Strengthening of masonry structures with an additional reinforcement

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Resumo

Este trabalho apresenta resultados de testes de carga de dois sistemas de resistência (o aço inoxidável reforçado e o novo reforço GFRP), com o objetivo de informar as possibilidades de sua utilização em alvenarias estruturais, resistindo à ação de forças normais e momentos fletores, quando se verifica experimentalmente o comportamento de perfis de aços com forma helicoidal, aplicado como reforço da alvenaria. Uma comparação entre a eficiência dos dois sistemas resistentes é apresentada, com o objetivo de coletar os dados básicos de projeto na análise estrutural dos elementos de resistentes da alvenaria, ainda não incluídas nas normas técnicas ČSN 731101 e ČSN 731102, da República Checa. O método permite um reforço na estrutura da alvenaria através de pré-tensionamento. Este sistema é capaz de redistribuir os esforços originados pelas cargas que atuam em um sistema estrutural.

Palavras-chave: *Resistência. Alvenaria estrutural. Aço reforçado. Reforço GFRP.*

Abstract

In this paper are presented results of the load tests of the two strengthening systems (stainless steel with helicoidally shape and in Czech Republic new developed GFRP reinforcement). The aim of performed work was to document possibilities of the use of those systems for the strengthening of masonry structures loaded with the interaction of a normal force and a bending moment and to verify experimentally the behaviour of specially shaped profiles of the reinforcement and the grout in masonry. Furthermore, to obtain comparison of efficiency of both reinforcing systems, to collect basic design data for the structural analysis of strengthened masonry members, as strengthening of masonry structures is not possible to design according to the Czech standards ČSN 731101 and ČSN 731102. The method of additionally inserted non-prestressed reinforcement allows additional strengthening of masonry structures without a necessity of large intervention into vaults especially in case of external application. This system is capable redistribute newly originated stresses from load that act on a strengthened construction. The aim of reinforcing is to restrict the development of existing cracks and eliminate possibly an origin of the new ones, and to improve load-bearing capacity of vaulted masonry constructions.

Keywords: *Strengthening. Masonry structures. Stainless reinforcement. GFRP reinforcement.*

1 Introdução

Masonry continues to be popular because of its relative simplicity of application in the technical practice. Indeed, for a new use of structural masonry reasonable constructional rules have to be required, because conventional approach based on the experience is unacceptable nowadays. In addition most of methods both of carrying capacity assessment and of strengthening for the existing masonry construction are increasingly based on analyses of mathematical simulation and appropriate (linear and nonlinear) computational models.

One method of load-bearing elements strengthening is application of additional external reinforcement into chases in masonry, which will provide stiffening and increasing of load carrying capacity of the individual load-bearing elements. This paper is based on the experiments in the field of masonry structures strengthening that were performed on Faculty of Civil Engineering Brno University of Technology. In up to now carried out experiments, strengthening of bended masonry beams and vaults with additional external reinforcement of system Helifix (stainless steel reinforcement) and GFRP reinforcement fully developed by solution of research task of Czech Ministry of Industry and Trade (MPO POKROK 1H-PK2/57) of additionally applied reinforcement were tested.

2 Pull-out tests and beam tests

From performed pull-out tests for some embedment lengths it is possible to claim following conclusions (ZLÁMAL; ŠTĚPÁNEK, 2004)

- The anchorage length should be considered as such embedment length of reinforcement, when the ultimate force carried-out by a rebar does not further increase with rising embedment length. The anchorage length can be then determined according to the mortar and masonry quality in the range between 300 and 550mm.
- The significant difference between the behaviour of special and concrete rebars has appeared when the peak tensile force was achieved. The special reinforcement is "screwed out" from the mortar but the rebar still carry the tensile force. In the case of concrete reinforcement with the groined surface is a failure of the anchorage zone brittle.

From the tests of the masonry strengthened beams result following conclusions (CZEMPIEL; ŠTĚPÁNEK, 2003)

- Until cracks in the masonry develop, all materials behave elastically according to Hook's law. As cracks develop in the masonry, the bending stiffness of the masonry element decreases. After the cracking has developed in the grout, the stiffness of the beam in bending abruptly decreases rapidly.
- It is obvious that there is significant difference between the experimentally obtained ultimate load-bearing capacity and the load-bearing capacity calculated in accordance with design standards.
- Masonry with the retrofit reinforcement satisfies the load bearing function even after cracking has occurred. The ultimate load can be achieved by a number of causes, either individually or in combination:
  - failure of masonry in its compressed area through crushing,
  - rupture of reinforcement in tension (this can appear only after the masonry has failed through cracking in its tensioned area),
  - failure of anchorage of the tensile reinforcement (collapse of the anchorage area),
  - failure of masonry element from the development of shear cracks near the obvious supporting joints.

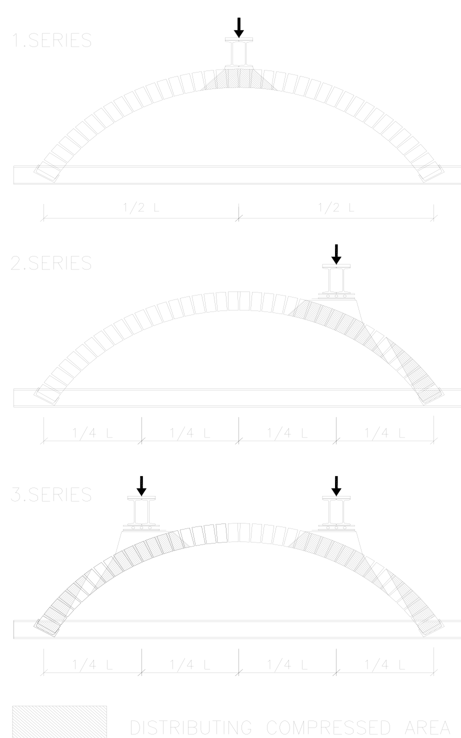

Fig. 1: Loading scheme of vaults and distribution of load in vaults

3 Strengthening of vaults

3.1 Description of experiment

Within experimental parts of the project three sets of masonry vaults for various loading types were manufactured. For the distinction of individual vaults is used notation jKi, where „j" corresponds to series number (1-3) and „i" to the strengthening method (1-3). The vaults were symmetrically loaded in ½ of the span - 1.series (j=1) - (Fig. 1), asymmetrically in ¼ of the span - 2.series (Fig. 1) and symmetrically in both quarters of the span - 3.series (j=3) - (Fig. 1). Each series consists of three vaults: non-strengthened one – comparative (i=1), a vault reinforced in two chases (i=2) and a vault reinforced in three chases (i=3). The vaults were bricked up from full burnt bricks on lime-cement mortar of the width 890 mm, span 2600 mm, deflection 750 mm and radius 1500 mm. Into every reinforcing chases were embedded 2 bars HeliBar of special helical shape of diameter 8 mm. For provision of static border conditions the vaults were bricked up into the steel frames, in order to prevent their horizontal and vertical displacement in the bedding.

It may be generally stated that the tests results will be valid for any kind of additionally applied reinforcement; but of course the actual physical-mechanical characteristics of materials used for strengthening are decisive.

3.2 Interpretation of tests results

From the comparison of the load-bearing capacity of the individual vaults in the series it results that essential growth of the load-bearing capacity was achieved especially in the case of 1st series and 2nd series of the vaults, namely more than eight multiple growth. It was related to the vaults stressed by either concentrated or one-sided load, at which the vaults were loaded by the interaction of normal forces and bending moments.

In the case of 3rd series the experiments did not prove the effects of strengthening by additionally inserted reinforcement on the vaults load-bearing capacity; no effects of reinforcement demonstrated themselves because the vaults were mainly compressed. The result values of the loading and corresponding deformations for all series for stainless steel reinforcement are presented in Table 1. The results of the tests for the second series with GFRP reinforcement are presented in Table 2.

In the case of non-strengthened vaults of 1st and 2nd series the failure was acute, main crack was opened and the vault ruptured. In the case of the strengthened vaults of 1st and 2nd series came to the gradual opening of separate cracks until the failure, which was accompanied by the rapture of reinforcement from the chases.

In the case of 3rd series of the vaults the failure of strengthened vaults was analogous to the failure of non-strengthened ones. The vaults in these series were mainly compressed, failure wasn't accompanied by the crack origin and the loss of stability was caused by very sudden crush of mortar in joints.

Table 1: Comparison of load-caring capacities and deformations of vaults – helicoidally reinforcement

j	i	Vault nr.	Maximal force $F_{max, jKi}$ [kN]	Maximal deformation w_{jKi} at $F_{max, jKi}$ [mm]	Comparison of deformations w_{jKi} at $F_{max, jK1}$ [mm]	F_{jKi}/F_{jK1}
1	1	1K1	4,737	3,233	3,233	1
	2	1K2	30,508	19,578	0,463	6,44
	3	1K3	39,98	18,614	0,309	8,44
2	1	2K1	4,933	1,678	1,687	1
	2	2K2	30,201	17,726	0,28	6,12
	3	2K3	43,756	16,884	0,331	8,87
3	1	3K1	368,584	8,975	8,975	1
	2	3K2	370,239	10,369	10,117	1
	3	3K3	439,772	8,9665	6,801	1,19

Table 2: Comparison of load-caring capacities and deformations of vaults – GFRP reinforcement

j	i	Vault nr.	Maximal force $F_{max,\,jKi}$ [kN]	Maximal deformation w_{jKi} při $F_{max,\,jKi}$ [mm]	Comparison of deformations w_{jKi} při $F_{max,\,jK1}$ [mm]	F_{2Ki}/F_{2K1}
2	1	2K1	4,772	2,47	2,47	1
	2	2K2	30,61	17,512	0,3	6,41
	3	2K3	40,111	14,9	0,087	8,41

Comparison of behavior of vaults strengthened with stainless steel and GFRP is shown on the Fig. 2. The deformation under acting load (denoted S ¼ under) and on the other side unloaded half of vault (denoted S ¼ unloaded) are drawn.

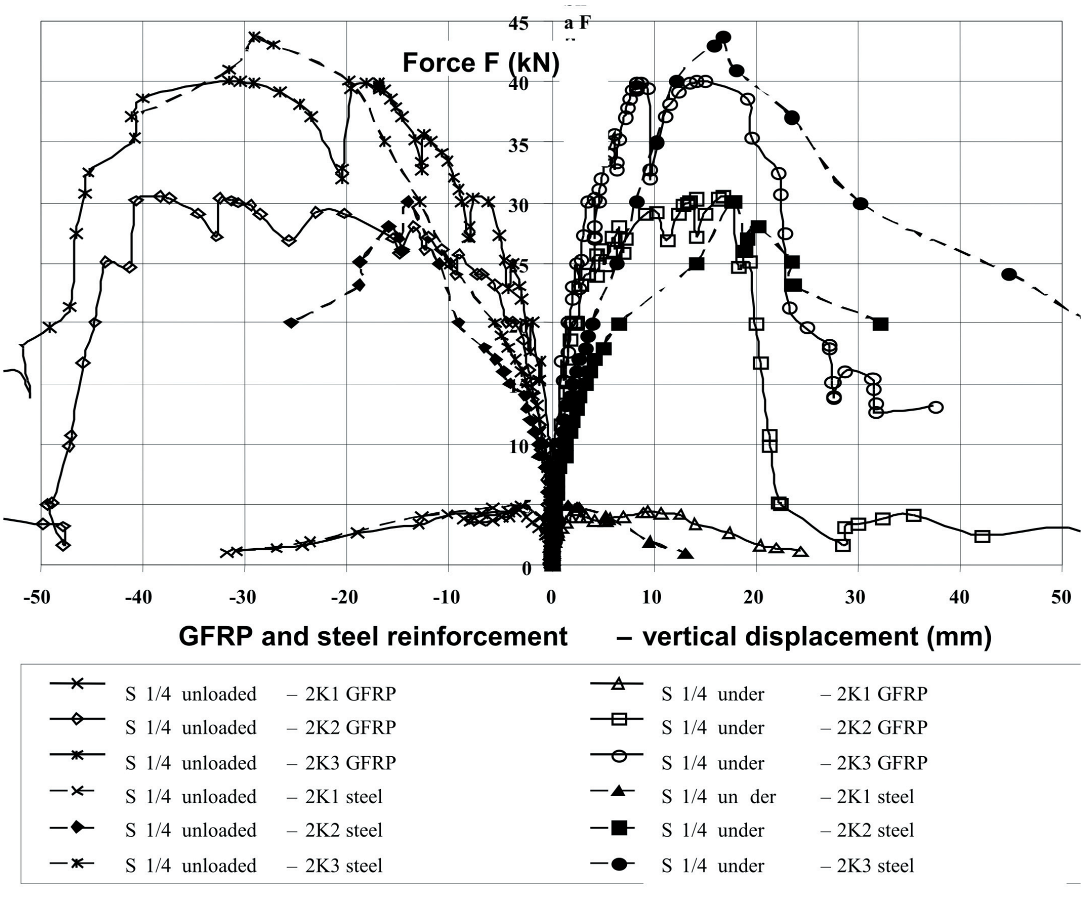

Fig. 2: Working diagrams of vault strengthened with stainless steel and GFRP reinforcement in 2 chases (vaults are loaded at ¼ of span)

4 Mathematical modeling

4.1 Base classification of models for masonry

A numerical model for the study of spatial structures, including masonry vaulted bridges, consisting of curved, three-dimensional members with variable cross sections, together with its application to the nonlinear geometric and material analysis of skeletal masonry can be based on the simplifications and approximations which are introduced below. Nonlinear material behavior is included in the model by means of elastic-plastic constitutive equations under shear and compressive stresses, while a linear-elastic perfectly brittle behavior is assumed in tension. The dependency of shear strength upon the applied compression is taken into account by means of the Mohr-Coulomb failure criterion. Nonlinear geometric affect caused by the imposition of the equilibrium condition upon the deformed configuration of the structure are considered, but it is assumed that the increments of both displacements and sectional rotations are moderately small.

The growing concern about masonry frames and bridges, both those still in use in the Europe and elsewhere and those of merely historical value, has produced a remarkable interest in the development of accurate, reasonably efficient methods for their analysis. Although most of the effort is concentrated on the assessment of single arches, there are also proposals specifically developed for the analysis of more complex structures, such as multispan arch bridges.

Most of the known studies are based on the following techniques of analysis:

(1) classical theories based on ultimate mechanisms;
(2) solid continuum mechanics with constitutive equations for masonry as a whole (continuum model, macro modeling);
(3) solid continuum mechanics combined with joints to simulate sliding or separation between masonry units (micro modeling);
(4) discontinuous deformation analysis (DDA).

4.2 Detailed classification of micro and macro models

Several attempts have been made to categorise computational modelling frameworks for structural masonry, where it's inherent discontinuous nature (unit, joint, interface) needs to be recognised. Perhaps the most appropriate categorisation comes from the "Delft School" (LOURENCO, 1994; LOURENCO, 1996; ROTS, 1997), where three principal modelling strategies are identified

(a) Detailed micro-modelling – units and mortar in the joints represented as continuum, whereas the unit/mortar interfaces are modelled by discontinuous elements
(b) Simplified micro-modelling – "geometrically expanded" continuum units, with discontinuous elements covering the behaviour of both mortar joints and interfaces
(c) Macro-modelling – where all three principal features of structural masonry are represented by an equivalent continuum

4.3 Mathematical model of arch in Atena program

Atena program determined for non-linear analysis on the base of FEM method is specially designed tools for computation simulation of the composite materials behaviour (ČERVENKA, V.; JENDELE, L., 2005). It enables simulate the behaviour both of unreinforced and reinforced structures with different types of reinforcing. Some materials models involve even fracture mechanics. Atena program is possible to use for the solutions of arbitrary numerical problems which can be described by the materials modes composed in the program.

From the presented facts it results that the most convenient model for describing orthotropic non-contiguous character of masonry is a micro-model. This model can describe not only the materials characteristic of individual materials (bricks, mortar), but also their co-acting that is in the mathematical model of masonry considered by 2D contact among the materials. This contact task describes in the best way the behaviour of masonry on the boundary of the masonry units and mortar.

A disadvantage of the micro-modelling is its high time-consuming of computation and extensive number of the physically-mechanical properties to be determined for the material behaviour description and for the contact behaviour description among individual materials. For the experimental results verification 2D and 3D mathematical micro-models are set up. The spatial behaviour of a masonry structure describes 3Dmodel.

Contact task describes the physical properties of a contact between two surfaces. The model of a contact in Atena is based on a model of the dry friction (Mohr-Coulomb) defined by the shear cohesion c and by the friction factor φ (1). Maximum shearing stress is restricted by a linear relation

$$\tau = c + \sigma \, tg\varphi, \tag{1}$$

where σ is a magnitude of the contacting pressure stress (positive value). The contact task is extended in addition by limited damage of the contact by a tension ft. The model has two types of parameters. First set of parameters is describing the real physical properties of interface: tensile strength ft, shear cohesion c and friction coefficient φ. They must correspond to real material properties. The second set of parameters are stiffness coefficients, which serve purely for numerical purpose. There are two stiffness coefficients, Knn (normal), Ktt (shear) and each has two values: basic and minimal. The basic stiffness represents the stiffness of the interface model in close state. Instead of rigid connection, the interface in closed state undergoes displacements according to the stiffness. The higher the stiffness the smaller are these displacements. Under compressive stresses these displacements generate a slight overlapping of interface lines. The minimal stiffness serves only for numerical purposes as “predictor” within the nonlinear interactive solution method.

4.4 Comparison of calculation model with experiments

2D model of vaults fully includes behaviour of micro-model with involvement of contact conditions among mortar and bricks. Models of unreinforced vaults are used especially to checkout parameters of the contact. In the case of reinforced vaults is in the structural model in Atena program also included reinforcement inserted into the chases. 2D model is unable to model precisely the reinforcement behaviour in the chases, unfortunately. For reinforcement is only implemented a presumption about its behaviour, namely by the bi-linear or tri-linear working diagram of steel. Into the calculation is also possible to implement a presumption about the reinforcement coherence with ambient material (bond-slip relation). The presumption about the reinforcement coherence with ambient environmental, in our case the special reinforcement of a helical shape, is possible to express on the bases of performed pull out tests at the BUT-FCE in Brno.

The problem of 2D model still remains in the description of the reinforcement behaviour with the bond as a complex in relation to the ambient masonry. The exact description of this behaviour will only enable 3D model, which will be able to describe also the contact tasks on the boundary between the bond and ambient masonry. Nevertheless, it should be point out that the reinforcement tear off from the chases originated even after the load-bearing capacity spending of a strengthened vault with retrofit reinforcement, i.e. at large displacements caused by such stresses that damaged the contact between the bond and ambient masonry.

In case of unreinforced vaults the computational mathematical models approximate to the behaviour of real vaults. From comparison of tests and mathematical modelling is possible to obtain/identify the physically mechanically characteristics of masonry (mortar, bricks, contacts characteristics).

In the mathematical model of additionally reinforced vault is very important element of reinforcement the mechanical bond with surrounding environment. If in mathematical model is full mechanical bond of reinforcement under consideration, thus there is not redistribution of tension in armature and reinforcement is under stress only in regions where will get to open of contacts. In other section of reinforcement is tension subsequently approximately zero (Fig. 4). With this it is also connected practically the same stiffness of a construction during loading. From experiments is however evident a fall of stiffness at the load rising. As to the mathematical model corresponds to the performed experiments this bond strength have to be included into calculation. Then the tension is redistributed in reinforcement (Fig. 5) that slip and decrease construction stiffness how is displayed in next diagram.

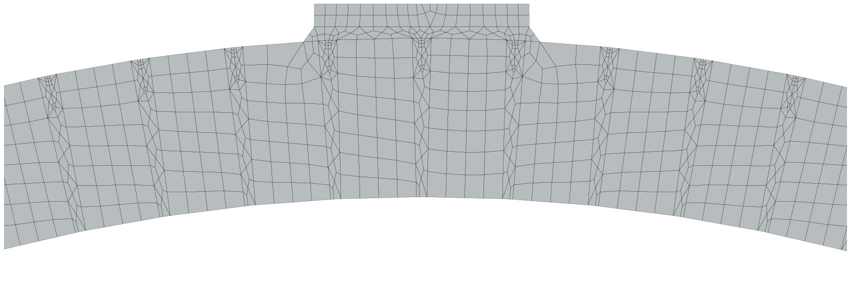

Fig. 3: Geometry of vault 1K2 with finite elements mesh in ½ of the span

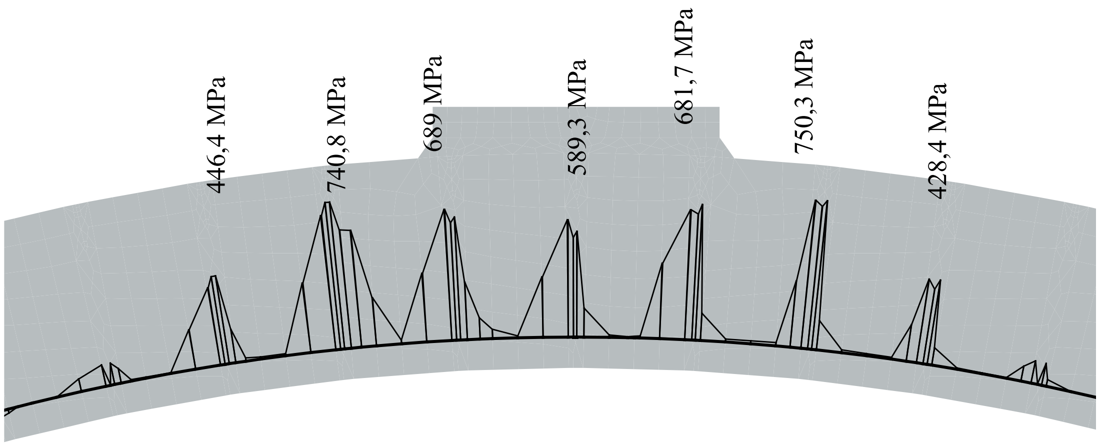

Fig. 4: Tension in reinforcement - full bond strength – vault 1K2

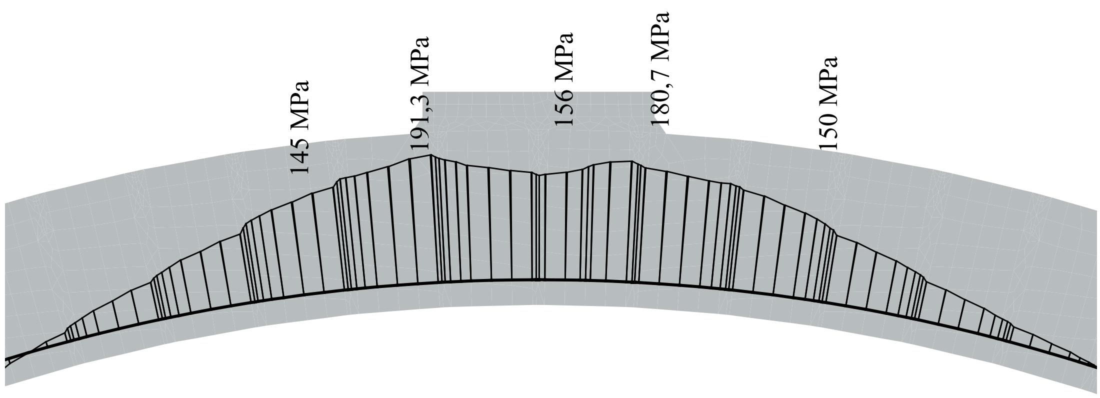

Fig. 5: Tension in reinforcement – with partial bond strength according CEB-FIB Model Code 1990 – vault 1K2

A most precise representation of the structure behaviour is 3D mathematical model. These models are indeed highly demanding on computational time and the work with micro models in 3D environment is also very difficult with regard to large number of the macroelements and contact surfaces.

5 Design algorithm

At present there is no simple normative basis for design of additionally inserted reinforcement for strengthening and/or stiffening of the masonry structures. Some of possibilities of the calculation and design of the masonry reinforced constructions are introduced in Eurocode 6 (1996), ČSN 731101 (2006) and ČSN 731102 (2006).

5.1 General presumptions

For proposal and checking calculation of additional reinforcing of the masonry by non-prestressed reinforcement in the area with tensioned reinforcement and compressed masonry was designed computational algorithm, which is based on the following presumptions (Fig. 6)

- masonry is loaded by combination of bending moment and compressive force and the algorithm is computed only in areas with tensioned reinforcement,
- masonry and mortar do not transfer tension stress,
- strain of the layers in a cross section is directly proportional to the distance of the layers from neutral axes of the cross section,
- largest strain of the layers of the individual materials is achieved at accomplishment at least the one from the following values:
- limit strain ε_{kc} in compressed masonry,
- limit strain ε_{st} in tensioned reinforcement,

- stress in the compressed area of masonry is determined from idealised elastic-plastic diagram expressing the masonry stress and strain dependence; alternatively may be determined providing that the stress in masonry is equal to the compressive strength of masonry and is equally distributed along the height which is equal to 80% of neutral axes distance from extremely stressed layers of the masonry in the cross section, as it is used in designed algorithm,
- stress in reinforcement is determined on the base of idealized elastic-plastic diagram expressing the stress and strain dependence of reinforcement (Fig 6.)

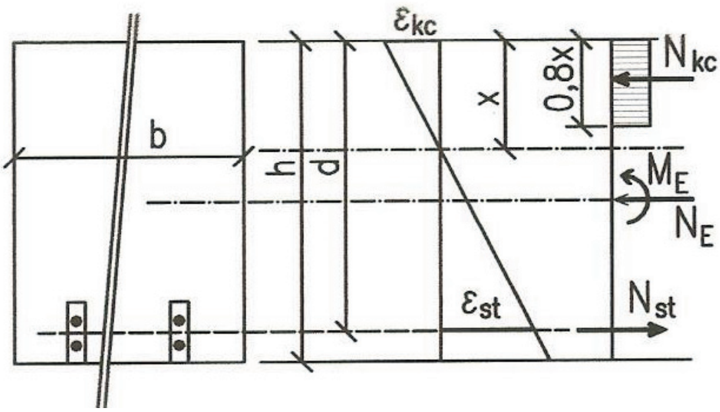

Fig. 6: Presumptions of limit strain method

5.2 Comparison of experiments with designed algorithm

For calculation of the cross section carrying-capacity is used designed algorithm based on above mentioned presumptions. The material characteristics were examined in the course of the tests. The behaviour of the materials is elastic-plastic and it is governed by the idealised working diagrams.

For determination of the characteristic compressive strength of masonry may be used calculation according to EC6

$$f_k = K f_b^{0,65} f_m^{0,25} \quad \text{[MPa]}, \tag{2}$$

where f_k is characteristic compressive strength of masonry, f_b is compressive strength of masonry units (mean value), f_m is compressive strength of common mortar (mean value) and K is appropriate constant according to EC6

For the comparison of algorithm with the experiments is used a mean value of the compressive strength of masonry, which is determined on the presumption of normal distribution of a quantity with standard deviation s=2. Than

$$f_k' = (1,645 * \sigma) + f_k \text{ [MPa]}, \tag{3}$$

where $f_{k'}$ s mean value of compressive strength of masonry. At fulfilment conditions of equilibrium in a cross-section, then

$$N_E = N_U = N_{kc} - N_{st} \quad \text{and} \quad M_E = M_U = N_{kc} * z_{kc} + N_{st} * z_{st}, \tag{4}$$

where N_{kc} is force in compressed masonry $N_{kc} = f_k' * b * 0,8x$, N_{st} is force in tensioned reinforcement $N_{st} = A_{st} * \varepsilon_{st} * E_s$ and z_{kc}, z_{st} are relevant arm of internal forces.

The results presented in Table 6 were obtained with the following input values

- cross sectional area of tensioned reinforcement A_{st}= 38,2 mm^2 for vaults jK2, and A_{st}= 57,3 mm^2 for vaults jK3
- reinforcement modulus of elasticity E_s= 50 GPa
- mean value of compressive strength of masonry f_k'= 6,6 MPa

6 Conclusions

The method of repairs and strengthening of the vaulted bridges using additionally inserted reinforcement has a wide usage. Its application is possible in the cases when in a structure either originates or may originate the tension stresses in unreinforced masonry, whose magnitude is close (or exceeds) to the strength of unreinforced masonry, i.e. in places where the cracks on a construction have been already developed, alternatively when their origin is expected, whereas it may dealt with the strength of masonry in plain tension, in tension in bending or in main tension.

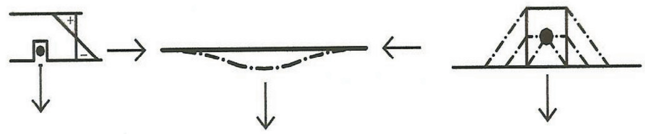

Fig. 8: Failure by rupturing of reinforcement from masonry chases

Except basic models of the failure, when the loss of load-bearing-capacity caused by the compression failure of masonry or the tensile failure of reinforcement and for the masonry structures with additionally inserted reinforcement which is placed in the compressed area, it is necessary to implement also further models for the construction failure: the failure of compressed reinforcement by buckling from chases (Fig. 8). Three partial models of the failure come into the account rapture of reinforcement from filler, rapture of reinforcement with filler from the chases, rapture of reinforcement with filler and with a part of masonry unit.

With that information would be possible to verify the influence of a distance of the chases with reinforcement on the load-bearing-capacity of a member.

On the basis of thus obtained results from numerical studies and on the base of the designed algorithm, it is possible to obtain simple constitutive relations for the evaluation and design of strengthening by simplified designed methods used in the practice.

Table 3: Comparison of designed algorithm with experiments – achieved calculation values

Vault nr.	M_u=M_E kNm	N_u=N_E kN	ε_{st}	σ_{st} MPa	x m	ε_{kc}
1K2	4,7	-19,7	0,0169	845	0,0110	-0,002
1K3	6,2	-25,8	0,0153	762,5	0,0148	-0,002
2K2	4,3	-10,7	0,0176	878	0,0077	-0,001
2K3	6	-15,5	0,0174	869	0,0139	-0,003
3K2	15,1	-316,4	0,0144	717,5	0,0731	-0,028
3K3	17,9	-375,7	0,0138	690	0,0767	-0,032

Annotation: At vault 3K3 supposed increasing of compressive strength of masonry on 7,6 MPa

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