THE BALTIC JOURNAL OF ROAD AND BRIDGE ENGINEERING
2018/13(3)

ISSN 1822-427X/eISSN 1822-4288
2018 Volume 13 Issue 3: 313–330
https://doi.org/10.7250/bjrbe.2018-13.419

STRENGTHENING OF ARCHED MASONRY STRUCTURES BY ADDITIONAL REINFORCEMENT: DESIGN APPROACHES AND COMPARISON TO EXPERIMENTS

MARTIN ZLÁMAL*, PETR ŠTĚPÁNEK

Institute of Concrete and Masonry Structures, Faculty of Civil Engineering, Brno University of Technology, Brno, Czech Republic

Received 18 April 2018; accepted 23 August 2018

Abstract. The primary aim of this article is to present design approaches for calculating the additional strengthening of masonry arches with the use of the Strut-and-Tie model and applicable standards and their comparison to the experiments. Experiments have proven the functionality of the described method of strengthening by additional inserted non-prestressed reinforcement from the face of the vault. The presented method is one of the methods of maintaining historical vaulted masonry structures, and is also used to improve the behaviour of newly designed masonry structures. This method of strengthening has its advantages, especially in the minimization of alterations to the structure and its simplicity of application. To compare the results and verify the vaults behaviour, experiments were performed with using a metallic helical reinforcement and non-metallic composite glass reinforcement. These experiments have demonstrated the significant influence of additional reinforcement on the carrying capacity of masonry vaults. The growth of bearing capacity was more than eight-fold. From a comparison of design approaches to experiments is evident that approaches to the design of additionally strengthened masonry based on valid standards are possible.

* Corresponding author. E-mail: zlamal.m@fce.vutbr.cz

The comparison of results moreover demonstrates the possibility of using approaches based on the Strut-and-Tie model.

Keywords: arched bridge, design approaches, glass fibre reinforced polymer (GFRP), masonry, strengthening, Strut-and-Tie model, vault.

Introduction

Masonry continues to be popular because of the relative simplicity of its application in technical practice. Indeed, the development of improved construction rules for newly designed masonry structures is currently greatly needed, as the conventional approach based on experience is unacceptable today. Also, most methods of carrying capacity assessment and strengthening methods of existing masonry structures are increasingly based on the analysis of mathematical simulations and appropriate (linear and nonlinear) computational models. One method of load-bearing elements strengthening is the application of additional reinforcement in chases in masonry on the bottom sides of vaults. These method provides stiffening and increases the load carrying capacities of individual load-bearing elements. This paper is based on experiments in the field of masonry structure strengthening which have been performed at the Faculty of Civil Engineering, the Brno University of Technology (BUT).

This paper presents the results of the load testing of masonry vaults strengthened with the metallic helical reinforcement system (Figure 1a) and with non-metallic glass reinforcement (glass fibre reinforced polymer (GFRP)) (Figure 1b). This GFRP reinforcement was developed on BUT and practically used on chosen constructions (Ďurech, Štěpánek, & Horák, 2010). This work aims to document the options available for the use of additional reinforcement in the strengthening of masonry structures loaded with the interaction of a normal force and a bending moment. The next aim is to experimentally verify the behaviour of

a) helical steel reinforcement

b) wrapped surface glass fibre reinforced polymer reinforcement

Figure 1. Reinforcement shape

different types of reinforcement - specially-shaped helical profiles and GFRP bars).

The method of additionally inserting of non-prestressed reinforcement is one of the several methods for strengthening of vaulted masonry structures (Alecci, Misseri, Rovero, Stipo, De Stefano, Feo, & Luciano, 2016; Anania, Badalà, & D'Agata, 2013; Borri, Castori, & Corradi, 2011; Fauchoux & Abdunur, 1998; Foraboschi, 2004; Oliveira, Basilio, & Lourenço, 2010; Paeglitis, Paeglitis, Vitiņa, & Igaune, 2013; Tao, Stratford, & Chen, 2011). The presented method allows the strengthening of masonry structures without the necessity for large-scale modifications to the structure of vaults (i.e., the excavation of infill), especially in the case of external applications. This system is capable of redistributing newly originated stresses from loads which act on a strengthened structure. The reinforcement aims to:

- restrict the development of existing cracks,
- prevent the origin of new cracks,
- improve the load-bearing capacity of vaulted masonry structure.

It is also to be noted, that further describe experiments correspond to the stress mode, that induces a combination of axial forces and bending moments and origination of tensile areas in the arched structure. Such a situation may occur, for example by the unbalanced moving of supports, application of the concentrated load. This assumption about the behaviour of the structure corresponds to the presented method for the strengthening of masonry arches and the presented design methods.

1. Description of the experiments

Within the experimental parts of the project, three sets of masonry vaults for various loading types were manufactured (Figure 2a–c, Figure 3a). For the distinction of individual vaults, the notation jK_i was used, where "j" corresponds to the series number (1–3) and "i" to the strengthening method (1–3). The vaults were symmetrically loaded in half span – first series (j = 1), asymmetrically in quarter span – second series (j = 2) and symmetrically in both quarters of the span – third series (j = 3) (Figure 2). Each series consisted of three vaults: a 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 constructed using burnt bricks and lime-cement mortar; the vault width was 890 mm, span 2600 mm, deflection 750 mm and radius 1500 mm. Two bars were embedded into each reinforcing chase. The first part of the experiments was performed with special helically shaped reinforcement with a diameter of 8 mm. A strengthened vault was reinforced with glass armature rebar (GFRP) with a diameter of 6 mm too to verify the behaviour of the tested vaults. Only asymmetrical loading was tested, at quarter span (second series) (Zlámal & Štěpánek, 2010). This glass fibre reinforced polymer reinforcement was simultaneously developed and tested at BUT (Girgle, & Štěpánek, 2016; Horak, Girgle, & Stepanek, 2013; Horák, Zlámal, & Štěpánek, 2014).

The last series of vaults were also loaded by dynamic loading; they have only loaded asymmetrically, at one of the quarter spans (second

a) symmetrical loading in half span

b) unsymmetrical loading in the quarter span

c) symmetrical loading in both span quarters

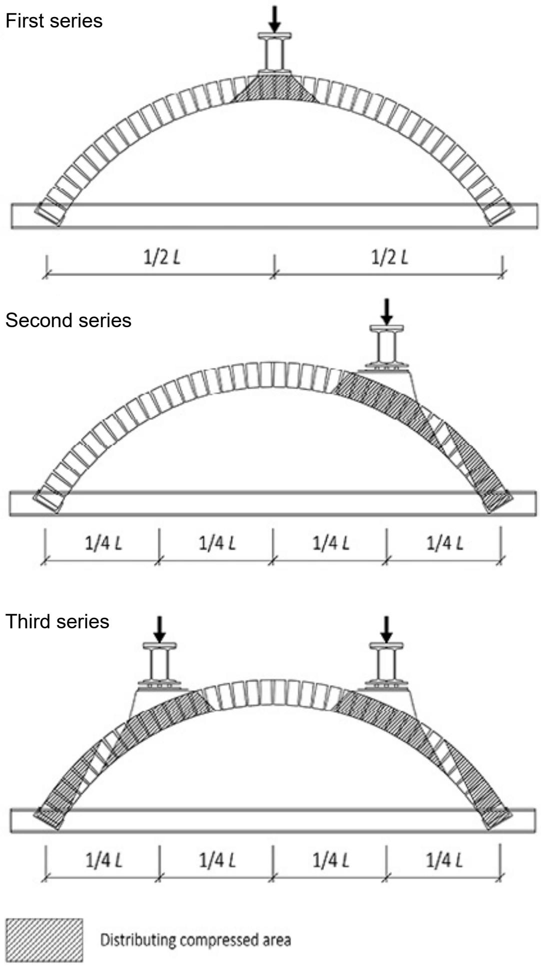
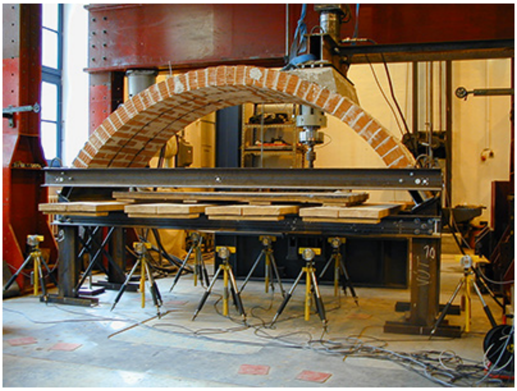
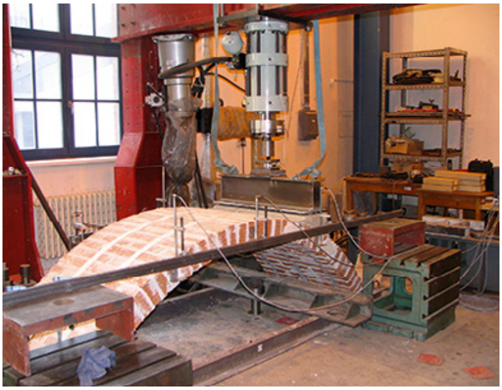


Figure 2. Loading schemes of vaults and the distribution of load in vaults



a) set-up of the static test



b) set-up of the dynamic test

Figure 3. Strengthened vault loaded asymmetrically at the quarter span

series – Figure 2b), because of the maximum influence of the additional reinforcement on the final load-bearing capacity of the vaults. These last series of the vaults were strengthened only with GFRP glass reinforcement. A dynamic hydraulic press initialized dynamic loading, and the deformation of the structure was monitored by inductive displacement transducers (Figure 3b).

1.1. Interpretation of test results – static test

From the comparison of the load-bearing capacities of the individual vaults in the series, it was seen that significant growth in load-bearing capacity was achieved mainly in the case of the first and second series of vaults. Increase of resistance is more than eight-fold. This growth in carrying capacity is observed for both types of reinforcement – helical metallic and GFRP non-metallic (Figure 4). It was related to the vaults loaded in the middle of the span or in the quarter span, where the vaults were stressed by the interaction of normal forces and bending moments. That is why, based on previous experiments, asymmetrical loading at quarter span was selected for vaults strengthened with GFRP reinforcement.

In the case of the third series, the experiments have shown the negligible effects of described strengthening method. The reinforcement did not affect the bearing capacity because the vaults were mainly compressed (Figure 2c). The resultant values of the loading and corresponding deformations for all series of vaults strengthened with metallic reinforcement are presented in previous papers (Zlámal & Štěpánek, 2010).

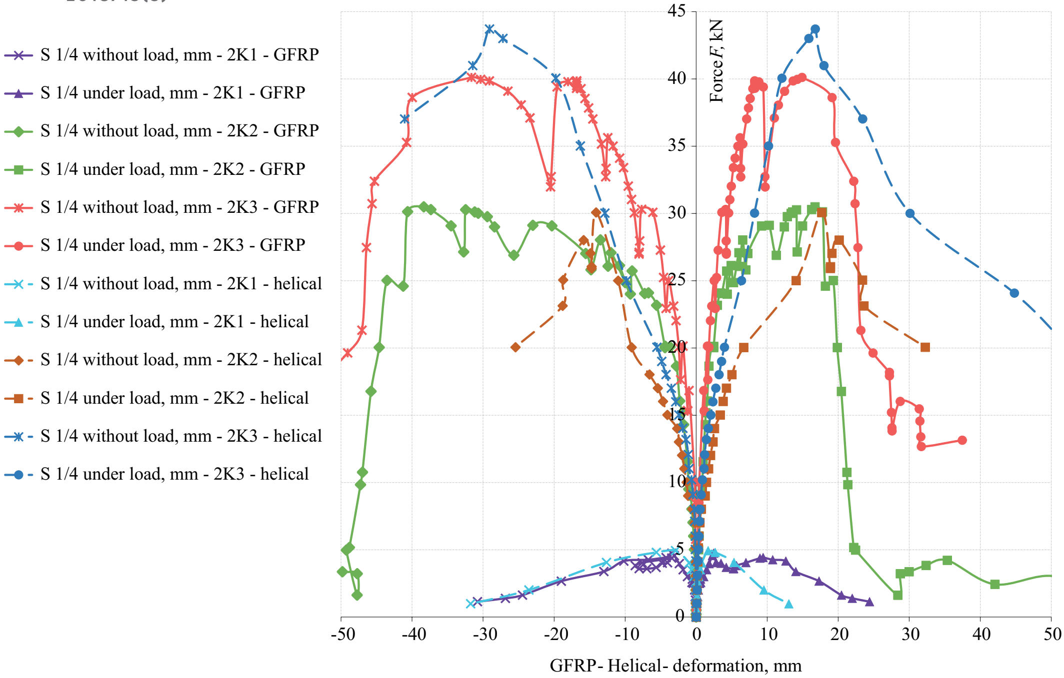


Figure 4. Comparison of deformations in vaults loaded at quarter span and strengthened with glass fibre reinforced polymer and metallic helical reinforcement – static test

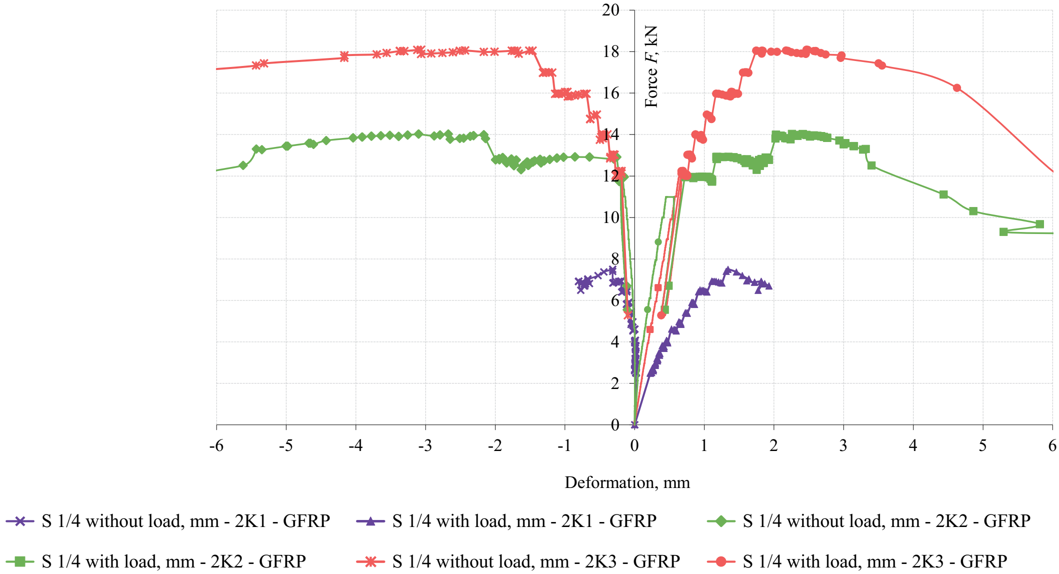
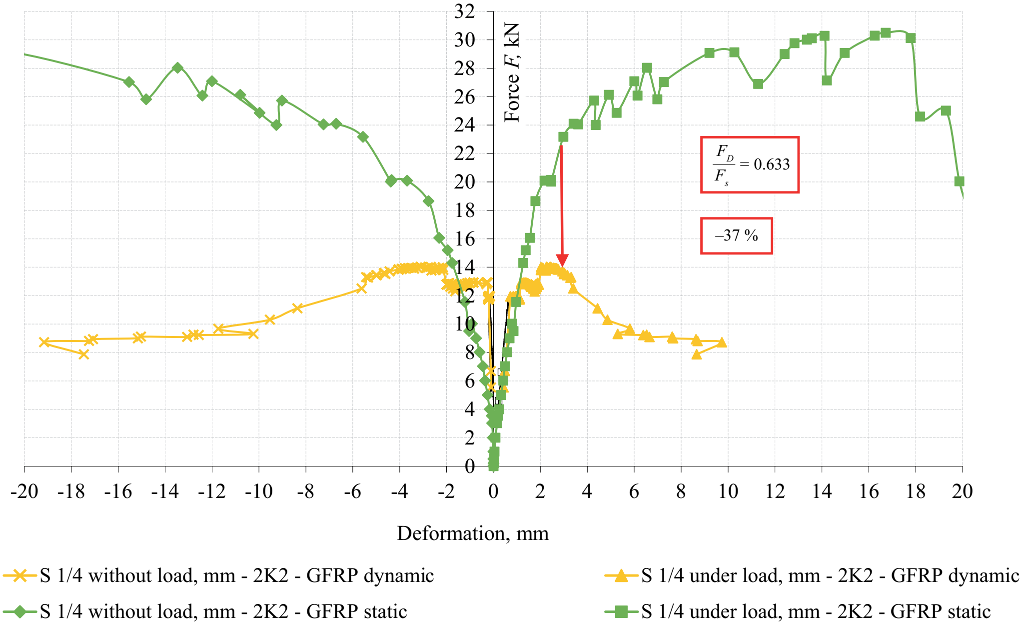
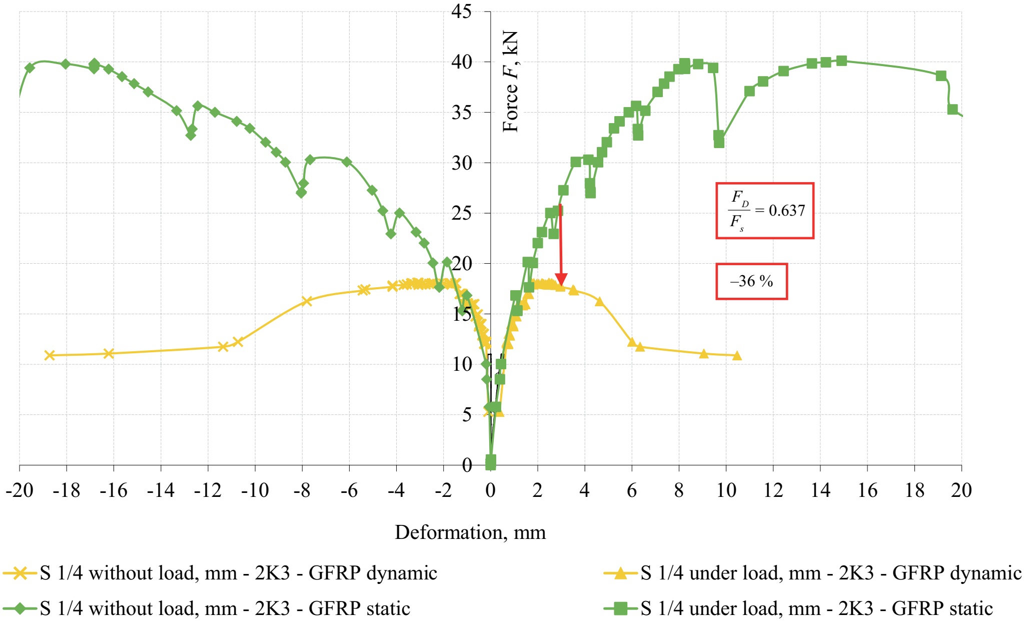


Figure 5. Comparison of deformations in vaults loaded at quarter span and strengthened with glass fibre reinforced polymer reinforcement – dynamic test



a) vaults strengthened by glass reinforcement with two chases



b) vaults strengthened by glass reinforcement with three chases

Figure 6. Comparison of static and dynamic tests

1.2. Interpretation of test results - dynamic test

Dynamic tests were performed on vaults loaded asymmetrically at quarter span and reinforced with glass reinforcement (GFRP). From the results of the dynamic tests, it is again visible that the load-bearing capacity of reinforced vaults (2K2, 2K3) increases compared to vaults which are unreinforced (2K1) (Figure 5).

However, the low number of tested specimens prevented comparison of unreinforced vaults to the test data from static experiments. The load-bearing capacity of unreinforced vaults loaded by dynamic loading is higher in comparison to that demonstrated in the static test. This behaviour probably occurs mainly due to non-homogeneity in masonry.

Strengthened vaults can be partially compared about their load-bearing capacity. The ratio of the load-bearing capacities of dynamically loaded vaults and statically loaded vaults (FD/FS – dynamic coefficient) with two reinforcing chases is 0.633 (Figure 6a) and with three reinforcing chases is 0.637 (Figure 6b). The obtained values of the presented dynamic coefficients correspond to the commonly used values. The comparison is performed for deformation of 3 mm.

2. Design methodology

It is possible to use several approaches for the methodology involved in the design of additionally strengthened masonry vaults. The design and assessment of a structure can be performed based on:

- experiments, eventually supplemented by mathematical models of strengthened structure;
- behavioural similarities between reinforced masonry and reinforced concrete structures, e.g., the Strut-and-Tie model (STM);
- current standards.

All these approaches proved the functionality of the system of masonry vault strengthening from the face side of the vault.

2.1. The Strut-and-Tie model of masonry

In the Strut-and-Tie model (STM) the complex flow of inner forces in a structure is idealised, e.g., trusses transfer a given external load of a structure over individual truss elements to the supports. Nevertheless, both original trusses and STM trusses consist of struts and ties connected to one another in knots (also referred to as knot zones or knot areas).

Struts are compressive members in the STM and represent a compressive field in a structure. Compressive stress passes mainly along the axes of the struts. Ties are tension elements in the STM and mostly represent reinforcement. Though they may also occasionally represent a stress field in a structure where the dominant principal tension stress is in the same direction as the tie. Knots are similar to joints in trusses, and their location is in places where forces are carried between struts and ties.

For a statically relevant stress field in the STM, the external load and reaction (border) forces must be in balance with the inner forces in each knot. Although the STM is known as a model that is applied to reinforced concrete structures, it can also be partially applied to reinforced masonry structures. In practice, the STM is most commonly used mainly for the shear masonry walls confined in reinforced concrete frames (Foraboschi & Vanin, 2013), masonry columns (Campione, Cavaleri, & Papia, 2016) and anchorage zones (Seim & Pfeiffer 2011). However, it must be noted, that when STM for masonry is used, it is necessary to modify assumptions about the behaviour of the STM which usually are valid when it is applied to reinforced concrete structures. These modifications are necessary because of the non-continuous orthotropic character of masonry.

Masonry consists of brick elements which are connected by a mortar which fills the joints among them. In addition, masonry is mainly used in unreinforced structures with low tensile strength. Therefore, the behaviour of the STM for masonry must be considerably modified in cases when a tie in the STM is in an area that is without reinforcement.

2.1.1. Carrying capacity of Strut-and-Tie model elements

Underlying assumptions and rules about the behaviour of the elements in the model are taken from the standard *ACI 318M-2:2001 Building Code Requirements for Structural Concrete – Appendix A: Strut-and-Tie Models*. The load-bearing capacity of compressive struts is defined by the carrying capacity of masonry under pressure, which is given regarding its dependence on the strength of brick elements and mortar. The carrying capacity of ties depends on whether the vault is reinforced. If it is, the carrying capacity of the ties is defined as the carrying capacity of the tensioned reinforcement. If it is assumed that the ties are located where masonry is unreinforced, the carrying capacity is defined as the tensile strength of masonry (brick or mortar) or the tensile strength of the interface between the brick elements and the mortar. It is also necessary to supplement STM assumptions about the behaviour of masonry for trusses parallel to the bed joint. With information regarding the contact conditions at the interface between the brick elements and the mortar the failure mode of masonry in shear along the bed joint is obtained. One of the models, on which the brick-mortar interface model can be based, is the dry friction model (Mohr–Coulomb model), i.e., a model defined by shear cohesion c and friction coefficient φ. A linear relation Eq. (1) limits the maximum shear stress.

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

where τ – shear stress, c – cohesion, σ – normal stress, φ – friction coefficient.

If ties limit values in tension for the unreinforced area are exceeded, than these ties should be eliminated from the STM. Of course, the ties elimination should only be done providing the assumption that the conditions for static balance are fulfilled.

The first model to be created was, therefore, a full truss model concerning geometry and the directions of the principal stresses (Figure 7). Than the STM is loaded in knots. Ties situated in structural areas without reinforcement, and in which the opening of the joints and the origination of cracks occur, are eliminated from the full STM (Figure 8).

The resulting diagram of the STM is thus created precisely by the elimination of ties from the STM and the insertion of joints into the structure. This model modification correspond to the real behaviour of a

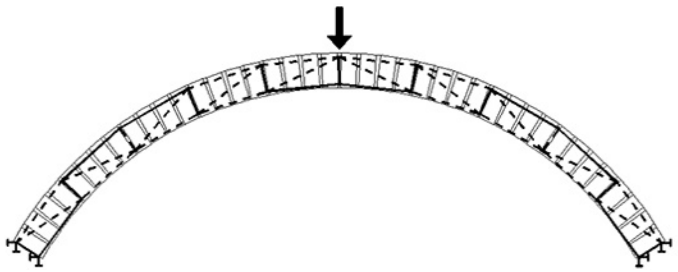


Figure 7. A complete Strut-and-Tie model of the vaults from the first series

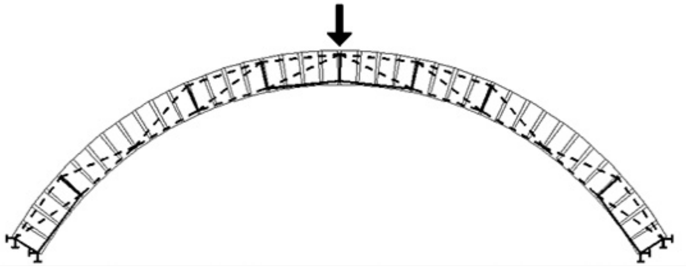


Figure 8. A modified Strut-and-Tie model of the vault from the first series

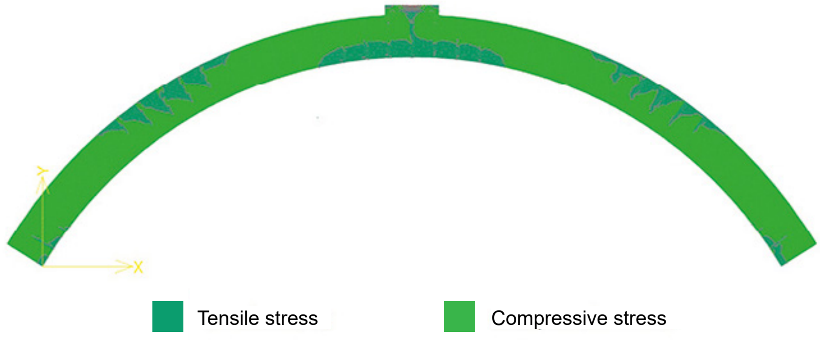


Figure 9. The distribution of principal stresses – the first series of vaults

vaulted masonry structure (Figure 9). In the case of the selected model, this adjustment was enabled by the static indeterminateness of the whole structure because of the pin supports.

2.1.2. Application of the Strut-and-Tie model to masonry vaults

A detailed STM for masonry structures separately describing the behaviour of individual elements (masonry unit, mortar, contact model) is unnecessarily complex. In addition, the results would probably misinterpret the behaviour of the masonry structure. It is, therefore, appropriate to separate the STM into larger entities and use the assumptions described above for the assessment of individual rods in the STM.

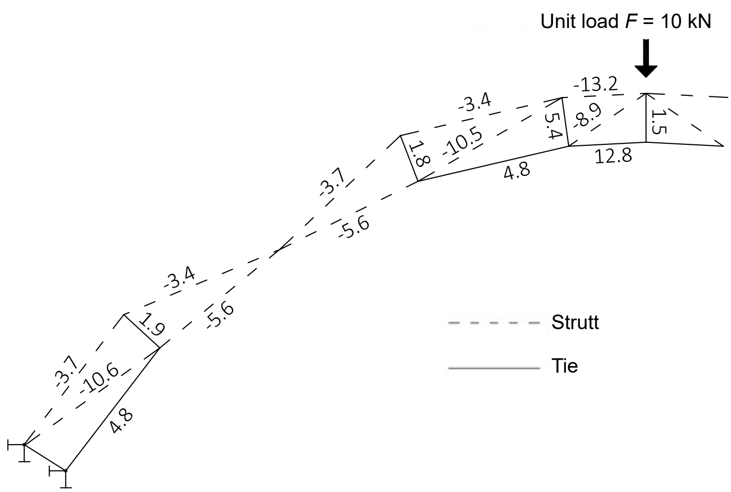


Figure 10. Internal forces determined in the modified Strut-and-Tie model

The internal forces in the STM are determined with the exclusion of ties in areas without reinforcement (Figure 10). To obtain a limiting bearing capacity for a structure, the STM is in the first step loaded by a unit load. Consequently, the inner forces (Ni) in the individual trusses are determined. Also is determined the bearing capacity of individual struts and ties ($N_{i,\lim}$).

The limit bearing capacity of individual elements of the modified STM is determined as follows:

- ties at the bottom face of the vault are represented by the resistance of reinforcement in tension;
- struts are represented by the compressive strength of masonry;
- the ties parallel to the bed joints are represented by resistance in terms of the shear resistance of joints of the dry friction model (Mohr–Coulomb model) Eq. (1).

The bearing capacity of individual struts and ties $F_{i,\lim}$ is determined as a multiple of the coefficient and the applied unit load. The coefficient is derived from the quotient of the individual truss bearing capacity $N_{i,\lim}$ and the achieved internal force in the STM for the unit load N_i Eq. (2). The limiting bearing capacity of the whole structure $F_{\lim}$ is defined as the minimum value of the achieved capacity of individual elements in the modified Strut-and-Tie model Eq. (3).

$$F_{i,\lim} = \frac{N_{i,\lim}}{N_i} \cdot (unit\ load);\ for\ i = 1 \sim n \tag{2}$$

$$F_{\lim} = \{\min F_{1,\lim}, F_{2,\lim}, \ldots, F_{n,\lim}\}, \tag{3}$$

where $N_{i,\lim}$ – bearing capacity of an individual element of STM, N_i – force in the individual element of STM developed by the unit load, $F_{i,\lim}$ – limit force in the individual element of STM, $F_{\lim}$ – bearing capacity of the whole structure.

2.1.3. Comparison of the Strut-and-Tie model to experiments

From the presented comparison of the selected STM with experiments (Table 1), it is evident that the STM discussed here is suitable and successfully describe the behaviour of reinforced vaulted structures stressed by a combination of normal forces and bending moments.

The ultimate carrying capacity of the vaults was reached when ties failed at the location of the tensioned reinforcement. Failure mode corresponds to the behaviour of the experimentally tested vaults, and the achieved values are approximate to the carrying capacity of the experimentally tested vaults.

Table 1. Comparison of the Strut-and-Tie model to experiments – achieved calculated values

Material	Vault No.	Limit loading		
		Strut-and-Tie model, kN	Experiment, kN	Difference, %
Helical steel	1*K*2	27.9	30.5	8.5%
	1*K*3	41.9	40.0	4.6%
	2*K*2	25.7	30.2	14.9%
	2*K*3	38.5	43.7	11.8%
Glass fibre reinforced polymer	2*K*2	24.8	30.6	19.0%
	2*K*3	37.1	40.1	7.5%
Average difference				11.1%

2.2. The designed algorithm

At present, there is no simple normative basis for the design of additionally inserted reinforcement for the strengthening and stiffening of masonry structures. Some of the options for the calculation and design of reinforced masonry structures are mentioned in *EN 1996-1-1 + A1:2013 Eurocode 6: Design of Masonry Structures – Part 1-1: Common Rules for Reinforced and Unreinforced Masonry Structures*.

2.2.1. General assumptions

A computational algorithm for the design and evaluative calculation of masonry with additional non-prestressed reinforcement was designed based on the following assumptions (Figure 11):

- masonry is loaded by a combination of bending moment and compressive force; the algorithm is computed only for areas with tensioned reinforcement;
- masonry and mortar do not transfer tensile stress;
- the strain of the layers in a cross-section is directly proportional to the distance of the layers from the neutral axes of the cross-section;
- the limit strain of the layers is achieved in at least the one of the individual materials;
- the stress in the reinforcement is determined based on an idealised elastic-plastic diagram expressing the stress and strain dependence of the reinforcement.

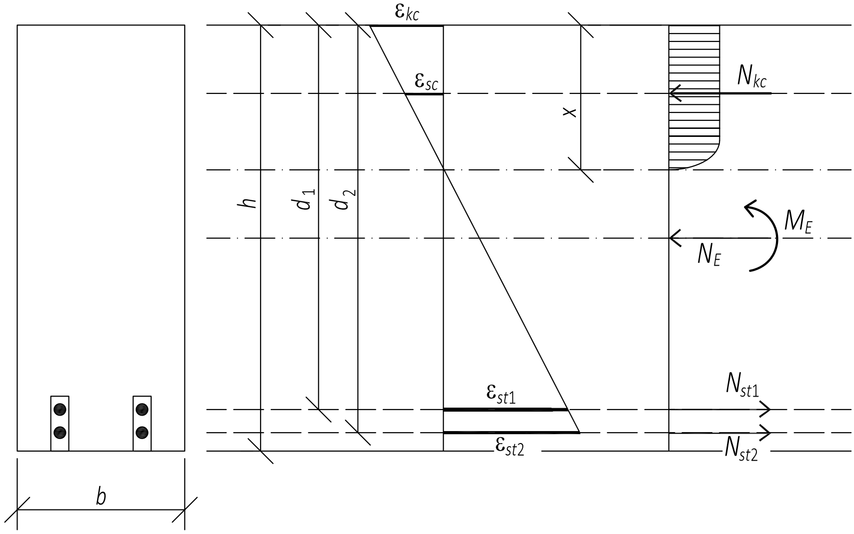


Figure 11. Assumptions of the limit strain method

2.3. Comparison of experiments to the designed algorithm

The algorithm designed by the assumptions mentioned above was utilised for the calculation of the cross section carrying capacity. The characteristics of the investigated materials were examined in the course of the tests. The behaviour of the materials is elastic-plastic, and idealised stress-strain diagrams govern it.

For the determination of the characteristic compressive strength of masonry, the following calculation Eq. (4) according to *EN 1996-1-1 + A1:2013* is used:

$$f_k = K \cdot f_b^{0.7} \cdot f_m^{0.3}, \tag{4}$$

where f_k – the characteristic compressive strength of masonry, MPa, f_b – the normalised compressive strength of masonry units (mean value,) MPa, f_m – is the compressive strength of general purpose mortar (average value), MPa, K – is a constant according to *EN 1996-1-1 + A1:2013*.

The results presented in Table 2 were obtained with the following input values:

- the cross-section area of tensioned steel helical reinforcement A_{st} = 38.20 mm^2 for vaults $jK2$, and A_{st} = 57.30 mm^2 for vaults $jK3$,
- the cross-section area of tensioned GFRP reinforcement A_{st} = 101.24 mm^2 for vaults $jK2$, and A_{st} = 151.86 mm^2 for vaults $jK3$,

Table 2. Comparison of the designed algorithm to experiments – achieved calculation values for helical and glass fibre reinforced polymer reinforcement

Material	Vault No.	Limit loading			
		$N_{Rd} = N_{Ed}$, kN	M_{Ed}, kN	M_{Rd}, kNm	Difference, %
Helical steel	1K2	-20.6	4.7	4.93	5.8%
	1K3	-26.4	6.1	6.72	10.2%
	2K2	-17.3	5.0	4.73	5.4%
	2K3	-24.4	7.2	6.85	4.9%
	3K2	-413.0	15.8	16.24	2.8%
	3K3	-487.0	16.7	16.34	2.2%
Glass fibre reinforced polymer	2K2	-17.5	30.6	4.59	4.3%
	2K3	-22.9	40.1	6.28	8.3%
Average difference					5.5%

- the GFRP reinforcement modulus of elasticity E_{GFRP} = 50 GPa,
- the average value of the compressive strength of masonry f_k'= 7.5 MPa is determined with the assumption of a normal distribution.

The values presented in Table 2 are obtained assuming the equilibrium of normal forces and bending moments in the critical cross-section (in the case of the presented experiments, critical cross sections correspond to the concentrated load position). The normal force N_{Ed} and the bending moment M_{Ed} corresponds to the internal forces in the vault when the maximum load is reached. The force N_{Rd} and the bending moment M_{Rd} then corresponds to the determined cross-sectional resistance. While calculating the force N_{Rd} is set equal to N_{Ed} and the bending moment M_{Rd} is calculated under the conditions of the general assumptions mentioned above so that the balance of forces applies.

From the presented comparison of bearing capacity acquired from the experiments and the calculation according to *EN 1996-1-1 + A1:2013* to average difference 5.5% (Table 2), it is evident that the approach specified in the standard applies to assess the load-bearing capacity of the vaulted masonry structure strengthened with additional reinforcement.

Conclusions

1. From the experiments, it is evident that reinforcement has an influence on the load bearing capacity of a structure, namely in the case of concentrated loading, asymmetrical loading or (in the

case of damaged structures) such phenomena as cracks, degraded materials, overloading or support the movement. In the case of undamaged, uniformly loaded structures without cracks, the influence of this type of additional strengthening is insignificant.

2. From a comparison of design approaches to experiments is evident that approaches to the design of additionally strengthened masonry based on valid standards are possible. The comparison of results moreover demonstrates the possibility of using approaches based on the Strut-and-Tie model.
3. It is necessary to highlight the fact that the method of additional strengthening of masonry structures described in this text is not a method dependent on any specific material base. It is possible to make the general claim that the test results are valid for random additionally applied reinforcement; the actual physic mechanical characteristics of the materials used for strengthening are of course decisive.
4. In principle, a correctly designed vaulted construction should fulfil three fundamental premises to be used safely:

- the immovability of supports is ensured;
- the resultant force of the inner forces passes through the core of the cross-section (at least for significant design conditions);
- any concentrated load on the vault is limited or eliminated.

5. The method of repairing and strengthening of vaulted masonry bridges and structures via the use of additionally inserted reinforcement has a broad range of uses. Its application is possible and appropriate namely in cases when compliance with the three fundamental prerequisites for the correct design of vaulted masonry structures does not occur.

Acknowledgement

This paper has been worked out under the project No. LO1408 "*AdMaS UP – Advanced Materials, Structures and Technologies*", supported by Ministry of Education, Youth and Sports under the "*National Sustainability Programme I*" and project No. FV10588 – "*New Generation of High-Performance Spatial Precast Structures with Improved Mechanical Resistance and Durability*", supported by the Ministry of Industry and Trade. The experiments were performed in cooperation with Institute of Metal and Timber Structures of the Brno University of Technology.

REFERENCES

Alecci, V., Misseri, G., Rovero, L., Stipo, G., De Stefano, M., Feo, L., & Luciano, R. (2016). Experimental investigation on masonry arches strengthened with PBO-FRCM composite. *Composites Part B: Engineering*, *100*, 228-239. https://doi.org/10.1016/j.compositesb.2016.05.063

Anania, L., Badalà, A., & D'Agata, G. (2013). The post strengthening of the masonry vaults by the Ω-Wrap technique based on the use of C-FRP. *Construction and Building Materials*, *47*, 1053-1068. https://doi.org/10.1016/j.conbuildmat.2013.05.012

Borri, A., Castori, G., & Corradi, M. (2011). Intrados strengthening of brick masonry arches with composite materials. *Composites Part B: Engineering*, *42*(5), 1164-1172. https://doi.org/10.1016/j.compositesb.2011.03.005

Campione, G., Cavaleri, L., & Papia, M. (2016). Advanced Strategies and Materials for Reinforcing Normal and Disturbed Regions in Brick Masonry Columns. *Journal of Composites for Construction*, *20*(4), 04016013. https://doi.org/10.1061/(ASCE)CC.1943-5614.0000660

Ďurech, D., Štěpánek, P., & Horák, D. (2010). Renovation of a bridge using GFRP reinforced concrete slab. In *Proceedings of the CESB 2010 PRAGUE - Central Europe towards Sustainable Building 2010: from Theory to Practice*, 30 June – 2 July 2010, Prague, Czech Republic. 1-13.

EN 1996-1-1 + A1:2013 Eurocode 6: Design of Masonry Structures - Part 1-1: Common Rules for Reinforced and Unreinforced Masonry Structures

Fauchoux, G., & Abdunur, C. (1998). Strengthening masonry arch bridges through backfill replacement by concrete. In *Proceedings of the 2nd International Arch Bridge Conference, Sinopoli ed.(Balkema, Rotterdam, 1998)* (pp. 417-422).

Foraboschi, P. (2004). Strengthening of masonry arches with fiber-reinforced polymer strips. *Journal of composites for construction*, *8*(3), 191-202. https://doi.org/10.1061/(ASCE)1090-0268(2004)8:3(191)

Foraboschi, P., & Vanin, A. (2013). Non-linear static analysis of masonry buildings based on a strut-and-tie modeling. *Soil Dynamics and Earthquake Engineering*, *55*, 44-58. https://doi.org/10.1016/j.soildyn.2013.08.005

Girgle, F., & Štěpánek, P. (2016). An anchoring element for prestressed FRP reinforcement: simplified design of the anchoring area. *Materials and Structures*, *49*(4), 1337-1350. https://doi.org/10.1617/s11527-015-0580-z

Horak, D., Girgle, F., & Stepanek, P. (2013). Effect of Concrete Cover on Developing Lengths of FRP Bars. In *Proceedings of the CESB 2013 PRAGUE - Central Europe towards Sustainable Building 2013: Sustainable Building and Refurbishment for Next Generations*, 26–28 June 2013, Prague, Czech Republic. 357-360.

Horák, D., Zlámal, M., & Štěpánek, P. (2014). Behavior of FRP- RC Members During Exposure to Fire, *Special concrete* and Composites 2014 - *Advanced Materials Research,* 1054: 27-32. https://doi.org/10.4028/www.scientific.net/AMR.1054.27

Oliveira, D. V., Basilio, I., & Lourenço, P. B. (2010). Experimental behavior of FRP strengthened masonry arches. *Journal of Composites for Construction*, *14*(3), 312-322. https://doi.org/10.1061/(ASCE)CC.1943-5614.0000086

Paeglitis, A., Paeglitis, A., Vitiņa, I., & Igaune, S. (2013). Study and Renovation of Historical Masonry Arch Bridge, *Baltic Journal of Road and Bridge Engineering*, 8(1): 32-39. https://doi.org/10.3846/bjrbe.2013.05

Seim, W., & Pfeiffer, U. (2011). Local post-strengthening of masonry structures with fiber-reinforced polymers (FRPs). *Construction and Building Materials*, *25*(8), 3393-3403. https://doi.org/10.1016/j.conbuildmat.2011.03.030

Tao, Y., Stratford, T. J., & Chen, J. F. (2011). Behaviour of a masonry arch bridge repaired using fibre-reinforced polymer composites. *Engineering Structures*, *33*(5), 1594-1606. https://doi.org/10.1016/j.engstruct.2011.01.029

Zlámal, M, & Štěpánek, P. (2010). Strengthening of Masonry Vaulted Structures. In *Proceedings of the CESB 2010 PRAGUE - Central Europe towards Sustainable Building 2010: from Theory to Practice*, 30 June – 2 July 2010, Prague, Czech Republic. 221-224.