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EXPERIMENTAL STUDY ON AN ARCH MASONRY BRIDGE

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ABSTRACT

The evaluation of the load-bearing capacity of historic bridges and the size of the impact due to live loads form the basis to assess the structural integrity of the building. The arch masonry bridge construction is commonly used throughout Europe and Asia. During the 19th century only this vaulted construction technique was used in Germany, so a large number still exist and are used today. They account for roughly 33% of all bridges in Germany. Due to the high durability of arch masonry bridges, the maintenance costs are not so high. Railway arch masonry bridges are about 20% of the total maintenance budget of the railway companies. The reconditioning and strengthening of historical arch masonry bridges is complex and is not routine engineering tasks. This process requires individual solutions since not every bridge is the same. The increasing live loads and material fatigue are factors that complicate the situation even more. Fast replacement of the existing bridges in not practicable. That is why they need to be preserved. Therefore an accurate assessment of the state of integrity is essential. As part of the research project experimental studies on a section of a 380m long arch bridge were carried out. The bridge was built in 1867 and consisted of 15 vaults. Overall, two separate load tests were carried out successfully by using the load model 71. In the first load test the bridge was loaded up to 5.6 MN in its original condition. For the second load test the spandrel of the bridge was removed and the cover of the multi-leaf pier was cut away. By exposing the three-leaf pier the load-bearing and composite behaviour could be monitored. The tested results were modelled in an FE-simulation software to gain a better understanding of the balance of construction.

KEYWORDS: *arch bridge, structural behavior, historic masonry*

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INTRODUCTION

Check calculations of historical bridges often ascertain low bearing capacities. On the other hand, experience shows that arch masonry bridges are mostly insensitive to excess load. The results of load tests on bridges cannot be reproduced in structural analysis [1]. An essential supporting element of an arch bridge is the vault construction. The evaluation of the load-bearing capacity in the actual state is associated with considerable uncertainty. One of the reasons is a lack of inventory documents and thus the important information about the building. In addition, the material parameters of historical inhomogeneous masonry are often insufficiently known. The reasons are variations in the raw material, in the production process and in the processing that lead to scattered material parameters.

Historical viaducts often have a spatial support structure which consists of the arch construction, the spandrel, the backfill and the foundation. In this case, the interaction between the single building elements and the foundation plays an important role. These individual building elements significantly contribute to the structural behavior of the whole building in terms of supporting effects [2]. The structural composition of piers in historical arch masonry bridges are often executed as a multi-leaf construction. This type of construction was already used by old master builders and had established itself by 19th century. In the past, multi-leaf walls were also used in churches and other buildings. Using materials with different stiffness in the pier, the force distribution of the resultant force of the arch also occurs differently. The size of the stiffness differences is an important indicator for the assessment of the load assumption and the load-bearing behavior of the entire bridge structure.

LOAD-BEARING BEHAVIOUR OF MULTI-LEAF WALLS

The basics of the load-bearing behaviour of multi-leaf walls are provided, among others, by *Warnecke* [3], *Egermann* [4] and *Binda* [5]. In [3] and [4], the compressive strength of the outer-leaf is determined using the method for single-wall masonry. *Warnecke* [3] describes the load-bearing capacity of solid single- and multi-leaf walls and their internal action-effects until collapse. The load-bearing behaviour of multi-leaf and cohesive walls is described using the composite-model in the area of load introduction. He also describes a moveable composite and makes assumptions about the toothing between the single leafs. To determine the permitted action-effects, an interaction-diagram is used.

Egermann [4] reduces the permissible compressive strength of the outer-leaf. Concurrent the permissible compressive strength of the inner-leaf is increased. In this way, a space stress state is defined. In this model a cohesive inner-leaf and smooth interface between inner- and outer-leaf are assumed.

Binda [5] investigated the load-bearing and composite behaviour of multi-leaf walls using FE-simulations (Finite Element) based on the plasticity theory. The individual masonry leafs are modeled as a continuum. The experimental investigations and the FE-simulations were compared and validated.

An example of FE-simulation with multi-leaf piers can be found in [6]. In this publication a three-vault masonry railway bridge with concrete backfill in different static models was investigated. Beginning with a simple rod model an elastic 2D and 3D Finite-Element-Model was used. Finally a nonlinear model was used here. Both middle piers consist of facing masonry and backfill. During investigation in 3D-model the arch was modeled with a shell element and bend-resistant contact to the pier. The result of FE-simulations shows that, in the range of the skewback, stress peaks occur. These stress peaks arise due to soft facing masonry and stiff backfill. In the investigation, it is made clear that the facing masonry is not affected by the arch load [6]. However, the impact of the thrust line course due to the different stiffness ratios in the pier is not discussed here. At load level of limit state of the load-bearing capacity there is not enough experience about the true structural behaviour. In addition, there are no experimental studies known about the load-bearing behavior of multi-leaf masonry piers. The demolition of the Aller-Bridge offered the rare opportunity to obtain measurements on an arch masonry bridge at a load level well above serviceability.

LOAD TEST ON THE ALLER-BRIDGE (VERDEN, GERMANY)

The arch masonry bridge that was used for experimental studies is a double-track bridge. The year of construction is 1867 and it had a total length of 380m. It was used to cross the Aller River. At the time of testing the south side of the bridge had six arches and the north side had seven arches in brickwork. The span of one arch was 14 m. Over the Aller River the bridge had a steel construction with seven spans. Originally this section also consisted of arch masonry. The piers had three-leaf masonry with brick stones for the inner-leaf and natural stones for the outer-leaf. Under the foundation of one three-leaf pier there was 80 timber piles. Longitudinally the arch was divided in two pieces each with a 4 m width. These arches were connected with two tension rods. Figure 1 shows the set-up of one arch masonry.

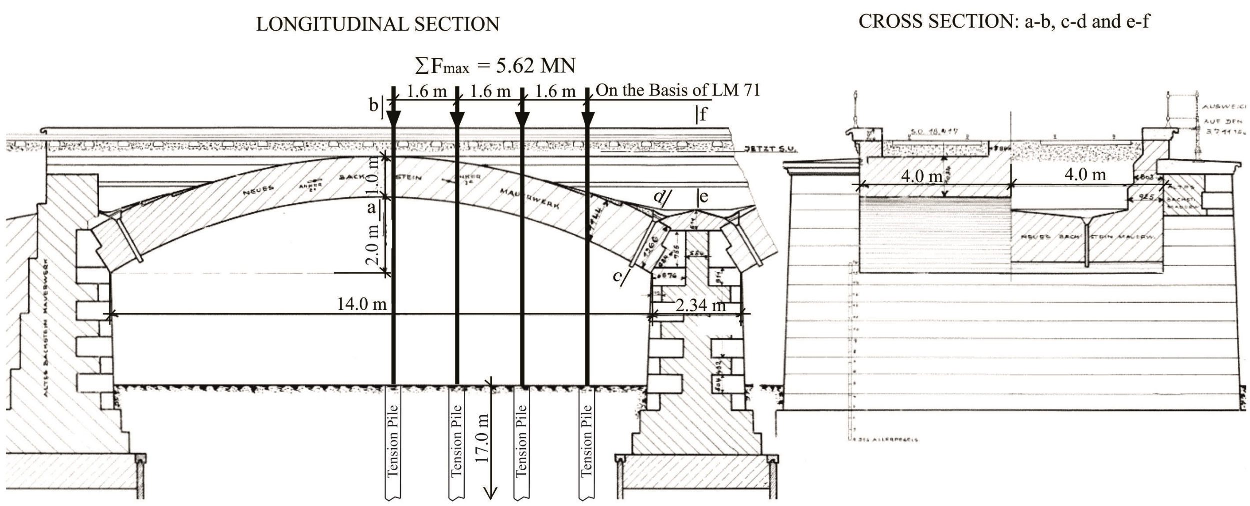

Figure 1: Longitudinal and Cross Section of the Old Aller-Bridge (Drawing from German Railway Company)

Load tests were conducted on the arch masonry bridge on 17.03.2016 and 16.06.2016. The aim was to investigate the load-bearing and composite behaviour of a three-leaf masonry pier. From the measured data of different stiffness conditions in three-leaf piers the influence of thrust line in the arch was investigated. Knowledge about load assumption in the area of the skewback is also of interest. The recording of deformations and elongations on the arch and pier could be realised using displacement transducers. Moreover, photogrammetry was used during the tests. In cooperation with *the Institute for Solid Construction at Leibniz University Hannover* and *the Institute for Experimental Mechanics at the Leipzig University of Applied Sciences* an interdisciplinary planning and implementation of both load tests was realized. The geodetics measurement were carried out by *the Institute for Applied Photogrammetry and Geoinformatics at the Jade University of Applied Sciences Oldenburg*. In order not to exceed the scope of this paper, in this case only analysis of the displacement transducers will be discussed. The investigation of deformations, deflections and stress conditions in individual bridge elements was conducted in advance in 2D FE-model (*ATENA*) by *Leipzig University of Applied Sciences* [7]. This provided findings about the expected conditions by using different load positions on the bridge. The decisive load position was determined to be most unfavourable at one third span length of the arch. In this case the collapse probability of the construction is higher and therefore significant. On the basis of load model 71 the load was applied on the arch. The fictitious load model 71 was developed on a deterministic basis [8] and introduced in 1971. This load model is anchored in current norms and directives [9], [10] and [11]. In Figure 1 the place of the load assumption points is illustrated. On the south side of the Aller arch No. 4 was selected for the load tests. Between both arch segments (each with the width of 4 m) there is a mortar joint. To get more deformations of the arch masonry bridge only one of these segments was loaded. Figure 2 is an overview of the situation.

Figure 2: Overview

In order to achieve a total load of 5.6 MN on the arch, a tie-back anchoring with four threaded bars was installed. These threaded bars passed the arch through boreholes with a diameter of 15 cm. In the ground four tension piles each with a length of 17 m were anchored. The load was distributed by a load distribution construction of eight load areas that was installed in the former track axis. The uniform load distribution was applied hydraulically and was managed by a pressure gauge. On the four hydraulic cylinders the forces were recorded through a load cell and monitored electronically.

The load was increased gradually to a maximum load of 5.6 MN. At the beginning a preload of 0.4 MN was applied to check the measured data and the function of the whole measuring chain. Then the load was gradually increased to approx. 1 MN and repeated two times. After each of the single load levels a relief to 0.1 MN base load was used. To demonstrate the load-bearing behaviour at serviceability and find comparisons to the first load-bearing behaviours, after the maximum load of 5.6 MN a load phase of 0.93 MN was used. In conclusion an endurance test with a load of 1.86 MN was used. The endurance test was to demonstrate if the damage caused by the high load levels would have an influence on the long term behaviour during a constant operational load and if the time dependent deformations would increase. Figure 3 shows the load cycle in the first load test. During the second load test the load phase 0.93 MN after the maximum load of 5.6 MN could not be implemented.

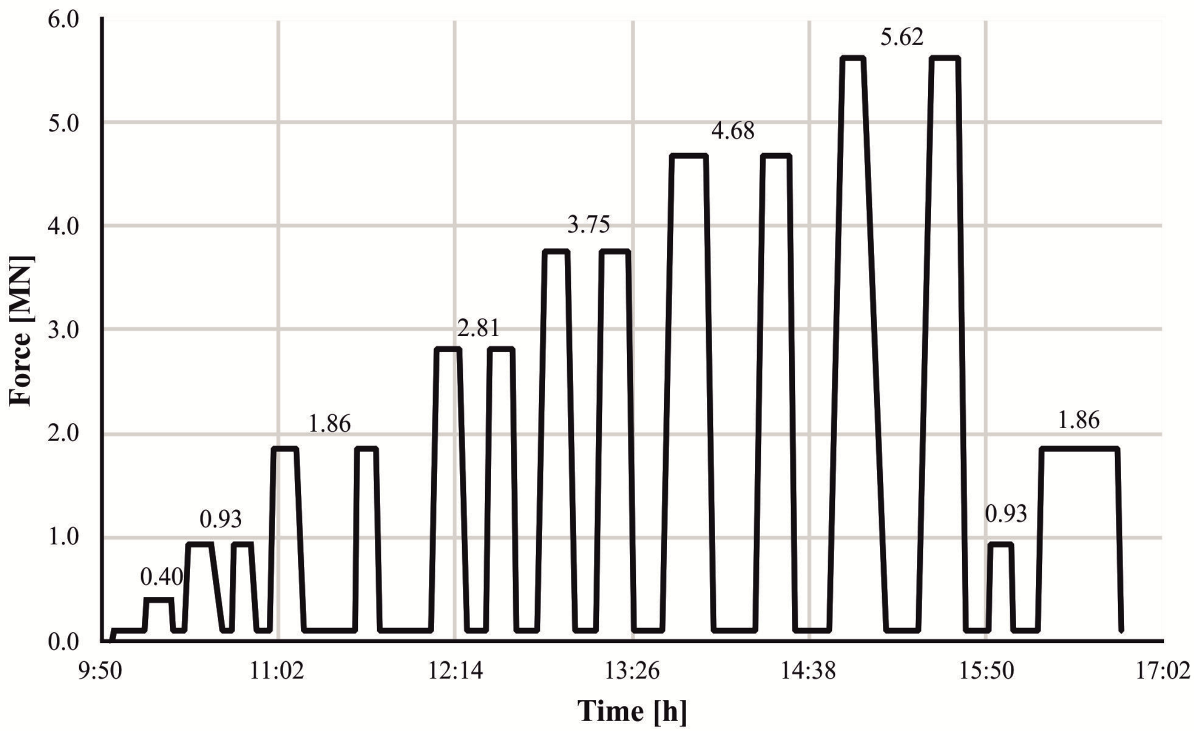

Figure 3: Load Cycle in 1st Load Test

In accordance with the demolition-plan of the *German Railway Company* the track and the railroad ballast were already removed. The arch was covered only by concrete slabs (thickness = 5 cm) that were affixed on a sealing coat. Apart from that, the bridge was in its original condition during the first load test. In the discharge area of the forces at the bottom of the arch (arch No. 4, left and right) the displacement transducers were installed to record the longitudinal and cross deformation. To record the horizontal and vertical deformations, boreholes were drilled in pier No. 4 next to the load unit. Inside the boreholes displacement transducers were installed. In the upper section of the pier more deformations were expected. The displacement transducers were attached in this area. In the opposite pier No. 4 complementary reference measuring points were installed.

Before the second load test was conducted on the side of the arch segment, the spandrel over arch No. 2, 4 and 5 to the top of the backfill was demolished. In this way, the stabilizing effect of the spandrel was partly excluded. For accurate investigation of deformation behaviour in the pier the cover of the three-leaf pier was cut away. Additional horizontal and vertical displacement transducers were attached on the exposed surface of the pier. For checking purposes, diagonal displacement transducers were installed. Figure 4 shows the state of the bridge during the second load test and the placement of all displacement transducers on the front of the arch No. 4 left to the pier No. 3. The other displacement transducers from the first load test were retained for verification.

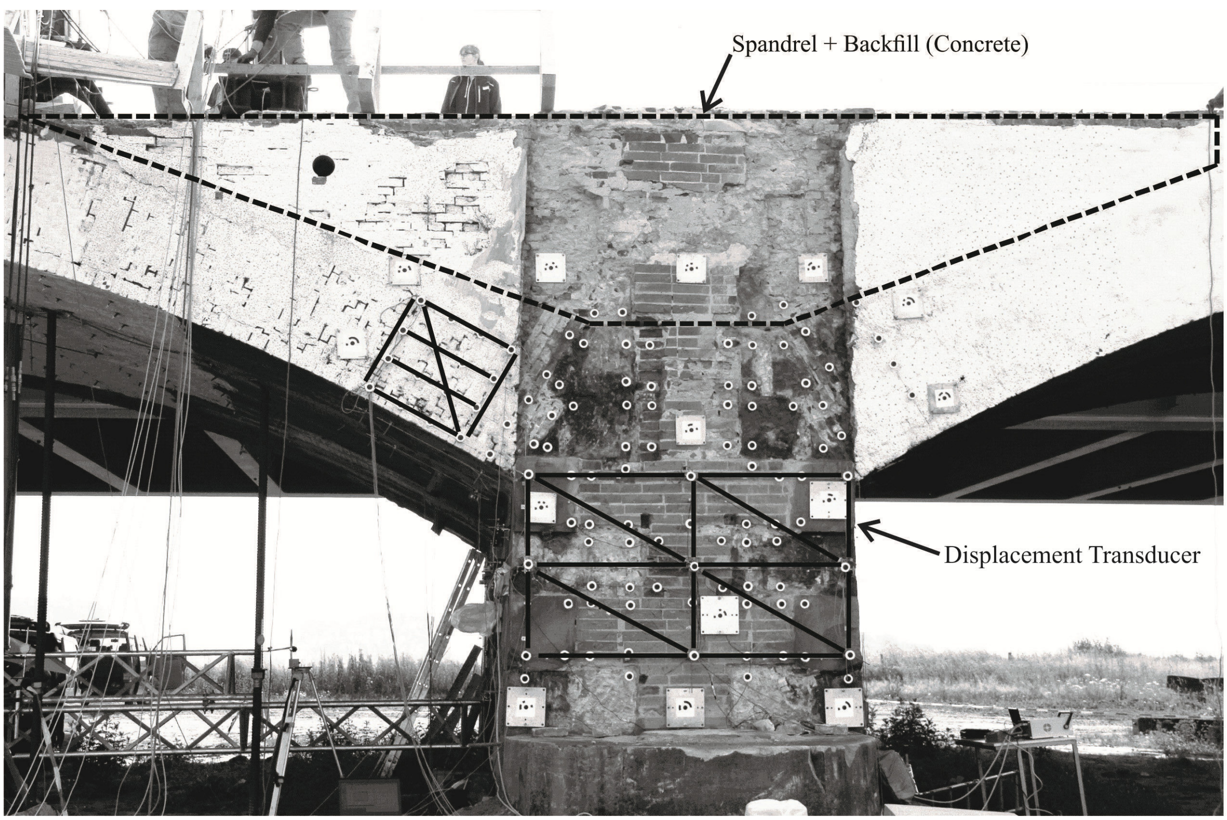

Figure 4: State of the Bridge during the 2nd Load Test (Pier No. 3)

ANALYSIS

During the load test the temperature was measured below and on top of the bridge. The measuring point on top of the bridge was exposed to direct sunlight while the one below the arch was in a shady location. Some displacement transducers were influenced by temperature. For the analysis of the strains measurements the temperature compensation was filtered out. The corrected strains behaviour (‰) at the load level 0.93 MN, 3.75 MN and 5.62 MN are shown in Figure 5 to Figure 10. The results of the first and second load test in the area between the bottom of the arch and the pier are verified. For the second load test the spandrel was demolished. However the backfill between the arches could not be removed. The strains at load level 5.62 MN of the second load test show a slight increase compared to the strains of the first load test at the same load level. It seems that the supporting effect of the backfill was still present. In Figure 6, Figure 8 and Figure 10 only three measured values are shown in the field of the bottom of the arch. The reason is that one of the displacement transducers was out of order during the second load test. The viewable area on the side of the arch is about 0.90 m but the actual thickness behind the spandrel is 1.27 m. For the rest of the arch thickness (0.37 cm) the results were extrapolated and are shown by a dashed line. The extrapolation illustrates that, in the upper half of the arch, elongations must have occurred. The results of the vertical strains in the pier at load level 5.62 MN show, on the top right, a strain of 1.57 ‰. If the measurement was correct, a crack formation would have set in. Observations proved the measurement to be false. It is assumed that the displacement transducer had an error. This measured value will be corrected by linear extrapolation. The results are shown inside the brackets. For first comparisons of the measured values a plain stress defined FE-model is used as a continuum. The investigations are made in the context of a master thesis [12]. The material parameters and the soil properties have been tested and were provided by the *German Railway Company* [13], [14]. The modulus of elasticity and the Poisson´s ratio are listed in Table 1. The plain stress defined FE-model has a thickness of 4 m (one arch segment) and a linear behavior of the material was used for the calculations. A validation between the measured values and FE-simulation is shown in Figure 10 and Figure 11 to illustrate a load level of 5.62 MN. The geometry of the bridge and the positions of the measuring points were modeled using the information of the laser scanning and equalised pictures. The strains are calculated from the deformations in the FE-model. The tension zone of the measured values at the bottom of the arch could not be reproduced in FE-Simulation. The comparison of the measurements in the pier are confirmed by the FE-simulation. On the left side of the pier upsettings were identified and on the right side elongations.

Table 1: Modulus of Elasticity and Poisson´s Ratio for 2D FE-Simulation

Building Element	Modulus of Elasticity [N/mm²]	Poisson´s Ratio [-]
Arch	4,000	0.2
Outer-Leaf of the Pier	13,000	0.2
Inner-Leaf of the Pier	1,000	0.2
Backfill	22,000	0.2
Soil	200	0.3

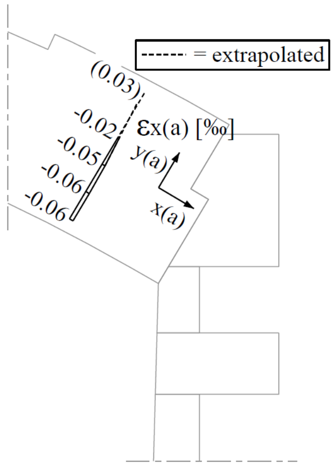

Figure 5: Strains in 1st Load Test with 0.93 MN (Arch No. 4)

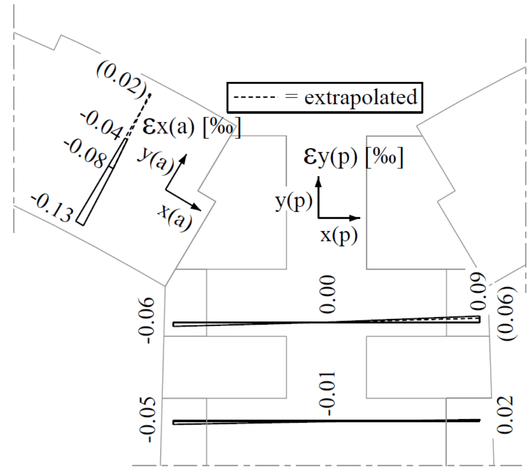

Figure 6: Strains in 2nd Load Test with 0.93 MN (Arch No. 4, Pier No. 3)

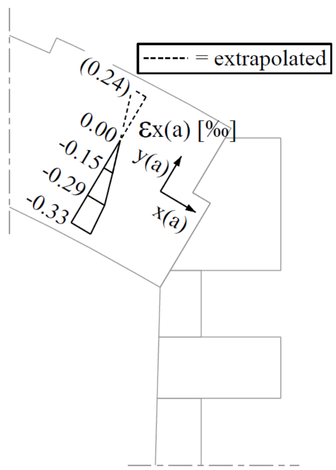

Figure 7: Strains in 1st Load Test with 3.75 MN (Arch No. 4)

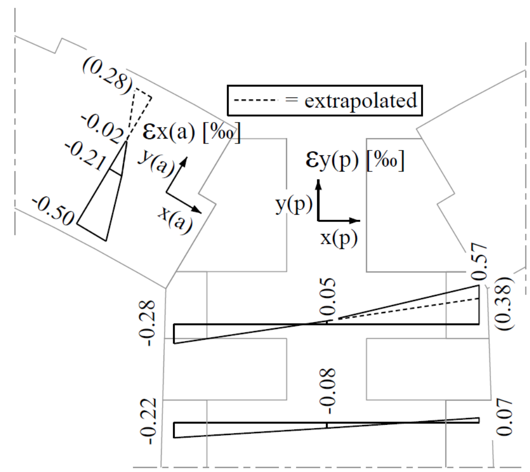

Figure 8: Strains in 2nd Load Test with 3.75 MN (Arch No. 4, Pier No. 3)

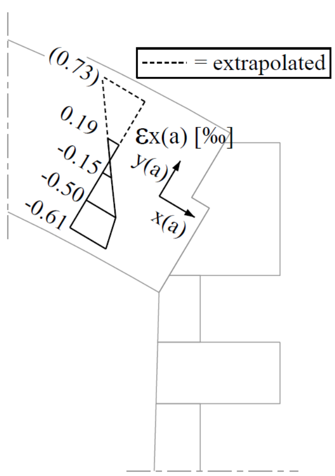

Figure 9: Strains in 1st Load Test with 5.62 MN (Arch No. 4)

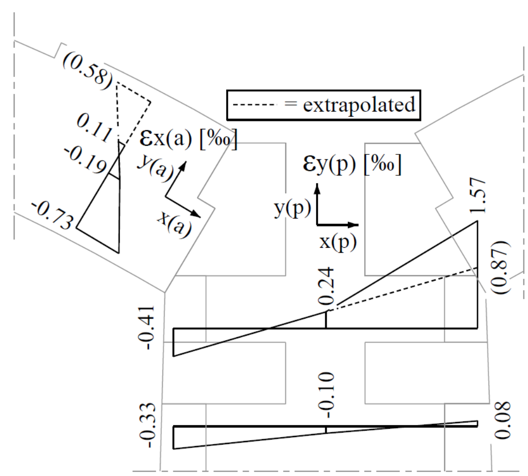

Figure 10: Strains in 2nd Load Test with 5.62 MN (Arch No. 4, Pier No. 3)

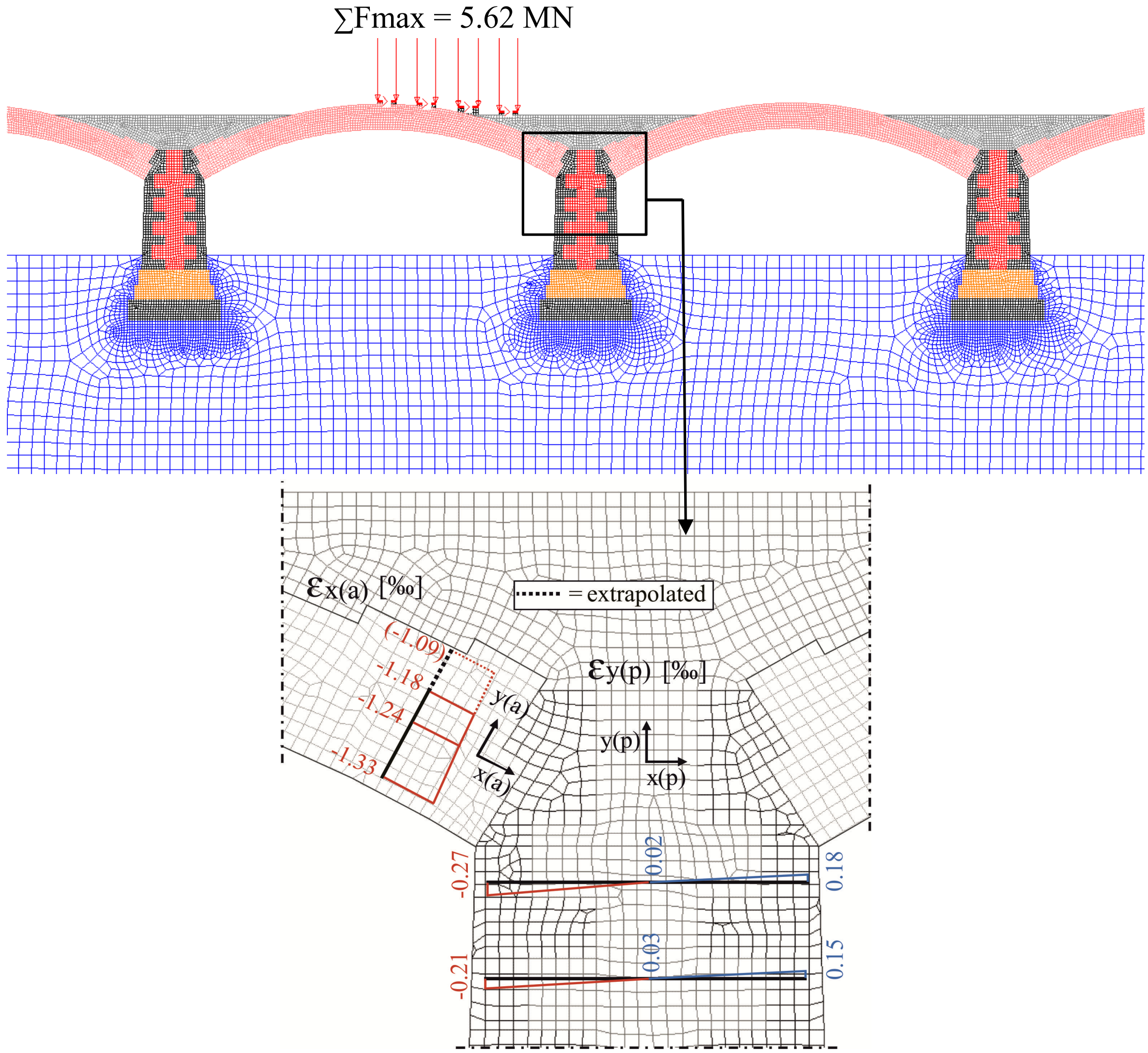

Figure 11: Overview and Partial View of FE-Simulation (∑Fmax = 5.62 MN) [12]

CONCLUSIONS

The evaluation of the measured values of the displacement transducers shows that 2D FE-simulations are a useful indicator of actual behaviour. However the results are not satisfactory yet. In the course of the dissertation project called "Experimental Structure Safety Assessment of Masonry – Development of Practice-Oriented Concept for Preservation of Bridges" which is carried out by the first author, materials of the Aller-Bridge are currently being tested. With the aid of 3D FE-models and in consideration of the results from the material testing, continually validations have to be implemented. The Aller-Bridge is going to be reproduced using information from laser scanning and equalised pictures. Additionally, the results of the photogrammetry and the measured values of the displacement transducers have to be superimposed. The position of the resulting force in the area of the bottom of the arch has to be determined in further studies. A verification of the size of the fixed-end moment between the arch and the pier is still to be done and will take into consideration the different stiffnesses of the three-leaf pier. Finally, the conclusions from the results of the FE-simulations should give insights into the formation of the thrust line in the arch.

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