Assessment of the loadbearing capacity of historic masonry

A. Sabha[1], C. Neuwald-Burg[2]
[1] *Büro für Tragwerksplanung, Am Weißiger Bach 39 a, D-01474 Weißig*
[2] *SFB 315, Universität Karlsruhe, Institut für Tragkonstruktionen, Forschungsgruppe Mauerwerk, Hertzstr. 16, D-76135 Karlsruhe, Germany*

Abstract

Historic masonry differs from modern masonry by construction and material. The assessment of its loadbearing capacity therefore needs a particular approach. Technical guidelines are neither available nor desirable, because each monument requires an individual solution. The focus of this paper is to show a sensitive procedure to analyse strength and loadbearing behaviour of existing masonry, which may be applied in most cases. Special attention is paid to a suitable safety concept for existing structures. Possible ways to assess masonry strength under concentric and eccentric compression are demonstrated. The loadbearing behaviour of different types of masonry, single and multiple leaf walls, is described. The paper also deals with sampling and testing material from historic masonry in order to assess the characteristic values needed in the analysis. The reflections are based on experimental work carried out on masonry pillars and multiple leaf masonry samples, as well as on finite element analysis.

Introduction

Masonry of historic buildings is a document that contains information about craftsmanship, technical knowledge and economic circumstances in former times.

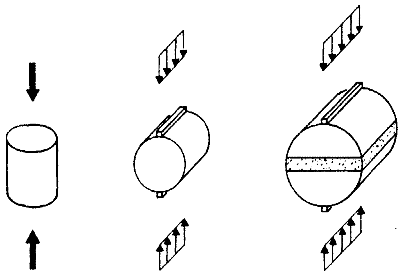

Figure 1: Tests on bedjoint cores allow thelower limit of masonry strength to be assessed [1]

Restoration and repair have to consider the importance of this information and conserve the material as authentic as possible. Though many features of the old constructions cannot yet be fully explained, a lot of knowledge has been acquired. The results of the bearing behaviour of masonry reported here have been obtained within the framework of a large research programme carried out within the SFB 315 (Special research area 'Conservation of historically important buildings') at the University of Karlsruhe. Investigations into the loadbearing behaviour of multiple leaf masonry were reported in companion papers to the StremaH [1, 2, 3]. Further investigations by means of more sophisticated finite element analysis have given an inside view into the loadbearing behaviour of single and multiple leaf masonry. Experimental investigations have shown that we tend to underestimate the bearing capacity of old masonry. The following paragraphs roughly describe the different investigations and their results, with a focus on practical use.

Different safety concepts required: historic and modern masonry

Current industrial codes aim to ensure safety in buildings based on particular safety concepts, which are usually written to design new structures. Employing these concepts to verify the safety-level of old structures bears numerous questions related to the statistical properties of material data. The safety design of new building guarantees a distinct safety for a determined lifetime. For historic structures the existing safety level must be assessed and from that an expected lifetime may be derived. A general concept of safety analysis for old buildings is difficult. Even though, a static proof frequently is necessary and the fact that a structure has survived for several hundred years cannot be accepted as a safety proof. If an irrevocable loss of substance in historic buildings has to be avoided, the goal must rather be to ensure higher reliability of the calculated results than to justify the minimum level of safety given by the codes. In many cases a historic building will not perform as required in the codes. It is then necessary to control the *reliability index*. Additional to the application of new computing methods and new developed approaches of calculating the bearing capacity, the intense inquiry of material data as well as deformation and loading control are further possibilities to use lower safety factors and lower service loads. A global safety factor as used in DIN 1053-1 [4], where load and resistance are considered to be equally reliable is not suitable for historic buildings. Partial safety factors as included in EC 6 [5] differentiate material resistance and different types of loading and therefore permit the safety-level to be assessed more adequately.

Concerning the material EC 6 gives a factor for the unit quality and for the workmanship. Applied on existing old masonry the unit category factor can be replaced by a multiplier for the number of test samples. By keeping the boundary values given in EC 6 this factor will take the value γ_m = 1,7 if more than 30 samples can be tested and γ_m = 2,0 for only 3 test samples. Instead of the workmanship-factor a value covering the testing quality should be introduced, which indicates the reliability.

According to EC 6 characteristic values are to be used. For loading these are mean values except for a variation coefficient > 1,0 or if there is sensibility against increase of dead load. For dead load a fraction value, for a probability of p = 0,02 is required. Moreover, there are partial safety factors for each load and load combination which allow more flexibility by justifying safety requirements for old structures.

Material testing

Masonry samples of sufficient size for direct compression tests can rarely be obtained from historic buildings. Non destructive in-situ tests sometimes may be employed but usually the masonry strength will be derived from the results of tests on units and if possible on mortar samples taken from specific areas of the structure. The appropriate choice of the samples is extremely important, because due to weathering, change of material etc. the mechanical characteristics vary a lot, even in very small areas of one wall. Choosing the weakest material will give safe results for compressive strength, but may evoke a wrong idea of stiffness. The samples should have been submitted to the same influences that are acting on the investigated part of masonry: The material should have the same age and exposure as well as a similar loading history. Unavoidable differences between the behaviour in a laboratory test and inside of a real structure must be covered by reduction factors. The masonry strength inside of the structure can be calculated from test results as

$$f_{ma} = \alpha_1 \cdot \alpha_2 \cdot \alpha_3 \cdot f_{cr}$$

Where f_{cr} the failure stress in test as 5% fractil
α_1 reduction factor for unconsidered influences such as ageing, loading history or moisture
α_2 factor for the consideration of short time effect
α_3 shape and slenderness conversion factor

Mortar strength is usually difficult to assess from direct testing. A very useful composite test (fig. 1) has been developed by Berger [1].

The calculation of old masonry according to current codes such as DIN 1053/1 usually leads to an underestimation of the real strength of existing masonry. This underestimation arises mainly from assumptions and generalisation regarding design

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methods, geometry and material laws, which often are not valid in the old masonry structures. In order to avoid exaggerated interventions and to optimise restoration work it is necessary to analyse the bearing capacity for each construction individually, by taking a maximum of specific parameters into account, such as mean strength, unit height to width to thickness-ratio, true bedjoint thickness, bond. Mechanical models based on elastic material laws have shown inadequate for historic stone-mortar-combinations. Soft mortar tends to plastisize already under low pressure. A model which describes the effect of plastification on the inner state of stress and on failure was established for different types of single leaf masonry in regular bond [6] (fig. 2).

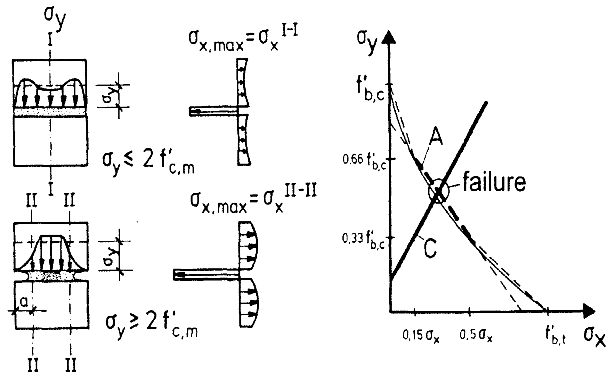

Figure 2: Failure criterion for masonry based on the model of plastisized bedjoints [6]

Compressive strength: Model of plastisized bed joints

This approach assumes a biaxial stress state in mortar and stone. According to FEM-Investigation it was possible to determine an envelope curve, defining the relationship between vertical and lateral stresses in the mortar. Using the failure

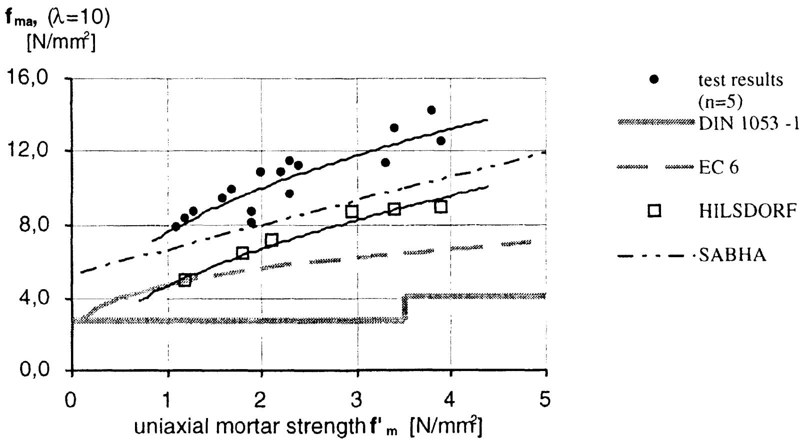

Figure 3: Test results compared to theoretical masonry strength.

criterion of stone material, the intersection between the two curves defines masonry failure. The distributions of lateral stresses in stone and mortar are also assumed to be uniform and equal. According to these results the bearing capacity of the combination stone-mortar is

$$f_{c,Ma} = \frac{2Kf_{c,m} + f_{t,s}}{K + \frac{f_{t,s}}{f_{c,s}}}, \qquad (1)$$

with
$$K = \frac{t}{b}\left(2{,}32\frac{f_{t,s}}{f_{c,s}} + 1{,}6\sqrt{\frac{b}{h}}\right)$$

where

$f_{t,S}, f_{c,S}$	- tensile and compressive strength of stone
f	- compressive strength of mortar
t, b	- bedjoint height and width
h	- stone-height

For brickwork the equation becomes

$$f_w = \frac{2Kf_{c,m} + 0{,}845f_{t,s}}{K + 1{,}042\frac{f_{t,s}}{f_{c,s}}} h \qquad (2)$$

with
$$K = \frac{t}{b}\left(2{,}42\frac{f_{t,s}}{f_{c,s}} + 1{,}6\sqrt{\frac{b}{h}}\right)$$

if the failure criterion for brick units suggested by HENDRY [7] is employed in its linear mode according to OHLER [8].

Tests on brickwork confirm that the model is still on the safe side, but much closer to reality than the codes or elastic models (fig. 3) [9].

Eccentric load

Plastisizing of the edges becomes extremely important especially under eccentric load where this effect causes an increasing accidental eccentricity. A series of tests on five-stack-bond brickwork prisms as well as tests on sandstone pillars with different types of mortar and different bedjoint thicknesses showed, that due to plastification a somehow premature failure occurs due formation of hinges in the plane of loading which leads to an opening of the bedjoints. This then causes important rotations which finally cause a loss of stability. The effect is worse for thick bedjoints and for rather soft mortars. Relatively small eccentricities may cause failure in this type of masonry [9, 10].

By comparing σ–ε-diagrams obtained from brickwork piers with weak mortars under centric load and under eccentricites e of 1/6 or 1/3 of the section thickness it was demonstrated that the behaviour under eccentric load cannot be predicted only from the centric compressive tests. Attention also must be paid when the loadbearing

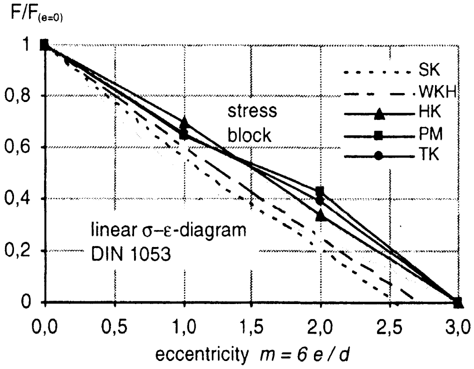

Figure 4:
Reductof the bearing capacity with increasing eccentricities for different types of brickwork.

capacity under eccentric load is derived from the crushing strength obtained from direct tests or calculated as above: Current reduction factors may be unsafe (fig. 4).

Sand stone pillars (three stack bond prisms) were investigated in the same way (centric load, $e=d/6$ and $e=d/3$) and simulated in a finite-element analysis. Using a linear-elastic-ideal-plastic material law and geometric non-linearity between stone and mortar, the possible stress fields in the stone under eccentric loads were revealed. Depending on the assumed material input data the failure was expected as splitting mode in the stone. The resulting normal stress distribution in the stone-mortar contact surface was non-linear. The analysis with relatively thick bed joints led to collapse, that means loss of stability Though this FEM-analysis must be considered as a qualitative rather than qualtitative description of the structural behaviour, because the non-linear material input data of stone and mortar only could be suggested, it allows to derive a suitable reduction factor for eccentric load. Obviously, the character of this reduction factor depends on the distribution of normal stress in the section. The test results showed a linear relationship between failure load and related eccentricity e/d. This indicates an approximately rectangular distribution of normal stress in masonry sections. The test results showed an additional relation between the reduction factor and the thickness of bed joints. According to the test results the reduction factor can be written as

$$\eta = \frac{d - 2e - 6,5e\frac{t}{d}}{d} \qquad (3)$$

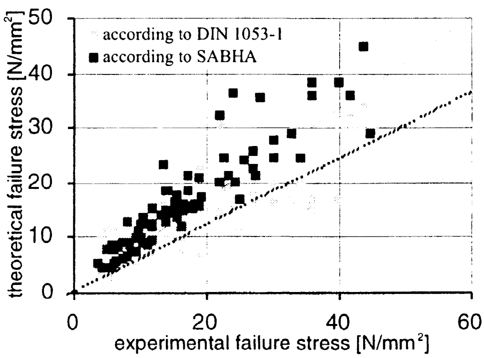

Figure 5: Centric and eccentric test results compared to predicted strength.

For stiffer mortar and for thin bedjoints sliding rather occurs than loss of stability. The failure mode, 'sliding' needs another definition of the reduction factor, although the reduction formulated above seems to be valid for the tests of sliding failure too. For this reason and for practicable formulation, this reduction factor will be generalised for all tests carried out. Hence it would be advisable to consider the influence of the bed-joint thickness on the limit eccentricity too. This limitation means either keeping stability sufficiently safe or limiting the tensile strain. The limit eccentricity can be approximately calculated as

$$e \leq \frac{d}{3+10\frac{t}{d}} \qquad (4)$$

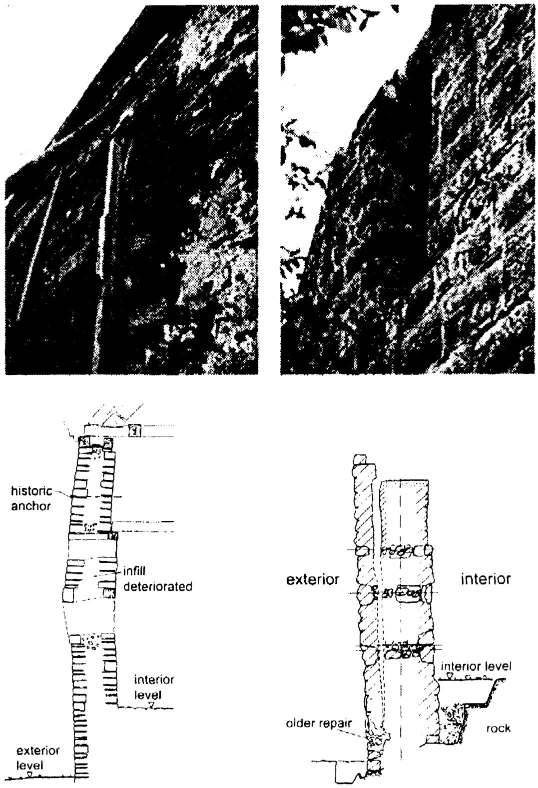

Figure 6: Damages on multiple leaf masonry sectionsfrom medieval buildings.

The tests illustrated above and further tests carried out by other authors with concentric or eccentric loads are calculated according to eq.2 and eq. 3 [11, 8]. Fig. 5 shows a comparison between the theoretical expected failure stress and the test results. The distances between the test point and agreement line indicate the validity of suggested formulation. For the application of the reduction factor eq. 3 it would be advisable to use it only for eccentricities with bending out-of-plane and for one unit masonry sections. Bending in plane and sections with more than one unit requires further investigations.

Multiple leaf masonry

Parameters

Historic masonry is frequently built in multiple leaf walls consisting, in general, of two outer walls and a more or less heterogeneous infill; with the thickness of these walls starting at 50 cm. An enormous variety of construction techniques can be observed. Detailed understanding of the behaviour of these structures is necessary to minimise restoration interventions for strengthening or repair. The bearing capacity of multiple leaf walls depend on the material and bond of outer shells and

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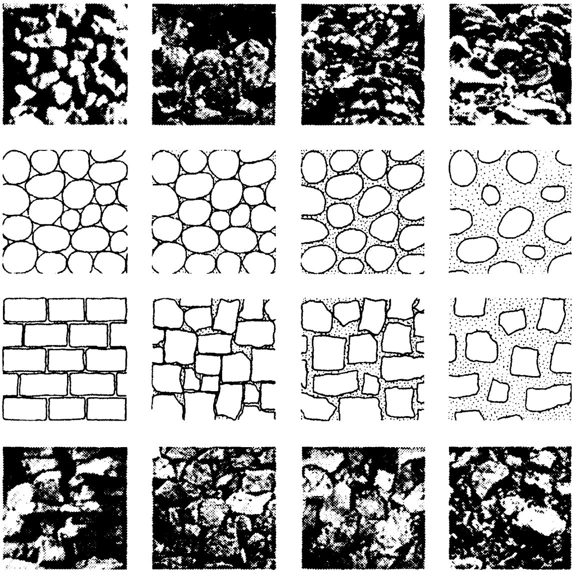

Figure 7: Loose infill material can be classified with respect to the percentage of fine particles, the size and shape of coarse material and the type of stratification.

infill, the structure of the contact surface, the static system of the outer shells and the type of loading. Figure 6 gives two different examples with their characteristic modes of decay. It is obvious that only in rare cases a simple linear elastic analysis is sufficient to describe these structures, but only the application of non-linear material laws is suitable to fully explain the features.

Outer leaves

The outer leaves usually are mainly submitted to normal stresses and out-of-plane bending. Their mechanical properties can be assessed in the same way as described for single leaf masonry.

Infill

The masonry infill may have a good bond, but often it is made up of rubber material of irregular shape and sizes with or without mortar. The most important characteristics then is cohesion, inner friction and the grade of anisotropy of the material. Infills can be classified by the quantitative stone-mortar-relationship and by a parameter for the stratification (Fig. 7).

Contact surface

The contact surface between inner and outer shells is significant for the interaction of the leaves. Five different types are to be distinguished. Rock mechanical models are suitable for their description (fig. 8) [12]. Especially for soft or loose infill, attention has to be paid if indentation may influence the location of shear planes (fig. 9).

Loose infill

Multiple leaf walls often embody a weakened or dried out core infill contained by facing walls. The stability of this type of section depends on the pressure exerted on the two outer leaves from the infield gravity weight. Various theories exist which deal with associated stress conditions (Silo theory, normal earth pressure and granular

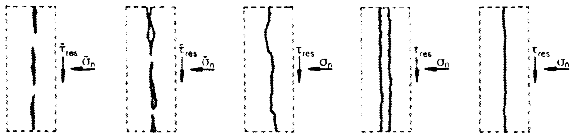

Figure 8: Interface modells employed in rock mechanics [12]

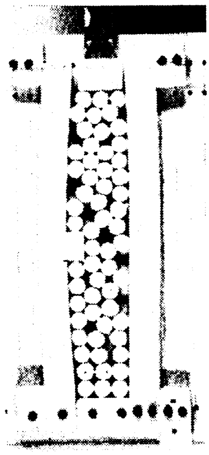

Figure 9: Indentation influences the location of shera plane within the infill.

particle models), but up to now, none of them has been proved reliable for multiple leaf masonry. A pilot study was therefore conducted in which corresponding experimental loading and failure modes were investigated on model walls [13]. The tests have been run on geometrically scaled model walls. Because of the essentially non-linear stress-strain nature of the model assembly, a correct analogue reproduction of self weight stresses was the overriding factor. Therefore the tests were performed in a geotechnical Centrifuge. The testing was carried out in the Cardiff Centrifuge Facility, England, supported by a collaborative E.P.S.R.C/D.F.G. Research Programme A total of 10 tests were undertaken in which fine and coarse single-size sands formed the model infill and the infill thickness was between one to five times that of the outer wall leaves. The test assembly enabled continuous observation of horizontal and vertical deformations and loadings on the external faces. With the more slender wall shapes it was possible to cycle the accelerations and this ultimately produced a failure in most cases. Different failure mechanisms were observed. A comparison between the test results and theoretical solutions indicate that both - silo theory as well as an earth pressure theory - may be applied. Only the latter approach allows to determine failure mechanisms (fig. 10) and to consider kinematic aspects. A modified earth pressure theory moreover can be generalised for cohesive infill materials and will therefore be recommended [14].

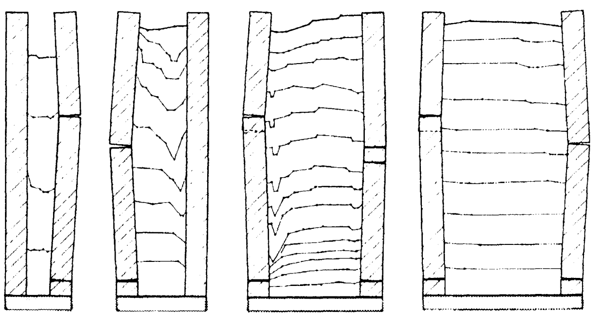

Figure 10: Failure mechanisms of multiple leaf walls observed in centrifuge tests.

Outlook

The focus of further research on the carrying capacity of single leaf masonry now has to deal with mechanical tests of infill materials.

Acknowledgements

The authors wish to acknowledge the advise and encouragement of Professor F. Wenzel at the University of Karlsruhe as well as the financial support of the Deutsche Forschungsgesellschaft DFG.

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