

AN EXPERIMENTAL STUDY OF THE COMPOSITE BEHAVIOUR OF MASONRY GEOMETRIC SECTIONS

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Abstract

The Vertical shear capacity of masonry geometric sections is an important parameter which is required for the design of a robust and seismic resistant masonry building. In order to evaluate the parameters that influence the vertical shear resistance of flanged sections of masonry, a series of specimens with I-shaped cross sections incorporating varying techniques to achieve monolithic structural action were tested (monolithic behaviour was achieved by using either masonry header units, or some form of steel shear connector). The shear capacity was assessed in each case and compared to predicted code capacities. Considerable strength reserves and shear ductility was observed in almost all cases.

Key Words

Vertical shear, flanged sections, bond pattern, numerical modelling

1 Introduction

Theoretical and experimental studies of brick masonry wall strengths generally have concentrated on the consideration of rectangular cross-sections. However, many walls in practical situations are stiffened by piers, returns or other flanged sections in order to increase their lateral resistance. To achieve effective composite behaviour for these types of walls a monolithic structural connection is required across the vertical joint between the two masonry components. The supporting flanges are usually linked to the web by masonry bond in the form of header courses or by metal shear connectors embedded in the bed joints across the shear plane with the vertical joint at the interface filled with mortar. This latter technique is useful where there is a difference in the bonding pattern on each side of a wall or where different materials are used side by side. Connectors are also useful when a damp-proof membrane is incorporated at the vertical joint between the ribs and the outer leaf of a wall to prevent the passage of moisture in highly absorbent units.

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The failure of walls made up of composite geometric sections can be either due to the failure of one section (i.e. flange or web), the failure of their connections or a composite failure as presented in Figure 1. This study focuses on flange-web interface behaviour (Figure 1 (d)).

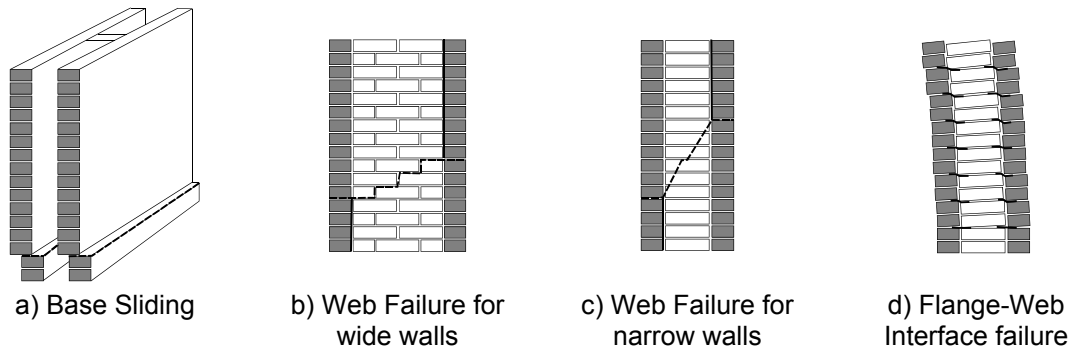


Figure 1 Modes of shear failure (after Simundic, 1997)

1.1 Previous research and code provisions

Only a limited amount of research has been carried out in this area. As a consequence, code design rules vary considerably from country to country and reflect the limited knowledge available. In-depth studies of this phenomenon have been carried out in relation to diaphragm wall behaviour by Phipps in the United Kingdom (Phipps, 1987), and this work was then extended in Australia by Phipps and Page (Phipps & Page, 1995a; Phipps & Page 1995b).

This general lack of knowledge has resulted in variable and conservative design provisions and excessively conservative strength predictions are thus obtained when these codes are used in the design of load bearing structures. One of the complicating factors in developing harmonized design provisions is the widely varying nature of wall types, construction practices and detailing in various countries.

In the latest draft of the new European Standard for masonry structures (prEN 1996, 2001) the vertical shear capacity of the connection between two adjacent walls is only very briefly discussed. This contrasts with the Australian Standard for masonry structures (AS3700 – 2001) which has quite specific (but probably conservative) provisions as outlined below:

a. For bonded sections with the web fully engaged in the flange:

The shear strength of the interface (f_{ms}^1) is taken as the lesser of $1.2 r_h$ MPa or 0.60 MPa (where r_h = the proportion of the shear plane intersected by the masonry units)

The header must be placed at least in every fourth course of the interface, and the shear stress at the interface can either be calculated using elastic theory (resulting in a parabolic stress distribution), or the complementary shear from an assumed uniform distribution can be used.

b. For shear connectors in the bed joints:

The connectors are designed to transfer all of the shear stress between the flanges and web in a mechanism involving the formation of two plastic hinges in the connector, with the distance between the hinges approximately equal to the thickness of the header joint. The shear capacity of an individual connector of a rectangular cross section is then given by:

$$V_c = \frac{0.75 f_{sy} \cdot r \cdot u}{12}$$

and for a circular cross section as:

$$V_c = \frac{f_{sy} \cdot d^2}{24}$$

Where, f_{sy} = characteristic yield strength of the steel connector, and r , u and d are the width, thickness and diameter of the connector (in mm) respectively.

2. Research Program

The overall research program had an experimental and a numerical component. The experimental component involved a study of the flange-web shear behaviour of a number of geometric sections with a range of bonding characteristics between the flange and web. The numerical component of the study involved the application of a simplified micro-modelling technique in modelling the above tests. Following the calibration and verification of the model, it was then used to analyse the significance of the parameters that might influence the shear behaviour of the specimens. This paper presents the results of the experimental study. The analytical study is reported elsewhere (Bosiljkov et al, 2004).

2.1 Description of the tests

Ten-course high “I” section specimens were used for all the shear tests (see Figure 2). The variables that were investigated were:

- Type of masonry bond (with a header in every 2nd, 3rd or 4th course across the shear plane). According to current Australian code provisions (AS 3700-2001), headers should be included in at least every fourth course of masonry or if the header spacing is variable, with the equivalent number of header units distributed uniformly throughout the interface area.
- Steel shear connectors (of rectangular cross-section) in every course across the shear plane.
- Masonry ties across the shear plane at a maximum spacing of 400mm (in accordance with AS 3700-2001) (these ties would normally be standard wire ties of medium or heavy duty).

A summary of the variables investigated is given in Table 1.

Table 1 Variables Investigated
(see also Figure 3)

Variable	Number Investigated	Comments
Clay Unit	1	Extruded unit (78x116x238 mm)
Mortar	1	Cement : Lime : Sand = 1:1:6 (volume ratio)
Masonry bond (SM2, SM3 and SM4)	3	Masonry header unit crossing the shear plane at every 2 nd , 3 rd and 4 th course (3 replicates for each series).
Shear connectors (SCT)	1	SC – rectangular sections 20mm x 30mm, 120mm long (3 replicates)
Wall ties (SMT)	1	WT – wire ties, 3.85 mm diameter, 2 wires per interface (6 replicates)
Type of loading	1	Monotonic
Number of specimens	18	Shear tests of flanged sections
Number of specimens	10	Compressive tests of masonry – standard test
Number of tests	50	Bond Wrench tests – standard test
Number of specimens	10	Splitting tensile tests of units – standard test
Number of specimens	10	Compressive tests of units – standard test

All the tests were carried out in the Civil, Surveying and Environmental Laboratory at the University of Newcastle. All the specimens were prepared by a professional mason. During the bricklaying retempering of the mortar was allowed. All the specimens were cured in laboratory conditions and were at least 30 days old before testing. The test set-up for the shear tests of the flanged sections is show in Figure 2.

A schematic arrangement of the 5 specimen types is show in Figure 3. In parallel with the 18 flanged section tests, a series of standard tests on the brick masonry and its components was carried out to establish its critical strength parameters for use in the analytical part of the investigation. These consisted of compression and indirect tension (splitting) tests on the units (10 tests in each case); masonry compressive strength (10 tests on prisms); and flexural tensile strength (bond wrench tests on 50 joints).

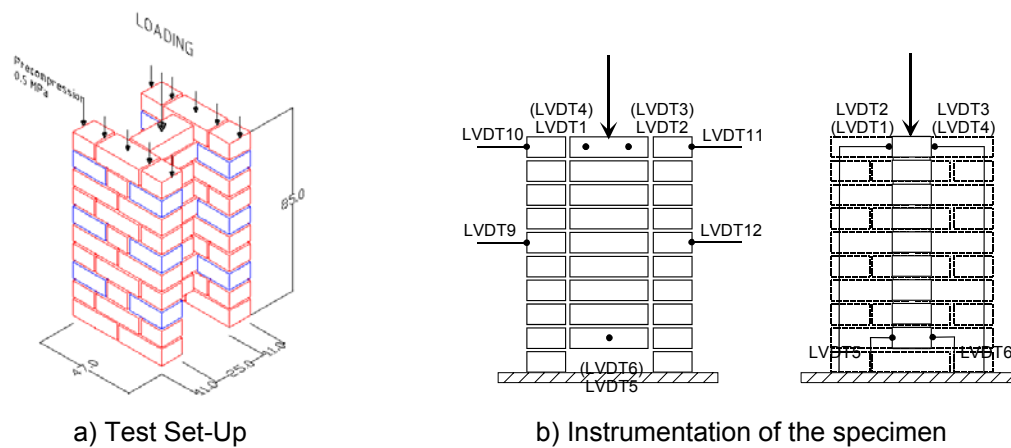


Figure 2 Test Set-Up and instrumentation of the specimens

Prior to applying a shear load to the I sections, a small precompressive stress of 0.5 MPa was applied to each flange to stabilise the specimen and simulate a typical level of pre-load. A compressive load was then applied to the web of the section to produce shear in the flange-web interface. The load was applied monotonically to failure at a rate of 0.1 mm/min for the bonded specimens, and 0.5mm/min for the shear connector and wall tie specimens. The relative vertical displacements between the web and the flanges (due to shear deformations along the flange-web interface), as well as any dilatency of the specimen were measured by LVDT's placed at the locations show in Figure 2(b).

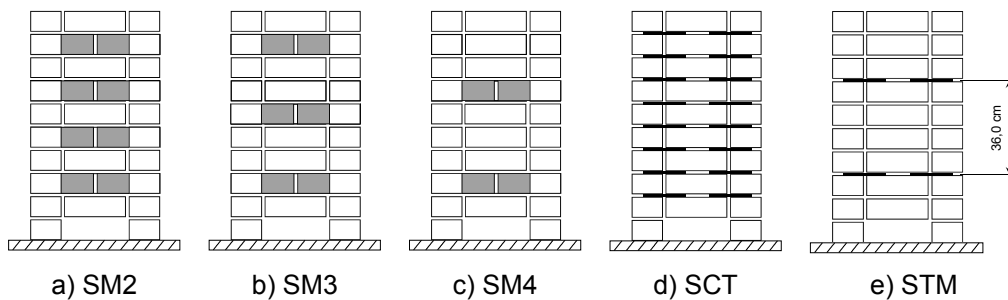


Figure 3 Type of specimens depending from the bonding

3 Results

The results of the standard tests on the masonry and its components are given in Table 2, and confirm that the masonry used was “typical” for Australian practice.

Table 2 Results of tests of the masonry and its constituents

Type of test	No. of spec.	Mean value [MPa]	C.O.V. [%]
Compressive test of the unit	10	29,86	12
Splitting tensile test of the unit	10	2,48	21
Bond Wrench test	10	0,93	19
Compressive test on 4-high masonry prism	5	8,09	16

Typical results of shear tests on the I-section specimens for different types of bonding are shown in Figure 4. Plots of applied load versus the differential average displacements between the web and the flanges (obtained from measuring devices LVDT1-4) are shown.

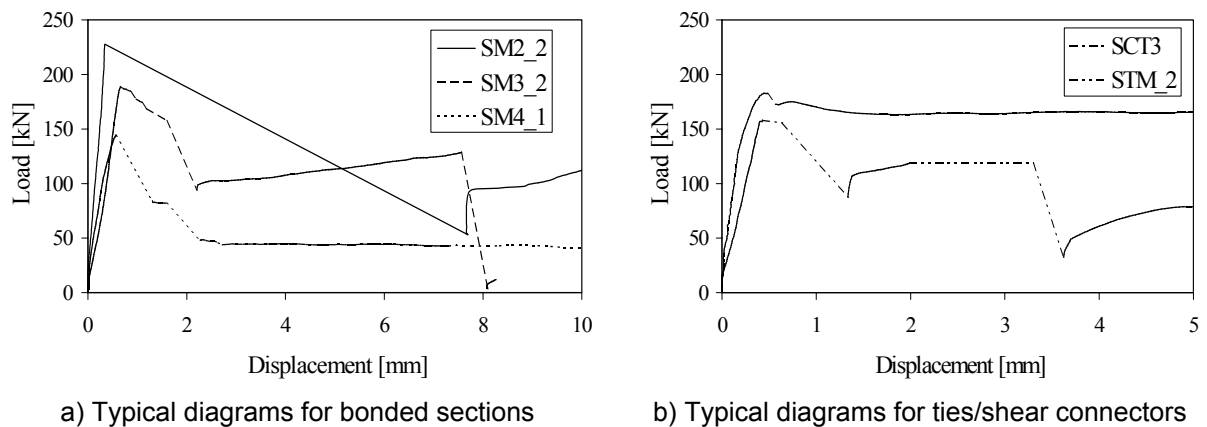


Figure 4 Load – relative web displacement for different bonding arrangements

The average values for each series are presented in the following table:

Table 3 Comparison between average experimental values

Series	F_{max} [kN]	d_{Fmax} [mm]	K [kN/mm]	F_u [kN]	d_u [mm]	μ
SCT	183.89	0.45	734.89*	137.92	10.01	22.23
(C.O.V.)	21%	15%	-	21%	-	21%
SM2	215.34	0.89	373.86	161.50	2.27	2.55
(C.O.V.)	-	-	-	-	-	-
SM3	183.64	0.50	356.90	137.73	1.36	2.69
(C.O.V.)	10%	11%	17%	10%	16%	10%
SM4	126.99	0.40	320.17	95.24	1.07	2.67
(C.O.V.)	22%	8%	39%	22%	32%	22%
STM	138.60	0.67	310.88	103.95	2.75	4.09
(C.O.V.)	30%	16%	22%	30%	70%	30%

* excluding values for SCT1

Where:

F_{max} =is maximum force;

d_{Fmax} =displacement at F_{max} ;

F_{max} ; K=effective shear stiffness calculated as $K=(F_{60\%} - F_{30\%})/(d_{60\%} - d_{30\%})$;

F_u =attained 75% of F_{max} in softening region;

d_u =displacement corresponding to F_u ;

μ = ductility defined as $\mu = d_u / d_{Fmax}$.

From the average experimental results shown in Table 3, it can be seen that the highest

shear interface). Reducing the bonding to every third or fourth course reduced the shear capacity by 15% and 41% respectively. The absolute values of shear capacity will be influenced by the strength of the brick in the bonding course. However, the relative strengths for the different bonding cases would be expected to be similar regardless of brick strength.

For the specimens with shear connectors (SCT) and wall ties (STM), mean shear strengths of 183.89 KN and 138.60 KN were obtained. The higher capacity for the section with a large number of connectors with rectangular cross-section is not surprising, but the wall tie specimens performed remarkably well considering the ties were located at the maximum code spacing. The capacity of these specimens (STM) was also 8% higher than that of the specimens with every fourth course bonded (the maximum spacing permitted by the Australian Code). The use of wire cavity ties as connectors therefore appears to be quite effective.

The “stiffest” section (in shear terms) before shear failure along the interface (“K” in Table 3) was clearly that with shear connectors in every course, with the stiffness being almost twice as high as any other detail. The “ductility” obtained for the wall tie specimens (SMT) or shear connectors (SCT) was also consistently higher than for specimens with conventional bonding using masonry units (SM2-4). However, for the SCT case, it should be noted that the failure mechanism of the SCT series involved vertical cracking in the flange normal to the shear plane in line with the connectors. This was different to the other bonding types and strongly influenced by the deformation of the shear connectors at the web-flange interface. For all other types, failure occurred as a typical shear crack at the web-flange interface (see Figure 5).

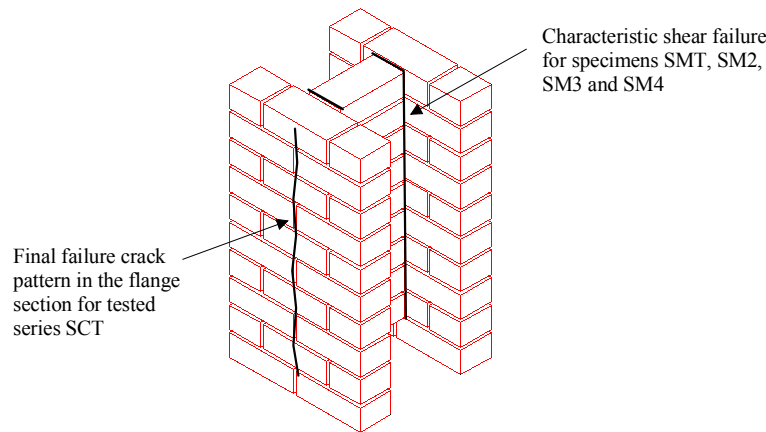


Figure 5. Damage propagation and failure mechanism

Comparison with Australian Code Provisions

The experimentally obtained shear strengths are compared with the Australian code provisions in Table 4.

Table 4 Comparison of average values with Australian code provisions

Series	$f_{ms \text{ exp}}$ [MPa]	f_{ms} [MPa]	Ratio $f_{ms \text{ exp}} / f_{ms}$
SCT	1.09	0.09	11.5
SM2	1.29	0.53	2.4
SM3	1.09	0.40	2.7
SM4	0.76	0.27	2.8
STM	0.82	0.01	105.6

Where, $f_{ms \text{ exp}}$ =experimentally gained shear strength, calculated as $f_{ms \text{ exp}} = F_{max} / (2A_{sh})$ and f_{ms} =shear strength according to provisions of Australian code

As can be seen from the Table 4, the Australian code provisions are conservative in all cases, and particularly conservative for the shear connector and wall tie cases. For the masonry bonded specimens the margin of safety appears reasonable and corresponds well with the results of Kok (1984) and Kua (1985).

The margin of safety for the steel connector specimens and particularly the wall tie specimens are much higher. The SCT results for the 10-course high specimens correspond well with the results of Simundic (1997), derived from 6-course high specimens with the same shear connectors placed in every course. The conservative nature of these results indicates that further research in this area is warranted to more carefully assess the current code provisions.

5. Summary and Conclusions

From the relatively limited number of tests performed, the following conclusions can be drawn:

- The highest shear capacity was obtained by masonry bonding across the shear interface at every second course. This capacity was reduced by 15% and 41% if the bonding occurred at every third or fourth course respectively.
- The shear capacities exhibited by the specimens with shear connectors in every course or with wire ties in every fourth course were lower but still significant. This type of detail has practical advantages as there are no constraints on the layout or the bonding pattern of the flanges of the cross-section.
- Reasonable (and conservative) agreement was obtained between the Australian code predictions and the observed capacity of the masonry bonded sections. The Code provisions significantly underestimated the capacity of the shear connector and wall tie specimens, with the wall tie case being particularly conservative. More research is required to clarify this situation.
- Before shear failure of the interface occurred, the shear connector specimens exhibited a shear stiffness twice that of all the other specimen types. The ductility after failure of both the shear connector and wall tie cases were both significantly greater than the ductility of the masonry bonded specimens.
- This study has provided direct input into a parallel analytical study which models the tests and critically assesses the influence of the various parameters. (Bosiljkov et al, 2004).

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