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EFFECTS OF SUPPORT CONDITIONS ON LINTEL-MASONRY INTERACTION

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ABSTRACT

This paper describes research into the behaviour of so-called “composite lintels” i.e. load bearing masonry in combination with prefabricated concrete lintels. Eighteen identical walls were loaded in plane to rupture. Nine layers of stretcher bond masonry, 562.5 mm in height, were built on prefab concrete lintels (60×100 mm^2) with a span of 2800 mm. The effects of two types of supports and two types of loading on the mechanical behaviour of in plane loaded composite lintels were studied. Roller supports were simulated by suspending steel blocks from the roof beam of the test frame. A support condition, often used in practice, was simulated by a layer of felt on a brick. Two series of six walls were symmetrically loaded at four points. A third series of six walls were asymmetrically loaded at one point. The mean failure shear load for the four point loading condition was V_{fail} = 31 kN. For the one point condition it was V_{fail} = 24.4 kN. On average, the ultimate load (F_{ult}) was 15% higher than the failure load (F_{fail}). Supported on rollers, three walls failed in the constant moment area (mid span). The fifteen other walls failed in the maximum shear load area near the supports. The height of the compression zone at mid span depended on the support condition and was largest for the felt support condition, where horizontal movement of the lintel was restrained. The support condition (rollers or felt) had a negligible effect on the load bearing capacity.

KEYWORDS: Composite lintel, support, shear, strength variation, load bearing masonry.

INTRODUCTION

Openings in walls are necessary to give entrance to buildings for people, furniture and daylight and for ventilation. As masonry has little tensile strength, a concrete beam called a lintel can be used to span openings. During construction, the lintel (and the masonry above it) must be temporarily supported. After removal of the supports, a self-supporting structure called “composite lintel” is obtained. This structure consists of two structural elements: a prefabricated concrete lintel and a number of layers of structural masonry. Both elements carry the load in combined action, as shown in Figure 1. It is assumed that the behaviour of composite lintels depends mainly on the condition of the supports, the manner of loading and the (shear) strength of the masonry, as described by Stafford Smith [1], Wood [2] and [3].

In the early 1950's, Wood [2] studied the stresses near the supports of concrete beams supporting masonry and the distribution of bending moments along the length of the beam (lintel) in combination with masonry deformation. In the lintel near the support, negative bending occurred, as compared to the positive bending at mid span. Until then (1952), triangular loading on a beam, supporting masonry, was assumed but Wood's study [3] resulted in new guidelines.

Figure 1a shows schematically the compressed arch which develops inside the masonry. The arch transfers the load to the supports via the lintel which is in tension. The height of the masonry has some effect on the shape of the arch, Wood [2] and [3]. Forces are transferred via shear and mechanical interlock between the lintel and the masonry. Besides that, the lintel supports the masonry below the arch which causes bending. Figure 1b shows schematically the stresses near the support. Based on the assumed stresses, the shear stress distribution along the length of the lintel is calculated and, in Figure 2, compared to the stresses found with Comsol simulations by Vermeltfoort and Van Schijndel [4] assuming full composite behaviour.

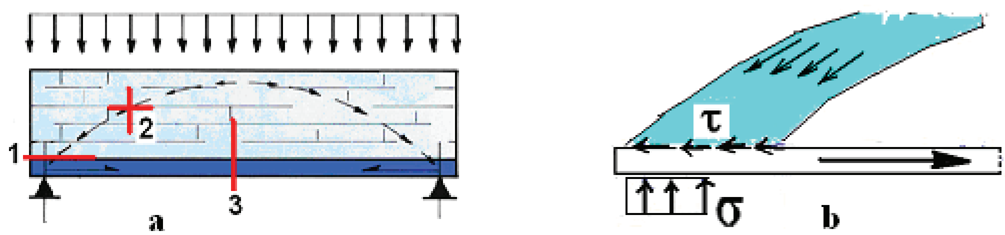

Figure 1: a) arching in a composite lintel and b) stresses near support.

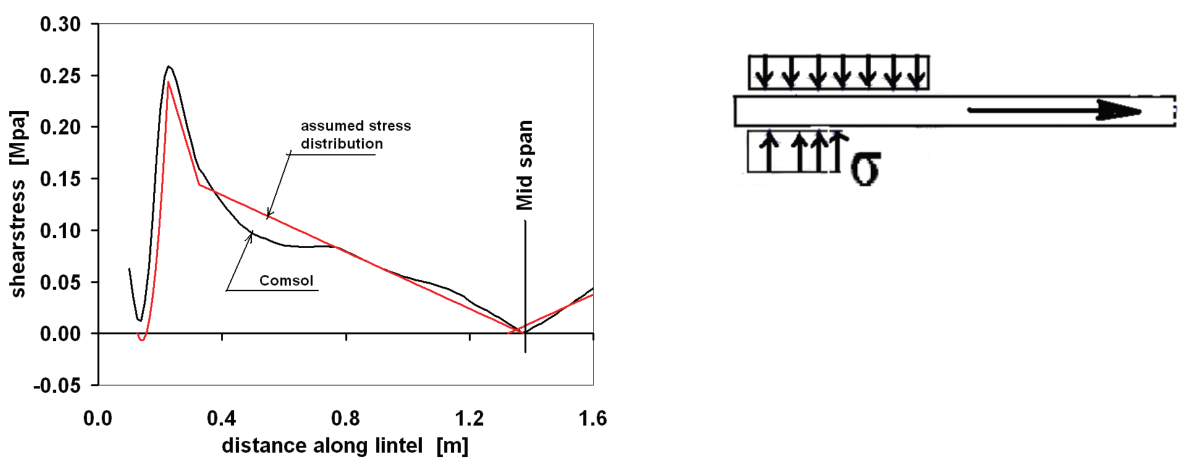

Figure 2: Shear stress distribution along the lintel-masonry interface; calculated on the basis of the stress distribution shown in Figure 1 and as the result of simulations [4].

Depending on the lay-out of the façade, movement joints are made in the brickwork [5]. The test walls may be seen as pieces of wall between two movement joints without the rest of the masonry. The supports of "combined lintels" prevent the elongation of the lintel when the wall is loaded and, consequently, the load distribution and behaviour of the test wall is affected.
According to [6] wall-lintel combinations must be loaded in two manners: 1) by four point loads for flexural resistance (S4P) and 2) by one point load at ¼ - ¾ of span for shear resistance (A1P).

With four forces at equal distances a uniform load is reasonably simulated. Figure 3 shows schematically the lines of thrust for the symmetric (S4P) and the asymmetric one point (A1P) loading situation and the loading scheme of the test wall.

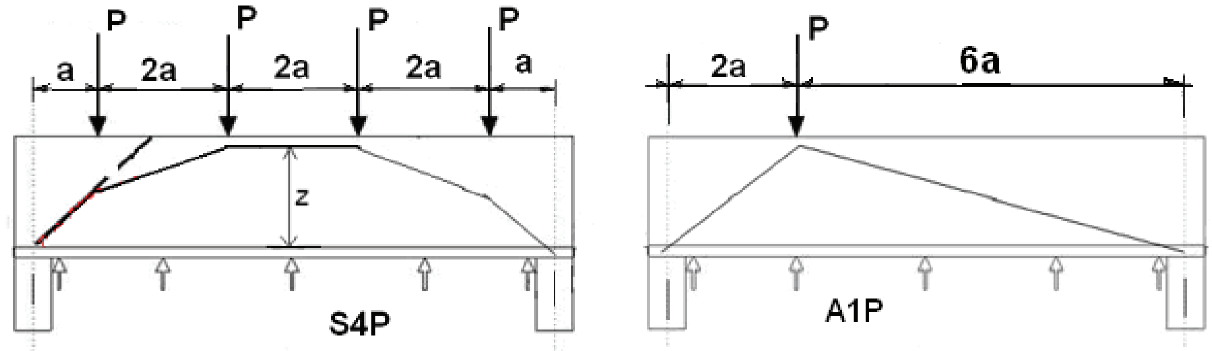

Figure 3: Lines of thrust (schematic) for the S4P and the A1P loading situation.

The force distribution in the test for composite lintels seems simple. The reaction forces can be derived from the test-measurements. When symmetrically loaded with two jacks, the reactions are equal to the applied jack loads. When loaded by one point load, the two reactions are approximately ¼ and ¾ of the jack load, respectively. Self-weight (approximately 2750 N) is neglected. The system is assumed to behave as a statically determinate system.

Establishing the exact values for bending moments is more difficult. The position of the support reaction – which is a resultant of stresses – is not exactly known while the stress distribution is irregular. When linear elastic material behaviour and no tensile strength are assumed, the position could be derived from displacements of the lintel-support interface. The calculated values for the bending moments differ by approximately 5% from the values for an estimated deviation of a few centimetres on a span of 2.80 m.

For a span (2800 mm) to height (630 mm) ratio of approximately four, bending could be critical at mid span. For an estimated internal lever arm of 550 mm and a tensile strength of 1770 MPa of the prestressing steel reinforcement with an area of 25 mm^2, the internal moment of resistance at mid span is approximately M = 25 kNm, corresponding with a maximum shear force V = 35 kN. If it is assumed that the compressive stress block is triangular, the resulting masonry compressive stress of 3.7 MPa is about half of masonry compressive strength.

Shear capacity must be controlled in a vertical section near the support [7]. When differences in Young's moduli of concrete and masonry are taken into account, and assuming uncracked linear behaviour a shear stress distribution as shown in Figure 4 is obtained, with a maximum value at the neutral axis. However, the shear capacity in the (horizontal) joint between lintel and masonry seems most critical, location 1 in Figure 1. In that area, stress distribution is not uniform as found in [4]. Further, after cracking, arching becomes prominent. Other possible failure modes are shear in the arch at other locations and failure of the arch due to splitting, location 2 in Figure 1. At location 3, bending stresses must be controlled.

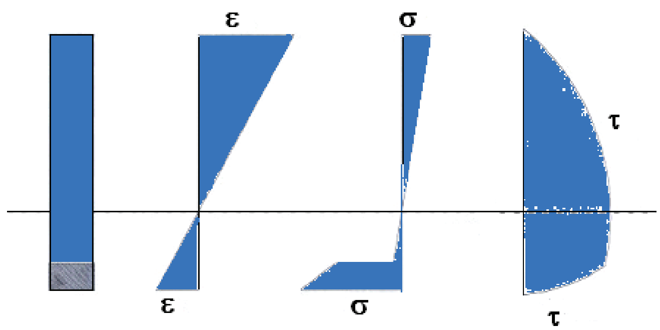

Figure 4: Strain and bending and shear stress distribution with stiffer lintel.

To estimate the capacity, assume that the contact area is 250 mm in length at the foot of the arch (Figure 1b) and the indicated stress distribution occurs. Then, a reaction force for the tested walls of 15 kN and a tensile force in the lintel of 19 kN is estimated using a shear strength, based on experiments [9] of:

$$\tau = 0.35 + 0.65\sigma \qquad (1)$$

However, the masonry will have some tensile capacity and, to obtain an arch which is completely ‘free’, the masonry has to crack. The load-bearing capacity of the uncracked masonry may be higher than the load-bearing capacity of the arch developed after cracking. Therefore, half of the walls that were S4P-loaded were pre-cracked by vertical joints, as shown in Figure 5.

The effect of the type of loading is assumed to be negligible because, when designing a composite lintel, the largest shear force in the system is checked against shear strength. No distinction is made for the type of loading that causes this shear force. The same result is expected for shear due to an equally distributed load or one point load. However, when loaded at one point, the shear distribution in the wall is different at both ends and different from the distribution for an equally distributed load (Figure 3). Consequently, one side is strongest. Loaded symmetrically, material properties determine which side is strongest.

Another reason for differences in load resistance of test walls is the difference in the length of the area with a constant bending moment. In the S4P loading situation this length is approximately one quarter span. In the A1P situation, theoretically, the moment has a maximum in only one section, Figure 3.

TEST PROGRAM

To study the effects of the support condition on load bearing behaviour the following two types of support were applied as indicated in the left column of Table 1.:
- Roller supports, with free horizontal movement and free translation (6 walls); and
- Felt supports, with more or less confined horizontal movement and free translation.

For the felt supported walls, as mentioned earlier, two types of loading were used:
- Loading at four points at equal distances (6 walls) and
- Loading at one point (6 walls).

Half of the four point loaded walls (indicated V1 through V9) were "pre cracked" by vertical header joints at one third span, Figure 5b. An additional series (3 walls) were supported on rollers but the lintel was stiffened by four external bars.

The test walls were built in the laboratory by an experienced mason in series of three in a 'practical' manner. The mortar joints were jointed [8]. The goal was to obtain masonry with uniform properties. Soft mud bricks, brand Rijswaard (mean compressive strength ±20 N/mm^2) were used in combination with a factory made mortar. More details on mortar and masonry shear testing are given in [9]. To get the quality as constant as possible, a factory made mortar was used with a mean compressive strength of f'_{mor} = 9.5 N/mm^2. This brick-mortar combination results in masonry with a compressive strength of f'_{rep} = 7.1 N/mm^2, according [7].

Prefabricated lintels (60x100 mm^2) were used with a concrete compressive strength of 55 N/mm^2. The lintels were prestressed with two, FeP 1770, 4 mm diameter bars (25,2 mm^2).

After building, the walls were wrapped in plastic to prevent extreme drying and stored under laboratory conditions. Testing took place at least six weeks after building. Support conditions were assigned to walls by drawing lots.

Figure 5: a) Building the test walls, in the TU/E laboratory. b) Test set up roller support with, in detail, open header joints above the lintel

The test frame was built using HE300B beams (300x300 mm^2). Two jacks and two load distribution beams were used to apply the load, Figures 3a and 3b. The force in each jack was measured with a load cell. The displacement of the bottom of the lintel was measured with five LVDTs mounted on a separate frame.
A roller support was simulated by suspending a steel block from the roof beam of the test frame using two threaded rods, 1500 mm in length. In this way, the steel support block rotated easily and moved freely in the horizontal direction. Another support condition, often used in practice, was simulated by a layer of felt on a brick. This brick was positioned in a two mm thick layer of gypsum on the same steel block as used for the roller support. However, this block was bolted to the bottom beam of the test frame. Each wall had either two roller or two supports on felt. The external stiffening bars used in the additional tests (A1, A2 and A3) are shown in Figure 6b. Figure 6a en Figure 6c show the other two types of support used in detail.

RESULTS

The main test results are given in Table 1.

Table 1: Test program and main test results: age, support and load type, mortar strength, failure and ultimate load, stiffness and ratio between estimated and measured failure load.

Test	Wall	Age	Supp.-	Load-	$f'_{mor.}$	F_{fail}	V_{avg}	F_{ult}	k	est.
#	#	days	type	type	MPa	kN		kN	kN/mm	-
1.1	5	41	Rol	S4P	7.44	61		68	60	1.03
1.2	11	34	Rol	S4P	6.51	69	32.5	75	66	0.98
1.3	4	42	Rol	S4P	7.73	65		71	63	0.96
1.4	V1	38	Rol	S4P	10.50	57		64	69	1.03
1.5	V3	36	Rol	S4P	12.20	58	28	64	66	0.98
1.6	V4	37	Rol	S4P	12.50	54		61	66	1.03
2.1	8	47	Felt	S4P	5.23	62		62	75	1.02
2.2	10	49	Felt	S4P	4.46	60	31	73	69	1.07
2.3	3	57	Felt	S4P	7.25	64		64	64	0.88
2.4	V6	66	Felt	S4P	9.20	38		42	43	1.30
2.5	V9	54	Felt	S4P	8.25	60	31	60	44	0.93
2.6	V7	65	Felt	S4P	5.47	61		83	44	0.92
3.1	12	70	Felt	A1P	4.93	42		42	57	0.95
3.2	7	76	Felt	A1P	4.66	36		36	69	1.06
3.3	2	85	Felt	A1P	7.99	30	24.4	40	60	0.96
3.4	6	88	Felt	A1P	7.21	28		28	61	1.04
3.5	9	83	Felt	A1P	5.31	28		29	65	1.22
3.6	1	92	Felt	A1P	8.71	31		31	44	0.81
A1	V2	55	Rol	S4P	9.47	41	21	49	56	
A2	V5	45	Rol	S4P	9.34	42		50	64	
A3	V8	42	Rol	S4P	5.81	80		89	60	

Wall#: serial number of walls, V1 through V9 were precracked as indicated in Figure 5,
f'mor: mean compressive strength of mortar prisms made simultaneously with test walls,
F_{fail}: applied load at the start of cracking,
V_{avg}: shear failure load, averaged value of three tests,
F_{ult}: ultimate recorded load,
k: stiffness (loading per mm deflection),
est: ratio between the estimated F_{fail} and the measured F_{fail}.

Figure 6: Supports in detail: a) roller, b) stiffened lintel and c) brick with felt.

Figure 7 shows the load-deflection diagrams for the three series of six test walls. These deflections of the bottom of the lintel are the measured displacements at mid span corrected for the displacements near the supports. After this correction the behaviour is almost linear until the first cracks appear. Cracking is accompanied by noises and, instinctively, the noise level was an indication for the crack width. In some cases, in the first part of the test, minor cracks were observed in the lintel and masonry but these cracks hardly affected the load-deflection behaviour.

After cracking, the behaviour is less regular and the stiffness becomes smaller than in the earlier phase. However, in this phase, the load first increases but may decrease later on. Finally, the deformation increases rapidly as the test wall (suddenly) fails and large cracks develop. From the walls loaded at four points, one roller supported and four felt-supported test walls collapsed suddenly. A sudden collapse also happened four times in the series loaded with A1P, Figure 7. The load-deflection (F-Δ) diagram of each separate test was used to establish the failure and ultimate load. To establish the stiffness factor (k), i.e. the ratio between load and deflection, a linear best fit for the first part in the diagram was used. Table 1 shows the values of k. The mean value is k = 60 kN/mm. Extreme values were found for walls 2.4, 2.5, 2.6 and 3.6 (k = 44 kN/mm) and wall 2.1 (k = 75 kN/mm).

Two types of failure patterns were observed. In the first pattern, a relatively large (1/3) part of the wall slides over the lintel. Shear stresses concentrate near one of the supports. In some cases small cracks were observed at mid span in the lintel. However, their width hardly increased after they appeared. The second pattern shows vertical cracks at mid span in the constant bending moment area. These cracks widen till the wall fails and two pieces of almost equal length remain. Figure 8 shows the typical crack patterns for 'shear' and 'bending' failure.

In some cases, the part of the lintel that rested on the support was not exactly perpendicular to the front wall surface. Consequently, these walls were more difficult to position properly in the test set up. This may have caused a negative effect on the load bearing capacity and also the deformation at the front and at the back of the wall will be different. In subsequent experiments, measurements should be taken at the front and at the back to study this effect.

In the additional series of three tests (A1,A2 and A3) the lintel was stiffened by four threaded rods. The failure load of one of these tests was almost double the mean value of the other two. The stiffness followed the same trend as the other walls, Figure 7. It is expected that this variation in resistance may have been caused by eccentricities due to the forces in the four stiffening bars.

The height of the compression zone at mid span was established with horizontal deformation measurements. Assuming a linear stress-strain relationship, and using Bernoulli's hypothesis, the trend line through the measured strains was used to establish the position of the neutral axis. The mean results are listed in Table 2.

Masonry compressive stresses for a moment at mid span of 25 kNm were estimated using a 390 mm high triangular stress block and a structural height of 600 mm. A maximum stress of 1.67 MPa which is approximately one quarter of masonry strength (7.1 MPa) was found. This means that, because at mid span masonry stresses are not critical, shear dominates the overall behaviour

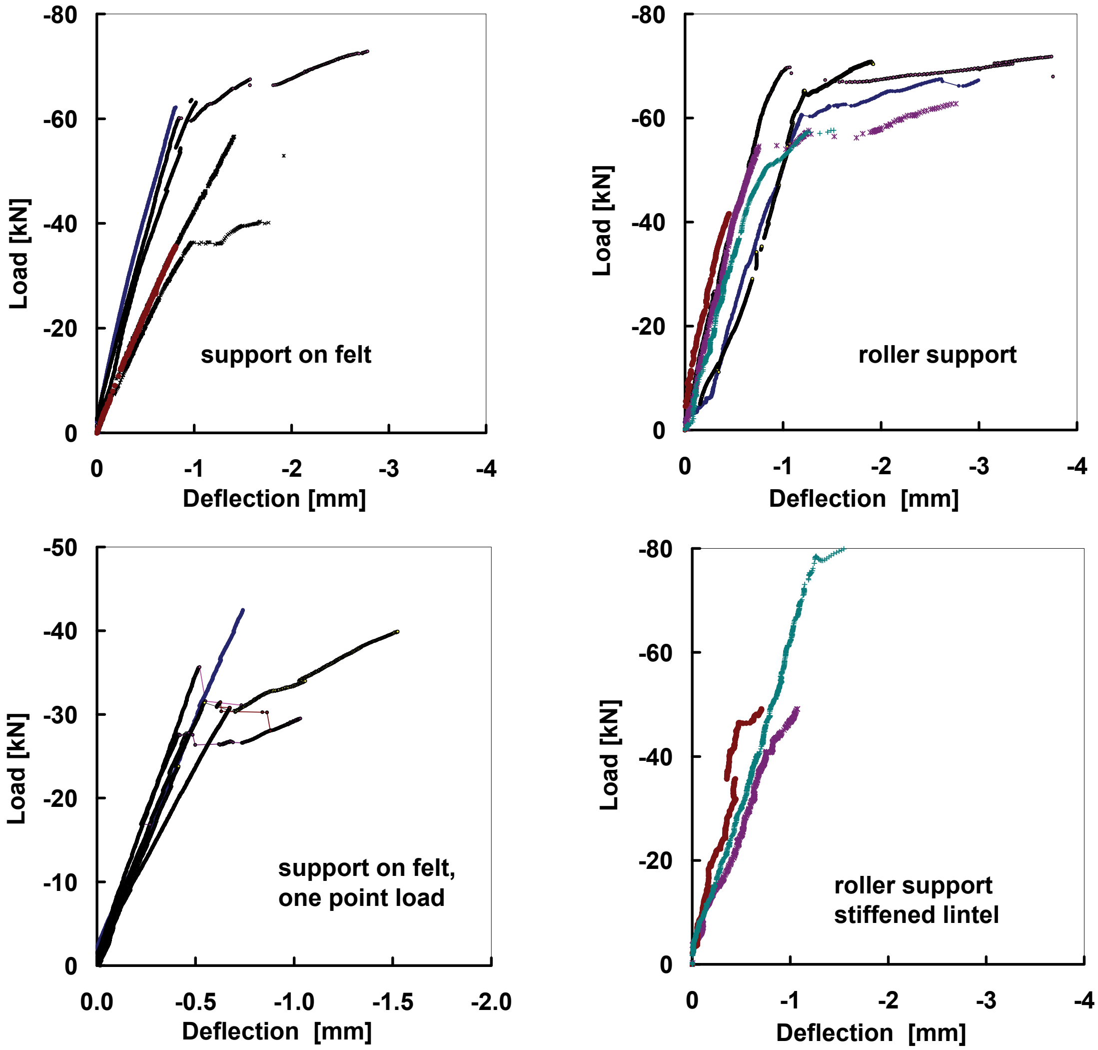

Figure 7: Measured net deflection versus total load for three series of six tests and one additional series of three with stiffened lintel.

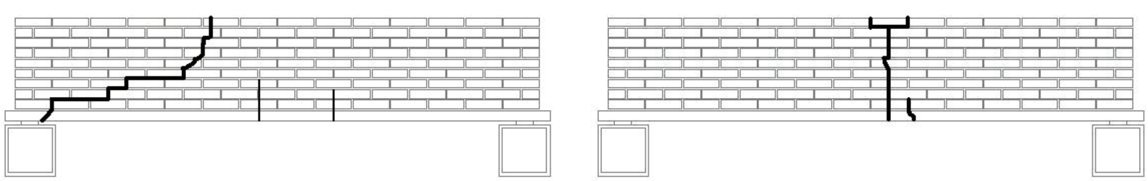

Figure 8: a) Crack pattern 1 "shear" b) Crack pattern 2 "bending".

Table 2: Mean height of the compression zone (in mm)

Roller supported		S4P Supported on felt		A1P Supported on felt	
test number	mm	test number	mm	test number	mm
1.1, 1.2, 1.3	---	2.1, 2.2, 2.3	446	3.1, 3.2, 3.3	260
1.4, 1.5 1.6	390	2.4, 2.5, 2.6	440	3.4, 3.5, 3.6	
A1, A2, A3	490				

DISCUSSION OF PARAMETER EFFECTS

Test parameters affected the failure load of composite lintels. Test parameters like: mortar compressive strength, age of the test wall, type of support and type of load introduction (S4P of A1P) were taken into account . Using multiple regression analysis, the following equation was found:

$$F_{est} = -1.74 \cdot f'_{mor} - 0.37 \cdot age + 5.22 \cdot LT - 1.28 \cdot ST + 71.65 \qquad (1)$$

with parameters as explained in Table 3. This relatively simple model without cross correlation effects taken into account gives a general impression of the contribution of several parameters to the estimated value of the failure load. Equation (2) is only valid for the parameter values used in this project. The word 'contribution' in Table 3 refers to the product of a coefficient from equation (2) and the difference between the maximum and minimum value of a parameter. For example the 'mortar' contributes -1.74*(12.5-4.5) = 14.0 kN.

Table 3: Parameters with values and contribution to the estimated value of the failure load.

		minimum	maximum		contribution	
F_{est}	estimated failure load				71.65	kN
f_{mor}	mortar compressive strength	4.5	12.5	MPa	-14.0	kN
age	age of the test wall	34	95	days	-21.5	kN
LT	load-type	A1P = 1	S4P = 4	--	+15.7	kN
ST	type of support	Rol = 1	Felt = 2	--	-1.3	kN

Even though the contribution of the parameters may suggest differently, the mean shear failure loads for the S4P loaded walls are almost independent from the support conditions. On the other hand, when loaded with one load (A1P) the shear failure load was 20% smaller when loaded with four loads (S4P). Mortar shear strength is not taken into account while only two or three shear specimens per wall were tested, which is too little to establish a reliable value for shear strength.

The felt-support condition simulated a practical situation. An attempt was made to simulate pinned support conditions using four external tensile bars to couple the supports horizontally, Figure 6b. However, these bars introduced unintended eccentricities in the lintel. Elongations in the length of the lintel were measured. Only the ones at mid span were used, together with other measurements, to establish the size of the compressed area. Displacements of the lintel-head in the direction of the length of the lintel were affected by movements perpendicular to the measuring direction (the lintel rotated at the support, its end moved upwards) and therefore not used. The observed crack patterns confirm the confining effects in the felt support condition. Two of the roller supported walls failed in bending with broken lintel reinforcement . All other walls failed in shear.

The pre-cracked walls (1.4, 1.5, 1.6 and 2.4) failed earlier than the others of the same series indicating the effect of the tensile strength of the masonry. Mortar compressive strength and age both have a negative effect according to equation (2).

The standard deviation of the ratio between the estimated and the measured value for failure load was approximately 6 kN (10%), Table 1. In the series loaded with one point load (series 3; shear resistance test) the largest differences were found; the lowest value was 0.80, the highest value1.21 indicating a larger variation in results than for the S4P type of loading.

CONCLUSIONS

Loaded on four points, the load bearing capacity of walls is independent from the support condition. Supporting the composite lintel on rollers or on felt does not result in significantly different shear resistances. The walls failed either in a bending failure mode or in a shear failure mode indicating the support condition effect.

In the roller support condition three walls failed due to rupture of the lintel reinforcement. This indicates that the ‘felt’ support condition had a positive effect on the tensile capacity of the lintel. For all other walls, shear was the dominant failure mode. The mean failure shear load for the four point loading condition was V_{fail} = 31 kN. For the one point loading condition it was V_{fail} = 24.4 kN.

The regression model showed an unexpected negative effect of age on the resistance.

Pre-cracking by leaving some head joints open had a negative effect on flexural resistance. Leaving these head joints open was effective because a relative strong type of mortar was used.

A possible cause for variation in results may be the position of the supporting surface of the lintel which, in some instances, was not perpendicular to the surface of the wall. This causes an irregular stress situation near the support.

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