Bed joint reinforcement and the shear capacity of masonry beams

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ABSTRACT: The design of reinforcement in masonry beams is strait forward. A few, relatively small diameter bars in one or two bottom layers is often sufficient to form a tie while in the masonry compressive struts develop. In the layers further away from the tensile area, often reinforcement is applied with the idea that this decreases the risk of crack development and might increase shear capacity. Masonry beams with a height of 490 mm, 550 mm or 625 mm and a span of 1400 mm, with and without extra bed joint reinforcement were tested. The results of these tests are presented and the effect of the extra reinforcement on shear capacity discussed. In a number of cases, bed joint reinforcement with a closed hoop configuration was applied. Extra reinforcement did not always increase shear capacity; on the contrary, the capacity was sometimes smaller because the extra reinforcement affected the capacity of the compression struts negatively.

1 INTRODUCTION

Structural engineers, familiar with the relatively low tensile strength of masonry, prescribe the use of reinforcement at places where tensile stresses might become critical. One of the solutions to span an opening in a masonry wall is to use reinforcement. A masonry beam, comparable with a concrete beam, is made and like any other structural member, the masonry beam is supposed to resist normal forces, shear forces and bending moments.

In case of a composite consisting of anisotropic materials, like masonry or concrete, failure due to flexure and shear can occur in several ways. In a beam design checks for bending at mid span and shear must be made, Christiansen, (2008), Vermeltfoort & Martens (2015). In concrete, reinforcement in the shape of stirrups increases shear bearing capacity. Vertical, i.e. perpendicular to bed joints, reinforcement in masonry is hardly possible to apply. However, can (extra) bed joint reinforcement increase shear capacity?

1.1 *Types of Reinforcement*

Bed joint reinforcement consists of steel wires and is available in various diameters and widths. The wire yield strength usually exceeds 450 MPa. To effectively make use of the applied reinforcement, sufficient bond is required. This is usually obtained by providing enough bond length or anchorage. The connection points of bars also increase load transfer from bars into the masonry. Two main types of bed joint reinforcement are available, the ladder type and the truss type, shown in Figure 1.

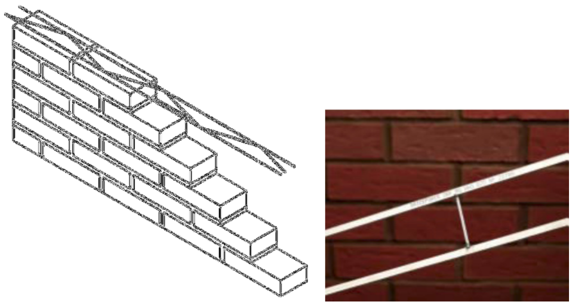


Figure 1 Example of the truss type bed joint reinforcement, redrawn from Drysdale [2008] and ladder type reinforcement.

1.2 *Contents of paper*

In this paper, the results of two series of tests, with and without extra shear reinforcement are presented and compared to results of 21 tests discussed earlier, Vermeltfoort (2010).

The focus is mainly on the effect of bed joint reinforcement in the neutral zone on load bearing capacity for which shear is considered the main characteristic. However, the stiffness of the tie is of importance. Because the critical amount of reinforcement in relation to bending is relatively small this results in a flexible tie. For practical reasons a heavier tie is usually not possible.

2 TEST PROGRAM AND SET UP

Table 1 gives an overview of the test program. Twenty walls were tested in three series, SA, LC and AM, in sets of two (10x) or three (2x). Results were discussed earlier (Vermeltfoort (2010). Eight walls, two times two sets (R1,1, R1,2 R2,1 and R2,2) were tested and the results are discussed in this paper. Each sets of tests had another main parameter.

Table 1 Overwiew series of tests and parameters

Serie	#	h mm	# of layers	type number
SA-A	3x	505	10	1
SA-B	3x	505	10	2
LC-A	2x	491	10	3
LC-B	2x	491	10	4
LC-C	2x	491	10	5
AM-B	2x st	865	13	7
AM-C	2x st	685	12	8
AM-D	2x st	490	10	9
AM-E	2x st	492	10	10
R1-1	2x	675	11	11
R1-2	2x	680	11	11.2
R2-1	2x	620	10	12
R2-2	2x	625	10	12.2

All walls were loaded till failure. Among the specimens variations were made in height and configuration of the bricks (st = stack bond, Series AM). The height of the specimens was altered to verify whether a steeper compression diagonal has a beneficial effect on the shear capacity and a continuous perpend joint (Series R2,1 and R2,2) was applied to activate the bending reinforcement immediately after starting of the loading of the specimens.

Characteristics of the specimens of series R1 and R2, given in Table 2, are:

- Span of 1350 mm
- Distance between to two point loads of 490 mm
- Thickness is equal to the brick width, 96 mm.
- Height of 613 and 675 mm respectively,
- Stretcher bond is applied
- Series R2 contains a continuous vertical joint at mid span from the bottom up to the ninth layer.

Table 2 Overview specimens.

type	height ##	mm	layers with reinforcement*) 1500	710	600 **)	#
R1-1	11	675	1, 2			2
R1-2	11	675	1, 2	7, 9	3, 5	2
R2-1	10	613	1, 2			2
R2-2	10	613	1, 2	7, 9	3, 5	2
					total	8

*) count from the bottom of the specimen.
**) length of reinforcement

2.1 *Materials*

2.1.1 *Bricks and mortar*

For all specimens, one kind of brick was used, brand Rijswaard which is a type of soft mud machine mold brick with sanded surfaces. The most important properties are listed in Table 3.

Table 3 Main properties of the bricks used.

size (l x w x h)	206 x 96 x 50	mm^3
density	1630	kg/m^3
normalized compressive strength	27.0	N/mm^2
initial water absorption	4.0	$kg/m^2/min$
when used building*	1.5	$kg/m^2/min$
bond strength	>0.20	MPa

*IRA when constructing masonry between 1.3 and 1.7 $kg/m^2/min$ (Haller getal). Bricks were pre wetted by submerging them for 3 minutes.

The mortar was Portland cement (CEMI based. Simultaneously with the building of the specimens, three mortar prisms per batch were made for compressive testing according to EN 1015-11.

The average compressive strength of the mortar prisms, made simultaneously with the main walls in this paper, is 5.51 N/mm2 which is approximately 10% higher than the value specified by the manufacturer. Mortar properties for other walls were also extensively tested as described in Vermeltfoort 2010.

2.2 *Bed joint reinforcement*

Epoxy coated Murfor RND/E, a truss type bed joint reinforcement, was used. The properties are presented in Table 4. Bonding of the short pieces of bed joint reinforcement, 710 and 600 mm in length, was improved by welding a cross bar at the end as shown in Figure 2.

Table 4 Properties of Murfor RND/E

Width of truss	50	mm
Length	1500, 600, 710	mm
diameter of outer bars	4.00	mm
diameter of center bar	3.75	mm
average tensile strength	729	N/mm^2

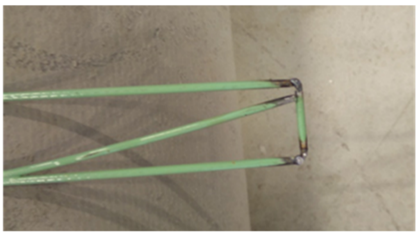
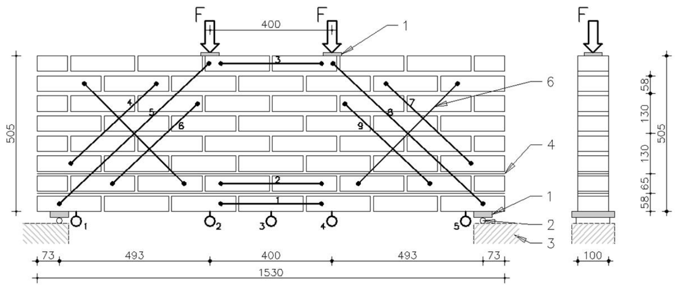


Figure 2 Welded end of the bed joint reinforcement forming a loop.

2.3 *Experimental set-up*

Figure 3 shows the set-up. The dashed lines indicate the bed joint reinforcement. The load from the hydraulic jack is transferred to the specimen as shown in Figure 3.



1. Steel platen 2. Steel cylinder support
3. Steel supporting block,
4, 5 and 7. Murfor reinforcement l=1530 mm, l = 600 mm and l = 710 mm. 6. LVDT's

Figure 3 Reinforcement and position measuring devices

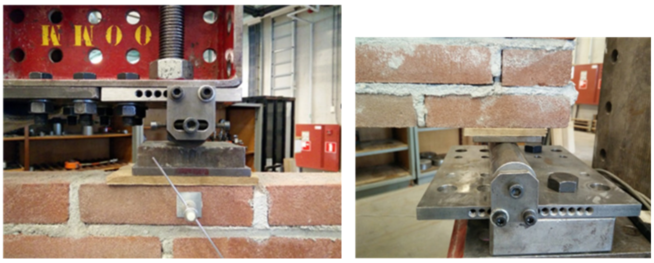


Figure 4 Load introduction point and support, soft board and steel platen used to spread the load evenly.

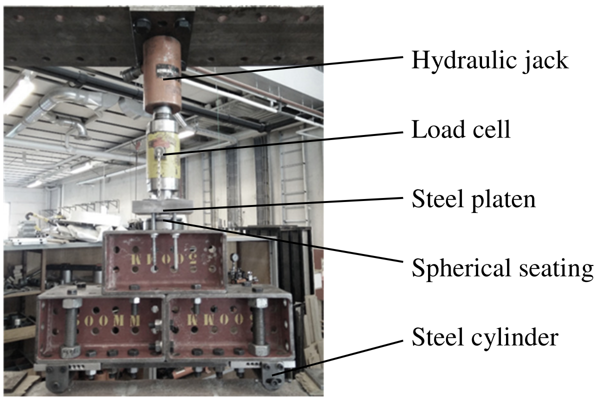


Figure 5 load introduction, via soft board

Mitu_1 and Mitu_5 (Figure 3) were intended for measuring the displacement of the specimens near the supports. Digital displacement gauges measured the vertical displacements at five positions, giving an indication of the curvature of the specimen. However this curvature was expected to be small and negligible.

3 EXPECTATIONS

The expectations of the experiments are predicted based on Eurocode 6 and previous research.

3.1.1 *Based on Eurocode 6*

The expected load bearing capacity of two types of specimens was estimated for a height of ten and eleven layers respectively, Christianson (2008), Vermeltfoort & Martens (2015). Both shear and bending resistance were calculated.

When considering bending, only the bottom two layers of bed joint reinforcement were taken into account. For shear no reinforcement was taken into account, because Eurocode 6 does not allow. The results from bending and shear calculations are shown in Table 6.

Table 5 Estimation of bending capacity based on Eurocode6.

specimen	#layers	height mm	As mm2	z mm	MR kNm	2F kN	τ MPa
	bending moment at mid span						
h10	10	613	72.35	517	18.7	86	0.85
h11	11	675	72.35	542	19.6	90	0.80
	shear (based on height times thickness)						
h10	10	613				38.8	0.38
h11	11	675				40.6	0.36

The specimens will fail due to shear rather than due to flexure. However, it still is expected that the maximum load will approach the loads given in Table 6 for bending rather than the loads given for shear.

This is probably due to the fact that the method of determining the shear resistance is based on experiments and not on a realistic physical model.

3.1.2 *Based on previous research*

Previous research, Vermeltfoort, (2008) indicates that an improvement in shear resistance was found by applying bed joint reinforcement, as indicated by the results given in Table 8. Compare the results of series SA-A with those from series SA-B and in a similar way those form series LC-A, LC-B and LC-C.

Below, ultimate load shear stresses are presented. Comparison at a lower load level, i.e. directly after significant cracking shows that differences are smaller.

series	τ	%
SA-A	0.53	100%
SA-B	0.91	172%
LC-A	1.39	100%
LC-B	1.90	137%
LC-C	2.09	150%

Based on the results presented above, it was expected the specimens in this research will fail at about the same or a perhaps a little higher load level. The specimens from series R1 and R2 are higher and in higher specimens the load is transferred more directly to the support due to arch action.

4 RESULTS.

Results from series R1 and R2 are presented, based on two levels: at ultimate load and at the start of significant cracking. These results are compared to the results of the earlier tests, Table 8.

4.1 *Load – mid-span deflection relation*

The load–mid-span deflection relation was obtained by combining the data of digital displacement gauges Mitu_1, 3 and 5. Mitu_1 and 5 were needed to measure the compression of the soft board at the supports to correct the readings from Mitu_3. Also, the graphs are shifted to intersect with the point where the total load and (corrected) mid-span deflection equals 10 kN and 0 mm respectively. This way, the settling of the specimen and test setup and compression of the soft board is taken into account. Therefore the graphs can be compared better. The mid-span deflection is presented in Figure 6 and Figure 7.

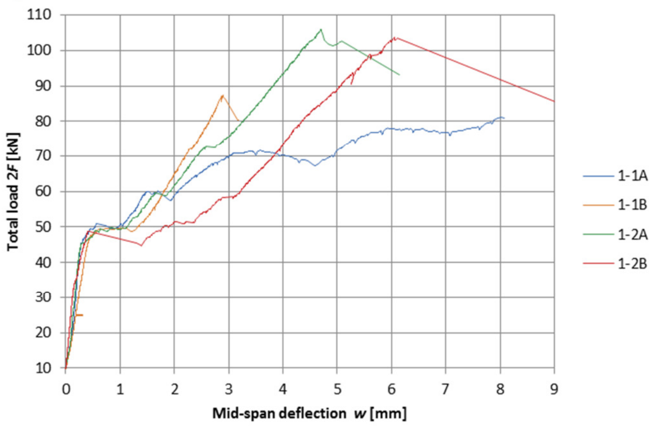


Figure 6 Load-mid span deflection of Series R1,1 and R1,2

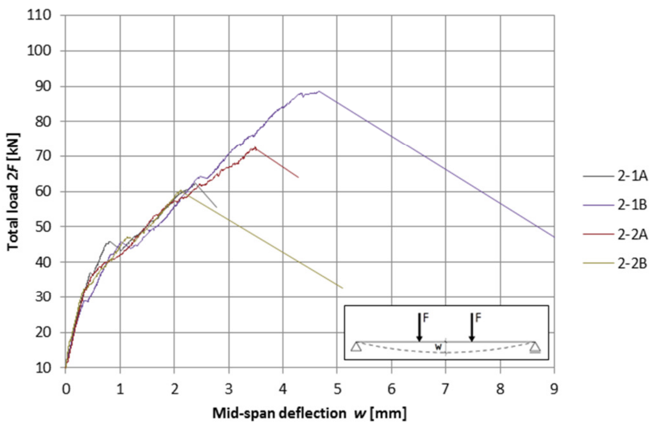


Figure 7 Load-mid span deflection of Series R1,1 and R1,2

In general the graph in Figure 6 shows linear elastic behavior until 2F ≈ 49 kN. Next, cracking occurs and the deflection increases while the load remains the same. Then the load increases and the deflection also, but in a less stiff way. At the same time cracks grow bigger and more cracks arise, until the specimen ruptures. Specimen R1-1A differs from the other specimens after the first crack occurred.

Table 6 Results. Fcrack and Fcollapse shear load

	Fcrack kN	Fcollapse kN	hoogte mm
SA-A-1	16.00	25.00	505
SA-A-2	21.00	26.00	505
SA-A-3	20.00	35.00	505
SA-B-1	20.00	44.00	505
SA-B-2	24.00	28.00	505
SA-B-3	23.00	44.00	505
LC-A-1	14.90	60.80	491
LC-A-2	12.40	51.50	491
LC-B-1	15.60	-	491
LC-B-2	17.80	76.40	491
LC-C-1	16.40	72.00	491
LC-C-2	21.00	96.80	491
AM-A-1 st	54.00	54.00	1053
AM-B-1 st	38.91	55.50	861
AM-B-2 st	39.73	66.00	868
AM-C-1 st	26.07	30.50	687
AM-C-2 st	28.62	37.50	682
AM-D-1 st	12.28	28.50	490
AM-D-2 st	17.65	22.50	490
AM-E-1 st	15.11	24.00	491
AM-E-2 st	16.00	20.00	492
R1-1A	25.5	40.6	675
R1-1B	23.65	43.65	675
R1-2A	22.5	53	680
R1-2B	24.35	51.85	682
R2-1A	20	31.15	620
R2-1B	20	44.3	620
R2-2A	20	36.3	625
R2-2B	20	30.2	625

The specimens of series R2, Figure 5 2, show different behavior. The load – mid-span deflection relationship of these ‘pre-cracked’ specimens’ is almost bi-linear with a kink at about 2F ≈ 35 kN.

At the kink, more and more cracks start to originate and grow bigger. This continues until failure. Although the behavior of all specimens is similar, the failure loads vary considerably.

4.2 *Loads at the onset of cracking*

The loads at the onset of cracking were, per test, derived from the load deflection graphs. From these loads shear stresses were established and plotted versus their type number in Figure 10. The mean value for all tests is indicated by a horizontal line. Maximum and minimum borders are also drawn, at values plus or minus two times the standard deviation.

No real extremes were found. The onset of cracking occurs on a similar load level (shear stress) for all types. Un cracked, masonry behaves "the same".

The load at which cracking started hardly differed for the specimens from series R1 and R2, type number 11 and 12,

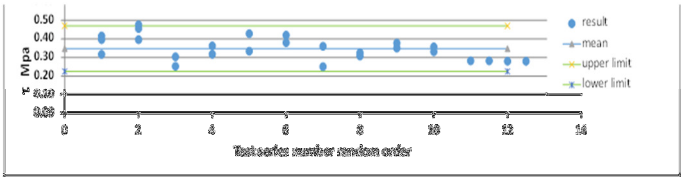


Figure 8 τ-crack results versus test type

4.3 *Failure loads (Maximum)*

In Figure 11 τ-ult is plotted versus the number of the series. The tests with a stiff tie are marked (series 3, 4 en 5) and not taken into account when calculating the mean value. The mean of the remaining series is indicated. The deviation (C.o.V.) for the ultimate loads is larger than the deviation for the loads at the onset of cracking. This shows the variation in bond and probably the negative effect of reinforcement in the relatively thin joints, i.e. a reinforcement diameter of 4 mm and a joint thickness of 11 to 12 mm.

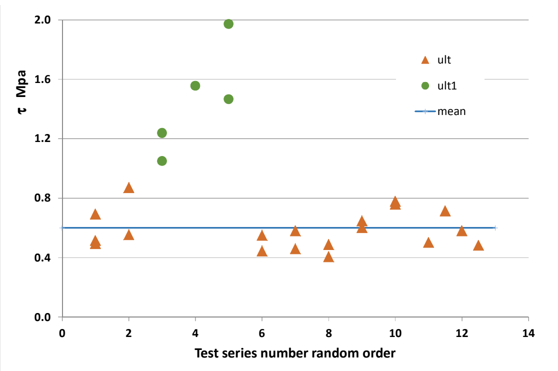


Figure 9 τ-ult results versus test series number

4.4 *Cracking patterns.*

Figure 12 shows examples of cracking patterns. The direction of cracks followed more or less the joint pattern; however, the load introduction at the support may have initiated the cracking process.

4.1 *Other observations*

1. (extra) reinforcement creates more connection especially after the masonry has cracked.

2. (shear)capacity, i.e. shear stress at the onset of cracking, with reinforcement is not significant higher than without reinforcement. Ultimate load has a larger deviation.

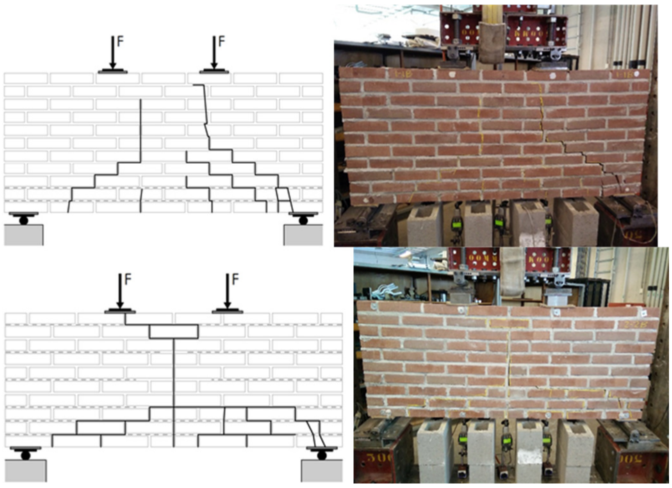


Figure 10 example 2-1B and 2-2B (pre cracked)

3. continuing reinforcement in the area around the neutral bending axis causes, after (bending) cracking occurs, additional tension in the shear area, Figure 11. This effect decreases the bearing capacity of the compression strut,.

4. the effect of (vertical) connection between compression zone and tensile zone (in concrete by vertical stirrups) has (not yet) been researched.

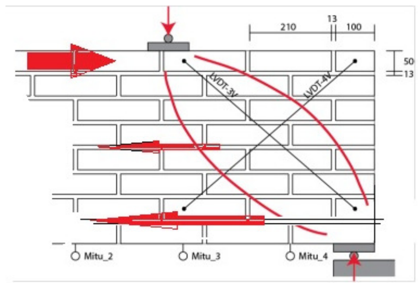


Figure 11 Equilibrium of forces on rigid body, showing the effect of tension in the reinforcement at mid height on the compression strut.

4.2 *Analyses*

4.2.1 *Analysis series R1*

In the load deflection behavior of the specimens of series 1, Figure 6, three phases can be distinguished as indicated in Figure 13.

First a linear elastic phase is recognized. The specimen is free of cracks. After cracking starts, stiffness decreases, and reinforcement becomes active.

The load remains more or less constant, phase 2, while deflection increases.

After some deflection, the loads starts to increase again, phase 3, the last phase. The cracks due to flexure will grow larger and new cracks due to shear will develop. This continues until the specimen ruptures.

The third phase differs for specimens with (R1-2) and without reinforcement (R1-1) in the shear zones. The R1-1 specimens behave in a stiffer way, but fail earlier than the R1-2 specimens which behave more flexible but fail at higher loads.

The R1-2 specimens behave more flexible than the R1-1 specimens, probably due to the manner of cracking. The cracks in the unreinforced specimens R1-1 are more widespread, while the cracks in the reinforced specimens are more concentrated in the bottom tie, due to the reinforcement in the shear zones. More cracks in the bottom reduce the stiffness of the tie and consequently, more deflection will occur.

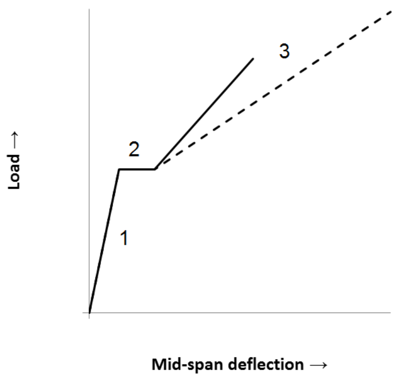


Figure 12 scheme of load deflection diagram, compare this Figure with Figure 6 and Figure 7.

4.2.2 *Analysis series R2*

In the load deflection diagrams of test series R2, Figure 7, only two phases can be distinguished, similar as the phases 1 and 3 indicated in Figure 13. However, phase two did not occur. The two phases are described below.

Phase 1 is the linear elastic phase, comparable with the linear elastic phase of the specimens in series R1. But, because the specimens were 'pre-cracked' by applying a continuous perpend joint in the middle, there is no horizontal cracking plateau, (phase 2 in series R1). Instead, the slope in the load – mid-span deflection relation changes when actual cracking occurs. The specimen becomes less stiff.

Due to the "constructed" crack, phase 2 is lacking.

Next, phase 3, similar behavior, like in series R1 occurs. More cracks develop and the existing ones grow larger until rupture occurs. In this series no clear distinction between the specimens without (R2-1) and with (R2-2) reinforcement in the shear loaded areas can be made. On average, the unreinforced specimens performed better.

Besides that, another phenomenon which occurred during testing was noticed. Specimens R2-1B and R2-2A showed horizontal cracks at their side edges causing the whole upper left part of specimen R2-2A to come loose.

These horizontal cracks occurred in layers with bed joint reinforcement. Probably the bond in this joint between the mortar and the bricks was not sufficient to transfer the shear forces.

4.3 *Shear load bearing capacity versus height.*

By plotting the shear load at the onset of cracking (Vcr) versus height (h) the following best fit relationship was established:

$$Vcr = 26.793h^2 + 23.187h - 0.49 \qquad (1)$$

With:

h = the height in meters and
Vcr = the shear load in kN.
R^2 = 0.9233

This best fit second degree relationship indicates that bending effects may play a role, indicated by the significant quadratic component. The ratio between bending capacity and height is quadratic.

In EC6 a linear relationship between height and Vcr is adopted. The higher loads in specimens with the stiffer tie (types 3,4 and 5) indicate also that the shear could not develop completely while bending was more dominant in the specimens with the slacker tie.

5 SUB SEQUENT RESEARCH

5.1 *Deep beam*

On the basis of the comparison of test results in this paper, it seems that it is a wrong idea to use more bed joint reinforcement in deep masonry beams.

This is not fully understood while deep concrete beams are reinforced over the full height. However, deep beam reinforcement is applied in concrete in both horizontal and vertical direction. Sub sequent research may give more insight.

5.2 *Larger support length*

The support length of the tested specimens was relatively short. Usually, more masonry is present besides an opening that can form support. Consequently, the tie can be anchored better, i.e. over a larger length.

In sub sequent research the possible positive effect of a larger anchorage length can be studied, in combination with variation in (extra) shear reinforcement.

5.3 *More reinforcement for bending*

The use of more reinforcement in the bottom part of the wall -i.e. a stiffer and stronger tie- possibly increases load bearing capacity. This idea is confirmed by the significantly higher results of tests with a stiffer/stronger tie (types 3,4 and 5).

Due to the applied relatively low amount of reinforcement, bending effects were more critical than shear effects in the other tests. More reinforcement and/or a better anchoring assure that the strut en tie model better can develop.

5.4 *Vertical shear reinforcement*

The effect of vertical shear reinforcement on bearing capacity has not yet been studied. However, first the practical consequences of applying, in an efficient way, vertical reinforcement in the shape of stirrups, like in concrete must be studied.

6 CONCLUSION

The test results (ultimate load and cracking load) of specimen series R1 correspond with the theoretical values and previous research. Also it seems bed joint reinforcement hardly increases the shear capacity, although there is a large spread.

The test results found for specimens of series R2 do not meet the expected outcome. The unreinforced specimens failed at a higher load level. The unexpected results within series R2 were probably caused by peak stresses at the points where the loads were introduced.

In the case of the specimens of series R2, two parameters were changed of which one unintentionally. The specimens of series R2 should have been eight layers high instead of ten.

It is recommended to perform more tests on this subject to increase the reliability of the test results, preferably supported by numerical analysis. To build a numerical model several subjects need to be investigated, like mortar-bed joint reinforcement and mortar-brick bond.

The results of the research described in this paper in combination with earlier work, Vermeltfoort & Martens (2015) show that:

- tie anchorage and tie stiffness are important. More reinforcement and/or a better anchoring assures that the strut en tie model better can develop.

- using a stiffer tie shear becomes more critical, this effect is not addressed by the usual design formulas.

- vertical shear reinforcement allows a strut and tie truss model to develop.

7 ACKNOWLEDGEMENTS

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