An investigation into the shear bond strength of masonry

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A research report submitted to the Faculty of Engineering and the Built Environment, University of the Witwatersrand, in fulfilment of the requirements for the degree of Master of Science in Engineering.

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DECLARATION

I declare that this research report is my own work. It is being submitted for the degree of Masters of Science in Engineering to the University of the Witwatersrand, Johannesburg. It has not been submitted before for any degree or examination to any other University.

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(Signature of Candidate)

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ABSTRACT

This study was undertaken to investigate the shear strength of masonry. Specimens were produced and tested. The failure mechanism, the peak shear strength as well as the force deflection curves for all specimens were recorded. The specimens failed at the shear bond eighty percent of the time. The positions in a wall where the bricks are attached to the mortar when constructing the wall, produces a weaker brick mortar bond than, where the mortar is applied to the bricks. The characteristic shear strength equations obtained for a double and a single wall through experimentation, gives shear strength values respectively of 4.2 and 2.8 times greater than the recommended equations of the European and South African codes of masonry design. After 3 mm differential settlement, a wall has to be repaired due to the rapid increase in shear stresses.

DEDICATION

This work is dedicated to my late Father, Mother, Sister and best friend Dr. Stavros Albanis. I also dedicate this work to the Hellenic community of South Africa, the Greek Orthodox Church of Saint Constantine and Helen, as well as, all my family and friends, for all their help and support.

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My thanks goes to Prof. H. C. Uzoegbo as well as the members of staff of the School of Civil and Environmental Engineering.

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Additionally, I wish to acknowledge with thanks contribution from Dr. Stavros Albanis my friends, and colleagues.

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The average stress	τ_u
The initial stress without any pre-compression applied	τ_o
The friction coefficient between the interfaces	μ
The applied pre-compression load	σ_n

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1. INTRODUCTION:

A wall resists, through its shear strength, forces acting perpendicularly to its face. Such forces are wind loads, lateral earth loads, seismic loads and forces due to stresses developed by differential settlement of the wall foundations. Therefore the role of the shear strength is a critical quantity determining the structural performance and structural integrity of a wall.

From the literature review it can be seen that the characteristic shear strength used in designing according to the Eurocode 6 and SANS 10164, is based on the assumption that the specimen will fail at the shear bond. One of the aims of this investigation was to obtain the frequency of shear bond failure occurring and the factors that affect shear bond failure occurring.

It was noted from the literature review that the experimental specimens were always subjected to horizontal forces in a direction parallel to the long side of the brick. This research considered the behaviour of the specimens subjected to horizontal forces acting perpendicular to the long face of a brick, a case study that reflects the most common and realistic response of a wall. The latter experimental results indicated a substantially lower the shear strength of a wall, in the order of over 30%.

It was noted from the literature review that there were no results for shear strength at different curing ages, this was seen as a gap in the topic and was researched.

A holistic approach to evaluate the shear strength of a wall is via laboratory experimentation and thereafter extrapolation of the produced results of the scale modelling. The materials used had known structural strength characteristics, forming the parameters of the experimentation. For this purpose, specimens were produced to obtain the change in shear strength with time, as well as the effect of different curing environments on the shear strength.

Moreover, the frequency of shear failure of the specimens was systematically recorded, and it was noted that a predictable dominant failure pattern occurred at the brick-mortar bond. The factors affecting the occurrence of this failure pattern were also noted.

Astonishingly, the results from the preliminary analysis of the tests as well as from the subsequent additional experimentation confirmed that the maximum shear stress occurred at a consistent deflection of about 10 mm, irrespective of the variables, i.e. magnitude of the applied load, direction of the applied force, curing regime, and mortar strength, of course within the boundaries of the selected values.

It was additionally noted that the shear stress-deflection curves followed a certain trend. The consistency of this result was tested, and a general stress deflection curve was obtained. This curve indicates the failure pattern of a wall subjected to differential settlement on its foundation.

The results are applicable only to cement stock bricks and mortar mix 2 (see Table 2-3) according to SANS 10164-1. The results are applicable for curing age up to 44 days. Long term durability performance tests were not undertaken. All other material properties used in the experiments are detailed in a subsequent section of this research report (section 3.2).

2. LITERATURE REVIEW

2.1 The shear bond strength in masonry

2.1.1 What is the shear bond strength in masonry

The shear bond strength in masonry is the force in shear required to "separate the units from the mortar and each other" (CEMEX, n.d.) .This is as shown in Figure 2-1 below

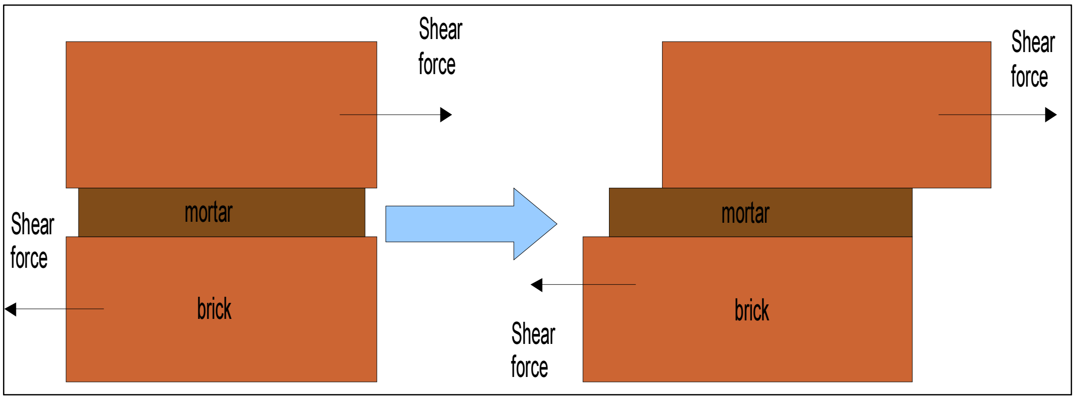


Figure 2-1 a diagrammatical explanation of the shear bond strength in masonry

The shear bond strength in masonry is the bond strength between the brick mortar interface.

The "Shear strength at the interface comes from friction due the asperities between the surface of mortar layer and the surface of the brick unit, and the chemical bond between mortar and brick units. Normal compression perpendicular to the interface further increases its shear strength because the asperities cannot easily slide over one another". (Mosalam K, 2009)

"The bond development in masonry is due to mechanical interlocking of hydrated cement-products into the pores of the brick". (Reddy, 2008)

According to (Mosalam K, 2009) this phenomenon can be represented by the Mohr-Coulomb Criterion as stated in equation 2-1 below.

$$\boxed{\tau_u = \tau_o + \mu\sigma_n} \qquad \textbf{Equation 2-1}$$

Where:

- τ_u -is the average stress
- τ_o - is the initial stress without any pre-compression applied
- μ – is the friction coefficient between the interfaces
- σ_n- is the applied pre-compression, i.e. the vertical load applied to the specimen before testing.

According to (Riddington, 1994) the linear relationship of shear stress to normal stress (pre-compression) is valid up to approximately 2 N/mm^2 of applied pre-compression, i.e. above 2 N/mm^2 the Mohr Coulomb Criterion is no longer valid**.**

"The bond between brick and mortar is derived from penetration of the mortar and hydration products, such as calcium silicate hydrates CSH, into the brick surface voids and pores" (Lawrence, 1987)

During cement hydration the 2 main by-products formed are the CSH gel and slaked lime (winters, n.d.).

The CSH gel formed gives the mortar its adhesive property allowing it to bond to the masonry unit while the slaked lime produces a weak layer at the interface. The addition of pozzolans to the cement as an admixture reacts with the slaked lime that has been produced to create a secondary CSH gel thereby increasing the bond strength (Shrive, 2001)

2.1.2 Why the shear bond strength is important

According to (Maheri, 2011) the shear strength in masonry is very important as it is the principle resisting force to seismic loads.

The shear bond strength is important as the strength of the masonry bricks is generally greater than that of the mortar so failure generally occurs at the joint (Maheri, 2011)and (Mosalam K, 2009).

2.1.3 Applied loads that cause shear bond failure.

- Wind loads
- Seismic loads
- Normal loads
- Settlement of the foundations
- Impact loads
- Lateral earth pressures

An example of shear bond failure caused by foundation settlement is shown in the Figure 2-2 below.

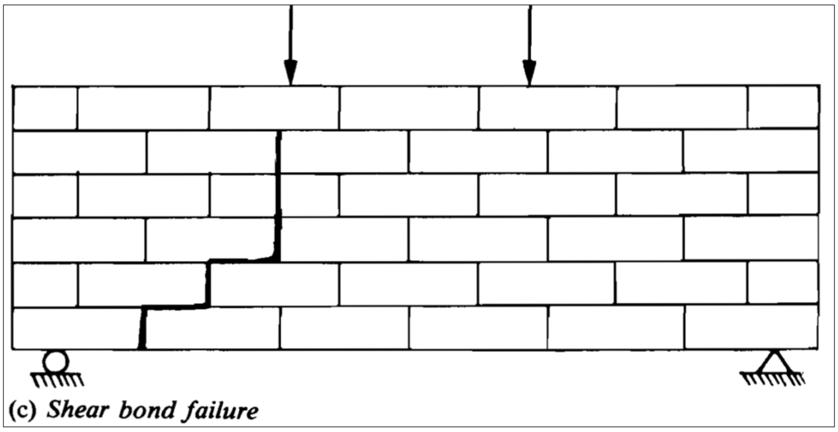


Figure 2-2 shear bond failure caused by foundation settlement (Ehsani R, 1997)

An example of shear bond failure caused by wind or normal loading is shown in figure 2-3:

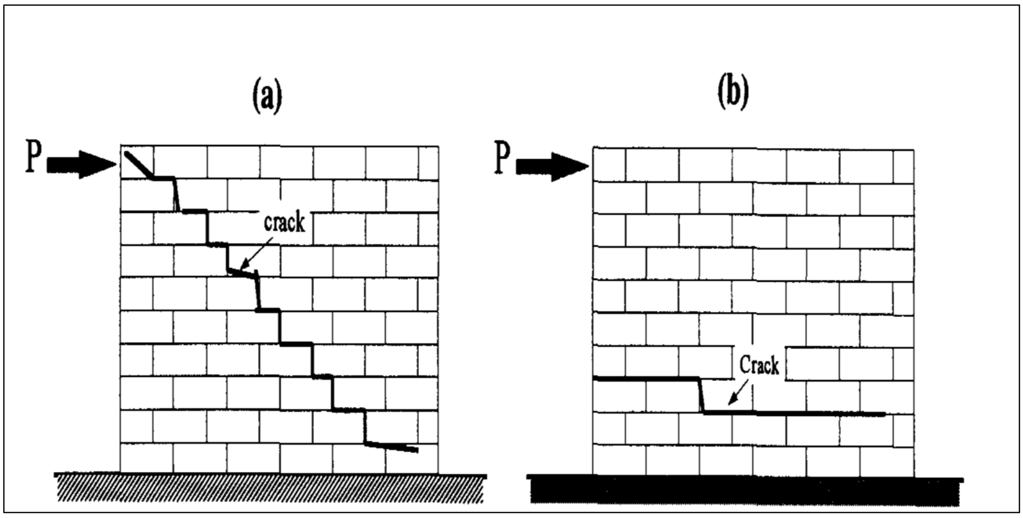


Figure 2-3 shear bond failure caused by applied loads (Ehsani R, 1997)

2.1.4 Factors affecting the shear bond strength in masonry

The shear bond strength in masonry is related to the bond strength in masonry. The greater the bond strength the greater the initial shear strength (τ_o).

The bond strength can be" affected by the mortar strength, mortar shrinkage, brick strength, joint thickness, interface morphology, and chemical bond. (Zhu, 1997)

2.1.4.1 Cement content

According to (CEMEX, n.d.), the bond strength in masonry is mainly dependent upon the cement content in the mortar, i.e. the greater the cement content in mortar the greater the bond strength.

2.1.4.2 Air content

According to (CEMEX, n.d.) , the air content is another important factor in the bond strength of masonry, i.e. the higher the air content the lower the bond strength.

2.1.4.3 Curing regime

According to the (Portland cement association, n.d.), recent experiments show a 300% increase in bond strength when the concrete masonry specimens are damp cured compared to when they are cured in the laboratory air.

(Maheri, 2011) Tested the bond strengths of mortar when the masonry specimen was moisture cured and found an increase of up to 50% in shear, tensile and flexural bond strengths.

2.1.4.4 Type of masonry unit

According to the (Portland cement association, n.d.), the bond strength is influenced by the texture of the mortar unit, i.e. the rougher the texture of the unit the greater the bond strength this allows for a greater "mechanical keying" with the mortar.

According to (Maheri, 2011) the Brick water absorption affects the bond strength because it determines the amount of water transmitted from the mortar to the brick. This controls the degree of hydration of the mortar and the amount of hydration products that will be transported and deposited in the masonry pores.

(Maheri, 2011) Found it is essential to wet the bricks before construction in arid regions so as to develop good bond strength. They also found that curing in a moist environment increases the shear bond strength.

2.1.4.5 Other factors

According to (CEMEX, n.d.) Bad workmanship and casting are important factors contributing to poor shear strength.

2.1.5 Test method for testing the shear bond strength as recommended by the (European Standard 1052-3, 2002).

Specimens can be couplets or triplets. A couplet specimen is defined as the configuration of two bricks bonded together with the means of mortar. A triplet specimen is defined as the configuration of three bricks bonded together with the means of mortar. See Figure 2-4 below. The European standard suggests the shear strength be tested using masonry triplets.

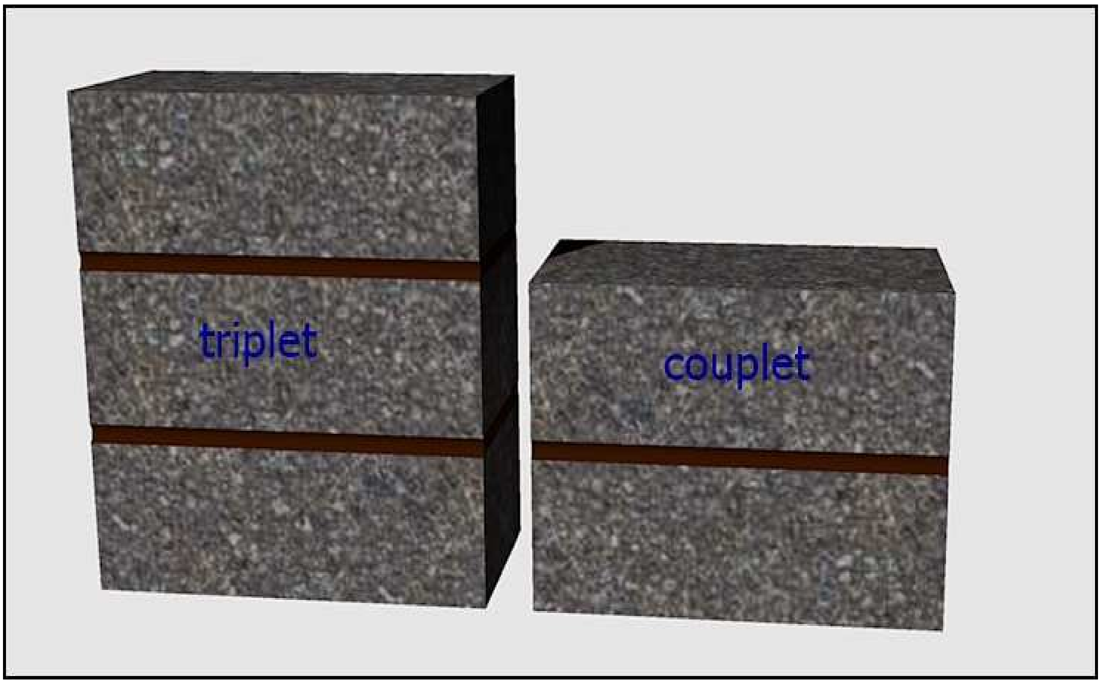


Figure 2-4 showing a triplet and couplet specimen

Figure 2-5 below shows the test configuration for shearing couplets. A load is applied to the centre brick. The other two bricks are supported by rollers and there is a pre-compression load applied horizontally to the specimen denoted by (2) in Figure 2-5. This configuration can be modelled as a simply supported beam, from this analogy one can see that a bending moment is induced; this bending moment is counteracted by the pre-compression load.

The magnitude of the three pre-compression loads to be used to obtain the characteristic shear strength equation is determined by the compressive strength of the bricks. A minimum of three specimens per pre-compression load are to be tested to obtain the characteristic shear strength equation as recommended by the code (European Standard 1052-3, 2002).

If the compressive strength of the bricks is 10 MPa, use pre-compression loads that give approximately 0.2 MPa, 0.6 MPa and 1.0 MPa. For units with compressive strength less than 10 MPa, use pre-compression loads of 0.1 MPa, 0.3 MPa and 0.5 MPa. (European Standard 1052-3, 2002)

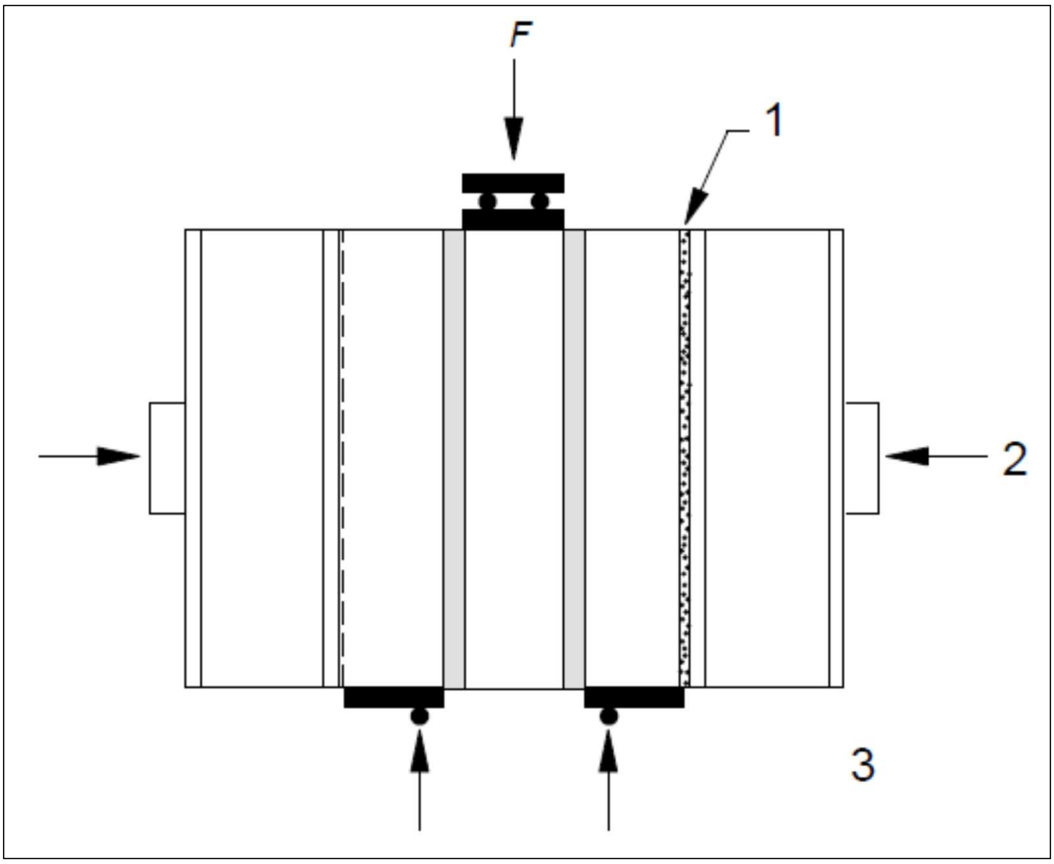


Figure 2-5 the test configuration of the specimens according to (European Standard 1052-3, 2002)

The European standard states that if during a laboratory test shear failure occurs either through the unit or by crushing or splitting in the unit, the results should be discarded and the test must be repeated. Hence the characteristic shear strength equation is based on shear bond failure.

From the code the possible failure patterns from testing specimens in shear are shown in Figure 2-6 below.

Types of failure

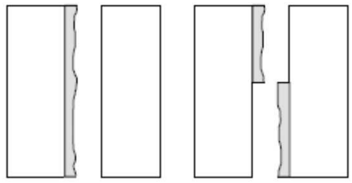


Figure A.1 - Shear failure in the unit/mortar bond area either on one or divided between two unit faces

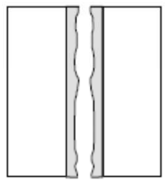


Figure A.2 - Shear failure only in the mortar

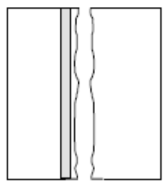


Figure A.3 - Shear failure in the unit

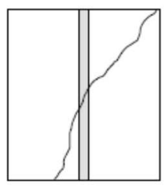


Figure A.4 - Crushing and or splitting failure in the units

Figure 2-6 the types of failure patterns for masonry specimens in shear (European Standard 1052-3, 2002)

2.2 Previous research done on the topic

2.2.1 Relationship between the shear bond strength and the compressive bond strength

(Reddy, 2008) , investigated the relationship between the shear bond strength and the compressive bond strength. In order to make this comparison for a certain masonry block and mortar mix without altering their respective compositions (Reddy, 2008) altered the texture of their masonry block specimens to increase the shear bond strength.

“Brick–mortar bond development is generally attributed to the mechanical inter-locking of cement hydration products into the surface pores of the bricks”, (Reddy, 2008) Therefore a rougher surface texture will give greater bond strength than a smooth surface due to the increase in size of surface pores.

2.2.1.1 Test method

(Reddy, 2008), tested the shear bond strength using masonry couplets as shown in Figure 2-7 below.

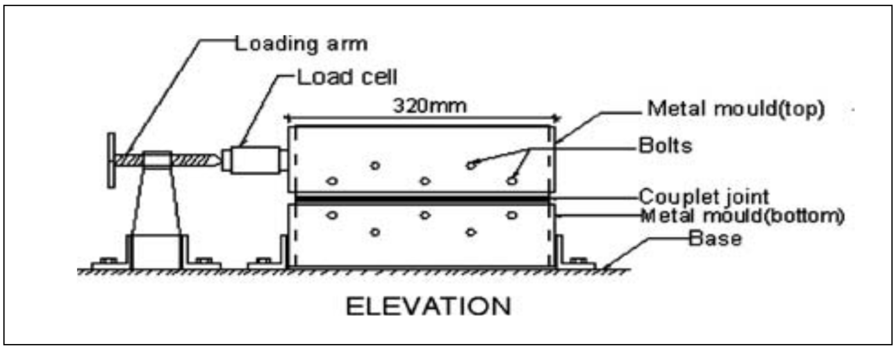


Figure 2-7 test configurations for testing couplet specimens in shear according to (Reddy, 2008)

2.2.1.2 Results and discussion

Effect of brick texture on shear strength

The results of the shear tests indicated that there was an increase of up to four times in shear strength comparing the specimens with a smoother surface texture to the ones with the rougher surface texture the range of results which they had achieved ranged from 0.21 MPa to 0.83 MPa.

Failure pattern

The testing regime identified four distinct patterns of failures as shown below in Figure 2-8, i.e.

(a) interface failure, separation of the bond at the block mortar interface
(b) block failure, shearing of the block
(c) mortar failure, shearing of the mortar horizontally, and
(d) both block and mortar failure (Reddy, 2008)

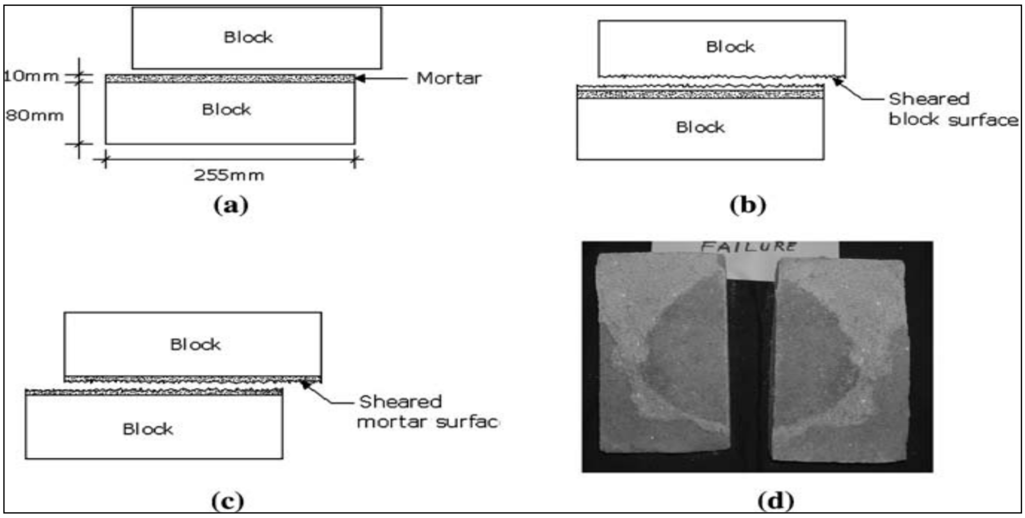


Figure 2-8 possible failure patterns according to (Reddy,2008)

(Reddy, 2008) Noticed that failure of the interface generally occurred if the shear strength was lower than 0.25 MPa as shown in Figure 2-8(a) above, if the shear strength was greater than 0.25 MPa then either the brick or the mortar will fail in shear as shown in Figure 2-8(b) and 2-8(c) above.

As the shear bond strength increases the bond strength increases the brick mortar specimen starts to display characteristics to that of a single masonry unit. A chain always breaks at its weakest link, so the masonry specimen fails at the weakest component .As soon as the masonry shear bond is greater than the brick or mortar shear strength the specimen would fail at the weaker of the two.

Stiffness

Other findings were that the shear bond strength only affected the masonry compressive strength when the stiffness of the brick was less than that of the stiffness of the masonry.

Effect of brick texture on shear strength

If the stiffness of the brick is less than that of the mortar an increase in shear bond strength increases the stiffness of the masonry unit (the brick and mortar), if the stiffness of the brick is greater than that of the mortar an increase in shear strength decreases the stiffness of the masonry unit.

2.2.2 An investigation of the behaviour of the mortar joint

According to (Abdou, n.d.), one of the principal failures of a masonry wall is due to failure in the shear bond in the masonry.

2.2.2.1 Test method

(Abdou, n.d.) Used loading and unloading cycles of shear tests to investigate the behaviour of the mortar joint .Numerous previous studies done to investigate the behaviour of the mortar joint were done using the triplet and couplet tests.

2.2.2.2 Results and discussion

Mohr Coulomb failure criterion

(Abdou, n.d.) Experimented using two types of specimens, i.e.

i. one ordinary brick type and

ii. One brick type containing a hole in it.

They observed that the experiments followed the Mohr coulomb friction law .i.e. the applied shear stress was linear to the applied pre-compression stress.

This indicates that the Mohr coulomb friction law is a good theoretical model for shear strength as it is also applicable when the wall experiences changes in loading.

Effect of brick texture on shear strength

The brick with the hole experienced a greater value of shear strength as expected due to the greater mechanical interlocking at the hole of the brick. The mortar joint were seen to observe elasto-plastic behaviour. (Abdou, n.d.)

2.2.3 A new failure criterion to describe the shear strength in masonry

(Ali, n.d.) , Investigated a new failure criterion, other than the Mohr coulomb failure criterion, for mortar joints subject to both shear and tension. This type of failure according to Ali and Page occurs when large concentrated vertical loads act on the brickwork and splitting occurs at the joint in the region below the load.

2.2.3.1 Test method

According to (Ali, n.d.), the failure criterion used is expressed in terms of the normal, parallel and shear stresses. According to (Ali, n.d.), the Mohr-Coulomb failure criterion does not include the parallel stress acting on the joint and therefore they have made a comparison between the Mohr-Coulomb failure criterion and their own which includes the parallel stresses.

The principal stresses were "obtained by subjecting brickwork triplets with sloping bed joints to biaxial tensile and compressive forces by varying the force ratio and the joint angle (Ali, n.d.)" as indicated in Figure 2-9 below.

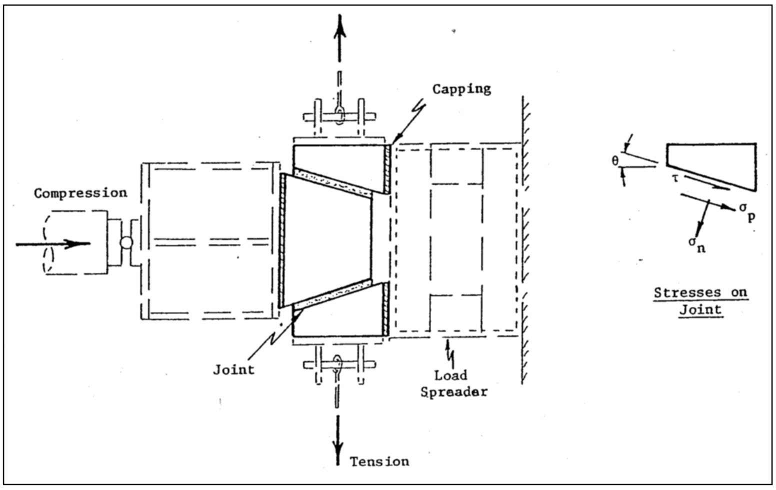


Figure 2-9: Testing apparatus (Ali,nd)

2.2.3.2 Results and discussion

(Ali, n.d.) Found out that the Mohr-Coulomb failure criterion gives a good approximation of the data however the failure surface that they have created by including the parallel stresses gives a greater representation of the data.

2.2.4 Effect of carbon fibre reinforced in on bond strength

(Zhu, 1997) investigated the use of carbon fibres in mortar to increase the bond strength according to (Zhu, 1997) the drying shrinkage of the mortar decreases the bond strength because the mortar shrinks but the bricks do not shrink, the addition of fibres decreases the shrinkage of the mortar thereby increasing the shear bond strength.

2.2.4.1 Test method

In order to test the shear bond strength (Zhu, 1997) employed their own test method to obtain pure shear, according to (Zhu, 1997) the triplet test used to test the shear bond strength does not give a result for pure shear stress.

This would be due to the bending moment resulting from the application of the load at the centre brick in the triplet test.

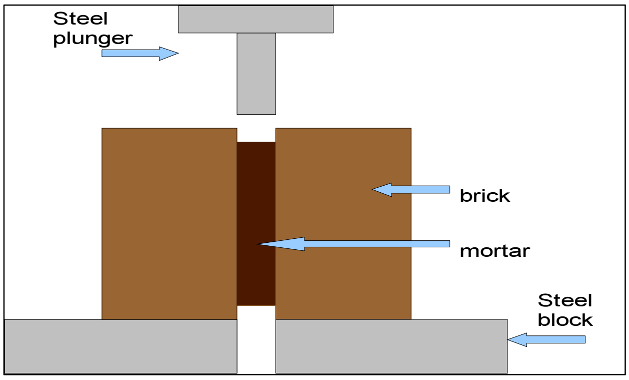


Figure 2-10: (Zhu, 1997) , has applied the load at the joint to obtain pure shear.

2.2.4.2 Results and discussion

(Zhu, 1997) Noted that the addition of 0.35% fibres in the mortar increased the shear bond strength by 89%.The optimum amount of carbon fibres was found to be 0.5% by the weight of cement.

2.3 Compare codes on the shear bond strength in masonry:

The European code for masonry design

2.3.1 Eurocode 6

In the European code the number after the letter "M" which denotes mortar, gives the 28-day-strength of the mortar in compression.

The Eurocode 6 part 1 of 1995 states that if the characteristic shear strength is required i.e. the average shear strength it should be found experimentally or from previous test data.

If this is not available or tests cannot be conducted then the masonry shear strength is to be taken as the least of the values below:

1. $\tau_u = \tau_o + 0.4\sigma_n$ **Equation 2-2**
2. Or $\tau_u = 0.065x\ f_b$ but not less than τ_o **Equation 2-3**

3. Or τ_u = the limiting value given in a Table 3.5 in the code, Table 2-1 below. **Equation 2-4**

Where

f_b - is the compressive strength of the masonry unit when the load is applied perpendicular to the bed face. Equation 2-3 suggests that the shear strength of the mortar unit is 6.5% of its strength in compression.

In the Eurocode 6 the characteristic shear strength equation is determined by the strength of the mortar as well as the type of mortar unit.

The type of masonry unit is determined by the volume of the holes as well as the volumes of the individual holes.

Using Equations 2-2, 2-3, 2-4 with Tables 2-1 and 2-2, one can calculate the expected shear stress values according to Eurocode 6 for the specimens to be tested.

From Table 2-2 of Eurocode 6 we can determine the group our mortar unit belongs to, since our unit does not comprise volume of holes greater than 25 % and no individual hole is greater than 12.5 % it falls into group 1.

Since cement stock bricks were used our initial and limiting shear stresses are obtained from Table 2-1.

Giving a characteristic shear strength equation of:

$$\tau_u = 0.15 + 0.4\,\sigma_n \quad \textbf{Equation 2-5}$$

And a limiting stress of 1.5 MPa,

The Compressive strength of the brick was found to be approximately 7.5 MPa at 28 days see Figure 3-18. Using equation 2-3, f_b, the other limiting shear stress is found to be 0.455 MPa.

Table 2-1: (Table 3.5, (Eurocode 6, 1996)) Values of the initial shear strength of masonry

Masonry Unit	Mortar	f_{vko} (N/mm^2)	Limiting f_{vk} (N/mm^2)	
Group 1 clay units	M10 to M20	0,3	1,7	
	M2,5 to M9	0,2	1,5	
	M1 to M2	0,1	1,2	
Group 1 units other than clay and natural stone	M10 to M20	0,2	1,7	
	M2,5 to M9	0,15	1,5	
	M1 to M2	0,1	1,2	
Group 1 natural stone units	M2,5 to M9	0,15	1,0	
	M1 to M2	0,1	1,0	
Group 2a clay units	M10 to M20	0,3	The lesser of longitudinal compressive strength (see note below) or	1,4
	M2,5 to M9	0,2		1,2
	M1 to M2	0,1		1,0
Group 2a and Group 2b units other than clay and Group 2b clay units	M10 to M20	0,2		1,4
	M2,5 to M9	0,15		1,2
	M1 to M2	0,1		1,0
Group 3 clay units	M10 to M20	0,3	No limits other than given by equation (3.4)	
	M2,5 to M9	0,2		
	M1 to M2	0,1		

NOTE For Group 2a and 2b masonry units, the longitudinal compressive strength of the units is taken to be the measured strength, with δ taken to be not greater than 1,0. When the longitudinal compressive strength can be expected to be greater than 0,15 f_b, by consideration of the pattern of holes, tests are not necessary.

Table 2-2: Geometrical requirements for Grouping of Masonry Units (Eurocode 6, 1996)

	Materials and limits for Masonry Units							
	Group 1 (all materials)		Group 2		Group 3		Group 4	
		Units	Vertical holes				Horizontal holes	
Volume of all holes (% of the gross volume)	≤ 25	clay	$> 25; \leq 55$		$\geq 25; \leq 70$		$> 25; \leq 70$	
		calcium silicate	$> 25; \leq 55$		not used		not used	
		concrete [b]	$> 25; \leq 60$		$> 25; \leq 70$		$> 25; \leq 50$	
Volume of any hole (% of the gross volume)	$\leq 12,5$	clay	each of multiple holes ≤ 2 gripholes up to a total of 12,5		each of multiple holes ≤ 2 gripholes up to a total of 12,5		each of multiple holes ≤ 30	
		calcium silicate	each of multiple holes ≤ 15 gripholes up to a total of 30		not used		not used	
		concrete[b]	each of multiple holes ≤ 30 gripholes up to a total of 30		each of multiple holes ≤ 30 gripholes up to a total of 30		each of multiple holes ≤ 25	
Declared values of thickness of webs and shells (mm)	No requirement		web	shell	web	shell	web	shell
		clay	≥ 5	≥ 8	≥ 3	≥ 6	≥ 5	≥ 6
		calcium silicate	≥ 5	≥ 10	not used		not used	
		concrete [b]	≥ 15	≥ 18	≥ 15	≥ 15	≥ 20	≥ 20
Declared value of combined thickness [a] of webs and shells (% of the overall width)	No requirement	clay	≥ 16		≥ 12		≥ 12	
		calcium silicate	≥ 20		not used		not used	
		concrete [b]	≥ 18		≥ 15		≥ 45	

[a] The combined thickness is the thickness of the webs and shells, measured horizontally in the relevant direction. The check is to be seen as a qualification test and need only be repeated in the case of principal changes to the design dimensions of units.

[b] In the case of conical holes, or cellular holes, use the mean value of the thickness of the webs and the shells.

2.3.2 SABS 0164-1 / SANS 10164-1

The South African code on Masonry design provides two equations to calculate the shear strength. The characteristic shear strength equations are determined by only considering the mix design of the mortar and the strength of the mortar.

Table 2-3: The shear strength equations from the South African code of masonry.

Mortar class	Calculate f_v from	Maximum allowable value of f_v MPa
I	0,35 + 0,6 g_A	1,75
II	0,15 + 0,6 g_A	1,4
where g_A =	the design vertical load per unit area of wall cross-section due to the design vertical loads calculated from the appropriate loading condition given in 4.2.1.	

Table 2-4: mortar class specification to SANS (10164-1)

1	2	3
Mortar class	Compressive strength at 28 d MPa, min.	
	Preliminary (laboratory) tests	Works tests
I	14,5	10
II	7	5
III	2	1,5

From Table 2-4 the mortar used in experimentation is mortar class 2. The compressive strength of the mortar was tested to be 7.5 MPa see Figure 3-18.

Therefore the characteristic shear strength equation according to the South African code would be:

$$\tau_u = 0.15+0.6\,\sigma_n \quad \text{Equation 2-6}$$

And the limiting shear strength would be 1.4 MPa.

The South African code suggests that the friction factor be less than a value of 0.6.

2.3.3 Summary

Table 2-5: The expected shear strength for the brick and mortar mix design according to the South African and European design codes

Codes	**Eurocode 6**	**SABS 0164-1 / SANS 10164**
Characteristic shear strength equation	$\tau_u = 0.15+0.4\ \sigma_n$	$\tau_u = 0.15+0.6\ \sigma_n$
Limiting stresses	1.5 MPa	1.4 MPa
Strength of brick in shear	0.455 MPa	

From Table 2-5 above certain differences between the expected shear strength values of the two codes can be seen.

- The friction factors used in the characteristic shear strength equation of the Eurocode is 0.4, whereas it is 0.6 in the South African code.
- The Eurocode 6 uses the type of mortar unit, to determine the characteristic shear strength equation. It also takes into account the strength of the masonry unit when determining the shear strength.

From Table 2-5 according to Eurocode 6 the limiting value for shear strength is taken as 0.455 MPa, the shear stress of the brick, this is less than the limiting stress of 1.5 MPa.

3. EXPERIMENTAL PROCEDURE:

3.1 Description of testing equipment:

3. 1.1 The Testing Equipment:

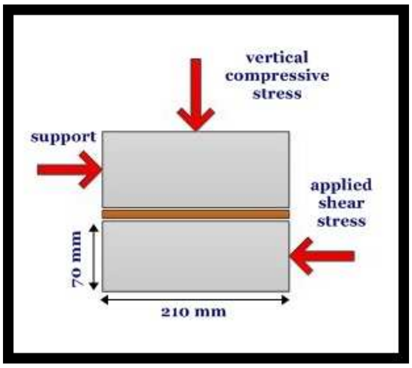


Figure 3-1: Descriptive setup of the shear strength test

In order to obtain the Characteristic shear strength equation, (see Equation 2-1) for the specimens tested, the shear strength is obtained for various constant vertical applied loads. This vertical load was previously denoted as the pre-compression load (σ_n). It specifies the compressive load applied to the line of bricks considered in testing see Figure 3-1. Details of the testing equipment are as follows:

The Applied shear force is applied in the longitudinal direction of the specimen, see Figure 3-1, this shall be denoted as test direction A. Test direction A can be used to simulate the effect of shear stresses acting perpendicular to the long face of a double wall(Load L acting on surface area B), see Figure 3-5.

The testing apparatus used is in the Laboratory of the Civil Engineering Department (at the university of Witwatersrand) is the big shear box, see Figure 3-2.

Figure 3-2: The testing apparatus of the Civil Engineering Department

The big shear box used to test the shear strength of the masonry couplets is shown in Figure 3-2, 3-3 it was manufactured in the University's workshop. The shear box consists of two cast iron boxes, see Figure 3-3-4; the bottom box is attached to a metal plate see Figure 3-3-1, This metal plate is supported by rollers and can translate freely in the horizontal direction of the force see Figure 3-3-3, it is prevented from rotating or moving in the vertical direction.

The top box sits on the bottom box and it is prevented from moving horizontally in the direction of the load by a thick side steel plate placed perpendicular to the force direction see Figure 3-3-1. It is prevented from moving downwards by the bottom box. The top box is free to rotate about the horizontal axis see Figure 3-3-4.

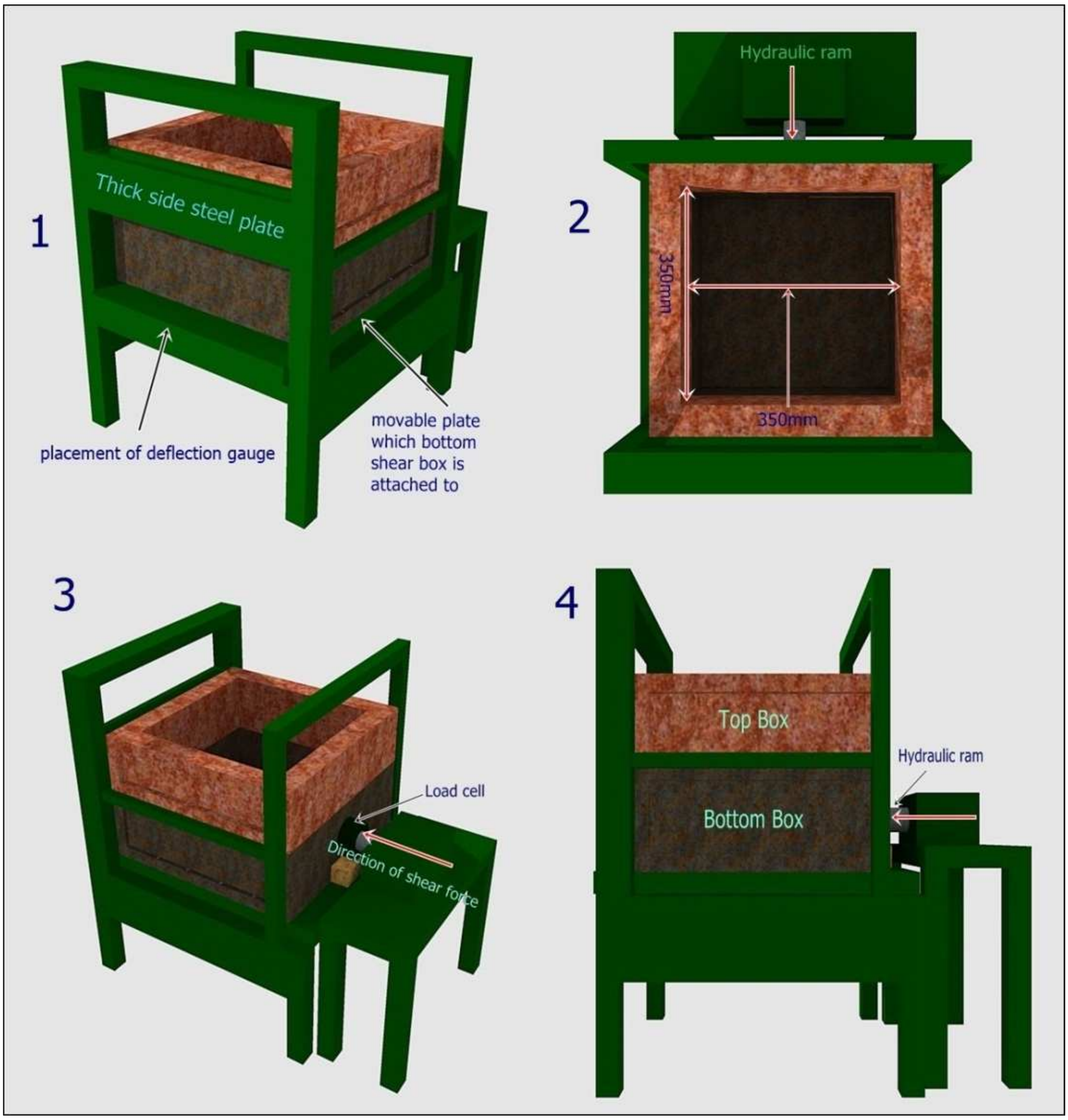


Figure 3-3: Schematic representation of the experimental apparatus

The shear force is applied by a hydraulic ram see Figures 3-3-2 and 3-3-4. The hydraulic ram applies the load to a load cell, which is centred at the bottom box. Subsequently the load cell transfers the load to the centre of the bottom shear box. The hydraulic ram applies the load at a constant deflection rate. The load is applied uniformly to the face of the brick.

Using the apparatus in figure 3-1 to test for shear produces the following results for each specimen:

1. The peak stress that results in the failure of the specimen

2. With correct extrapolation of the results, the horizontal applied force at a constant deflection rate, imposed by the hydraulic ram, can simulate the behaviour of a wall subjected to *differential settlement of the foundations.*

The top and bottom boxes of the big shear box with dimensions of 350 mm x 350 mm x 250 mm, figure 3-2-2, are larger than the couplet specimens, which have dimensions of 100 mm x 210 mm x 150 mm see figure 3-19. This implies that the top and bottom boxes of the shear box should be modified to reduce its size to the size of the couplet. This modification was implemented in the laboratory and comprised of two components. A steel adaptor for the top shear box and a concrete adaptor in the bottom shear box (see Figure 3-9). The first is hereafter referred to as the *steel mould* and the latter as the *concrete mould*.

3.1.2 The Concrete Mould:

The concrete mould is shown in Figure 3-4. It is slightly smaller in surface area than the inside of the top and bottom boxes of the shear box so that it can be easily lowered into the bottom shear box. It had a compressive strength of approximately 30 MPa at 28 days. It was fibre reinforced to prevent cracking at the corners of the interface between the concrete mould and the specimen.

Part A in Figure 3-4-3 is the hollow space where the specimen is positioned into the concrete mould. Part A is slightly bigger in surface area than the average brick so that the specimen can fit easily into the concrete mould. The depth of the concrete mould is 120 mm on the outside. On the inside, at the position where the specimen is placed, it is 62 mm deep from the upper edge of the concrete mould (see Figure 3-4-1). Bearing in mind that the height of a commercial cement stock brick is approximately 70 mm, the specimen after being placed in part A will protrude approximately 8 mm above the concrete mould. 10 mm of brick is required to protrude above the concrete mould instead of 8 mm (see Figures 3-9-2 and 3-10); Packing material was used to achieve this 10mm protrusion (see Figure 3-12-5). If the brick does not protrude from the concrete mould, the specimen will fail at the bond between the brick and mortar which is not necessarily the weakest point of the specimen.

The concrete mould weighed 40-50 kg. A thin material strip of approximately 1.5 m in length (see Figure 3-12-1) having a thickness fitting through the interface between the mould and the bottom shear box was used to lower the concrete mould into the bottom shear box. This strip was placed in the middle of the concrete mould. This strip could not be removed until the experiment is completed. In order to achieve stability of the concrete mould inside the bottom shear box, two additional strips were attached to the bottom sides of the concrete mould as shown in Figure 3-4-4.

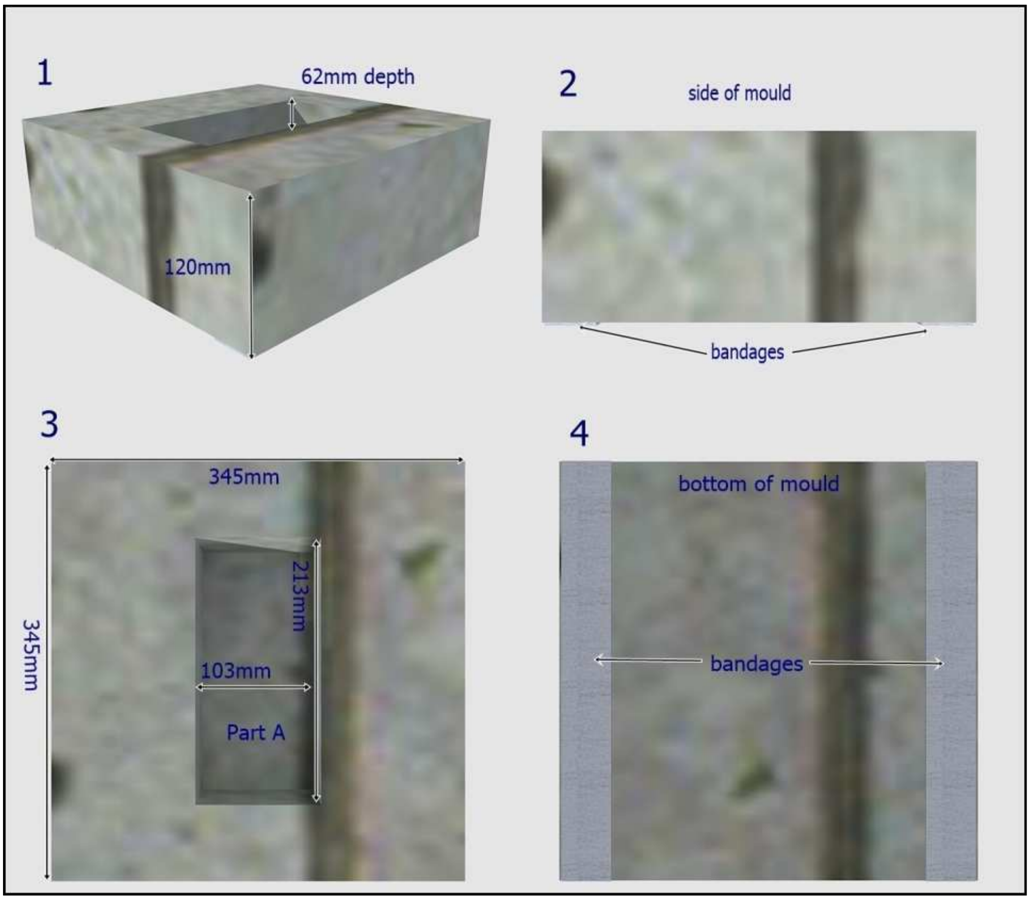


Figure 3-4: Concrete Mould used in testing

3.1.3 The Steel Mould:

A steel mould adaptor was manufactured for the top shear box; Figure 3-6-1 below shows the initial steel mould. This steel adaptor was made to hang from the top of the top shear box (see Figures 3-6-4, 3-10 and 3-13-4). The sides of the steel adaptor were made to be flush against the sides of the top shear box in the direction of the applied shear load. Its function would be to transfer the force to the top shear box without been deformed, bent or buckled. This steel mould was to be used in all laboratory experiments.

Using a basic analysis it was seen that the 3 mm plate used to create the steel mould would not buckle under the applied load (assumed the maximum shear stress of 1.75 MPa given by the South African code, SANS 10164-1). Although the initial steel mould passed the criteria for non-buckling the plate was still deforming during testing under the applied load.

The initial steel mould (Figures 3-6-1, 3-6-2 and 3-6-3) was then reinforced by two angle irons below the horizontal surface and two above it as shown in Figures 3-6-4, 3-6-5 and 3-6-6. This reinforcement minimized the deflections to an insignificant figure.

The surface area of Part A of the steel mould is slightly larger than that of the average top (or bottom) brick surface. This serves two purposes,

1. The couplet specimens can easily fit into the steel mould,
2. The couplet carries the vertical compression load on its own without any assistance from the steel mould.

Part B is the right-hand side surface of the steel mould (see Figure 3-6-6) against which the specimen is placed flush before the testing load L is applied on the opposite side of the mould (see Figure 3-6-3). In this case the horizontal force applied is acting from right to left. It has to be ensured that the specimen is flush against the surface of part B so that any dynamic load affecting the force-deflection curves can be avoided.

The height of part B is 55 mm including the 3 mm thickness of the plate. This makes the upper surface of the top brick of the couplet specimen to lie 5 mm above the part B. On the same token, the lower surface of the top brick of the couplet specimen protrudes 10mm below the bottom of the steel mould (see Figures 3-6-1 and 3-6-2). The reasons for the top and bottom protrusions are:

1. The 10 mm protrusion of the brick below the surface of the steel mould ensures that the specimen will not be forced to fail at the brick-mortar bond. The measurement of 10mm is chosen because it is the thickness of the mortar used in testing.
2. The 5 mm protrusion of the upper surface of the top brick from the top of part B ensures that the vertical load is applied solely to the couplet specimen and no component of it is transferred to the steel mould.

The steel mould in Figures 3-6-4, 3-6-5 and 3-6-6 is used to simulate the shear force acting perpendicular to the long face of a double wall as seen in Figure 3-5. A double wall is approximately 230 mm thick, it consists of two single walls connected by a 10 mm vertical layer of mortar see Figure 3-5. The experimental setup produces results without the vertical mortar layer. However, the vertical mortar layer is in compression and not shear. Therefore, the extrapolation of the experimental results to determine the strength of a cantilever wall in shear should approximate the reality adequately, see Figure 3-5. The steel mould in Figure 3-6-4, 3-6-5 and 3-6-6 is used to simulate test direction A see Figure 3-1).

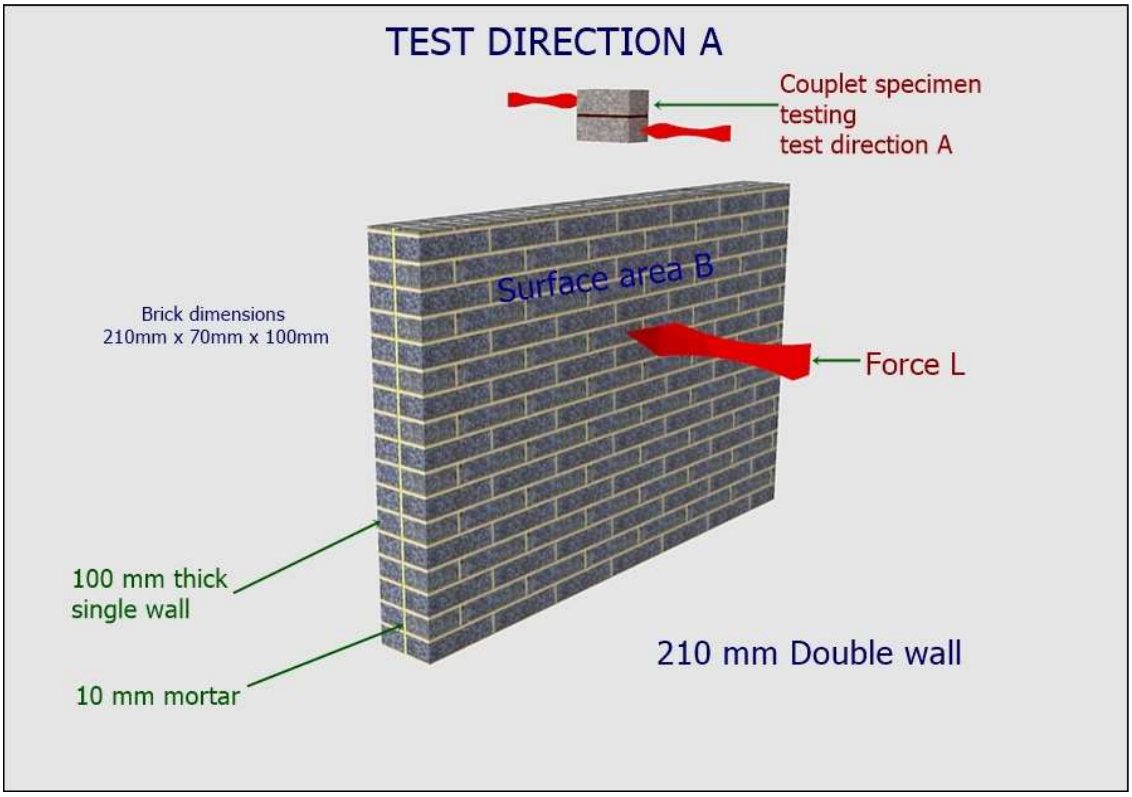


Figure 3-5: A schematic representation showing how Test direction A models shear stresses acting on a double wall, modelling a 100 mm portion of a double wall (test direction A)

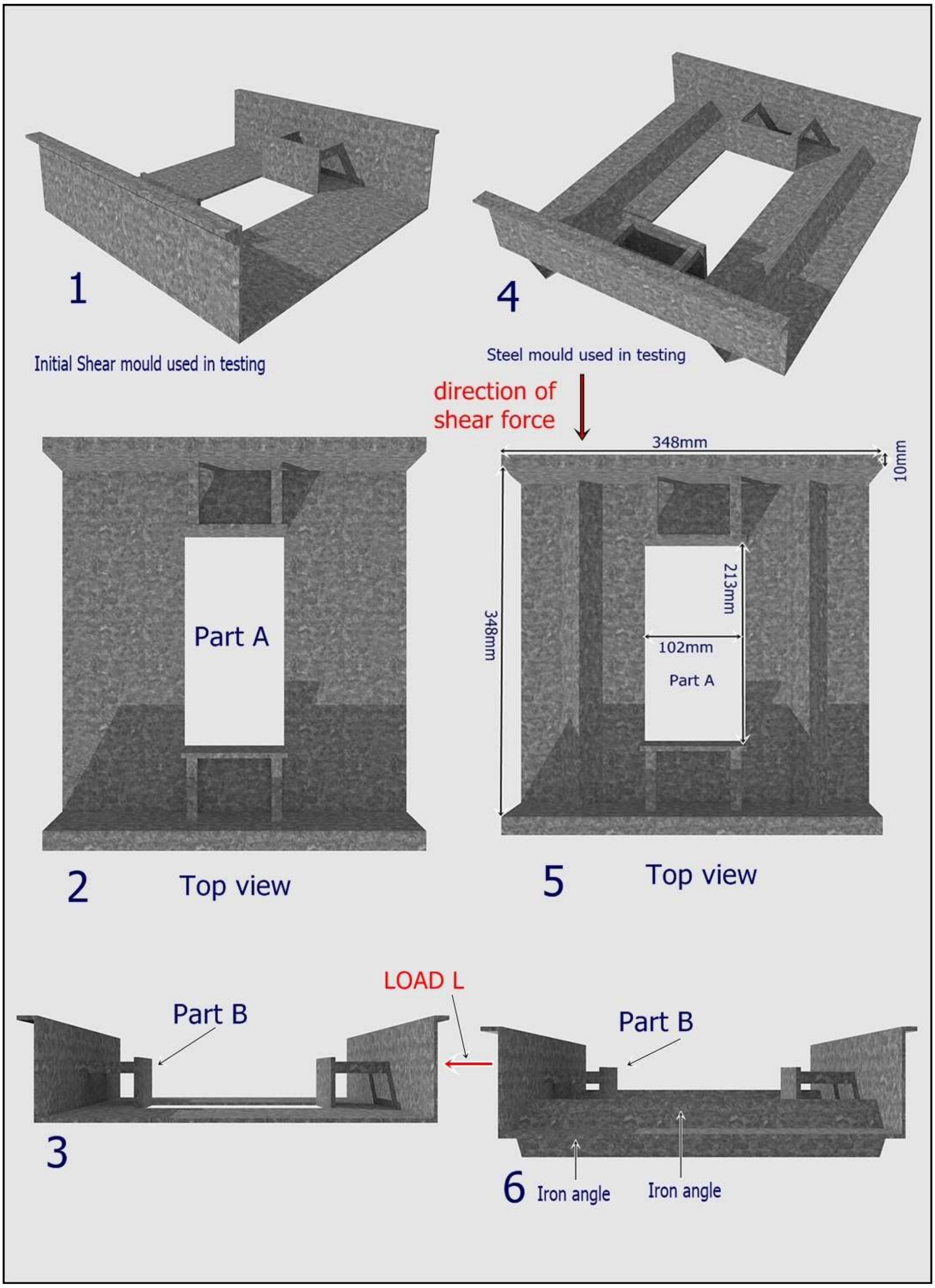


Figure 3-6: It shows the steel mould used to model the testing of shear stress in a double wall, modelling a 100 mm portion of a double wall (test direction A)

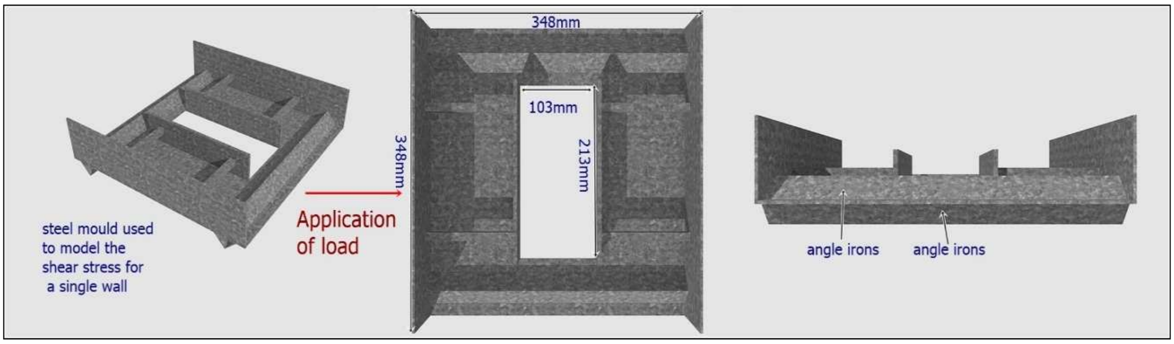


Figure 3-7: It shows the steel mould used for testing shear stress in a single wall, modelling a 210 mm portion of a single wall (test direction B referring to Figure 5-5).

3.1.4 Set Up For Experimentation:

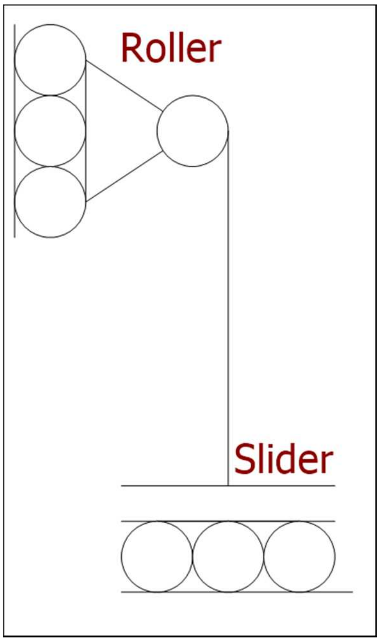


Figure 3-8: modelling of the supports of the shear box

Figure 3-8 models the supports of the shear box apparatus. The roller represents the support on the top box acting opposite the applied shear force. The slider represents the supports on the bottom box which is not fixed as it translates freely in the horizontal direction but can still take moments. The column is a representation of the specimen which is placed in the shear box. There are two forces acting on the specimen horizontal force acting at the slider and vertical force acting at the roller.

In order to test the specimens using the original model of the shear box machine, the specimen with thin steel plates was stacked on all four sides in the concrete mould so that the concrete mould would act as a fixed support to the specimen. The concrete mould was packed on all four sides inside the bottom box, making it fixed onto the bottom box allowing it to transfer all the forces into the bottom box.

The other reason for fixing the specimen to the concrete mould is to prevent it from rotating as can be clearly seen from the model. This configuration in Figure 3-8 introduces a bending moment in addition to the shear force and if allowed the specimen would rotate which introduces other forces.

The configuration of the specimen inside the rig is shown in Figure 3-9 below.

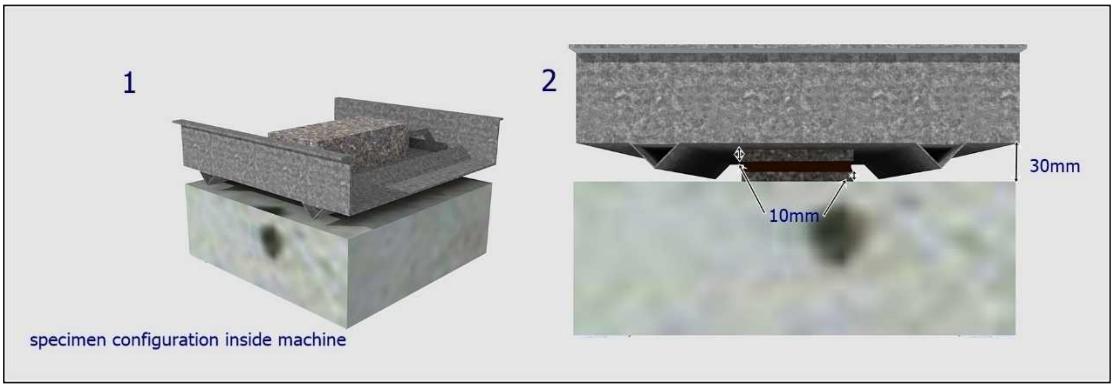


Figure 3-9: The configuration of the specimen and steel and concrete moulds in the machine

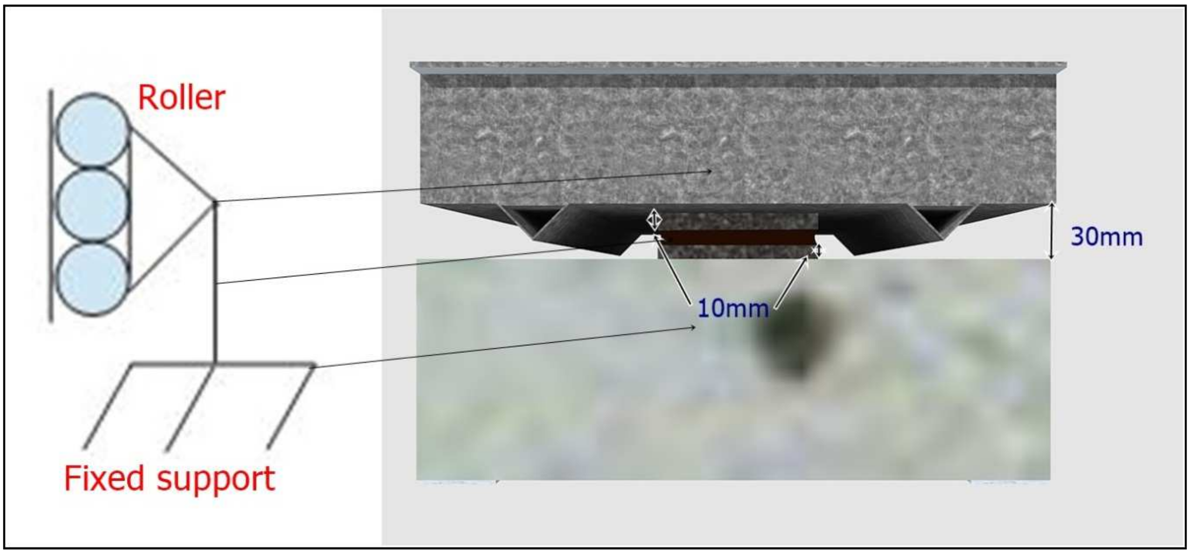


Figure 3-10: Support conditions of the specimen

The specimen can be modelled as a column with fixed base and propped at the top see Figure 3-10, where the column represents the 10 mm mortar and the 10mm of brick on either side of the mortar protruding from the concrete and steel moulds respectively.

To limit the applied moment acting on the specimen, see figure 3-10, the top shear box was prevented from rotating, by placing two steel I sections between the steel bars on the machine and the top shear box see Figure 3-11, Figure 3-14, Figure 3-15. The original steel beams used in testing were adapted see Figures 3-11-2, 3, 4.A piece of 20 mm depth and 375 mm length was cut from both steel section see Figures 3-11-1, 2, 3. These pieces were cut from the beams to prevent loading from the beams on to the initial load (figure 3-12-2).

Placing these beams onto the top box would increase the friction forces rapidly between the top and bottom boxes and these forces would no longer be considered negligible. These additional forces were then obtained through experimentation.

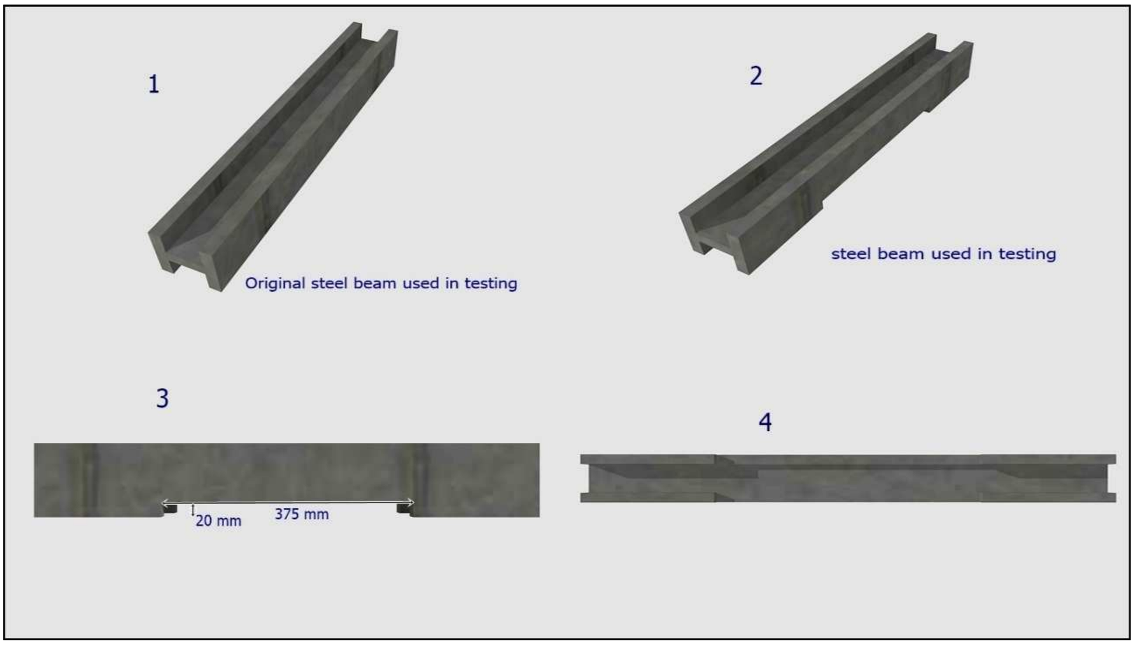


Figure 3-11: Steel beams used in testing

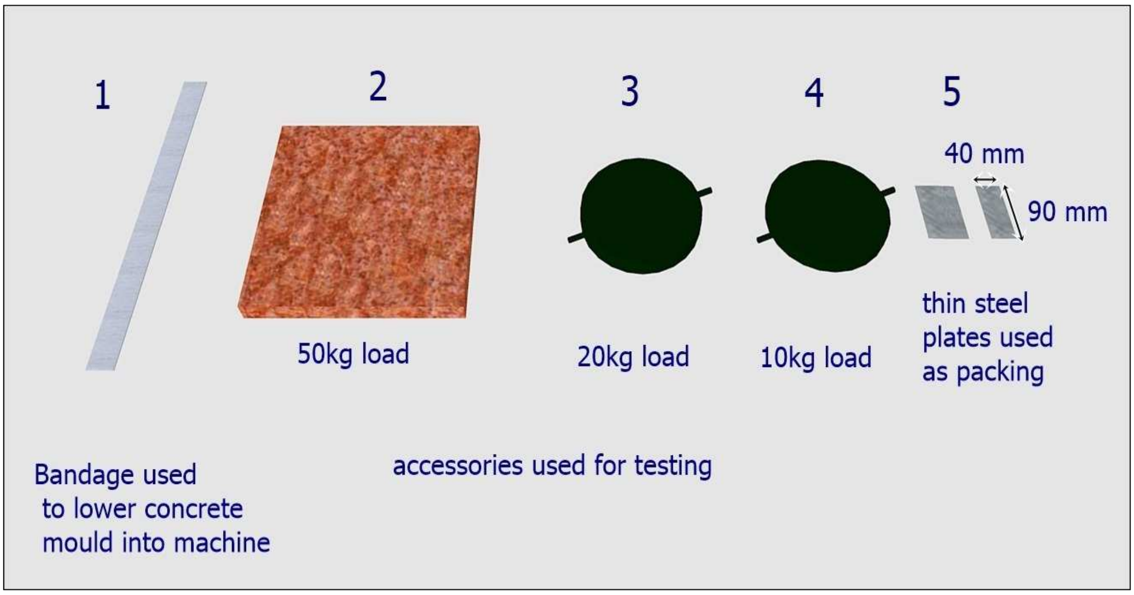


Figure 3-12: accessories used in testing

Figure 3-12 shows all the accessories used during testing, the bandage which was used to lower the concrete block into the machine, Figure 3-12-1.The 50 kg load (47 kg actual weight) was used to apply the initial vertical load onto the specimen had dimensions smaller than that of the inside of the steel mould so that it would easily fit into the steel mould without transferring its load to the steel mould i.e.

all of its load would be transferred to the specimen Figure 3-12-2. Four 20 kg weights and two 10kg weights were placed on top of the 50 kg load to vary the vertical load so that results may be obtained for different depths of a wall Figure 3-12-3 and Figure 3-12-4. The thin steel plates above were used as packing material Figure 3-12-5.

Figure 3-13 is used to illustrate the placement of the specimen into the shear box before testing.

1. The shear box is cleaned the bottom and top boxes are aligned, see Figure 3-13-1.
2. The concrete box is placed inside using the bandage; packing material is placed on all four sides of the shear box so that it fits fixed to the bottom box, see Figure 3-13-2.
3. The specimen is then placed into the concrete mould and is packed using the thin steel plates to fix it to the inside of the concrete mould, see Figure 3-13-3.
4. Next the steel mould is placed on top, see Figure 3-13-4.
5. The Vertical load required for testing is applied. Explained in detail below.
6. The steel beams are placed on top of the top box to prevent rotation see Figures 3-14 and 3-15.
7. The Horizontal deflection gauge is placed in the direction of the load see Figure 3-3-1.

The Horizontal shear force is applied. Peak stresses and force deflection curves are recorded.

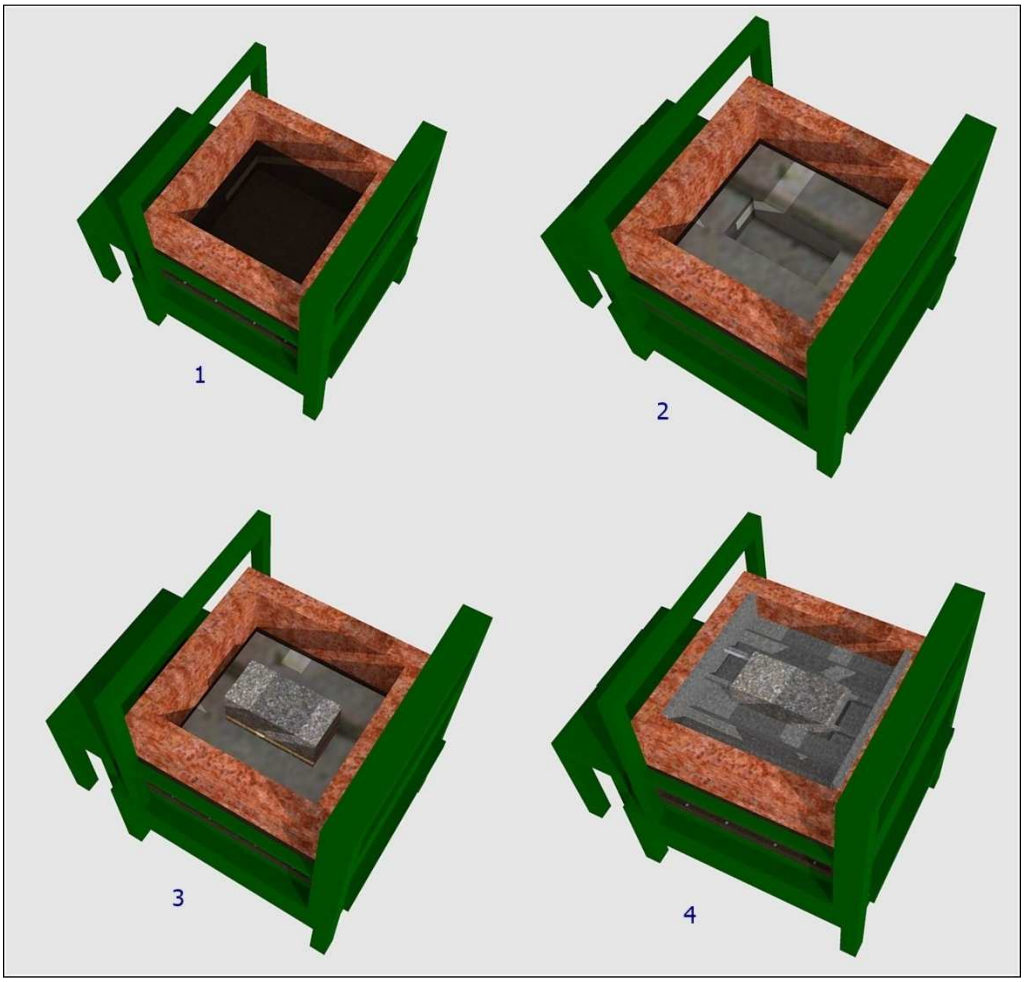


Figure 3-13: Setup of the equipment before testing

Four distinctive vertical loads were applied during testing to obtain the characteristic shear strength equation (equation 2-1) see Figures 3-14 and 3-15, these vertical loads will be referred to as load configurations.

- The first load configuration models the shear force acting between the top two rows of brick in a wall; see Figures 3-14-1-2 and Figure 4-1.
- The second load configuration models the shear force acting between two rows of brick at a metre depth from the top of the wall see Figures 3-14-3-4 and Figure 4-1.
- The third load configuration models the shear force acting between two rows of bricks at a 2 metre depth from the top of the wall see Figures 3-15-1-2 and Figure 4-1.

- The fourth load configuration models the shear force acting between two lines of brick at 3 metre depth from the top of the wall see Figures 3-15-3-4 and Figure 4-1.

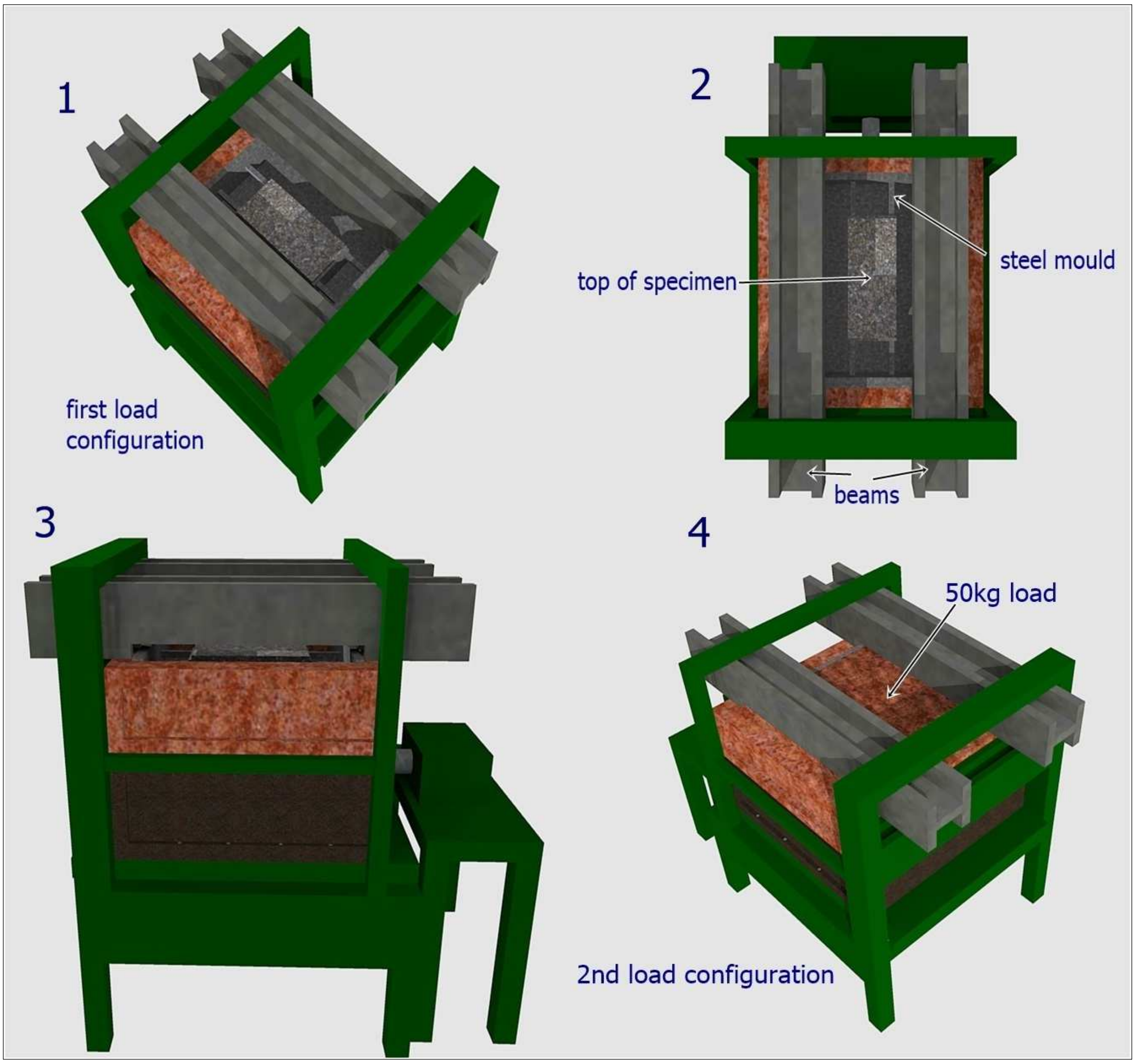


Figure 3-14: Schematically representation of the 1st and 2nd load configurations used in testing

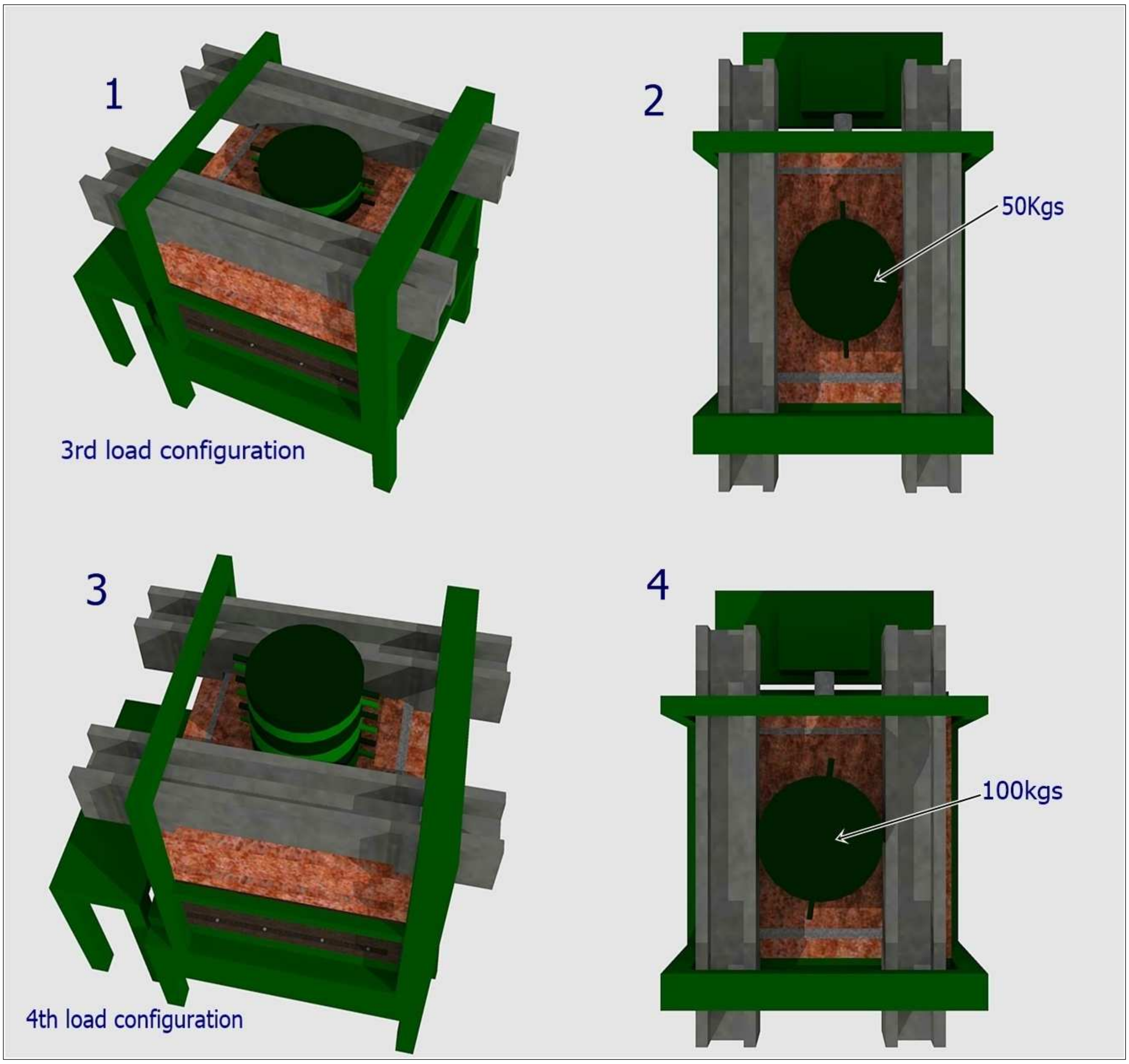


Figure 3-15: Schematically representation of the 3rd and 4th load configurations used in testing.

3.2 Materials used:

Cement of Grade 32.5 N

The cement of grade 32.5 N was used because it is the cement commonly used in masonry in South Africa.

Pit sand from a local hardware was used. Figure 3-16 shows the grading distribution of the pit sand used.

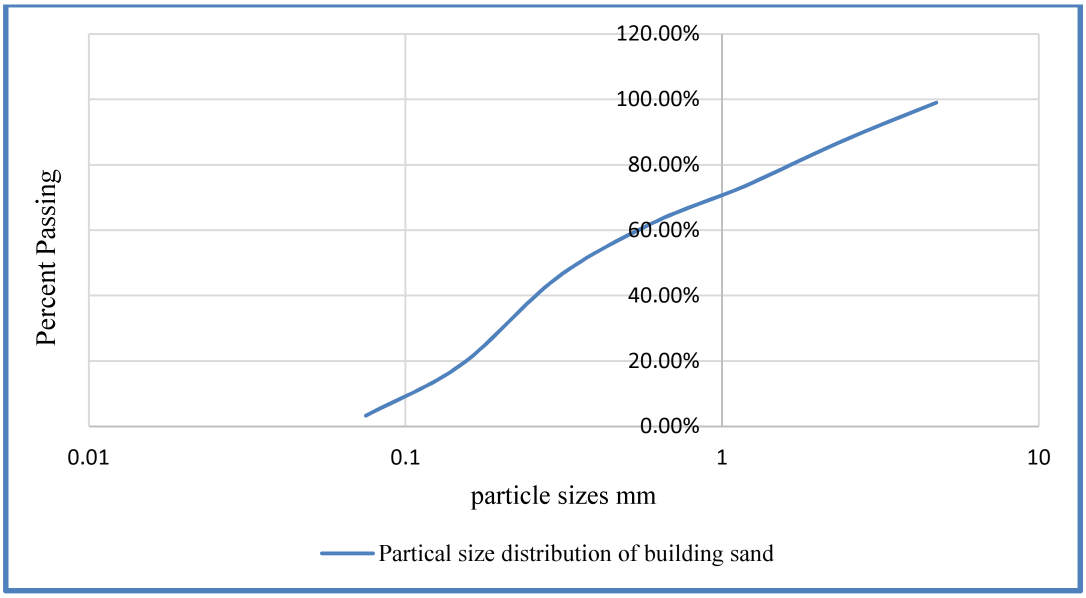


Figure 3-16: The particle size distribution of the building sand used in testing

From the sieve analysis it can be seen that less than 35% of the soil is fine, making this a course soil. More than 50% of the coarse material is finer than 2mm making this a sand and between 5-15% of the material are fines, making this a silty-sand as can be seen above the soil contains particles of many sizes.

For well-graded sand the coefficient of uniformity (C_u) is greater than 6 and the coefficient of curvature (C_c or C_z) is between 1 and 3. (Sivakugan, 2000), From the above graph the Coefficient of uniformity was computed to be 4.9 which is less than 6 and the coefficient of curvature was computed to be 0.65 which is not between 1 and 3 making this a poorly graded soil. Hence the sand used was a poorly graded silty-sand (SPM).

The loose density of the sand was computed as 1189 kg/m^3 and the compacted density of the cement from the manufacturer was given as 1500 kg/m^3, which can be easily computed on the basis that a sack of cement has a weight of 50 kg and a volume of 33 litres.

On site three wheelbarrows (approximately 66 litres volume each) of sand are mixed with one sack of cement to produce a mix design of 1:6 in volume proportions of cement to sand. The sand used in-situ is loose and not compacted hence the loose density was used in the calculations for the mix proportions.

Out of all the available masonry units in the market, cement-stock bricks were used. They were used because they are the most commonly used bricks in South Africa. These bricks have flat rough surfaces they do not contain holes, they were handpicked to be of the same dimensions and have similar size and distribution of pores on the surface to eliminate an additional variable. The bricks were purchased and were already dry when used in the experiments.

Their batch designated a nominal compressive strength of 7 MPa and an average weight of 2.3 kg. The average dimensions of the bricks used were 100mm x 70mm x 210mm

Figure 3-17: It shows the shape and the sizes of the pores of an average brick used in the experimentation

3.3 Mix design:

The design mix can be summarized as follows:

1. A mix design of 1 part cement to 6 parts sand per volume was used. It is referred to as mortar mix 2 in the South African code and it is used for, "Normal loadbearing applications, as well as parapets, balustrades, retaining structures, and freestanding and garden walls, and other walls exposed to possible severe dampness" according to (CNCI, 2009).
2. A water to cement ratio of 4:3 in kilograms was used, it was found to give the most consistent mix.
3. A mortar mixer was used to mix the mortar.
4. The mortar making procedure:
    a. First the sand and cement were place into the mortar mixer they were then mixed for 60 seconds; to produce a uniform mix of sand and cement with no bunches.
    b. Second water was poured into the new uniform mix at a constant rate with the mixer still in operation this occurred for another 90 seconds.

The mortar mix was tested and the compressive strengths were obtained by producing mortar cubes and testing them at 28 days. The samples were produced for the three curing regimes that were tested, curing in water, air, and in a specially designed solar chamber. Mortar cubes were also produced for an alternative mix whereby the water to cement ratio was reduced.

The curing regime affects the mortar strengths in compression see Figure 3-18. Curing in air gave the greatest compressive strength which was followed by curing in water and then curing in the solar chamber. See chapter 4-1.

According to the European code (Eurocode 6) this makes the mortar an M7 because it has 28 day strength of approximately 7 MPa when cured in water.

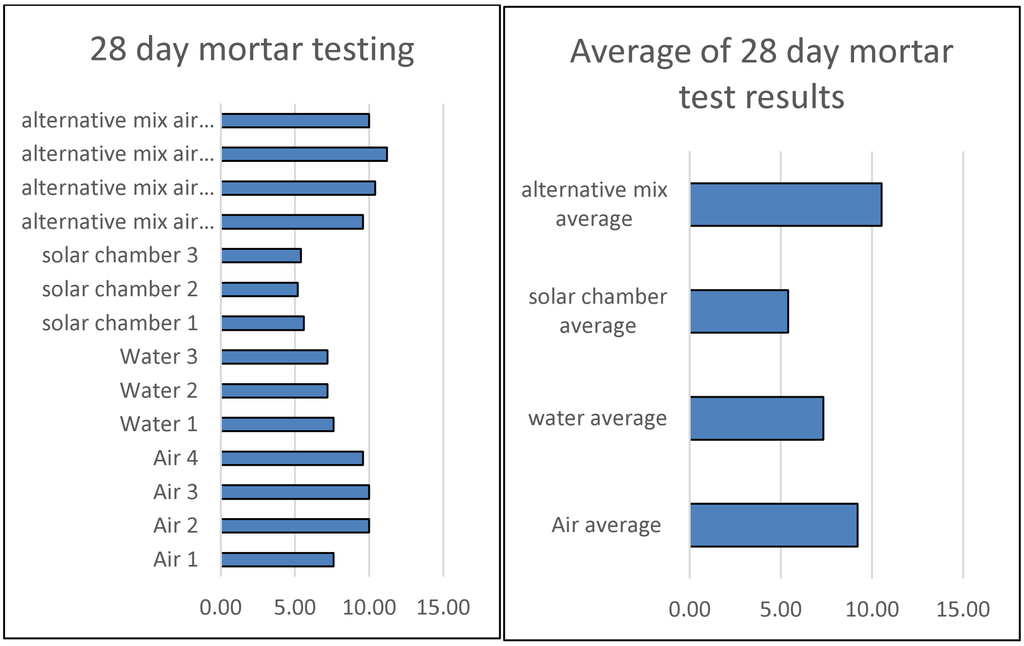


Figure 3-18: The compressive strength of the mortar used in testing according to the curing regime

3.4 Production of Specimens

The specimens produced to test the shear strength were brick couplets. The configuration of two bricks with the mortar in between forms a brick couplet, see Figure 3-19.

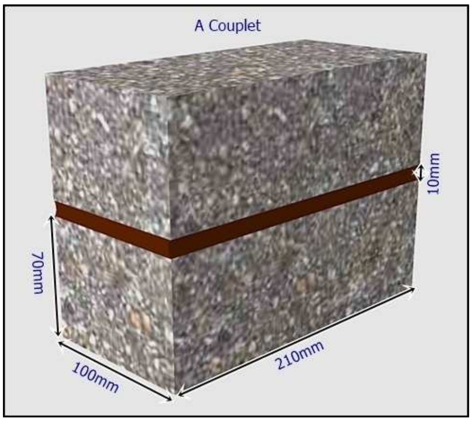


Figure 3-19: A dimensioned representation of the couplet used in the study

Since cement bricks were used and as cement is hygroscopic, the dry state of the bricks would absorb part of the water from the mortar, weakening the mortar thereby leading to false experimental conclusions. In order to avoid the above distortion, the individual bricks were pre-wet before application of the mortar.

Mortar was applied within one hour of being mixed as specified according to (Eurocode 6) (European Standard, 2002).

Only six specimens were produced at a time, due to the amount of mortar the mixer was able to mix. The brick couplets had to be level so to prevent any additional moment being induced from the vertical load placed on the specimens.

The mortar thickness used for all specimens was 10mm which complied with the specified thickness of between 8 and 15 mm recommended by the (European Standard, 2002).

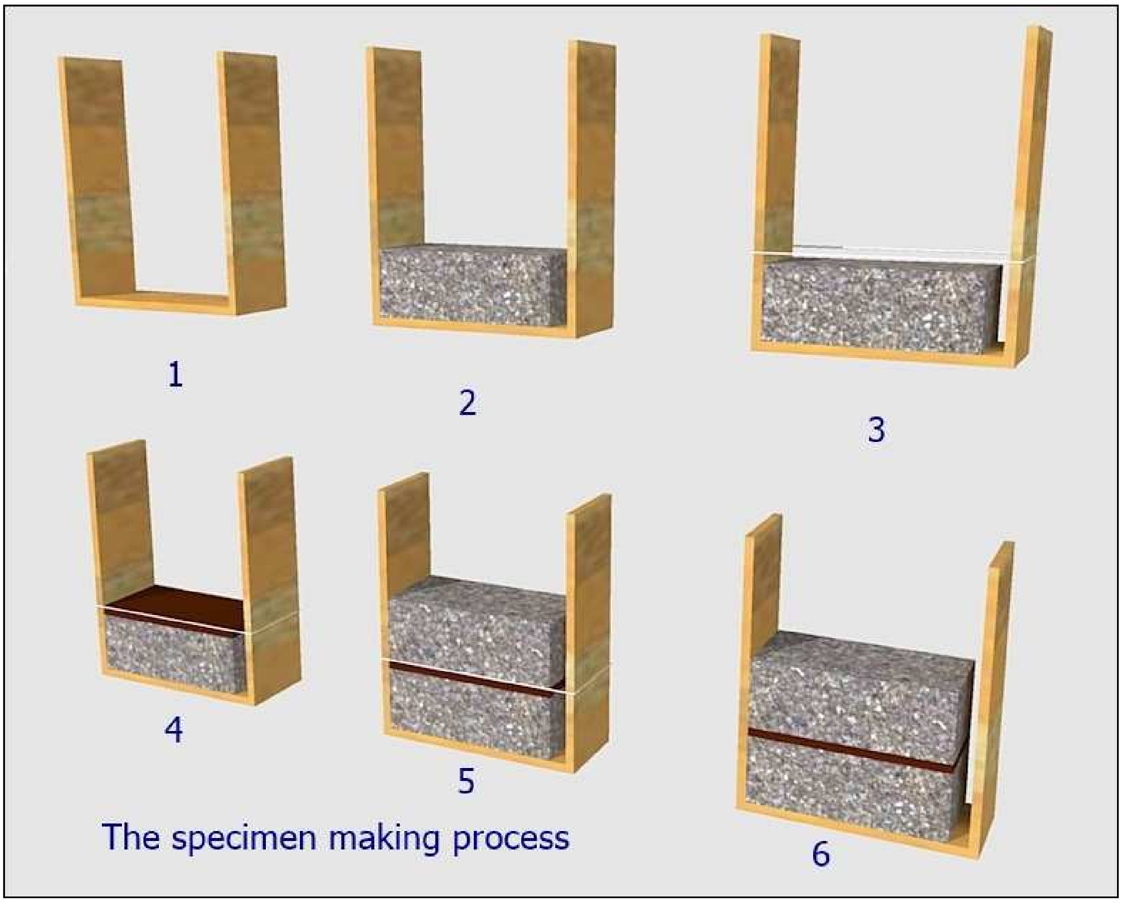


Figure 3-20: The couplet making process

In order to achieve a 10 mm thickness and ensure that the top of the couplet is level since all the bricks are not exactly the same. A wooden device was used as shown in figure 3-20.

1. The wooden device was placed on a level surface, the horizontal part of this device is 90 degrees to the vertical part this device has a width of 100mm (the same as an average brick), however its length is greater than the length of a brick to provide sufficient work space and ease of removal of specimen .(Figure 3-20(1))
2. The pre-wet brick was placed on the wooden device. (Figure 3-20(2))
3. A thin string was placed 10 mm above the brick on all sides and is made as level as possible. (Figure 3-20(3))
4. The mortar was applied to the bottom brick then levelled. The thin string makes it easy to level the mortar at a thickness of 10mm. (Figure 3-20(4))
5. The top brick was placed on top, the specimen was lightly tapped to ensure full contact between the brick and the mortar, it was then verified that the specimen was level. (Figure 3-20(5))
6. The string was carefully removed without disturbing the specimen. (Figure 3-20(6))

The couplet specimen was then removed and placed on a board nearby. Once all specimens were produced, a load was placed on top of each specimen in order to model the conditions on site i.e. the subsequent bricks that are placed on top of this line of bricks before the mortar dries. As soon as the mortar dries it has no affect but if the mortar is still wet the extra weight assists the mortar in penetrating further into the pore structure of the bricks anchoring itself, thereby increasing the shear strength. One has to bear in mind that the cement-stock bricks have more and bigger pores than the stock bricks.

Due to the availability of weights, 7 kg were placed on top of three specimens equating to the pressure of one line of bricks.

The specimens were covered with plastic sheeting for 24 hours to allow them to cure in a controlled environment. The specimens were then cleaned marked on the top brick, and placed in its respective curing regime, air, water or the solar chamber,(described in chapter 4), for the required number of curing days, 7, 14, 28 and 44 days.

The specimens cured in air and in the solar chamber were removed from the curing regime within an hour prior to testing. To the contrary, the specimens cured in water were removed 24 hours before testing to allow them to dry. When one tests the water specimens while wet, the results cannot be compared with that of specimens cured in air and in the specially designed solar chamber as the bricks have variable moisture content. Moreover, when testing specimens while still wet, it was noticed that the bricks tended to be more brittle. This would lead to a foreseeable risk of getting incomparable results concerning the shear strength.

4. TEST SPECIMENS:

In the initial phase of testing specimens were produced and cured in water at 7 and 28 days. From the initial results it was noted that there was a direct correlation between the failure load and the deflection. It was noted that most of the specimens reached the maximum load at a deflection of approximately 10 mm.

Shear load versus deflection curves were produced and a distinctive pattern was found. From the preliminary results there were large deviations in the results, between the specimens so it was decided that six specimens would be produced for each test undertaken instead of three. This gave more consistent results.

4.1 Curing regimes

The specimens were cured in 3 different curing regimes (in summer) namely water, air and a specially designed solar chamber.

The solar chamber was used to obtain the effect of a hot humid moist environment. This sort of environment is seen in South Africa in Durban, KwaZulu-Natal. According to the literature a more humid environment should increase the shear bond strength.

The solar chamber is a rectangular plastic tank of dimensions; length 1300 mm, breadth 600 mm and height 700 mm; with a glass lid of average thickness 0.9 mm. The chamber is white in colour, to reflect the rays of the sun and keep the specimens at a uniform temperature during extreme hot weather conditions. Gravel of average size 19 mm were placed at the bottom of the chamber to a depth of about 200 mm and potable water 100 mm underneath to keep a higher constant relative humidity inside the chamber. The chamber was tightly sealed after placing the specimens inside.

Specimens were cured in water in a plastic curing tank under controlled conditions in the lab. This models shear stresses on a wall that retains water.

The specimens that were cured in air were sealed in a plastic bag to produce a controlled environment. Specimens were cured in Air to attempt to model the site conditions in the Gauteng environment.

4.2 Vertical applied load (pre-compression load)

Four load configurations (pre-compression loads) were used; Figure 4-1 is used to indicate the physical representation of the loads on a 3.5 m height wall. The first load configuration is used to obtain the shear stresses between the first two lines of bricks. The second load configuration is used to obtain the shear stresses at a metre depth. The third load configuration is used to obtain the shear stresses at two metres depth. The fourth load configuration is used to obtain the shear stresses at three metres depth, see Figure 4-1 and Table 4-1.

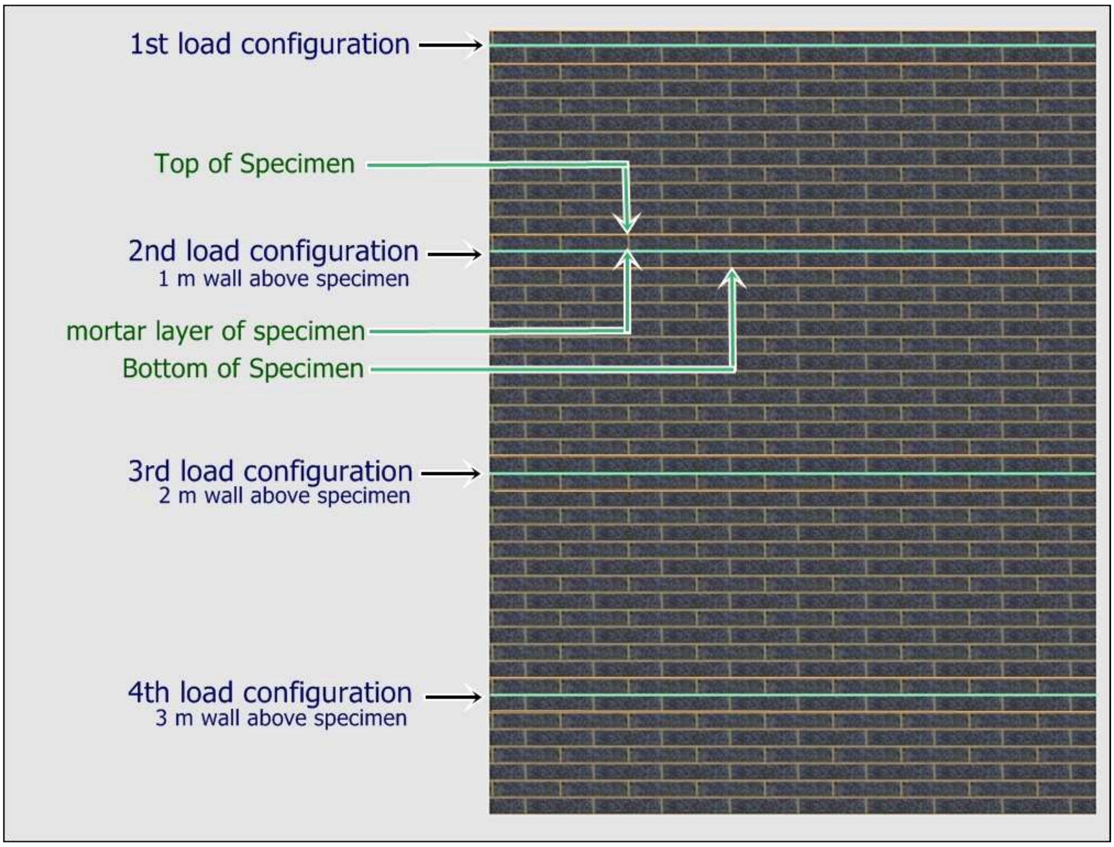


Figure 4-1: The vertical pre compression load as a height of wall

These heights were obtained using the design load for the density of masonry of 2200 kg/m^3.The density of the specimen is calculated as 1500 kg/m^3.

These vertical loads were chosen due to availability of equipment as a well as to see the effect of using smaller vertical pre-compression loads on the friction factor (μ) obtained compared to using larger loads as suggested by the Eurocode.

In load configuration 1, no vertical load is applied to the specimen.

Table 4-1: It gives the number of rows of brick and mortar that is represented by each vertical load.

Correlation of vertical weight to height of wall above application of horizontal force					
Load configuration	Applied vertical force		Rows of bricks+mortar	Height of brick+mortar	Height of wall
2	47 kg	0.46 kN	12.47 rows	0.08 m	1.00 m
3	97 kg	0.95 kN	25.75 rows	0.08 m	2.06 m
4	147 kg	1.44 kN	39.02 rows	0.08 m	3.12 m

From the load configurations above one could obtain the shear stresses due to a shear force at any position in the wall i.e. any height from the top of the wall by interpolation.

4.3 Test Direction

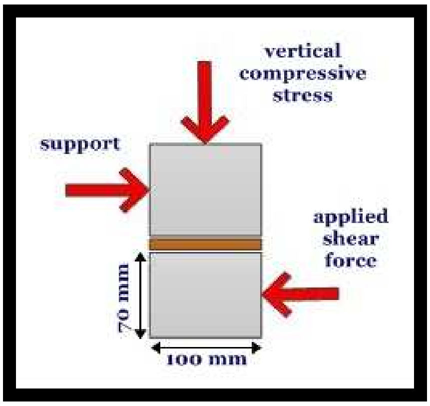


Figure 4-2: Apparatus used to test specimens in shear

The literature review (chapter 2) revealed that the horizontal shear force was always applied in the longitudinal direction of the bottom brick of the couplet specimen; i.e. Test direction A, see Figure 3-1. In chapter 3 it was revealed that test direction A can be used to simulate the shear force acting perpendicular to the long face of a double wall as seen in Figure 3-5.

Specimens were produced and tested applying the horizontal shear force perpendicular to the longitudinal direction of the bottom brick of the couplet specimen as show in Figure 4-2 this shall be denoted as test direction B. Test direction B can be used to simulate the effect of shear stresses acting perpendicular to the long face of a single wall (Load L acting on surface area B) i.e. a single wall subject to lateral loads, see Figure 4-3.

Single walls are commonly used in bathrooms, kitchens, balconies as well as other internal walls in buildings.

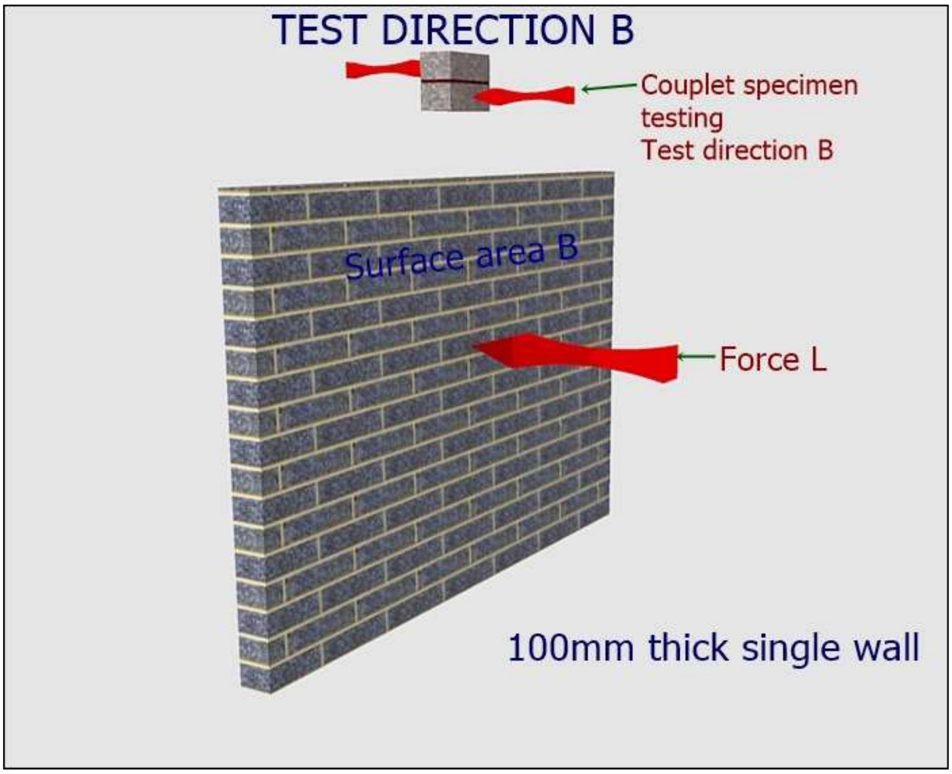


Figure 4-3: A schematic representation showing how Test direction B models shear stresses acting on a single wall, modelling a 210 mm portion of a single wall (test direction B)

4.4 Specimens produced for testing:

4.4.1 Test specimens produced:

Table 4-2 shows all the specimens that were produced for testing. The curing regimes, load configuration as well as the test directions have been previously explained in the chapter. The change in curing age of the specimen as well as the change in water cement ratio is described below:

- It was decided to test specimens at different ages in order to see how the shear strength varies with time, i.e. at what age is the Peak shear bond strength achieved? At what age can I apply loads to the wall and the approximate magnitude of these loads? These questions are useful if one is in the construction business he knows at what age he can apply loads to a load bearing wall.
- The effect of reducing the water to cement ratio from the optimum water to cement ratio, to see if it would increase the strength as it does in compression. From the literature it should decrease the shear strength because there is less water available to allow the cement to be deposited in the pore structure of the bricks. The expected result would be a decrease in shear strength. The Water cement ratio was reduced for one batch of specimens.

Table 4-2: Specimens Produced for testing

Curing regime	Load configuration	Vertical load	curing age (days)	Water/ cement ratio	testing direction models	No. of specimens
AIR	1	Self weight	7,14,28,44	4/3	Test Direction A (double wall)	24
	2	50Kg	7,14,28,44			24
	3	100kg	7,14,28,44			24
	4	150kg	7,14,28,44			24
	1	Self weight	28	7/6	Test Direction A (double wall)	6
	2	50Kg	28			6
	3	100kg	28			6
	4	150kg	28			6
	1	Self weight	28	4/3	Test Direction B (single wall)	6
	2	50Kg	28			6
	3	100kg	28			6
	4	150kg	28			6
Water	1	Self weight	28	4/3	Test Direction A (double wall)	6
	2	50Kg	28			6
	3	100kg	28			6
	4	150kg	28			6
Solar chamber	1	Self weight	28	4/3	Test Direction A (double wall)	6
	2	50Kg	28			6
	3	100kg	28			6
	4	150kg	28			6
Total number of Specimens						192

4.4.2 Testing schedule:

Table 4-3: Schedule of producing and testing specimens

Plastic W/C 4/3	7-days			14-days			28-days			44-days		
	set of	cast date	test date	set of	cast date	test date	set of	cast date	test date	set of	cast date	test date
Self Weight	6	17	24	6	7	20	6	1	28	6	1	44
50kg	6	18	25	6	8	21	6	2	29	6	2	45
100kg	6	19	26	6	9	22	6	3	30	6	3	46
150kg	6	20	27	6	10	23	6	4	31	6	4	47

Water W/C 4/3	28-days		
	set of	cast date	test date
Self Weight	6	5	32
50kg	6	6	33
100kg	6	7	34
150kg	6	8	35

Solar W/C 4/3	28-days		
	set of	cast date	test date
Self Weight	6	9	36
50kg	6	10	37
100kg	6	11	38
150kg	6	12	39

Plastic W/C 7/6	28-days		
	set of	cast date	test date
Self Weight	6	13	40
50kg	6	14	41
100kg	6	15	42
150kg	6	16	43

Above is a time schedule for producing specimens it is indicated in blue and runs for 20 continues days. The testing of the specimens starts on day 20 and runs till day 47. The above table does not include the production and testing of specimens for the testing which models single walls (Test direction B) as this was undertaken much later.

5. RESULTS AND DISCUSSION:

5.1 Failure patterns

5.1.1 Introduction

In chapter 2 it was revealed that the characteristic shear strength equation (Equation 2-1) used in design is obtained from specimens that have failed in shear bond failure during testing. The validity of using this equation in design depends on the frequency that the specimen will fail in shear bond failure. A wall that fails at the shear bond can be repaired easier than one that fails through the brick. Provided shear bond failure occurs the position in a wall where it is most likely to occur can be predicted see section 5-1-7. The factors that affect the frequency of shear bond failure occurring becomes important.

The factors investigated were:

1. The age of the specimen at testing, section 5-1-2.
2. The effect of the curing environment, section 5-1-3.
3. Varying the Water cement ratio, section 5-1-4.
4. The Test direction (see section 4-2), section 5-1-5.

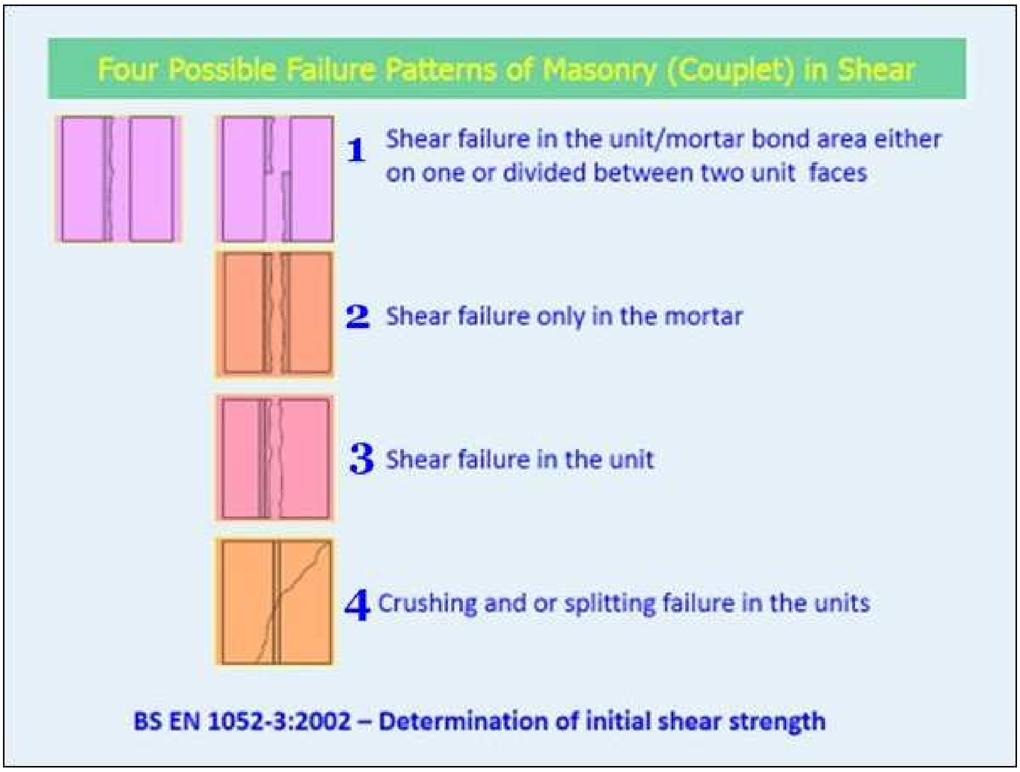
Shear failure between the bond between the brick and mortar will be denoted as failure pattern 1, see Table 5-1-1

Shear failure in the mortar only will be denoted as failure pattern 2, see Table 5-1-2.

Shear failure in the unit will be denoted as failure pattern 3, see Table 5-1-3.

Crushing, Splitting of the specimen will be denoted as failure pattern 4, see Table 5-1-3.

Table 5-1 shows the likely failure patterns of the specimens in shear from BS EN 1052-3:2002.



5.1.2 Frequency of failure patterns occurring according to age of specimen at testing.

Specimens were produced and cured in air for 7, 14, 28 and 44 days before testing (see table 4-2, rows 1-4). These specimens were tested in test direction A see Figure 3-5.

All specimens tested at 7 and 14 days failed according to failure pattern 1(see Table 1-1), i.e. shear bond failure. This result did not vary with change in load configuration (vertical pre-compression load) see Figure 4-1.

The specimens tested at 28 days failed according to failure patterns (see figure 5-1):

1. Failure pattern 1 with an average frequency of occurrence of 61 percent
2. Failure pattern 4 with an average frequency of occurrence of 35 percent
3. Failure pattern 3 with an average frequency of occurrence of 4 percent

For 28 day testing the frequency of failure pattern 1 occurring increased with increase in vertical pre-compression load see Figure 5-1. The increase in vertical load counteracts the applied moment during testing, the applied moment that would be caused in a wall experiencing shear, forcing the load to act horizontally and not at an angle as the moment would induce.

Initially at 7 and 14 days the bond between the brick and the mortar is weaker than the shear strength of the brick or the shear strength of the mortar. As the bond strength increases, at 28 days an above, the specimen reaches a point where it acts as a single unit resulting in failure by failure pattern 4. This indicates that the specimens tested at 7 and 14 days have not yet attained their maximum strength.

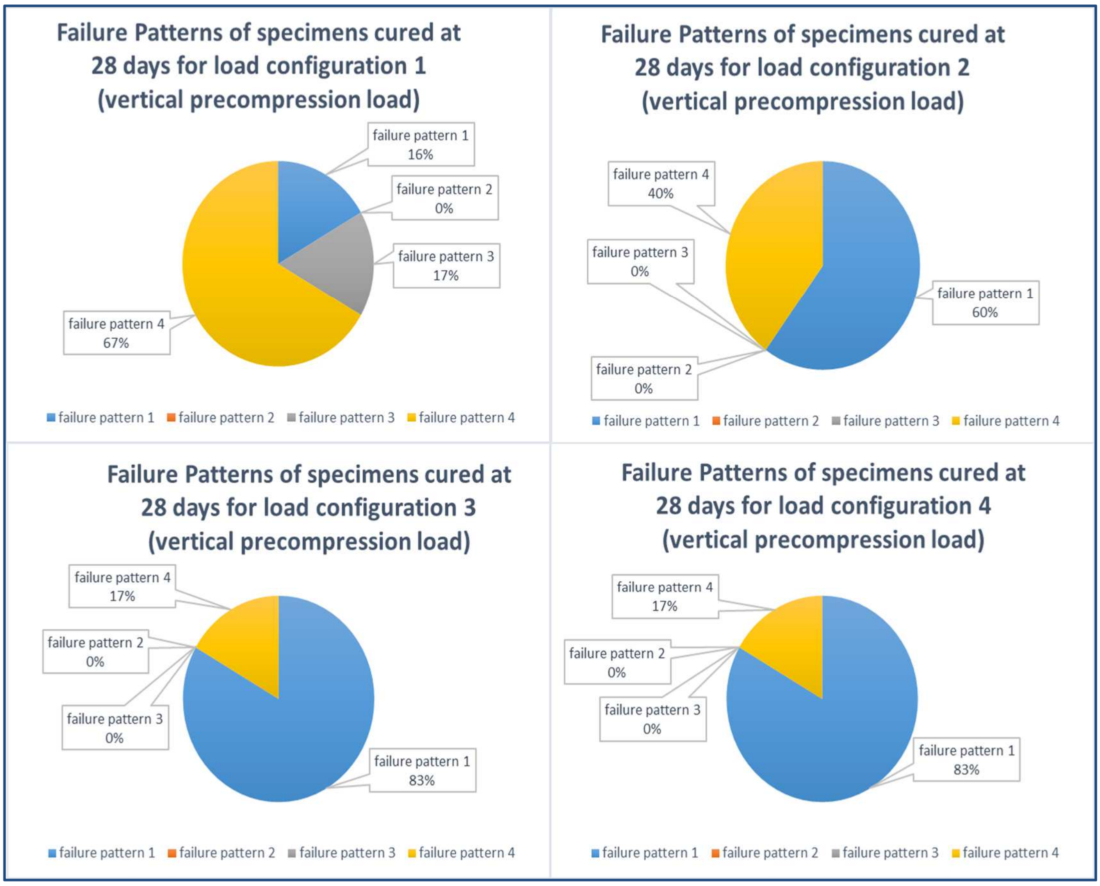


Figure 5-1 Frequency of a certain failure pattern occurring for specimens tested at 28 days for each load configuration used in testing.

The specimens tested at 44 days failed according to failure patterns:

1. Failure pattern 1 with an average frequency of occurrence of 82 percent
2. Failure pattern 4 with an average frequency of occurrence of 18 percent

Similar to testing at 28 days the frequency of failure pattern 1 occurring increased with increase in vertical pre-compression load see Figure 5-2, for testing at 44 days, explained above.

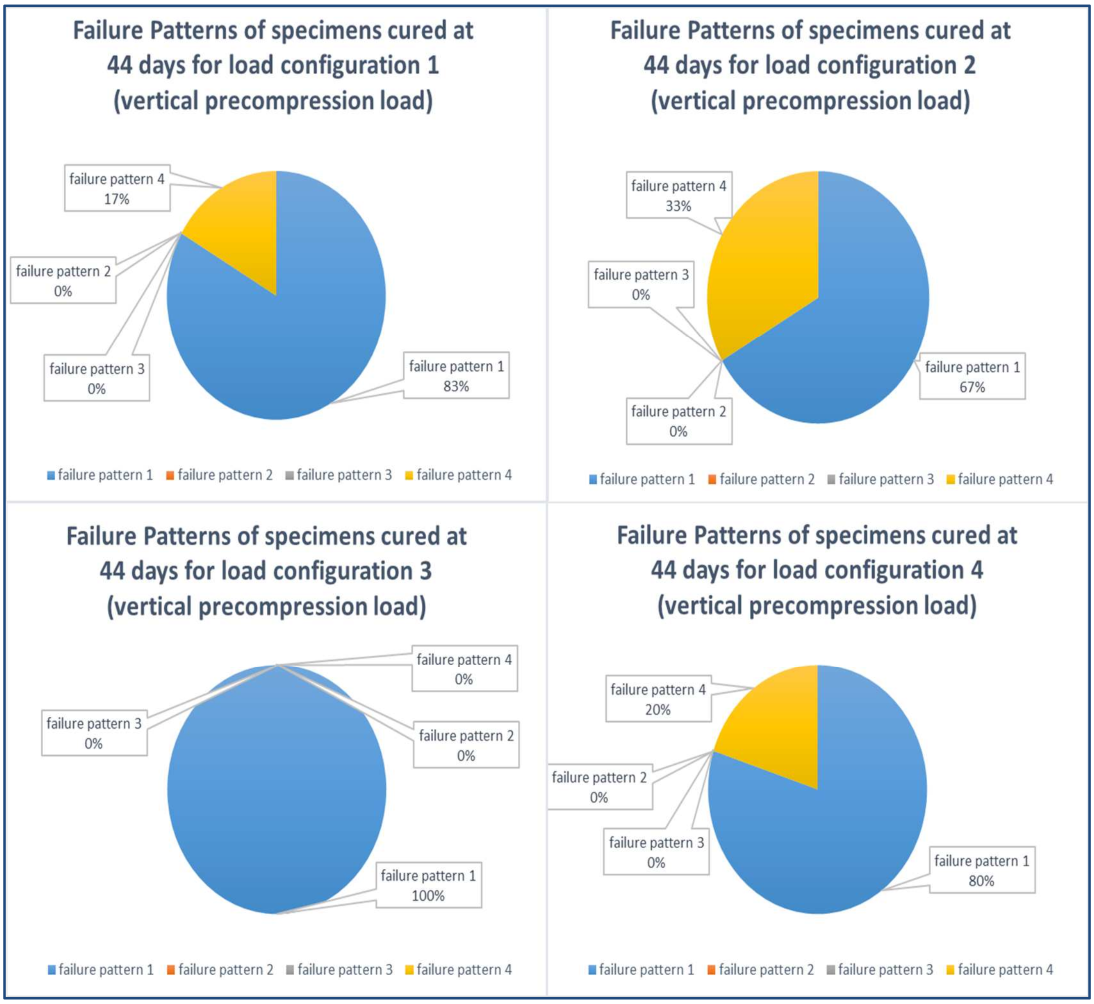


Figure 5-2 Frequency of a certain failure pattern occurring for specimens tested at 44 days for each load configuration used in testing.

The frequency of occurrence of failure pattern 1 decreases with increase in age to an average frequency of 72 percent. After 28 days it remains the most probable failure pattern. The second most probable failure pattern is failure pattern 4.

The frequency of failure pattern 1 occurring increases with increase in vertical load for specimens tested in Test direction A.

5.1.3 Frequency of failure patterns occurring according to Curing regime

Specimens were produced and cured in air, water and the solar chamber for 28 days before testing (see table 4-2, rows 1-4 and rows solar chamber). These specimens were tested in test direction A see Figure 3-5.

For results on the specimens cured in air see section 5.1.2, Figure 5-1.

All specimens cured in water and the solar chamber failed according to failure pattern 1(see Table 5-1), i.e. shear bond failure. This result did not vary with change in load configuration (vertical pre-compression load) see Figure 4-1.

Hence curing in an environment with a high moisture content increases the frequency of occurrence of failure pattern 1 (shear bond failure). After the specimens (that were cured in water and the solar chamber) were tested the mortar felt moist. The specimens cured in air that was tested at 28 days and 44 days did not feel as moist at the bond between the brick and the mortar. Curing in an environment with high moisture content does not allow the mortar to dry at the bond, making the bond the weakest position in the specimen (specimens cured in water and the solar chamber).

5.1.4 Frequency of failure patterns occurring according to change in mix design.

Specimens were produced and cured in air, using two different water to cement ratios were tested at 28 days, (see table 4-2, rows 1-4 and rows 5-8). These specimens were tested in test direction A see Figure 3-5.

For results on the specimens produced with a water to cement ratio of 4/3 see section 5.1.2, Figure 5-1.

All specimens produced with a water cement ratio of 7/6 failed according to failure pattern 1(see Table 5-1), i.e. shear bond failure. This result did not vary with change in load configuration (vertical pre-compression load) see Figure 4-1.

This was expected; there is less water, so the hydrated cement products are not easily transported to the pores of the bricks, producing a weaker bond.

5.1.5 Frequency of failure patterns occurring according to test direction.

Specimens were produced and cured in air for 28 before testing (see table 4-2, rows 1-4 and rows 9-12). These specimens were tested in test direction A see Figure 3-5 and test direction B see section 4-3.

For results on the specimens tested in test direction A at 28 days see section 5.1.2, Figure 5-1.

The specimens tested in test direction B failed according to failure patterns (see figure 5-3):

1. Failure pattern 1 with an average frequency of occurrence of 72 percent
2. Failure pattern 4 with an average frequency of occurrence of 28 percent

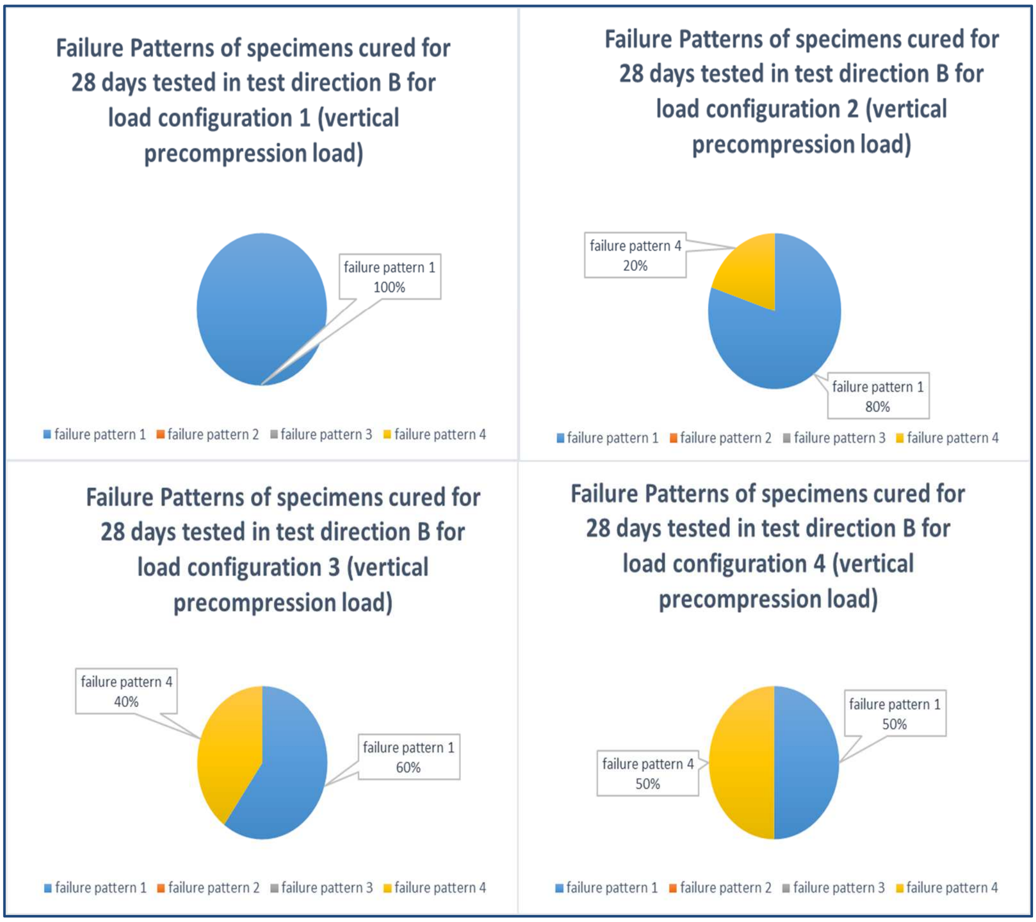


Figure 5-3 Frequency of a certain failure pattern occurring for specimens tested at 28 days in test direction B for each load configuration used in testing.

For testing in test direction B the frequency of failure pattern 1 occurring decreased with increase in vertical pre-compression load see Figure 5-3. Unlike the specimens tested in test direction A, the increase in vertical load in this case assists the applied moment during testing, the applied moment that would be caused in a wall experiencing shear, forcing the load to act at an angle. This indicates that the compressive load sits on the specimen with a greater stability when testing in test direction A than test direction B.

Test direction B failed 72 percent of the time in shear bond failure, whereas test direction A failed 60 percent of the time in shear bond failure. Testing in test direction A yields higher shear strength values and more brittle failure (failure pattern 4).

5.1.6 Summary of frequency of failure patterns for all specimens tested.

For all specimens tested, Failure pattern 1 i.e. shear bond failure is the most probable failure pattern with a frequency of occurrence of 89 percent. The second most probable failure pattern is failure pattern 4 with a frequency of occurrence of 10 percent, failure pattern 3 has a frequency of occurrence of 1 percent. None of the specimens failed with failure pattern 2 (shear failure through the mortar), this is probably due to the thickness of the mortar.

This frequency of occurrence of shear bond failure of 89 percent validates the use of the characteristic shear strength equation (equation 2- 1) in design.

5.1.7 Interesting property of the shear bond failure found during testing which can be used to predict planes of weakness in a wall.

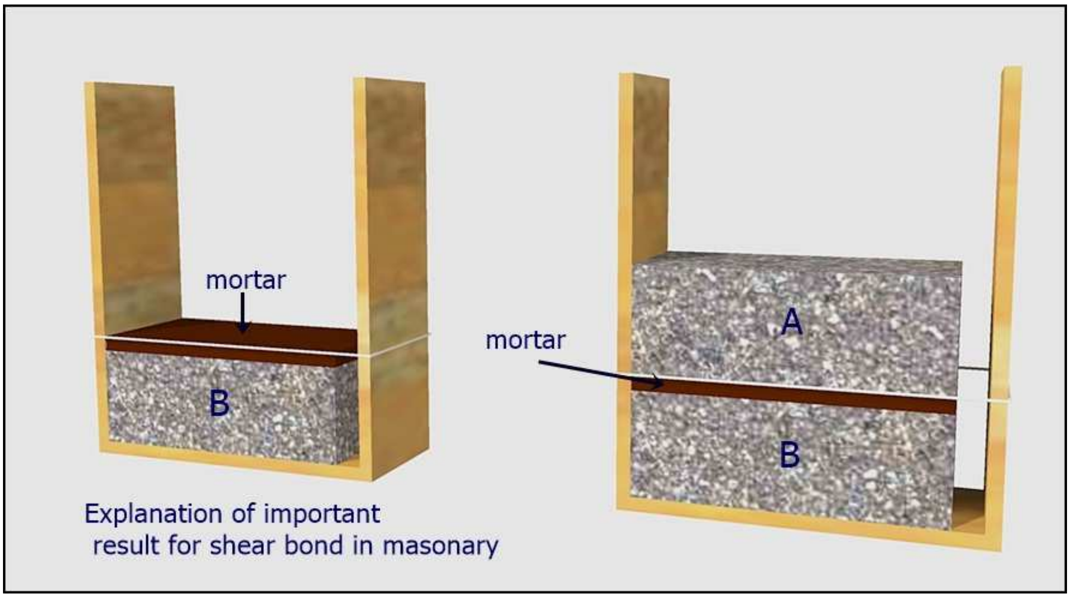


Figure 5-4 Picture used to illustrate the specimen making process.

When producing the specimens, brick A in Figure 5-1 above was always marked, brick A being the top brick as marked see Figure 5-4. During testing for *all specimens* tested it was noted that if the specimen failed with failure pattern 1, i.e. shear bond failure it would always fail at the bond between brick A and the mortar. Hence the bond between brick B and the mortar was stronger than that between brick A and the mortar.

To verify that this was not due to the method of testing or the apparatus, several specimens were tested inverted .i.e. the horizontal force was applied to Brick A instead of Brick B. This did not alter the result .i.e. the bond between brick A and the mortar is weaker than the bond between brick B and the mortar

The reasons for the occurrence of this phenomenon (refer to figure 5-4):

1. When producing the specimen brick B is placed on the wooden device (section 3-4), using a trowel mortar is applied to the top of brick B, with the assistance of gravity and the pressing of the mortar on to the surface of brick B with the trowel the mortar is allowed to protrude into the surface of brick B, from the literature the shear bond strength is developed from the mechanical interlocking of the mortar into the pore structure of the brick. Brick A is placed on top of the mortar and tapped this allows the mortar to further protrude into brick B .Gravity works against the bond between brick A and the mortar, the mortar is not pressed on to brick A, hence the result that the mortar does not protrude the same amount into the pore structure of brick A as it protrudes into Brick B, creates a stronger bond between Brick B and the mortar than brick A and the mortar.
2. After the mortar is placed onto brick B the top surface of the mortar is exposed to the air for the period before brick A is placed on top, this allows the surface of the mortar to dry before Brick A is placed on top. Water at the surface allows the mortar to travel into the pores of brick the brick. Since there is less water on the surface of the mortar applied to brick A than on the surface of the mortar when applied to brick B, Once again the mortar cannot protrude into the pore structure of brick A as it protrudes

into the pore structure of brick B using water as a means of transportation. If an adhesive begins to dry it does not bond as well.

3. There is a smaller air content on the surface of brick B than that of the surface of brick A, in brick B more of the air contained in the pore structure is replaced by mortar than that in brick A. According to the literature the greater the air content the weaker the bond between the brick and the mortar.

This result is dependent on the frequency of failure pattern 1(shear bond failure) occurring. This is the second reason the frequency of failure pattern 1 was discussed in detail in the last section, see table 5-1.

The Minimum frequency of occurrence of failure pattern 1 is 16 percent and it is for specimens tested under load configuration 1, i.e. in the first 2 rows in a wall at a curing age of 28 days, see figure 4-1. With increase in load this frequency increases to an average frequency of failure of 61 percent. The average frequency of failure for all specimens tested at ages greater than 28 days (in test direction A) is 88 percent. The average frequency of failure for all specimens tested at ages greater than 28 days (test direction B) is 72 percent. An increase in moisture content of the curing environment increases the frequency of failure pattern 1 occurring, see table 5-1.

The average frequency of occurrence for all specimens tested was 89 percent however this is not a true average (not all specimens were tested at 28 days or more than 28 days). The average for all specimens tested at 28 days (or more than 28 days) is 86 percent, .Hence the likelihood of occurrence of failure pattern 1 is 86 percent.

The result that the bond between brick B and the mortar is stronger than that between brick A and the mortar is independent of the age of the specimen, the curing regime, the mix design, the test direction, or the load configuration (the vertical applied load) (see figure 4-1),see figure 5-4. Since this result is independent on the load configuration (vertical load), i.e. the position in the wall where shear failure is takes place does not matter. From this result one can conclude that the wall in figure 5-5 is likely to fail at the failure planes (planes of weakness in a wall), shown in white in figure 5-6, 86 percent of the time when exposed to shear forces. (These failure planes represent failure at the brick mortar bond).

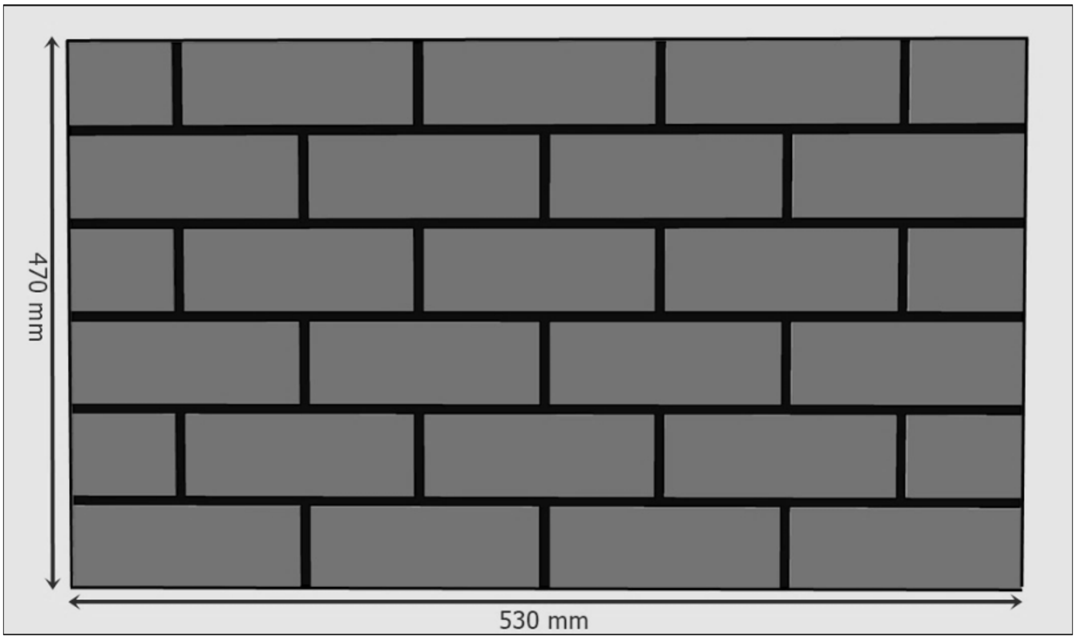


Figure 5-5 Example of a masonry wall bonded by mortar

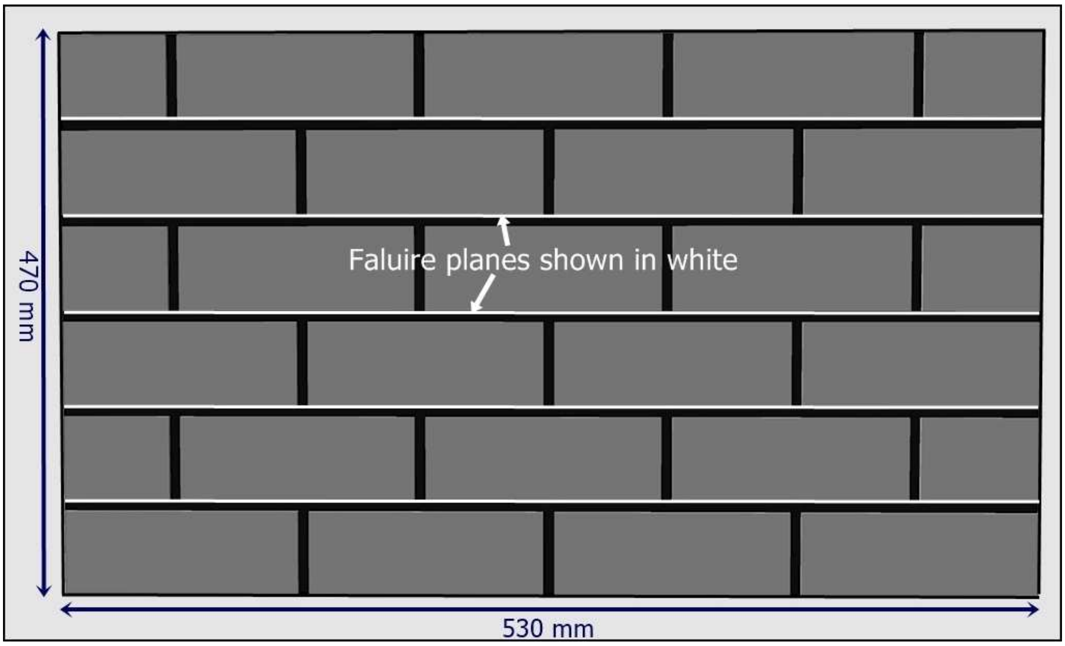


Figure 5-6 Failure planes of a masonry wall bonded by mortar

If we consider a wall that is built from left to right as shown in Figure 5-7, the failure planes (planes of weakness in the wall) are shown in white in Figure 5-8. The frequency of occurrence is 86 percent.

Figure 5-7 A masonry wall constructed from left to right.

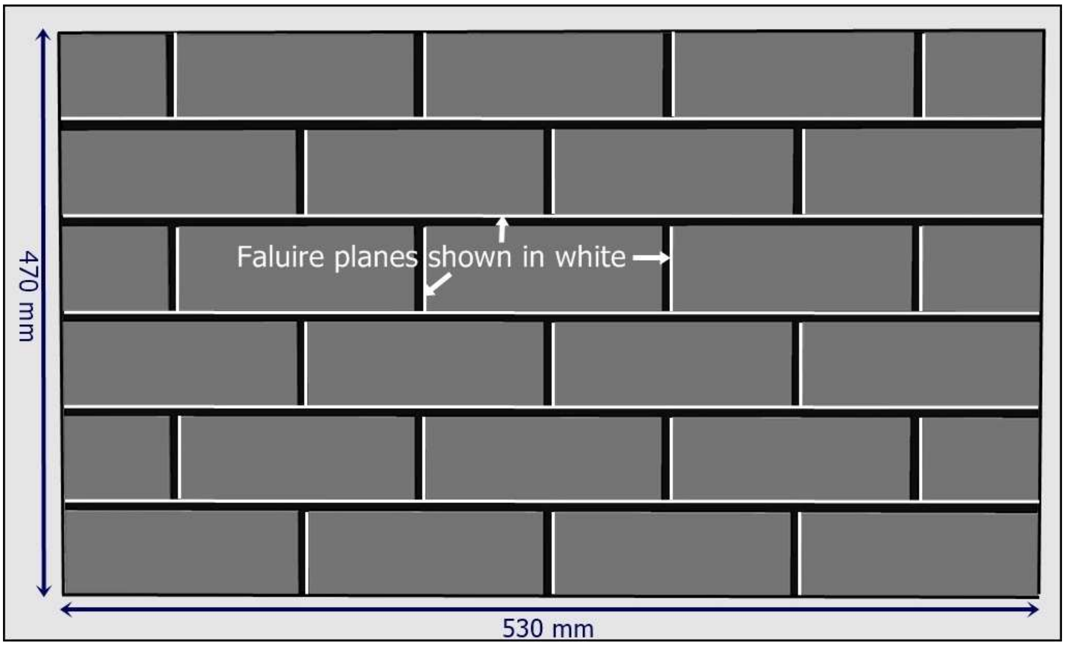


Figure 5-8 Failure planes of a wall constructed from left to right

The positions in a wall where the bricks are attached to the mortar (when constructing the wall), produces a weaker bond than where the mortar is applied to the bricks. Hence these positions become the weakness planes of a wall exposed to shear forces.

In conclusion the failure patterns above have shown that failure pattern 1 is the most probable failure pattern which validates the use of the characteristic shear strength equation in design. From the results one can easily predict where a wall is most likely to fail if exposed to shear forces. This can be useful in the rehabilitation of a wall. However these results are limited as only one type of brick and mortar unit were used. Since this is a useful result and can have practical implications, it is suggested that one should test specimens using different types of mortar units to see if it renders the same result.

5.2 deflection at max shear stress and stress strain curve

From the initial tests it was noted that all specimens failed at a deflection of about 10mm.The deflection at the max shear stress gives one an indication of the maximum displacement the specimen can undergo and after this point of displacement it can no longer resist the applied load. It also gives an indication of the magnitude of the differential displacement that a wall can undergo until it can no longer resist the applied shear stresses. This is an interesting result specimens were tested to identify factors which affect this result.

The factors investigated were:

1. The age of the specimen at testing.
2. The effect of the curing environment.
3. Varying the Water cement ratio.
4. The Test direction (see section 4-2).

For Specimens tested in test direction A (figure 3-5) at 44 days, (Table 4-2 rows 1-4).The average deflections of the specimen at failure were (see Figure 3-9):

1. For load configuration 1 the average deflection at failure is 9.40 mm with a standard deviation of 1.39mm.

2. For load configuration 2 the average deflection at failure is 9.82 mm with a standard deviation of 1.51mm.
3. For load configuration 3 the average deflection at failure is 10.89 mm with a standard deviation of 0.70mm.
4. For load configuration 4 the average deflection at failure is 9.28 mm with a standard deviation of 1.33mm.

It may be concluded that the deflection at failure does not vary with change in load configuration (vertical compression), i.e. this result is valid for the entire height of wall. The average deflection is found to be 9.85 mm, which is close to the 10mm expected failure.

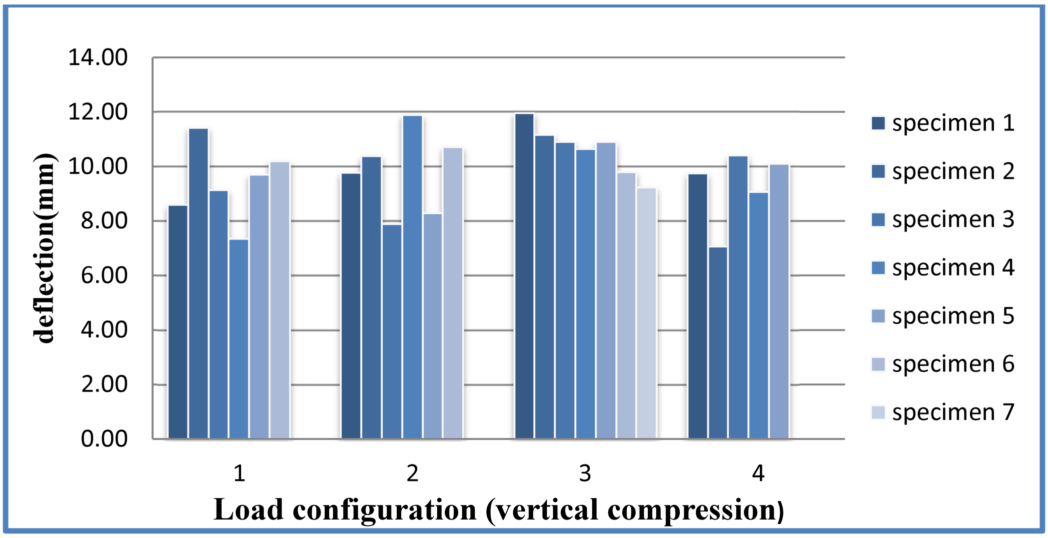


Figure 5-9 Deflection at Max Shear stresses for 44 day testing (test direction A) for each load configuration

5.2.1 Effect of Change in Test direction (see section 4-3).

For Specimens tested in test direction B (section 4-3) at 28 days, (Table 4-2 rows 9-12).The average deflections of the specimen at failure were (see Figure 3-10):

1. For load configuration1 the average deflection at failure is 13.73 mm with a standard deviation of 3.26 mm.
2. For load configuration 2 the average deflection at failure is 14.37 mm with a standard deviation of 1.51 mm.

3. For load configuration 3 the average deflection at failure is 9.96 mm with a standard deviation of 2.34 mm.
4. For load configuration 4 the average deflection at failure is 9.75 mm with a standard deviation of 2.44 mm.

One can conclude that the deflection at failure does not vary much with height. The average deflection is found to be 11.95 mm, which is larger than the 10mm expected failure.

The change in test direction from test direction B to test direction A increases the deflection at failure to an average of 12 mm. Hence a single wall deforms more at failure than a double wall.

For test direction B the results vary and the averages tend to be greater than 10 mm. However one can still conclude that after 10 mm deflection the wall can no longer resist any applied stresses (see Figure 3-10).

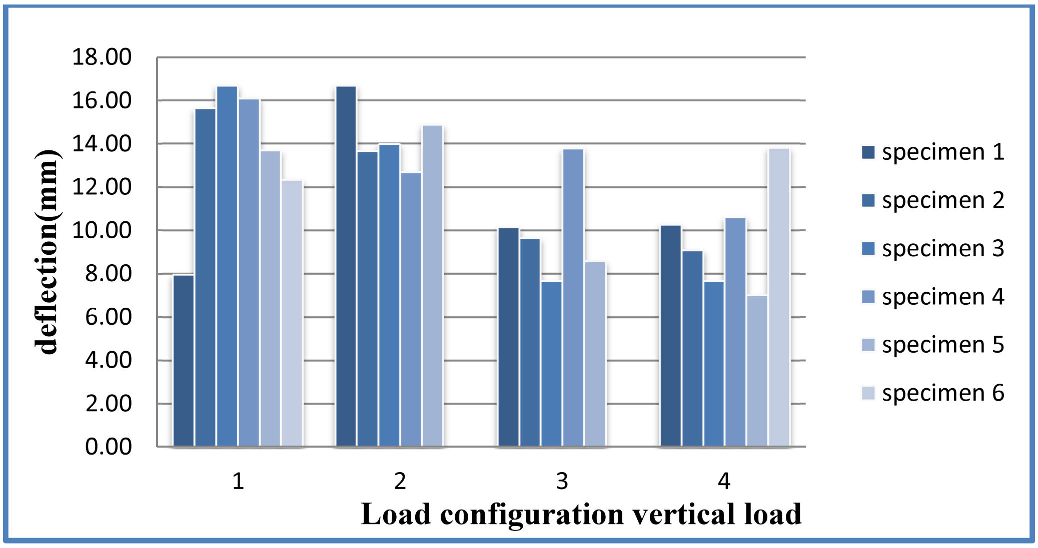


Figure 5-10 Deflection at Max Shear stresses for 28 day testing (test direction B) for each load configuration

5.2.2 Effect of Change in age of specimen.

The specimens tested at 7 and 14 days show an average deflection of 8.8 mm. As the specimen ages this deflection increases to an average of 11 mm, see figure 5-

11. A wall that is at age below 28 days can no longer resist external forces if it deforms greater than 8 mm

The first graph is used to show that the result of 10 mm deflection is not sensitive to age; the data gives an average deflection value of 10.57 mm with a standard deviation of 2.85 mm, see figure 5-11.

5.2.3 Effect of using a weaker mix.

The specimens tested with a reduced water cement ratio had a weaker brick mortar see section 5-5 .These specimens gave an average deflection value of 8.34 mm with a standard deviation of 1.47 mm, see Figure 5-12. From this result one can conclude that the stronger the brick mortar bond, the greater the deflection at failure.

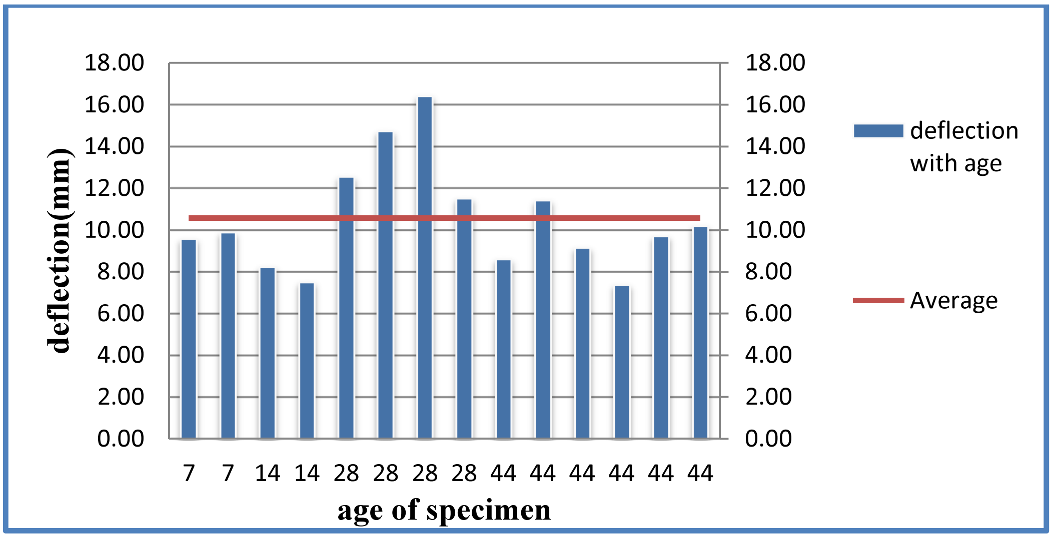


Figure 5-11 Deflection at Max Shear stresses for different curing ages

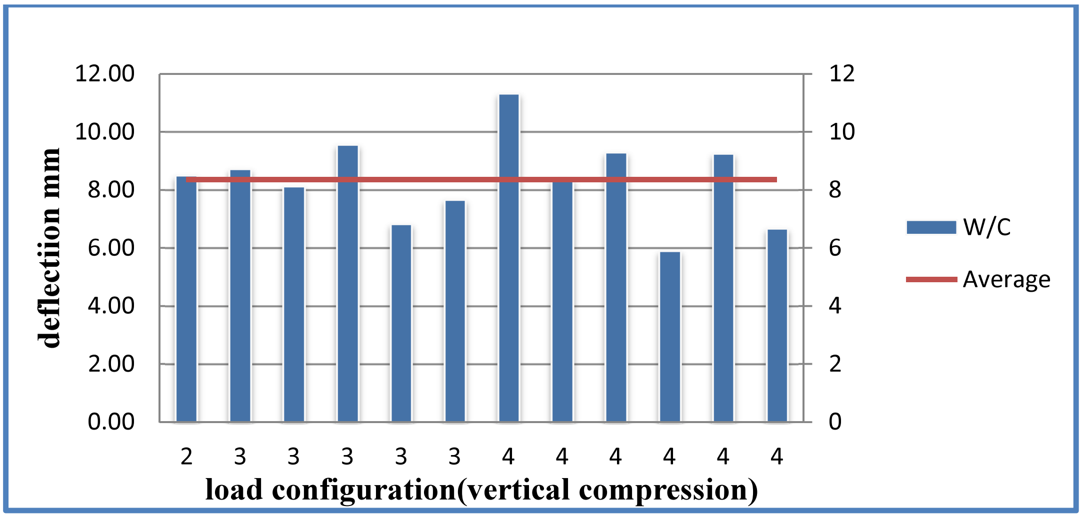


Figure 5-12 Deflection at Max Shear stresses using a water/cement ratio of 7/6 for different load configurations.

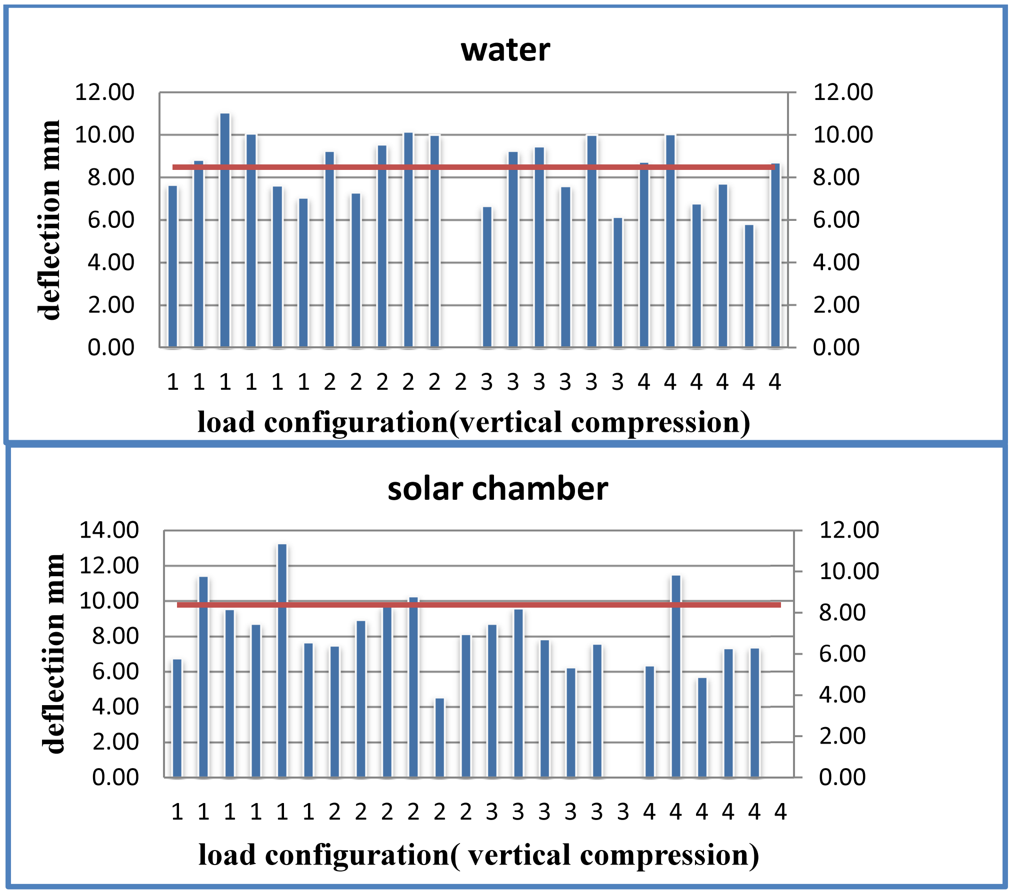


Figure 5-13 deflection at Max Shear stresses for water and solar chamber curing regimes

5.2.4 Effect of curing environment

Curing in a moist environment decreases the deflection at failure. The average deflection at failure for specimens cured in water was 8.49 mm with a standard deviation of 0.68 mm. The average deflection at failure for specimens cured in the solar chamber was 8.4 mm with a standard deviation of 1.74 mm, see Figure 5-13.

From these results one can conclude that after 8 mm displacement a wall can no longer resist the applied shear stresses.

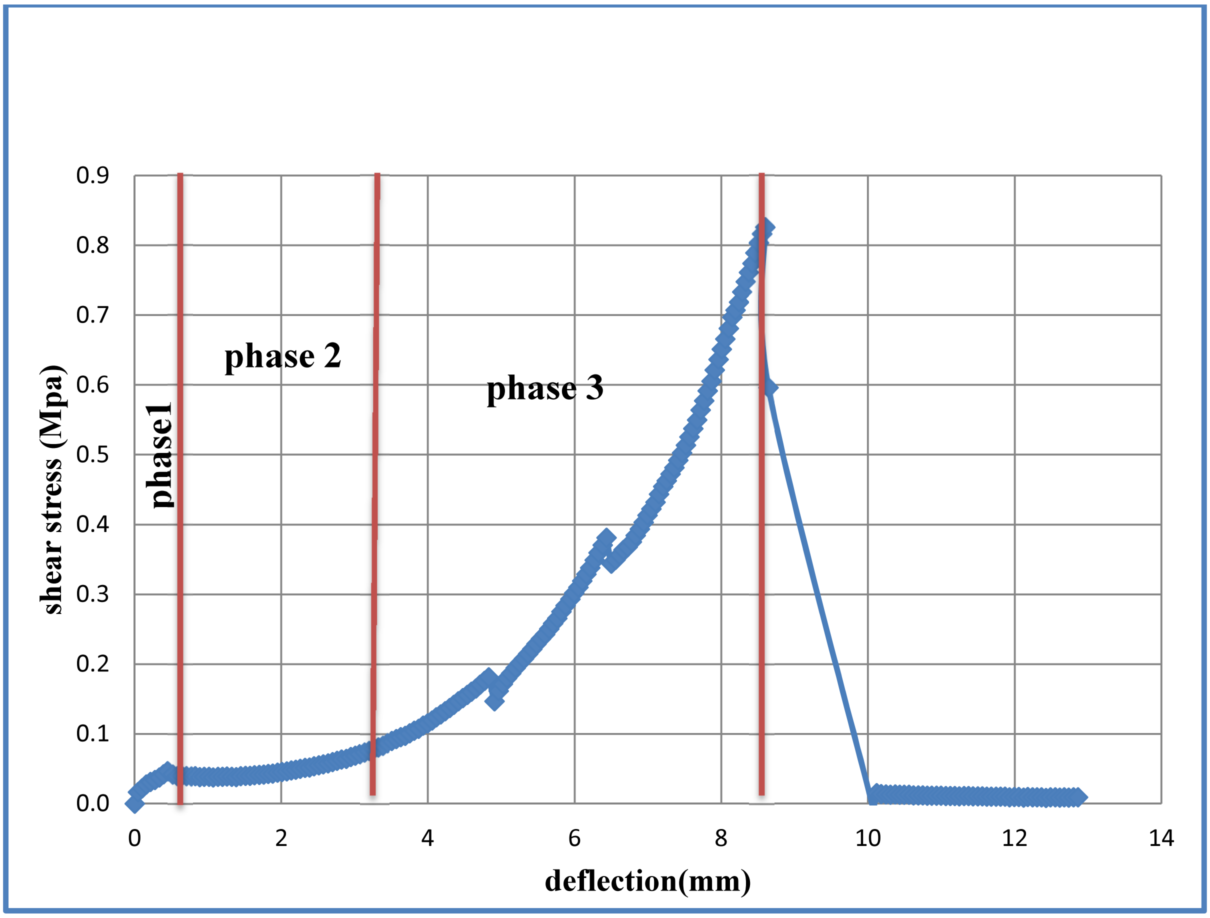


Figure 5-14 General pattern found for the shear stress deflection curve

Figure 5-14 is a shear stress deflection curve describing the failure pattern seen for all specimens tested. Practically this graph indicates the rate of the Shear stress induced for every millimetre of deflection up until failure. The interpretation of this curve indicates the shear stress induced for every millimetre of differential settlement.

The first phase of the curve indicates initially that there is a great increase in shear stress experienced by the wall from 0 to approximately 0.4 mm of deflection; this is observed from the gradient of this segment of the curve. In phase 2, from approximately 0.4 mm to 3 mm the gradient of the curve is not as steep as was indicated in the results section 5.2 for up to 3mm deflection one can predict the stresses along any height of wall. In Phase 3 the rate of change of shear stress increases exponentially with increase in deflection until the specimen suddenly fails at a deflection of approximately 10 mm. From this one can conclude that sudden failure occurs for a wall subject to differential settlement after 3mm of differential settlement has occurred.

5.3 Change in shear strength with age.

Specimens were produced and cured in air for 7, 14, 28 and 44 days before testing (see table 4-2, rows 1-4). These specimens were tested in test direction A see Figure 3-5 using load configuration 1 (vertical load) see Figure 4-1. Reasons for obtaining the change in shear strength with time (see section 4-3-1).

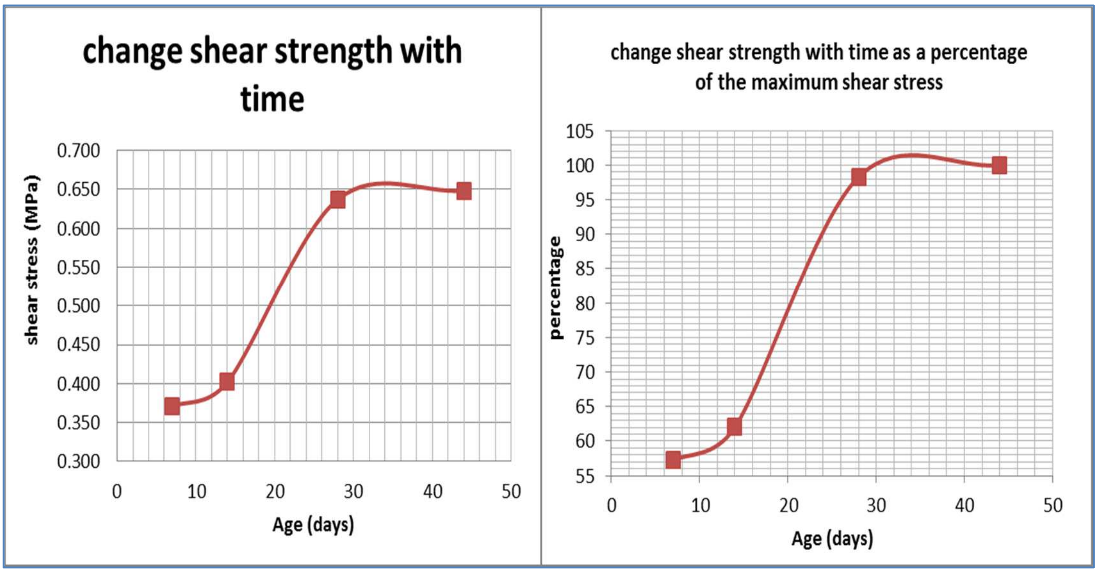


Figure 5-15 the average change in shear strength with time for load configuration 1 (see Figure 4-1)

The 14 day shear strength of the specimen is 7.7 percent greater than the shear strength at 7 days. From 14 to 28 days the shear strength increases rapidly. The 28 day shear strength of the specimen is 58 percent greater than the shear strength at 14 days. At 28 days the shear strength of the specimen has reached its maximum value, (see Figure 5-5).

Since these values were obtained from the testing of load configuration 1 (see section 4-2), the values for shear strength in the Figure 5-15 can be compared to the initial shear strength (τ_o) of 0.15 MPa as recommended by the codes (see Table 2-5 and section 2-3). At 7 days the average shear strength of the specimens are 2.5 times greater than the recommended value of the code. This factor shall be referred to as a safety factor. At 14 days this safety factor increases to a value of 2.7. At 28 days and older the safety factor reaches a peak value of 4.3, (see Figure 5-16).The curve in Figure 5-16 has been assumed to follow the trend of a second order polynomial graph, the equation of this graph can be used to obtain the safety factors from ages 7-28 days, however it is not recommended that one obtain a safety factor for ages before 7 days. It is recommended that at day zero when the wall is built it should be assumed that the shear strength is zero; A linear relationship for the safety factor of the initial shear strength from zero to 7 days of age should also be assumed. The gradient for this relationship is found to be 0.3535; the number of days that give a safety factor of 1 is 2.8 days, so at 3 days the strength of the wall is equal to the recommended initial shear strength as given by the codes (with no safety factor applied to it), The wall should be undisturbed from age zero to 3 days.

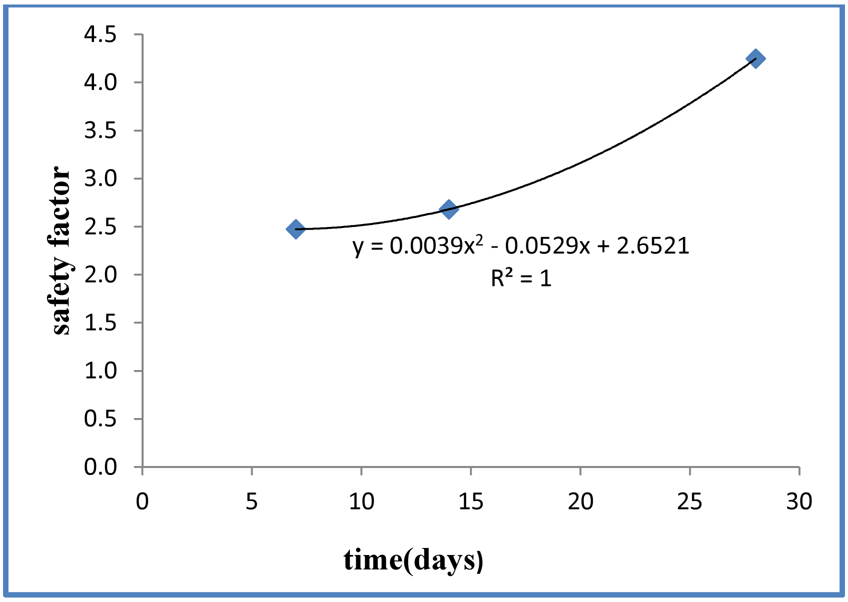


Figure 5-16 the change in safety factor for the initial shear stress with time

From these results it is recommended that one loads a wall in shear only a week after construction. The 7 day strength of the wall has a safety factor for shear strength of 2.5 compared to the recommended maximum design load for shear stress given by the codes.

5.4 Shear strength of a double and a single wall

5.4.1 The shear strength of a double wall at age greater than 28 days

It is important to know the maximum stresses a wall can take once it reaches its full strength (after 28 days see section 5-3). Specimens were produced and cured in air for 44 days before testing (see table 4-2, rows 1-4). These specimens were tested in test direction A see Figure 3-5. Test direction A can be used to simulate the effect of shear stresses acting perpendicular to the long face of a double wall (Load L acting on surface area B), see Figure 3-5. The results from these tests were compared to the recommended shear stresses as given by the codes.

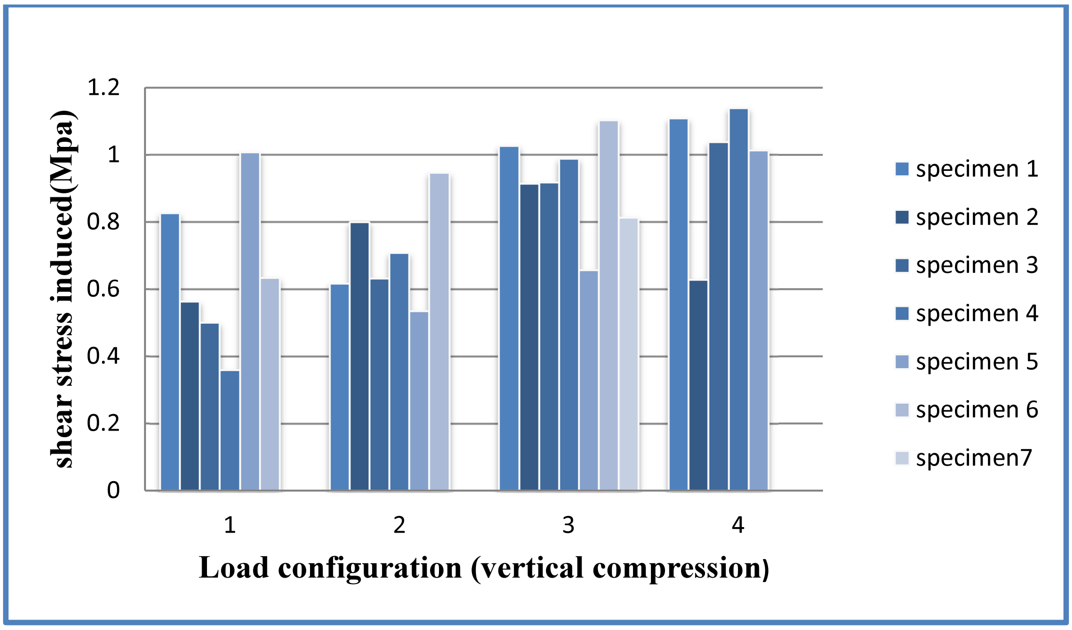


Figure 5-17 the peak shear stress found for specimens tested in test direction-A at 44 days

The peak shear stresses found for specimens tested at 44 days in test direction A are, see (Figure 5-17):

1. For load configuration 1 the average stress was found to be 0.65 MPa and had a standard deviation of 0.233 MPa.
2. For load configuration 2 the average stress was found to be 0.71 MPa and had a standard deviation of 0.148 MPa.
3. For load configuration 3 the average stress was found to be 0.92 MPa and had a standard deviation of 0.147 MPa.
4. For load configuration 4 the average stress was found to be 0.98 MPa and had a standard deviation of 0.206 MPa.

What can be noted from these results is that as the vertical load increases the shear stress increase. The standard deviation decreases, accept load configuration 4 were the standard deviation increases. As the vertical load increases the results are more accurate.

Using the peak stresses for specimens tested at 44 days in test direction A for different load configurations one can obtain the characteristic shear strength equation (equation 2-1) for a double wall (constructed from cement stock bricks bonded by 10 mm thick mortar having the same mix design as used in testing).

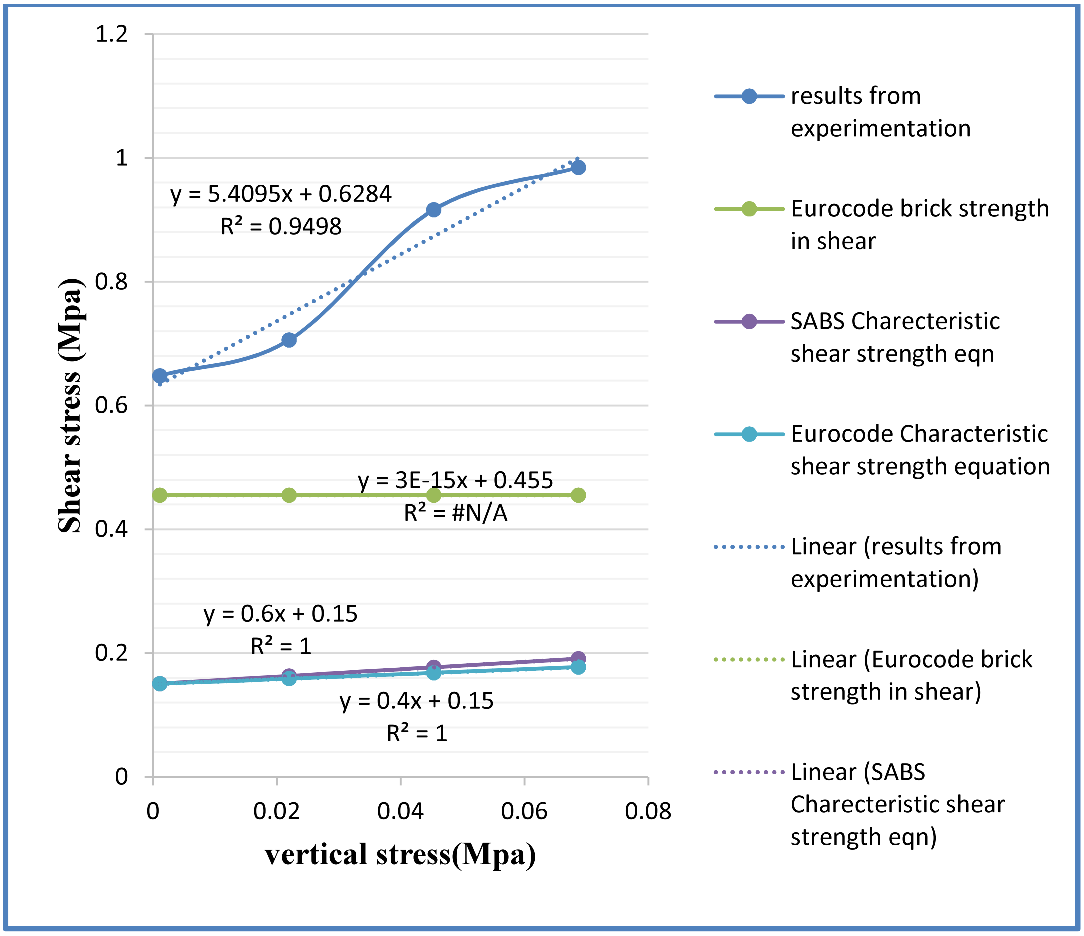


Figure 5-18 the characteristic shear strength equation for a double wall

Assuming a linear trend the characteristic shear strength equation for a double wall (constructed from cement stock bricks bonded by 10 mm thick mortar having the same mix design as used in testing) is $\tau_u = 0.6284 + 5.4095\sigma_n$, see Figure 5-18.

The characteristic shear strength equation for a double wall was compared with the recommended characteristic shear strength equations as given by the South African (equation 2-6) and European (equation 2-5) codes see Figure 5-18. This comparison was done to obtain a value of safety factor against the recommended values as given by the codes. The safety factor for the initial shear stress (τ_o) for both codes was calculated to a value of 4.2 and the safety factors for friction (μ) were calculated as 13.5 for the Eurocode and 9 for the South African code, see Figure 5-19.

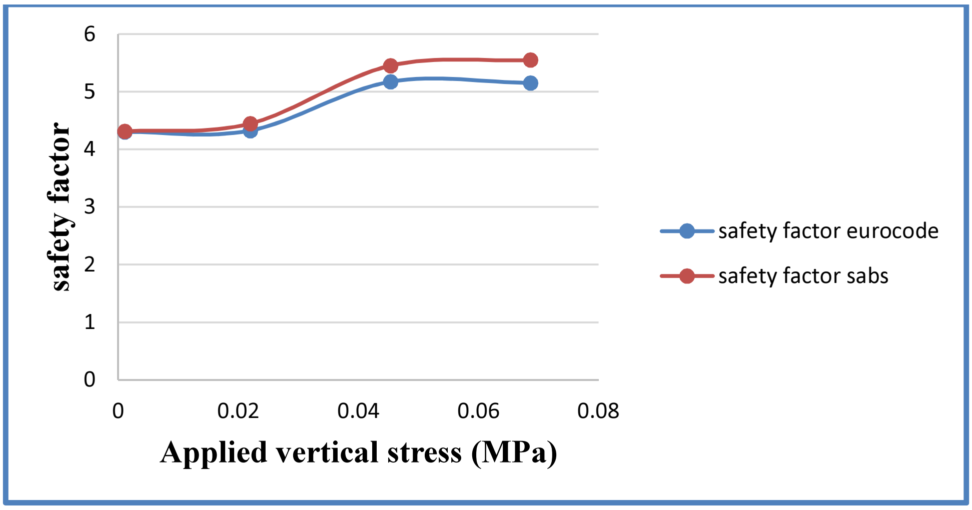


Figure 5-19 Change in Safety factor for the characteristic shear strength equation of a double wall with change in applied vertical load.

As the vertical stress (Load Configuration, The position in the wall according to height) increases the safety factor for shear strength increases until it reaches a maximum value of 5, see Figure 5-19.

The characteristic shear strength equation obtained for a double wall through experimentation, gives shear strength values 4.2 times greater than the recommended equations of the code, see Figure 5-19. Hence the safety factor for the shear strength of a double wall is 4.2.We design it to take 4.2 times less shear stress using the European and South African codes.

5.4.2 The shear strength of a single wall at age greater than 28 days

It is important to know the maximum stresses a wall can take once it reaches its full strength (after 28 days see section 5-3). Specimens were produced and cured in air for 28 days before testing (see table 4-2, rows 9-12). These specimens were tested in test direction B test direction B see section 4-2. Test direction B can be used to simulate the effect of shear stresses acting perpendicular to the long face of a single wall (Load L acting on surface area B), see Figure 4-3. The results from these tests were compared to the recommended shear stresses as given by the codes.

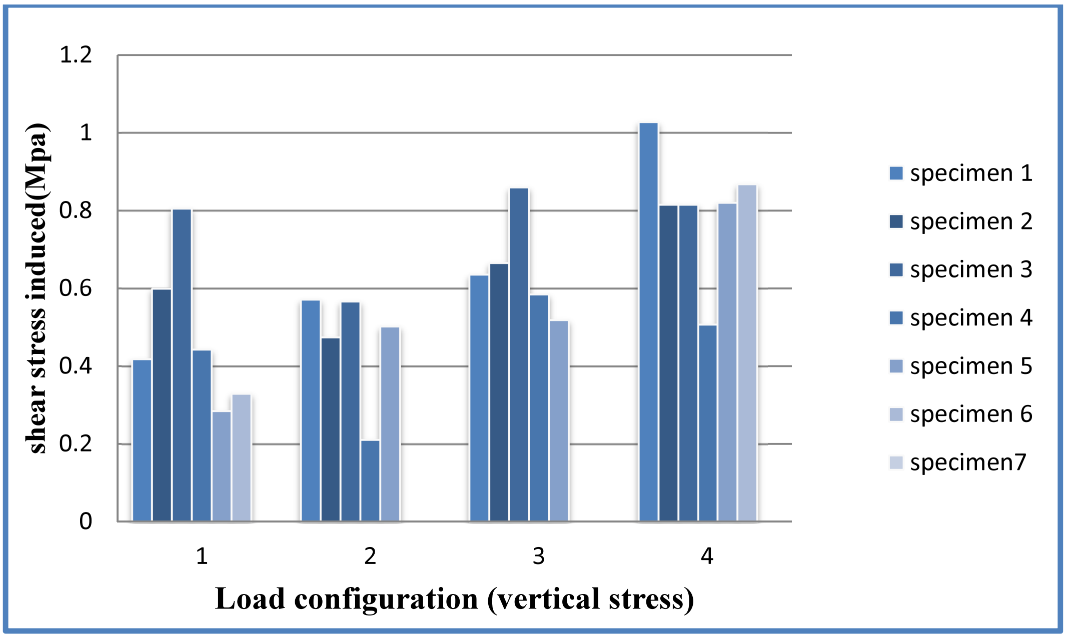


Figure 5-20 the peak shear stress found for specimens tested in test direction-B at 28 days

The peak shear stresses found for specimens tested at 28 days in test direction B are, see (Figure 5-17):

1. For load configuration 1 the average stress was found to be 0.48 MPa and had a standard deviation of 0.193 MPa.
2. For load configuration 2 the average stress was found to be 0.47 MPa and had a standard deviation of 0.148 MPa.
3. For load configuration 3 the average stress was found to be 0.65 MPa and had a standard deviation of 0.128 MPa.
4. For load configuration 4 the average stress was found to be 0.81 MPa and had a standard deviation of 0.169 MPa.

What can be noted from these results is that as the vertical load increases the shear stress increases. The standard deviation decreases. As the vertical load increases the results are more accurate.

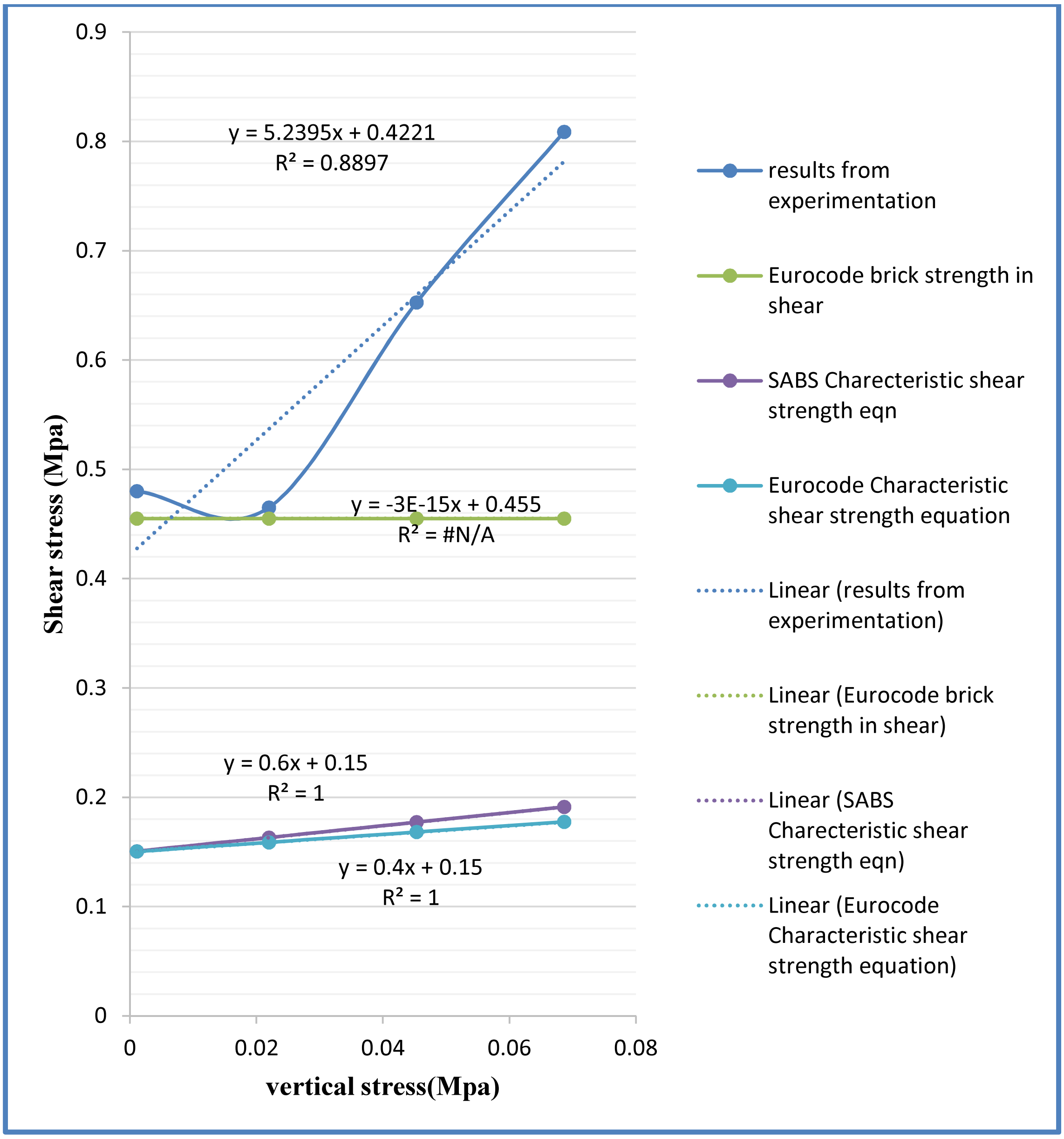


Figure 5-21 the characteristic shear strength equation for a single wall

Using the peak stresses for specimens tested at 28 days in test direction B for different load configurations one can obtain the characteristic shear strength equation (equation 2-1) for a single wall (constructed from cement stock bricks bonded by 10 mm thick mortar having the same mix design as used in testing).

Assuming a linear trend the characteristic shear strength equation for a single wall (constructed from cement stock bricks bonded by 10 mm thick mortar having the same mix design as used in testing) is $\tau_u = 0.4221 + 5.2395\sigma_n$, see Figure 5-21.

The characteristic shear strength equation for a double wall was compared with the recommended characteristic shear strength equations as given by the South African (equation 2-6) and European (equation 2-5) codes see Figure 5-21. This comparison was done to obtain a value of safety factor against the recommended values as given by the codes. The safety factor for the initial shear stress (τ_o) for both codes was calculated to a value of 2.8 and the safety factors for friction (μ) were calculated as 13.1 for the Eurocode and 8.7 for the South African code, see Figure 5-22.

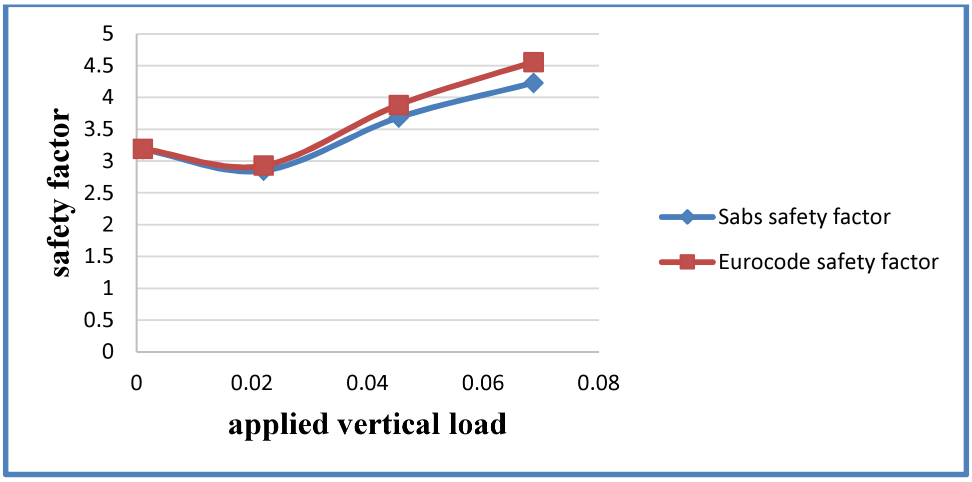


Figure 5-22 Change in Safety factor for the characteristic shear strength equation of a single wall with change in applied vertical load

As the vertical stress (Load Configuration, The position in the wall according to height) increases the safety factor for shear strength increases until it reaches a maximum value of 5, see Figure 5-22.

The characteristic shear strength equation obtained for a single wall through experimentation, gives shear strength values 2.8 times greater than the recommended equations of the code, see Figure 5-22. Hence the safety factor for the shear strength of a single wall is 2.8.We design it to take 2.8 times less shear stress using the European and South African codes.

5-4-3 Comparison between the shear strength of a double wall (test direction A) and a single wall (test direction B)

From the results for the change in shear strength with change in age of specimen it is clear that at 28 days the specimen has reached its maximum strength, see section 5-3. Hence the 44 day data of test direction A can be compared to that of test direction B.

The difference in shear strength between test direction A and test direction B for each load configuration (vertical compressive load, see figure 4-1) is:

1. For load configuration 1 the strength of the specimen tested along test direction A is 1.35 times greater than that of test direction B.
2. For load configuration 2 the strength of the specimen tested along test direction A is 1.52 times greater than that of test direction B.
3. For load configuration 3 the strength of the specimen tested along test direction A is 1.40 times greater than that of test direction B.
4. For load configuration 4 the strength of the specimen tested along test direction A is 1.22 times greater than that of test direction B.

The lowest increase in strength from the above results is 22 percent, which is a large increase in strength, since the bonded areas of the specimens are the same, which indicates that the direction in which the force acts has an influence on the strength of the wall.

The average change in shear strength was found to be a value of 37 percent.

From the above results one can conclude that as the vertical stress increases, the increase in shear strength from test direction A to test direction B decreases. These results can be represented by a third order polynomial equation, see Figure 5-23. As the vertical stress increases, the percent increase in shear strength from test direction A to test direction B decreases and tends to a value of zero. This third order polynomial curve does not reach a Y value of Zero as the vertical stress increases. It is suggested that one use this trend line to obtain the increase in shear resistance from a normal stress of 0 MPa to a normal stress of 0.07 MPa.

For vertical stresses greater than 0.07 MPa one can use a 2nd order polynomial to represent the data, see Figure 5-24.

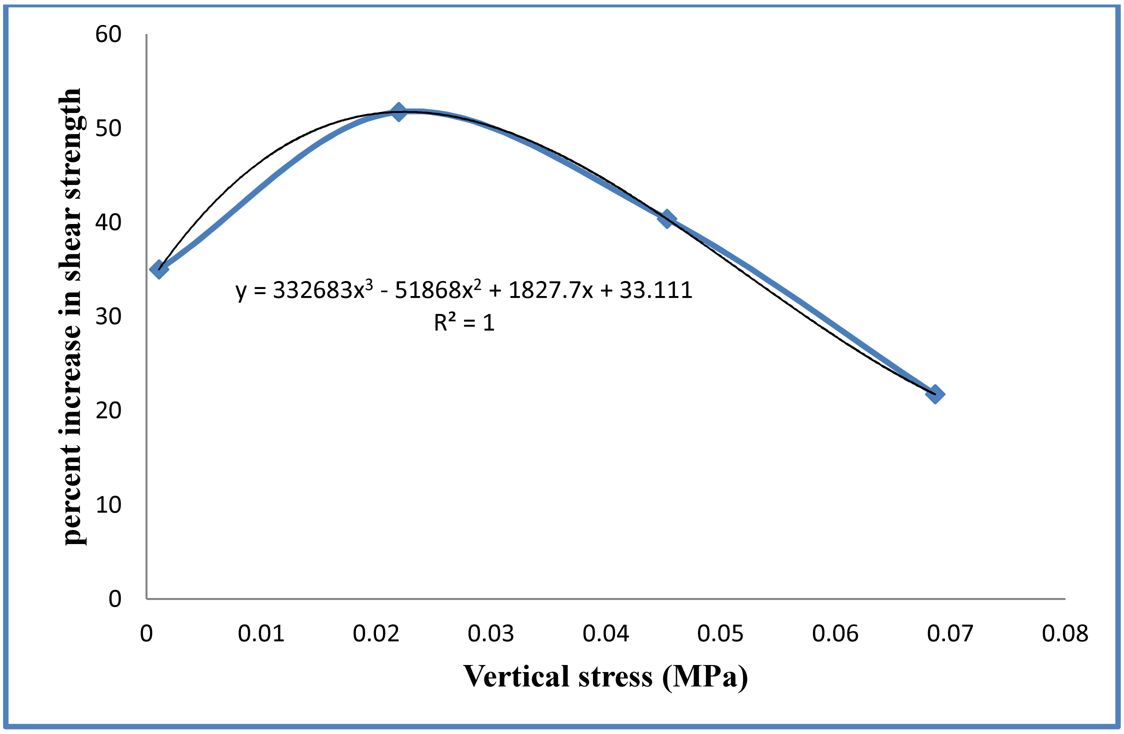


Figure 5-23 Percent increase in shear strength from test direction A- B for different vertical loads

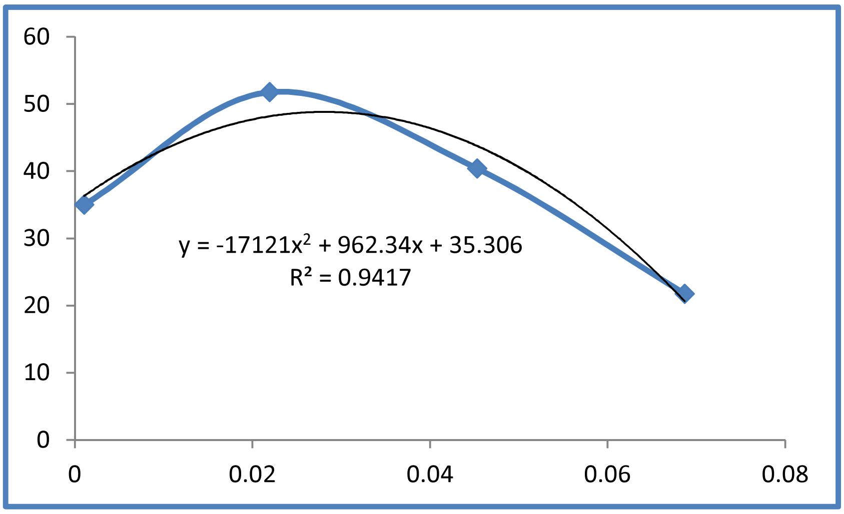


Figure 5-24 Percent increase in shear strength from test direction A- B for different vertical loads

The normal stress at which there is no longer an increase in shear stress from test direction-A to test direction-B , was obtained using a 2nd order polynomial to represent the data, see Figure 5-24. This normal stress is calculated as 0.0815 MPa.

From the data it is clear that the increase in shear strength decreases with height of wall, Hence a double wall has an increased shear strength compared to a single wall for vertical compressive stresses less than 0.0815 MPa. Assuming a density of wall of 2200 kg/m^3 this would relate to a free standing wall of 3.7 meters in height without any additional vertical loads applied to it.

A general increase in shear resistance of about 37 percent was observed in double walls compared to single walls for free standing walls of height less than 3 metres.

From the results one can conclude that the length of the specimen the load has to shear through has an influence on the shear strength of the masonry, i.e. the longer this length the greater the shear force has to be for failure to occur.

5.5 Change in shear strength by reducing the w/c ratio

The effect of reducing the water to cement ratio from the optimum water to cement ratio on the shear strength of masonry is important. Conditions on site are not controlled the amount of water added to the mortar mix is determined by the site workers it is not controlled. The mortar with w/c ratio of 7/6 was inconsistent and consisted of lumps. The mortar with water cement ratio 4/3 was smooth and consistent.

Specimens were produced and cured in air using two different water cement ratios, they were tested at 28 days, (see table 4-2, rows 1-4 and rows 5-8). These specimens were tested in test direction A see Figure 3-5.

For results on the specimens produced with water to cement ratio of 4/3 see section 5.4.1.

The peak shear stresses found for specimens produced with a water to cement ratio of 7/6 for load configurations 3 and 4 are:

1. For load configuration 3 the average stress was found to be 0.77 MPa and had a standard deviation of 0.155 MPa.
2. For load configuration 4 the average stress was found to be 0.62 MPa and had a standard deviation of 0.165 MPa.

For load configuration 3, the specimens produced with a water/cement ratio of 4/3 gave a shear strength 20 percent greater than those produced with a water/cement ratio of 7/6.For load configuration 4, the specimens produced with a water/cement ratio of 4/3 gave a shear strength 58 percent greater than those produced with a water/cement ratio of 7/6.

For a w/c ratio of 7/6 for load configuration 3 the safety factor was found to be an average value of 4.3 compared to 5.2 for a water to cement ratio of 4/3. For load configuration 4 the safety factor was found to be an average value of 3.5 compared to the safety factor of 5.2 for a water to cement ratio of 4/3. This gives an indication of why the safety factors for the recommended shear values of the code are so large.

The water cement ratio of the mix is important. When one produces mortar it should be smooth and cohesive, if this is not the case than the optimum amount of water has not been added to the mortar mix, this will result in a weaker brick mortar bond.

5.6 Stresses caused by differential settlement.

When using the shear box machine, the shear force is applied to the specimens at a constant deflection rate. Two sets of results are produced, peak shear stresses and the shear stress for every millimetre of deflection. It was realised that the force applied at a constant deflection rate can be modelled as a foundation settling at a constant deflection rate. A double wall consists of two single walls bonded by a mortar layer. Testing in Test direction A can be used to model a double wall were one of the single walls is subjected to foundation settlements i.e. the soil on side B of the wall settles at a constant deformation rate, brick wall side B moves at a constant deflection rate while brick wall side A remains stationary, see Figure 5-25.

From the results of the experimentation one can obtain the shear stresses induced on a double wall due to foundation settlement.

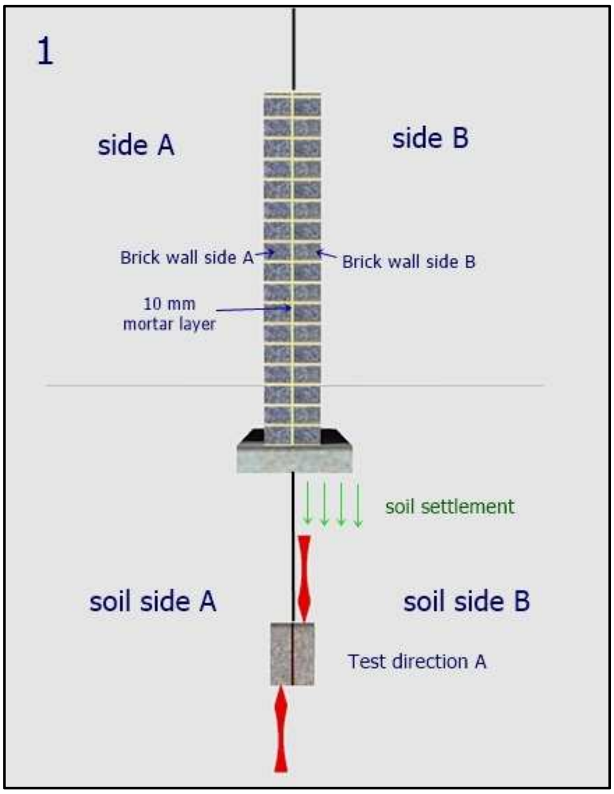


Figure 5-25 Schematic representation of how test direction A can be used to model Foundation settlements in a double wall

5.6.1 A freestanding double wall subject to stresses due to differential settlement of the foundation.

In a double wall were one of the single walls is subjected to foundation settlements (Assuming no lateral loads are applied to the wall) can be modelled by Test direction A (see figure 3-1) using load configuration 1(see section 4-2). See Figure 5-25.

Specimens were produced and cured in air for 44 days before testing (see table 4-2, rows 1-4). These specimens were tested in test direction A see Figure 3-5.

The shear stress induced for every mm deflection can be represented by a 2nd order polynomial curve, see Figure 5-26. After 3mm deflection the stresses increases rapidly for every millimetre of deflection, see Figure 5-26. After 3mm deflection the stresses increase with increase in deflection follows the geometric sequence with r equal to a value of approximately 1.5 (with r being the common ratio),see Figure 5-6.

These results were obtained from the testing using load configuration 1 (see section 4-2), therefore they can be compared to the initial shear strength (τ_o) of 0.15 MPa as recommended by the codes (see Table 2-5 and section 2-3). After 6mm deflection the shear stress induced exceeds the recommended shear stress as given by the codes, see Figure 5-26.

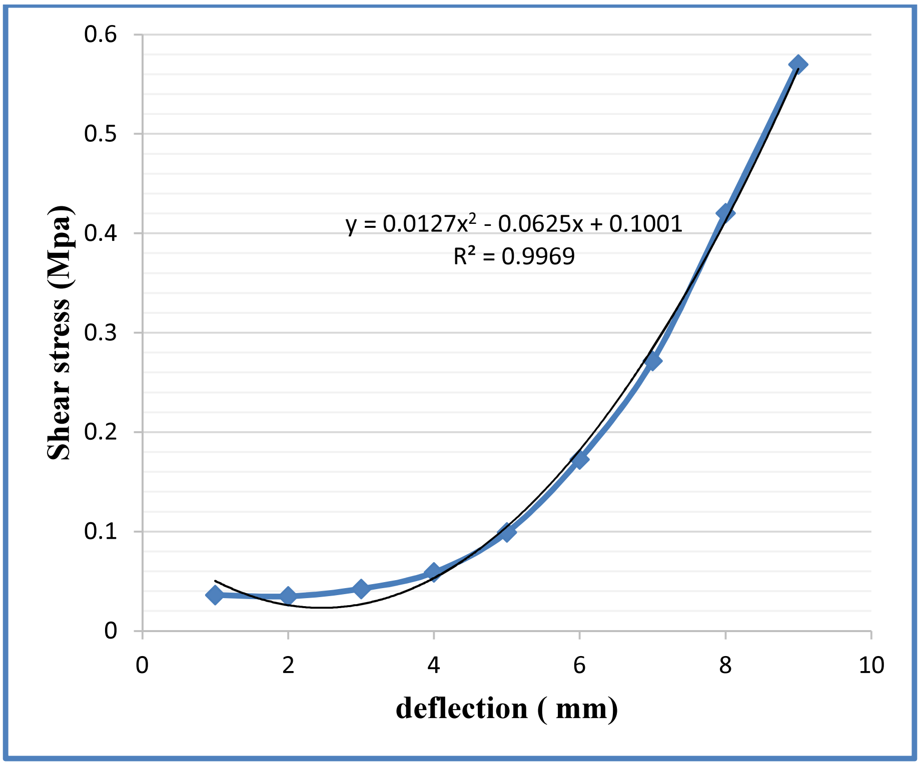


Figure 5-26 Shear stresses induced for every mm deflection for load configuration1 (vertical compression)

In conclusion after 3 mm differential settlement, a wall has to be repaired due to the rapid increase in shear stresses after 3mm of differential settlement.

5.6.2 A double wall subject to lateral stresses and stresses due to differential settlement of the foundation.

In a double wall were one of the single walls is subjected to foundation settlements (Assuming lateral loads are applied to the wall) can be modelled by Test direction A (see figure 3-1). For lateral load applied to a wall all load configurations are considered, see Figure 5-27.

Specimens were produced and cured in air for 44 days before testing (see table 4-2, rows 1-4). These specimens were tested in test direction A see Figure 3-5.

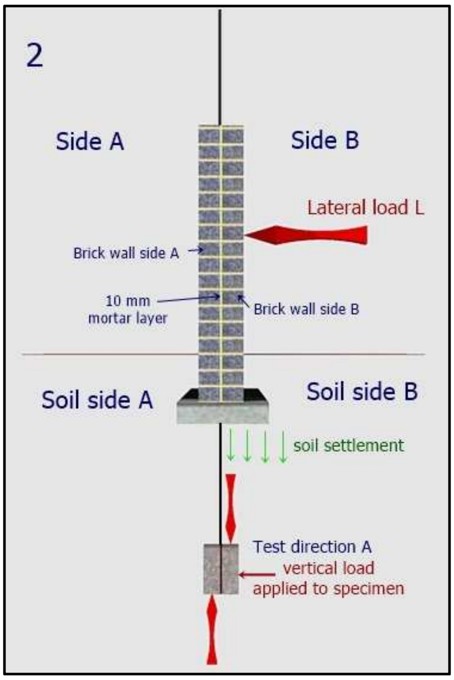


Figure 5-27 Schematic representation of how test direction A can be used to model Foundation settlements in a double wall

Shear stresses induced for 1mm differential settlement

The shear stress induced for 1 mm deflection for different vertical applied loads are (see figure 5-28):

1. For load configuration 1 the average stress was found to be 0.036 MPa and had a standard deviation of 0.0097 MPa.
2. For load configuration 2 the average stress was found to be 0.039 MPa and had a standard deviation of 0.012 MPa.
3. For load configuration 3 the average stress was found to be 0.042 MPa and had a standard deviation of 0.0097 MPa.
4. For load configuration 4 the average stress was found to be 0.059 MPa and had a standard deviation of 0.0059 MPa.

What can be noted from these results is that as the vertical load increases not only does the shear stress increase but the standard deviation also decreases, the results obtained are consistent. From the above results a certain trend can be noticed as the vertical load increases the shear stress induced also increases.

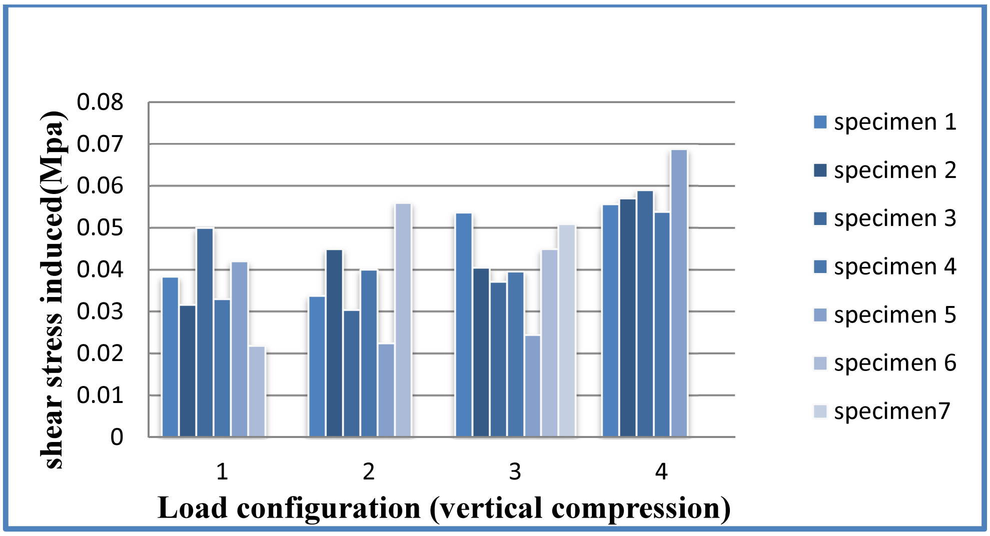


Figure 5-28 Shear stresses induced for 1mm deflection for different vertical compressions

The results in Figure 5-28 were plotted on a straight line curve to obtain a relationship between the vertical applied stress and the shear stress induced for 1mm deflection see Figure 5-29. Two curves were produced. The first curve is produced by taking the average value for each applied vertical load; it is represented by the linear equation $y = 0.3206x + 0.0326$ and has a correlation with the data of $R^2 = 0.8103$. The second curve is produced by omitting the results which are outliers; it is represented by the linear equation $y = 0.3012x + 0.0332$ and has a correlation with the data of $R^2 = 0.9009$.

The factor for R^2 is close to 1, using the second curve (equation 5-1) one can approximate the shear stress induced for 1 mm differential settlement for any lateral load applied to a double wall, see Figure 5-27.

$$\tau_u = 0.3012\ \sigma_n + 0.0332. \qquad \textbf{Equation 5-1}$$

Where:

τ_u – The average shear stress induced for 1mm deflection in MPa.

σ_n – Lateral load applied to the wall in MPa.

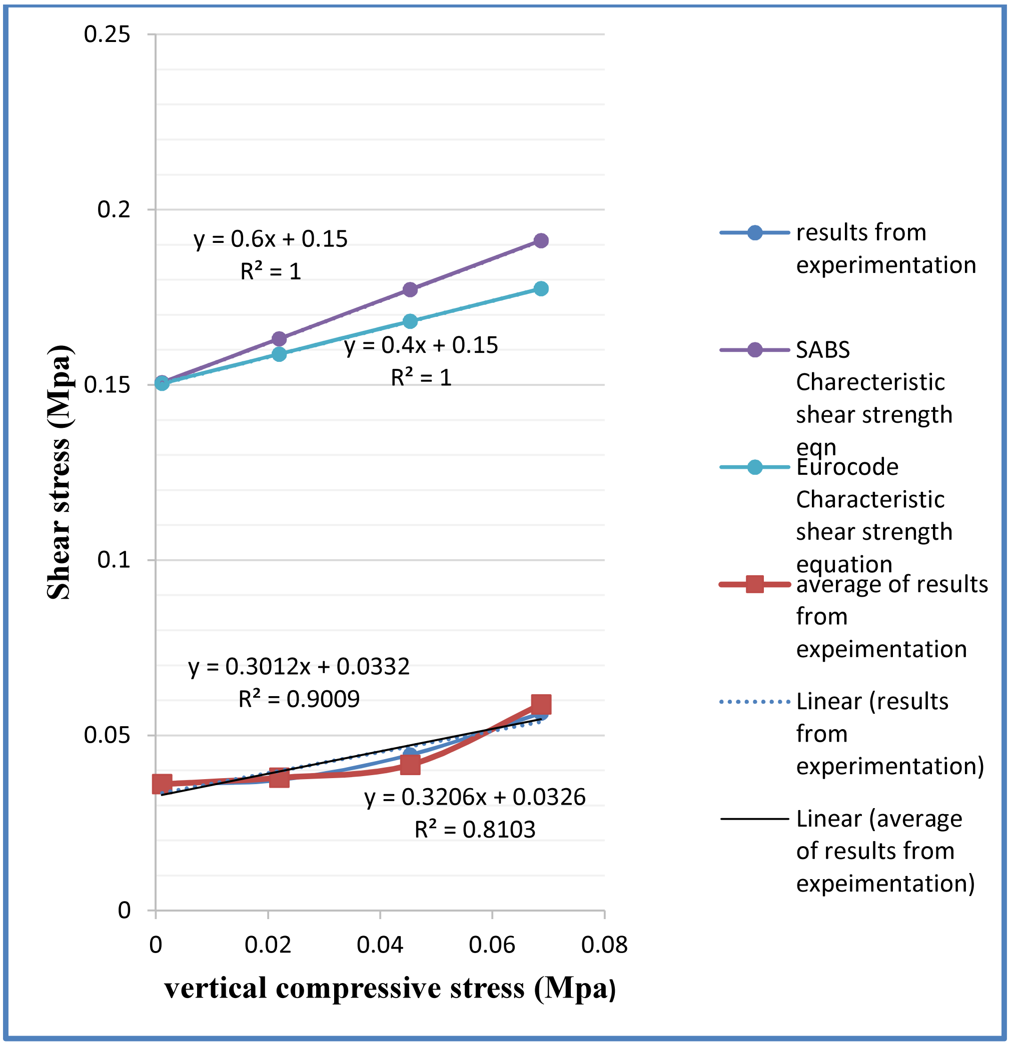


Figure 5-29 Shear stresses induced for 1mm settlement for different vertical compressions

Shear stresses induced for 2mm differential settlement

The shear stress induced for 2mm deflection for different vertical applied loads are (see figure 5-30):

1. For load configuration 1 the average stress was found to be 0.035 MPa and had a standard deviation of 0.0105 MPa.
2. For load configuration 2 the average stress was found to be 0.046 MPa and had a standard deviation of 0.0235 MPa.

3. For load configuration 3 the average stress was found to be 0.045 MPa and had a standard deviation of 0.0094 MPa.

4. For load configuration 4 the average stress was found to be 0.068 MPa and had a standard deviation of 0.0079 MPa.

What can be noted from these results is that as the vertical load increases not only does the shear stress increase but the standard deviation also decreases. From the above results a certain trend can be noticed as the vertical load increases the shear stress induced also increases. If one looks at the data for 1mm deflection the results are more consistent, and show a more definite trend than that of 2mm, the shear stresses increase by a small magnitude for the first three load configurations; however for the fourth load configuration the increase in magnitude is greater.

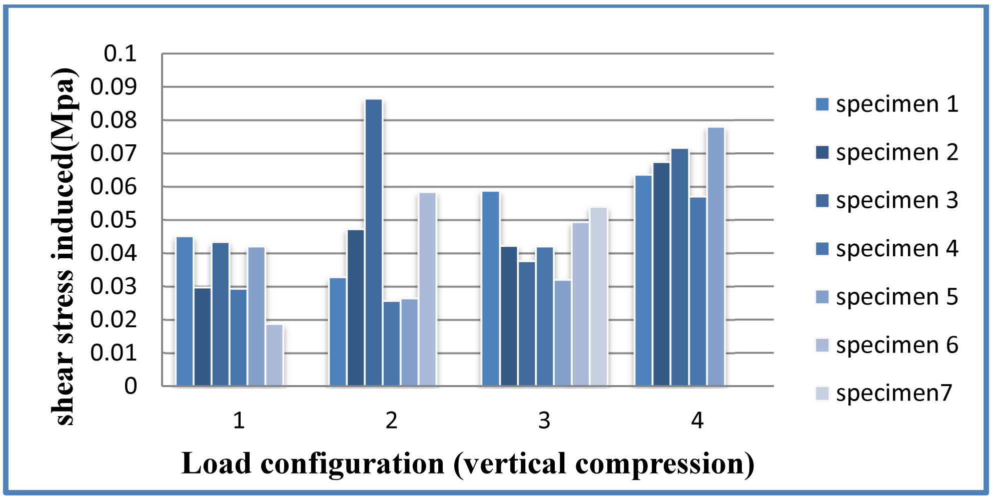


Figure 5-30 Shear stresses induced for 2mm deflection for different vertical compressions

The results in Figure 5-30 were plotted on a straight line curve to obtain a relationship between the vertical applied stress and the shear stress induced for 1mm deflection see Figure 5-31. Two curves were produced. The first curve is produced by taking the average value for each applied vertical load; it is represented by the linear equation $y = 0.4309x + 0.0337$ and has a correlation with the data of $R^2 = 0.837$. The second curve is produced by omitting the results which are outliers; it is represented by the linear $y = 0.4425x + 0.0358$ and has a correlation with the data of $R^2 = 0.8897$.

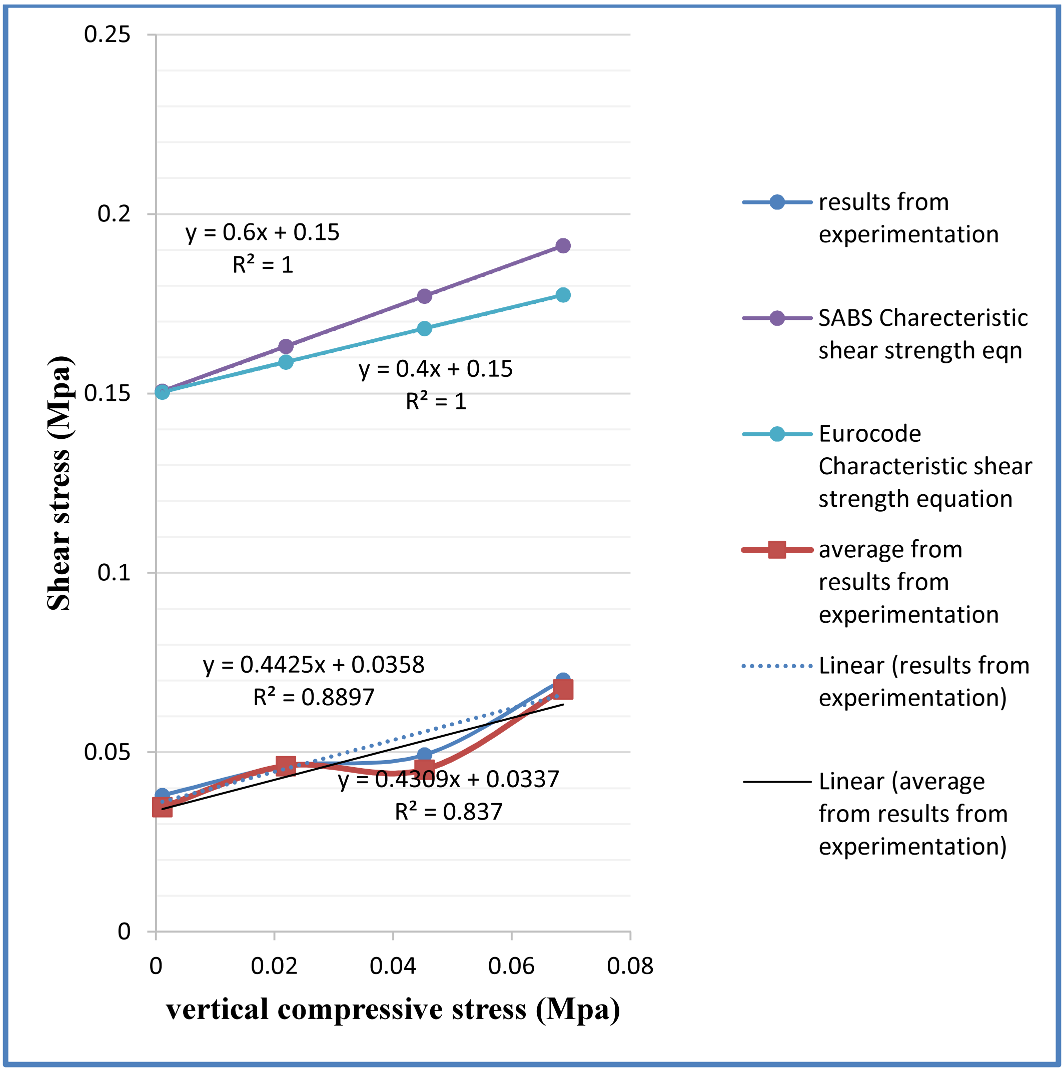


Figure 5-31 Shear stresses induced for 2mm deflection for different vertical compressions

The factor for R^2 is close to 1, using the first curve (equation 5-2) one can approximate the shear stress induced for 2 mm differential settlement for any lateral load applied to a double wall, see Figure 5-27.

τ_u = **0.4309 σ_n + 0.0337**. **Equation 5-2**

Where:

τ_u– The average shear stress induced for 1mm deflection in MPa.

σ_n – Lateral load applied to the wall in MPa.

Shear stresses induced for 3mm differential settlement

The shear stress induced for 2mm deflection for different vertical applied loads are (see figure 5-32):

1. For load configuration 1 the average stress was found to be 0.042 MPa and had a standard deviation of 0.0183 MPa.
2. For load configuration 2 the average stress was found to be 0.059 MPa and had a standard deviation of 0.0417 MPa.
3. For load configuration 3 the average stress was found to be 0.049 MPa and had a standard deviation of 0.0126 MPa.
4. For load configuration 4 the average stress was found to be 0.079 MPa and had a standard deviation of 0.0137 MPa.

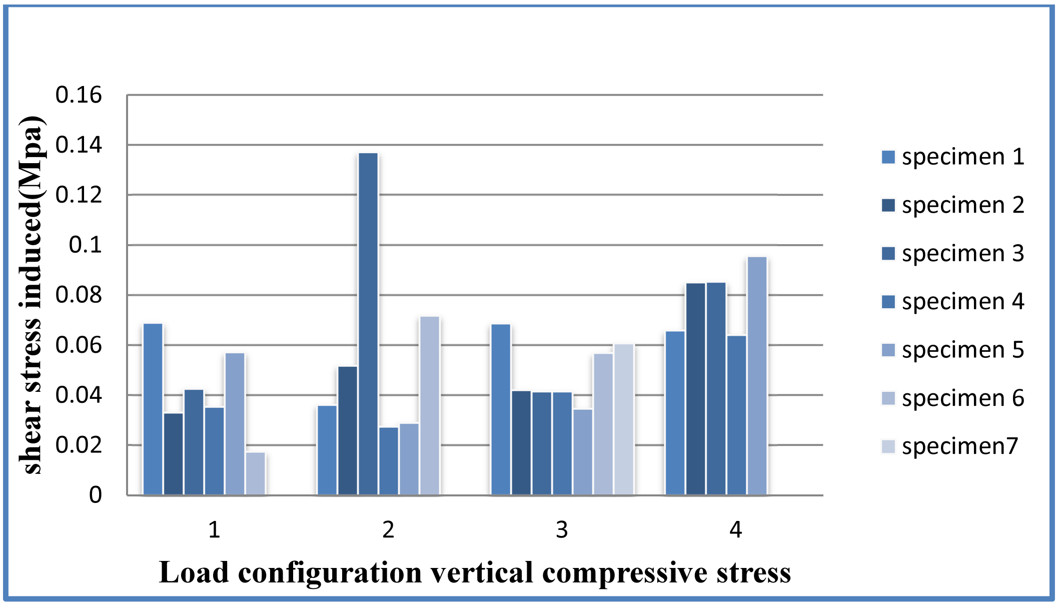


Figure 5-32 Shear stresses induced for 3mm deflection for different vertical compressions

From the above results a certain trend can be noticed as the vertical load increases the shear stress induced also increases. The values of shear stress are similar the follow a trend however there greater disparities are seen in the data than that for 2 mm and 1mm deflection. The standard deviation for all specimens tested has increased rapidly.

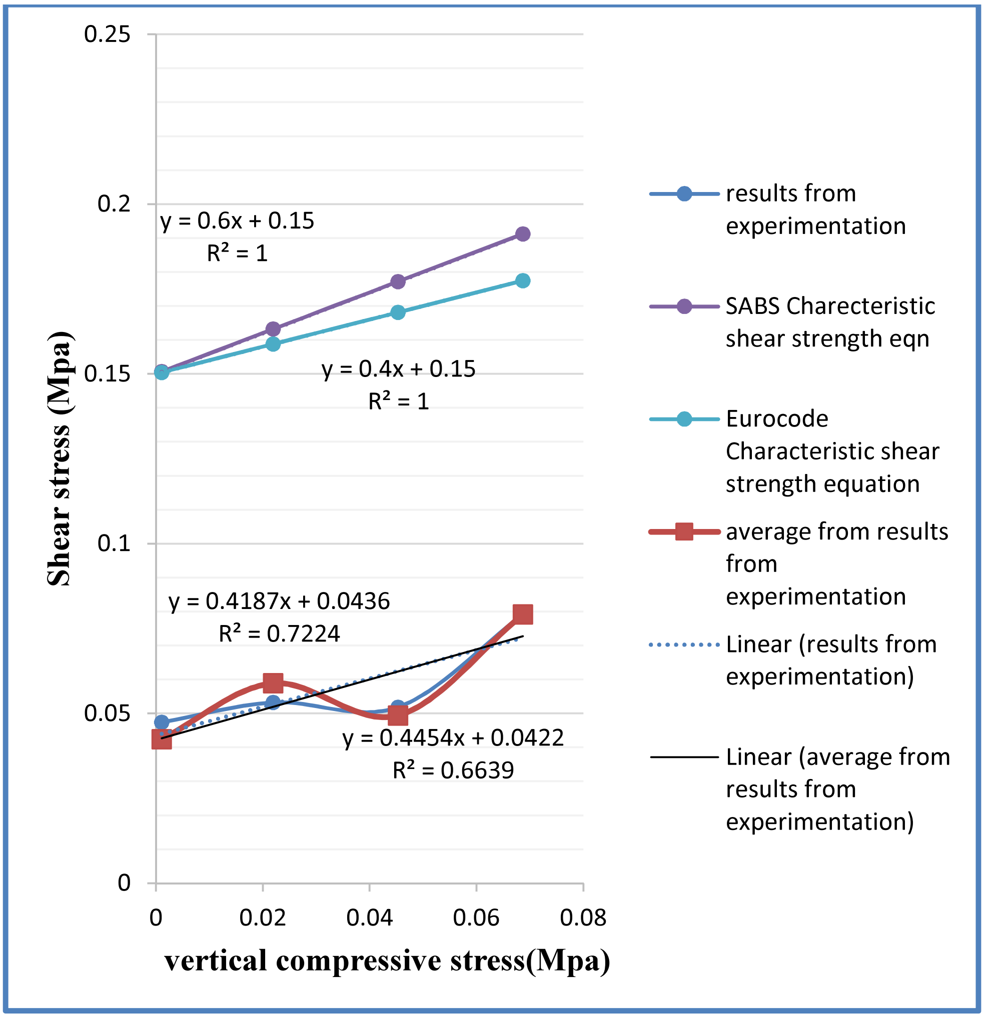


Figure 5-33 Shear stresses induced for 3mm deflection for different vertical compressions

The results in Figure 5-32 were plotted on a straight line curve to obtain a relationship between the vertical applied stress and the shear stress induced for 1mm deflection see Figure 5-33. Two curves were produced. The first curve is produced by taking the average value for each applied vertical load; it is represented by the linear equation $y = 0.4454x + 0.0422$ and has a correlation with the data of $R^2 = 0.6639$. The second curve is produced by omitting the results which are outliers; it is represented by the linear $y = 0.4187x + 0.0436$ and has a correlation with the data of $R^2 = 0.7224$.

The factor for R^2 is close to 1, using the second curve (equation 5-3) one can approximate the shear stress induced for 3 mm differential settlement for any lateral load applied to a double wall, see Figure 5-27.

τ_u = **0.4187 σ_n + 0.0436**. Equation 5-3

Where:

τ_u– The average shear stress induced for 1mm deflection in MPa.

σ_n – Lateral load applied to the wall in MPa.

Summary of relationships found that represent the shear stresses induced by differential settlements

One can approximate the shear stress induced for differential settlements from 0-3mm for any lateral load applied to a double wall, (see Figure 5-27),using equations 5-1,5-2 and 5-3.

- For 1mm, τ_u = **0.3012 σ_n + 0.0332**. Equation 5-1
- For 2mm, τ_u = **0.4309 σ_n + 0.0337**. Equation 5-2
- For 3mm, τ_u = **0.4187 σ_n + 0.0436**. Equation 5-3
- Where:
- τ_u– The average shear stress induced for the deflection considered in MPa.
- σ_n – Lateral load applied to the wall in MPa.

To obtain the shear stresses for deflections such as 1.5mm one would have to interpolate between the values given by these curves.

Shear stresses induced for greater than 3mm differential settlement

For 4mm differential settlement and greater than 4mm of differential settlement, the results vary considerably and a relationship between the shear stresses and vertical compression is no longer linear. However one can produce a set of curves which shows the change in shear stresses with change in differential settlement for each load configuration (vertical compression applied), see Figure 5-34.

From Figure 5-34 below, certain conclusions can be made.

1. From 4mm differential settlement to 5mm differential settlement the shear stresses double.
2. From 5mm to 6 mm differential settlement the shear stresses increase by a factor of about 1.7.
3. At 6mm differential stress the shear stresses induces are greater than the recommended maximum shear stress of the codes. I.e. according to the code the wall has failed.
4. From 6 mm to 7 mm differential settlement the shear stresses double.
5. From 7 mm to 8 mm differential settlement the shear stresses increase by a factor of 1.5.
6. From 8 mm to 9 mm differential settlement the shear stresses increase by a factor of 1.5.
7. At 9 mm differential settlement the shear stress is greater than the recommended shear stress of the bricks as recommended by the Eurocode.

As seen in section 5.6.1 (No lateral load applied), After 3mm deflection the stresses increase with increase in deflection following the geometric sequence with r equal to a value of approximately 1.5 (with r being the common ratio),see Figure 5-34.

In conclusion after 3 mm differential settlement, a wall has to be repaired due to the rapid increase in shear stresses after 3mm of differential settlement.

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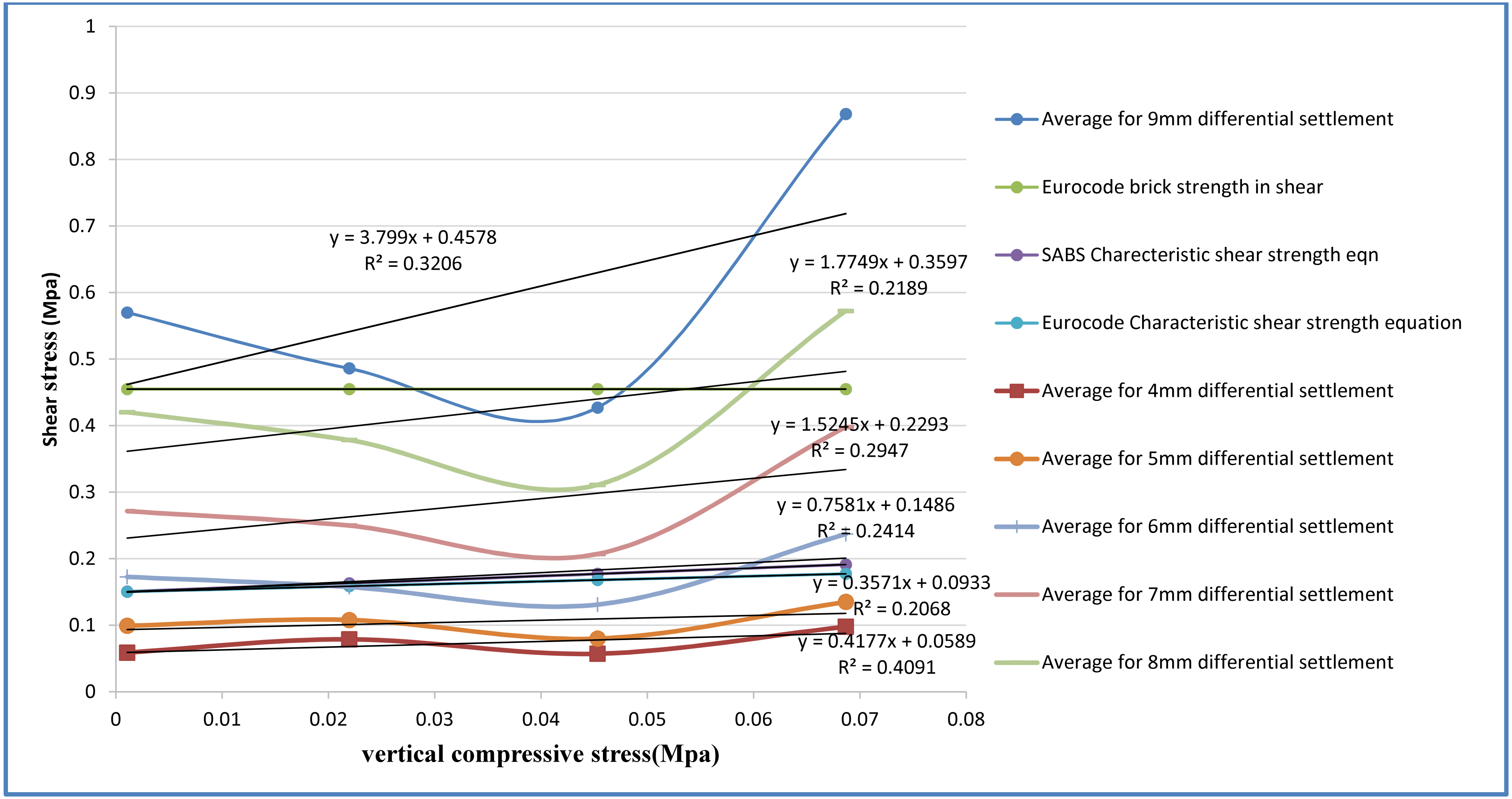

Figure 5-34 Shear stresses induced for different deflections for different vertical compressions

6.Conclusions and recommendations:

6.1 failure patterns

- The most common failure pattern for the brick specimens was failure occurring at the bond between the brick and the mortar, with an average frequency of occurrence for all specimens tested of above 80 percent.
- When considering a double wall with shear stress applied to it, the frequency of occurrence of shear bond failure increases with increase in vertical compression. When considering a single wall with shear stress applied to it, the frequency of occurrence of shear bond failure decreases with increase in vertical compression.
- The positions in a wall where the bricks are attached to the mortar (when constructing the wall), produces a weaker brick mortar bond than where the mortar is applied to the bricks.
- Curing in an environment with a high moisture content increases the frequency of occurrence of shear bond failure.

6.2 deflection at max shear stress and stress strain curve

- The max shear stress of all specimens tested occurred at about 10 mm of deflection.
- The stronger the brick mortar bond, the greater the deflection at failure.
- After 8 mm displacement a wall can no longer resist the applied shear stresses.
- A wall exposed to differential settlements of magnitude greater than 3mm fails suddenly.

6.3 Change in shear strength with age.

- The 14 day shear strength of the specimen is 7.7 percent greater than the shear strength at 7 days.

- From 14 to 28 days the shear strength increases rapidly. The 28 day shear strength of the specimen is 58 percent greater than the shear strength at 14 days.
- At 28 days the shear strength of the specimen has reached its maximum value.
- A wall of 3 days age has achieved the recommended shear strength used in design for both the Eurocode 6 - Design of masonry structures - Part 1-1: General and SANS 10164-1. The structural use of masonry Part 1: Unreinforced masonry walling

6.4 Shear strength of a double and a single wall

- The characteristic shear strength equation obtained for a double wall through experimentation, gives shear strength values 4.2 times greater than the recommended equations of the codes (Eurocode 6 - Design of masonry structures - Part 1-1: General and SANS 10164-1. The structural use of masonry Part 1: Unreinforced masonry walling) i.e. We design a double wall to take 4.2 times less shear stress using the codes.
- The characteristic shear strength equation obtained for a single wall through experimentation, gives shear strength values 2.8 times greater than the recommended equations of the codes (Eurocode 6 - Design of masonry structures - Part 1-1: General and SANS 10164-1. The structural use of masonry Part 1: Unreinforced masonry walling) i.e. We design a single wall to take 2.8 times less shear stress using the codes.
- A general increase in shear resistance of about 37 percent was observed in double walls compared to single walls for free standing walls of height less than 3 metres.
- From the results one can conclude that the length of the specimen the load has to shear through has an influence on the shear strength of the masonry, i.e. the longer this length the greater the shear force has to be for failure to occur.

6.5 Change in shear strength by reducing the w/c ratio

- The water cement ratio of the mix is important. When one produces mortar it should be consistent, if this is not the case than the optimum amount of water has not been added to the mortar mix, this will result in a weaker brick mortar bond.

6.6 Stresses caused by differential settlement.

- After 3mm deflection the stresses increase with increase in deflection following the geometric sequence with r equal to a value of approximately 1.5 (with r being the common ratio)..
- After 3 mm differential settlement, a wall has to be repaired due to the rapid increase in shear stresses.
- After 6mm of differential settlement the shear stresses exceed the maximum stresses as recommended by the codes. (Eurocode 6 - Design of masonry structures - Part 1-1: General and SANS 10164-1. The structural use of masonry Part 1: Unreinforced masonry walling).

6.7 Recommendations

- All the results obtained from this study are limited to the use of cement stock bricks, it is recommended to undertake tests using other types of bricks
- It is recommended that shear strength tests should be performed for specimens cured from 1 to 7 days
- It is recommended that shear tests be conducted on actual walls and compared to the results obtained when testing shear specimens in the laboratory.

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Appendix

Conference report submitted to ACCTA 2013-Advances in Cement and Concrete Technology in Africa.

Johannesburg, South Africa, January 28-30, 2013

Investigation into the shear bond strength in masonry

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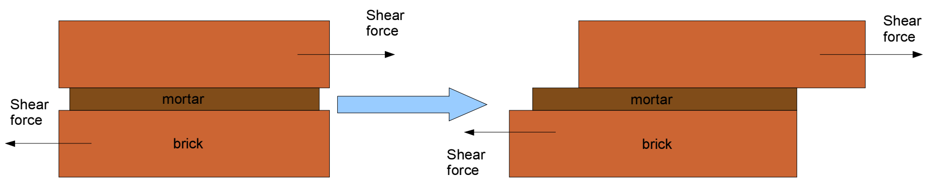
Abstract

The investigation of the shear bond strength in masonry is important as it is one of the principal failure mechanisms in masonry. Experiments were produced for 7 and 28 day testing using a mortar mix of 1:6 by volume. Major findings for the experiments conducted may be summarized as follows. It was found that as the age of the specimen increased the resistance of the couplets increased simultaneously. At 28 days it was found that the couplets failed at a constant average deflection of approximately 10 mm. A graph giving the failure pattern of the couplets at shear was also obtained.

Background theory

The shear bond strength in masonry is the force in shear required to "separate the units from the mortar and each other" [1] this is as shown in the figure below.



The" Shear strength at the interface comes from friction due the asperities between the surface of mortar layer and the surface of the brick unit, and the chemical bond between mortar and brick units. Normal compression perpendicular to the interface further increases its shear strength because the asperities cannot easily slide over one another". [2]

According to [2] this phenomenon can be represented by the Mohr-Coulomb Criterion as stated below.

$$\tau_u = \tau_o + \mu\sigma_n$$

Where:

τ_u =the average stress

τ_o = the initial stress without any pre-compression applied

μ = the friction between the interfaces

σ_n =the applied pre-compression

Materials and Equipment

Out of all the available masonry units on the market, cement-stock bricks were used. These bricks have flat rough surfaces they do not contain holes. The bricks were purchased and were already dry when used in the experiments. Their batch designated a nominal compressive strength of 7 MPa.

The mortar was produced using a cement of grade 32.5N.Building sand was obtained from the local hardware.
Cement to sand ratio of 1:6 per volume was used and gave a 28 day cube compressive strength of 4 MPa. A water to cement ratio of 4:3 was used in all experiments this was found to give a consistent mix. In relation to the 4:3 water to cement ratio that was used it should be noted that the water requirement for sand cement mixes are much greater than for concrete mixes. [3]

Both sides of the bricks were pre-wetted before applying the mortar to the bricks.

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The configuration of two bricks with the mortar in between forms a brick couplet as is shown in figure 1 below. Brick couplets were produced and cured in a water tank in ambient temperature for 4, 7 and 28 days before testing.

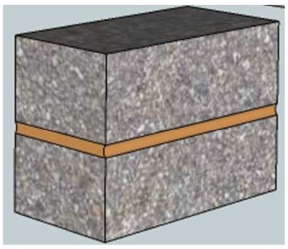


Figure 1

The experimental apparatus used was the shear box testing machine, figure 2, used for obtaining the shear strength of soils. Two customized components consisting of a concrete mould, figure3, and a steel plate figure 4 which were created in the workshop.

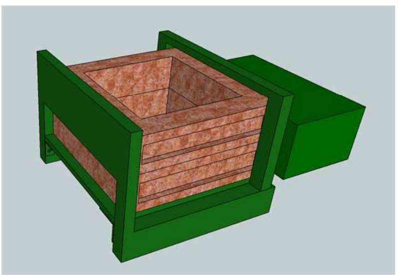


Figure 2

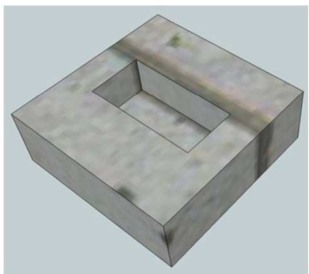


Figure 3

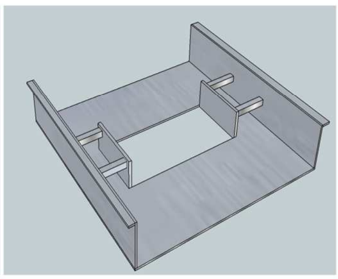
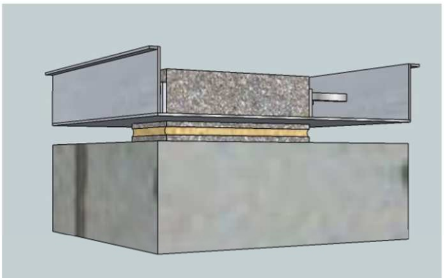


Figure4

The shear box consists of two cast iron boxes. The top box is immovable contrary to the bottom one, which translates freely horizontally according to the designed horizontal external load, applied to it. In the bottom box, a concrete mould, with a hollow core of the size of a brick, contained by a steel frame in its external perimeter, was placed to enable the testing of placed right on the top of the brick couplet, accommodated the upper part of the brick couplet, allowing the 15% of the bottom part of the upper brick to remain outside the mould. In order to identify the failure pattern of the couplet either due to brick or mortar shear failure .i.e. the area of the couplet exposed to the applied shear force was the mortar plus 15% of each individual brick.



Methodology

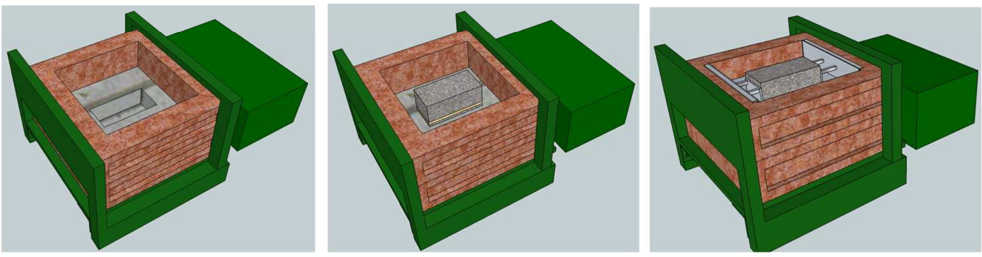
Cement bricks were used and as cement is hygroscopic, the dry state of the bricks would absorb part of the water from the mortar, weakening the mortar thereby leading to false experimental conclusions. In order to avoid the above distortion, the individual bricks were pre-wet before application of the mortar.

Mortar was applied within 1 hour of being mixed according to [4]

The mortar was applied to one brick the other brick was placed on top. The top brick was then tapped three times with the handle of the trowel to ensure full contact between the mortar and the surface of the brick. A compressive load of seven kilograms, giving a stress of 3.2kPa over the brick surface of 100mm x 210mm, falling into the recommended values as stipulated by [4]) of two to five kPa, was then applied to the couplet for 24 hours. Thereafter the load was removed and the couplet was placed in a water curing tank , contrary to the [4] which states the specimens must be cured in air, for 7 or 28 days depending on the scheduled experiment.

The brick couplet was removed from the curing tank a day before testing took place

The brick couplet was placed in the concrete mould the steel mould was then placed on top a constant vertical load was applied to the couplet; the vertical loads applied were of magnitude 50,100 and 150 kg. The horizontal shear force was then applied till failure .The horizontal shear force and deflection were then recorded.



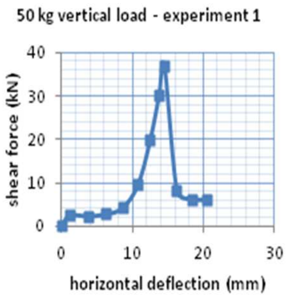
Experimentation Strategy

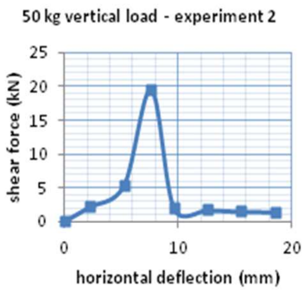
The main objective of the experiments was to find the maximum horizontal shear force responsible for the failure of the couplets

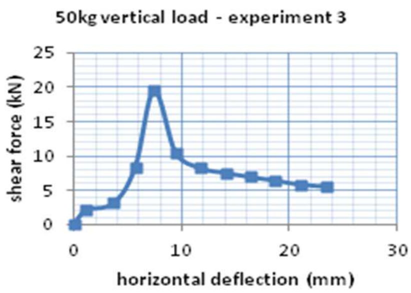
Experimental Results

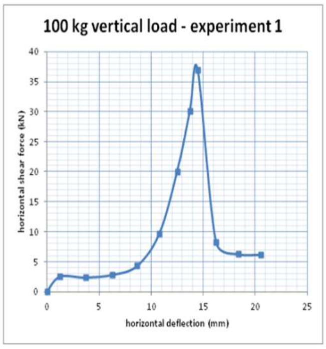
The experimentation commenced in April but until June they were aimed at familiarizing with the available equipment. It was found that certain modifications / adjustments had to be effected in the apparatus, e.g. to avoid lateral movement of the couplets, achieve a perfectly horizontal application of the force and deflection, etc.

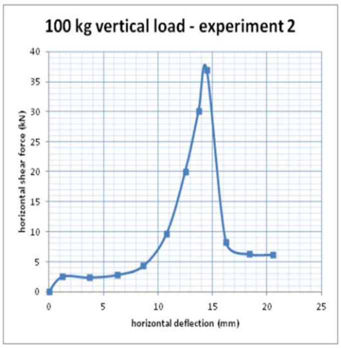
The following graphs summarize the bulk of the actual results obtained from the experiments conducted so far.

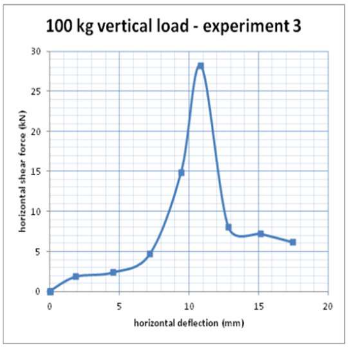


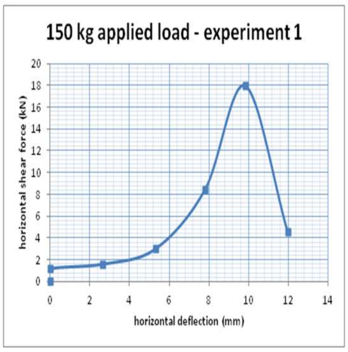


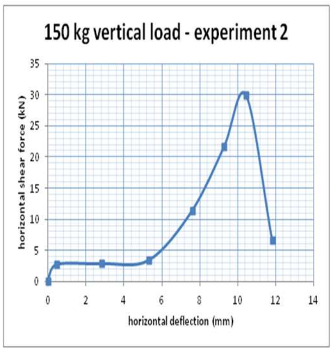


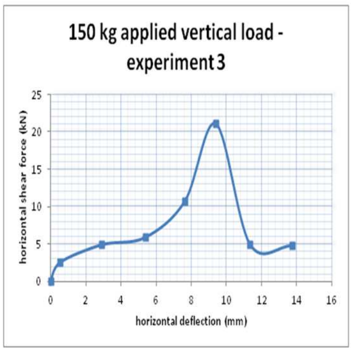


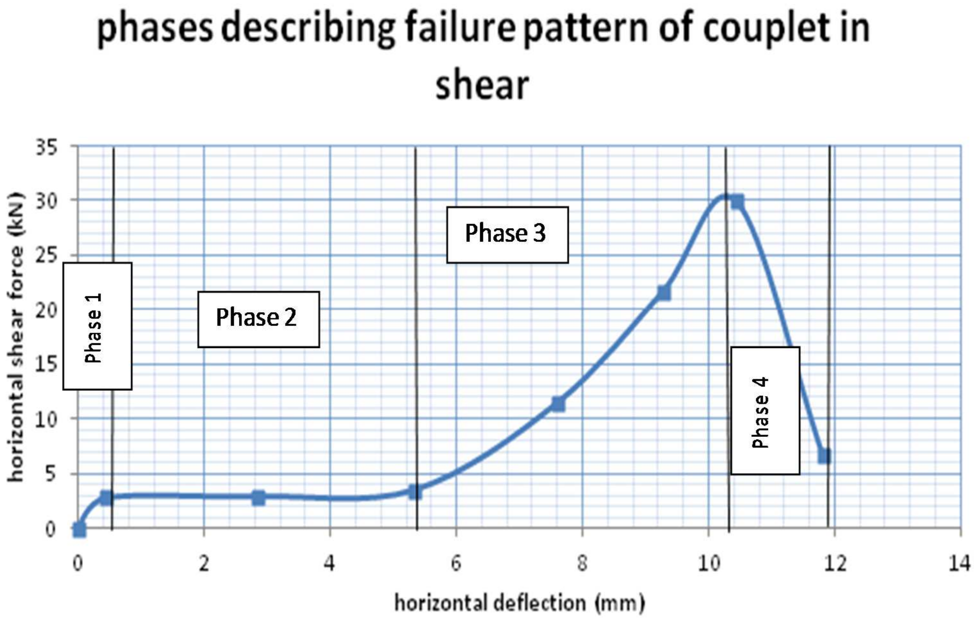












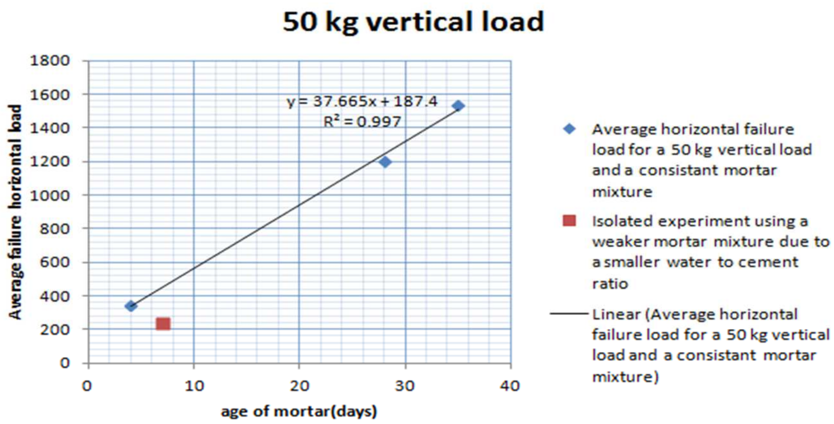
The graph shows the average failure pattern of a brick couplet in shear at 28 day testing.

The first phase of the curve indicates initially that there is a great initial resistance to the applied shear force as can be seen by the steep gradient of the curve. An explanation for this phenomenon could be that initially the load is resisted by the bond between the particles at phase 2 this bond breaks and for approximately 5mm of deflection the specimen cannot carry any additional load the specimen deflects to a point the beginning of phase 3 where it starts resisting load once again. this resistance is much greater than the initial resistance which is indicated by the gradient of the curve which is much steeper this could be due to the rearrangement of the molecules. The specimen reaches a point where it suddenly fails phase 4 at this phase the shear load is only resisted by friction between the particles as can been seen by the rapid decrease in load and rapid increase in deflection.

Preliminary Results Analysis

The vertical load applied on the couplet, represents the weight of a wall above the couplet were the horizontal force is applied.

Correlation of vertical weight to height of wall above application of horizontal force										
Applied vertical force				Rows of bricks+mortar		Height of brick+mortar		Height of wall	
50	kg	0.49	kN	13.27	rows	0.08	m	1.06	m
100	kg	0.98	kN	26.54	rows	0.08	m	2.12	m
150	kg	1.47	kN	39.81	rows	0.08	m	3.19	m



It was found that as the age of the specimen increased the vertical force required to shear the specimen increased this is an expected result as the mortar ages it increases in strength.

It was found that for 7 day testing compared to 28 day testing the specimen took longer to reach its maximum load it was also noted that the specimen did not fail abruptly as it had failed for the 28 day testing. This could be attributed to the water content in the mortar at 7 days compared to the water content in the mortar at 28 days less water in the mortar relates to a more plastic and less elastic mortar. At 7 days the mortar still contains enough water so the molecules can rearrange themselves much more easily then at 28 days were a more abrupt failure occurs. Similar to concrete the specimens have shown a more brittle nature as they age when tested in shear.

A trend was found for 28 day testing of the couplets the specimens reached their maximum load, their failure load, when the deflection of the specimen became approximately 10mm.This trend was seen regardless of the applied vertical load. This is a valuable result from this result we may be able to identify if a wall has failed .i.e. the instant the deflection of the wall is greater than 10mm it may be concluded that the wall has failed in shear.

Conclusions and direction of work

The preliminary result of failure occurring at 10mm deflection is an interesting find however more tests are required in order to achieve a better confidence in the results. It will be useful to know if this deflection at failure is sensitive to changes in the mix design, changes in the environment in which it is cured and how sensitive it is to the age of the specimen. More tests are required to obtain answers to these questions. Specimens will be produced for 7, 14 and 28 day testing, specimens will be produced using 3 different water: cement ratios, specimens will be produced for the 3 curing regimes namely air water and the solar chamber. From these tests a better analysis of the results can be obtained as well as a greater insight into the failure pattern of the couplet may be obtained.

References

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[2] K. Mosalam, L. Glascoe, J. Bernier, 2009,-'Mechanical Properties of Unreinforced Brick Masonry, Section1'

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