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EXPERIMENTAL AND NUMERICAL SIMULATIONS OF COLLAPSE OF MASONRY ARCHES

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Abstract: *The paper presents experimental and numerical simulations of collapse of masonry arches with backfill. Research on three masonry arch models loaded until failure is shown. In the framework of the paper details related to performance of experiments as well as to numerical modelling are given. Precise description of result obtained from the real tests and the virtual simulations are carefully compared and the most important conclusions are drawn. The laboratory tests are performed on models representing masonry arch structures with backfill – corresponding to bridge spans or vaults in buildings. The assumed destructive loading is the concentrated force located at a quarter or in the middle of the span representing live loads. The numerical simulation of the tests is based on nonlinear analysis carried out by means of Finite Element Method (FEM). For the calculations advanced two-dimensional FE models including all the most important material and geometrical nonlinearities are applied. A precise technique for modelling of the masonry arch barrel is proposed and carefully described. Destruction of the structures under loading is being precisely analysed. A special attention is paid to detection of cracks' (plastic hinges') formation, recording the ultimate load values and monitoring the final collapse mode of the whole structures.*

1 INTRODUCTION

Masonry arches covered with backfill are important elements of many structures including bridges and buildings. They have been used as structural elements for ages however existing methods of analysis of the arches are still not perfect nor very precise. Especially advanced numerical approaches based on Finite Element Method (FEM) require extensive experimental data for calibration of the calculation model including selection of proper values of material and geometrical parameters. Therefore performance of tests on real structures of that kind is exceptionally desirable research. It is also important taking into account very limited number of similar test results available on literature what can be related to complexity of the task.
Within the framework of this paper results of tests and analyses of laboratory models of brick masonry arches covered with backfill are presented. Two types of typical backfill material are considered: sand and granite aggregate. The laboratory tests carried out at Cracow University of Technology were based on loading the models with a concentrated force till their collapse. Numerical analyses performed afterwards at Wroclaw University of Technology were aimed at reliable simulation of the laboratory tests. Results of both experimental and numerical simulations were carefully compared and discussed.

2 DESCRIPTION OF LABORATORY MODELS

The tested laboratory models consisted of: masonry arch supported by reinforced concrete abutments, fill material above the arch, end and side walls surrounding backfill material (Figure 1). The arch was built of clay bricks and lime mortar. The end walls were made of reinforced concrete whereas side walls were made of OSB or Plexiglas board stiffened with steel elements. Since the side walls were non-structural elements, therefore between walls and the arch vault about 15 mm wide gaps were left.

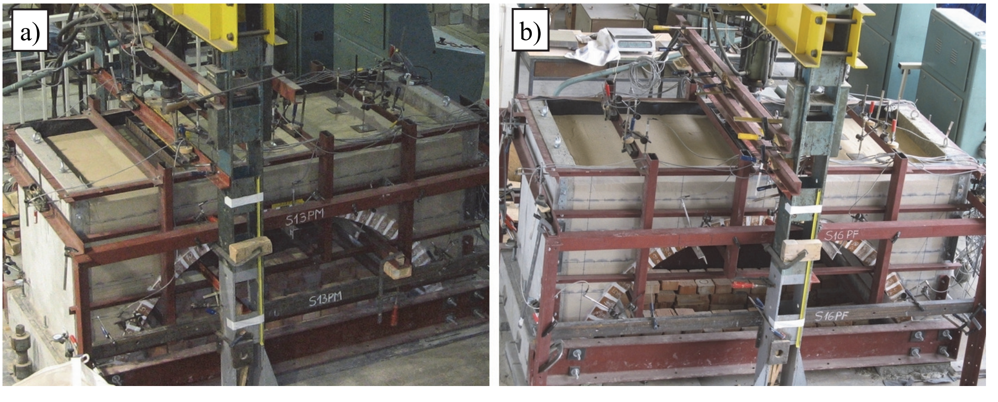

Figure 1: Laboratory models: a) load at a quarter span; b) load at the mid-span

In the tests sand with grain size 0.125-1 mm or granite aggregate with grain size 8-16 mm were used as the backfill. The fill material was placed and compacted every 200 mm thick layers. The total depth of the fill at the crown was equal to 200 mm for models AGQ and ASQ or 80 mm in case of model ASM. Geometrical parameters of laboratory models like:

arch internal span *L*, rise of the arch *R*; arch thickness *T*; arch width *W*; backfill depth over the crown *H* and load location *x*/*L* are presented in Table 1 and in Figure 2.

Model	*L* [mm]	*R* [mm]	*T* [mm]	*W* [mm]	*H* [mm]	*x*/*L* [-]
AGQ					200	0.25
ASQ	2000	730	125	1040	200	0.25
ASM					80	0.50

Table 1: Geometrical parameters of the laboratory models

3 SCOPE OF THE TESTS AND MEASUREMENTS

The aim of the presented experiments was to determine load-carrying capacity and examine the general behaviour of buried masonry arches. During the tests load, radial displacements of vault (U1, U2, U3) and vertical displacements of the loading beam (UP) were measured Figure 2). These parameters are the most representative and useful data in tests of real structures [3]. The arrangement of displacement transducers (LVDT) is given in Figure 2.

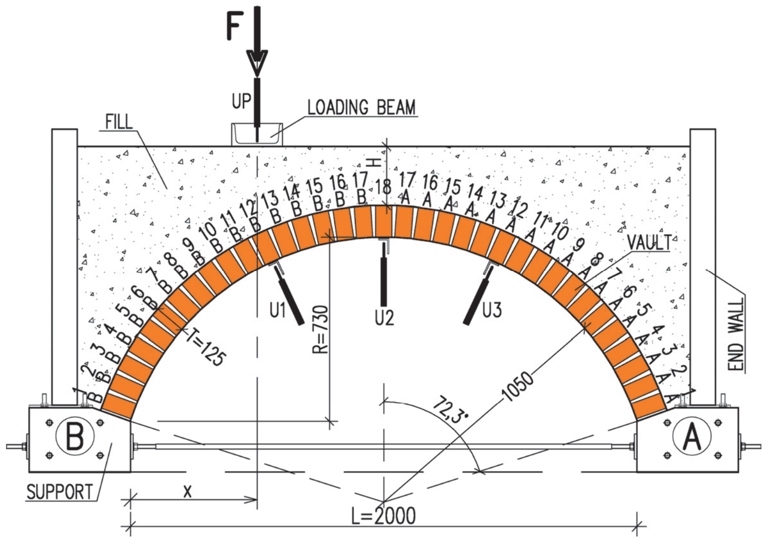

Figure 2: Geometry of the vault, dimensions in mm. Arrangement of displacement transducers (LVDT), brick courses numbering

Moreover during the tests development of cracks in masonry joints was observed. The load was applied to the top of the fill material and was being increased continuously until failure. Models AGQ and ASQ were loaded at a quarter span whereas model ASM was loaded at the mid-span.

Both models loaded at a quarter span failed due to formation of four-hinge mechanisms at load of 58.2 kN – AGQ and 37.2 kN – ASQ (Figure 3). In case of arch loaded at the mid-span five-hinge mechanism was observed at load of 78.5 kN. Modes of failure for all tested models are presented in Figure 5-7. In the same figure load-displacement (*P*-*u*) plots are given.

Figure 3: Hinges of the four-hinge mechanism - model ASQ

It can be seen from *P*-*u* diagrams that during loading process some unloading took place. The models were unloaded because instability of the loading beam occurred. After releasing of the load the loading beam was stabilized and then test was restarted. Detailed description of the models, testing procedure and test results are given in [5], [8] and [9].

4 DESCRIPTION OF NUMERICAL MODELS AND ANALYSIS

For the purpose of computer simulations of the laboratory tests presented in previous chapters two-dimensional (2D) FEM models were applied (according to methodology elaborated in [1], [6], [7]). The models represented all main components of the laboratory structures: arch barrel, backfill, loading beam and ending walls outside the arch (see Figure 4). The boundary conditions representing support for the arch barrel springing and side (spandrel) walls surrounding the backfill were assumed as rigid – therefore plain strain (PE) state for the model elements was defined. For modelling of the arch barrel and the backfill 4-node linear elements were selected. The end walls and the loading beam were modelled by means of rigid elements. Between these elements and the backfill as well as between the backfill and the arch barrel contact edges were defined, which provided transfer of compression and shearing (limited by coefficient of friction μ_w or μ respectively) and allowed separation of contacting edges without generation of tension.

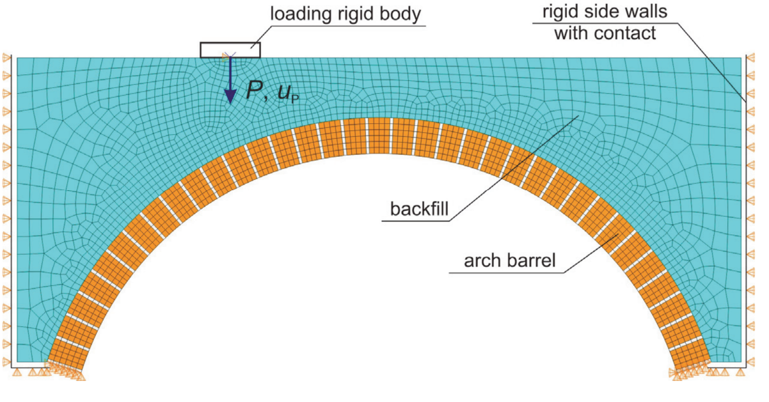

Figure 4: FE model applied in computer simulations of the laboratory tests

The masonry arch barrel was discretised with elements corresponding to alternate layers of bricks (together with head joints) and radial mortar joints (with unit weights denoted by γ). The material of brick layers was defined by orthotropic elastic-perfectly plastic model with yielding determined by compressive strength $f_c^{\,b}$. Special attention was paid to modelling of the mortar joints which are the critical areas of the arch where damage (due to crushing or cracking) occurs first at loading. Thus the mortar joints were described by elastic-brittle material model with *Concrete Damaged Plasticity* formulation of degradation [4] (modulus of elasticity $E_x^{\,m}$, Poisson's ratio ν^m) with yielding strengths at compression $f_c^{\,m}$ and tension f_t. Refined stress-strain relationships for hardening/softening at both compression and tension were also defined. The material properties of the masonry arch components assumed as constant in all analysed models are given in Table 2.

Material (*i*)	γ^i [kN/m^3]	$E_x^{\,i}$ [GPa]	ν^j [-]	$f_c^{\,i}$ [MPa]	f_t [MPa]
brick (*b*)	17.0	20.0	0.20	20.0	-
mortar (*m*)	17.0	0.25	0.16	0.50	0.10

Table 2: Material properties for masonry arch components applied in FE models

The material of backfill was defined by elasto-plastic model with Drucker-Prager yielding surface. Material properties assumed in numerical analyses (like unit weight γ_s, modulus of elasticity E_s, Poisson's coefficient ν_s, angle of internal friction ϕ, cohesion c and dilation angle β) are given in Table 3.

Material	γ_s [kN/m^3]	E_s [MPa]	ν_s [-]	ϕ [deg]	c [kPa]	β [deg]	μ [-]	μ_w [-]
sand	18.9	40	0.3	38	7	15	0.5	0.3
granite	14.0	30	0.3	46	5	40	0.7	0.5

Table 3: Material properties for sand and granite applied in FE models

The analysis is carried out in two consecutive steps: in the first one self-weight of the structural components is applied and in the second one the external P load is added. The external load is applied by means of a rigid element (20 cm wide) acting along the contact edge on the top of the backfill over the quarter (in models AGQ and ASQ) or over the middle (in model ASM) of the span. The boundary conditions imposed on the rigid body constrain its horizontal translation. In the second step the loading process is controlled by the vertical displacement u_P of the rigid element. This method enables easy reaching the ultimate load and its clear determination as the highest value of the loading force taken from the relationships between the displacement u_P and the reaction force P acting on the rigid body.

5 COMPARISON OF EXPERIMENTAL AND NUMERICAL TESTS

The measured and calculated values of selected quantities presenting behaviour of the three laboratory and numerical models under loading are carefully compared. The results in the form of relationships between displacements u_i and loading force P are shown at diagrams in Figure 5-7. In the figures modes of failure received during the tests as well as from the numerical analyses are also presented.

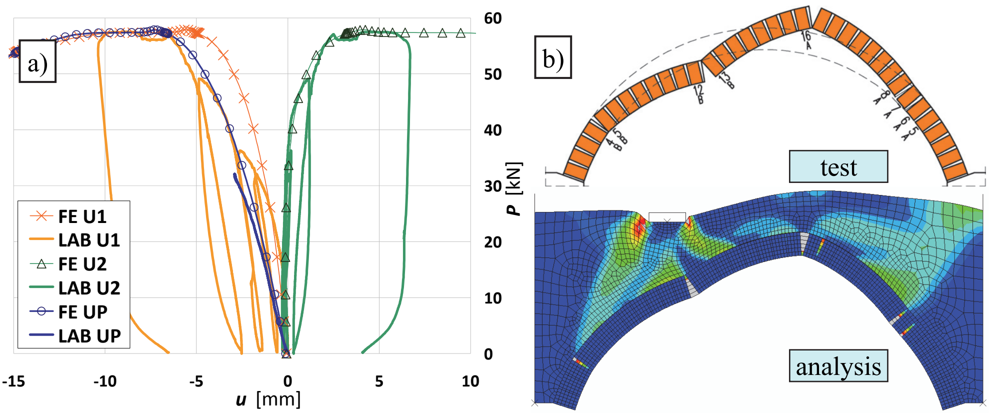

Figure 5: Results for model AGQ: a) P-u_i relationships, b) modes of failure

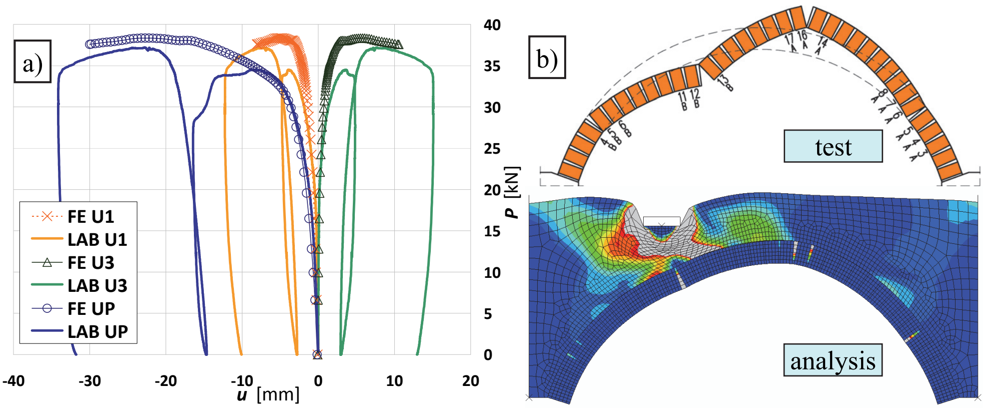

Figure 6: Results for model ASQ: a) P-u_i relationships, b) modes of failure

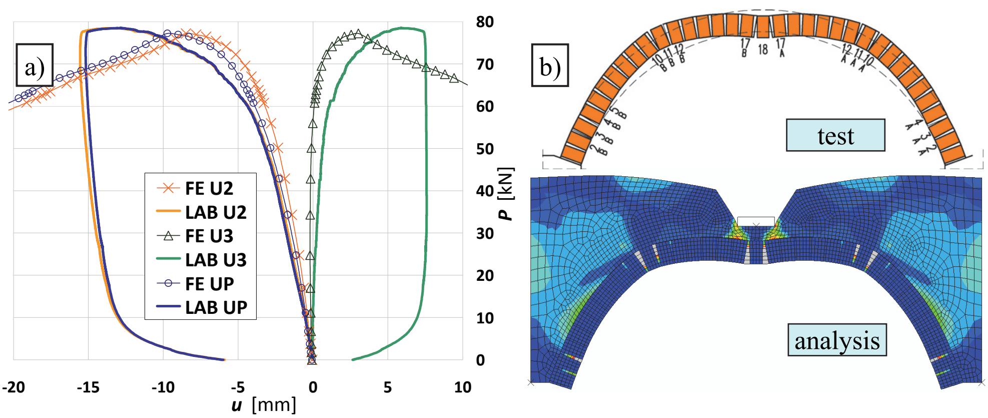

Figure 7: Results for model ASM: a) P-u_i relationships, b) modes of failure

A good agreement between experimental and numerical results was reached, mainly in terms of detected modes of failure (locations of plastic hinges) and ultimate loads.

6 CONCLUSIONS

The presented paper shows two-stage process of development of precise methodology for prediction of the load carrying capacity of masonry arch structures. The first phase was careful planning and performance of the laboratory tests and the second one was related to computer simulations of these tests. Both stages provided large amount of strict physical data and practical conclusions useful in assessment of similar structures.

One of difficulties encountered during laboratory tests was instability of the loading beam. The effect was mainly related to the loading method. The load was applied to a beam by means of a hydraulic actuator via ball-and-socket joint which allowed beam rotations. Observed instabilities could be also related to small area of the loading beam and relatively low stiffness of the fill. Occurrence of the instabilities imposed the need of repeating the loading and unloading (visible in *P*-*u* diagrams) of the structure and correction of loading beam arrangement before reaching the ultimate load.

Some unexpected material properties' values applied in numerical models provided the best representation of the laboratory tests. For example, modulus of elasticity of the granite fill was assumed at unusual low level to correspond to low stiffness of the backfill itself. The effect of low backfill stiffness at the early stage of loading could however be related to low compaction of the fill over the arch. Due to limited space of the backfill in the models very significant influence of the end walls on results of calculations was identified regarding vertical displacements of the fill contacting with the walls. Therefore careful selection of the friction coefficient for this contact played an important role in establishing the load carrying capacity of the structure.

Also important influence of cohesion parameter c was found. In the analyses it was assumed equal to 5-7 kPa to provide numerical stability of the solution, although such value does not correspond to strength of theoretically non-cohesive materials (like sand or granite). However for values of c lower than 5 kPa much larger discrepancies between numerical and experimental results were received even in initial stages of loading.

Discrete modelling of masonry arch barrel distinguishing areas of bricks and mortar joints enabled better representation of the structural behaviour at the limit state than do homogenous models based on macro-modelling of masonry (applied e.g. in [2]). Strictly localised plastic hinges (in most cases concentrated in single joints) were successfully represented by means of the FE models.

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