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Creep Behaviour of Masonry Structures – Failure Prediction based on Archeological Model and Laboratory Tests

Sven Ignoul
Triconsult NV, Industriepark 1241/Bus 4, B-3545 Halen

Luc Schueremans, Jan Tack, Lieselotte Swinnen, Sylvia Feytons
Department of Civil Engineering, KULeuven, Kasteelpark arenberg 40, B-3001 Heverlee

Luigia Binda
Politecnico di Milano, Dipartimento di Ingegneria Strutturale, Italy

Dionys Van Gemert[1], Koenraad Van Balen[1,2]
[1]Department of Civil Engineering and [2]R. Lemaire Centre for the Conservation of Historic towns and buildings, KULeuven, Kasteelpark arenberg 40, B-3001 Heverlee

ABSTRACT: Masonry, subjected to significant compressive stresses, tends to creep. This creep behaviour is modelled using a rheological model, consisting of several basic springs and dashpots placed in series or in parallel. Qualitatively, this rheological model is capable of describing the creep behaviour in a rational way. The main disadvantage of the model towards its practical applicability however, is the significant number of parameters involved. Up to 15 parameters need to be estimated based on experimental data. These data are gathered by means of short term compressive tests on masonry wallets, as well as (accelerated) creep tests on masonry wallets. The final goal is to predict the service life of a masonry structure subjected to the creep failure mode. Since all parameters involved in the analysis show a significant scatter, a probabilistic analysis is put forward.

1 INTRODUCTION

Masonry, subjected to significant compressive stresses, tends to creep. This creep behaviour, depending on the level of the stress state, might lead to material collapse, which was demonstrated by the sudden collapse of for example the civic tower in Pavia (It, 1989) and the San Nicolo Cathedral of Noto (It, 1996). Also other cases in the past are relatively well documented, for example the collapse of the spire of Chichester Cathedral (UK, 1861).

After the collapses in Italy, fundamental and extensive experimental research was started up at the Politecnico di Milano under the supervision of L. Binda (Binda et al. 2001, Anzani et al. 2001). Also in Belgium collapses occurred (Kerksken 1991) and therefore collaboration was set up between Politecnico di Milano and KULeuven through the Socrates academic exchange program.

The creep behaviour is modelled using a rheological model, consisting of several basic springs and dashpots placed in series or in parallel. The global Burgers model has mainly three parts: a Maxwell spring modelling the elastic phase, a Kelvin element representing the primary creep phase and a Maxwell-Bingham element to estimate the secondary creep phase. The tertiary creep phase, leading the masonry to fail, is modelled by adding static and viscous damage parameters respectively to the first and latter component. Qualitatively, this rheological model is capable of describing the creep behaviour in a rational way. The main disadvantage of the model towards its practical applicability however, is the significant number of parameters involved. Up to 15 parameters need to be estimated based on experimental data.

These data are gathered by means of short term compressive tests on masonry wallets, as well as accelerated creep tests on masonry wallets. Additionally, real creep tests are started up to validate the rheological model resulting from the short term data. In the ongoing experimental program at the K.U. Leuven one single type of brick is combined with 4 different types of mortar: cement mortar, hydraulic lime mortar, air hardening lime mortar and a cement-lime mortar.

Purpose is to investigate the effect of differences in stiffness between brick and mortar on the long term creep behaviour.

The final goal is to predict the service life of a masonry structure subjected to the creep failure mode. Since all parameters involved in the analysis show a significant scatter, a probabilistic analysis accounting for the scatter is preferred. The framework for this type of analysis is already well established. Difficulty however is to gather the required information. First, the limit state function or failure mode needs to be well known. The better the knowledge on the limit state function, the more accurate the estimated service life will be. Subsequently, the variables involved are modelled using random variables, accounting for the present uncertainty. In order to have an acceptable representation of the parameters by means of a probability density function, sufficient data are required, which is not a simple matter in the case of relatively expensive and time consuming tests as (accelerated) creep test. The prediction model is presented and first analysis results are discussed.

2 CREEP – VISCO-ELASTIC MODEL

In general, the linear visco-elastic model is composed from a series system of a Maxwell element and a Kelvin element, see Fig. 1. Both consist of a spring and dashpot in series and in parallel respectively. The stiffness and relaxation time of the springs and dashpots are denoted E^K,E^M, τ^K, τ^M, in which the subscript K refers to Kelvin and M to Maxwell. The Maxwell spring describes the instantaneous deformation, the Kelvin element describes the primary creep phase. A friction element (Bingham element) is added before the Kelvin dashpot, to introduce a minimum stress level for the secondary creep phase to start. This rheological model is able to describe the primary and secondary creep phase.

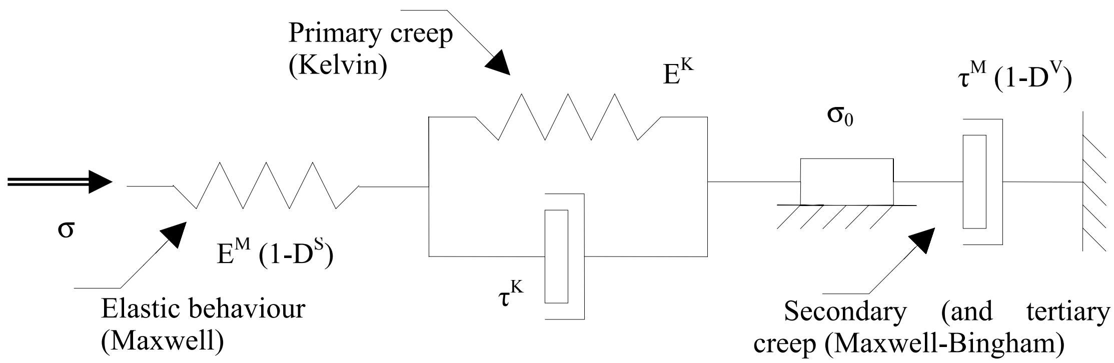

Figure 1: Schematic representation of the Burgers model including damage variables (adopted from (Papa and Talierco 2005)

The reduction in stiffness with increasing compressive stress as well as the tertiary creep phase, encountered in practice, can only be described by introducing additional damage variables. Two sets of damage variables are introduced. The first set, called static damage variables D^S, account for the damage that occurs with increasing stresses. The second set of damage variables, denoted viscous damage, D^V, represents the damage accumulation with constant loading that increases with increasing time.

In case of a constant stress level ($\overline{\sigma}$), the strain ($\varepsilon(t)$) as a function of time can be written as:

$$\begin{aligned} \varepsilon(t) &= \varepsilon^{el} + \varepsilon^{K} + \varepsilon^{M} \\ &= \frac{\overline{\sigma}}{E^{M}\left(1-D^{S}\right)} + \frac{1}{E^{K}}\left(1-\exp\left(\frac{-t}{\tau^{K}}\right)\right)\overline{\sigma} + \frac{t}{E^{M}\tau^{M}}\,\frac{H(\overline{\sigma}-\sigma_{0})}{\left(1-D^{V}\right)}\overline{\sigma} \end{aligned} \tag{1}$$

In which H() is the Heavyside function. For the viscous damage (D^V) variable, often evolution laws for brittle materials are used, such as for rock salt for example (Chan et al. 1992, La Borderie 1990). For the static damage (D^S) variable, an evolution law for concrete is adopted (La Borderie 1990).

3 EXPERIMENTAL RESEARCH PROGRAM

In order to have an acceptable representation of the parameters in the model and to better understand the behaviour of masonry under persistent loads, a number of different tests were set up at the Reyntjens Laboratory of the K.U. Leuven. In addition, several accelerated creep tests performed at the Politecnico di Milano were used to investigate the process of fitting the 15 model parameters in an optimal and objective sense (Feytons 2005). The latter originated from remains of the Civic Tower of Pavia, collapsed in 1989.

Table 1: General overview of test program at KULeuven [Swinnen and Tack, 2005]

Samples	Tests performed and material characteristics derived
Brick: Quirynen, module 50	Compressive tests ($f_{c,b}$;E;ν) and 3-point bending tests ($f_{t,b}$): $f_{c,b}$=9.97±2.24N/mm²;E=1314N/mm²;ν=0.10;$f_{t,b}$=3.12±0.47N/mm²(20)
Mortar:	Compressive tests ($f_{c,m}$); 3 point bending tests ($f_{t,m}$):
- air hardening lime	$f_{c,m}$=0.79±0.09MPa(10);$f_{t,m}$=0.52±0.03MPa(5);E=100MPa(2);
- hydraulic lime	$f_{c,m}$=4.47±0.28MPa(10);$f_{t,m}$=1.34±0.11MPa(5);E=532MPa(2)
- cement	$f_{c,m}$=34.5±3.02MPa(10);$f_{t,m}$=5.82±0.24MPa(5);E=3325±1643MPa(10);
- hybrid lime/cement	$f_{c,m}$=4.20±0.90MPa(12);$f_{t,m}$=1.13±0.24MPa(6);E=235±190MPa(10);
Masonry columns:	Compressive test (f_c,E):
- air hardening lime	f_c=4.76N/mm²(2); E =1374N/mm² (2)
- hydraulic lime	f_c=6.3±0.9N/mm²(3); E =2136±45N/mm² (3)
- cement	f_c=7.3±0.2N/mm²(3); E =2601±160N/mm² (3)
- hybrid lime/cement	f_c=7.3±0.3N/mm²(3); E =2902±320N/mm² (3)
Masonry columns:	accelerated creep test: $f_{c,creep}$ and t: time to failure
- air hardening lime	Planned 01/10/2006 (accelerated carbonation executed)
- hydraulic lime	$f_{c,creep}$=5.9±0.2N/mm²(3); t= 1000-4500 min
- cement	$f_{c,creep}$=5.2±0.4N/mm²(3); t= 1000-4500 min
- hybrid lime/cement	$f_{c,creep}$=6.8±0.3N/mm²(3); t= 1000-4500 min
Masonry columns:	Creep tests – stress level: 50%; 65% and 80% of f_c:
- air hardening lime	Planned 01/10/2006 (accelerated carbonation executed)
- hydraulic lime	Ongoing (3) - started dd.: 04/10; 04/10; 11/10/2004
- cement	Ongoing (3) - started dd.: 08/01; 26/03; 11/10/2004
- hybrid lime/cement	Ongoing (3) - started dd.: 05/10; 11/10; 13/10/2004
Legend: presentation of test results: *average value±standard deviation* (*no. of samples*)	

The necessary data are gathered by means of monotonic compressive tests on masonry wallets, as well as accelerated creep tests on masonry wallets. Additionally, real creep tests are started up to validate the rheological model resulting from the short term data. In the ongoing experimental program one single type of brick is combined with 4 different types of mortar: cement mortar, hydraulic lime mortar, air hardening lime mortar and a cement-lime mortar. Purpose is to investigate the effect of differences in stiffness between brick and mortar on the long term creep behaviour. The brick type and mortar composition were chosen to be representative for the historical masonry encountered in Belgium. For each type of mortar, 9 wallets were built, Fig. 2a. For the monotonic compressive tests (3 tests) as well as for the accelerated creep tests (3 tests) a hydraulic test device was used in combination with LVDT's (to measure the deformations). For the real creep tests (3 tests), separate steel frames were constructed and hydraulic jacks were used, in combination with a demec-measuring device to measure the deformations, Fig. 2b.

A general overview of the test program performed at KULeuven is presented in Table 1. Detailed results can be found elsewhere (Swinnen and Tack 2005, Deswert and Vanderstraeten 2004). Simultaneously deformation measurements are executed on real structures during ongoing restoration works.

3.1 *Creep model - parameter estimation based on experimental data*

The results of the compressive test and accelerated creep tests are used to determine the different parameters of the rheological model (Feytons 2005, Tack and Swinnen 2005). Afterwards a verification of the rheological model will be obtained by means of the real creep tests. The model parameters obtained from the monotonic compressive test and accelerated creep test are outlined in Table 2. The details on the meaning of the different parameters and their estimation can be found elsewhere (Feyons 2005). From Table 2 it is clear that derivation of some of the parameters is straight-forward, whereas others are much more difficult to capture in an objective way. Several parameters required in the static (D^S) and viscous (D^V) damage are given a constant value over the different types of mortar. Their meaning can be argued.

Table 2 : Parameters of rheological model and source tests (Feytons 2005)

Test	Parameter in rheological creep model			Hydraulic lime mortar	Cement mortar	Hybrid mortar
Mono-tonic com-pres-sive test	Poisson's ratio	ν [/]		0.10	0.48	0.22
	Bingham element	σ_0		4.62	4.16	6.16
	Static Damage	D^S	Y_{0c} [MPa]	3e-3	7.1e-3	1.1e-2
			Y_{0t} [MPa]	3e-5	7.1e-5	1.1e-4
			A_c [MPa]	800	800	800
			B_c [/]	1.2	1.2	1.2
			A_t [MPa]	9000	9000	9000
			B_t [/]	1.05	1.05	1.05
	Maxwell element	E^M [MPa]		4651	5275	4038
		τ^M [s]		59526	32147	46490
accel-erated creep test	Kelvin element	E^K [MPa]		1959	1764	2112
		τ^K [s]		78907	96643	90866
	Viscous damage	D^V	X_{1c} [$s^{-1}MPa^{-1}$]	0.67	10.18	0.5
			X_{2c} [/]	3.23	0.90	1.1
			X_{1t} [$s^{-1}MPa^{-1}$]	7.7	7.7	7.7
			X_{2t} [/]	0.35	0.35	0.35

3.2 *Ongoing creep tests (Deswert and Vanderstraeten 2003), (Tack and Swinnen 2005)*

In 2003, a number of masonry wallets with surface dimensions ±200x300 mm² and a varying height of about 850 mm, consisting of 14 bricklayers and mortar joints with a thickness of approximately 10-12 mm were constructed. At the ground surface the columns are placed on a concrete pedestals, see Fig. 2a.

(a) (b)

Figure 2 : (a) Masonry wallets; (b) Test set up real creep test

At this moment, the creep tests of only 3 mortar types are ongoing. Each of the wallets is subjected to a different stress level, namely: 50%, 65% and 80% of the obtained compressive strength, Table 1. The wallets with the air hardening lime mortar are prepared to ensure a complete carbonatation of the mortar. This way, an extra parameter was avoided (the amount of carbonatation). In most historical buildings complete carbonatation of the lime mortar can be expected. These wallets will be tested after completion and evaluation of some of the ongoing creep tests.

The preliminary results of the real creep tests until the last measurement of 14/04/06 show that all wallets have a linear increasing deformation (vertical as well as horizontal) as a function of time. This means that all wallets still are in the secondary creep phase. Fig. 3 illustrates the average vertical deformations for the wallets constructed with 3 different types of mortar, subjected to three different stress levels.

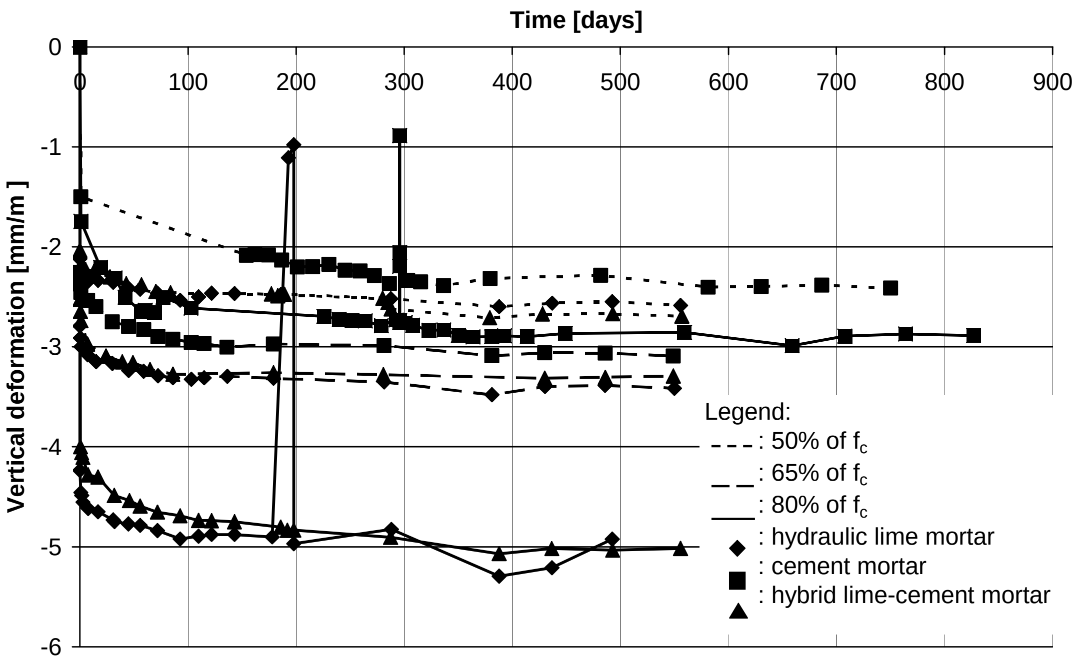

Figure 3 : Creep tests - average vertical deformations of masonry pillars as a function of time

The graphs show clearly a higher deformation level for higher load levels, except for the cement mortar at 80% of its compressive strength. This sample has been built in a former test program, clearly demonstrating deviating stiffness properties. For two wallets an unload and reload cycle was included. The unloading revealed a significant portion of non-reversible deformation. After reloading, the strains reached the original level again. In relation with the different types of mortar used, it can be stated that the behaviour of the different samples is quite similar. Also in the secondary creep phase, a difference in speed of vertical deformations is hardly noticeable. One could even state that the lower stress levels only exhibit a very limited creep behaviour. The latter demonstrates that probably higher stress levels are required to enforce creep behaviour within an acceptable test period. On the other hand, when unloading takes place, some permanent plastic deformation is envisaged.

3.3 Discussion of results

In general, the creep model is very well able to simulate the accelerated creep tests. Drawback of the model however is the lack of appropriate static and viscous damage models for masonry. Up till now, models originating from rock or salt are being used. In general, the number of parameters is large (6 for D^S and 4 for D^V). Therefore, the determination of the different parameters sometimes becomes subjective. This is further increased due to the low number of tests performed in combination with the high scatter of the obtained data and the observed dependency in between the different parameters. In general it is not possible yet to predict the behaviour of the real creep tests, based on the test results of the monotonic compressive tests and accelerated creep tests in an objective way.

3.4 Data from real structures – future goal

In parallel with the laboratory tests on masonry wallets, deformation measurements are being performed on several real historical masonry structures. In general, a convergence measuring device is used to measure horizontal deformations between walls or pillars. At this moment in time, the long-term behaviour of following structures is being monitored on a regular basis, all located in Belgium (Ignoul et al. 2005, Schueremans et al. 2006):

- the church of Saint Catharina at Duisburg;
- Our Mary Basilica at Tongeren;
- Saint-Jacobs Church at Leuven;

In all cases the monitoring campaign covers the period during and after restoration works took place in which the structural layout altered significantly. In the near future, detailed measurements with fibre optic devices are planned during and after restoration of the 's Hertogenmolens at Aarschot, Belgium. The results of these measurements give an indication of the time needed for an historical monument to adapt to the newly created structural situation. The objective is to link these data with the overall deformations obtained from a structural analysis, also accounting for the time-dependent creep behaviour of the masonry. For example, the results for the church of St. Catharina at Duisburg (Belgium) reveal that 4 years after the tensioning of 3 new vault anchors, the structure is still adapting to this 'new' situation. It is clear that this information is of major importance to judge the overall actual safety of the structure and predict its service life in an objective way.

4 PROBABILISTIC MODEL – SERVICE LIFE PREDICTION

4.1 General framework

To predict the service life of the structure, the general load-resistance model, can be adopted [Melchers, 1999]. In general a limit state function g() has to be defined, that outlines the safe (g()>0), unsafe (g()<0) and critical (g()=0) situation. Additionally the probabilistic model for the different random variables needs to be available.

Based on the accelerated creep tests, it was difficult to retrieve a consistent limit state function. Based on a linear regression through the stress-strain points at collapse, following empirical limit state function is established (Tack and Swinnen 2005):

$$g(\varepsilon(t),\varepsilon_{ult}(\sigma(t)))=(-0.014\sigma(t)+0.091)-\varepsilon(t) \quad (2)$$

This limit state function is based on both the stress and strain at a certain moment in time. Of course, the better the limit state function, the better the outcome of the reliability analysis. This is mainly a matter of insight into the structural behaviour and acquiring sufficient data that are lacking for the moment. The actual strain ($\varepsilon(t)$) is obtained from a stepwise function based on eq. 1, applied for the different stress levels in the actual load path.

The failure probability is calculated following:

$$p_f = P[g(\varepsilon(t),\varepsilon_{ult}(\sigma(t)))<0] \quad (3)$$

This analysis can be performed for different values of time (t). With preset values of target failure probabilities, the corresponding service life can be derived. As an example the failure probability (p_f) was calculated for the accelerated creep test on hydraulic lime mortar for which experimental data were available (Swinnen and Tack 2005). In the accelerated creep test, the loading path starts with an initial compressive stress level equal to 3.70 N/mm² and an increase of the compressive stress level each 6000s with a step of 0.2 N/mm². The wallet collapsed after 72000s (20 hours) at a compressive stress level of σ_c=5.9 N/mm². Based on the experimental data of the accelerated creep tests on wallets with hydraulic lime mortar, the main creep parameters were defined as random variables, Table 3. Lacking more detailed information, all variables were considered Gaussian distributed. The parameters not listed in Table 3, were assumed constant.

Table 3 : Random variables involved, adopted from (Swinnen and Tack 2005)

Random variables	Probability density function	mean	Standard deviation
E^M [MPa]	Gaussian	2200	200
τ^M [s]	Gaussian	2.7 10^5	0.5 10^5
E^K [MPa]	Gaussian	20000	7000
τ^M [s]	Gaussian	14000	5000
D^S	Gaussian	Nominal values, Table 1	=20% × mean
D^V	Gaussian	Nominal values, Table 1	=20% × mean
σ (applied stress) MPa]	Gaussian	Nominal value from loading path	=7% × mean

Using an (Importance Sampling) Monte Carlo analysis, an accurate outcome of the failure probability could be obtained for different values of the service life (t_L):

- t_L = 6000 s: p_f = 0.0035 and β= 2.6;
- t_L = 72000 s; p_f = 0.60 and β= 0.75.

It is clear that after the first loading step, the actual failure probability is relatively low. Although, with the uncertainty taken into account, the remaining reliability index β does no longer reply to general safety levels preset in European Standards. In the latter, a target value equal to β_T = 3.8 ($p_{f,T}$ = 7.2 10^{-5}) is proposed for ultimate limit states. At the final loading step, after 72000s, the failure probability increased up to an unacceptable level of p_f = 75%, which is not equal to 100% because of the biased limit state function and the limited accuracy in the statistics used. Despite these inconveniences, it is clear that this framework provides a powerful tool in decision making, on condition of three important items: a accurate and unbiased creep model, an appropriate limit state function, and finally probabilistic models for the random variables involved. These essential ingredients will remain in focus during further research.

5 CONCLUSIONS

Simple monotonic and accelerated creep tests can be set up to determine the parameter set for masonry wallets and to simulate real creep behaviour. For the moment, obtaining a consistent data-set is not straight-forward. Real creep tests still are required to be executed in order to control and validate the obtained parameters. At the Reyntjens Laboratory of the K.U. Leuven, 9 real masonry creep tests are ongoing since 2004. 3 More creep tests are planned on air-hardened lime mortar.

Since all parameters involved in the analysis show a significant to large scatter, a probabilistic analysis accounting for the scatter is preferred. In order to have an acceptable representation of the parameters by means of a probability density function, sufficient data are required, which is not a simple matter in the case of relatively expensive and time consuming tests as (accelerated) creep test. The prediction model is presented and first results are available. The model can be improved when the results of the real creep tests become available.

More theoretical research is needed in order to find a more objective way to determine the different parameters in the model and to optimize the probabilistic model. Two Ph. D. research projects have been started recently to tackle this problem (Ignoul 2003, Verstrynge, 2006). Emphasis will be on a consistent modelling of the static and viscous damage and its parameter estimation, an update of the probabilistic service life prediction and the application of these tools in the structural analysis of real case studies.

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