



THE RESPONSE OF STEEL FRAMES INFILLED WITH CASIEL WALLS TO IN-PLANE MONOTONIC LOADS

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Abstract

The influence of calcium silicate element (CASIEL) infills and the effect of an initial gap below the roof beam on the behaviour of steel frames were investigated. Four large-scale CASIEL infilled steel frames were subjected to monotonically increasing in-plane loads at roof beam level. Two of the frames had 12 mm gaps between the roof beam and the wall. The measured stiffness and strength of the walls are presented and compared with some theoretical analyses. Crack patterns are also presented. Behavioural differences from traditional masonry (brick) infills are highlighted. It is affirmed that contact or separation at the frame-wall interface plays a significant role in the behaviour of these infilled frames.

Key Words

Infilled frames, CASIELs, thin bed mortar, frame-wall interface.

1 Introduction

An increasingly popular method of wall erection is the use of calcium silicate elements (CASIELs) in thin bed mortar. CASIELs are large building 'stones', produced by mixing sand, lime and water, moulding and curing under pressurized steam. The term 'element' is used to distinguish them or their size from traditional units and blocks. The nominal dimensions of elements are 900 mm x 600 mm x various thicknesses (100 to 300 mm), and they weigh approximately 100 to 280 kg per piece. CASIEL walls differ from traditional brick walls in the sense that, in the former, joints are fewer and much thinner. These CASIEL walls are quite often built in between beams and columns. Unless purposely isolated from the frames that surround them, the CASIEL walls act compositely with these frames resulting in stiffer and stronger frame behaviour. This influence of CASIEL walls can, if not taken into consideration, lead to damages to finishes, cracking in the walls, and/or overstressing of certain sections of the columns. On the other hand, their strengthening and stiffening influence can be exploited to provide lateral stability to building structures.

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In order to evaluate the influence of CASIEL infills on the behaviour of infilled frames, four full size tests were carried out. Monotonic racking loads were applied at roof beam level. The influence of an initial gap between the roof beam and infill wall was investigated. Experimental results are compared with theoretical analyses proposed by other researchers. The behaviour of these frames is compared and contrasted with that of steel infilled frames with traditional masonry (brick) infills

2 Background

Interest in exploiting the in-plane stiffness of brickwork infills to resist blast loading, reportedly (Mainstone 1970), led to early exploratory tests in infilled frames in the 1950s. Further interest developed as Wood (1958) sought to use 'modern' frame design methods which involved no-sway on stability of tall building structures. Subsequent developments in understanding of the behaviour and analysis of infilled frames could, generally chronologically, be delineated through the following key stages:

- Development of Equivalent Diagonal Strut analogy (Polyakov 1960, 1963).
- Evaluation of the role of frame-infill interface conditions (Stafford-Smith 1960, 1966)
- Recognition of non-linear behaviour of interface stress displacement behaviour.
- Finite element modelling of infilled frames (Chrisafulli 2000).

Although a lot of research has been done on infilled frames, there has not yet been any research involving infill walls constructed from CASIELs. Most full-scale experiments have been on frames infilled with clay brick or concrete block masonry. The much larger size of CASIELs and much fewer and thinner joints distinguish these walls from traditional masonry, hence meriting a specific study into their behaviour when used as infill walls.

3 Description of Experiments

The experiments whose results are reported here are part of an ongoing experimental programme of ten full scale tests with accompanying 'shadow' tests of the materials used. As laid out in Table 1, the parameters being investigated are: (i) the influence of relative stiffness, denoted by λ , of the bounding frame to that of the CASIEL infill wall, (ii) the influence of an initial gap between the roof beam and the infill wall and (iii) the effect of a 'bearing wedge', at the top corners of the frame. Only details of the first four tests are presented in this paper.

Table 1: Overview of experimental programme

Test	Low λ	High λ	No gap	Top Gap	Bearing wedge	No bearing wedge
1 & 2	√		√			√
3 & 4	√			√		√
5 & 6		√		√		√
7 & 8		√		√	√	
9 & 10		√	√		√	

3.1 Test set-up

Two rigid triangular frames, connected to each other at the vertices, as shown in Figure 1, formed the reaction frame. European HE 300B profiles were used for the triangular frame members. A specimen was 'slotted' in the space between the triangular frames. A 2 MN loading jack mounted at roof beam level was used to apply a racking (horizontal) load. At the bottom of the leeward column the specimen rested on a block of steel – part of the reaction frame – providing restraint to outward movement in the horizontal and vertical directions. The bottom of the windward column was bolted through the flanges (see Figure 2) to a 20 mm thick steel plate, which was in turn fixed to the reaction frame. This connection was intended to provide maximum vertical restraint and minimal horizontal restraint, thus approximating a roller support.

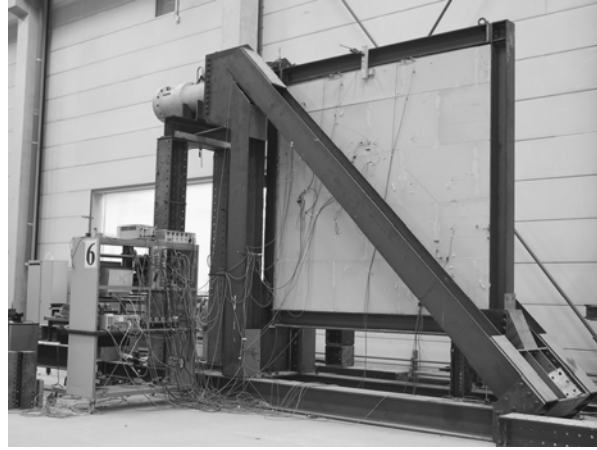


Figure 1 Test rig

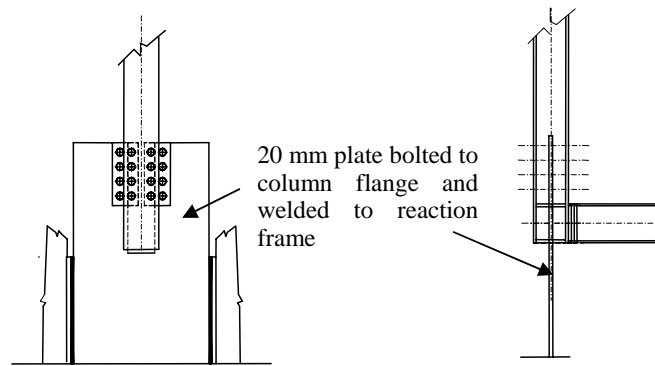


Figure 2 Connection of windward column to support plate.

3.2 Materials and specimen preparation

Preparation of a specimen involved the assembling of a bare frame, building of small specimens for material tests and of the wall into the frame and curing under normal laboratory conditions for three weeks.

Each frame was 3 m long and 3 m high. Columns were made out of European HE 180 B profiles while beams were from HE 200 B profiles. At each connection 4 bolts of size M24 and grade 10.9 were used. The bare frame was hoisted into the experimental rig and its stiffness, within the elastic range, measured by loading it up to a horizontal racking force of 20 kN. The frame was then taken out of the rig and placed vertical on the floor.

A mason from a factory that produces CASIELS was hired to erect the wall in a manner as similar to site practice as possible. The CASIELS used were 150 mm thick. Thin bed mortar was applied on the flange surfaces of the leeward column and floor beam. Starting with this corner, CASIELS were hoisted, with a crane, into position and laid up until the windward column. Using a pointing tool to pack ordinary mortar, the 20 mm gap between the wall and windward column was filled as completely as possible. Subsequent layers of CASIELS were laid in a similar manner. Two of the infilled frames had 12 mm initial gaps between the roof beam and the wall while the other two had that gap packed with ordinary mortar.

Samples for compressive, tensile, and shear testing of mortar and joints were made at the same time as the wall was constructed. Figures 3 and 4 show the test arrangements used for joint shear tests and four point bending tests.

The specimen and samples were then left undisturbed until the day of testing.



Figure 3 Shear test set-up



Figure 4 Four point bending test set-up

3.3 Measurement scheme

Figure 5 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 (with numbers indicated) on the four corners of the specimen. Gaps and slip at the frame-wall interface as well as across and along joints, in the wall were measured at specified points. Rosettes were placed on a 500 mm x 500 mm grid in order to measure strains at different points on the wall. Measurements of the displacements of the corners of the infilled frame and the strains (not discussed herein) at various locations on the wall were recorded every three seconds.

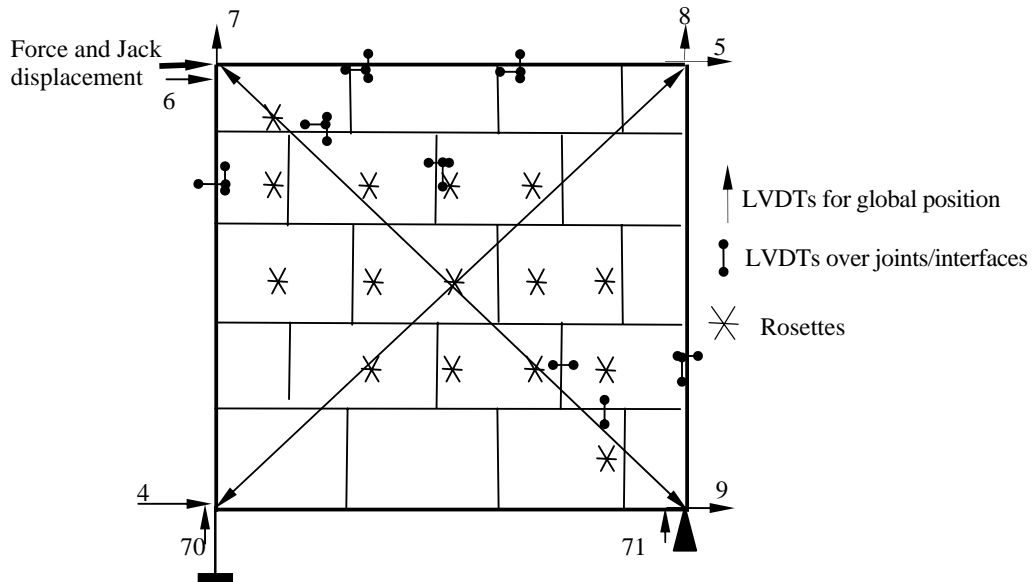


Figure 5 Measurement scheme.

3.4 Testing procedure

Load application, at roof beam level, was displacement controlled, at 1 mm/minute. At the beginning of each test the specimen was loaded up to 20 kN and unloaded. This was done in order to verify that the measuring system was working properly, and to close up initial contact tolerances between the frame and the support rig. The load was then increased until maximum load, typically accompanied with cracking, had been attained.

4 RESULTS

4.1 Strength characteristics of wall materials

Tables 2 to 5 show values of the wall material characteristics. These values were obtained from compressive, bending and shear tests on CASIEL prisms, mortar prisms and couplets.

Table 2 Compressive strength of CASIEL prisms

Frame	Test	Comp. strength (N/mm ²)	Mean Comp. strength (N/mm ²)
1	1	15.89	14.09
	2	14.01	
	3	12.36	
2	1	16.38	14.55
	2	14.60	
	3	12.67	
3	1	*	14.20
	2	15.02	
	3	13.38	
4	1	17.23	15.00
	2	14.81	
	3	12.98	

*Result discarded

Table 3 Compressive and tensile strength of mortar

Frame	Compressive strength (N/mm ²)		Tensile strength (N/mm ²)	
	Thin bed mortar	General purpose Mortar	Thin bed mortar	General purpose Mortar
1	12.41	-	5.51	2.85**
2	13.13	4.70	4.83	1.63
3	15.42	7.30	3.70	1.67
4	15.68	7.93	5.01	2.09

** A different type of mortar was used

Table 4 Shear bond strength

Precompression (N/mm ²)	Shear strength (N/mm ²)			Mean Shear Strength (N/mm ²)	Shear values
Infilled Frame					
	2	3	4		
0.20	0.56	0.74	0.79	0.78	Cohesion = 0.68 N/mm ²
	0.87	0.76	0.93		
0.60	1.02	1.06	1.12	1.00	Shear Angle =0.53
	0.88	0.89	1.05		
1.20	1.14	1.26	1.24	1.20	
	1.20	1.19	1.18		

Table 5 'Tensile' bond strength from 4 point bending test

Frame	Test	'Tensile' strength (N/mm ²)	Mean 'Tensile' strength (N/mm ²)
1	1	0.83	0.80
	2	0.83	
	3	0.75	
2	1	0.75	0.76
	2	0.67	
	3	0.89	
3	1	1.12	0.91
	2	0.87	
	3	0.75	
4	1	0.61	0.76
	2	0.86	
	3	0.82	

4.2 Load deflection behaviour

The behaviour of all infilled frame specimens was generally similar and is described with the aid of the load deflection curves in Figure 6. Prior to maximum load, three stages can be identified in the response of the specimens to loading. There was an initial, stiff stage which was succeeded by an intermediate, less stiff, stage, followed by a (final) stiff stage. Table 6 shows the stiffnesses, for the linear parts of the graphs, and maximum loads for the four infilled frames. The end of the initial range was accompanied by a bang sound and the appearance of cracks along the frame-wall interface near the tension corners. In INFRA 1 and INFRA 2 (see Figure 6), these initial cracks propagated from the tension corners along both the column and beam-wall interfaces. In infilled frames with a gap at the top, i.e. INFRA 3 and INFRA 4, the crack propagated from the bottom tension corner along the beam-wall interface. During the intermediate range, the wall adjusted its position within the bounding frame. This adjustment was associated with a displacement range in which the infilled frame was relatively flexible.

The infilled frames with gaps at the top were significantly more flexible than the others in this range. Once the wall had locked up with the frame at the compression corners, the final stage started. In some cases, during this stage, horizontal cracks along the bed joints appeared. This stage ended with a loud bang and a long diagonal crack.

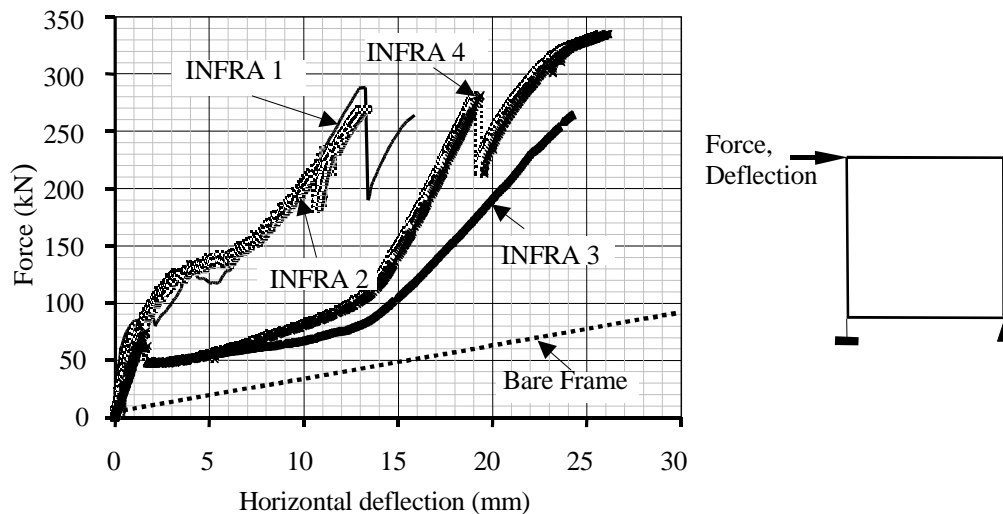


Figure 6 Load displacement diagrams.

Table 6 Summary of test results.

Infilled Frame	Stiffness of bare frame (kN/mm)	Age of wall (days)	Initial stage stiffness (kN/mm)	Intermediate stage stiffness (kN/mm)	Final stage Stiffness (kN/mm)	Ultimate load (kN)
INFRA 1	2.7	22	111	*	28	292
INFRA 2	3.8	32	101	*	19	272
INFRA 3	3.7	19	52	3.2	17	270
INFRA 4	3.3	33	74	5.5	30	337

*Load-displacement curve is not linear.

4.3 Failure mode and crack pattern

Failure was taken as attainment of maximum load which, in three cases, coincided with diagonal cracking. In INFRA 1 and INFRA 4, horizontal cracks along joints appeared, resulting in temporal load drops. In these cases, especially for INFRA 4, the crack below the first layer of CASIELs was particularly large. For INFRA 1, maximum load coincided with horizontal joint cracking below the first layer of CASIELs and was followed by a diagonal crack at a lower load. The first cracks were at frame-wall interfaces. In INFRA 2, some crushing of CASIELs occurred at the loaded corner. The specimens could then take on higher loads until failure. At failure, sudden diagonal cracks, generally parallel to the compression diagonal, appeared. These cracks passed through CASIELs as well as joints between the elements. Figures 7 and 8 show pictures of two of the cracked infill walls.

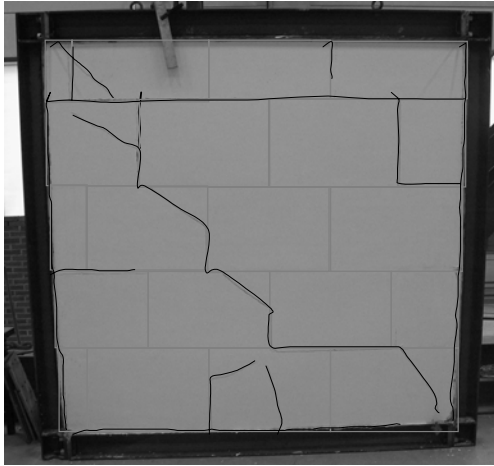


Figure 7 Crack pattern for INFRA 1

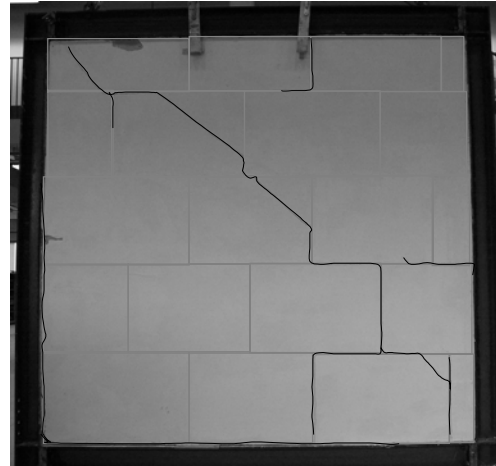


Figure 8 Crack pattern for INFRA 3

4.4 Discussion

Differences in the stiffnesses of the bare frames can be attributed to variations in tightening of bolts at the connections. Although efforts were made to tie the bolts in a uniform way, it was difficult to ensure and assume that this could be achieved. The manner of tightening the bolts was improved after the first test, hence, the lower stiffness in case of the first bare frame. These differences were, however, considered to have had little effect on the overall behaviour of the infilled frames.

The stiffening influence of the CASIEL infill walls on frames is self-evident. As can be seen in Table 6 the bare frames had stiffnesses in the order of 3.5 kN/mm while the infilled frames had initial stiffnesses ranging from 52 to 110 kN/mm and final stiffnesses of 17 to 30 kN/mm. Final stage stiffnesses were much lower than initial stage stiffnesses because of the reduction in contact lengths at frame-wall interfaces and increasing shear deformations in the joints.

The influence of a gap at the top of the wall is equally self-evident. INFRA 3 and INFRA 4 were less stiff than INFRA 1 and INFRA 2. Due to the gap at the top, it took a larger horizontal displacement of the load point to lift up the wall on the windward side so that it could lock up with the frame. Once the wall locked up with the frame, it behaved like infilled frames without initial gaps.

The ultimate failure loads were similar for all the infilled frames. INFRA 4 behaved somewhat differently in the final stages. At a load of 285 kN a large crack opened up below the top CASIEL layer. This layer then began to slide over the joint pushing against the leeward column. The infilled frame could then take on a higher load up to a diagonal crack failure at 337 kN.

4.5 Comparison with theory

The most common analytical approach to the behaviour of infilled frames is the equivalent strut diagonal. In this model, the infill wall is substituted with a compression diagonal and the infilled frame analysed with classical methods. Different values and expressions have been proposed for the geometric properties of the diagonal strut. Holmes (1961) proposed a width of one third of the diagonal length for the equivalent diagonal strut. By drawing free body diagrams of the frame and the strut and equating the resulting diagonal deformations he proposed the expression in Equation (1) for the horizontal failure load.

$$H = \frac{24E.I_c.\varepsilon_w.d}{h^3(1 + \frac{I_c}{I_b}\cot\theta)\cos\theta} + \frac{1}{3}d.t.f_c.\cos\theta \quad (1)$$

where E = Young's modulus for frame material,
 I_c = second moment of area of the column,
 I_b = second moment of area of the beam,
 ε_w = strain in the infill at failure,
 d = diagonal length,
 t = wall thickness,
 θ = $\tan^{-1} h/l$,
 h = height of the infilled frame,
 l = length of the infilled frame.
 f_c = the compressive strength of the infill.

Stafford-Smith proposed contact lengths, α_h and α_l along the column and beam respectively. Expressions for α_h and α_l are given by Equations (2) and (3). In these equations the factor λ , defined in Equation (4), expresses the relative stiffness between the wall and the frame. Using these contact lengths, Hendry (1998) proposed an equivalent strut width as given in Equation (5). Assuming different failure modes in the infill – thus joint shear failure, diagonal tensile cracking or corner crushing and their associated values of f_c , the compressive strength - the load causing failure in the equivalent strut can be calculated and added to the resistance of the bare frame to give the total horizontal failure load.

$$\alpha_h = \frac{\pi}{2\lambda_h} \quad (2)$$

$$\alpha_l = \frac{\pi}{\lambda_l} \quad (3)$$

$$\lambda.l = l.4\sqrt{\frac{E_w.t}{4E_f.I_f.l}} \quad (4)$$

$$w = \frac{1}{2}\sqrt{\alpha_l^2 + \alpha_h^2} \quad (5)$$

where: E_w = Young's modulus for the wall material,
 E_f = Young's modulus of frame material,
 I_f = second moment of area of the column, for Equation (2), or beam for Equation (3),
 t = wall thickness,
 l = length of the column, for Equation (2), or beam, for Equation (3),
 l' = length of wall side.

Although collapse theories have been proposed, they have not been used to estimate failure for these infilled frames. This is because the plastic hinge mechanisms assumed in these methods were not observed in these tests.

Ultimate loads using the Holmes and Stafford-Smith approaches were estimated. The following values were used in calculations: $E_f = 210000 \text{ N/mm}^2$, $\epsilon_w = 0.6 \text{ mm/m}$ (obtained from strain measured near the loaded corners on the walls, at failure), $f_c = 8 \text{ N/mm}^2$ (estimated), and $E_w = 8000 \text{ N/mm}^2$. The predicted ultimate loads were 1731 kN and 475 kN for the two methods respectively. For horizontal shear cracking in the joints, Stafford-Smith's method estimates a horizontal load of 270 kN. Holmes method overestimates the experimental ultimate load of average 293 kN by more than 5 times. This discrepancy is due to an overestimation of the width of the compression strut. In addition, this method assumes crushing failure in the infill whereas diagonal tension was the primary failure mode in the test. Stafford-Smith's prediction is more realistic. It still overestimates the strength though, probably because the assumed value of ultimate compressive strength of 8 N/mm^2 is too high.

By using the same material characteristic values, and using a strut width according to Stafford-Smith, an elastic analysis of the frame yields a horizontal stiffness of 110 kN/mm. This compares well with the experimental values for infilled frames 1 and 2. This suggests that the diagonal strut approach is a reliable beginning point in analysing these infilled frames.

4.6 Comparison with traditional masonry infills

Dawe (1989) conducted similar experiments on steel frames infilled with 200 mm x 200 mm x 400 mm concrete block masonry. He made observations that are very similar to the ones herein reported, except in two respects. Firstly, in Dawe's experiments, the intermediate relatively flexible stage (refer to section 4.2 above) was not observed. Secondly, there were a greater number of cracks observed in Dawe's experiments. These differences can probably be attributed to the greater number of joints in the walls used in Dawe's experiments. As such, in Dawe's case initial cracks are more local than global in effect. In this sense, on a global scale, it can be said that the behaviour of the CASIEL infills tended to be more brittle than traditional masonry.

5 Conclusions

In order to investigate the behaviour of CASIEL infilled steel frames, and the influence of a gap between the roof beam and the wall, four large-scale specimens were tested. The results were compared with some theoretical analysis and traditional masonry infilled steel frames.

As noted by other researchers, infill walls tremendously increase the stiffness of frames. The presence of top gaps resulted in lower stiffness of the infilled frames as compared to those with mortar packed gaps. Changes in the contact conditions at the frame-wall interface had a remarkable influence on the overall behaviour. CASIEL infills behaved similar to traditional masonry infills, although, the former had fewer cracks and more brittle behaviour.

The diagonal strut approach gives a good approximation of the initial stiffness of the infilled frames. The onset of cracks however changes the configuration of the structure. Holmes' proposed method overestimated the strength of the infilled frame while Stafford-Smith's approach yielded predictions that were close to experimental results. It is apparent that contact and separation at the interfaces play a key role in the overall behaviour of these types of infilled frames. Tests with different relative frame to wall stiffnesses and different frame-wall interface details are ongoing.

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