary stiffness although it does not reduce the cracking load. Increasing the frame-to-wall stiffness ratio increases the primary stiffness and the diagonal cracking load. By using a bearing wedge at the top corners, the influence of the top gap is significantly reduced. This may be significant in developing a construction technique for industrial application of infilled frames. The global responses, together with the strain distributions derived from rosette measurements on the walls provide a data base for calibration of a finite element model.

KEYWORDS: infilled frames, CASIELs, thin layer mortar, stiffness, bearing wedge

INTRODUCTION

Calcium Silicate Elements (CASIELs) are commonly used nowadays and provide a rapid labour-saving wall erection technique [1]. CASIEL walls are often placed in steel or reinforced concrete frames. The incidental interaction between these walls with the bounding frames produces infilled frame behaviour [2], [3]. Although infilled frame behaviour has been researched for over five decades, there is still little understood and much less applied in design. In addition, large-scale test programmes are very scanty [5]. CASIELs have not previously been studied for this application.

Theoretically, steel frames contribute ductility and infill walls contribute stiffness to infilled frames. The infill wall acts as a diagonal brace to the frame. The effectiveness of the diagonal brace depends upon the frame-to-wall stiffness ratio, the contact, bond and shear characteristics at the frame-wall interface and the strength of the infill under biaxial loading [5]. In this investigation large scale experiments are used to study the influence of (a) a structural configuration factor: the frame to wall stiffness ratio, (b) an interface detail factor: a gap below the roof beam, and (c) a novel construction technique: the use of a corner bearing wedge.

EXPERIMENTS

The objectives of the experiments were, firstly, to observe and measure the response of steel-CASIEL infilled frames to in-plane monotonic loading and, secondly, to observe and measure the influence of the aforementioned parameters on the response in terms of overall stiffness of infilled frames, stress/strain distributions in the structure, cracking loads and the pattern of damage in the structure.

A purpose designed and built reaction frame was used as a platform to mount and load the specimens. The reaction frame is composed of twin triangular frames, one on either side of the specimen, connected through rigid steel members at their vertices. The members of the twin triangular reaction frame were fabricated from European HE 300 B profiles. The test arrangement is shown in Figure 1. At the leeward support, the twin triangular frames were bolted to a heavy steel block. This block provided the specimen with restraint from horizontal and vertical outward displacement. At the windward side, two types of supports, shown in Figure 2, were used during the investigation. In the first type, used for the first four tests, a slender steel plate was bolted to the reaction frame at the bottom and the specimen above. In the second type, used in the last six tests, four steel rods were used to tie the reaction frame to the specimen. Both types of support were intended to provide high vertical restraint and as low a horizontal restraint as possible. In this way, for evaluation of the behaviour of the structure, the windward and leeward supports would be modelled as a roller and a pin support, respectively.

Each infilled frame was nominally 3000 *mm* by 3000 *mm*. Steel I-sections, with semi rigid bolted connections were used for the bounding frame. The infill walls were constructed from 897 *mm* x 594 *mm* x 150 *mm* CASIELs in thin-layer mortar. Five different types of specimen, in duplicate, were used. Table 1 shows the main characteristics of the specimens, namely: (a) strong or weak frames; (b) frames with or without gaps, and; (c) frames with corner bearing wedges. A specimen with corner bearing wedges is also shown in Figure 1.

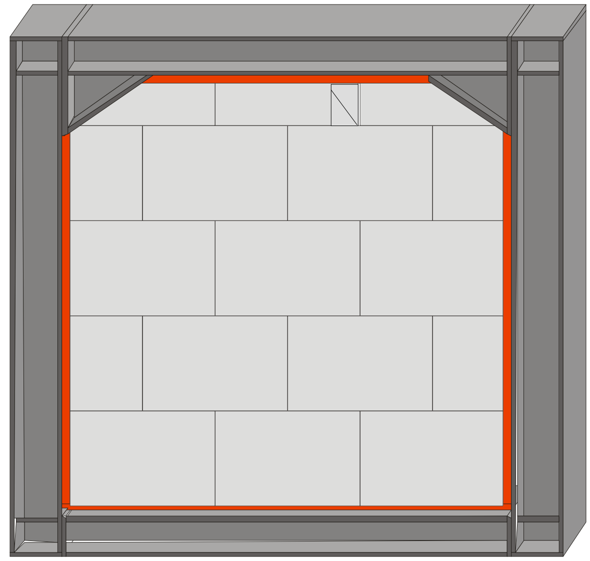
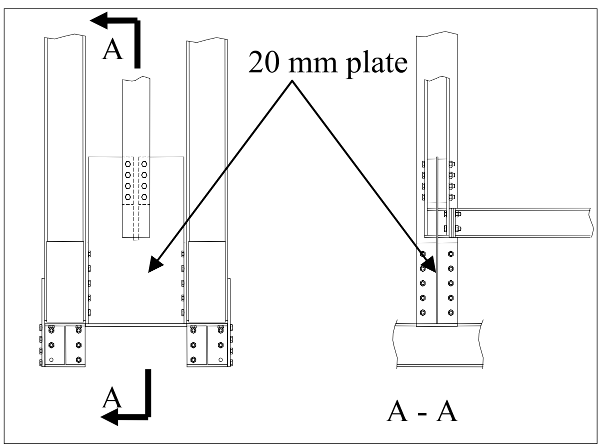


Figure 1 – Test arrangement with specimen mounted (left) and specimen with corner bearing wedges (right)



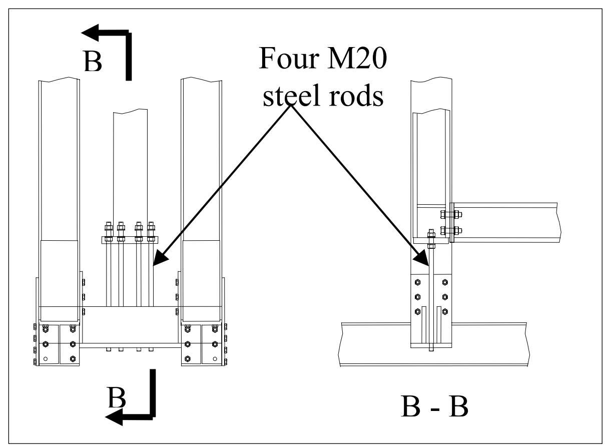


Figure 2 - Windward support, type 1 (left) and type 2(right)

Table 1 - Types of infilled frame specimens

Specimen Type	TEST	Beam Section	Column Section	Wall Thickness (*mm*)	Gap Below Roof Beam (*mm*)	Bearing Wedge
1	TEST 1 & TEST 2	HE200B	HE180B	150	Nil	Nil
2	TEST 3 & TEST 4	HE200B	HE180B	150	12	Nil
3	TEST 5 & TEST 6	HE240M	HE240M	150	Nil	Nil
4	TEST 7 & TEST 8	HE240M	HE240M	150	Nil	Present
5	TEST 9 & TEST 10	HE240M	HE240M	150	12	Present

Weak frames for Specimen Type 1 were constructed from HE 200 B sections ($I_x = 5696 \times 10^4$ mm^4) for beams and HE 180 B sections ($I_x = 3831 \times 10^4$ mm^4) for columns. For each connection, a 15 mm thick beam end plate was bolted with 4 M20 bolts to the column flange. The strong frames were constructed from HE 240 M ($I_x = 24290 \times 10^4$ mm^4) steel sections all round. Beam end plates of 30 *mm* thickness were bolted to stiffened column flanges. Back plates of 15 mm thickness were welded to the column flange. Tests of the stiffness of bare frames showed that the strong frames, at an average of 10.0 *kN/mm* were 3 times stiffer than the weak frames, at an average of 3.3 *kN/mm*.

When fitting an infill wall in a frame, tolerance gaps exist between the edges of the wall and the surrounding frame. Gaps may also be caused by shrinkage of the infill wall. In these experiments, weak frames with and without gaps between the top of the infill wall and the roof beam were compared. A comparison was also made between strong frames with corner bearing wedges with and without gaps. For Specimen Type 1, the 12 mm gap between the wall and the roof beam was packed with ordinary mortar while, for Specimen Type 2 an open gap was left. Observations of other researchers, which have now been corroborated by results from the current research, indicated that the presence of interface gaps reduces the stiffness of infilled frames during the early stages of loading. This is because of a delay in interlocking of the wall and frame, causing large deflections at this stage. Closing interface gaps by packing mortar, as was done in this research, is a slow process and does not guarantee consistent filling of the gap. In order to eliminate the negative influence of the top gap and at the same time to remove the necessity of filling it with mortar, a novel construction technique was investigated. The basic idea of the technique was to improve the contact between the frame and the wall at the frame corners. This was investigated by the use of Specimen Type 4 and Specimen Type 5. In Specimen Types 4 and 5, triangular corner bearing wedges, shown in Figure 1, were bolted to beam and column flanges at the top corners of the frames. The surfaces of the flanges were the bearing surfaces through which the load would be transmitted to the infill wall. The only difference between Specimen Type 4 and Specimen Type 5 was that the top gap was packed with mortar in the former and left open in the latter.

Figure 3 shows the arrangement of Linear Variable Displacement Transducers (LVDTs) and rosettes on the specimen. The position of the specimen in relation to the ground was measured by LVDTs as indicated by numbers 4, 5, 6, 7, 8, 9, 70, 71 at the corners of the specimen. These LVDTs were fixed to a separate measuring frame. In order to decipher the strain distribution in the wall, rosettes were placed on a 500 *mm* by 500 *mm* grid on the wall. The grid was arranged with a bias to cover the area along the compression diagonal since most of the deformations were expected to take place there. Gaps and slip at the frame-to-wall interface as well as across and along joints in the wall were measured by LVDTs at specified points. The applied force and displacement of the loading jack were also measured by LVDTs built into the load cell.

A deformation-controlled load was applied, at 1 *mm/min*, using the 2 *MN* hydraulic jack mounted at the roof beam level.

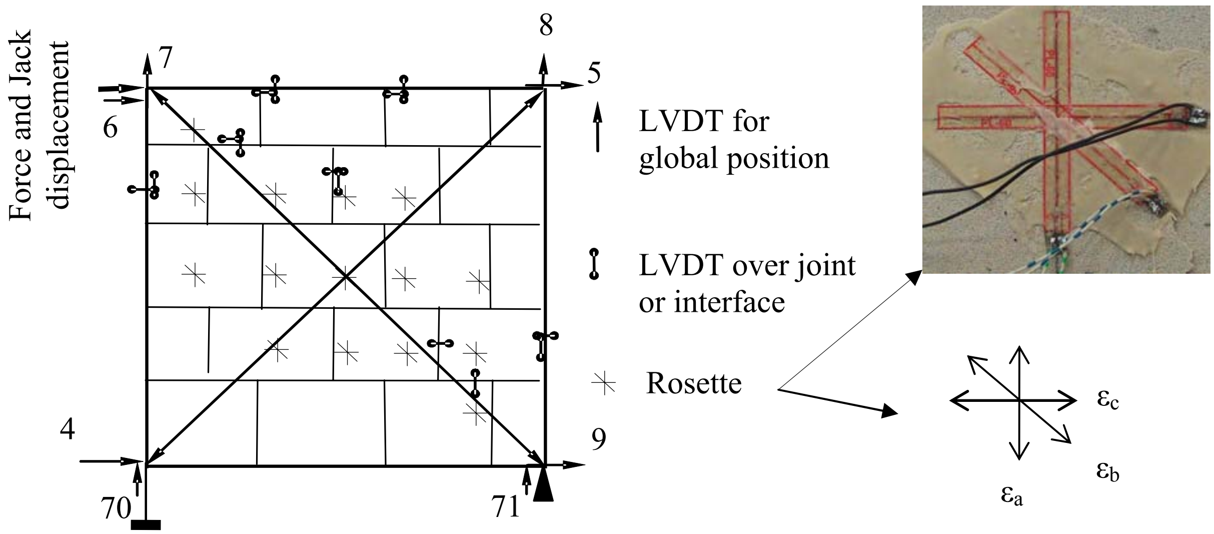


Figure 3 - Measurement scheme and rosette

RESULTS

From rosette measurements at different locations on the wall, principal stresses were estimated. For a 45^o rosette such as shown in Figure 3 it can be shown (4) that the principal stresses are given by Equation 1 and the direction of the principal planes can be determined by Equation 2.

$$\sigma_{1,2} = \frac{E}{1-\nu} \cdot \frac{\varepsilon_a + \varepsilon_c}{2} \pm \frac{E}{\sqrt{2}\,(1+\nu)} \cdot \sqrt{(\varepsilon_a - \varepsilon_b)^2 + (\varepsilon_c - \varepsilon_b)^2}$$

Equation 1

$$\tan 2\theta = \frac{2\varepsilon_b - \varepsilon_a - \varepsilon_c}{\varepsilon_c - \varepsilon_a}$$

Equation 2

where E and υ are the modulus of elasticity, and Poisson ratio respectively;
ε_a, ε_b and ε_c are the measured strains in the three known directions and
θ is the inclination angle of the principal stress to the direction of ε_a.

From auxiliary tests on the CASIELs, the average value of E was 6000 *N/mm*^{2}. Although tests indicated that the value of υ varies with the level of stress, an average value of 0.2 was used in estimations of principal stresses. Values and directions of principal stresses at different locations and different loading stages were calculated. As an example, principal stresses at various measuring points from TEST 6 are shown as Mohr's circles in Figure 4. Compressive direct stresses are shaded while the tensile stresses are unshaded. The directions of the major compressive principal stresses are indicated by arrow axes, with the magnitude indicated by the length of the arrow, at each Mohr's circle. The plots are for loads 50 *kN*, 200 *kN*, 300 *kN* and 380 *kN* respectively. TEST 6 cracked at 391 *kN*.

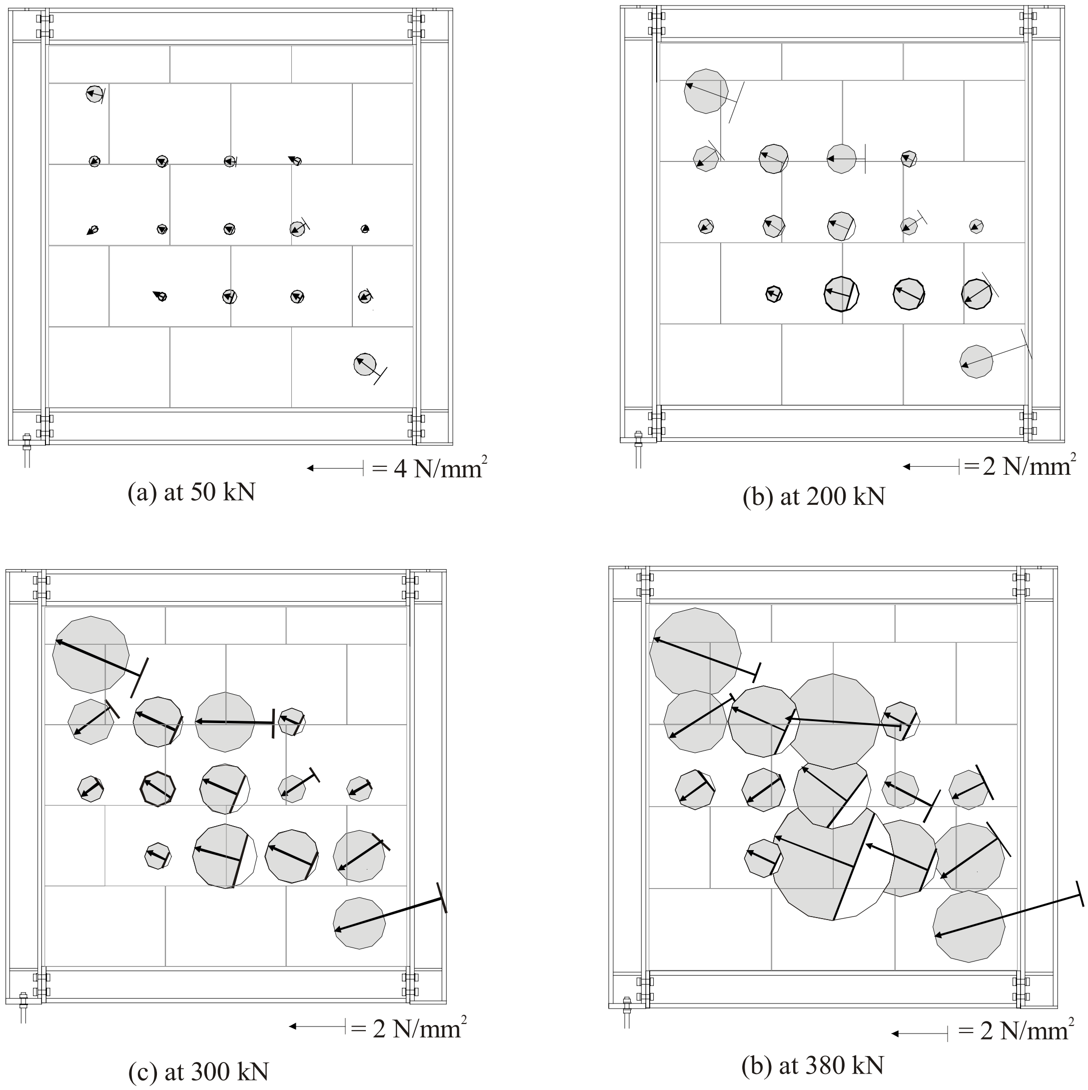


Figure 4 - Principal stress distributions for INFRA 6

The patterns of principal stress distribution show the following:

(a) The load is transmitted principally through diagonal strut action. This can be seen by the fact that the further the point is from the diagonal the smaller the diameter of Mohr's circle.
(b) The diagonal strut formation is more defined at higher loads than at lower loads. This is seen in the example of INFRA 6 by rather undifferentiated sizes of Mohr's circles at the low load of 50 *kN* compared to clear size distinctions at the higher loads of 300 and 380 *kN*.
(c) Compressive principal stresses are higher in the proximity of the loaded corners.
(d) Tensile principal stresses are higher in the central region of the wall.

Load deflection diagrams for all the tests are shown in Figures 5 – 7 with the key values of stiffness and cracking loads extracted to Table 2. Typically, each load deflection diagram shows a very high stiffness initially, followed by a transition when separation between wall and frame takes place at the tension corners. The transition is succeeded by a linear load-deflection response up to the development of a sudden major crack. Major cracking occurred predominantly due to diagonal tension, although in some cases shear sliding along the top-most bed joint was observed. Diagonal cracking was accompanied by as much as a 30% drop in the load. When shear sliding occurred, the stiffness of the infilled frame was almost instantly recovered. Subsequently, although already cracked and subjected to additional cracking, the infilled frames could bear higher loads.

In addition, from Figures 5 – 7 and Table 2, the following can be deduced:

Influence of gap: Comparing results of Specimen Type 1 to those of Specimen Type 2 reveals that a top gap:

(a) caused a temporary drop in the load when frame-wall separation occurred at the tension corners, as seen in Figure 5;
(b) caused large deflections in the transition stage. In this phase, the wall moved within the bounding frame and eventually locked with the frame at the compression loaded corners;
(c) caused a reduction in the primary stiffness;
(d) did not change the diagonal tension cracking load.

Influence of frame size: Comparing results of Specimen Type 1 with those of Specimen Type 3 reveals that increasing the size of the frame:

(a) smoothed the transition of stiffness when frame-wall separation occurred;
(b) increased the primary stiffness of the infilled frame and increased the major cracking load. Although this increase is expected by virtue of the higher load resisted by the stiffer frame, the increase is not necessarily directly proportional to the increase of the frame stiffness. For instance, at a deflection of 5 mm, the difference in loads resisted by the weak frame of stiffness 3 *kN/mm*, and strong frame of stiffness 10 *kN/mm* would be 35 *kN*. In Figure 6 comparing TEST 6 with TEST 2, the corresponding difference in the load resisted by the infilled frames is 170 *kN*.

Influence of corner bearing wedge: An increase in the load without appreciable deflection as seen in Figure 7 for TEST 9 and TEST 10 can be explained by slight movement of the wall to enable contact at the loaded corner which briefly reduces the stiffness of the infilled frame. However, the wall quickly locks up with the frame causing it to be very stiff until frame-wall separation starts causing the infilled frame to become less stiff again. A comparison of results between Specimen Type 4 and Specimen Type 5 shows that a corner bearing wedge:

(a) increases the load at which frame-wall separation occurs;
(b) increases the primary stiffness of the infilled frame. The introduction of the load into the infill panel is better, due to the better interface contact. Thus the earlier observed influence of a gap is essentially eliminated. (This assumes that the gap has the same influence in both weak and strong frames);
(c) does not alter the major cracking load.

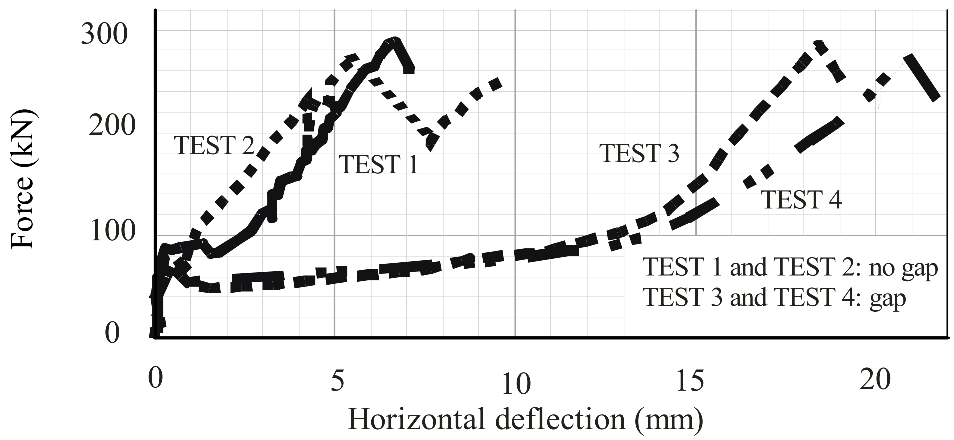


Figure 5 - Load deflection diagrams - Influence of gaps

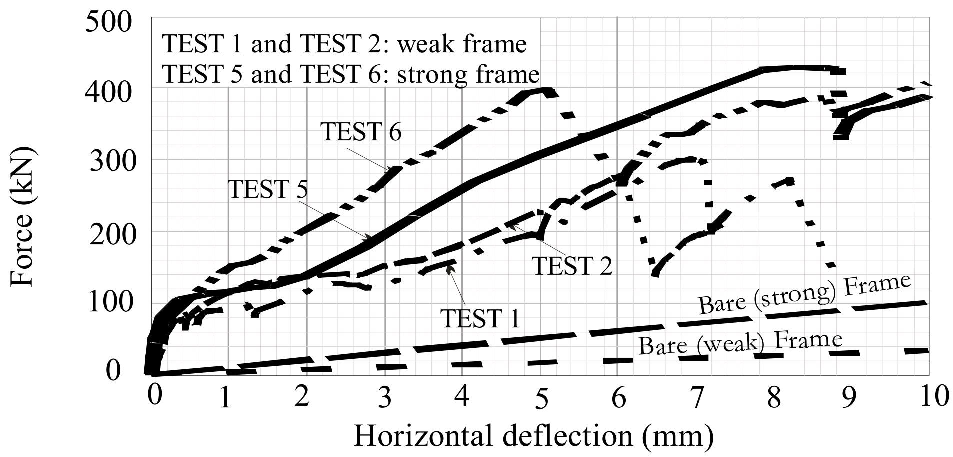


Figure 6 - Load deflection diagrams - Influence of frame size

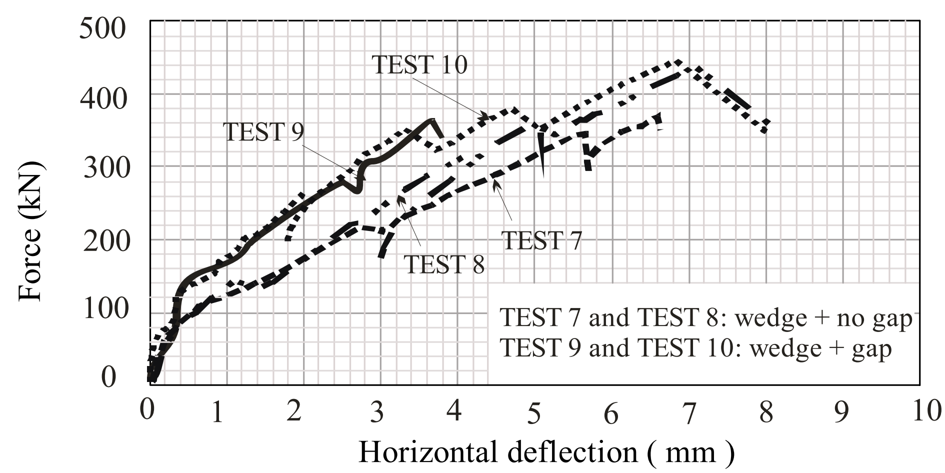


Figure 7 - Load deflection diagram; Influence of corner bearing wedge

Table 2 - Stiffness characteristics and cracking loads

Sp. type	CODE	Primary Stiffness (*kN/mm*)	Av. Primary Stiff. (*kN/mm*)	Shear Slip Load (*kN*)	Diagonal cracking load (*kN*)	Av. diag. cracking load (*kN*)
1	**INFRA 1**	42	41	**not observed**	293	284
	INFRA 2	39		**235**	275	
2	**INFRA 3**	29	33	**not observed**	270	278
	INFRA 4	36		**285**	285	
3	**INFRA 5**	54	59	**not observed**	420	405
	INFRA 6	63		**not observed**	390	
4	**INFRA 7**	50	57	**340**	365	398
	INFRA 8	63		**not observed**	430	
5	**INFRA 9**	74	75	**350**	370	365
	INFRA 10	75		**290**	360	

COMPARISON WITH ANALYTICAL MODELS

Several formulae for the equivalent diagonal strut width have been proposed in literature. Holmes [6] proposed that the width of the equivalent diagonal should be taken as a third of the diagonal length. Stafford-Smith [7] used the theory of a beam on an elastic foundation to derive expressions for the contact lengths at the frame-wall interfaces. Hendry [8] used these expressions as a basis for estimation of an equivalent strut width. Table 3 compares the stiffnesses of the infilled frames from elastic analyses using diagonal widths according to Holmes and Hendry, to primary stiffnesses obtained from experiments. Material values were obtained from auxiliary tests for CASIELs and from the literature for steel. Thus the elastic moduli for steel and CASIELs were taken as 205,000 N/mm^2 and 6,000 N/mm^2, respectively, while Poisson's ratios were taken as 0.3 and 0.2 respectively. Both theoretical methods overestimate the stiffness by more than double. It is thought that the much lower stiffness from the experiments is due to a stress distribution in the infill panel that leads to a much narrower equivalent width than that proposed in these models. The cause of the wide discrepancy still begs clarification.

Table 3 - Comparison of stiffness between analytical approaches and experiments

	Width of diagonal strut (*mm*)		Primary Stiffness (*kN/mm*)	
	Specimen Type 1	**Specimen Type 3**	**Specimen Type 1**	**Specimen Type 3**
Holmes [6]	1431	1431	124	124
Hendry [8] based on Stafford-Smith [7]	1071	1546	110	125
Experiment	-	-	41	59

CONCLUSIONS

Ten large-scale steel frames infilled with CASIEL walls were subjected to in-plane monotonic horizontal loads at roof beam level. The variables investigated were the presence of an initial gap below the roof beam, the frame-to-wall stiffness ratio and the influence of a top corner bearing wedge. Stress distributions derived from rosette measurements on the panel showed that the wall essentially acts as a bulging diagonal strut. Load-deformation curves show an initially high stiffness which transits into a less stiff linear primary stiffness. The deflection range in which the transition took place was longer for infilled frames with a top gap. During this transition, the wall separated from the frame at the two tension corners and adjusted within the frame until it was firmly locked up at the compression corners. In all specimens, major cracking occurred by sudden formation of two cracks cutting through the CASIELs oriented along the compression diagonal. Shear cracking along the bed joint below the topmost CASIEL layer was observed in some specimens, although when this happened, the frames almost instantly recovered their stiffness. An initial top gap resulted in long transition and a reduced primary stiffness although it did not reduce the cracking load. An increased frame-to-wall stiffness ratio increased the diagonal tension cracking load. By using a bearing wedge in the top corners, the influence of the top gap was significantly reduced. This may be significant in developing a construction technique for industrial application of infilled frames. Due to the limited number of tests conducted for each parameter, the conclusions drawn from these results need to be corroborated with more tests and or numerical analyses.

ACKNOWLEDGEMENTS

This research is supported by the Technology Foundation STW, applied science division of NWO and the technology programme of the Ministry of Economic Affairs. The support of the staff of the Pieter van Musschenbroek Laboratory at the Eindhoven University of Technology is gratefully acknowledged.

REFERENCES

1. Berkers, W.G.J. Building with calcium Silicate elements. Proceedings of the 4th International Masonry Conference, vol. 1 (7). 1995. pp 176-177.
2. Mainstone, R.J., & Weeks, G.A. The Influence of a Bounding Frame on Racking Stiffnesses and Strengths of Brick Walls, Proceedings of the 2nd International Brick Masonry Conference. Stoke-on-Trent, England. 1970. pp 165-171.
3. Ng’andu, B.M., Vermeltfoort A.T., Martens D.R.W. The response of steel frames infilled with CASIEL walls to in-plane monotonic loads. Proceedings of the 13th International Brick and Block Masonry Conference. Amsterdam, vol. 2. July 2004. pp 219 – 228.
4. Case J. & Chilver A.H. Strength of Materials and Structures. Edward Arnold. London, United Kingdom. 1971.
5. Dawe, J.L., and Seah, C.K. Behaviour of masonry infilled steel frames. Canadian Journal of Civil Engineering, 16(6), 1989. pp 865-876.
6. Holmes, M. Steel frames with brickwork and concrete infilling, Proc. I.C.E., Volume 19, 1961. pp 473-478.
7. Stafford-Smith, B. Lateral stiffness of infilled frames. ASCE Journal of the Structural Division. Volume 88 (ST6). December 1962. pp 183-199
8. Hendry A.W. Structural Masonry, Second edition. Macmillan. London, United Kingdom. 1998.