REPRINT 172

Lateral loads on masonry walls

By Å. Hallquist
Norwegian Building Research Institute

NORWEGIAN BUILDING RESEARCH INSTITUTE

OSLO 1970

1. INTRODUCTION

Due to insufficient knowledge of the structural mechanics of laterally loaded masonry walls, the lateral load capacity cannot be determined analytically. Existing design methods are based on empirical data and highly approximate calculation methods and are not considered to be a rational approach to the design problem. A rational design must be based on a method that is representative for the performance of the structure in use.

In most cases the designed walls have had satisfactory bearing capacity, however, the factor of safety might have been unduly high thus resulting in uneconomical design. Where failure has occurred, in adequately supported wall panels, it has been by bond failure at the brick mortar interface although in panels with good bond strength, tension failure in the bricks and mortar has taken place. Few failures of either type due to wind load have been reported in Norway. In Great Britain high winds have caused severe damages to external infill brickwork panels [1]. In Sweden cracking of masonry basement walls caused by earth pressure has been reported to be a problem [2]. By experience one knows that this is also a problem in Norway, but the severity of the problem has not been documented by a field investigation of buildings in use.

As a rule, however, national building codes with a few exceptions are restricting the use of masonry walls by not allowing tensile stresses to occur in such walls. If this rule was strictly enforced it would mean that unreinforced masonry walls could not be used as infill panels or in the top stories of buildings where the vertical loads are small.

There are several reasons why tensile stresses are not allowed, one being the lack of better design methods. To develop stress analysis design methods, test data for masonry walls must be available to verify the methods. To contribute such data, NBRI has carried out tests to study the effects of horizontal loading on brick cavity walls. The main objective of the research programme was to try to develop analytical methods based on these tests [3].

2. MATERIALS, TEST SPECIMENS, CURING CONDITIONS, AND TEST APPARATUS

2.1. Materials

2.1.1. Masonry units

Brick and concrete masonry walls were tested. Table 1 shows the average test results for solid bricks (used for six walls 1,20 m x 2,55 m) and for perforated bricks (used for two walls 4,50 m x 2,45 m) tested according to [4].

Table 1. Material test data for solid and perforated bricks.

Type of bricks	Dimensions, mm	Density, kp/dm^3	Compressive strength kp/cm^2	Initial rate of absorption g/dm^2/min
Solid	240 x 122 x 65	1,79	359	33
Perforated	222 x 105 x 62	2,12	521	9

Three types of mortars were tested, a cement-lime-, a masonry cement-, and a cement mortar. Table 2 gives the test results for the cement-lime mortar used for the wall panels. Type 1 mortar was used for solid and type 2 for perforated bricks. Testing methods were according to [5].

Table 2. Material test data for cement-lime mortars.

Type no	Proportions of Cement, Lime and Sand /by weight/	Modulus of rupture, kp/cm^2	Compressive strength, kp/cm^2	Bond strength, kp/cm^2
1	1 : ^{1} : 8	33	78	1,1
2	1 : ^{1} : 8	32	66	3,4

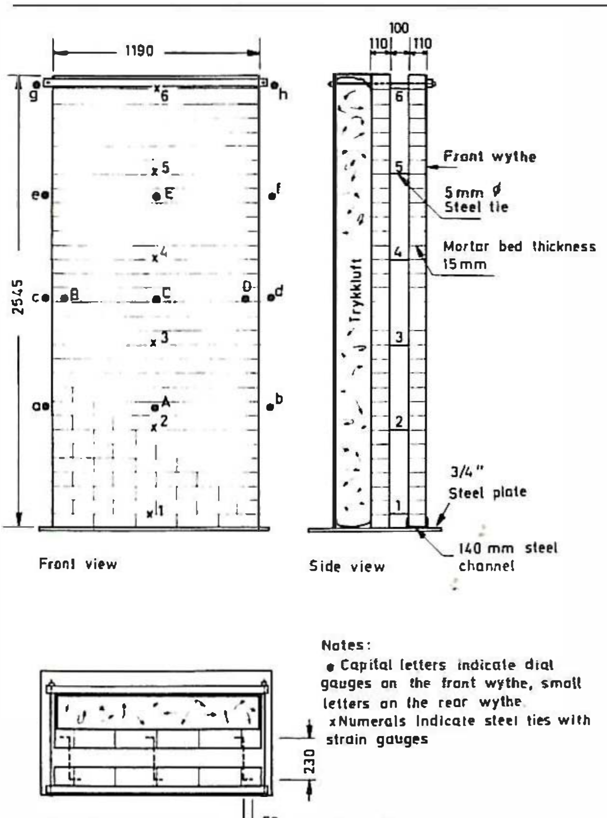


Fig. 1. Dimensions and test arrangement for 1,20 x 2,55 m cavity walls, test series 1.

2.2 Test specimens

2.2.1. Cavity walls 1.20 m

Six cavity walls were built using solid bricks and type 1 mortar. The dimensions are shown in *Fig. 1.* The front wythe in test series 1 (3 walls) was placed in mortar in a steel channel welded to a steel plate. Both wythes in test series 2 were placed in mortar on steel wideflange beams that were tied together by steel rods welded to the beams. The wythes were connected by 3 steel ties with diameter 5 mm placed in the mortar bed joint in every sixth run. On the steel ties along the vertical centerline of the wall, 2 strain gauges were glued on opposite sides.

2.2.2. Cavity walls 4,50 x 2,45 m

Two cavity walls were also built using perforated bricks and type 2 mortar. The dimensions are shown in *Fig. 2.* Both wythes were placed in mortar on a concrete slab anchored to the structure below. Ties similar to those described before was used to connect the two wythes. The ties were spaced at 0,50 m o.c. horizontally and vertically. At the edges of the walls, steel ties anchored to columns or welded to steel channels anchored to the columns were fitted to the mortar bed joints. Steel ties with strain gauges were located as shown in *Fig. 2.* Wall »A» was supported at the top by a steel beam anchored to the structure above, wall »B» was not supported at the top.

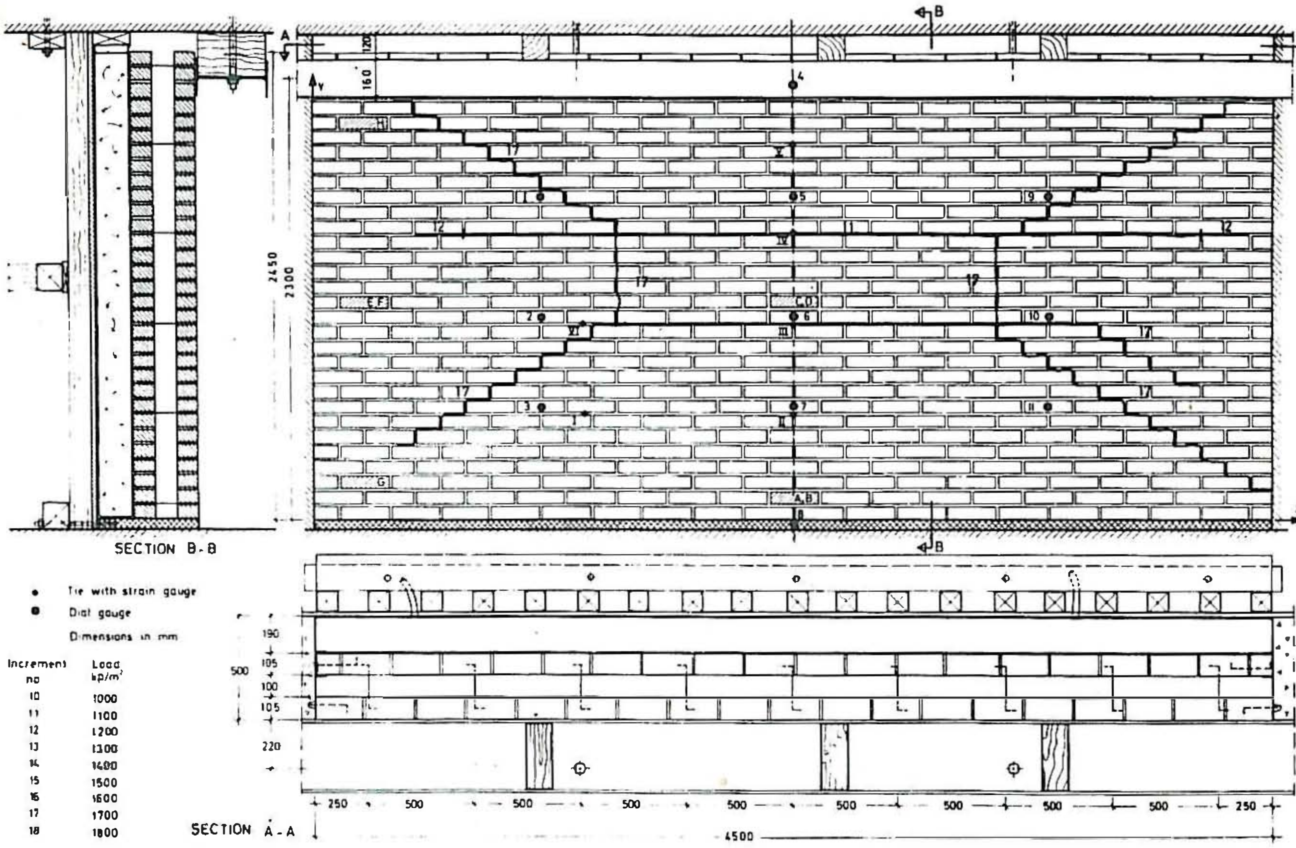


Fig. 2. Dimensions and test arrangement for wall A. The crack pattern is marked on the front wythe.

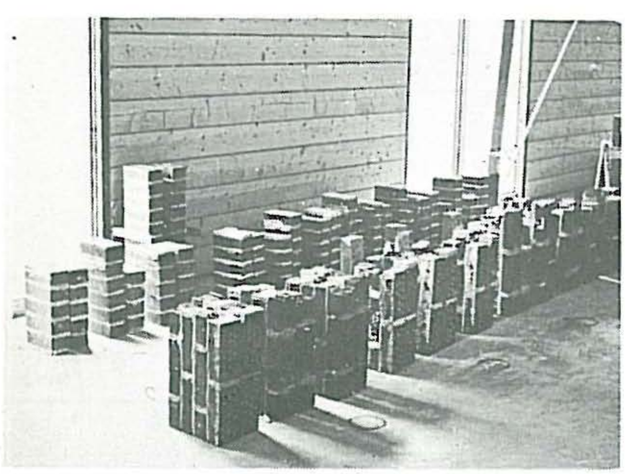


Fig. 3. Piers and small wall panels tested to failure.

2.2.3. Piers and small wall panels

Piers and wall panels were built using each of the sample masonry units. The specimens built with perforated bricks are shown in *Fig. 3*.

2.3. Curing conditions

The test specimens were cured for 28 days in the laboratory at a temperature of approximately 20°C. For the first 14 days they were covered by plastic sheeting. After the sheeting was removed the specimens were subjected to unconditioned air varying in relative humidity between 30 and 40%.

2.4. Test apparatus

The cavity wall specimens were loaded uniformly by inflating a plastic bag. The pressure in the bag was measured using a glass tube filled with water. Vertical load on 3 of the 1,20 m x 2,55 m spceimens (test series 2) was transferred by a hydraulic piston. Deflections were measured using dial gauges with a 1/100 mm scale. A static strain instrument with a scale reading 5 microstrains was used for the straingauge measurements.

In the flexural tests on piers and wall panels the load was also transferred by a hydraulis piston onto an electric loading ring and the applied load read on the straingauge instrument. Deflections were again measured using dial gauges with a 1/100 mm scale. The inclination of the specimens at one of the supports was measured with a Klinometer, Model no 544, with a 1 second scale.

3. TEST PROCEDURE AND RESULTS

3.1. Cavity walls 1,20 m x 2,55 m

The loading frame, containing a plastic bag was fastened to the steel plate or the steel beam at the bottom, and to the front wythe at the top. In test series 1 the bag was filled with compressed air in load increments of 20 kp/m^2 until failure. In test series 2 the front wythe was initially loaded with a 5 ton vertical load and then the uniform load was applied to the rear wythe in increments of 20 kp/m^2 until reaching 150 kp/m^2. The bag pressure was kept constant at that level and the vertical load increased to 10 tons and thereafter in increments of 10 tons until failure. The deflection of each wythe and the strain increment.

The failure in both series occurred at mid-heigth with the opening of a horizontal joint in each wythe. *Fig. 4* shows the measured deflection and the calculated force in the steel ties along the vertical centerline for wall series 1. Table 3 gives the failure loads.

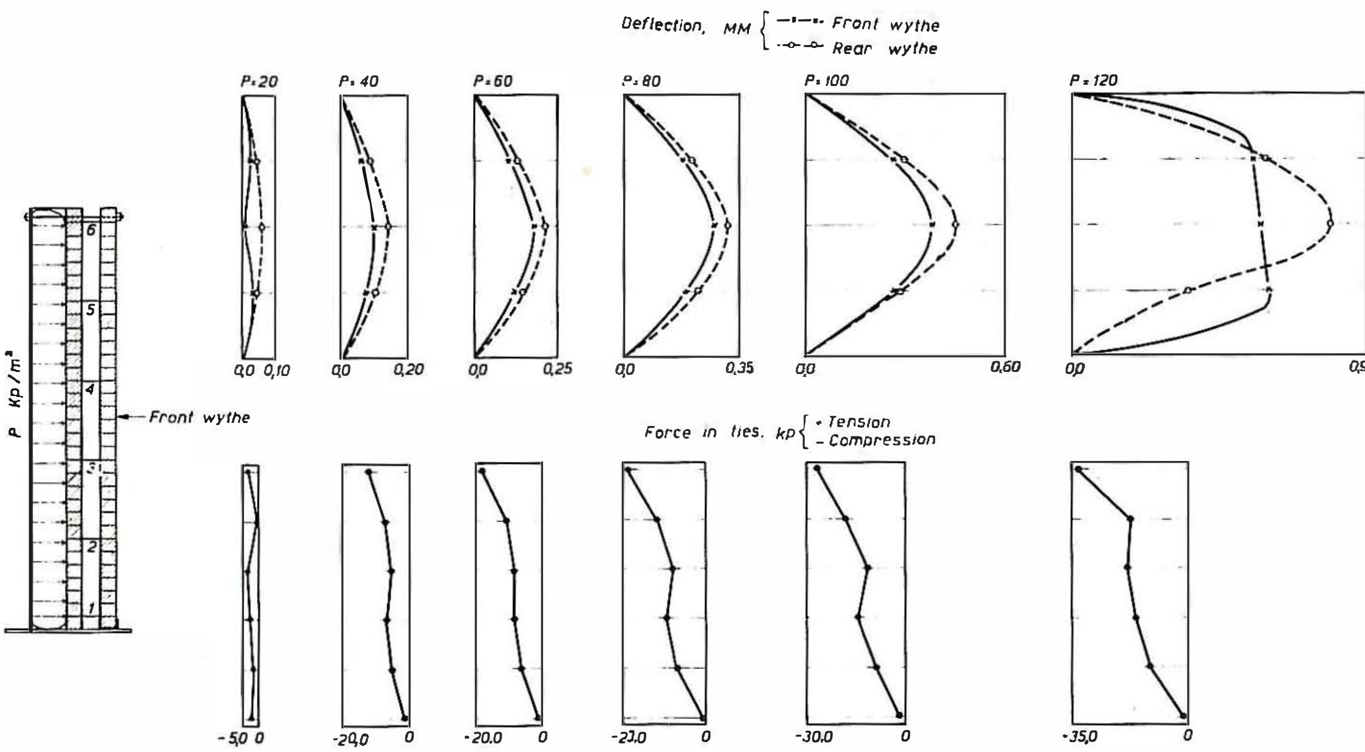


Fig. 4. Test series 1. The curves show measured deflections and the force in the steel ties connecting the wythes, each point on the curves represents the average of 3 measurements.

Table 3. Failure loads for cavity walls 1,20 m x 2,55 m.

	Test specimens					
	Series 1			Series 2		
	1	2	3	1	2	3
Uniform load on the rear wythe, km/m^2	138	260	134	150	40	150
Vertical load on the front wythe, kp	–	–	–	100	5	66

3.2. Cavity walls 4,50 m x 2,45 m

The loading frame, containing a plastic bag, was fastened to the concrete slab at the bottom and anchored to the structure above at the top. The load was applied in increments of 100 kp/m^2 until failure. The deflection of the front wythe and the strain in the steel ties were read at each load increment.

The crack pattern for wall A (supported at the top) is shown in *Fig. 2 and 5*. The crack pattern for wall B (unsupported at the top) is shown in *Fig. 6*. *Table 4 and 5* give the measured deflections of the front wythe.

To compare the loads carried by each wythe, the compressive force in the steel ties has been transformed into force per m^2 of wall area. See *Table 6*.

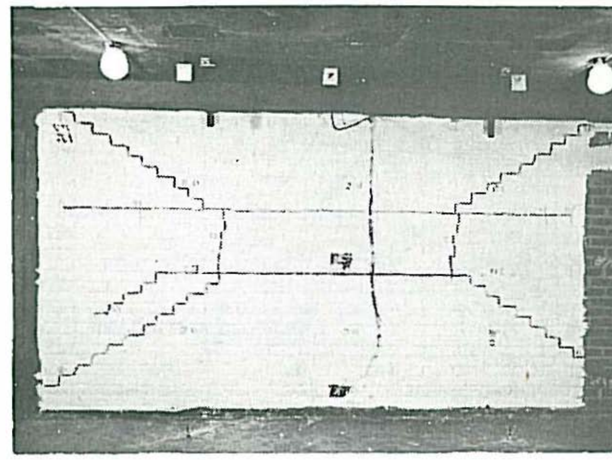


Fig. 5. The crack pattern in the front wythe of wall A at a uniform load of 1700 kp/m^2.

3.3. Piers and small wall panels

The piers and the small wall panels were turned on side and supported on rollers. A linear load was applied at midspan and increased in equal increments until failure. The deflection and the inclination at one of the supports were read at each load increment. The type of failures are shown in *Fig. 3*.

The average modulus of elasticity determined for nine piers each consisting of 10 bricks were 81,000 kp/cm^2 and for nine wall panels each consisting of 12 bricks were 153,000 kp/cm^2. The average modulus of rupture for the piers was 10,0 kp/cm^2 and for the wall panels 26,1 kp/cm^2.

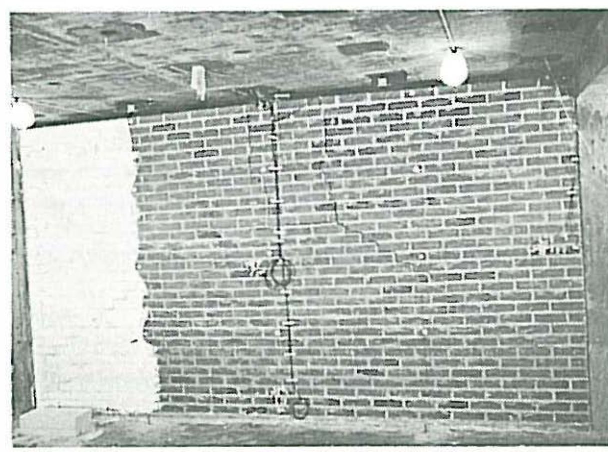


Fig. 6. The crack pattern in the front wythe of wall B at a uniform load of 1400 kp/m^2.

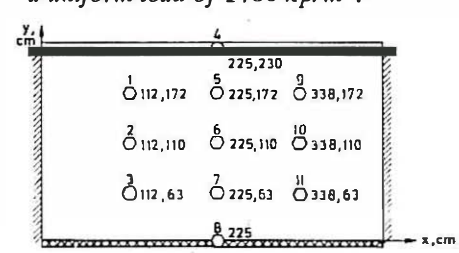


Table 4. Deflections of front wythe, wall A.

Load	Deflections, 1/100 mm										
kp/m^2	1	2	3	4	5	6	7	8	9	10	11
100	5	5	8	2	8	6	3	0	6	5	3
200	11	12	12	3	16	15	7	0	12	11	7
300	17	18	16	7	24	23	13	0	18	17	11
400	24	25	21	9	35	32	20	2	25	24	16
500	31	33	26	12	44	42	26	3	32	32	22
600	41	44	35	16	59	58	38	5	42	43	29
700	50	53	41	18	70	69	46	5	51	52	35
800	60	64	49	20	83	84	56	6	63	64	44
900	71	77	57	23	99	101	68	6	74	76	52
1000	83	91	66	27	113	119	79	7	87	90	62
1100	145	148	97	27	197	198	123	11	149	148	95
1200	163	166	108	29	219	219	136	11	165	166	105
1300	183	185	120	33	243	250	153	11	184	189	119
1400	203	207	133	36	267	280	170	10	205	215	134
1500	223	229	147	40	292	313	189	11	226	241	150
1600	245	254	161	43	320	348	211	11	254	272	169
1700	276	296	189	49	361	–	–	–	–	–	–

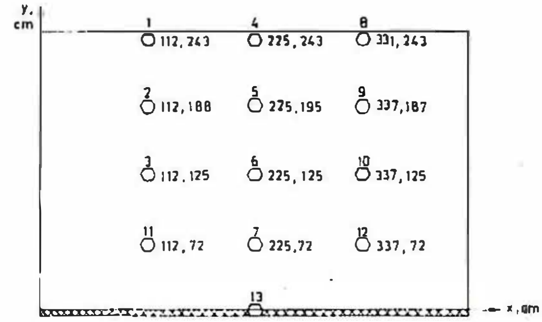


Table 5. Deflections of front wythe, wall B.

Load	Deflections, 1/100 mm												
kp/m²	1	2	3	4	5	6	7	8	9	10	11	12	13
100	28	25	18	36	33	16	15	24	22	17	10	10	0
200	69	60	45	92	84	65	27	61	55	44	27	27	0
300	122	105	80	161	145	113	65	112	96	76	47	47	0
400	178	155	117	288	212	165	94	158	140	111	70	68	0
500	236	204	154	322	284	218	114	216	189	149	91	91	0
600	269	255	192	403	355	272	154	271	236	185	113	112	0
700	355	304	219	484	425	326	184	326	283	221	135	134	2
800	411	354	268	569	499	384	217	382	332	261	160	158	3

Table 6. Comparison of loads carried by each wythe, wall A.

Uniform load,	Force, in kp/m² wall area, transferred bv tie no					
kp/m²	I	II	III	IV	V	VI
100	25	34	42	42	34	42
200	50	76	84	84	59	76
300	67	109	127	109	84	109
400	84	126	160	142	118	134
500	101	143	202	176	160	160
600	101	84	220	194	202	194
700	151	101	252	218	260	227
800	194	151	294	244	328	268
900	244	278	294	260	402	302
1 000	278	328	402	252	496	336
1 100	252	286	975	9	650	580

4. VERIFICATION OF A CALCULATION METHOD

4.1. Description of the method

The calculation method is based on a finite element procedure for displacement analysis of plate bending, employing rectangular elements. The fundamental idea of the finite element method is to represent the actual structure by a finite number of individual elements, interconnected at a finite number of nodal points. The stiffness of the idealized structure is obtained by adding the stiffness of individual elements.

The use of the finite element technique makes the method suitable also for analysis of structures with openings.

At NBRI, an extensive computer programme has been developed based on the particual type of rectangular elements described by Hansteen [6]. The calculations in the present paper have been made by Harald Hansteen and Gunnar Granheim, NBRI, using this programme.

4.2. Comparison of measured and calculated deflections

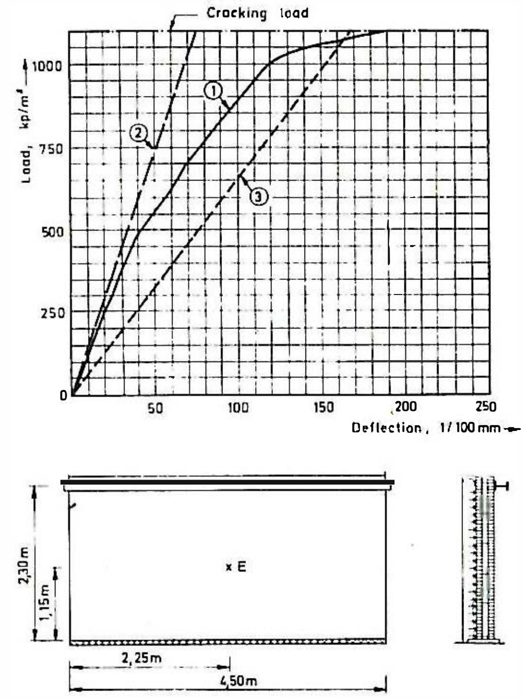
The calculations have been made using a modulus of elasticity E_y = 81,000 kg/cm^2 (vertically) and E_x = 153,000 kp/cm^2 (horizontally) in the plane of the wall. These values were determined in the bending tests on piers and small wall panels. Poisson' ratio was chosen as 0,2.

Walls A and B were partly fixed at three supports and respectively freely supported – or unsupported at the top. The measured deflection [1] ought to be somewhere between the calculated values for completely fixed walls [2] and for freely supported walls [3]. *Fig.* 7 and *8* show a graphical comparison of the values for the point with maximum deflection in both walls.

4.3. Comparison of cracking stresses and modulus of rupture

In wall A the first crack appeared at midheight in a mortar bed joint at a uniform load of 1100 kp/m^2. The calculated stress in the vertical direction was 6,8 kp/cm^2 (fixed) and 13,0 kp/cm^2 (freely supported). These values show good agreement with the average modulus of rupture of 10,0 kp/cm^2 for 9 piers.

A vertical crack appeared at the top of wall B at a uniform load of 900 kp/m^2. The calculated stress in the horizontal direction was 16,5 kp/cm^2 (fixed) and 34,9 kp/cm^2 (freely supported). Compared with the average modulus of rupture for 9 wall panels, 26,6 kp/cm^2, the calculated cracking stresses are considered to be in good agreement.

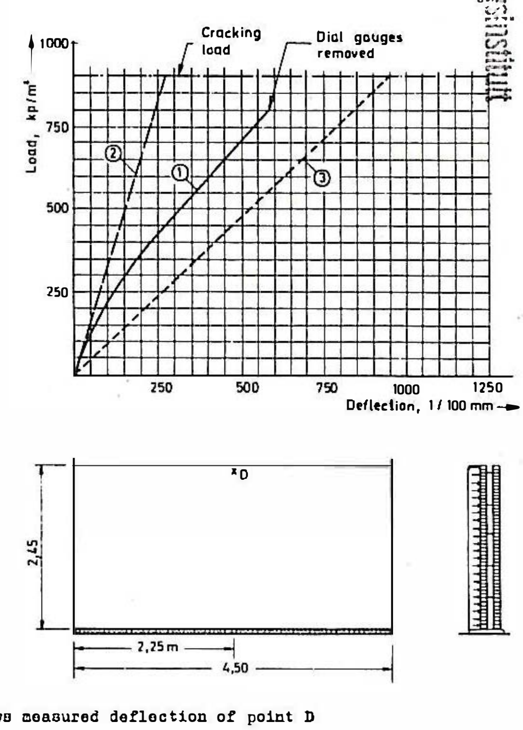


Curve 1 shows measured deflection of point E

Curve 2 shows calculated deflection of point E with the wall fixed at three edges and freely supported at the top

Curve 3 shows calculated deflection of point E with the wall freely supported on all sides

Fig. 7. Comparison between measured and calculated deflection by the finite element method.



Curve 1 shows measured deflection of point D

Curve 2 shows calculated deflection of point D with the wall fixed at three edges and free at the top

Curve 3 shows calculated deflection of point D with the wall freely supported at three edges and free at the top

Fig. 8. Comparison between measured and calculated deflection by the finite element method.

5. DISCUSSION

5.1. Materials

The solid bricks used for 1,20 m x 2,55 m cavity walls are not representative of the type of bricks recommended for exterior walls. The bricks had a powdery layer on the surface and gave low bond strength, see Table 2.

The strength properties for the perforated bricks and the cement-lime mortars are considered to be representative of materials used for exterior brick walls in Norway.

The material test data presented are for identification of the products being used. This identification is very important as factors affecting the tensile bond strength will not be discussed in detail in this paper. A survey of the subject is made in [7]. Included in that report is an extensive list of references.

5.2. Interaction between wythes in cavity walls

Both wythes in the 1,20 m x 2,55 m cavity walls attained the same arc of bending, see Fig. 4, indicating that a lateral load will be shared by the wythes in proportion to their stiffness. Hence in the calculation, the Section Modulus of the cavity walls has been obtained by adding the modulus for each wythe.

5.3. Lateral load-bearing capacity

The elastic properties and the strength in bending, of masonry walls built using a specific kind of mortar and masonry units, can be determined by bending tests on piers and small wall panels. If the load when the first crack appears is used as failure criterion, the factor of safety can be fairly low as the load-bearing capacity of the structure is just partly utilized. If the load when the full crack pattern is developed is used, the factor of safety must be increased. The first crack in the structure is considered to be best basis for selecting the factor of safety.

The structure could preferably be divided into classes with structures without special control of materials and workmanship in the lowest classes. The factor of safety must therefore be higher in the lowest classes than in the highest classes for which continual control of materials and workmanship is assumed. The highest class could. for instance, be used for large daring walls in industrial buildings.

Because just two large walls have been tested, one must have reservations about the conclusions. However, the tests provide evidence that non- loadbearing walls can be designed using a calculation method based on the theory of elasticity for thin anisotropic plates in bending.

6. A SIMPLIFIED DESIGN METHOD FOR LATERALLY LOADED MASONRY WALLS

6.1. Applicability

For simple design cases deflection and moment coefficients have been worked out in two tables for uniformly loaded walls without openings and supported on four sides. For more complicated design cases, for example walls with openings, varying degrees of restraint at the edges or non-uniform loads, one is at the present time depending on the computer programme that is well adapted for handling the above-mentioned cases.

6.2. Design formulas

To calculate the maximum deflection and the maximum moment due to a uniform lateral load the following formulas may be used:

Notations:
w = maximum deflection
α = coefficient calculated by using the computer programme
q = uniform load
b = height of the wall
a = length of the wall
D = factor calculated by the given formula
h = thickness of the wall
υ = Poisson's ratio
x = the horizontal direction in the plane of the wall
y = the vertical direction in the plane of the wall
E = Modulus of Elasticity
m = maximum moment
β = coefficient calculated by using the computer programme

The coefficient α and β are listed in Tables 7 and 8 for walls respectively freely supported and fixed at the edges.

6.3. Material constants

The Modulus of Elasticity must be determined for the specific combinations of materials being used. This can be done by bending tests on piers and small wall panels as described in the paper.

Poisson's ratio has not been determined in the test programme. The tables 7 and 8 are based on a maximum value of $\nu_x = 0{,}2$. To determine the influence of a different value of ν the product of ν_x and ν_y has been chosen to be 0,04 making $\nu_x = 0{,}316$ and $\nu_y = 0{,}126$. The influence on the deflection is negligible and on the moment about 16%. See Table 7. This is considered to be tolerable as a normal factor of safety for masonry walls is 4.

Table 7. Coefficients in formulas for calculation of maximum deflection and moment for uniformly loaded walls freely supported on four sides.

Modulus of Elasticity	Poisson's ratio		Ratio: Length Heigth	Deflection coefficients	Moment coefficients	
E_x/E_y	ν_x	ν_y	a/b	α	β_x	β_y
1,5	0,2	0,133	2,0	0,0094	0,0449	0,093
			2,5	0,0109	0,0423	0,107
			3,0	0,0119	0,0415	0,116
2,0	0,2	0,1	2,0	0,0087	0,0511	0,087
			2,5	0,0105	0,0473	0,103
			3,0	0,0115	0,0461	0,112
2,5	0,2	0,08	2,0	0,0083	0,0570	0,082
			2,5	0,0101	0,0522	0,099
			3,0	0,0113	0,0503	0,110
2,5	0,316[1)]	0,126	2,0	0,0082 (0,2%)	0,0657 (15,3%)	0,084 (2,3%)
			2,5	0,0101 (0,2%)	0,0606 (14,0%)	0,100 (1,3%)
			3,0	0,0112 (0,1%)	0,0584 (16,2%)	0,111 (0,9%)

Table 8. Coefficients in formulas for calculation of maximum deflection and moment for uniformly loaded walls with fixed edges on four sides.

Modulus of Elasticity	Poisson's ratio		Ratio Length Heigth	Deflection coefficients	Moment coefficients			
					At midspan		At supports	
E_x/E_y	ν_x	ν_y	a/b	α	β_x	β_y	β_x	β_y
			2,0	0,00245	0,0173	0,0401	–0,0681	–0,0802
1,5	0,2	0,133	2,5	0,00259	0,0167	0,0423	–0,670	–0,0829
			3,0	0,00262	0,0169	0,0427	–0,0659	–0,0831
			2,0	0,00237	0,0197	0,0386	–0,0790	–0,0784
2,0	0,2	0,1	2,5	0,00256	0,0187	0,0418	–0,0788	–0,0824
			3,0	0,00261	0,0186	0,0426	–0,0766	–0,0831
			2,0	0,00229	0,0220	0,0372	–0,0884	–0,0767
2,5	0,2	0,08	2,5	0,00253	0,0205	0,0412	–0,0873	–0,0818
			3,0	0,00260	0,0199	0,0425	–0,0861	–0,0831

6.4. Allowable stresses

The allowable stresses will depend on the material properties of the masonry units and the mortar, the bond strength, and the workmanship. These factors will not be discussed in the paper and consequently quantified allowable stresses will not be recommended.

7. CONCLUSIONS

The objective of the work described in this paper was to develop an analytical stress analysis design method for masonry walls subjected to lateral loading. The effects of vertical loads are not dealt with in the paper.

A method has been developed and verified by tests on brick masonry walls. Because just two large walls have been tested, one must have reservations about the conclusions. However, the tests provide evidence that:

- Masonry walls subjected to a uniform lateral load will act as elastic plates in bending. The walls may be designed using calculation methods based on the theory of elasticity for thin anisotropic plates in bending.
- In cavity walls both wythes will attain the same arc of bending when connected by four steel ties with 5 mm diameter per square meter wall area. The Section Modulus for the wall can be determined by adding the Section Modulus for each wythe.
- Materials constants required to be known when using the above-mentioned calculation methods can be determined by bending tests on piers and small wall panels.

REFERENCES

1. Bradshaw, R.D. and F.D. Entwisle. Wind forces on non-loadbearing brickwork panels. CPTB (Clay Products Technical Bureau), Technical note, vol.1, no 6, 1955
2. Jordtryck mot motfylld murad källarvägg. Av Ivar Tengvall, Sven-Erik Rehnman och Bengt Broms och Leif Eliasson och Bernt Aldin. Sthm 1968. Byggforskningen. Rapport 6, 1968
3. Hallquist, Åge. Fasthetsegenskaper for horisontalbelastet murverk. Lateral loads on masonry walls. Oslo 1968. Norwegian Building Research Institute. Report, 56
4. Murtegelnormer, antagna at gälla från 1. april 1955 av Svenska Technologföreningen genom Svenska Arkitektföreningen och Svenska Väg- och Vattenbyggares Riksforbund samt Sveriges Tegelindustriförening. Sthm. 1959
5. Mørtel-Mur-Puss. By Henry Duhrkop, Vitold Saretok, Tenho Sneck and Sven D. Svendsen. Oslo 1966. Norwegian Building Research Institute. Handbok, 20
6. Hansteen, Harald, Finite element displacement analysis of plate bending based on rectangular elements. Oslo 1966. Norges byggforskningsinstitutt. Særtrykk, 132
7. Högberg, Erik. Mortar bond. Sthm. 1967. Byggforskningen. Rapport 40, 1967.

SUMMARY

Masonry walls bearing uniform lateral loading have been tested at NBRI. The programme was undertaken to study the stiffness and strength of masonry walls loaded laterally to provide a better understanding of their structural mechanics.

Brick cavity walls supported on 1-4 sides have been tested, six walls with dimensions 1,20 m x 2,55 m and two with dimensions 4,50 m x 2,45 m. Piers and small wall panels have been tested in bending to determine the stiffness and strength of masonry walls supported vertically or horizontally. The bond strength between brick and mortar and the strength properties of brick and mortar have also been determined.

In the wall tests the dial gauge readings indicated that both wythes got approximately the same deflection and the strain gauge readings on steel ties connecting the wythes indicated that about half the load was carried by each wythe. Hence it was concluded that the bending moment caused by lateral loads is divided between the wythes according to their stiffness in bending. The tests where one wythe in addition was loaded with a vertical load were found to be inclusive due to the fact that small uncontrolled eccentricities when applying the load will cause a large increase in bending moment.

The test data for the walls have been used to verify theoretical calculation methods. Good agreement is found treating the walls as elastic anisotropic plates. To use the method the stiffness in both directions in the plane of the wall must be known (determined by bending tests on piers and small wall panels).

Design tables for masonry walls bearing lateral loads have been worked out in this paper based on our tests and a computer programme for elastic anisotropic plates.

Reprint from CIB Symposium on bearing walls
Warsaw 1969