EXPERIMENTAL ANALYSIS OF THE SHEAR CAPACITY OF INTERCONNECTED BLOCKWORK WALLS

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ABSTRACT
The shear strength of the vertical interfaces is a key parameter to guarantee both the flange contribution to a shear wall and the possibility of shear force transference between interconnected walls subjected to vertical loading. In this regard, this paper presents the results of an experimental investigation to study the behavior of vertical interfaces of interconnected concrete and clay blockwork masonry walls. Four series of tests in H-shaped specimens were conducted to determine the bonding pattern effect (running bond and U-steel anchor) on the shear strength of the web-flange connection. Face shell bedding was used to construct the specimens. The specimens were vertically loaded on the web with a uniformly distributed pressure. Strains, displacements, cracks and the ultimate load were analysed and the shear strength was compared with experimental results obtained by other researchers and with values reported in NBR 15812-1 (2010), NBR 15961-1 (2011), AS 3700 (2002) and CSA S304.1 (2004). The experimental results showed that the type of connection can significantly influence the shear strength of vertical interfaces and that there are characteristic values which are larger than those currently adopted by several codes.

KEYWORDS: masonry, concrete block, clay block, shear strength

INTRODUCTION
When masonry blockwork walls are connected and subjected to different loads, there is interaction between them. The interaction is dependent on the shear capacity of the interface. In practical situations, many walls are stiffened by flanged sections in order to increase their lateral resistance. To guarantee the flange contribution to the shear wall and the possibility of shear force transference between interconnected walls, the shear strength of the vertical interfaces should be ensured. Tests on H-shaped walls have revealed that more studies and improvement are needed on the topic of shear strength in interconnected masonry walls.

The type of connection between the walls influences the way that interaction occurs. The supporting flanges are usually linked to the web through the running bond. In that case, interaction might be higher because the interface plane crosses units which may be considered common to the connected walls. Alternatively, the connection between the walls can be through horizontal steel bars or trusses, or U-steel bars anchored into grout in block holes and embedded in the bed joints and extended across the shear plane with the vertical joint filled with mortar at the interface.

Masonry walls with flanges have already been studied by some researchers. Lissel et al. [1] tested shear walls with flanges and observed the influence of the type of connection on their behavior and strength. The tests indicated that bonding pattern of the web-flange connection affect the shear strength of masonry. Bonding the web-flange connection with brickwork clearly increases the shear strength of this joint. Hence, a series of specimens with H-shaped cross sections was tested in an attempt to determine the effect of bond pattern on the strength of the web-flange intersection. According to Lissel et al. [1], the results of these tests indicate that the mechanical interlock of a bonded web-flange connection provides a significant structural advantage over a tied connection. The forces applied on the specimens with bonded wall are three times the forces applied on specimens with tied connection. Camacho et al. [2] tested H-shaped walls made of clay blocks in small scale to determine the shear strength of interfaces. Silva [3] repeated the tests carried out by Camacho in full scale specimens. Capuzzo Neto et al. [4] conducted new studies, contributing towards the understanding of the wall interaction phenomena. The author sought a better representation of possible stress trajectories along the building structure, including proposing an H-shaped specimen to evaluate the shear strength of the vertical interface. These specimens were used in this work to obtain the shear strength through experimental tests. Moreira [5] carried out a comparative analysis of three types of connections between structural masonry walls under vertical loads. The test specimens were H-shaped third scale walls with three types of connection: running bond interconnected masonry walls, connection obtained by means of steel trusses and U-steel bars anchored in grouted holes. Bosiljkov et al. [6] analyzed the significance of some parameters that influence the vertical shear strength of interfaces between flanged masonry walls made of clay bricks, using different types of bonding patterns.

Other researchers also have developed studies in this area, including Simundic [7], Yoshimura et al. [8]; Modena et al. [9]; Mauríc io [10]; Drysdale et al. [11]. However, there is still a lack of knowledge of vertical shear capacity. According to Bosiljkov et al. [6], two of the complicating factors in developing harmonized design provisions are the widely varying nature of wall types and construction practices and detailing in various countries. As a consequence, code design rules vary considerably from country to country and reflect the limited knowledge available.

EXPERIMENTAL DETAILS

For the physical and mechanical characterization tests, concrete and clay hollow blocks were used for the construction of the masonry specimens. The concrete and clay hollow blocks had a percentage of holes equal to 45 and 62, respectively.

The mortar utilized for bed joints was a mixture of cement, hydrated lime and sand. The thickness of the bed joints was approximately 10 mm. Face shell bedding was used on the construction of the specimens.

Uniaxial compression tests for each material were carried out on 12 blocks, 12 mortar specimens, 12 prisms and 6 wallettes, according to EN 1052-1 [12], NBR 13279 [13], NBR 15812-2 [14], NBR 15961-2 [15]. Tests on 12 units were carried out to determine the splitting tensile strength of masonry units according to ASTM C1006 [16]. Table 1 summarises the characteristics of the materials used in the research and the results of the tests on masonry specimens.

Table 1: Preliminary Test Results

<table>
<tr><td colspan="10">Physical and mechanical characteristics</td></tr>
<tr><td rowspan="4">Blocks</td><td rowspan="2">Material</td><td rowspan="2">Dimension (mm)</td><td rowspan="2">Weight (kg)</td><td colspan="3">Compressive Strength (N/mm^2)</td><td colspan="2" rowspan="2">Young’s Modulus (N/mm^2)</td><td rowspan="2">Splitting tensile strength (N/mm^2)</td></tr>
<tr><td>Average</td><td colspan="2">Characteristic</td></tr>
<tr><td>Concrete</td><td>390x190x140</td><td>12.70</td><td>10.21</td><td colspan="2">8.68</td><td colspan="2">9910</td><td>0.91</td></tr>
<tr><td>Clay</td><td>290x190x140</td><td>5.43</td><td>13.07</td><td colspan="2">9.44</td><td colspan="2">4282</td><td>2.11</td></tr>
<tr><td rowspan="2">Mortar</td><td colspan="2">Composition (C:L:S) in volume</td><td colspan="2">Composition (C:L:S) in weight</td><td colspan="3">Average comp. strength (N/mm^2)</td><td colspan="2">Young’s Modulus (N/mm^2)</td></tr>
<tr><td colspan="2">1:1:6</td><td colspan="2">1 : 0.66 : 8.21</td><td colspan="3">3.52</td><td colspan="2">6800</td></tr>
<tr><td rowspan="4">Prisms</td><td rowspan="2">Material</td><td colspan="2" rowspan="2">Dimension (mm)</td><td colspan="4">Compressive Strength (N/mm^2)</td><td colspan="2" rowspan="2">Young’s Modulus (N/mm^2)</td></tr>
<tr><td colspan="2">Average</td><td colspan="2">Characteristic</td></tr>
<tr><td>Concrete</td><td colspan="2">390x590x140</td><td colspan="2">5.16</td><td colspan="2">4.00</td><td colspan="2">8171</td></tr>
<tr><td>Clay</td><td colspan="2">290x590x140</td><td colspan="2">3.04</td><td colspan="2">2.40</td><td colspan="2">3153</td></tr>
<tr><td rowspan="4">Wallettes</td><td rowspan="2">Material</td><td colspan="2" rowspan="2">Dimension (mm)</td><td colspan="4">Compressive Strength (N/mm^2)</td><td colspan="2" rowspan="2">Young’s Modulus (N/mm^2)</td></tr>
<tr><td colspan="2">Average</td><td colspan="2">Characteristic</td></tr>
<tr><td>Concrete</td><td colspan="2">790x990x140</td><td colspan="2">4.88</td><td colspan="2">4.13</td><td colspan="2">7900</td></tr>
<tr><td>Clay</td><td colspan="2">590x990x140</td><td colspan="2">2.90</td><td colspan="2">2.61</td><td colspan="2">3084</td></tr>
</table>

Figure 1 and Figure 2 show the co mpressive strength tests and spli tting tensile strength tests of concrete and clay blocks. Figure 3 shows the compressive strength tests of prisms and wallettes.

a) Concrete units in compressive strength test b) Clay units in compressive strength test

Figure 1 – Test setup of the compressive strength tests of units.

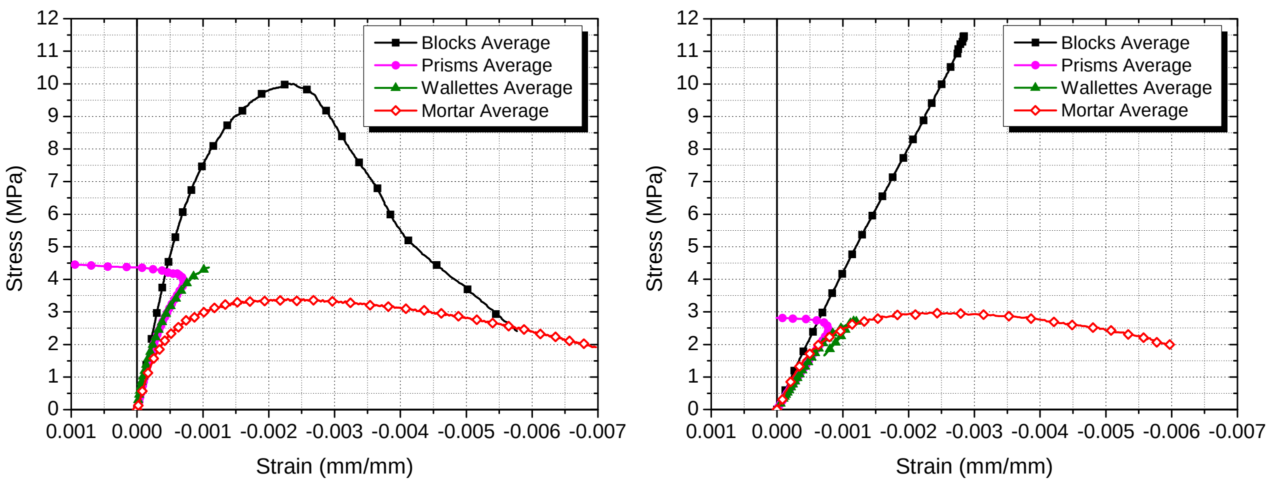
a) Concrete units in splitting tensile strength test. b) Clay units in splitting tensile strength test.

Figure 2 – Splitting tensile strength tests of concrete and clay units.


a) Concrete prism b) Clay prism c) Concrete wallete d) Clay wallete

Figure 3 – Test setup of the compressive strength tests of prisms and walletes.

Figure 4 shows the stress-strain diagram results of all compressive tests. In all cases, a brittle failure of the specimens was observed, as expected.



a) Blocks, prisms and walletes of concrete, and mortar used in this specimens

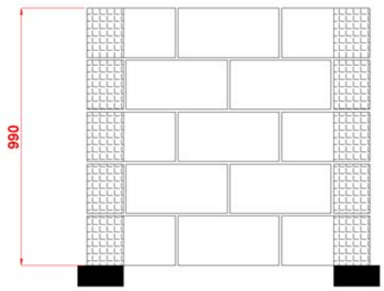
b) Blocks, prisms and walletes of clay, and mortar used in this specimens

Figure 4 – Stress-strain diagrams of the specimens in compressive strength tests.

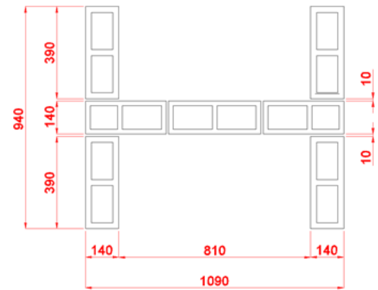
Four series of six specimens with H-shaped cross section with five courses were tested to determine the shear strength of the web-flange intersection. The specimens were prepared by a professional mason and were cured in laboratory conditions for 28 days before testing. They were constructed using the same blocks and mortar as the preliminary tests. All specimens were constructed with face shell mortar bedding.

The first and the second series were constructed with running bond in the flanges and concrete and clay blocks were used, respectively. The third and fourth series were constructed using clay and concrete blocks, respectively, and two U- steel anchors of 10 mm diameter per flange, placing one every two courses. There was a continuous vertical mortar joint in the tied wall. Figure 5 summarises the types of bonding which were investigated and all the features of the tested specimens.

a) Interlocking concrete blocks
(Series I)

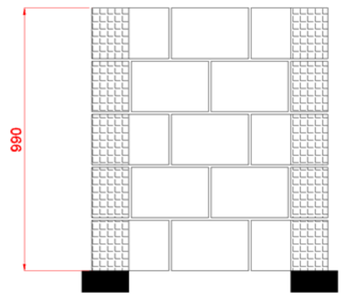


b) Interlocking concrete blocks
(Front view)

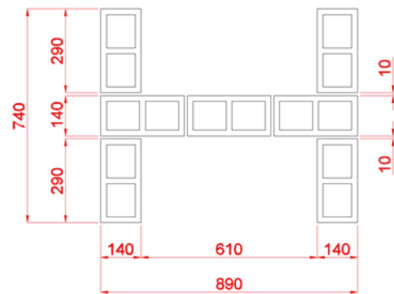


c) Interlocking concrete blocks
(Top view)

d) Interlocking clay blocks
(Series II)

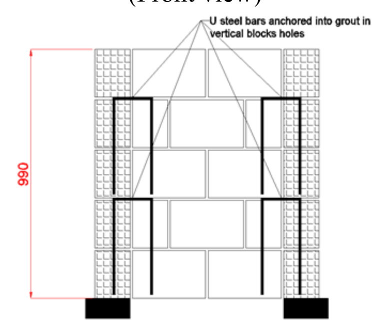


e) Interlocking clay blocks
(Front view)

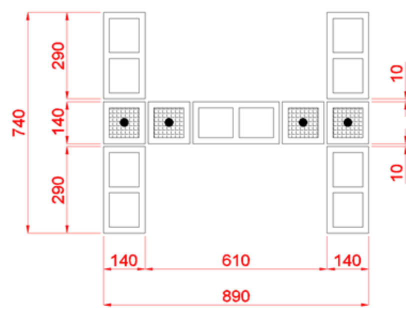


f) Interlocking clay blocks
(Top view)

g) U-steel anchor (clay blocks)
(Series III)

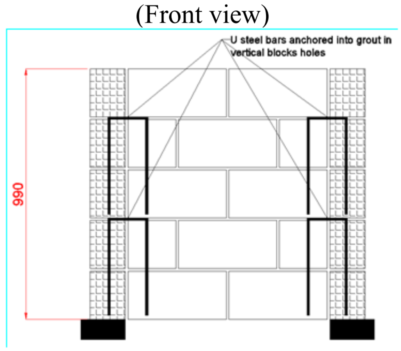


h) U-steel anchor (clay blocks)
(Front view)

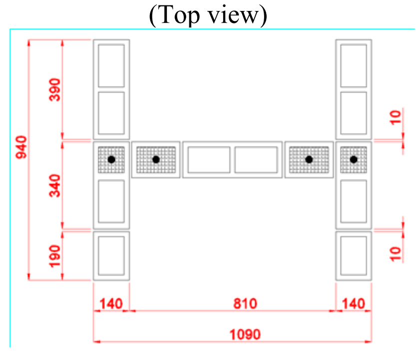


i) U-steel anchor (clay blocks)
(Top view)

j) U-steel anchor (concrete blocks)
(Series IV)



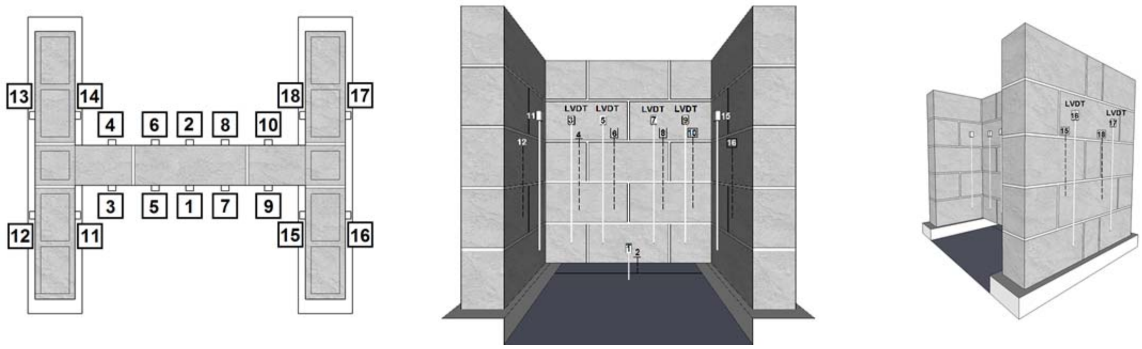
k) U-steel anchor (concrete blocks)
(Front view)



l) U-steel anchor (concrete blocks)
(Top view)

Figure 5 – Geometric specification of the specimens (dimension: mm).

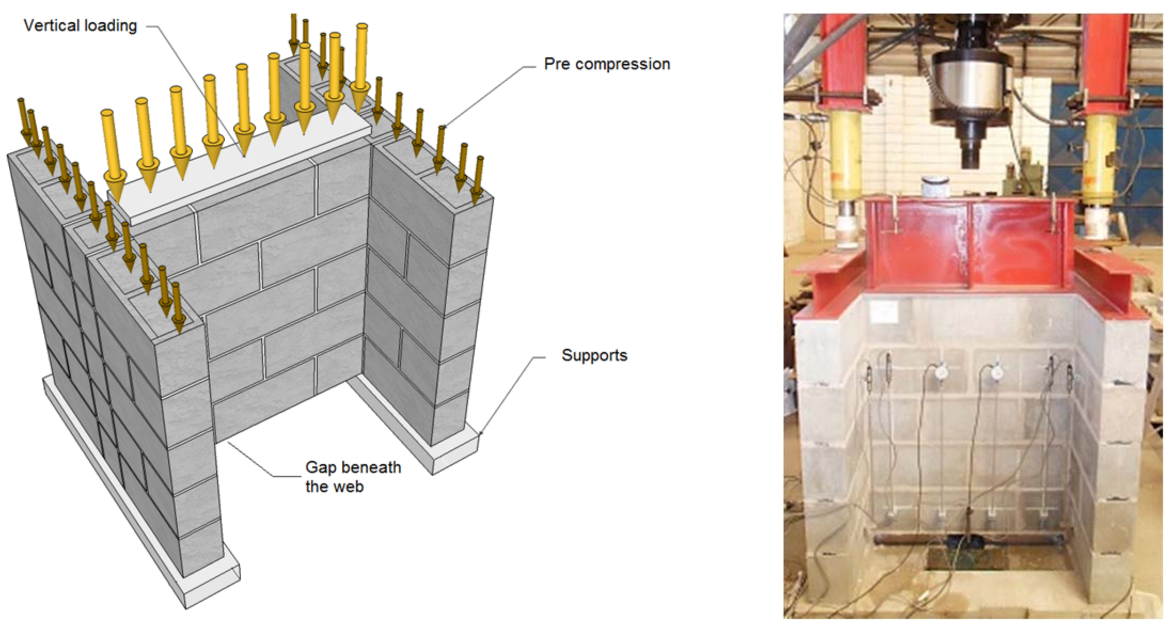
As shown in Figure 6, the relative vertical displacements between the web and the flanges (due to shear deformations along the flange-web interface) were measured using LVDTs located in the sample and connected to the actuator at a rate of 0.001 mm/s.



a) Top view b) Front view c) Side view

Figure 6 – Instrumentation of the specimens.

Prior to applying a shear load to the H sections, a small pre-compressive stress of 0.5 MPa was applied to each flange to stabilize the specimen and to avoid bending the flanges. According to Bosiljkov et al. [6], the level of pre-compression in the flanges also influences the shear resistance, but only up to a certain limit (of approximately 0.5 MPa). A compressive load was applied on the top web of the sections under displacement control to produce shear in the flange-web interface. The load was applied monotonically to failure. The test setup is shown in Figure 7.



a) Scheme for testing b) Test setup

Figure 7 – Test setup for testing vertical shear resistance.

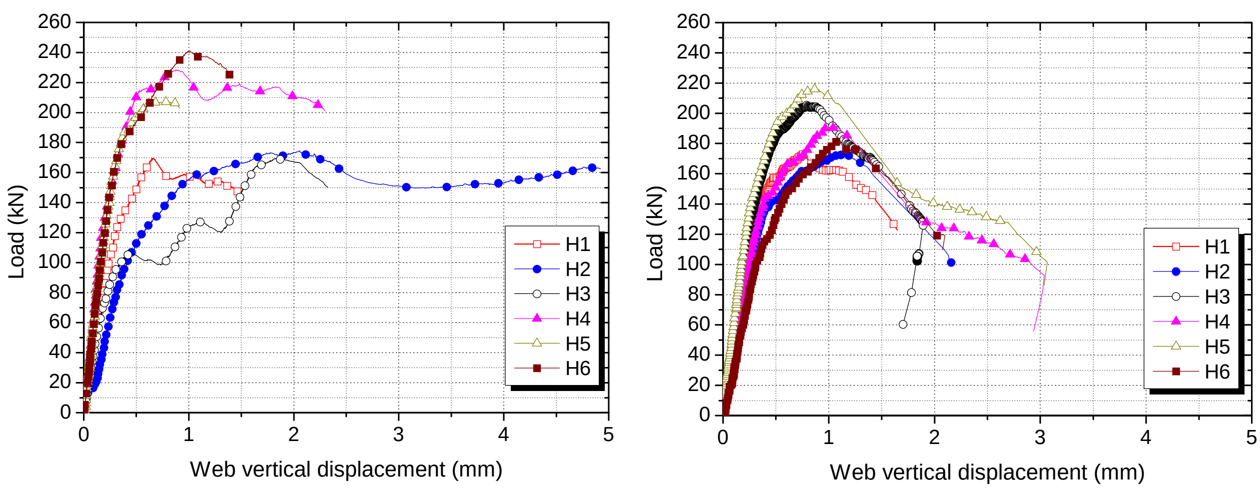
RESULTS AND DISCUSSION

For all series, failure occurred as a shear crack at the flange-web interface. Running bond connection clearly increased the shear strength of the interface. The failure occurred in the blocks due to the accumulated stress on interlocking blocks. Failure did not occur simultaneously on both sides and load capacity increased after initial cracking. In the first two series, the web showed a wide deformation and many cracks occurred before failure. See Figure 8.

a) Cracking patterns of concrete specimens b) Cracking patterns of clay specimens

Figure 8- Failure mode of running bond specimens.

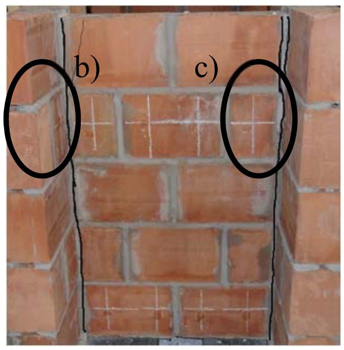
The average load-displacement data for each of the running bond specimen are shown in Figure 9. The initial branch of the load-displacement graph appeared to be linear, up to 75% of the ultimate load in almost all specimens. The cracks on the web started at approximately 50% of the ultimate load.



a) Displacement of web (concrete specimens) b) Displacement of web (clay specimens)

Figure 9 – Running bond specimen load-displacement relationships.

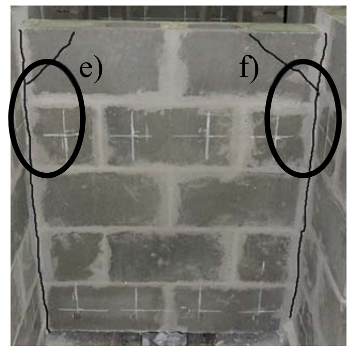
In the third and fourth series, the test showed very short strain of the web and no cracking before failure. Almost pure shear failure was achieved at the flange-web intersection. In this case, the failure was initiated by shear slip along the continuous vertical mortar joint at the flange-web intersection. It can be clearly seen that the vertical head joint at the flange-web intersection failed, while the flanges and the web showed almost no cracks, resulting in a rigid body behavior of the web (Figure 10). Dowel effec t of the U- steel anchor and shear friction along the failure surface likely caused the postcracking load resi sting mechanism. Significant reserv e capacity was maintained, even after large amounts of slip had occurred.



a) Cracking patterns of specimens (clay specimens).

b) Deformed U-anchor (Right side)

c) Deformed U-anchor (Left side)



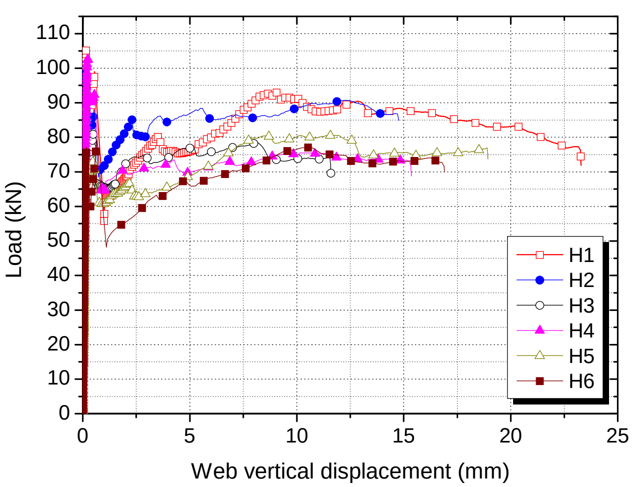
d) Cracking patterns of specimens (concrete specimens).

e) Deformed U-anchor (Right side)

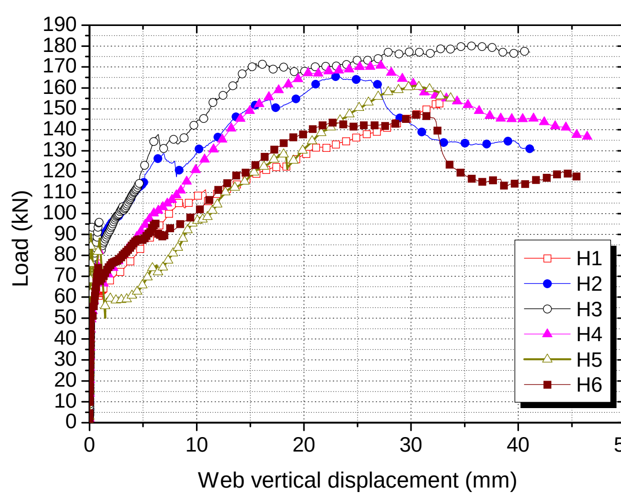
f) Deformed U-anchor (Left side)

Figure 10- Failure mode of U-steel anchored specimen (clay specimens).

The average load-displacement data for each of the U -steel anchored clay specimen are shown in Figure 11a. The initial branch of the load-displacement graph appeared to be linear, up to 90% of the ultimate load, matching to a visible crack along the web-flange intersection. For concrete specimens, the initial branch of the load-displacement graph appeared to be linear up to 50% of the ultimate load and after that, there was a significant increase of shear force along a nonlinear branch (Figure 11b).



a) Displacement of web (clay specimens)



b) Displacement of web (concrete specimens)

Figure 11- U-steel anchored specimen load-displacement relationships.

The ultimate capacity, the average and characteri stic shear strength o f the specim ens, among others, are shown in Ta ble 2. Test results of seve ral researchers and the values obtained in this current study are summarized in Table 3.

Table 2: Test Results of H-shaped specimens

Series	Material	Type of Bonding	Specimen	Ultimate Load (kN)	Shear strength (MPa)	Average shear strength (MPa) and coefficient of variation (%)	Characteristic[♦] shear strength (MPa)
I	Concrete	Running bond	1	169.45	0.61	0.72 15.88%	0.60
			2	174.57	0.63		
			3	170.78	0.62		
			4	228.45	0.83		
			5	207.07	0.75		
			6	241.13	0.87		
II	Clay	Running bond	1	172.95	0.63	0.69 9.34%	0.58
			2	172.69	0.63		
			3	205.29	0.75		
			4	190.80	0.70		
			5	216.21	0.79		
			6	182.12	0.67		
III	Clay	U-steel anchor	1	105.45	0.42 (*)	0.38 12.04%	0.32
			2	99.00	0.40 (*)		
			3	94.43	0.38 (*)		
			4	102.78	0.41 (*)		
			5	80.59	0.32 (*)		
			6	79.08	0.32 (*)		
IV	Concrete	U-steel anchor	1	154.19	0.70 (*)	0.76 6.01%	0.63
			2	165.55	0.75 (*)		
			3	180.63	0.82 (*)		
			4	170.84	0.78 (*)		
			5	161.02	0.73 (*)		
			6	147.60	0.67 (*)		

(*) Referred to the vertical area corresponding to four courses.

(♦) 5% lower bound assuming normal distribution.

Table 3: Test Results of several researchers for the vertical shear strength

Researcher	Material	Number of specimens	Scale	Running Bond (MPa)	U-Steel anchor (MPa)
Simundic (1997)	Clay brick	3	Full	1.09	
Moreira (2007)	Clay block	6	1:3	0.92(*)	0.42
Capuzzo Neto (2005)	Clay block	5	1:3	1.10(*)	
Camacho (2001)	Clay block	2	1:3	0.39(*)	0.41
Silva (2003)	Clay block	2	Full	0.76	0.49
Mauricio (2005)	Concrete block	3	Full 1:4	0.57 1.18 (*)	
Drysdale et al. (2008)	Concrete block	5	Full	1.04	
Current research	Concrete block	6	Full	0.60	0.63
Current research	Clay block	6	Full	0.58	0.32

(*) Scale factor used: 0.48 (Capuzzo Neto(2005) and (Maurício(2005))

In the absence of experimental tests, some codes present allowable stresses or characteristic shear strength, which can be used in structural design. According to both NBR 15812-1 [17] and NBR 15961-1 [18], the characteristic shear strength is 0.35 MPa for running bond walls. The code does not specify requirements for walls with other types of connection, stating the need for experimental results. Table 3 shows that most researchers have obtained larger values. The Australian Standard for masonry structures (AS 3700 [19]) also has specific provisions for characteristic shear strength to running bond walls and also has requirements for the shear strength of connectors across mortar joints. The characteristic shear strength required in the design for masonry in ordinary running bond, built of other than AAC units, is $1.2 \cdot rh$ (MPA), where rh is the proportion of the vertical shear plane that is intersected by masonry header units. In the present study, $rh = 0.5$, resulting in 0.6 MPa as the characteristic shear strength value. Note that this result is consistent with the results found in the current study. The Canadian Standard (CSA S304.1, [20]) establishes that where wall intersections are bonded so that at least 50% of the units of one wall are fully engaged in the other wall, the vertical shear at the intersection shall not exceed $0.16\sqrt{f'_m}$, where f'_m is the characteristic compressive strength of masonry in MPa units. Applying that formula to the data obtained in this research, the characteristic shear strengths would be 0.32 MPa for concrete blocks and 0.25 MPa for clay blocks. Note that these values are slightly less than the shear strength specified in the Brazilian Code and roughly half the values obtained in the present work.

To complete the study of specimens tied by U-steel anchors, a finite element based numerical analysis will be developed in order to obtain a simplified design model, inspired by the one reported by Drysdale et al. [11].

CONCLUSIONS

The type of bonding in web–flange connections has a significant influence on their behavior. Concrete specimens of different bonding types showed nearly the same shear strength, while clay specimens showed roughly a 50% loss of shear resistance in the case of U-steel bars connections. This is probably related to the fact that the steel bar tears the face shell of the clay block. Regardless the material, the running bond connection, without steel bars, showed a brittle failure.

When analyzing the preliminary experimental results, there are characteristic values which are larger than those currently adopted by some codes. It is noteworthy that in the usual practice in Brazil there is no information about a large number of walls which present pathological conditions due to shear at the vertical interfaces, suggesting the need for calibration of the strength limit provided by the Brazilian Code.

Results pointed out some aspects and indicators in the behavior of flanged walls that should be numerically and experimentally confirmed in order to clarify the behavior of these structures and to allow the development of accurate design models.

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