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Survey and preliminary assessment of a baroque vault for refurbishment planning

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Abstract: Designing the modification of buildings for a new use is a costly and time-consuming process, before which it is often necessary to conduct a preliminary assessment of the feasibility of the intended project. The article discusses the procedure for such a preliminary cost-effective assessment using the example of a historic building with vaulted ceilings. For older historical buildings, archival documents specifying their geometry, construction details and material characteristics are typically not available. For the preliminary design, it is possible to obtain information about the dimensions of the load-bearing elements non-destructively. However, the knowledge of the exact material properties of the masonry exceeds the possibilities of an inexpensive assessment, which must take advantage of the data available in the literature for similar structures. This article presents a methodology and an example of the measurement and assessment of the mechanical response of the Baroque vault to static loading. It shows the process of obtaining data on the geometric shape of the structure by means of geodetic surveying and geometric radar measurements of the thickness of vaults whose upper faces are not accessible. Two models are used to calculate the internal forces in the structure. The first planar model considers a simplified barrel vault strip according to the focused geometry with respect to geometric nonlinearity and without tension, the second model is a linear spatial model of the entire vault. The calculation was performed in ANSYS 17.2, the input parameters of the material are taken from the literature. The calculated stresses in the vault are compared with the values of the design strength of brick masonry, taking into account the characteristic compressive strength of the masonry based on published known data obtained during tests of similar historical materials. The results are then used to decide on the feasibility of the intention to adapt the building for a new use.

Keywords: baroque vault, non-destructive survey, mechanical response, load carrying capacity estimate

INTRODUCTION

Care for the long-term sustainable preservation of a quality environment is increasingly rising interest in extending the functionality of existing buildings and new uses. The designer must thus meet the requirements for design renovation, conversion, alteration, rehabilitation, adaptation, retrofitting, improvement or modernisation of the buildings. These modifications can generally be included under the umbrella term refurbishment. Refurbishing and repurposing buildings is nothing new in history, as in many cases these were the best solutions for buildings that had lost their function or become obsolete. They are activities that, together with maintenance, have accounted for almost fifty percent of construction production in the last twenty years in Europe. This trend is likely to grow, as it is estimated that by the middle of our millennium, eighty percent of the buildings occupied will be buildings standing today. Refurbishment has a number of advantages over new construction or the demolition and rebuilding of structures. In addition to the economic benefits of reducing costs and shortening construction time, environmental benefits are also obtained, especially the re-use of brownfields and substantial reduction of waste production. Simplifying the preparation and permissions for the construction and, last but not least, the preservation of cultural heritage and its use for the creation of a high-quality life also seem important. More recently, refurbishment planning has included carbon footprint assessment and energy performance solutions (Konstantinoua, Knaack, 2011; Tejedor et al., 2022). There are also potential disadvantages associated with refurbishment, which primarily entail requirements for design interventions that make buildings structurally safe and robust (UKEssays, 2018). Here, the designer often encounters a lack of information about geometry, materials and construction details, which affects not only the design process, but also brings uncertainties in assessing reliability and often problems when carrying out construction work (Lorenzoni et al., 2016). It is therefore necessary to devote sufficient time and financial resources to comprehensive surveys using modern means of non-destructive or gently destructive testing.

A comprehensive and detailed survey, including reliable monitoring of the condition of a historic structure, is a costly and time-consuming task for which investors usually do not have the resources when deciding whether and how to use an existing building. They need to know indicative data on the suitability of the building for the intended new use. Estimating the functional potential of a building in terms of its construction quality is essential information for deciding on the feasibility of the project, the necessary scope of interventions and the resulting costs. Therefore, it is useful to perform a preliminary assessment of the response of the structure to the applied load in the first step, using an appropriate low-cost approach. The aim of the preliminary assessment is not a detailed analysis of the condition of the structure in question and its behaviour under load, which would normally require a costly comprehensive survey with sampling of building materials and the most accurate determination of their properties. In order to decide on the implementation of a new use project, a less costly preliminary assessment of the load-bearing structures with respect to the load is sufficient in the initial phase, which will reveal their quality and the need for possible reinforcement if the load-bearing capacity is not adequate. An example of such an affordable procedure is presented in the following paragraphs.

ASSESSMENT CONDITIONS AND METHODOLOGY

The intention to change the use in selected areas of the former Jesuit College in Telč on Zachariáše z Hradce Square No. 2 in the Czech Republic required a preliminary assessment of the behaviour of the vaults therein subjected to the intended live load of 5 kNm-2. The contracting authorities provided copies of the construction drawings of the premises in question for analysis. However, the construction documentation did not contain detailed data on the geometry of the vaults, which would have allowed the creation of a computational model. Furthermore, documentation on the thickness of the vaults was unavailable, as was the height of the embankments and the dimensions and composition of the floor layers above the vaults. The material properties of the building materials used for the construction of vaults were also unknown.

Consequently, the analysis was preceded by local investigations, the aim of which was to survey the geometric shape and to non-destructively examine –by using georadar– the composition of inaccessible parts between the lower surface of the vaults and the upper surface of the ceiling. The geometry of the extrados and intrados of the vaulted ceiling was precisely determined by geodetic surveying. The internal composition was estimated according to the response of passing electromagnetic waves. Material properties were estimated based on the experience of the authors with similar historical buildings and a literature search. The data so obtained were used for the analysis. The calculated stresses were tentatively compared with the estimated computational strength of masonry. Therefore, the present analysis does not replace the design static calculation of vaults and must be understood as a highly qualified expert estimate of the behaviour of the structure on acting and considered loads. The results are naturally usable as input data in the processing of a detailed static calculation by a future designer.

GEOMETRY OF THE VAULTS

The geometry of the lower surfaces of the vaults was measured by an authorized surveyor and the measured vaults are shown in Fig. 1. Figure 2 shows the measured values of the vertical coordinates of the shape of the edges of the passage of individual vaults from rooms 1.17 and 1.18 and their ground plan. During the local investigation for radar measurements, it was revealed that the thicknesses of the perimeter walls on individual floors are not the same; the division of rooms by transverse walls is not identical either. For the calculation, the vaults were spatially modelled according to their undivided shape on the floor below the vaults.

Fig. 1. Barrel vaults with lunettes under room 1.11 (top), cross vaults under room 1.11 and room 1.12 (middle), barrel and cross vaults under rooms 1.17 and 1.18 (bottom). (Photo: Válek, 2021)

Georadar, ground radar, or GPR (short for Ground Penetrating Radar) is an instrument that uses a radar signal to survey and then display the internal structure under the surface being examined. It is a geophysical method that can be used for a wide range of non-destructive exploration tasks such as those involved in laying down the foundation of buildings, archaeology, or the detailed exploration of the subsurface defects of buildings. The basic principle of measurement is the pulsed emission of electromagnetic waves (< 1 ns), which passes through the given environment (the material under study). In engineering and construction applications, frequencies ranging from 300 MHz to 2.5 GHz are used. Reflected waves are received back by the antenna and displayed according to the time of arrival and in relation to the position of the antenna. Reflection or alteration of the signal occurs when it passes through an environment with different permittivity (e.g. rock-air interface) and conductivity (e.g. presence of iron fasteners).

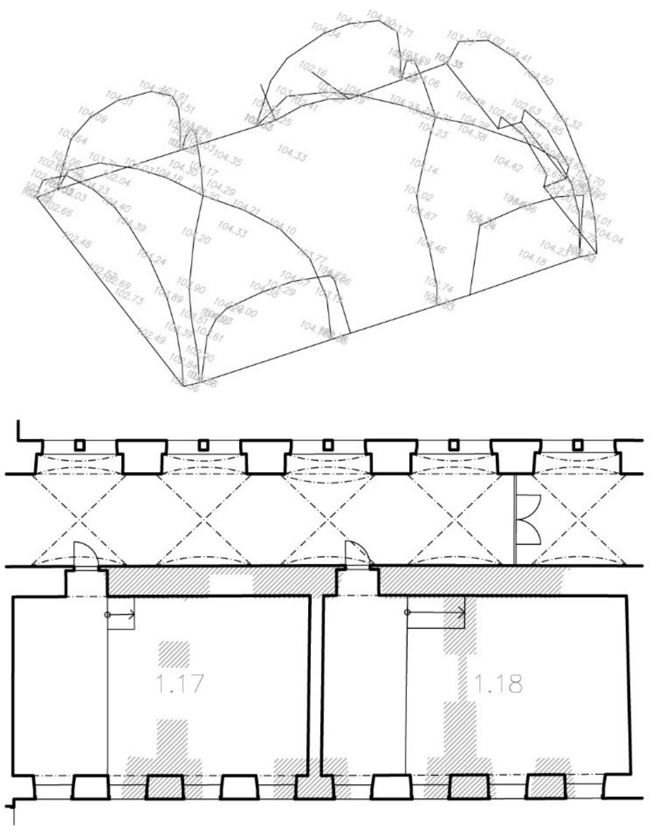

Fig. 2. The shape of the vault edges under rooms 1.17 and 1.18 (top), floor plans of the rooms concerned, with the indication of the supporting walls on the floor below. (Source: Zadražil, Buzek, 2021)

The parameter measured is time. If the speed of wave propagation in a given material or materials is known, then the depth where the signal changes or reflections occur can be determined. When measuring, the method most commonly used is the movement of the antenna in a straight line in the plane that relates to the surveyed object. If there is a different object or cavity in the material environment being examined, the so-called non-homogeneity, from which the sent waves are reflected, then when approaching the antenna, the distance determining its depth is shortened and when it is zoomed out, this distance is subsequently extended. In radargrams, these reflections create hyperbolas. For subsequent interpretation and localization of findings, it is necessary to know the position of the antenna during measurement. The antenna displacement is most often recorded by a measuring wheel, which measures the linear distance from the starting point.

For the radar measurements the space of room 1.18 was divided into 6 longitudinal and 4 transversal mutually perpendicular straight line profiles, which formed a rectangular grid. The longitudinal profile guidance was adapted to the position of the windows in the room (Fig. 3). The starting point was located in the northeast corner of the room at a distance of 300 mm from the walls. The IDS type ground radar was used for the task. With regard to the resolution and the expected depth range, a 900 MHz and a 2 GHz antenna was used. The linear distance when the antenna was moved along the profile was measured by a wheel. After initial authentication, the following system settings were selected: A depth resolution of 1,024 samples/scan was chosen for radar scans, the time limit was 60 ns for the 900 MHz antenna and 20 ns for the 2GHz antenna, the signal velocity was estimated at 140 mm/ns and the signal was read by 10 mm in the direction of the antenna movement.

Fig. 3. Room 1.18 (yellow) and measurement area (orange). The red dots show the initial positions of the longitudinal (L) profiles, which were positioned according to the central axis of the windows. (top) Orientation marking of the square network of longitudinal (L) and transverse profiles (T) in room 1.18. (Author: Válek, 2021)

Slice software was used for evaluation. To visualize the reflections, the signal was linearly amplified with depth, an interval of displayed frequencies was selected and the distance between the antenna and the top was subtracted. To better visualize non-homogeneities and eliminate noises, the signal was "smoothed out" by averaging over 3x3 boxes. From the course of hyperbolic reflections, the speed of signal propagation was retrospectively refined. These were 120 mm/ns (2 GHz) and 110 mm/ns (900 MHz) in room 1.11, and 130 mm/ns (2 GHz) and 120 mm/ns (900 MHz) in room 1.18 and were considered average for these two environments. In general, the depth determination of the signal reflection given in radargrams based on the average is only indicative, as it depends on the actual velocity of signal passage through the material, which can be inhomogeneous in depth and along the profile. However, the degree of simplification is normally suitable for the required resolution. The selected radargrams are presented in Figs. 4 and 5.

A vault is visible on all longitudinal profiles (L10005–10). Its top is at a depth of about 40 cm in the northern part of the room and about 50 cm in the southern part of the room (room 1.18 is above two rooms on the ground floor, which have different vaults). The profiles captured the main vault (L10005, L10007, L10010) and the vault with lunettes (L10006, L10008). On the L10007 profile it is possible to document the structure section. Parallel reflection with the floor at a depth of about 8-10 cm can be associated with a modern structure, which includes a concrete slab with reinforcing bars spaced at 15 cm. The thickness of the vault structure is about 25 cm at the top. It is quite homogeneous without significant reflections: it might consist, for example, of brick masonry.

The L10010 profile shows the ceiling structure above the next room. The interface of the new floor structure is at a depth of approximately 22 cm. The vault structures are of a similar character. Radargrams showed vault structures in all the rooms studied. The density of the selected network allows them to be described sufficiently. In all rooms a concrete slab is probably present with reinforcement in both directions. The tops of the vaults are at a depth of 40–50 cm under the floors of rooms 1.11 and 1.18. This interpretation is a qualified guess based on measured data. For display, the depth signal amplification was used as the average over the entire length of the profile, as well as the average signal pass rate. This setting corresponded to a simplified model that was suitable for the situation. However, when interpreting depth data or comparing signal intensity, it was necessary to take into account the fact that the environment examined was not ideally homogeneous.

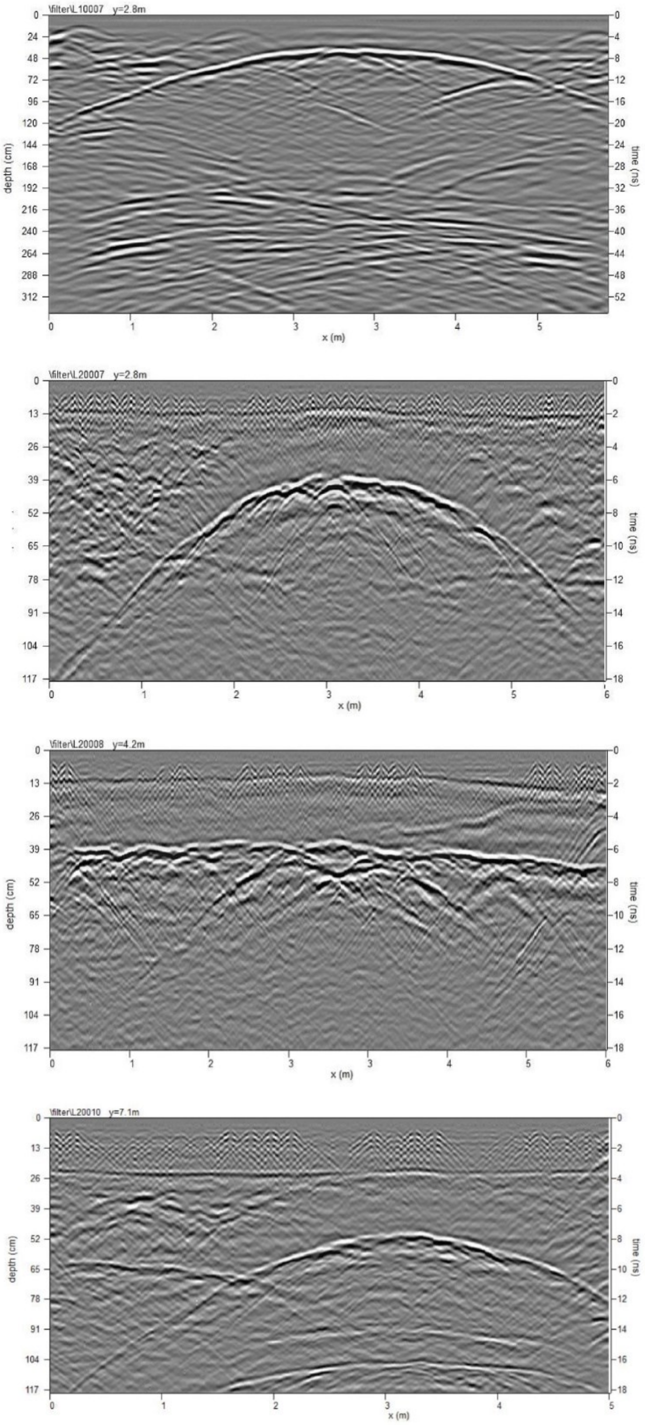

Fig. 4 (left). Radargrams in room 1.18 – profiles along the long direction. The upper two records (profiles L1/L20007 show the barrel vault in perpendicular section measured with different frequencies; the higher-frequency antenna allows to distinguish masonry from the infill material and the floor texture. Profile L20008 runs across the top of the side lunettes of the vault. Profile L200010 is a section through the vault in the adjacent room. (Author: Válek, 2021)

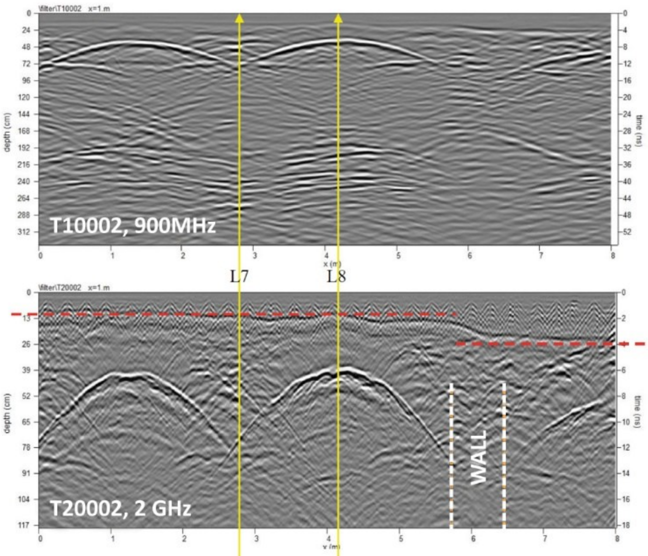

Fig. 5. A vault is visible on all transversal profiles. (Author: Válek, 2021)

STRESS ANALYSIS

For the stress calculation in the vaults under room 1.17/1.18, both the above-mentioned estimated dimension of the vault thickness of 25 cm and the more likely dimension of 30 cm were used. In addition, radar-estimated thicknesses of embankments and floor layers were used. As for the vaults, these have relatively complicated material properties with a rugged spatial geometry in which imperfections can have a significant impact on the stress distribution in the structure and the calculation should therefore ideally consider material and geometric nonlinearity. The demands on such a calculation are disproportionately high in comparison with the importance of the construction being considered, and therefore the calculation using two simplified models was chosen. The first planar model considers a strip of barrel vault with a width of 1 m without taking into account the load from the transverse vault and is made according to the focused geometry taking into account geometric nonlinearity and excluding tension. The second model was a linear spatial model of the entire vault according to the geodetic survey (see Fig. 2) in room 0.22A. The resulting stresses to be compared to standard limits were chosen from both models such as to be on the safe side according to engineering judgement. The calculation was performed with ANSYS 17.2 using SHELL181 and MASS21 finite elements. Input parameters of the calculation are the following:

Room floor 22: ±0.12 m

Room floor above room 22: + 5.0 m

Vault thickness: 0.25 m (0.3m), ρ_c = 1,900*1.1 = 2,100 kg/m^3

Thickness of backfill: 0.7 – 3.0m ρ_z = 1,800*1.1 = 1,980 kg/m^3

Thickness of floor above room: 0.12 m ρ_b = 2,400*1.1 = 2,640 kg/m^3

Live load: p = 5.0 * 1.1 = 5.5 kN/m^2

Modulus of brick elasticity: $E_c = 5.0$ GPa

The input data adopted and the ANSYS software generate results which correspond with the measurements acquired during full scale tests of masonry vaults (Khorkov et al., 2023). The new live load represents approximately 25% of the weight of the vault structure loaded under its own weight, including backfill and floor. For the planar model, the cross-section geometry was taken from the geodetic measurements for points 310–320. A strip of barrel vault with a width of 1 m was considered. The barrel vault in the longitudinal direction is relieved by an oblique vault in the transverse direction leading to the windows and doors. In a planar model, it is therefore necessary to estimate an increase of the load of the main barrel vault from the transverse vault. The barrel vault is by the walls on about 35.5% of the length of the room on one side and 34.3% on the other side. The resulting stress values obtained by calculation for a running meter of barrel vault must therefore be multiplied approximately by the coefficient 1/0.35 = 2.86 to obtain an estimate of the actual stress on the barrel vault (Fig. 6). According to this simplified type of analysis, the tensile stresses will not occur while the maximum compressive stress reaches 1.8 MPa approximately.

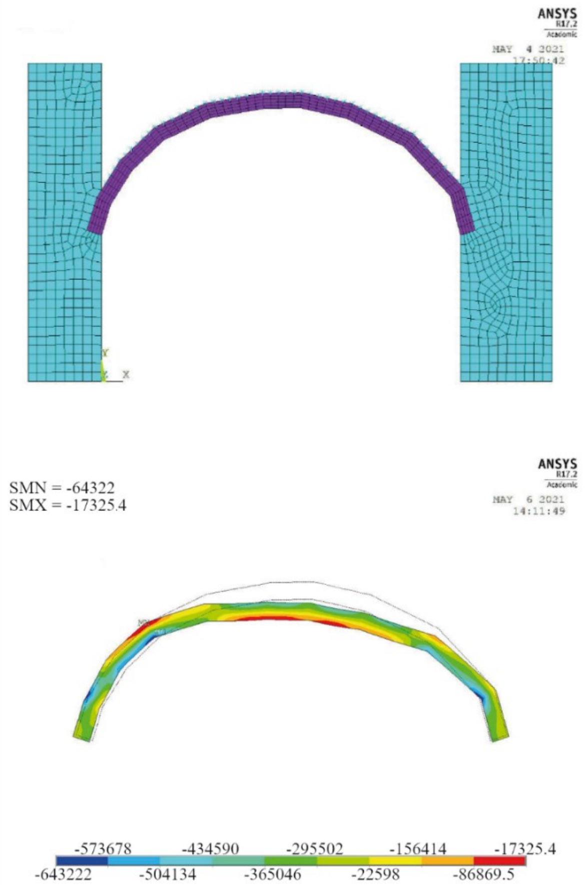

Fig. 6. Planar computational model (top), the course of the main stresses for calculation with live load and vault thickness of 25 cm. (Author: Bayer, 2021)

The maximum compressive stress on the model, taking into account the 25 cm thickness of the vault, is approximately 5.2 MPa (see Fig. 8) and the most stressed point is the contact edge between the longitudinal and transverse vaults at the lower surface. In the middle of the barrel vault belt above the place of fixing, maximum tensions of up to 1.8 MPa occur at the upper surface, which in fact cannot be completely transmitted through the masonry and will lead to a redistribution of stress, which will increase at this point the theoretically calculated compressive stresses at the lower surface. With the lower surface in the same place, the compressive stress in one direction is approximately 0.867 MPa (see Fig. 9) and in the other direction the compressive stress is approximately 0.322 MPa (see Fig. 9). On the other hand, it should be noted that in this place, due to stress, theoretically the vault belt should buckle towards the backfill, which in fact would be effectively prevented by the backfill, and it is therefore possible that the actual level of pressure stresses may be lower than the calculation shows. The spatial computational model was constructed in such a way that the surveyed geodetic points were connected by straight lines and planar surfaces (Fig. 7). Unrealistic stress concentrations can occur at the edges between the edges of the planes of the spatial model, because in reality the vault surface is rounded.

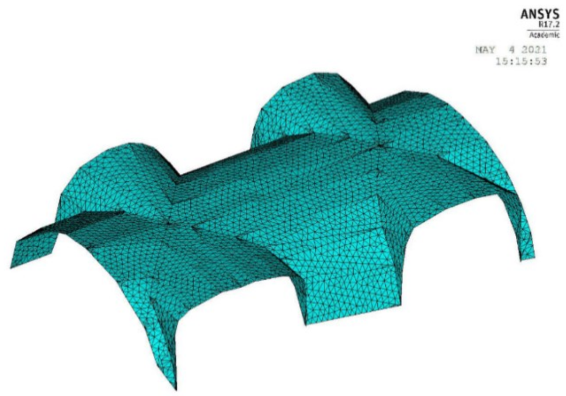

Fig. 7. Spatial computational model. (Author: Bayer, 2021)

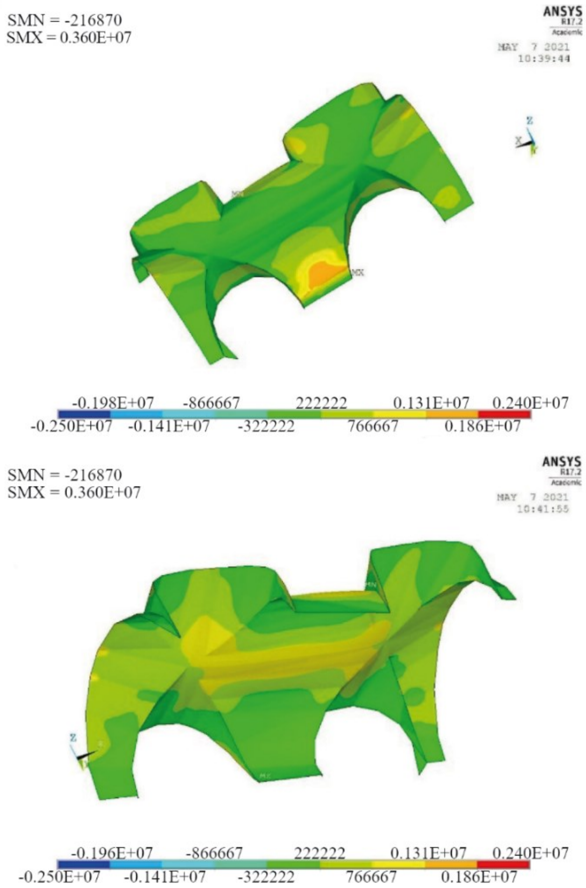

Fig. 8. The first main stress on the upper surface, (top), and the lower surface (bottom). Vault thickness 25 cm, + 5.5 KN/m² live load. (Author: Bayer, 2021)

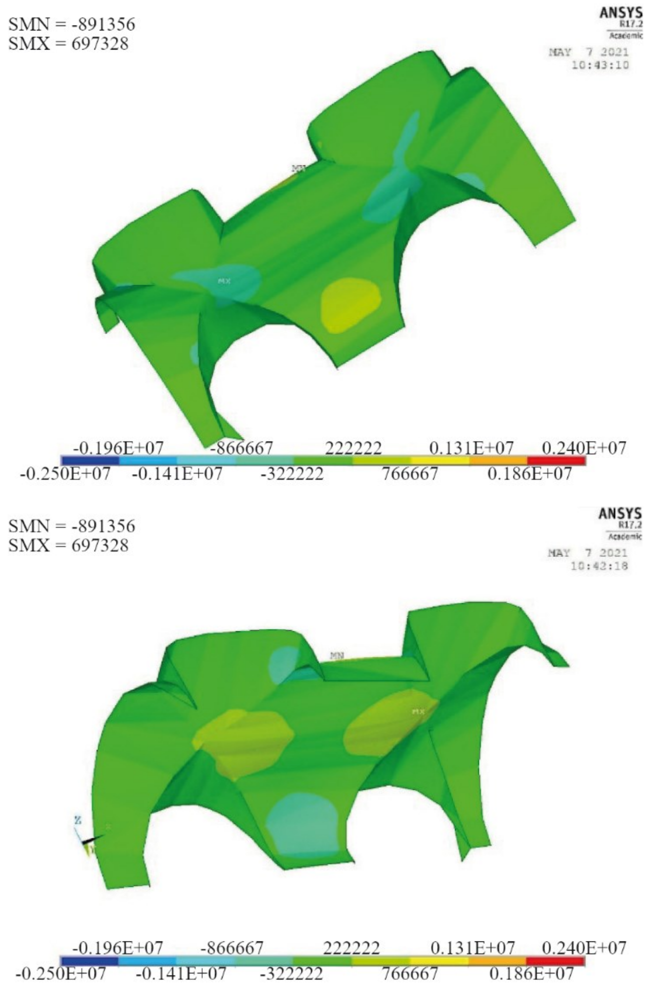

Fig. 9. The second main stress on the upper surface, and the lower surface. Vault thickness 25 cm, + 5.5 KN/m² live load. (Author: Bayer, 2021)

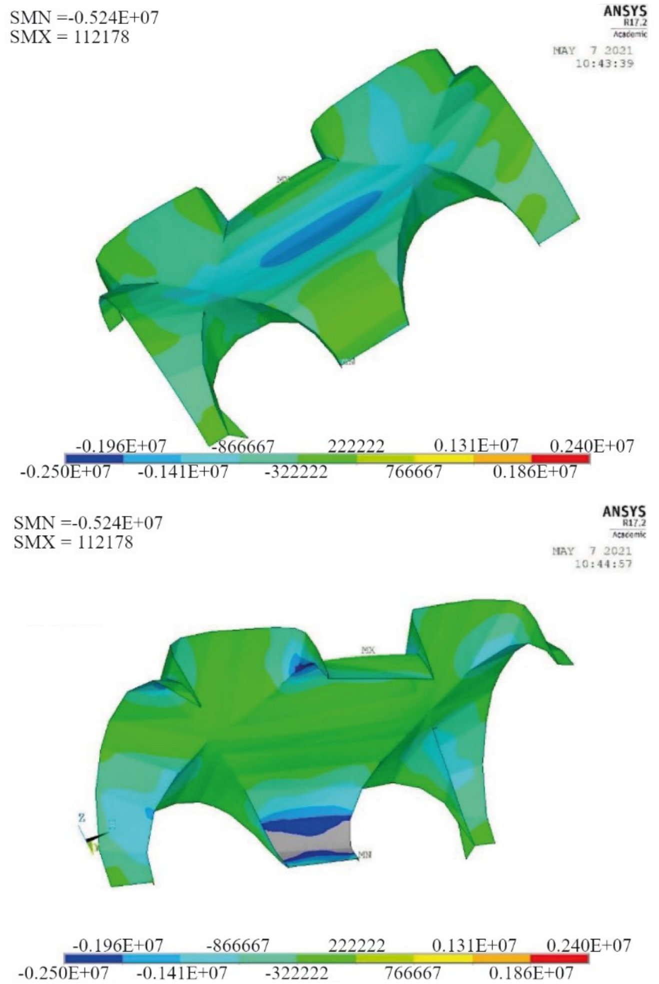

Fig. 10. The third main stress on the upper surface, and the lower surface. Vault thickness 25 cm, + 5.5 KN/m² live load. (Author: Bayer, 2021)

At the top of the vault there are compressive stresses around 2 MPa (see Fig. 10, top) compared to tensile stresses on the lower face of about 1 MPa (see Fig. 10, bottom). This corresponds closely to the simplified planar analysis above with 0.643*2.86 = 1.840 MPa in compression.

LOAD CARRYING CAPACITY ESTIMATION

The stresses calculated in the vault can be compared with the values of the design strength of the brick masonry. The actual strength of the bricks and the strength of the mortar is unknown. If we assume that the bricks used have an average strength comparable to today's bricks, i.e. 50 MPa and a normalized masonry strength at compression f_b = 50 * 0.77 = 38.5 MPa and the mortar had an assumed strength of at least 1 MPa, the characteristic compressive strength f_K of the masonry would then be in keeping the Eurocode 6 (European Commission, 2005), eq.3.2 at K = 0.5

$f_K = K \,.\, f_b^{0.7} \,.\, f_m^{0.3}$ = 6,438 kPa

But bricks may have less strength. According to tests conducted by the Brno University of Technology (Anton et al., 2016) the strength of old bricks from the last quarter of the 19th century was found to be in the range of 8.3–12.3 MPa at natural humidity. The characteristic strength when applying an average unit strength (brick) of 10.3 MPa and 1 MPa for mortar would then be only

$f_K = K \,.\, f_b^{0.7} \,.\, f_m^{0.3}$ = 2,558 kPa

Sýkora et al. (2018) gives an example of tests of brick strength from the monastery on Republic Square in Prague (dating from 1638) with an average value of 13.65 MPa. Further from the palace in Na Kampě Street in Prague 1 (dating from 1652), average values of various batches of bricks were 15.4 MPa and 25.3 MPa. This gives the characteristic strength for the bricks from 1638: 3,116 kPa and bricks from 1652: 3,390 kPa or 4,799 kPa. Sýkora and Holický (2014) tested the materials of the 17th century church and found the strength of bricks and the mortar to be 23.9 MPa and 1.44 MPa, respectively. The characteristic strength when applying the average strength would then be

$f_K = K \,.\, f_b^{0.7} \,.\, f_m^{0.3}$ = 5,145 kPa

For comparison of the calculated stresses with the expected strength of the masonry, this last value is probably the best description of the strength of the masonry of the vaults of the former Jesuit college, because, according to the authors' experience, the usual strength of high-quality structures mortar from the 17th century is somewhat higher than 1.5 MPa and the use of high-quality bricks in the vault can be assumed. According to the results of the above-mentioned studies, it can be estimated that the characteristic strength of masonry from the 17th century ranges from 3,116 kPa to 4,799 kPa. The design values of the masonry strength can then be considered to be at the sub-coefficient γ_m = 2 in the range from 1,558 kPa to 2,400 kPa. Stress calculations were made for vault thicknesses of 25 cm according to georadar data and 30 cm, which was the usual size of historical bricks, in Moravia standardized only by the decree of the Moravian-Silesian Governorate from 1810, with the size of the "zdice" being 111/2 × 53/4 × 23/4 inches (Lower Austrian inch – 302 × 151 × 72 mm). However, in previous times, the dimensions of the bricks could vary greatly.

With the presumed vault thickness of 25 cm and 30 cm respectively and the application of a constant and useful load of 5 kN/m^2, the following stress values were obtained. A linear calculation of the barrel vault belt aggravated by the supported parts of the cross vault revealed a maximum compressive stress in the vault of 1,722 kPa for a vault thickness of 25 cm and 1,840 kPa. These are values that lie within the limits of the design values of the strength of the masonry. The spatial model considers the vault as a shell, transmitting even bending stresses, which were calculated in large areas of the vault. These stresses are considerable even when unloaded, and the model does not adequately reflect the actual behaviour of the vault, because there are no defects on it today. It is therefore very conservative.

In conclusion, it can be stated that the stresses calculated in the vault from the effect of the considered live load of 5 kN/m^2 represent an increase in stress of approximately 25% compared to the stress from the dead load of the vaulted ceiling structure. The calculated stresses on the simplified model are around the design values of the masonry strength and it is not guaranteed that the structure will meet the calculation of the standard design of the vault. However, it must be said that with better knowledge of the material characteristics of the structure and, ideally, with verification by a load test, the load of 5 kN/m^2 could be considered. The design values reduce the estimated strength of the masonry by half. A more accurate determination of the strength values of historic masonry requires extensive experiments to be carried out on test specimens prepared from historical bricks and replicated mortar (Xia et al., 2022). Of course, the results of such tests can also be used to improve stress analyses based on more accurate inputs of material characteristics. But this is beyond the scope of preliminary assessment.

CONCLUSION

By reconstructing and modernizing historic buildings, it is possible to achieve their adaptation for new uses, meeting new requirements also in terms of expected and safe behaviour under load. Many unused buildings are becoming the subject of investor interest in using them for new functions, which, however, may not always be in line with the technical capabilities of the building. Early assessment of the potential for refurbishment thus significantly saves investors' time and money and helps them in their decision-making. A significant proportion of the existing building stock is made up of historic buildings, estimated at 30% in Europe. A typical construction type seen in older historical buildings incorporated brick vaults with unknown bearing capacity. This paper presents an example of a procedure for fast non-destructive survey and assessing the load-bearing capacity of a brick vault with unknown technical parameters.

Current technological tools for exploring the geometry of vaults are shown, which provide information about their shape and thickness of masonry even in situations where access is not possible. The obtained data allow the creation of numerical models for the calculation of internal forces and for preliminary estimation of the load-bearing capacity of the vault. A simplified planar model of the barrel vault section as well as the linear spatial model of the entire vault according to the geodetic survey analysed with the ANSYS 17.2 provided reasonable results for further computations. For the first estimates of the load carrying capacity of a vault, it is possible to use the values of mechanical properties of vault masonry built in the corresponding time period and style, published in the literature. Similar approach has been successfully applied for advanced non-linear and limit analysis procedure (Milani et al., 2014).

A preliminary assessment is not a substitute for a detailed analysis of an existing building. However, it provides the investor with a reliable and low-cost estimate of the structure's response to loads, thus opening or closing the way to the implementation of any plan for the new use of historic buildings with vaults. In this example, the result obtained did not give one hundred percent certainty of the possibility of using the building without the need to strengthen the vaulted ceilings for the intended project of a multi-purpose cultural facility where a large number of people might gather. The potential investor then withdrew its plan on the basis of this preliminary assessment.

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