Codes, guidelines and methods for safeguarding safety and significance

Structural Analysis of Historic Construction – D'Ayala & Fodde (eds)
© 2008 Taylor & Francis Group, London, ISBN 978-0-415-46872-5

Use of reliability methods for evaluating safety of historic structures

L. Schueremans & E. Verstrynge
Civil Engineering Department, KULeuven, Heverlee, Belgium

ABSTRACT: The architectural preservation process is generally based on a sequence of anamnesis and analysis, diagnosis, therapy, control and prognosis. In the analysis phase, an objective way to assess the safety of the structure is essential. The outline of the assessment process is described both in the ISCARSAH Recommendations and ISO13822, referring to determining the actual safety or reliability of existing (historical) structures. This paper focuses on the use of reliability based assessment techniques, using a probabilistic analysis accounting for the uncertainty within the analysis in an objective manner. As a result the user obtains a value for the actual reliability or failure probability. Besides a general description of its possibilities within the analysis of historical structures, an application is treated to illustrate the methodology as well as its limitations.

1 INTRODUCTION

Safety, reliability and risk are key issues in the preservation of our built, cultural heritage.

Both ISCARSAH Recommendations (ISCARSAH, 2005) and ISO13822 cover the assessment of existing structures. Within a paragraph of the latter, it is indicated that the standard is also applicable toward historical structures, "*provided additional considerations are taken into account concerning the preservation of the historical appearance of the structure and the preservation of its historical material*". The ISCARSAH recommendations are set align with ISO13822. Vice versa, it is proposed to add a Historical Structure Annex to the standard to provide additional considerations to the application of the ISO13822 standard to historical structures. Besides several other items, this annex could address the special characteristics of historical structures, including elements related to structural analysis and target reliability level.

In addition, this annex would also elaborate more on clause 8.1 of ISO13822 "*Assessment of Safety*", taking into account the longer life experience of many of these historical structures and the integration of all of the various components of the structural evaluation taking into account the complexity of history and the more extensive uncertainties that exist with older structures.

Structural evaluation is one of the determining factors in the preservation and use of the built cultural heritage. A conservative evaluation might lead to an increased level of intervention of the structure and therefore result in a loss or major alteration of its authenticity. In addition, the excessive scope of intervention can add unnecessary cost and compromise the viability of the conservation project.

The present raises the need for a reliability based assessment of existing structures. Powerful methods are available for the calculation of structural safety values. These permit to calculate the global probability of failure of complex structures, relying on deterministic techniques able to calculate the stability state for a prescribed set of parameters (ISO2394, 1998).

This paper illustrates the overall framework and motivates how these techniques can be a valid tool to evaluate the safety of existing historical structures.

Reference is made to adjacent research fields and practice in which the techniques are (more) widely spread and used, their advantages and disadvantages as well as the challenges that still need to be met for its general applicability within the field of historical structures.

Special attention goes to available reliability algorithms, (commercial) software available and their requirements, links with generic finite element codes covering the structural analysis, target failure probabilities for historical buildings and material properties.

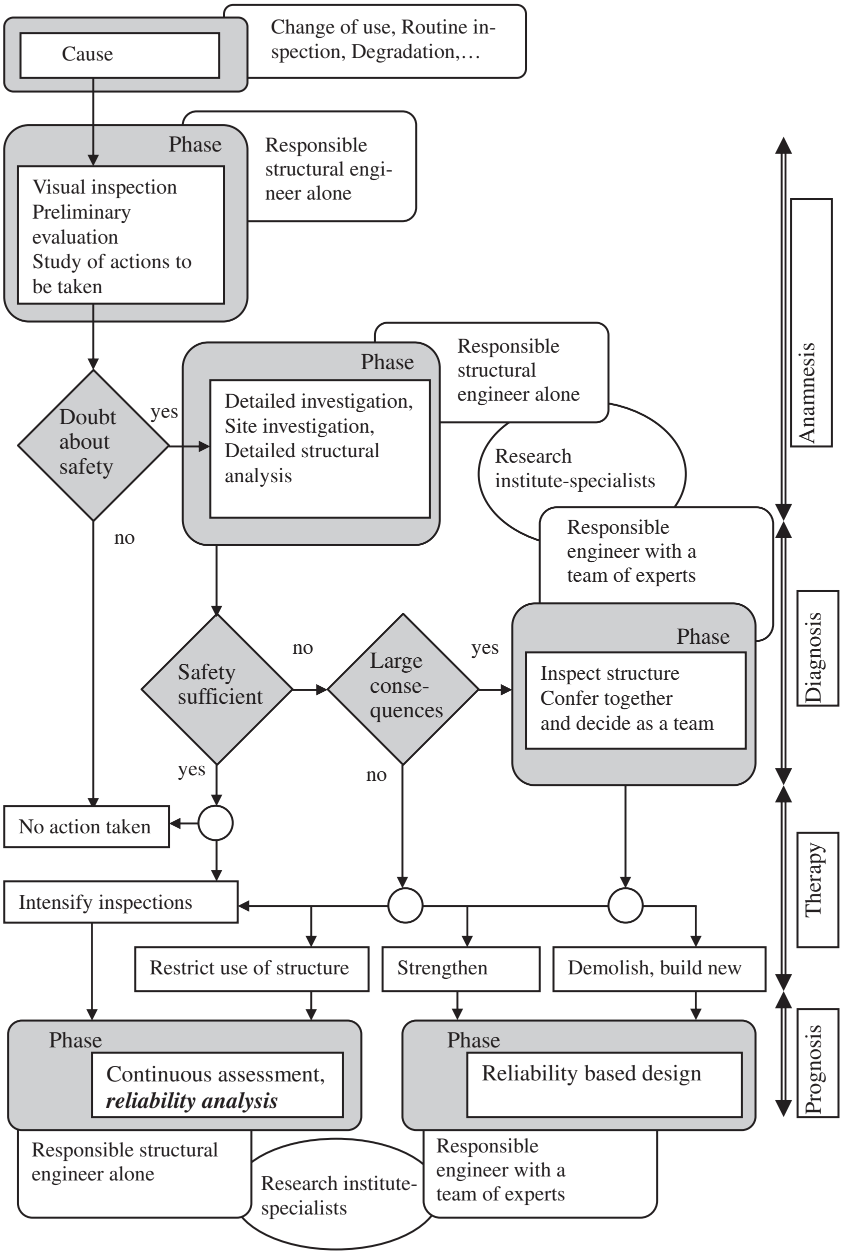

Figure 1. Preservation process (Schueremans, 2001).

2 PRESERVATION PROCESS

The preservation process is outlined within both the ISCARSAH Recommendations and schematically in Annex B of ISO13822. A similar layout, focusing on the different phases within the process (a sequence of anamnesis and analysis, diagnosis, therapy, control and prognosis), is given in Figure 1. In all three flow charts, it is in the detailed analysis phase that the reliability question is put forward. Is there a need for intervention,

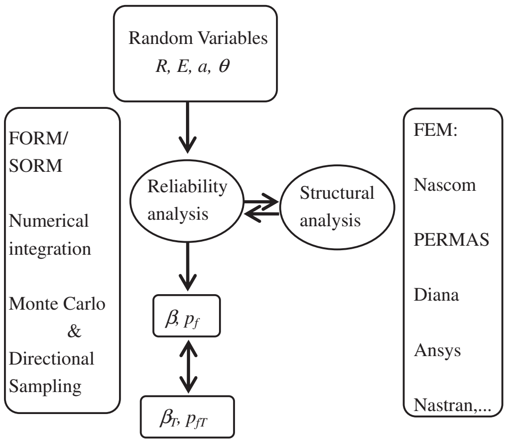

Figure 2. Reliability framework – requirements.

is the actual reliability sufficient? It is in this detailed analysis phase that a reliability based analysis delivers added value.

3 RELIABILITY BASED ASSESSMENT

3.1 *Framework*

To be able to make a probabilistic design or assessment according to a preset target failure probability, following requirements need to be fulfilled, Figure 2:

– accurate reliability algorithms in a user friendly environment;
– knowledge of random variables for the design variables (R: resistance, E: sollicitions, a: geometry, θ: model uncertainty) and availability of materials models and related data;
– appropriate structural models;
– preset target failure probabilities ($p_{f,T}$) or corresponding target reliability values (β_T).

3.2 *Target failure probabilities* $p_{f,T}$

For the decision process that follows up on a safety assessment of an existing structure, it is important to define target safety levels that can serve as decision criteria. In relation to fulfilling such criteria, existing (historical) structures will differ from new structures at the design stage. Furthermore, determining the target probability of failure is not a technical problem solely. Whether or not an historical building should meet the target probability of failure value $p_{fT} = 5.10^{-4}$ according to Eurocode (EN1990, 2002), is subject of discussion. Several authors suggest widening the discussion and proposing a differentiation with respect to various parameters. An overview is given elsewhere (Schueremans, 2001):

– Number of people put to danger (n_p);
– the type of possible damage (life injury, economical damage, social-cultural damage, environmental damage);
– the preset service life of the construction (t_L);
the level in which people are exposed to risk (public buildings, bridges, off-shore constructions, ...) and;
– the level in which people are warned beforehand and can be put to safety (gradual failure with visible damage against sudden collapse without warning) (W).

$$p_{f,T} = \frac{10^{-4} S_c t_L A_c C_f}{n_p W} = \frac{10^{-4} \times 0.005 \times 100 \times 1.0 \times 1}{10 \times 0.3} = 1.67\ 10^{-5} \quad (1)$$

The formula proposed in eq. 1 to determine nominal target failure probabilities is a mix of proposals presented by different authors. It is very suitable as it accounts for a social criterion (S_c) that can be reinterpreted to encapsulate the importance of historical buildings or the preservation value. This is indicated in Table 1 (in Italic).

3.3 *Reliability algorithms and (commercial) software*

The generalized reliability problem can be outlined starting from the basic reliability problem. The basic reliability problem considers only one load (E) and one resistance (R). Starting point is that both the load effect (E) as the resistance (R) are random variables. Each is described by a known probability density function: $f_E(e)$ and $f_R(r)$. In general, both the load effect (E) and the resistance (R) are a function of time t. The safety limit state will be violated when at a certain moment in time, $R(t) - E(t) < 0$. The chance or probability that this will happen equals the probability of failure p_f. As both R and E are a function of time, p_f is also a function of time. Because of mathematical complexity, the probability density functions are transformed into time-invariant probability density functions. As a consequence, the reliability analysis is performed for a preset reference period or so-called design service life t_L:

$$p_f = P[R - S < 0],\ \textit{for a reference period}\ t_L \quad (2)$$

Eq. (2) is generalized as follows:

$$p_f = P[g(R,S) < 0] = \int_{g(R,S)<0}\int f_{R,S}(r,s)\,drds,\ \textit{for}\ t_L \quad (3)$$

in which $g(R, E)$ is called the limit state function (LSF); the probability of failure is identical with the

Table 1. Factors influencing the nominal target probability of failure (used values in bold).

t_L: residual service life [years]:	**100**
n_p: number of lives put to danger:	**10**
Economical factor	C_f
not serious	10
serious	**1**
very serious	0.1
Warning factor	W
Fail-Safe condition	0.01
Gradual failure with some warning likely	0.1
Gradual failure hidden from view	**0.3**
Sudden failure without previous warning	1.0
Activity factor	A_c
Post-disaster activity	0.3
Normal activities:	**1.0**
Buildings or Bridges	3.0
High exposure structures (offshore)	10.0
Social criterion factor (Preservation value)	S_c
Places of public assembly, dams = *historical buildings of great importance for mankind, listed by UNESCO e.g.*	**0.005**
Domestic buildings, offices, trade buildings, industrial buildings = *listed historical buildings of national importance*	0.05
Bridges = *listed historical buildings of regional importance*	0.5
Towers, masts, off-shore structures = *not-listed historical buildings*	5.0

probability of limit state violation and $f_{R,E}(r, e)$ is the joint probability density function.

This limit state function defines 3 different regions:

$$g(R,S) > 0, \textit{ the safe region,}$$
$$g(R,S) = 0, \textit{ the critical situation and} \quad (4)$$
$$g(R,S) < 0, \textit{ the unsafe region.}$$

Adding the other sources of uncertainty (a: geometry; θ: model uncertainty) eq.4 can be further generalized into:

$$p_f = P[g(R,S,a,\theta) < 0] = \int_{g(R,S,a,\theta)<0} \cdots \int f_{R,S,a,\theta}(r,s,a,\theta)\,dr\,ds\,da\,d\theta, \textit{ for } t_L \quad (5)$$

Whatever numerical method is used, several evaluations of the limit state function ($g()$) are required

Table 2. Development of integrated software for structural reliability analysis.

Name of software tool	Structural analysis-FEM	Reliability analysis method MC	FORM/SORM	Use of RS
NASREL	NASCOM		•	
COMREL	/	•	•	
SYSREL	/	•	•	
PERMAS-RA	PERMAS	•	•	•
PROBAB	DIANA	•	•	•
NESSUS	ABAQUS	•	•	•
SBRA	ANSYS,... included	•		
OPTIMUS	NASTRAN	•	•	

Legend: FORM: First Order Reliability Method; SORM: Second Order Reliability Method.

to quantify an accurate system failure probability of complex structures with a large number of random variables. The most obvious techniques are based on simulation procedures, such as Monte Carlo (*MC*) or Directional Sampling (*DS*). Main disadvantage of these procedures is the large number of samples and thus calls to the limit state function and thus finite element model to come up with an accurate value of the failure probability. To meet this disadvantage, use is made of the Response Surface (*RS*) technique. This might be a simple low order polynomial response surface, but, in case of complex structural behavior, more universal response surfaces, such as Neural Networks, Splines or Kriging prove to be beneficial (Schueremans & Van Gemert, 2005). It is clear that an integrated communication in between the structural analysis and the reliability algorithm are a prerequisite to enable probabilistic design or assessment for real applications.

Several research programs have led to the development of integrated software. Some of the combinations in between the structural model (most often a finite element code) and probabilistic algorithms are listed in Table 2. These developments reply to the demand of integrated and user-friendly software applications. For example the probabilistic shell around DIANA is particularly suitable, because of the available numerical constitutive relations for masonry available (Lourenço, 1996).

3.4 *Advantages and drawback – reliability based assessment*

Major advantages of the methodology are:

– An objective value for the resulting failure probability (p_f) is obtained, accounting for the present uncertainty in an unbiased manner;

– Sensitivity coefficients (α_i) are obtained for all random variables included within the reliability analysis. These sensitivity coefficients directly reflect the importance of a specific parameter within the analysis. And therefore, these parameters are to be considered with priority when gathering more accurate information or designing a structural intervention procedure;
– The impact of structural interventions can be assessed in a pre-design phase in a similar manner, allowing different alternatives to be judged in an objective manner.

The major drawbacks are:

– For complex structures, an automatic interface is required with an accurate unbiased structural model of the structure. Often, this will be a finite element model of the structure, in which use is made of typical non-linear constitutive relations to simulate the real structural behavior. This interface is not available for all (generic) finite elements codes or numerical tools used for assessment at this moment;
– The outcome of the reliability analysis thus is related to the accuracy of the underlying structural model. Model uncertainty is often added to cover the uncertainty within the accuracy of the numerical model;
– The outcome of the reliability analysis is directly related to the accuracy of the probabilistic models of the random variables involved. Often, the number of data available is (too) limited to establish reliable probabilistic load or material models;
– The processing time is related to the complexity of the problem, the shape of the limit state function, the resulting reliability and the number of random variables included within the analysis. Processing time increases in cases of discontinuous limit state functions with strong curvatures and large dimensional problems having a small failure probability. This however is often the case in historical structures.

3.5 *Probabilistic models for random variables*

The reliability based assessment requires probabilistic models for the different random variables involved within the analysis. Certainly the Resistance variables (R) pose difficulties within the context of historical structures. These are related to the probabilistic modeling of material properties, required within the constitutive modeling.

The joint committee of Structural Safety has set up several workgroups who take action to establish probabilistic material models. For several construction materials, models have been established, for example: steel (2002), concrete (2002), soil and timber (2006). For masonry, and more specifically historical masonry with its irregular possibly multi-leaf layout, no probabilistic material model is available at this moment. Some preliminary proposals have been made by few authors (Schueremans and Van Gemert, 2001, 2007; Proske et al. 2006).

For the load effect, an international consent is available on the probability density functions to be used. A non-limitative reference list of probability density functions for the main design variables is given in Table 3 (Diamantidis, 1999, 2001; JCSS, 2006).

Table 3. Reference probability density functions for the main design variables.

Name of variable	Symbol	Dim.	PDF	Mean value μ_x	Standard deviation σ_x
Action Effects (E)					
Permanent	G	kN/m^2	Normal	G_k	$0.10\mu_X$
Imposed – 5 year	Q	kN/m^2	Gumbel	$0.2\,Q_k$	$1.10\mu_X$
Imposed – 50 year	Q	kN/m^2	Gumbel	$0.6\,Q_k$	$0.35\mu_X$
Wind – 1 year	W	kN/m^2	Gumbel	$0.5\,W_k$	$0.40\mu_X$
Wind – 50 year	W	kN/m^2	Gumbel	$0.7\,W_k$	$0.25\mu_X$
Resistance (R)					
Steel	R	kN/m^2	Lognormal	$R_k + 2\sigma_R$	$0.08\mu_R$
Timber	R_m	kN/m^2	Lognormal	$R_{m,0}$	$0.25\mu_R$
Concrete	f_c	kN/m^2	Lognormal	$f_{c,0}$	0.06–$0.30\mu_R$
Masonry	f_c	kN/m^2	Lognormal	/	/
Geometry (a)					
Geometry	a	/	Normal	a_0	Variable
Model uncertainty (θ)					
On action	θ_E	/	Normal	1.00	0.10
On resistance	θ_R	/	Normal	1.10	0.07

3.6 *Developments in adjacent fields of research/application*

Probabilistic techniques are not common use in daily design or assessment. The methodology is mainly used when the complexity of the problem is beyond the calibration domain of current codes, when the safety of the structure is beyond the traditional design or when the consequences of failure are large. Remark that these boundary conditions often apply for historical buildings.

To put the methodology in a broader reference frame, several examples of various application domains demonstrate its applicability in design (T2881, 1999; Diamantidis, 2001): design of ships and

Figure 3. Global overview of the critical part A – city side.

off-shore structures; design of tunnel segments (TNO, Delft, Nl); safety assessment of the London Eye (UK, TNO Bouw); risc analysis of the derailment guidance for high speed line (Netherlands, Simtech); Risc-analysis Jamuna Bridge (Haskoning, Bangladesh), service life prediction LNG Terminal (Dabhol, India, KULeuven); infrastructure dams such as Dijkring or Oosterschelde-stormvloedkering (Bouwdienst Rijkswaterstaat, Nl).

4 APPLICATION

Possible applications are multiple. Up till now, applications cover mainly structural components or structures that can be simulated with relatively simple models (with respect to: structural models, number of random variables covering the problem), or complex structures are reduced to models with acceptable complexity (Schueremans et al., 2001; Schueremans & Van Gemert, 2004).

The example treated, clearly illustrates the added value of the application of this type of analysis both in the assessment of the existing historical structure and in the pre-design phase of possible interventions.

4.1 *Romanesque city wall of Leuven*

The Romanesque city wall of Leuven (B) dates from 1150. The medieval wall is in a severe state of decay.

The part studied in this paper is a piece of the edifice between the former Biest- and Minneporte (2 gates). This part of the Romanesque city wall near the river Dijle has a total length of about 150 m and comprises 2 towers.

This example only covers the most critical part of the wall, Figure 3. A global view of the structural layout of the wall is given in Figure 4. The wall consists of an inner or city side (B) and an outer or rural side (A). Round foundation arches of 2 m high and 3.5 to 4 m wide carry the outer wall. The inner wall is a continuous arcade of 4 m wide round arches with their tops 3 m above the outer arches. A walkway of 0.90 m width (C) is present on top of the arcades. The outer wall was equipped with shooting holes centred in the arches of the inner part (D) and crowned by a parapet also bearing shooting holes.

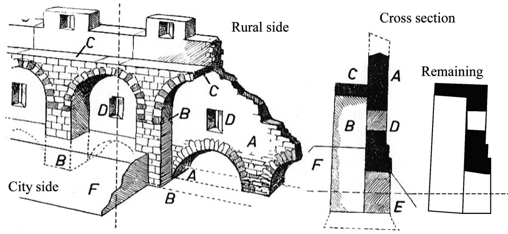

Figure 4. Reconstruction of structural layout of city wall.

These are no longer present. On the city side as well as on the rural side, the wall was lined with a sloped embankment (F, E) covering the foundation pillars and arches. This embankment is no longer present at the rural side, and only partly at the city side. For the construction of the wall, a local type of lime-sandstone (Diegemse Zandsteen) was used. An iron containing sandstone (Diestiaan sandstone) was mainly used for decorative purposes, for example in the inner arcades.

4.2 *Safety assessment*

A single arcade – the repetitive structural element - is taken as an individual control unit. To clarify the benefit of using a reliability based assessment, different levels to assess the safety are used:

- Level 0: To obtain a measure for the remaining safety margin, the structure is checked using nominal values for the applied loads and resistances. The resistance-load ratio (r) is used as a measure for the remaining safety margin with respect to a certain ultimate limit state;
- Level I: The analysis is performed according to the partial safety factor method;
- Level III: The analysis is performed using probabilistic evaluation algorithms based on sampling techniques. An accurate value for the system reliability is obtained. The same limit state functions that are used for the Level I analysis, are accounted for.

The probability density function and the parameters for the different random variables are listed in Table 4. These cover the different types of uncertainties encountered during the survey: accuracy of the geometry, the uncertainty on the material properties (subsoil and stone masonry), actual loads and future loads.

Following limit states are checked, Table 5, (definition of symbols, see Figure 5):

- Rotational-equilibrium. The centre of gravity is determined ($y_{g,tot}$), accounting for the structural geometry and the slant of the wall. As long as the value is positive, the centre of gravity remains within the cross section, thus the rotation limit state

Table 4. Romanesque city wall – Random variables.

Random variable	PDF	Mean value	Std dev.
Initial assessment – n = 23			
Load:			
ρ_m [kN/m^3]:density of masonry	N	19	1.9
Geometry (n = 17)			
Geometry of wall	N	nom	0.05
Resistance of subsoil:			
c [kN/m^2]: cohesion	LN	30.11	15.29
φ [°]: friction coefficient	N	25.56	4.59
γ_{dr} [kN/m^3]: dry density	N	16	1.6
Resistance:			
f'_m: stone masonry strength	LN	23.2	4.3
Uncertainty:			
ε [m]: model uncertainty	N	0.0	0.01
Strengthening with reinforced concrete foundation slab – additional random variables: n = 23 + 5			
Load:			
P [kN]: permanent load	N	40	4
q [kN/m^2]: floor load	LN	5	2
$\boldsymbol{\rho}_b$ [kN/m^3]: proper weight	N	25	1
Geometry (reinforced concrete slab):			
l_{slab} [m]: length of slab	N	4	0.05
w_{slab} [m]: width of slab	N	4	0.05

Legend: N: Gaussian; LN: LogNormal: PDF: probability density function.

Table 5. Results of structural reliability – original situation.

LSF// assessment level	Rotation equilibrium [m]	Stresses in the masonry [N/mm^2]	Stresses in the subsoil [N/mm^2]
Level 0		$\sigma_{m,max} = 0.74 <$ $f_{c,m} = 23.2$ **r = 30.5**	$\sigma_{gr,max,pl} = 0.50 <$ $d_g = 0.52$ **r = 1.04**
Level I	$y_{g,tot} = 0.35 > 0$ (OK) $e_{tot} = 0.53 >$ d/6 = 0.29 **NOT OK**	$\sigma_{m,max,d} = 1.00 <$ $f_{c,m,d} = 16.2$ OK	$\sigma_{gr,max,el,d} =$ 0.90 > dg,d = 0.19 **NOT OK**
Level III	System reliability $\beta = \mathbf{1.2} < \beta_T = \mathbf{4.2}$ Corresponding failure probability: $\mathbf{p_f = 0.11 > p_{f,T} = 1.7\ 10^{-5}}$		

is met. The eccentricity (e_{tot}) is checked. Whenever the eccentricity exceeds the mid-third ($d/6$), part of the cross-section is in tension. As use is made of a non-tension material model, the force equilibrium is met using compressive stresses only. This leads to an increased stress level;

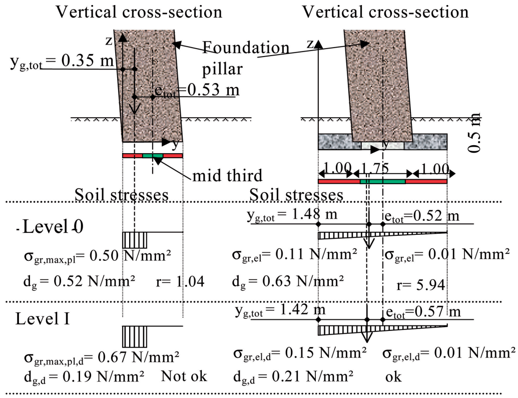

Figure 5. Stress distribution in the subsoil at foundation level – left: original situation; right: with foundation strengthening.

– Compressive stresses are calculated in the masonry ($\sigma_{m,max}$) as well as in the soil at foundation level ($\sigma_{gr,max,el}$ and $\sigma_{gr,max,pl}$). Maximum stresses are found at the foundation tip of the pillars, due to the slant of the wall. For both materials, a non-tension material model is used. For the soil, a linear-elastic (*el*) as well as an elastic-plastic (*pl*) material model is used.

The results of the safety assessment of part A of the wall are listed in Table 5. From the Level 0 and Level I analysis, it is clear that the structural stability is in doubt. The remaining safety ratio ($r = 1.04$) is limited. The limit state function of the stresses in the subsoil is violated. The probabilistic method offers an objective way to assess the remaining safety, accounting for the present uncertainties: $p_f = 0.11$. This value does not meet the preset target value.

4.3 *Strengthening – reliability based design*

The lack of safety originates from the limited load-bearing capacity of the soil in combination with the large slant of the wall. This is partly caused by the removal of the original sloped embankments at the rural and city side. At present, the top part of the foundation is above the original ground level. Thus, the depth of the foundation decreased significantly. To increase the safety to an acceptable level, two options are available:

– Widen the foundation at the support. This will reduce the soil stresses, see Figure 5;
– Increase the load-bearing capacity of the soil by restoring the original sloped embankment at rural side. The latter possibility however was not chosen by the responsible authorities.

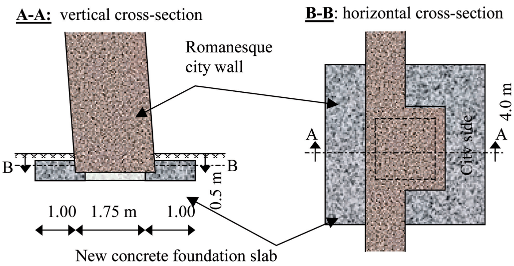

Figure 6. Foundation strengthening – new reinforced concrete foundation slab.

Table 6. Summary of structural safety for part A of the wall – strengthened situation.

LSF// assessment level	Rotation equilibrium [m]	Stresses in the masonry [N/mm^2]	Stresses in the subsoil [N/mm^2]
Level 0		$\sigma_{m,max} = 0.74 <$ $f_{c,m} = 23.2$ **r = 30.5**	$\sigma_{gr,max,pl} = 0.11 <$ $d_g = 0.52$ **r = 5.94**
Level I	$y_{g,tot} = 1.77 > 0$ (OK) $e_{tot} = 0.35 <$ $d/6 = 0.62$ **OK**	$\sigma_{m,max,d} = 1.00 <$ $f_{c,m,d} = 16.2$ OK	$\sigma_{gr,max,el,d}$ $= 0.15 >$ $dg,d = 0.21$ **OK**
Level III	System reliability $\beta = 4.8 < \beta_T = \mathbf{4.2}$ Corresponding failure probability: $\mathbf{p_f = 9.4\,10^{-7} > P_{f,T} = 1.7\,10^{-5}}$		

For this part of the wall, a strengthening of the foundation is proposed. A new concrete foundation slab will be established at the basis of the existing foundation, see Figure 6 for a schematic representation. This reduces the soil stresses at the support. The effect of these strengthening measures on the structural safety is recalculated. The results are summarized in Table 6 and visualized in Figure 5. In all cases, a sufficient safety margin is obtained.

4.4 *Discussion of analysis results*

In the analysis performed, a system reliability has been calculated referring to 3 failure modes. To do so, use was made from improved simulation techniques (Monte Carlo, Directional Sampling). The techniques and detailed outcome results are described elsewhere (Schueremans & Van Gemert, 2004). It although required on average $n_{LSFE} = 250$ limit state function evaluations. Since for this simple example an analytical expression is available, it does not require to much computational effort. For more complex structures, this might lead to large computational effort.

5 CHALLENGES

Within the analysis performed in general, one sees that:

- The use of correlated random fields describing the heterogeneity of for example the masonry walls or the layered subsoil is not often used. Although theoretically possible, its still is not generally applied within practice, also because of lack of data. As a result, mapping of surface field data on heterogeneity, weak spots, defects, is seldom included;
- The inclusion of monitoring data and Bayasian updating will reduce spread on the random variables and therefore is an undervalued tool at this moment in view of intelligent monitoring techniques development;
- Time variant/dependent analyses accounting for material degradation or the time dependent long term behavior of construction materials used is still in research phase (Verstrynge et al., 2008).

6 CONCLUSIONS

For the evaluation of the bearing capacity of existing structures, the interest in probabilistic evaluation methods is growing. The methodology is placed in a reference frame and outlined using the generalized reliability problem. The focus of the application is mainly on the structural safety that is assessed at different levels. Because of the uncertainties on geometry, soil resistance and loading, a structural evaluation is also performed based on probabilistic techniques. For the reliability analysis, use is made of simulation procedures. This is done, first to check the present safety of the wall and second to propose a consolidation and strengthening treatment. In both cases, the probabilistic evaluation method results in an accurate value of the failure probability. In combination with a preset target value, this results in an objective way to assess the safety. It is clear that more complex structures, e.g. combinations of vaults, arches and columns, will also lead to more complex formulations. As a consequence, the mathematical quantification of risk or safety will be aggravated accordingly.

In general, the tendency towards level III methods is mainly a matter of computational effort, continuous improvement of reliability algorithms, availability of material data and user-friendly software applications. Because of the increasing computational capacity and speed, probabilistic design according to a preset safety level, is within reach. On an international basis, the tendency from design (or "way of thinking") from a partial safety factor method towards a probabilistic method – reliability based design – is clearly visible.

ACKNOWLEDGEMENTS

The authors express their thanks to the Flemish Fund for Scientific Research (FWO) for the doctoral grant, offered to Els Verstrynge.

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