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RELIABILITY OF MASONRY PANELS SUBJECTED TO IN-PLANE SHEAR

E. Brehm[1], J. E. Dickie [2] and S. L. Lissel [3]

[1] Research associate, PhD candidate, Institute for Concrete and Masonry Structures, Technische Universität Darmstadt, Petersenstr. 12, 64287 Darmstadt, Germany, brehm@massivbau.tu-darmstadt.de

[2] PhD student, Department of Civil Engineering, The University of Calgary, 2500 University Drive NW, Calgary, AB, T2N 1N4, Canada

[3] Associate Professor, Department of Civil Engineering, The University of Calgary

ABSTRACT

Masonry shear walls are the major lateral load-carrying elements in masonry structures. Due to the fact that failure of the member might directly be followed by collapse of the structure, reliability analysis of masonry walls subjected to in-plane shear is important. Nevertheless, reliability of these members has not been subjected to extensive research due to the complex load-carrying behaviour. In this paper, an approach for the assessment of the reliability of masonry walls subjected to in-plane shear using analytical models is examined. An example wall is designed according to Canadian codes and subsequently analysed using probabilistic methods. The reliability of the wall will be determined and a parameter study will be performed. In the last step, partial safety factors for the safe design of masonry walls subjected to in-plane shear will be recommended.

KEYWORDS: in-plane shear, masonry, reliability, stochastic analysis

INTRODUCTION

The key objective in structural design is the design of sufficiently safe and reliable structures. While safety commonly refers to the absence of hazards, reliability is a calculable value that can be determined by probabilistic methods. In current structural design codes, the demands of safety are accommodated by the use of partial safety factors which can be derived from probabilistic analysis. Unlike other materials in construction, the reliability of masonry members has not been subjected to extensive research in the past. Recent research (see [1]) showed the necessity for a probabilistic approach to masonry structures.

Masonry walls subjected to in-plane shear, exhibit complex load carrying behaviour which is difficult to describe by analytical models. Hence, finite element analysis combined with software for reliability analysis have been preferred in the past. However, since the numerical modeling of masonry is also difficult and a large database of experiments is required for the calibration of the FE model, in this paper several analytical approaches will be compared to test data and assessed.

RELIABILITY OF STRUCTURES

The most important requirement for structures is reliability. The term reliability concerns every aspect of a structure, structures have to be reliable when it comes to load bearing capacity as well as serviceability. In design, every parameter is uncertain to some extent. The uncertainty may be in the strength of materials as well as in dimensions and quality of workmanship. All parameters, further referred to as basic variables, influence the properties of a member. Reliability is linked to the probability that a member will exceed a certain limit state. This can be described by so called limit state functions. For ultimate limit state, the limit state function can be written as follows:

$$Z_{(R,E)} = R - E \tag{1}$$

where R is resistance and E is load effect.

In the case where R = E, the ultimate limit state is reached. It can be seen from this equation that the safety of a member can be defined as the difference between the resistance and load effect. It has to be noted that R and E are independent random variables in many cases, so they have to be described by means of stochastics. Therefore a stochastic model, mostly consisting of a probability distribution and the corresponding moments (e.g. mean, standard deviation) is required for every basic variable.

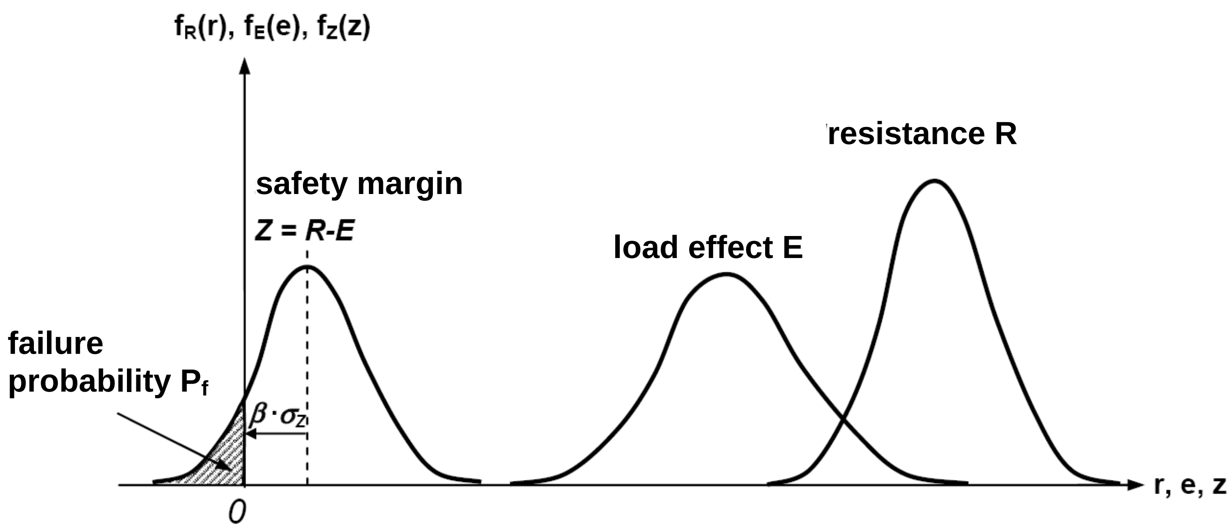

Figure 1: Definition of failure probability [1]

The failure probability can be computed by probabilistic methods such as SORM (Second Order Reliability Method) or Monte Carlo-simulation. For further information see [2].

For the description of the resistance proper models are required that describe the load carrying behaviour realistically. Contrary to design models, a model that underestimates the load carrying behaviour is not sufficient for probabilistic analysis.

To find a measure for reliability that can be defined independently from the type of distribution of the basic variables, the reliability index β_R according to [3] has proven useful. The major advantage of this definition is that only the mean, m_z, and standard deviation, σ_z, of the basic variables need to be known.

$$\beta_R = \frac{m_Z}{\sigma_Z} \tag{2}$$

With this measure, target reliabilities can be defined. Ideally, target reliabilities are based on a complex optimization process accounting for aspects of safety as well as economic requirements. In the past, target reliability has mostly been determined on an empirical basis. Since the target reliability has a major influence on safety factors, setting too large of a target reliability will lead to uneconomic design. More information can be found in [1], [4] and [5].

The Joint Committee on Structural Safety (JCSS) [6] gives the target reliabilities depending on the failure consequences as shown in Table 1.

Table 1: Target reliabilities according to [6] for an observation period of 1 year

relative cost for enhancing the structural reliability	failure consequences Minor[a)]	 Average[b)]	 Major[c)]
large	β=3.1 ($P_f \approx \cdot 10^{-3}$)	β=3.3 ($P_f \approx \cdot 5\cdot 10^{-4}$)	β=3.7 ($P_f \approx 10^{-4}$)
medium	β=3.7 ($P_f \approx 10^{-4}$)	β=4.2 ($P_f \approx 10^{-5}$)[d)]	β=4.4 ($P_f \approx 5\cdot 10^{-6}$)
small	β=4.2 ($P_f \approx \cdot 10^{-5}$)	β=4.4 ($P_f \approx 5\cdot 10^{-5}$)	β=4.7 ($P_f \approx 10^{-6}$)

[a)] e.g. agricultural building
[b)] e.g. office buildings, residential buildings or industrial buildings
[c)] e.g. bridges, stadiums or high-rise buildings
[d)] recommendation for regular cases according to JCSS 2001

These target reliabilities are considered to be sufficient for most cases and will be taken as reference for further calculations. Another recommendation is given by the German code DIN 1055-100 [7]. There, a value of β_{target} = 4.7 is given for a 1 year observation period.
A full probabilistic approach for design is difficult since stochastic models have to be known for all basic variables and good prediction models are required. To simplify design, the semi-probabilistic partial safety concept is applied in most design codes. In this concept, the partial safety factors for different basic variables make it possible to account for different scatter of the variables. A typical application of partial safety factors is presented by the following equation:

$$\gamma_E \cdot E \le \frac{R}{\gamma_R} \tag{3}$$

where E is load effect and R is resistance. The safety factors, which are greater than unity, are represented by γ_i.

SHEAR STRENGTH OF MASONRY WALLS

Masonry members subjected to shear show a complex load-carrying behaviour. There is, however, a general consensus on the 3 main in-plane failure modes in masonry which include: flexural failure (tension at the heel or crushing at the toe), sliding failure in one or multiple bed joints, and diagonal tensile failure of the panel, which may be combined with sliding failure of the joints. Cracks are typically diagonal and stepped in nature but may also traverse through units as shown below.

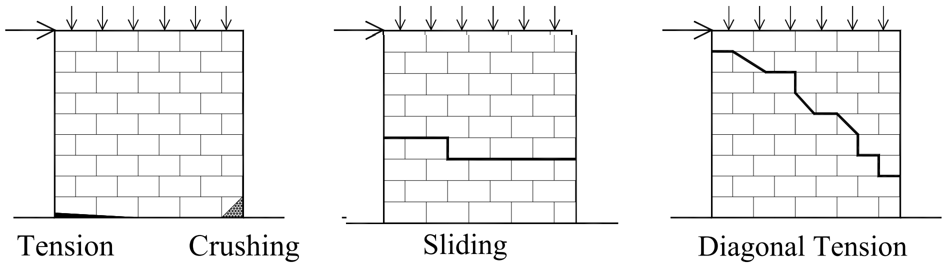

Figure 2: Typical failure modes for in-plane shear failure of masonry

For further information on the load-carrying behaviour of masonry walls subjected to in-plane shear see [8] and [9].

ANALYSED WALL

The wall analysed consists of hollow concrete blocks with dimensions and properties chosen to make shear failure become dominant. The hollow blocks are fully grouted to achieve full bond with the vertical reinforcement which prevents flexural and sliding failure. Horizontal reinforcement is assumed to be placed in every second bed joint, at a spacing of 400 mm. The axial load applied to the walls was determined assuming a 20 cm reinforced concrete slab spanning over 5 m and supporting dead and live loads of 4.8 kPa each. This results in an axial load of 12.5 kN/m dead load and 12.5 kN/m live load applied to the wall. The wall is designed to reach its full diagonal shear capacity at a horizontal load of 135kN.

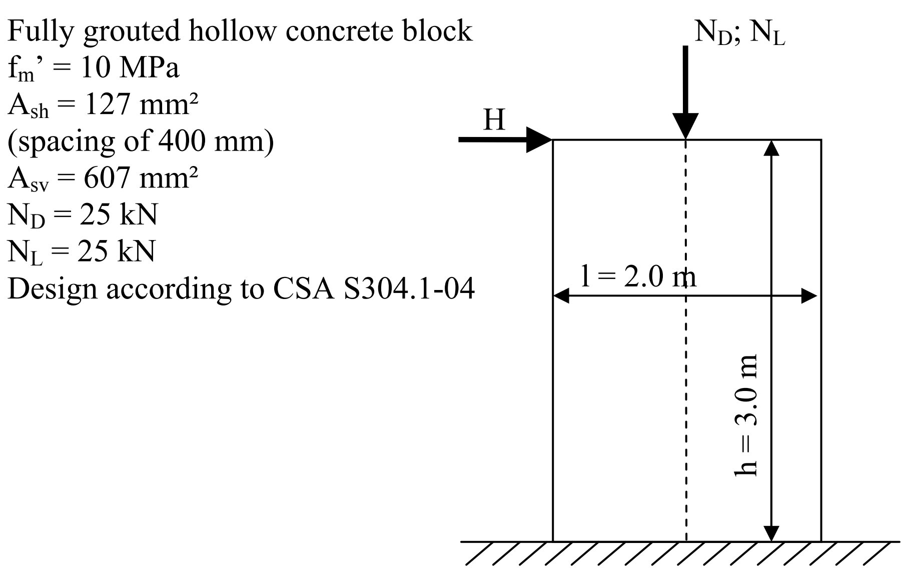

Figure 3: Analysed wall panel

CHOICE OF SHEAR MODEL

Every model is uncertain and so there has to be a random variable in the reliability analysis taking into account the deviations of the prediction model. This basic variable is referred to as model uncertainty on the resistance. To obtain realistic results in the reliability analysis, the most realistic model has to be chosen. In order to find an appropriate model, i.e. the one which gives the least uncertain prediction, several prediction models for the shear capacity of unreinforced masonry panels were evaluated by comparison with test data. Here, models from several international standards and some scientific models were analysed.

The Canadian masonry standard, CSA S304.1-04, [10] determines the shear strength based on all three modes of failure: flexure, diagonal, and sliding. The US standard, ACI 530-08/TMS 402-08/ASCE 5-08, [11] is the most similar to the Canadian standard. The only difference is that the US standard only includes provisions for sliding shear for reinforced sections for autoclaved aerated concrete masonry. The Australian standard, AS 3700-2004, [12] is more simplistic and bases the masonry shear stress solely on the aspect ratio of the wall. It accounts for neither axial load nor for sliding shear failure. The New Zealand standard, NZS 4230-2004, [13] is one of the more complex models. This model accounts for the shear resistance provided by the masonry, reinforcement, and axial load. Eurocode 6 [14] also accounts for the shear resistance provided by these three sources, but does not account for sliding failure. Anderson and Priestley [15] proposed yet another shear model after a review of the equations proposed by Shing et al. [16] and Matsumura [17]. The equation was developed using statistical data fitting. It accounts for the degradation of shear strength when the wall is subjected to cyclic loading into the inelastic range. This equation also accounts for all three types of shear failure. The National Earthquake Hazards Reduction Program (NEHRP) developed an equation similar to that of Anderson and Priestley, but with an additional factor, M/(VL), to account for the wall aspect ratio. Finally, [18] developed the most recent shear model. This model is based on that of NEHRP, but with some modifications. The masonry shear resistance includes a parameter to account for longitudinal steel. The shear resistance provided by axial loading was modified to account for a compression strut at an angle to the wall axis. This results in a greater contribution from squat walls than provided by slender walls. For shear resistance provided by reinforcement, the effective depth was reduced due to the assumption that reinforcement at the ends of the wall is not developing.

Evaluation of the test-to-prediction ratio gives the stochastic parameters presented in Table 2. The database consists of test results from walls with large reinforcement ratios. It is found that the model according to Anderson and Priestley [15] gives the best prediction due to the small CoV and was therefore chosen for further study.

Table 2: Stochastic parameters for $V_{test}/V_{prediction}$ for various models

	CSA S304.1-04	ACI 530-08	AS 3700-2001	NZS 4230-2004	EC 6	Anderson & Priestley	NEHRP (1997)	Voon & Ingham
St. Dev.	0.44	0.40	0.68	0.36	0.39	**0.16**	0.38	0.30
Mean	2.05	1.65	1.64	1.60	1.39	**0.97**	1.45	1.32
CoV	0.22	0.25	0.41	0.23	0.28	**0.17**	0.26	0.23

STOCHASTIC MODEL

Every basic variable in the chosen model has to be represented by a stochastic model. Commonly, stochastic data requires large databases. Since probabilistic analysis is gaining more and more acceptance within the construction industry, the number of stochastic models that can be found in the literature increases. In particular, values for material strength can be found more easily since strength tests are part of the quality control of producers. However, some values such as cohesion are difficult to find due to lack of data.

The reliability analysis was executed using the following limit state function for shear failure:

$$g(x) = R - E = \Theta_{R,s} \cdot \left(0.24 \cdot \sqrt{f'_m} \cdot l \cdot t + 0.25 \cdot (N_D + N_L) + 0.5 \cdot A_{sh} \cdot f_s \cdot \frac{0.8 \cdot l}{s} \right) - \Theta_E \cdot H \qquad (4)$$

where $\Theta_{R,s}$ is the model uncertainty for shear resistance, f'_m is the masonry compressive strength, N_D is axial force due to dead load, N_L is axial force due to live load, A_{sh} is the cross-sectional area of horizontal reinforcement, f_s is the yield strength of reinforcing steel, l is wall length, t is wall thickness, s is spacing of horizontal reinforcement, H is horizontal (wind) load and Θ_E is the model uncertainty for load effects.

The model uncertainties are always major parameters in a reliability analysis. Here, the model uncertainty for shear resistance $\Theta_{R,s}$ has been chosen on the basis of test data (see previous paragraph). The model uncertainty for the load effect Θ_E is chosen according to the JCSS Probabilistic Model Code [19].

The horizontal load H is modelled by the *Gumbel* distribution. This is the extreme value distribution of the group of exponential distributions, which includes the *Gaussian* distribution, and therefore the *Gumbel* distribution is widely used in probabilistic design. Additionally, it allows different observation periods T to be considered due to its properties concerning variability over time. It was found that while the mean increases the standard deviation stays the same for different observation periods. This leads to decreasing CoV (CoV is the ratio of standard deviation to the mean) for longer observation periods. Furthermore, the 98%-quantile of a 1-year observation is close to equal to the mean of the 50-year observation, as seen in Figure 4. Many codes give the unfactored wind load as the one occurring once in 50 years. This is taken as the basis for the analysis and this wind load is converted to the wind load for a 1-year observation.

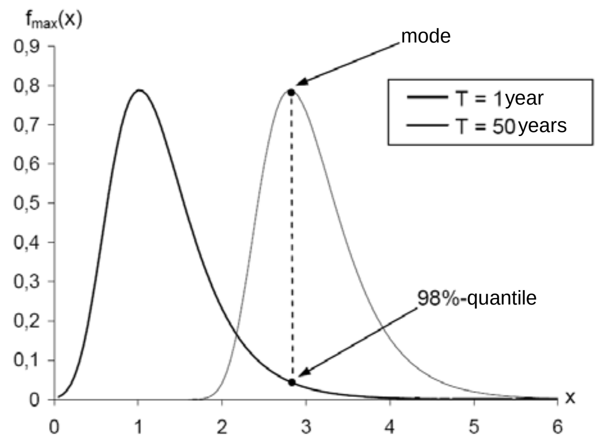

Figure 4: Gumbel distribution for different observation periods [1]

The mean values for dead and live load are also chosen to represent the characteristic values according to current codes on the basis of the distributions and coefficients of variation (*CoV*) given in Table 3.

The stochastic model includes four kinds of probability distributions: the aforementioned *Gumbel*, and the *Gaussian*, *lognormal and Gamma* distributions. The Gaussian distribution allows for negative and positive values and is widely used within every field of engineering. However, negative values for material strength and other parameters are not logical. Therefore, the *lognormal* distribution which only permits positive values is used for material strengths and model uncertainties. The *Gamma* distribution is used to determine the point-in-time value of the live load and so to account for favourable effects

Table 3: Stochastic Model Parameters

Basic variable	Type of distribution	Mean	CoV
$\Theta_{R,s}$	lognormal	0.97 [-]	17 %
Θ_E	lognormal	1.0 [-]	10 %
H	Gumbel	0.075	31 %
f_s	lognormal	430 MN/m²	4 %
f_m	lognormal	14.1 MN/m²	20 %
N_G	Gaussian	0.025 MN	3 %
N_Q	Gamma	0.063 MN	100 %
l	constant	2.0 m	-
t	constant	0.19 m	-
h	constant	3.0 m	-
A_{sh}	constant	127.0 mm² a)	-
A_{sv}	constant	607.0 mm²	-
a) spacing of 400 mm			

The parameters have been estimated on the basis of [19] and [20]. For the parameters of the distribution of the masonry compressive strength refer to [1] and [22].

RESULTS AND ASSESSMENT

The reliability analysis was performed using the Software COMREL [23] which includes various procedures. For this analysis, SORM was used.

The governing parameter of a masonry wall subjected to in-plane shear is the wall length l_w. In Figure 5, the reliability index β is shown versus the wall length. It can be seen, that the reliability increases non-linearly with growing wall length. Additionally, it can also be seen that the wall length of 2.0 m is enough to provide sufficient reliability since the target reliabilities according to DIN 1055-100 and JCSS are exceeded for l = 2.0 m. To reach the target reliabilities, wall lengths of 1.64 m (JCSS) and 1.98 m (DIN 1055-100) are sufficient. Further research and development of the prediction models therefore might lead to economic enhancement of the walls.

Flexural failure was also included in the analysis but did not dominate due to the design of the wall. The analysis was executed for an observation period of one year.

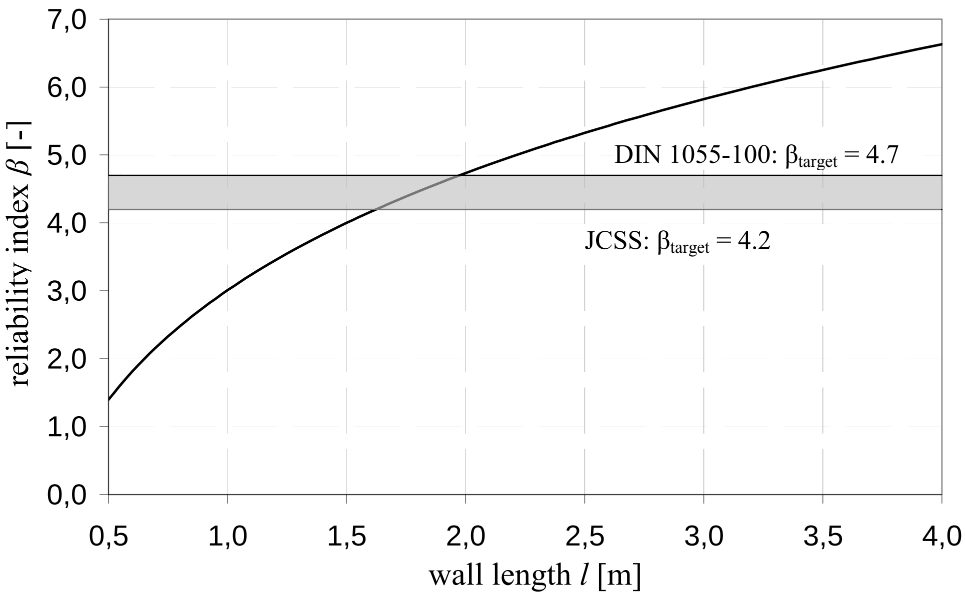

Figure 5: Reliability index β vs. wall length

Another outcome of the reliability analysis is determination of the sensitivity values α_i. These values define the design value of a basic variable together with the reliability value β and the standard deviation σ. The sensitivity values can be seen as a measure for the influence of a basic variable on the reliability; the larger the sensitivity, the larger the influence. As expected, the horizontal load H is the parameter with the largest influence. DIN 1055-100 gives an estimate for the sensitivity of load effects of $\alpha_E = 0.8$. The obtained value meets this recommendation. The model uncertainties have the second largest influence which underlines the importance of precise prediction models.

Table 4: Sensitivity values for H_k = 135 kN

Basic variable	Sensitivity value α
$\Theta_{R,s}$	0.44
Θ_E	-0.27
H	-0.83
f_s	0.03
f_m	0.19
N_D	0.02
N_L	0.04

Additionally, the values α_i can be used to calculate the partial safety factors. These are defined as follows:

$$\gamma_E = \frac{E_d}{E_k} \tag{5}$$

$$\gamma_R = \frac{R_k}{R_d} \tag{6}$$

where E_d is the factored load effect, E_k is the corresponding characteristic value, R_d is the factored resistance and R_k is the characteristic value of the resistance.

The characteristic values represent quantiles of the respective probability distribution of a basic variable. The design values can be determined depending on the sensitivity values, the target reliability, mean and standard deviation of every basic variable. Since every distribution requires different equations, refer to [20] for further information.

The obtained partial safety factors can be taken from Table 5. Compared to the recommendations of CSA S304.1-04, it can be seen that the required factor on the horizontal load to reach the target reliability according to the JCSS [6] is larger than the recommendation. The fact that the required partial safety factor on the live load is supposed to be larger, has often been examined in reliability analysis, see [1] and [24]. However, the favourable effect of the live load and the larger safety factor on the masonry strength equalize the effects of the small safety factor on the horizontal load.

Table 5: Partial Safety Factors for β_{target} = 4.2

Basic variable	Partial Safety Factor	Recommendation of CSA S304.1-04
H	1.78	1.50
f_m	1.25	1.667
f_s	1.26	1.176
N_D	0.75	0.90

Note that the live load is not to be applied in design since it is acting favourably. However, it is included in the reliability analysis to account for the permanently existent part of the live load. So, neglecting the live load in design is an additional element of safety.

CONCLUSION

In this paper, the reliability of a reinforced masonry shear wall was analysed. Therefore, several models were compared to test data to find an appropriate theoretical model for analysis. The model of Anderson & Priestley [15] was found to provide the best match to the experimental data available. A stochastic model was set up and the analysis was performed by application of SORM. The reliability index β was computed and compared to recommended values. It was found that the wall provides sufficient reliability so that further optimization is possible. Partial safety factors were provided and compared to the recommendations of the CSA.

REFERENCES

1. Glowienka, S. (2007) “Zuverlässigkeit von Mauerwerkswänden aus großformatigen Steinen”, doctoral thesis, Institut für Massivbau, Technische Universität Darmstadt, Germany, in German
2. Melchers, R. E. (1999) “Strutural Reliability Analysis and Prediction”, John Wiley & Sons Ltd., Chichester, England
3. Benjamin, J. R. and Cornell, C. A. (1970) “Probability, Statistics and Decisions for Civil Engineers“ McGraw-Hill Book Company, New York
4. Rackwitz, R. (2004) „Zuverlässigkeit und Lasten im konstruktiven Ingenieurbau“, TU Munich, Munich, Germany
5. Rosenblueth, E. and Mendoza, E. (1971) “Optimization in Isostatic Structures” Journal of the Eng. Mech. Division, ASCE, 97, EM6
6. JCSS (2001) “Probabilistic Assessment of Existing Structures”, RILEM Publications, S.A.R.L.
7. DIN 1055-100 (2001) “Einwirkungen auf Tragwerke - Teil 100: Grundlagen der Tragwerksplanung - Sicherheitskonzept und Bemessungsregeln”, Beuth Verlag, Berlin
8. Kranzler, T. (2008) “Tragfähigkeit überwiegend horizontal beanspruchter Aussteifungsscheiben aus unbewehrtem Mauerwerk”, doctoral thesis, Institut für Massivbau, Technische Universität Darmstadt, Darmstadt, Germany
9. Lissel, S.L., Shrive, N.G., and Page, A.W. (2000) “Shear in Plain, Bed Joint Reinforced and Post-tensioned Masonry” Canadian Journal of Civil Engineering, Vol. 27, pp. 1021-1030
10. CSA S304.1-04 (2004) “Design of Masonry Structures,” Mississauga, ON.
11. TMS 402-08/ACI 530-08/ASCE 5-08 (2008) “Building Code Requirements for Masonry Structures,” Boulder, CO
12. AS 3700-2001 (2001) “Masonry Structures,” Sydney, Australia
13. NZS 4230:2004 (2004) “Design of Reinforced Concrete Masonry Structures,” Wellington, NZ
14. Eurocode 6 (2003) “Design of Masonry Structures,” Brussels
15. Anderson, D.L. and Priestley, M.J.N. (1992) “In Plane Shear Strength of Masonry Walls,” 6th Canadian Masonry Symposium, Saskatoon, SK
16. Shing, P.B., Schuller, M., Hoskere, V.S., and Carter, E. (1990) “Flexural and Shear Response of Masonry Walls”, ACI Structural Jounal, Vol 87 No 6, Nov.-Dec.
17. Matsumura, A. (1987) “Shear Strength of Reinforced Hollow Unit Masonry Walls”, Proceedings, 4th North American Masonry Conference, Los Angeles, CA.
18. Voon, K.C., and Ingham, J.M. (2007) “Design Expression for the In-Plane Shear Strength of Reinforced Concrete Masonry,” Journal of Structural Engineering, Vol. 133, pp. 706-713

19. JCSS (2003), "Probabilistic Model Code", online available at www.jcss.ethz.ch
20. Spaethe, G. (1992) "Die Sicherheit tragender Baukonstruktionen" Springer, Vienna
21. Gumbel, E. J. (1958) "Statistics of Extremes" Columbia University Press, New York
22. Schueremans, L. (2001) "Probabilistic evaluation of structural unreinforced masonry", doctoral thesis, Catholic university of Leuven, Leuven, Belgium
23. RCP (2004) "STRUREL, a Structural Reliability analysis program system" RCP GmbH, Munich
24. Graubner, C.-A., Brehm, E. and Glowienka, S. (2008) "Economic Potentials of Probabilistic Optimization Methods", Beton- und Stahlbetonbau, Volume 103, ISSN 0005-9900, Ernst & Sohn, Berlin, in English