Church enclosure walls bearing capacity estimations and its validation on 3D models

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Abstract. The paper deals with the nonlinear computational modeling of the baroque enclosure masonry walls. The main tasks are input parameters for efficient advanced numerical tools and techniques, which are based on nonlinear and quasi brittle constitutive FEM modeling. For the work was used the knowledge and results from the Broumov Group Churches survey acquired in the frame of international SAHC university cooperation. The goal of the contribution are real bearing capacity parameters of the composite enclosure walls, which leads from standard homogenization techniques. With regards to the material micro modeling of different wall configuration in longitudinal and transversal direction was done with ATENA 2D software to evaluate the safe bearing capacity of the walls. The set of models aim to assess the bearing capacity of the enclosure wall, which is the main structural element in the church. In detail, we will present results from the numerical investigation, and partial in situ testing, from Vižňov, Ruprechtice and Otovice. Finally, we will present calculation of cracks propagation on full 3D church models. Concerning historical structures, it is one way how to validate the quality of estimations and validate numerical models.

1 Introduction

The Czech Republic is blessed with the rich history of events. These events caused building up of many architectural treasures, which nowday became a symbol of the rich history. Within this wide variety of monuments, baroque architecture can be considered as a heart of this region's legacy. One such case is the case of Broumov Group of Churches [1], which is very significant not only for its unique Baroque architecture but also for the short duration of construction and relation between the single client and a single family of architects. The nonlinear computational modeling of the structure was carried out to validate the cause of the current crack propagation. Material parameters were estimate using preliminary findings, namely from geotechnical background studies. The set of models aims to assess the bearing capacity of the enclosure wall, which is the main structural element in the church [2,3,4]. The results from three sites, from Vižňov (St. Ann

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Church), more details in [5], Ruprechtice (St. Jacob Church), all details in [6], and Otovice (St. Barbara Church), details [7,8] are presented in the contribution.

Fig. 1. (Left) St. Ann 1724-728, (Middle) Church of St. Jacob 1720-1723 and (Right) St. Barbara 1725-1727.

2 St. Ann Church investigation

The longitudinal wall sample and Schmidt hammer investigation. The surface hardness and superficial strength of the stones can be tested by a non-destructive test performed by the Schmidt hammer. This test can yield the useful value of the superficial strength of the stones through the available transformation criteria provided by the hammer manufacturer [9].

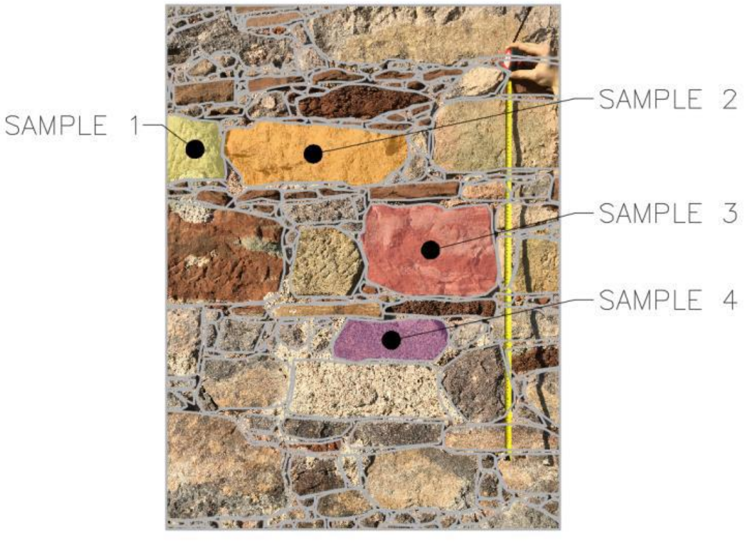

Fig. 2. Wall 1 sample considered in the analysis.

To best describe the strength of the different stones, present in the outer layer of the masonry wall, it was necessary to describe the strength of these different stone units. For the same reason, two different spots were chosen to perform the Schmidt hammer test on different types of stones. The wall samples which were chosen are of size 1 meter by 1 meter at two different locations in the church. Different combinations of the stones to build the masonry of the first spot are shown in Fig. 2.

As it can be seen from the above figure, four different types of stones were chosen to perform the superficial strength test. On each stone sample the rebound hammer was performed for 10 times and then the average value of the rebound number was chosen, this average number of rebound numbers was transformed into the equivalent strength values from the formula provided by the manufacturer of the equipment [9]. Following are the tables showing the strength of these sample stones obtaining by nondestrictive testing.

Table 1. Table showing the superficial strength of the stone samples in Wall 1.

Stone Sample	Average Rebound Hammer Value	C.O.V. [%]	Equivalent Strength [MPa]
Sample 1	52.48	9.45	41.48
Sample 2	52.83	8.80	41.96
Sample 3	43.20	10.14	29.81
Sample 4	49.45	12.00	37.47

These values of the strength of the stones are superficial values and they do not represent the true strength of the stone samples but provide with the reliable values of the strength which are useful while categorizing the stone properties for the finite element analysis of the wall samples. Anyway, concerning sandstone destructive testing in the Broumov region it was reached the peak strength value around 40 [MPa].

2.1 Simple compression test carried out in numerical way

2.1.1 Longitudinal wall configuration

The walls are modeled as the combination of various stone units, lime mortar, and rubble masonry infill. From the different available stone units, mainly three different types of stones were observed to be present in the walls outer leaf, namely red sandstone, grey sandstone, and basalt. These stones are bonded together with the lime mortar [10, 11]. While for the internal leaf, rubble masonry was considered [12]. Here the values of the mechanical parameters used are derived from both the typical values of such stones and the values obtained by the Schmidt hammer test. Observing the degraded state of the outer leaf [13, 14, 15], it was considered to use the lower bound value as a general for these types stones present in the wall, which will be a conservative approach and will result in the lower bearing capacity of the wall. Furthermore, for modeling of the longitudinal wall section, since only a single layer of masonry can be modeled in this 2D model [16, 17, 18], to account for the effect of the multi-leaves wall, reduced parameters are applied on the outer wall masonry blocks that are modeled. The reduction factor is obtained by modeling only the outer leaf in the sectional wall subjecting this wall to the same uniform loading, a load-displacement curve is obtained. The Young’s modulus and the yield strengths are modified such that a load-displacement curve that is comparable to that obtained from the three-leaves wall model. The ultimate reduction factor used is a factor of 0.33 for Young’s modulus and 0.8 for the yield strength and the shear strength [10, 19]. The mechanical parameters considered in the analysis are listed in Table 2. The results of numerical modeling are depicted in Fig. 3. and Fig. 4. Homogenized bearing capacities for longitudinal wall, coming from the numerical analysis, are listed in Table 3.

Table 2. The superficial strength of the stone samples in Wall 1.

Material Type	Young's Modulus E [GPa]	Poisson's ratio ν	Tensile Strength [MPa]	Compression Strength [MPa]	Fracture Energy in Tension *Gf* [N/m]	Unit Weight [kN/m^3]
Lime Mortar	0.126	0.17	0.1	1.5	10	20
Red Sandstone	20	0.2	1.5	30	43.5	21
Grey Sandstone	13	0.2	2	20	58	21
Green Sandstone	8	0.2	1.2	12	34.8	21
Rubble Masonry	0.7	0.2	0.1	2	10	20
Steel Plate	200	0.3	-	-	-	0

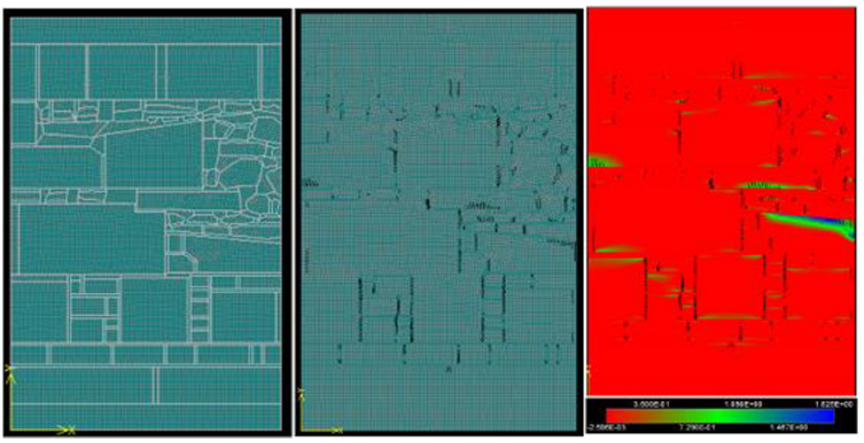

Fig. 3. Longitudinal Wall 1 configuration; crack patterns; maximum principal stress.

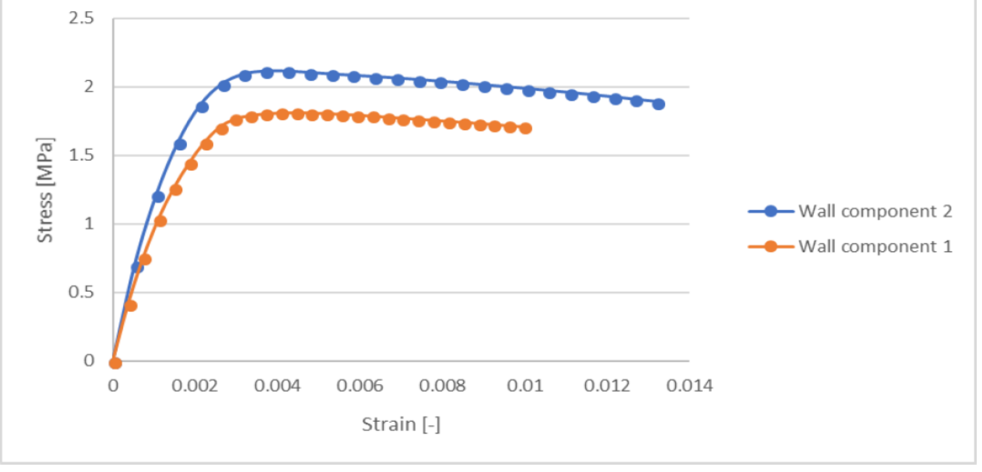

Fig. 4. Stress vs Strain plot of longitudinal wall configurations.

Table 3. Parameters for longitudinal wall configurations.

Wall Configuration	Peak Compressive Stress [MPa]	Estimated Tensile Strength [MPa]	Young's Modulus [GPa]
1	1.81	0.18	0.94
2	2.11	0.21	1.01

2.1.2 Transversal wall configuration

The analysis was performed on two different configurations of the walls. The differneces were in the bricklayer assembling of the wall, as it is shown in Fig. 5. We performed uniaxial compression test in the numerical way. The compression test was driven by deformation, which was applyed on the top of sample. All results and details are presented in [5].

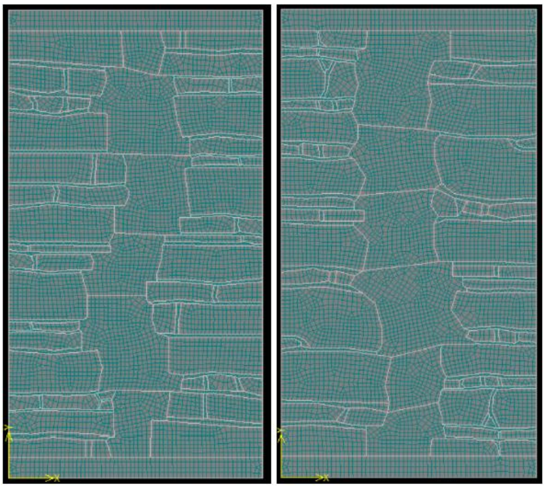

Fig. 5. Transversal wall configuration 1 (Left) configuration 2 (Right).

The numerical analysis leads to information including the bearing capacity and many others, which are listed in Table 6.

Table 4. Parameters for transversal wall configurations.

Wall Configuration	Peak Compressive Stress [MPa]	Estimated Tensile Strength [MPa]	Young's Modulus [GPa]
1	1.90	0.19	1.08
2	2.08	0.20	1.03

3 Bricklayer assembling of the St. Jacob and St. Barbara Churches

We have quite a lot information about bricklayer assembling due to delamination of render. Concerning the stones parameters we have information mainly thrue nondestructive testing, partialy we have a few information from destructive testing, as well. Anyway, we can compare them with geotechnical databases describing the stones parameters in the given locality. Thus we can carried out the compresive testing in the similar numerical way as it presented in the previous section. At first, brief visualization and some results from the locality Ruprechtice (St. Jacob Church). Bricklayer assembling is shown in Fig. 6.

Fig. 6. Bricklayer assembling of the St. Jacobs wall; wall 1 (left) and wall 2 (rigth).

The results obtained by the analysis are listed in Tables 5. and 6. All details are in [6].

Table 5. Parameters for longitudinal wall configurations (St. Jacob).

Wall Configuration	Peak Compressive Stress [MPa]	Estimated Tensile Strength [MPa]	Young's Modulus [GPa]
1	3.60	0.36	0.95
2	3.55	0.35	0.81

Table 6. Parameters for transversal wall configurations (St. Jacob).

Wall Configuration	Peak Compressive Stress [MPa]	Estimated Tensile Strength [MPa]	Young's Modulus [GPa]
1	2.95	0.29	1.35
2	3.23	0.32	1.30

Finally, we can briefly introduce visualization and some concluding results from the locality Otovice (St. Barbara Church). Information about the bricklayers skills and bricklayer assembling are presented in Fig. 7. Results of homogenization process are listed in Tables 7. and 8. More details are in [7,8].

Table 7. Parameters for longitudinal wall configurations (St. Barbara).

Wall Configuration	Peak Compressive Stress [MPa]	Estimated Tensile Strength [MPa]	Young's Modulus [GPa]
1	4.14	0.41	1.26
2	5.07	0.50	1.05

Fig. 7. Bricklayer assembling of the St. Barbara wall; wall 1 (left) and wall 2 (rigth).

Table 8. Parameters for transversal wall configurations (St. Jacob)

Wall Configuration	Peak Compressive Stress [MPa]	Estimated Tensile Strength [MPa]	Young's Modulus [GPa]
1 interlocking	5.95	0.6	0.58
2	4.24	0.42	0.47

We are pleased to can state that results are consistent with Italian standard Circolare 617 (2009) [9].

4 Examples of validations on 3D numerical models

Crack patterns propagation was selected in the St. Jacob Church as an example. Crack patterns were visualized by the survey in site. The question is causality of the failure For FEM modelling was chosen in this case software DIANA [22]. DIANA offers different constitutive models for a wide number of analyses. "Total strain based cracked model" was selected for a cracking analysis in the St. Ann Church masonry. Not well working gutters cause non uniform settlement due to deterioration of the sand stones in the foundation. We can see in Fig. 8. The 3D model of the St. Jacob Church, where we highlighted red zone around the gutter vertical downpipes. The subsoil resistance is described by springs of resistance 21,9 (MPa). In the red zones was applied additional settlement 2.5 cm. Incremental strategy was divided to 25 steps. The masonry parameters, used in numerical calculation, are listed in Table 9.

Table 9. Parameters for transversal wall configurations (St. Jacob).

E (GPa)	v	f_c (MPa)	G_c (N/m)	f_t (MPa)	G_t (N/m)	P (kN/m^3)
1.3	0.2	3.0	7200	0.15	50.0	20.0

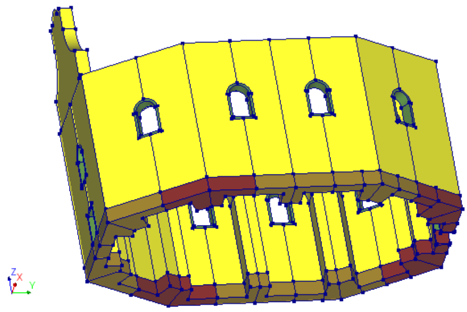

Fig. 8. Effect of soil deterioration of subsoil; in red zone was applied aditional settlement 2.5 cm.

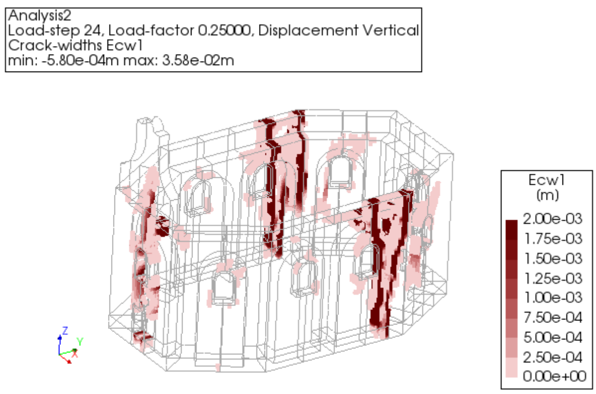

Fig. 9. Calculated cracks due to deterioration of foundations.

From the above figure the crack strain obtained in the structures can be found which occurred due to the increased differential settlements. Here it is interesting to notice that most of the crack strains that can be observed here were found to be in accordance of the damage observed on the site. This supports the considered hypothesis behind the occurrence of damages (due to differential settlements). More comments, scenarios of calculations and other details are in [6].

Non uniform settlement of the St. Barbara Church foundations was as the second example. For FEM modeling was chosen in this case software ATENA. For constitutive modeling was chosen similar nonlinear quasi-brittle constitutive model as in the previous case. Quite conservative parameters for numerical modelling are listed in Table 10.

Table 10. Material properties for the numerical model (St. Barbara).

E (GPa)	v	f_c (MPa)	G_c (N/m)	f_t (MPa)	G_t (N/m)	P (kN/m3)
2.0	0.2	2.9	9100	0.2	70.0	20.0

Degree of damage can be classified as Negligible (width up 0.15 mm), Very slight (width around 1 mm), Slight (width up 5 mm) Moderate (width from 5 mm to 15 mm), Severe (width from 15 mm to 25 mm), Very severe (width more than 25 mm). To consider the effect of differential settlement in the numerical model, vertical displacements were added in areas showing signs of damage or detachment in the foundation stones that were identified through visual inspection. The loads were then applied to the structure in two different intervals. In the first interval, the loads comprised of the self-weight and the dead load were applied to the structure, along with a spring stiffness of 26.36 MPa. This caused a uniform settlement of 18 mm. Additional settlement of 80 mm was applied in the highlighted region, see Fig. 10.

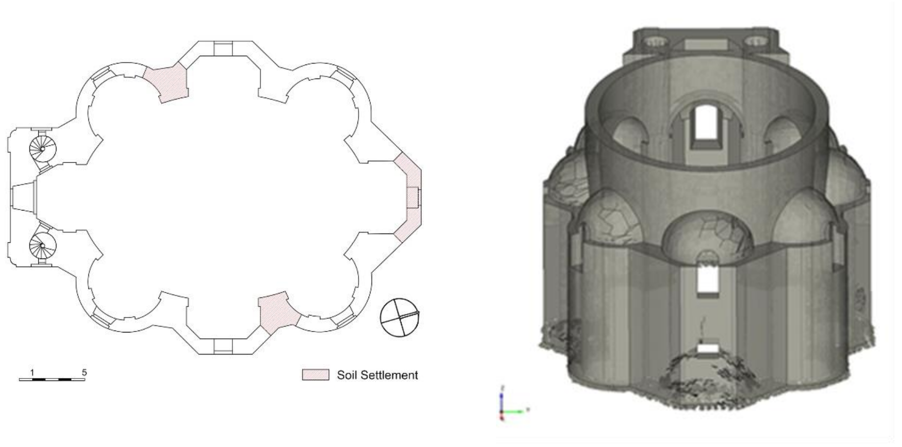

Fig. 10. Additional settlements and filtered crack width showing 15 mm (Severe) degree of damage.

Cracks of width larger than 15 mm corresponding to "Severe" degree of damage and expected to disrupt the functioning of the structure were filtered from the smaller cracks, Fig. 10. It was observed that at load step 80 which accords with 80 mm displacement showed a complete arch-like crack formation of thickness 15mm at the rear wall of the church. It is important to note here that, although this load can completely detach the section of the wall under the window, the failure is very local, therefore it is possible, that the superstructure would still prevent collapse. However, it is evident that the wall completely detaches itself from the superstructure and so, a differential settlement of 80 mm was taken as the failure criterion for the church. More interesting and complex information can be found in [8].

5 Concluding remarks

Large amount of analyses, smart numerical calculations and laboratory testings were done in the frame of SAHC Universities cooperation. So we have a lot of independent observations and opinions about the current state of the Broumov Group Churches. We are proud, that the SAHC program was awarded by the EU Prize for Cultural Heritage / Europa Nostra Awards 2017. The introduction of the work of SAHC group is seen in the short movie [20]. From the survey in site and consequent analysis it is seen, that significant role plays the date of realization, the locality, the bricklayer skills and the amount of money. The recommended values of the walls bearing capacites were validated and this can be compared in [21]. Generaly, the bearing capacites of wallls are considered as sufficient. But the poor drainage of rainwater caused the deterioration of foundations and consequently propagation of cracks due to non-uniform settlement. So in the future, it is necessary to pay attention to this phenomena.

The authors highly appreciate the financial support of Ministry of Culture Czech Republic DG16P02R049 NAKI II.

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