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THE INTERACTION BETWEEN FLOOR SLABS AND MASONRY WALLS – EXPERIMENTAL AND NUMERICAL INVESTIGATION IN COMPARISON WITH EUROCODE 6

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Abstract

The structural capacity of a masonry wall subjected to vertical loads depends on the slenderness of the wall, the effective load and the distribution of the bending moments. The assessment of the wall bending moment is difficult, since some relative rotation will occur between the wall and the floor. The Eurocode 6 contains a simplified method for calculating such bending moments induced by the floor or roof system. Experimental investigations, carried out at Dresden University of Technology, showed that the calculated moments according to EC 6 will be too conservative. In order to assess the complex behaviour of slabs and walls at their connection, a finite element model was used. The results confirm the conclusion derived from the experimental investigation. A clearly smaller effect of the moments as it has been calculated with the equations according to Eurocode 6 occurs.

Key Words

Slab-Wall Interaction, Bending Moments, Experimental Test, Numerical Simulation

1 Introduction

According to prEN1996-1-1 at the ultimate limit state the design value of the vertical load applied to a masonry wall, N_{Sd}, shall be less or equal to the design value of the vertical load resistance of the wall, N_{Rd}, such that

$$N_{Sd} \le N_{Rd} \tag{1}$$

The design value of the vertical resistance is:

$$N_{Rd} = \Phi_i \times f_k / \gamma_M \times b \times t \tag{2}$$

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f_k is the characteristic compressive strength of masonry
γ_M is the partial safety factor for the material
t is the thickness of the wall

The factor Φ_i is the capacity reduction factor at the top or bottom of the wall, allowing for the effects of load eccentricities. At the mid-height of the wall the reduction factor takes the load eccentricity and slenderness into account. In both cases, the factor may be based on a rectangular stress block. For load eccentricity effects at the top or bottom of the wall, Φ_i is given by:

$$\Phi_i = \left(1 - 2\frac{e}{t}\right) \tag{3}$$

The eccentricity e is defined by:

$$e = \frac{M}{N} + e_{he} + \frac{h_{ef}}{450} \geq 0.05t \tag{4}$$

M is the design value of the bending moment resulting from the deflection/rotation of the floor slabs
e_{he} is the eccentricity at the top and bottom of the wall, if any, resulting from horizontal loads (for example wind; $e_{he} = M_h / N$)
$h_{ef} / 450$ is the initial eccentricity

The value of N_{Rd} depends on the material parameters of the wall and on the bending moment transmitted at the connection of floor slabs and walls. The code prEN1996-1-1 contains a method of calculation for these moments.

2 Calculation method according to prEN1996-1-1

The calculation method for bending moments is given in Annex C of the code. Figure 1 shows a simplified frame diagram, based on which the bending moment can be calculated. The bending moment at the top of the wall can be found from 'frame a' with:

$$M_1^* = \frac{\frac{n_1 E_1 I_1}{h_1}}{\frac{n_1 E_1 I_1}{h_1} + \frac{n_2 E_2 I_2}{h_2} + \frac{n_3 E_3 I_3}{l_3} + \frac{n_4 E_4 I_4}{l_4}} \left[\frac{w_3 l_3^{\,2}}{4(n_3 - 1)} - \frac{w_4 l_4^{\,2}}{4(n_4 - 1)} \right] \tag{5}$$

n_i is the stiffness factor of the members and it is taken as 4 for members fixed at both ends and otherwise 3;
E_i is the modulus of elasticity of member i, where i = 1, 2, 3 or 4;
I_i is the second moment of area of member i, where i = 1, 2, 3 or 4;
$l_i ; h_i$ is the clear height or clear span of the members
w_3 , w_4 is the uniformly distributed design load on member 3 and 4

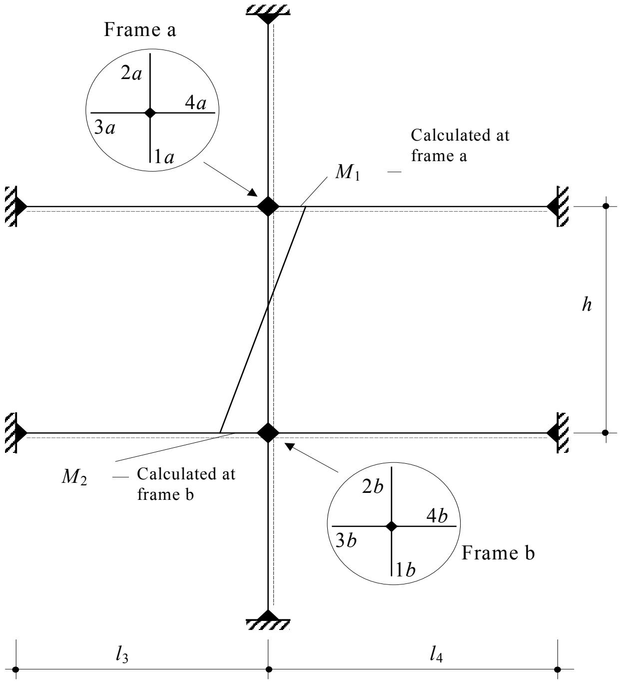


Figure 1. Simplified frame diagram acc. to prEN1996-1-1, Annex C

The bending moment at the bottom of the wall can be found in the same manner for 'frame b'. The calculated Moment M* can be reduced by the reduction factor $1-k/4$, if the average compressive stress is higher than 0.25MN/m². The value k has to be smaller than 2 and is defined by:

$$k = \frac{\frac{n_3 E_3 I_3}{l_3} + \frac{n_4 E_4 I_4}{l_4}}{\frac{n_1 E_1 I_1}{h_1} + \frac{n_2 E_2 I_2}{h_2}} \leq 2 \tag{6}$$

The design moment is then given by:

$$M_1 = M_1^* (1 - k/4) \tag{7}$$

3 Experimental investigations

3.1 Description of test structure

To verify the validity of the equations given in prEN1996-1-1, an experimental test series on a two storey, full size frame structure was performed. The test objectives were to determine the following:

- Joint bending moments, based on various floor slab loads and varying magnitudes of wall precompression
- Wall end rotations and wall deflections based on various floor slab loads and varying magnitudes of wall precompression

The general set up of the test structure and the overall dimensions are listed in Figure 2:

Floor slab:	reinforced concrete	
	width	1.00 m
	span	5.00 m
	thickness	0.16 m
Walls:	masonry	
	with	1.00 m
	height	2.00 m
	thickness	0.115 m
	units:	calcium silicate units
	(w/l/h-11,5/24,8/12,3cm³)	
	mortar:	thin layer mortar

Figure 2. Two-storey, full-size frame structure

3.2 Material properties

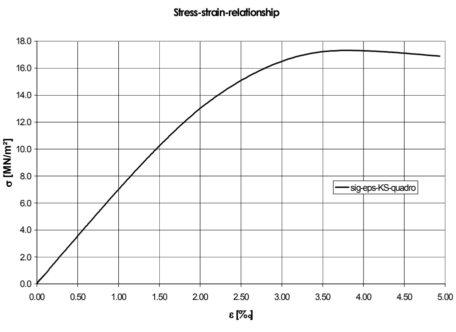
In order to simulate the experimental test with the help of the finite element method the detailed knowledge of the material properties was necessary. Therefore different tests were carried out. Figure 3 shows the test setups for determining the stress-strain relationship and the tensile strength of the masonry units.

Figure 3. Determining the stress-strain-relationship and tensile strength of the units

The following parameters could be estimated based on the experimental test:

Masonry units:

- Density $\rho_b = 1.912\ t/m^3$
- Compressive strength $f_b = 18.2\ MN/m^2$
- Tensile strength $f_{bt} = 1.46\ MN/m^2$
- Modulus of elasticity $E = 7143\ MN/m^2$
- Poisson ratio $\nu_b = 0.17$



Mortar

- Density $\rho_m = 1.43\ t/m^3$
- Compressive strength $f_m = 11.33\ MN/m^2$
- Modulus of elasticity $E = 7970\ MN/m^2$

Concrete

- Density $\rho_c = 2.33\ t/m^3$
- Compressive strength $f_c = 32.14\ MN/m^2$
- Tensile strength $f_{ct} = 2.6$ MN/m²
- Modulus of elasticity $E = 28821\ MN/m^2$
- Poisson ratio $\nu_c = 0.17$

3.3 Loading sequence and instrumentation

The tests were conducted using the initial position of the structure (before the formwork was removed) as the reference position. Two independent loading systems were used. One system applied an axial load to the wall using hydraulic jacks. These jacks were located at the top of the upper wall. With the second system, load could be applied to the floor slab at quarter points via one hydraulic jack. The jack load was distributed on the slab by seven steel beams. The applied loads were measured using load cells located under every jack. The magnitude of active wall precompression (i.e jack loads excluding dead load of structure) varied from 0kN/wall up to 100kN/wall. The two floor-slab loads were 0.95kN and 4.70kN at each load point. Measurements of deflection and rotation were taken at several location of the structure. A total of 15 inductive displacement transducers were used on the middle axis of the walls and slab. The rotation of the walls and the floor-slab at the connection was measured using electronic levels. For this, a total of five levels were installed. Additionally, the magnitude of the reaction of the slab was measured with two load cells. The locations where the measurements were taken are shown in Figure 4.

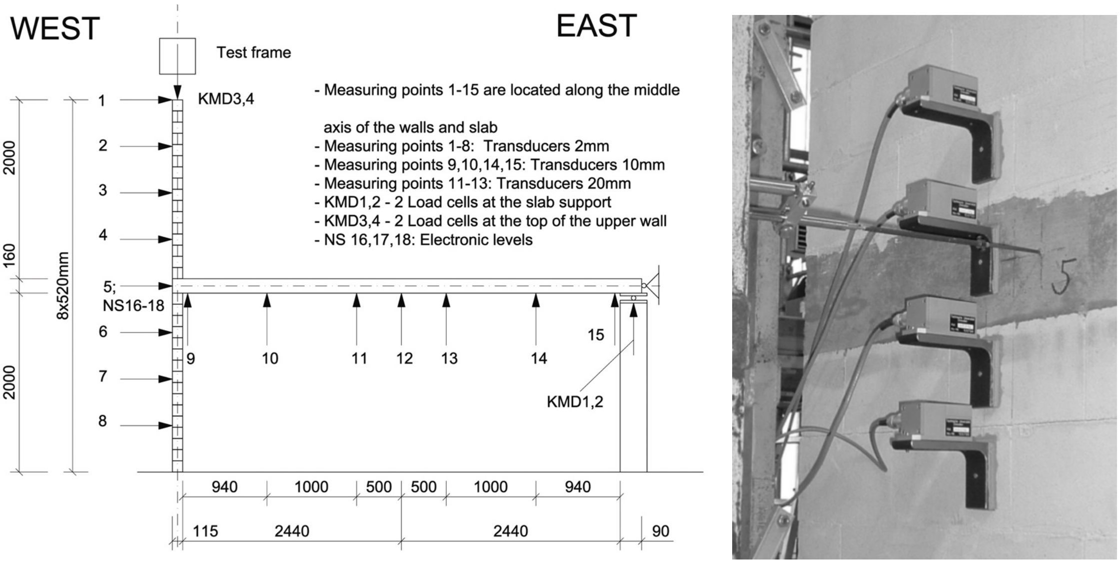


Figure 4. Instrumentation setup and locations of electronic levels around the joint-area

3.4 Results – experimental investigation

The recorded data confirms the predicted behaviour of the structure. Wall precompression greatly influences the joint stiffness. For load steps with a large load on the floor-slab and minimal wall precompression, large cracks in the masonry bed joints occurred. At load step 20, the crack length or depth was more than 8.5cm.

According to Awni, the slab restraining moments due to floor load application may be computed on the basis of the following simple system.

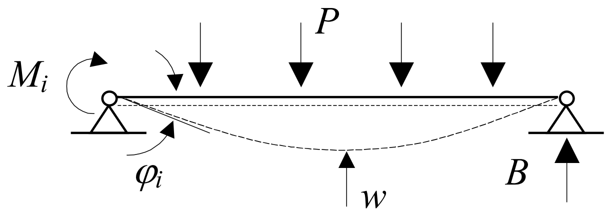


Figure 5. Beam with four point loads and restraining moment

The following relationship from simple mechanics can be achieved:

- Based on midspan deflection

$$M = \frac{16EI}{l^2}w - \frac{41}{768}Pl \tag{8}$$

- Based on slab end rotation

$$M = \frac{3EI}{l}\varphi - \frac{33}{64}Pl \tag{9}$$

w, φ	are the deformations due to slab loads
P	is the applied floor load
EI	is the bending stiffness of the slab

Unfortunately, the bending stiffness of the RC-slab is not known. Based on the fact that the moments calculated according to equations (8) and (9) have to be equal, a fictitious value of EI can be determined. With the help of this value, the slab restraining moments can be calculated. The results are shown in Table 1.

Table 1 Slab restraining moments

Load Step	w	φ	N	P	EI	M_φ	M_w	ΔM
	mm	°	kN	kN	MNm²	kNm	kNm	kNm
1	1.817	-0.0579	0.00	0.95	2.97	-0.674	-0.674	-
20	12.446	-0.4360	0.00	4.70	2.40	-1.289	-1.289	**-0.615**
3	1.080	-0.0045	50.00	0.95	2.54	-2.344	-2.344	-
11	8.694	-0.2759	50.00	4.70	3.05	-3.434	-3.434	**-1.090**
5	0.636	0.0210	100.00	0.95	2.61	-3.033	-3.033	-
8	5.939	-0.1716	100.00	4.70	4.06	-4.933	-4.933	**-1.899**

The value ΔM is the slab restraining moment induced by the difference of the slab load between load step 1 and 20; 3 and 11and 5 and 8 ($\Delta P = 3.75kN$). As mentioned above, additional load cells were installed in order to measure the vertical load at the support conditions of the slab. With the help of these values, the slab restraining moment can also be determined.

Table 2 Slab restraining moments

Load Step	B	M_B	P	G	M_P	M_G	M_{P+G}	ΣM	ΔM
	kN	kNm	kN	kN	kNm	kNm	kNm	kNm	kNm
1	11.79	59.308	0.95	20.140	9.477	50.652	60.130	-0.822	
20	19.10	96.073	4.70	20.140	46.889	50.652	97.541	-1.468	**-0.646**
3	11.50	57.845	0.95	20.140	9.477	50.652	60.130	-2.285	
11	18.70	94.061	4.70	20.140	46.889	50.652	97.541	-3.480	**-1.195**
5	11.30	56.839	0.95	20.140	9.477	50.652	60.130	-3.291	
8	18.35	92.301	4.70	20.140	46.889	50.652	97.541	-5.240	**-1.950**

4 Numerical simulation

4.1 Specifications of the numerical model

The experimental test was simulated with the help of an engineering simulation software. Therefore, the model was created within the ANSYS program system. In order to describe the behaviour as realistically as possible, the masonry units, mortar joints and the reinforced concrete-slab were modelled separately. This allows to consider the specific properties of each component. The masonry units and the were RC-slab were modelled with SOLID65 elements. This is an eight-node 3-D structural solid-element as shown in Figure 6. It is capable of cracking in tension and crushing in compression. In concrete applications, for example, the solid capability of the element may be used to model the concrete while the rebar capability is available for modelling reinforcement behaviour. During the experimental investigations cracking processes in the bed joints of the masonry walls could be observed. In order to simulate this, contact elements (CONTA173) were applied. These elements are able to transmit pressure and shear forces, but they can prevent tensional stresses. The failure law for the material was the Willam-Warnke five parameters model. Based on these criterions, a non linear analysis was performed with the material properties obtained from the experimental tests (see section 3.2).

4.2 Results – numerical simulation

Figure 6 shows the loading of the slabs and the deformed FEM-model for load step 20.

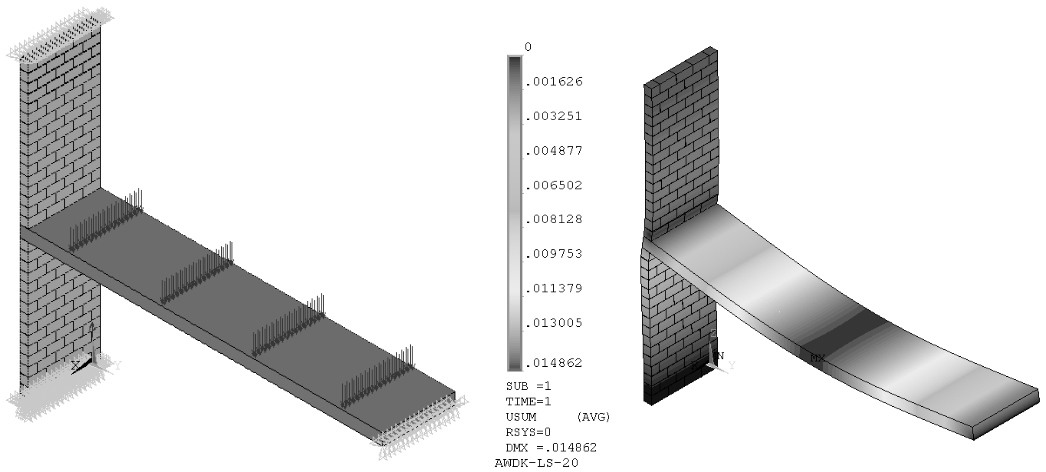


Figure 6. Applied loads and boundary conditions (left) and deformation (load step 20)

The results of the numerical simulation were in close agreement with the experimental test for all simulated load steps. The cracking of the RC-floor slab and the cracking of the bed joints near the connection of the slab and the walls could also be seen in the numerical simulation.

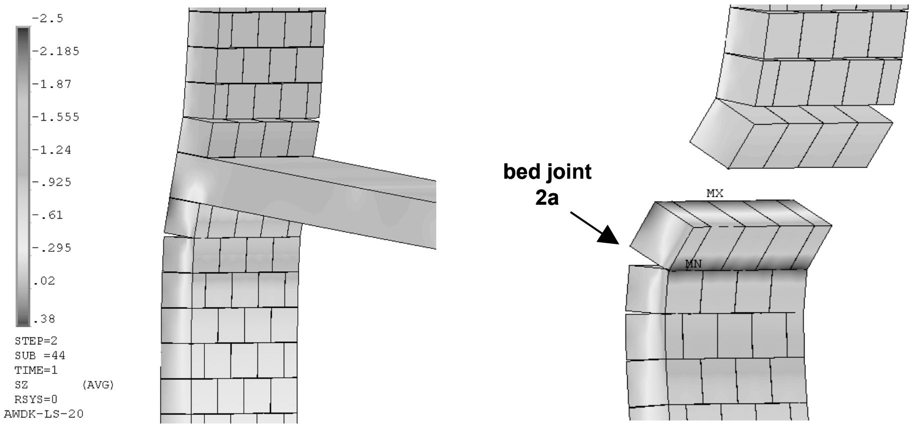


Figure 7. Vertical stresses (SZ) and cracking of the joints (Load step 20)

Especially the simulated length of the crack in the bed joint coincided very well with the experimental results.

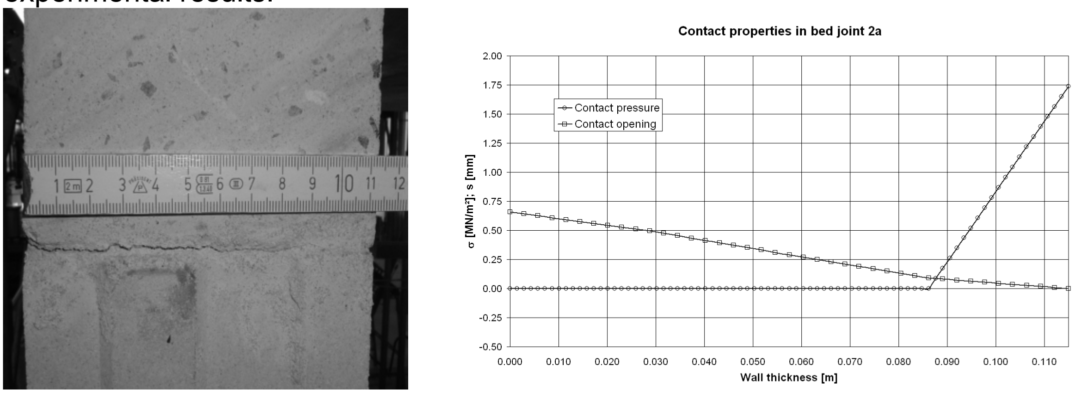


Figure 8. Length of the crack in bed joint 2a (left: experimental test; right: FEM)

The slab restraining moments were determined using the calculated supporting reaction B (similar to the procedure in section 3.4, see also Figure 5). Table 3 lists the results, which were obtained.

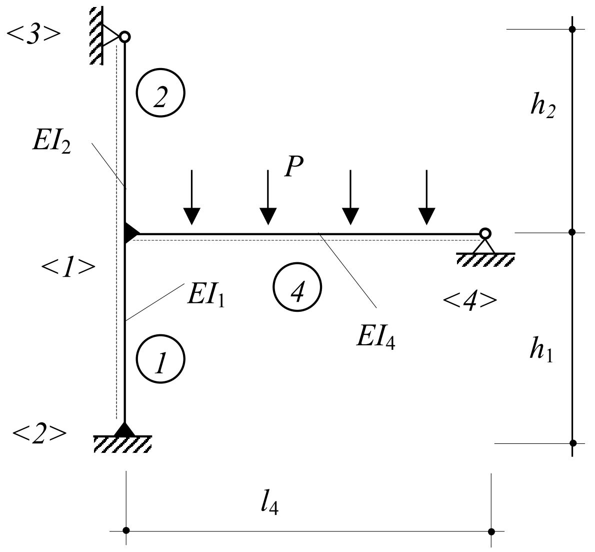
Table 3 Slab restraining moments

Load Step	B	M_B	P	G	M_P	M_G	M_{P+G}	ΣM	ΔM
	kN	kNm	kN	kN	kNm	kNm	kNm	kNm	kNm
1	11.76	59.15	0.95	20.14	9.48	50.65	60.13	0.98	-
20	19.17	96.43	4.70	20.14	46.89	50.65	97.54	1.12	**0.14**
3	11.53	58.00	0.95	20.14	9.48	50.65	60.13	2.13	-
11	18.72	94.16	4.70	20.14	46.89	50.65	97.54	3.38	**1.25**
5	11.27	56.69	0.95	20.14	9.48	50.65	60.13	3.44	-
8	18.32	92.15	4.70	20.14	46.89	50.65	97.54	5.39	**1.95**

5 Bending moments according to prEN1996-1-1

For a comparison of the obtained experimental and numerical data, the results according to prEN1996-1-1 were needed. The design procedure, explained in section 2 of this paper, was applied. The procedure provides the bending moment at the top of the wall. For comparison, the slab restraining moments are of interest. Therefore, the

procedure was modified so that the slab restraining moments could be calculated. These bending moments only have to be distributed according to the stiffness relation of the both walls in order to receive the wall bending moments (at the top and at the bottom of the wall). Therefore all conclusions regarding the slab restraining moments will be also valid for the wall end moments.



- $E_{1,2} = 5329\ \text{MN/m}^2$ *(Experimental test)*
- $E_{1,2} = 12340\ \text{MN/m}^2$ *($1000 f_k$ acc. to prEN1996-1-1)*
- $I_{1,2} = 1.276 \cdot 10^{-4}\ m^4$
- $h_{1,2} = 2.00\ m$
- $E_4 = 28821\ \text{MN/m}^2$
- $I_4 = 3.413 \cdot 10^{-4}\ m^4$
- $l_4 = 5.03\ m$
- $P = 3.75\ kN$

Figure 9. System and parameters for the calculation of the slab restraining moment

The slab restraining moment is given by:

$$M_4^{\ *} = \frac{33}{64} Pl \cdot \left(1 - \frac{\dfrac{3EI_4}{l_4}}{\dfrac{4EI_1}{h_1} + \dfrac{3EI_2}{h_2} + \dfrac{3EI_4}{l_4}} \right) \qquad (10)$$

Table 4 Slab restraining moments acc. to prEN1996-1-1

Modulus of Elasticity for masonry	Slab restraining moment	Reduction factor acc. to equation (6)	Reduced moment
$E_{1,2} = 5329\ MN/m^2$	$M_4^{\ *} = 2{,}788\ kNm$	$k = 2{,}0$	$M_4 = 1{,}394\ kNm$
$E_{1,2} = 12340\ MN/m^2$	$M_4^{\ *} = 4{,}698\ kNm$	$k = 1{,}07$	$M_4 = 3{,}441\ kNm$

6 Comparison and conclusion

The calculated bending moments according to prEN 1996-1-1 are different from the experimental test moments. Especially for small vertical loads in the masonry walls the test values are much smaller. With the help of the reduction factor according to equation (6) the results can be better adapted to the reality, but they are still too high for small vertical loads. For thicker walls, this tendency will intensify greatly, because of the higher elastic stiffness of the walls.

The modulus of elasticity of the masonry material has a great impact on the results. According to prEN1996-1-1 the value for the investigated masonry is given by $1000 f_k$ ($E = 12340 MN/m^2$). The value obtained by experimental tests was only $430 f_k$ ($E = 5329\ MN/m^2$). The calculated moments diverge up to 40% before using the reduction factor and up to 60% after applying the reduction factor (see *Table* 4).

The slab restraining moment is influenced a great deal by the vertical loads acting in the walls. However, the diagram in Figure 1 does not allow such affects. In other words, for every vertical load in the masonry walls, the slab restraining moment will be equal based on the model of the diagram. The experimental and numerical investigation, however provide quite different results.

The experimental data and the results of the numerical simulation are in very close agreement. FEM models can provide a deeper insight into the behaviour of masonry structures. The cracking phenomenon in the bed joints as well as in the reinforced concrete-slabs can be studied in greater detail with the help of numerical simulations. If the mechanical properties of the materials are evaluated by realistic testing methods, then the behaviour predicted by the FEM model will be reasonably accurate. FEM-models can be used to improve the traditional calculation method in order to receive more realistic results. This is the aim of a research project currently under way at the Department of Structural Design at Dresden University of technology.

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