



SHEAR, BOND AND 2D COMPRESSIVE PROPERTIES OF THIN BED MORTAR MASONRY

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Abstract

Thin bed mortar is used more and more in The Netherlands for its ease in appliance and esthetical qualities. Therefore, more knowledge of its mechanical properties is needed. Stack bonded specimens, one brick size in section, were tested for shear, bond and compressive strength. Further, 25 wallettes were loaded in compression, equally in two directions. For these tests a test rig was developed and tested. In subsequent research, this test rig will be used for testing other ratio's of vertical and horizontal loads. Specimens with filled head joints were obviously stronger and stiffer than specimens with open head joints. When the bed joints were more completely and more regularly filled, strength and stiffness are higher too.

Key words

Thin bed mortar, bi-axial compression, failure process, shear strength.

1 Introduction

Traditionally, bricklaying is piling bricks on top of each other and the mortar allows for variations in the size of the bricks. This old skill is disappearing since it is too expensive due to the labour intensity and because the working conditions are not appealing to young people. Therefore, new ways to build with brick have to be found. Thin bed mortar showed to be easy to apply when building with calcium silicate elements [Berkers, 1995] and AAC-elements and the clay brick industries studied the possibilities of using thin bed mortar in combination with brick sized units. It showed that using thin bed mortar, a wall can be built faster than when the traditional brick laying technique is used. The first buildings already have been erected, [Martens 2002].

The size tolerance of the bricks is critical for the joint thickness and the quantity of mortar that is needed. A pump system is developed in order to be able to apply the mortar faster on the wall. A special stand is used to apply mortar easily to the headers of the bricks before they are placed. However, if this 'handling' is no longer needed, material and labour will be saved. Therefore, the effect on strength of the filling of the

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head joints was studied. It is assumed that the head joint filling only has a small effect on (vertical) compressive strength.

More knowledge of the mechanical properties was obtained using stack bonded specimens, that were tested in shear, bond and compression. Further, 25 wallettes were loaded in compression, equally in two directions.

2 Test Set-up

2.1 Materials

All specimens were made using red soft mud clay bricks, brand Rijswaard and thin bed mortar, brand Ankerplast. The mortar was delivered in two parts. The main material properties are given in Table 1.

Brick properties were taken from [CUR 193]. Mortar properties were experimentally established according NEN 3835. Therefore, three 40x40x160 mm³ prisms were made of each of the 11 batches. These prism specimens were tested in bending and subsequently the two rest pieces were tested in compression. The results showed a tendency for strength increase with specimens age and a clear distinction between delivery 1 and 2. Using Statgraphics software [NN, 2000] for the multiple regression analysis of the data, the following relationship was found:

$$f_{c,mor} = 26.13 - 5.633 \cdot D + 0.0878 \cdot A \quad (1)$$

with: $f_{c,mor}$ = mortar compressive strength,

D = parameter for the delivery number: $D = 1$ or 2 for delivery 1 or 2

A = parameter for the age of the specimen in days, $16 < A < 134$

The probability values for this relationship are $P_D = 0.0385$ for D and $P_A = 0.0192$ for A , which means that these parameters are both statistically significant at the 95 % confidence level.

Table 1 Mechanical properties of bricks and thin bed mortar.

		Brick, acc. to NEN 2489	Mortar, delivery 1 acc. to NEN 3835	Mortar, delivery 2
dimensions	mm ³	206*96*50		
compressive strength	N/mm ³	27	28.3 (3.7)	17.5 (1.57)
bending strength	N/mm ³	2.47	6.1 (1.1)	4.9 (0.7)
modulus of elasticity	N/mm ³	4000		
mass by volume	kg/m ³	1630	1814	1816

C.o.V. in % between ()

2.2 Specimens

The main test walls (MTW) and stack bonded specimens (SBS) were cut from walls using a water cooled circular diamond saw. The main test walls measured 630 x 630 x 96 mm³. The stack bonded specimens measured 210 x 96 x 370 mm³, see Figure 1.

2.3 Building of the walls.

Mortar was mixed in a ratio of 5.5 litres of water to 25 kilos (one bag) of dry factory made mortar, using a hand held mixing machine. Before using the bricks, they were rubbed against each other to remove the sanding and to allow for a better mortar-brick bond. The mortar was applied as usual in practice, using cord guiding. Instead of using a pump and nozzle system, first, a piping bag (usually used to put whipped cream on a cake) was used. In practice, the advantage of using a 'piping bag' is that there is only little to clean, and the work can be interrupted more easily.

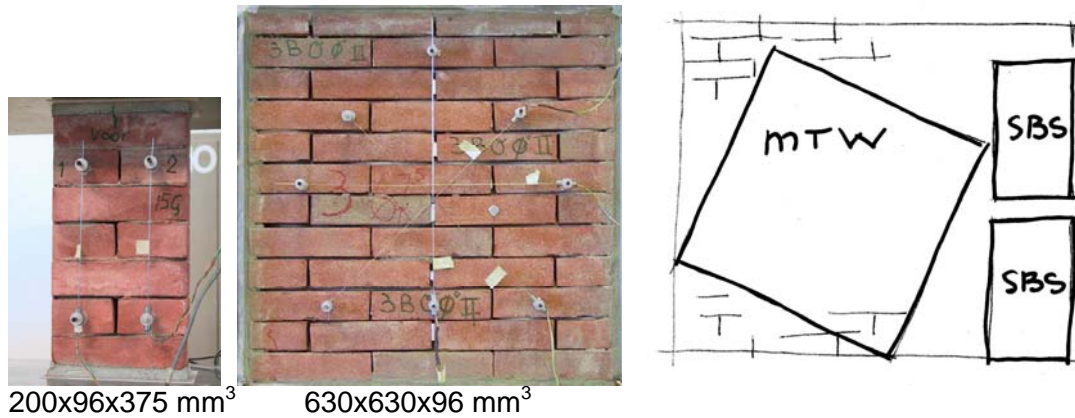


Figure 1 Stack bonded specimen and main test wall, cutting scheme.



Figure 2 Positioning of the bricks, using cord guiding.

Some force is needed to press the mortar out of the bag. Therefore, later a trowel was used. Consequently, the joints have a more irregular shape than when the nozzle system would have been used.

After the mortar needed for a few bricks was applied, the bricks were pushed into the mortar. Starting from the back and rotating the brick over its length, the mortar was pushed to the 'clean' side of the wall, leaving a notch-like opening at the clean side of the wall. Walls with filled and unfilled header joints were built.

The finished walls were covered with plastic sheets to prevent them from extreme drying conditions.

2.4 Preparation of the specimens

The wallettes (MTWs) were cut from the larger walls, Figure 1. Then they were positioned in the test set-up on steel bars with a diameter of 10 mm and the moving load platens were positioned, leaving a space of 10 mm all around. Next, these spaces were filled with a high strength pointing mortar, brand Beamix 5*, by using 'pointing' tools. Two 0.05 mm thick Teflon sheets and one PVC protection sheet were placed between the steel loading beams and the pointing mortar to reduce friction.

The SBS specimens were capped outside the machine. Therefore they were positioned on a flat steel platen in an amount of fresh mortar and a joint of 10 mm thickness was formed. Then, mortar was placed on top of the specimen and levelled by pushing a piece of plywood into the fresh mortar. A spirit level was used to obtain a surface parallel to the bottom surface.

2.5 Test frame

The MTWs were tested in a specially built test frame. The design was based on ideas from Page [1989] and a 3D-test rig originally built to test concrete cubes, [Van Mier, 1997] later adapted by Vonk and Van Geel. The columns and beams of this frame were made out of double HE300B steel profiles. The holes in these profiles allowed for simple and strong bolted connections. Two times two 500 kN jacks were mounted in the frame, Figure 3. The load was transferred into the specimen using 'ball bearings' and steel profiles with a section of 100 x 100 mm². The four jacks (marked .5) were connected via a distribution unit to a 1000 kN jack (marked 1 in Figure 3).

This 1000 kN jack was placed in a Schenck 2.5 MN testing machine and loaded, using the displacement control of this machine. Via the hydraulic system, all four jacks smoothly received the same oil pressure and consequently the specimen was loaded equally both horizontally and vertically. In Figure 3, **p** is a sensor for the oil pressure, **m** is a manual oil pump and **l** indicates the position of the LVDT's on the specimen.

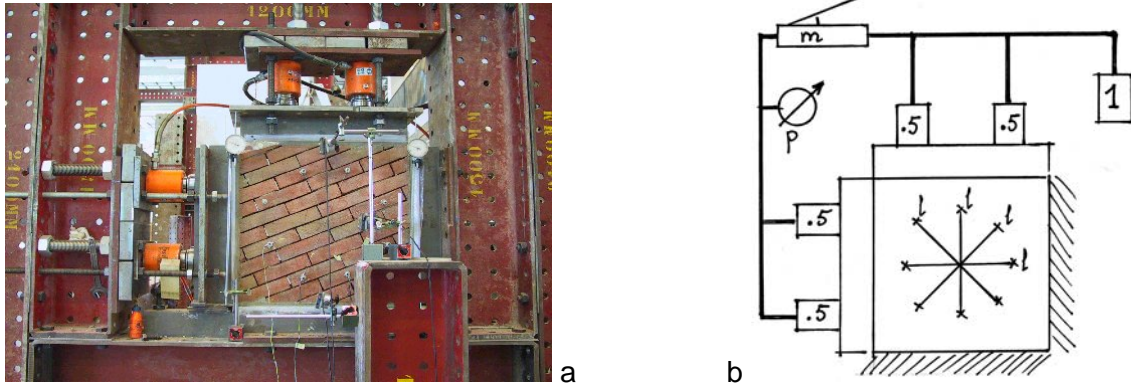


Figure 3 a) Test frame and b) hydraulic scheme and position of LVDTs

2.6 Measurements.

The specimens (SBS) were tested in a 2.5 MN compression testing machine. Deformations were measured with two LVDT's on the front and two on the back. These tests were performed with a loading speed of 0.1 mm/min.

In the MTW tests, oil-pressure in the system was recorded, in combination with the load applied on the 1000 kN jack. Deformations of the wallettes (MTWs) were measured with four LVDT's on the front and four on the back side, parallel and perpendicular to the bed joints and under an angle of plus or minus 45°.

3 RESULTS

3.1 Stack bonded specimens

Uni-axial compression tests were performed on both stack bonded specimens with open and with filled head joints. The specimens with open head joints had a compressive strength of 11.5 N/mm² and an E-value of 2990 N/mm². For the filled head joint specimens these values were $f_{c,mas} = 13.4$ N/mm² and $E = 3770$ N/mm² respectively. It showed that the deformation on the side where the joints were not fully filled were largest, due to the bending of the specimen. The E-values were established using the averaged result of the four LVDT measurements.

Bond strength tests were performed on similar specimens as used for the compression tests. The specimens were loaded in four point bending. The specimens had various lengths and therefore they were broken in two, giving one result. Then, the rest pieces were used in a similar way to obtain another result. The effect of the weight of the specimen was taken into account. At first, the span was 480 mm and later 162 mm. The distance of the two point loads was 55 mm, which was about the thickness of one brick (50 mm) and one joint (3 to 4 mm).

The gross area of the section was $A_{gro} = 198$ cm², the gross section moment is $W_{gro} = 315$ cm³. The averaged net area of 18 sections was established using the accurately measured dimensions of the circumference of the bonded area. In average, $A_{net} = 129$ cm² (C.o.V. was 20%) and the net section moment was 162 cm³ (C.o.V. = 31 %).

It showed that the strength related to the net area of the section was approximately twice the strength obtained with the gross section. The flexural bond strength was

0.85 N/mm² (C.o.V. was 32 %) when the theoretical gross section moment of $W_{gro} = 315 \text{ cm}^3$ was used. When W_{net} was used the averaged flexural bond strength increased to 1.71 N/mm² (C.o.V. was 36%).

Shear strength was established using three brick, two joint specimens, similar to the SBS used for compressive testing. The specimens were loaded in shear under three different prestresses [EC6, 1996]. From the shear load deformation diagram the shear strength (maximum value, F_{max}) and the residual strength (dry friction, F_{res}) were established, see Figure 4.

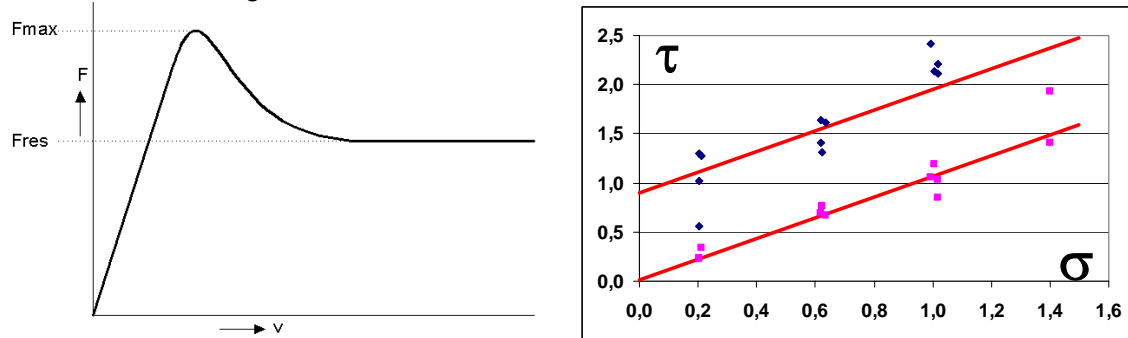


Figure 4 Shear-load versus displacement. Prestress shear strength relationship

Using multiple regression, the following relationship between preload and shear strength was found:

$$\tau = 1.15 \sigma + 0.835 \quad (2)$$

When the residual values are also taken into account, the relationship becomes:

$$\tau = 1.065 \sigma + 0.892 * C \quad (3)$$

in which: τ = shear strength,

σ = prestress perpendicular to the shear surface

C = parameter for the uncracked ($C = 1$) or the cracked situation ($C = 0$)

The friction coefficient related to the gross section in the cracked situation was 1.065.

3.2 Main test walls.

In Table 2 the averaged values for compressive strength of each series of similar tests are presented. In Figure 5a the strength of each specimen is plotted versus its age, and in Figure 5b the averaged strength of each series is plotted versus the bed joint orientation. The lowest result (7.9 N/mm²) was found for the single specimen with filled head joints and a bed joint orientation of 45°. This specimen had poor bed joint filling. Further, some practical irregularities occurred during testing. Therefore, this result may be omitted. The averaged values are: $f_{c,mas} = 10.1 \text{ N/mm}^2$ for the open and $f_{c,mas} = 11.6 \text{ N/mm}^2$ for the filled head joint masonry. The SBS results were higher as a result of the smaller slenderness.

Table 2 Averaged compressive strength; a) of MT-walls versus bed joint orientation, b) acc to EC 6 and c) of SB Specimens in N/mm²

head joint	bed joint orientation to vertical axis						EC6	SBS
	0°	22.5°	45°	67.5°	90°	avg		
open	9.5	9.8	10.4	11.1	9.9	10.1		11.5
filled	12.9	11.3	7.9	11.7	12.3	11.6	10.0	13.4

open: 3 tests per orientation, filled: 2 tests per orientation except 45°.

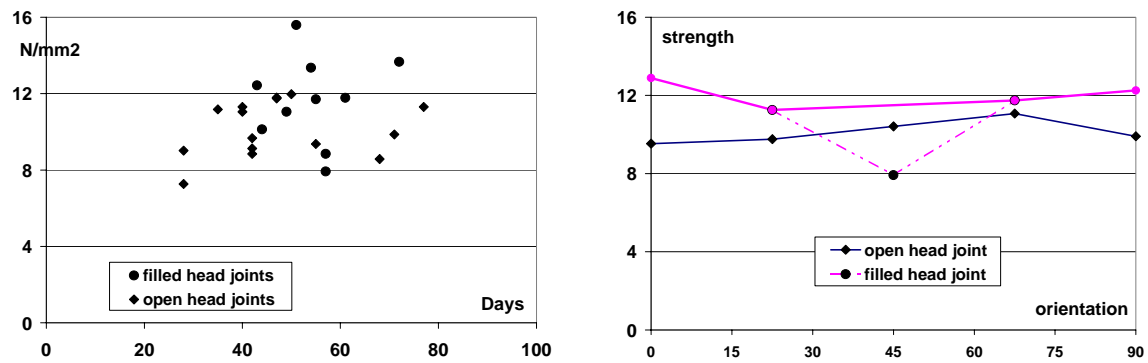


Figure 5 Compressive strength versus age and versus bed joint orientation (averaged)

3.2.1 Relationships between strength and test parameters

The parameters of this test series were: the mortar compressive strength (M), the specimens age in days (A), the bed joint orientation with respect to the vertical (O), the way of filling the head joints, filled or not deliberately filled, (F). Using multiple regression analyses [Statgraphics, 2000] the following relationship was established for the main parameters in the 25 tests:

$$f_{c,mas} = 5.031 + 0.215 \cdot M - 0.0121 \cdot O + 3.201 \cdot F - 0.0062 \cdot A \quad (4a)$$

or

$$f_{c,mas} = 5.097 + 0.202 \cdot M - 0.0117 \cdot O + 3.056 \cdot F \quad (4b)$$

with $f_{c,mas}$ = masonry compressive strength. The effect of age (A) was neglected in relationship (4b) while the variation of strength related to the aging of the specimen shows to be relatively small, contrary to the results of the mortar prisms where some age effect was found. The difference between the weakest and the strongest mortar was 18 N/mm^2 , or a maximum contribution to $f_{c,mas}$ of $18 \cdot 0.215 = 3.87 \text{ N/mm}^2$. It showed that the contribution of orientation is minor. The probability values (P) are given in Table 3. There is not a statistically significant relationship between the variables at the 90% confidence level. However, when the not significant parameters O and A are removed from the model, the reduced model becomes:

$$f_{c,mas} = 6.668 + 0.126 \cdot M + 2.439 \cdot F \quad (5a)$$

or

$$f_{c,mas} = 10.137 + 1.509 \cdot F \quad (5b)$$

with probability values of $P_M = 0.206$ and $P_F = 0.026$ in (5a) and $P_F = 0.054$ in (5b) respectively. Then, the effect of filling is still significant on the 95% confidence level.

Table 3. Contribution of parameters in model (4a)

	Regressor		variabele	P	Range
M	mortar strength	N/mm ²	0.2150	0.21	$16,4 > M > 34,4$
O	Orientation	deg (°)	-0.0121	0.47	$0 > O > 90$
F	Open/filled head joints	---	3.2010	0.06	0 or 1
A	Age of specimen	days	-0.0062	0.87	$28 > A > 77$

According to EC6 Draft prEN 1996-1-1 [EC6, 1996] the characteristic compressive strength for the thin bed mortar masonry used equalled 10.0 N/mm^2 . However, in [EC6, 1996] it is assumed that the mortar strength has no effect on compressive strength in this type of masonry. This is in contradiction with the experimental findings.

In this project however, the joint thickness was approximately 4 to 5 mm, while [EC6, 1996] refers to a joint thickness smaller than 3 mm. Further, the joints were not completely filled and the amount of filling varied considerably. No correction was made for the height over thickness ratio. However, bending of the specimen due to the unfilled parts of the bed joint affected the results as well.

The results of the Mutitoyo's that measured the deformations perpendicular to the 630 mm square surface showed that the specimens moved and bended, which may be explained by the fact that the joints were not completely filled at one (the esthetical) side of the wall. Deformation due to bending was also found in numerical simulations which were made for complete joint filling, and for 85% and 70% filling respectively. Figure 6 shows the deformation in horizontal and vertical direction. The specimens centre moved 0,11 mm to the left. The vertical deformations for a load of 10.74 N/mm² shown in Figure 6 right, resulted in an overall E-value of 3618 N/mm².

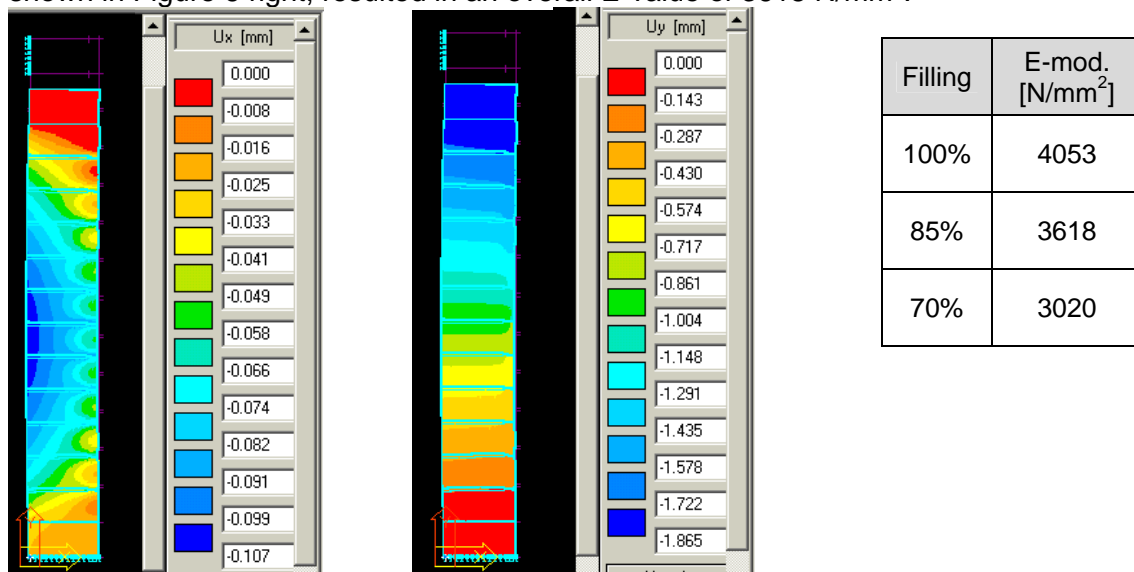


Figure 6 Numerical simulation of wallette (MTW) behaviour, 85% filling, deformation in X (left) and Y direction and E-values.

3.2.2 Failure

The following three phenomena were observed in the failure process of the wallettes (MTWs): 1) Spalling of the bricks, with fragments of approximately 20 mm thick, 2) vertical splitting of the specimen and 3) bending of the specimen. Figure 8 shows remaining pieces after testing.

As in practice, the joints were not completely filled to keep one surface of the wall clean and appealing. In this way, the recessed joints formed notches. Because a high strength, thin bed mortar was used, it may be assumed that stress concentrations occurred at the end of the mortar joint. These kind of stress concentrations were confirmed in numerical simulations, Figure 7. Consequently, tensile stresses develop in the brick which may cause premature failure. This also explains the spalling of the bricks.

Averaged compressive stress in the bed joint equalled 12,69 N/mm² while the peak stress was 28,85 N/mm². It may be concluded that peak stresses acted very locally while the ratio between averaged stress and applied stress was $10.74/12.69 = 0.85$, which is almost equal to the filling ratio. The tensile peak stress in the brick equalled almost 2.0 N/mm², the tensile strength of the bricks was 2.4 N/mm².

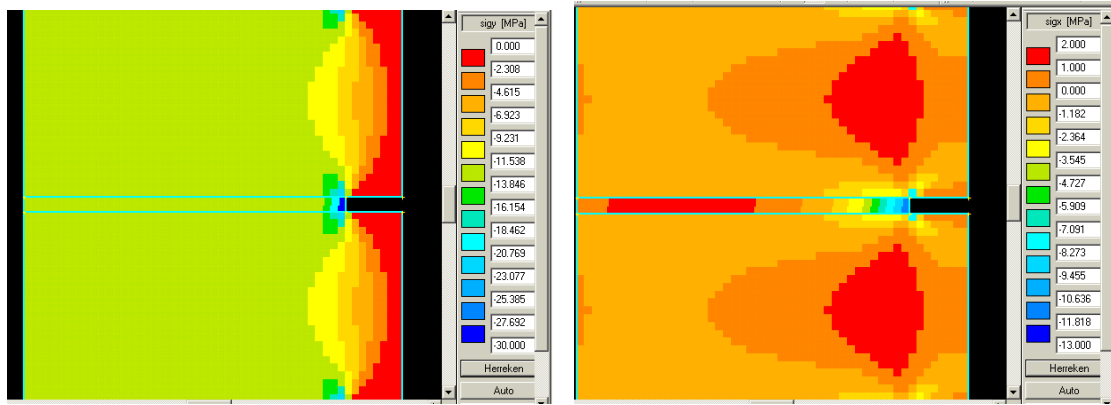


Figure 7 stress concentrations at the notch, numerical results in Y and X direction

The joints were reasonable filled at the back of the MTWs. In combination with the already mentioned notch at the front side, the load was transferred with an eccentricity of approximately 10 mm from the centre of gravity of each joint section.

In specimens where the joints were better filled, less spalling occurred. These specimens were stronger. In those cases the more traditional way of splitting failure could be observed.



Figure 8 Remaining pieces showing spalling at the 'notch' i.e. the end of the bed joint

3.2.3 Stiffness obtained from LVDT results.

Deformations of the wallettes (MTWs) were measured using LVDTs in four directions, parallel and perpendicular to the bed joints and under an angle of plus or minus 45° with the bed joint. Due to bending, large differences were found between the results measured at the front and the back of the specimens. Therefore, these results were averaged, resulting in the strain in the centre of the specimen in the four mentioned measuring directions.

To establish E-values, a linear best fit of the form $\sigma = E \cdot \varepsilon + C$ was established on a part in the σ - ε graph between approximately 15% and 80% of the failure load. The obtained E-values are presented in Table 4. For each measuring direction the average and C.o.V. are given first. Then, the extreme values in each group were omitted and the averaged values and C.o.V. for the reduced number of tests are given as well.

It can be seen that E_{bed} , measured perpendicular to the bed joint is smaller than the E_{head} , measured parallel to the bed joint, due to the different amount of mortar per unit length and due to differences in brick properties.

Values for masonry with filled head joints are larger than those for masonry with open head joints, as expected. Measured in bed joint direction the values are largest, both in the filled and the open head joint situation.

For comparison, the E-value in diagonal direction was estimated ($E_{d,est}$) using the results of measurements in vertical, horizontal and diagonal direction and the following equation:

$$\frac{2}{E_{d,est}^2} = \frac{1}{E_{hor}^2} + \frac{1}{E_{ver}^2} \quad (7)$$

while $\Delta l_{diag}^2 = \Delta l_{hor}^2 + \Delta l_{ver}^2$. Under the assumption that the horizontal and vertical gauge length are equal, the length of the diagonal is $\sqrt{2}$ times this gauge-length. Using formula (7) the found diagonal values match the experimental results reasonably well. Compare the values for $E_{d,est}$ in Table 4 with the $E_{d,avg}$ values.

Table 4 Averaged values for the modulus of elasticity measured in various directions, reduced number of test-results when extreme values are omitted.

Open head joints

	E_{bed}		E_{head}		$E_{d,est}^{(1)}$	E_{diag1}		E_{diag2}		$E_{d,avg}^{(2)}$
AVG	3873	4054	6506	5848	4567	4318	4686	5054	5281	4657
C.o.V.	24	18	35	17		23	10	26	22	
# test	15	14	15	13		15	13	15	14	

Filled head joints

AVG	4644	n.e.	6383	6914	5452	5207	n.e.	5323	n.e.	5265
C.o.V.	22		27	13		18		23		
# test	10		10	9		10		10		

1) using formula (7)

2) $1/E_{d,avg} = 1/E_{diag1} + 1/E_{diag2}$

n.e. = no extreme value

4 Discussion

The effects of the filling of head joints were disrupted by the (unintended) use of two mortar deliveries and changes in the application method of the mortar. Therefore, in sub sequent research, the joints should be filled more uniformly, and the depth of the recessed joint controlled more systematically. In statistical sense, the relatively large number of tests was positive. A large deviation was found but the results from various methods compared reasonably well.

Whether the use of a linear regression model is allowed, may be questioned since the parameters not necessarily have a linear relationship and cross correlations were omitted.

In the analysis of the shear test results the theoretical model is linear so a linear regression model fitted quite well. For the analysis of deformation measurements an attempt was made to incorporate the diagonal results.

Because of the recessed bed joints, (notches) the specimens were not loaded in the centre of gravity of the specimen and this resulted in bending of the specimen. In subsequent research, the effect of bending should be controlled as well.

The filling of the joints depends on the method used for the application of the mortar and has a direct effect on strength. To study certain effects, the recessing should be controlled. Perhaps for experimental purposes, the joints should be filled completely first and then raked using a tool to become a predetermined depth. The amount of filling should be visually inspected. In subsequent research, more accurate control of the depth of the recessing is necessary.

The effect of 'notch-action' of the bed joints should be taken into account when formula (6) taken from [EC6, 1996] is used.

As expected, no effect of bed joint orientation was found under the equal horizontal and vertical loads that were applied. In subsequent research, the load ratio will be varied. The Teflon sheets allowed for free movement, however some small effects of asymmetry of the test set-up were found. The asymmetry was caused by the fact that only one horizontal and one vertical load platen moved, while the other two platens were rigidly connected.

5 Conclusions

There was no significant effect of mortar compressive strength on masonry compressive strength. Mortar compressive strength increased with age.

The amount of joint filling both in bed and head joint, directly affects the compressive strength.

The use of the net area properties versus the gross area properties gave a negligible improvement of the deviation of the results.

Bricks spalled because the unfilled part of the bed joint acted as a notch and caused peak stresses.

Table 5 Resume of averaged strengths and E-values in N/mm².

Head joint	Bond	Shear	$f_{c,mas}$	E_{head}	E_{bed}
Open	--	--	10.1	5600	4000
Filled	0.85	0.85	11.6	6500	4500

Gross area: $A = 96 \times 206 = 198 \text{ cm}^2$, $W = 315 \text{ cm}^3$.

E_{head} measured // bed joint, E_{bed} measured \perp bed joint.

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