# ASSESSMENT OF MASONRY ARCH BRIDGES WITH A SHALLOW OR PILE FOUNDATION

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## ABSTRACT

This paper is about the assessment of masonry arch bridges in the Netherlands, whether or not founded on wooden foundation piles. A large part of the public road network in old city centres is still formed by masonry arch bridges crossing the canals. The general question of the local authorities is: "Can these masonry arch bridges withstand motorised traffic including busses and occasionally heavy trucks?" Nonlinear finite element analysis in PLAXIS is applied for the assessment of the failure behaviour of the masonry structure and its shallow or pile foundation. The structural limits are related to cracking, continuous deformations and instability. A proof-loading of a real bridge increased the reliability of the calculation methodology. With NLFEA it can be demonstrated that these historic bridges still meet the current loads from heavy motorised traffic. With a quick scan method based on the MEXE theory, a large group of arches can be quickly classified, so the assessment with NLFEA can start with the riskiest objects. In many cases lifetime extension of these bridges is possible, whether or not by strengthening the superstructure. This means that the shallow foundation or pile foundation is reused.

**Keywords: masonry arch bridge, NLFEA, PLAXIS, proof-loading, assessment, residual lifetime, wooden foundation piles**

## INTRODUCTION

The masonry arch bridge is one of the oldest known type of bridge structure. In the historic cities in the Netherlands, a large part of the public road network is still formed by masonry arch bridges. Figure 1 gives an overview of typical terminology for masonry arch bridges. For most of these single span masonry arch bridges the (exact) geometry and the type of foundation material qualities, design criteria, and design loads are unknown. Many bridges even have an unknown year of construction. Over the lifespan, the loading has developed from pedestrians and horses to motorised traffic with busses and (occasionally) heavy trucks up to 60 metric tons. Over the years, degradation of the masonry occurs and sometimes cracks appear. Large cracks can occur in the abutments in case of settlements of the wooden pile foundation due to loads higher than the capacity of the piles. In addition, (biological) degradation of wood reduces the capacity of these piles.

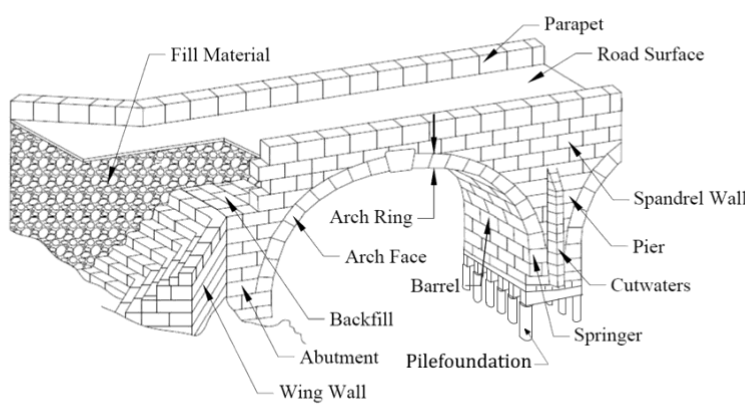


**Fig.1.Terminology of a masonry arch bridge on a wooden pile foundation, by T. Siwowski (2018).**

The question of the local administrators is how the structural safety can be ensured over the next decades in accordance with the Eurocode and the national codes for existing structures (NEN 8700-series). Research started with developing a reliable assessment methodology for the masonry arch bridge based on destructive investigation and current assessment codes. In addition, the profound research into the geometry, material decay and structural behaviour of wooden piles adds vital knowledge for reliable assessment. For the assessment and prioritisation of a larger group of masonry arch bridges, the MEXE-theory is introduced as a quick-scan method to focus on the most critical arch structures.

## COMPARISON OF ASSEMENT METHODS

Different types of calculation models are validated by comparing the outcome with lab tests found in the literature. The validation is based on three lab tests with destructive loading of masonry arch bridges: two Bolton laboratory tests (BLT 3-1 and BLT5-2 with spans of 3 and 5 m and Salford laboratory tests (SALT-1) with a span of 3 m, by M. Gilbert et al. (1994/1995/1997/2007). The BLT tests examine the capacity of a massive arch (BLT3-1) and multiple stacked arches (BLT5-2). The SALT-1 test has a limestone fill and encountered a horizontal displacement of 3 mm of the support. The model results are validated with the following programs: LimitState:Ring, SCIA Engineer, PLAXIS and DIANA FEA. Some important conclusions are the following: the angle of internal friction (phi) of the fill-material has a considerable influence on the failure load for SCIA Engineer and LimitState:Ring. The value phi has a direct link to the passive soil pressure coefficient. However, strong doubt is in place whether the passive soil pressure will be (fully) mobilised. Deflections are needed for building up this passive soil pressure. Increasing displacements of the supports (abutments) leads to lower failure loads, where the passive soil increases the capacity of the structure. PLAXIS only shows these phenomena. Additionally, the shallow foundation of the bridge can only be modelled in PLAXIS, where SCIA Engineering and LimitState:Ring need assumptions for boundary conditions. Additionally, the finite element analysis solver DIANA is used for multiple case studies and found fit for purpose. The soil-structure interaction can be simulated, and the masonry can be modelled with a total strain rotating crack model (TSRC-model). The TSRC-model follows the smeared cracking where cracking is distributed in the elements and occurs in the direction of the main tensile stress. Due to extensive compression, the model allows the material to crush.

In conclusion, the soil-structure interaction, the physical and geometrical non-linear behaviour of the structure, and the shallow foundation are best to be modelled in PLAXIS or DIANA.

## DESTRUCTIVE RESEARCH FOR GEOMETRY

One arch bridge was appointed as a pilot structure. On this bridge, different techniques were tested to find the missing but crucial geometrical parameters as the shape of the arch, the arch thickness and the foundation type and depth.

The pavement and soil fill are temporarily removed for an approximate 0.5 m wide strip in the longitudinal direction of the bridge. Now the topside of the masonry bridge can be measured with a level. Lidar 3D mapping for the bottom of the masonry arch is used. Cores are drilled in multiple locations to determine and validate the geometry of the bridge. Both the core length and the borehole can be measured. Especially the level of foundation and the thickness of the abutments are to be determined by drilling because excavation is often not possible. Assumptions for the foundation method were needed and have large influence on the results. Suspicions of a wooden pile foundation have not been verified.

In another project, already lower limit values for typical (Dutch) masonry were set and coded in NPR9998+C1 2020. Expert judgement of an experienced inspector with a profession in historic masonry can point out the masonry quality. This can be done by visual inspection of the masonry on mortar fill, coherence of the masonry and by knocking on the bricks to search for hollow sounding masonry.

Often the advice is to repair the local degradations in the masonry to restore to an adequate level. In the case of clay brickwork dated before 1945 that has a clear, bad quality concerning the fill of the mortar, layout or bonding pattern, the lower limit values are advised to decrease by 40 %. The partial factor for the masonry can be set to $\gamma m=2.2$ (from NEN-EN1996-1-1), consisting of a model factor and a strength factor for variation in shape and physical properties.

## PROOF-LOADING

The information from the destructive research is used to set up a 2D PLAXIS model to make a prediction of the deflections during the proof-loading. The proof-loading is executed with a heavy mobile crane with 3 (close spaced) axles loaded to 21.5 metric tons each. Maximum deflections of 1.7 millimetres are measured at the top of the arch during the proof-loading. The behaviour of the arch bridge was still in its linear phase because permanent deflections did not occur. Also, repetition of a loading position results in the same deflection.

With the proof-loading, the calculation methodology is calibrated by post diction. Suspicions of a wooden pile foundation are present because the calculated displacements, including a pile foundation, match the actual displacements from proof-loading.

## STRUCTURAL ASSESSMENT

*Model setup*

The modelling is done in the software programme PLAXIS which has its focus on geotechnical engineering. A physical and geometrical non-linear finite element analysis is proposed to model the failure behaviour of the arch structures. The arches are continuously modelled including backfill as a 1.0 m wide strip, with 2D non-linear plane strain FEA. Updated mesh is applied, which includes the deformed mesh as a starting point for each load step. The deformations from the safety analysis are fictitious and are only used to drive the process and to indicate the failure mechanism.

The masonry can be modelled by the Mohr-Coulomb model. The cohesion is set to half the compression strength of the lower limit value. The angle of internal friction is set to zero. The tension cut-off (maximum tensile stress) is equal to the flexural tensile strength perpendicular to the ribbon joint and the Poisson value is 0.2. A low value for the tensile strength of 0.005 N/mm$^2$ is applied in the ULS to prevent overestimation of the failure load. The SLS uses the lower limit value from the NPR9998+C1 2020 for the tensile strength. An alternative is to model the masonry with a Concrete-model. The tensile strength, according to the code, can be used without overestimation of the failure load. When the tensile strength is reached, the fracture energy determines the moment of cracking. However, low values for the fracture energy (0.01 kN/m or lower) may lead to numerical instability. Thereby, the calculation time is longer for the Concrete-model.

The material properties of the soil are based on available soil surveys from the administrator or open source. In general, a Hardening Soil model is applied when the bridge is founded on a sandy soil. The Hardening Soil model is an advanced model for the simulation of soil behaviour, by M. van der Sloot (2019). The soil stiffness is described more accurately than by the Mohr-Coulomb model, by using three different input stiffnesses: the triaxial stiffness E50, the triaxial unloading stiffness $E_{ur}$ and the oedometer loading stiffness $E_{oed}$. In contrast to the Mohr-Coulomb model, the Hardening Soil model also accounts for stress-dependency of stiffness moduli. This means that all stiffnesses increase with pressure. Hence, all three input stiffnesses relate to a reference stress, usually taken as 100 kPa (1 Bar). Besides the model parameters mentioned above, initial soil conditions, such as over consolidation ration (OCR), play an essential role in most soil deformation problems. This can be considered in the initial stress generation from construction phasing.

Construction phasing includes excavation of the pit with low (ground)water level, construction of the bridge including the fill and extreme values for the (ground)water level are applied for the calculations in the different limit states.

The maximum element size is limited to ensure that the constitutive model does not exhibit a relapse in the stress-strain relationship during the load cycles, the contour of the bridge with varying thickness is captured well and the expected damage distribution within the masonry arch is captured well. The RTD 1016 prescribes a maximum element size of L/50 for these types of structures. However, a minimum of 4 elements across the thickness of the arch ensures the mentioned aspects, which is often finer for typical Dutch arch bridges.

*Safety Philosophy*

The non-linear behaviour of masonry is to be examined by the three proposed methods from the FIB Modelcode par. 9.2 (see below). Safety can be proved with each self-contained method.

1. The global resistance factor (GRF): This method can be compared to the former overall safety factor method in the Dutch design codes using allowable stresses. This method calculates with characteristic values on the capacity side (masonry and passive soil) and at the loading side. Between the strength and the capacity, an overall safety factor is introduced.

2. The partial factor method (PF-method) is the method currently used in Eurocode to reach an acceptable structural reliability. The PF-method has partial factors applied to the capacity side (characteristic/material factor) and at the load-side (characteristic*load factor).

3. The estimation of coefficient of variation of resistance method (ECOV) is a more probabilistic based approach. Two calculations are made, one with characteristic values for strength and one with mean values. The difference between both determines the standard deviation in the results. Then the design values for strength parameters are defined by the standard deviation. At the load-side is a load factor introduced. Due to the long calculation time, this method is only applied in exceptional situations.

*Limit states*

The ultimate limit state examines the destructive load of the masonry arch bridges. The GRF and PF-method are applied to the nonlinear finite element analysis for safety verification. The traffic load model from Eurocode consists of the design tandem and distributed traffic load. The traffic load is applied to multiple unfavourable locations in the driving direction of the bridge. The distributed traffic load is positioned in the positive influence area of the concentrated traffic load. In this way, the bridge is symmetrically and asymmetrically loaded. Mirror-symmetry is used to minimize the loading positions in case the bridge and soil fill is symmetrical in the vertical axis at midspan.

Research from P. Foraboschi (2004) shows four types of failure mechanisms by symmetric or nonsymmetric loading. Governing is the load at which at least three plastic hinges in the arch occur and failure comes after (Fig.2-left). Figure 2 (right) shows the typical representation of the one-sided symmetric shear failure mechanism developed beneath a shallow footing, resting on the crest of a c-φ soil slope, according to G. Meyerhof (1957). This can be considered a shallow strip foundation as an abutment of an arch bridge resting on the crest of a cohesionless soil slope of the canal. Many other recent research works have been conducted on the effect of slope on bearing capacity using finite element modelling and energy dissipation methods.

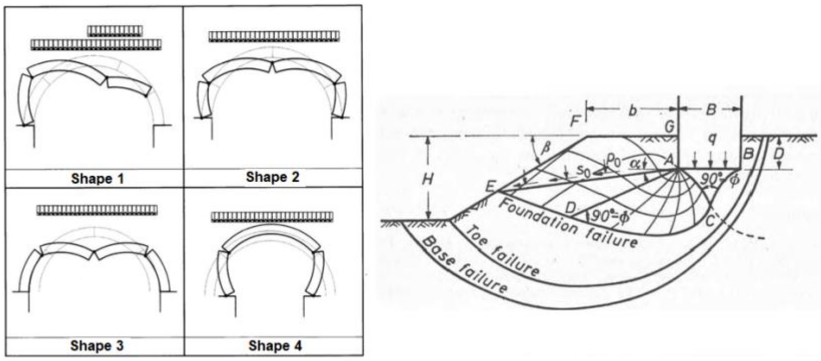


**Fig.2. Failure shapes of arches (left) and failure mechanism of the shallow foundation of the arch bridges (right).**

The masonry arch shows failure behaviour when multiple plastic hinges occur due to exceeding the compression strength of the masonry. The compression arch fails due to crushing of the masonry material collected by a small fraction of the cross section. The Mohr-Coulomb model does not include crushing of the material; however, the approximation of the failing masonry is clearly visible. Figure 3 presents the compression stresses in a masonry bridge loaded by a design tandem of 600 kN in total and a distributed mobile load of 9.0 kN/m. The maximum compression stress $\sigma_{c,max}$ for the GRF method is the characteristic value divided by the partial factor for the masonry strength and the required load factor ($\sigma_{c,max}$=5000/2,2 *1,2=2725 kN/m$^2$). The starting of three plastic hinges is visible at the extrados below the applied load and two at the intrados of the arch.

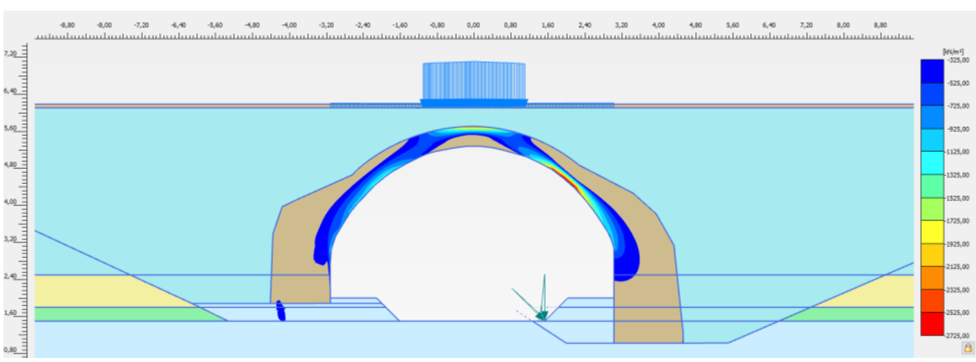

**Fig.3. Example of compressive stresses (σ'1) in an arch due to traffic load.**

The traffic load can be further increased to examine how close to failure the structure is. The fictitious deflection is shown in Fig. 4 to give an impression of the failure shape. The deflection is fictitious because of applied safety factors and load values. A shape 2 (Fig.2) is found in this specific case.

Existing structures are only to be examined in the ultimate limit state in a public law sense. However, in a private law sense, examination of the serviceability limit state is meaningful. Additional to the ULS, the serviceability limit state (SLS) is considered for durable conservation of the historic bridges. Cracking of the masonry due to overloading by traffic is undesirable. The criteria of exceeding the flexural tensile strength of the masonry (Fig.5) in combination with deflections of multiple millimetres are tested for. These calculations indicate the expected damage in terms of cracking. Because of the use of lower limit values of the masonry and modelling a continuous material, only an indicational limit is found based on allowable stresses. Cracks due to overloading by traffic are, in general, perpendicular to the driving direction. The advice for the local administrators is to perform an inspection when incidental heavy traffic has crossed the bridge.

The deflection is approximately 6 mm in the top of the arch, corrected for the deflection of the abutments, see Fig. 6. Horizontal soil pressure at the abutments causes an inward pressure where the displacement of the abutments is also inward. This is an opposite effect in case the bridge is loaded by traffic on top of the arch. Therefore, including the construction phase in modelling causes tensile stress in the top of the arch cross section and compression in the bottom of the cross section. This clamping effect increases the moment of cracking (exceeding the flexural tensile strength).

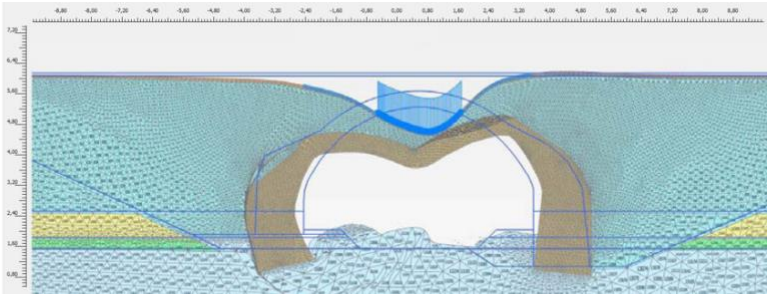

**Fig.4. Example of the failure shape of an arch.**

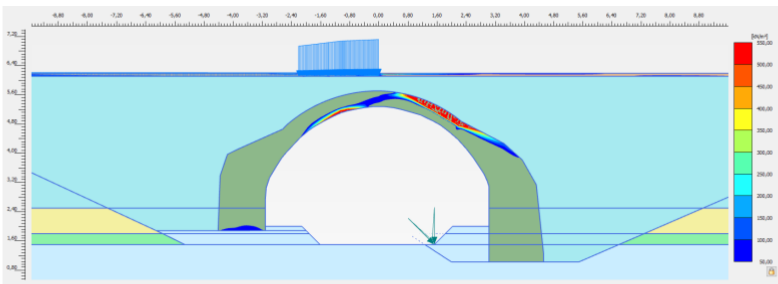

**Fig.5. Example of exceeding the flexural tensile strength of the masonry, red parts.**

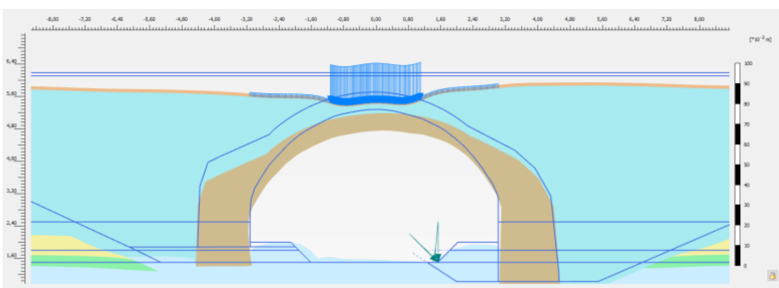

**Fig.6. Example of the deflection of an arch due to traffic load.**

## INSTABILITY OF THE SHALLOW FOUNDATION

The settlement behaviour of the individual abutments due to the load transfer to the soil has an influence on the bearing capacity of the total masonry arch structure. For the assessment of the masonry arch structures multiple different positions of the traffic load are considered. In the ULS, the permissible compression strength may not exceed in the residual pressure arch by PF-method: 5000/2,2= 2272 kN/m$^2$ and GRF-2 method: 5000/2,2*1,2= 2725 kN/m$^2$.

In Fig. 7 an example of a historical masonry arch structure is shown where geotechnical bearing capacity of the soil below the left abutment is insufficient, and the corresponding critical sliding plane is shown.

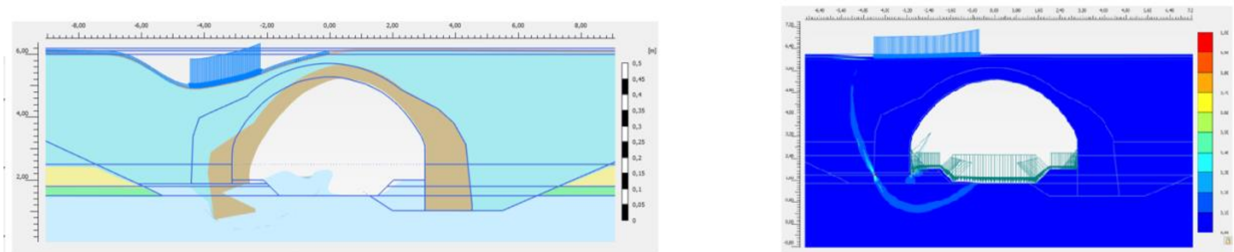

**Fig.7. failure of the geotechnical bearing capacity of the left abutment (top) and the corresponding critical sliding plane: base failure (bottom).**

In Fig.8.1 an example is shown of a masonry arch structure with two arches and a shared abutment, where the traffic load is centred above the shared abutment. The results show a settlement of this abutment of 85 mm due to overloading of the shallow strip foundation. The associated compression stresses for the same situation show that the pressure arch is already clearly visible, but the maximum permissible compressive strength is not yet mobilized across the pressure arch width.

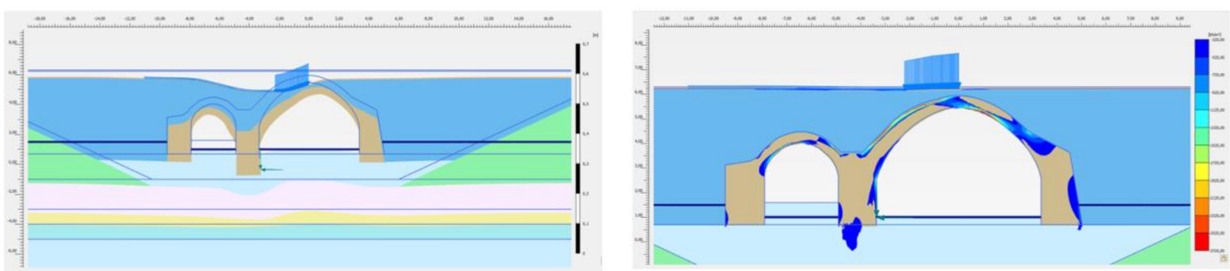

**Fig.8.1 Exceeding the geotechnical bearing capacity of the shallow foundation (top) and the corresponding compressive stress plot (bottom).**

An example of the assessment of a masonry cellar structure (in Dutch: werfkelder) is seen in Fig.9.2, which shows overloading of the shallow strip foundation, where the solution is to perform two calculations. First, the assessment of the shallow foundation by spreading the traffic load to the bottom of the strip foundation, with the limitation of the width of the cellar. Second, the assessment of the arch with the traffic load spread to the centre of the arch and increasing the stiffness of the soil directly below the foundation level until an equal magnitude of deformation of the shallow foundation is found to the first calculation.

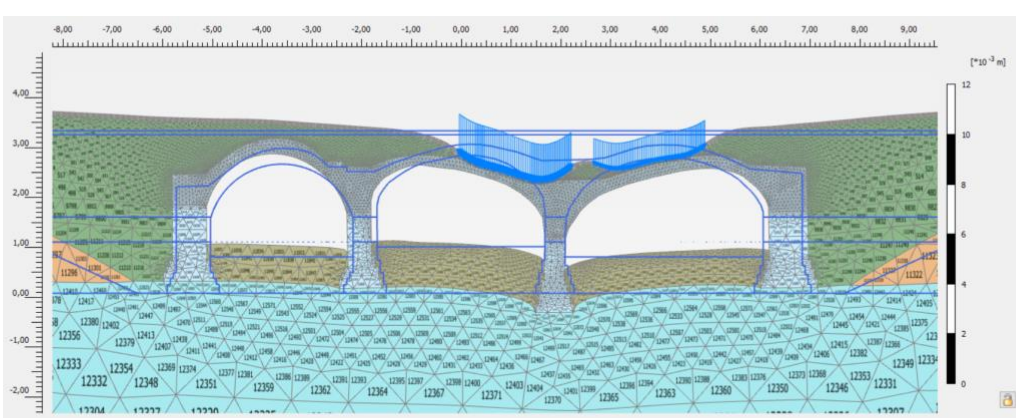

**Fig.9.2 Exceeding the geotechnical bearing capacity of the shallow foundation.**

In general, the masonry arch bridge is a robust type of structure. These structures are built with large overcapacity compared to the relatively small loading at the time of the construction. This overcapacity allows the transformation of the road traffic towards heavy motorised vehicles. However, aging and decaying masonry, and slender and relative flat arches with little soil fill on top of the arches, have, in general, a low capacity. In these cases, a reduced traffic load model for bridges in the local road network from NEN8701 appendix C, can prevent disapproval of the bridge. Another option is to assess a load restriction by signage, in accordance with NEN8701 appendix B. The road authority can issue separate exemptions to users (specific vehicles) for the given bridge. In case of insufficient arch-capacity and undesirable load restrictions, strengthening or replacement of the arch is possible while reusing the abutments. Multiple practical examples are known for reuse of the abutments and rehabilitation of the superstructure: replacement of the masonry arch by a concrete arch while retaining the masonry abutments, strengthening of the masonry arch by a concrete inner or outer shell, bridging of the arch by a concrete slab founded on the existing abutments or by a new pile foundation (the remaining function is soil retainment and aesthetic).

## WOODEN FOUNDATION PILES

In the Western and Northern part of the Netherlands, where the load bearing sand layer is at a deep level, these historical masonry arch bridge structures are founded on wooden foundation piles. Also, for the wooden foundation piles, the (exact) length, material quality, pile tip diameter, design criteria and design load are unknown. In addition, the resistance of these piles decreases over time due to, among other things, biological degradation (fungi and bacteria). For most of the bridges, degradation by fungi is not an issue because the piles are completely below the waterline. The current calculation method (NEN8700-series and F3O guideline) is based on many assumptions, both during foundation inspections and the translation of the inspection results to the normative cross-section near the pile tip. The current method of examining the material properties of the wooden pile foundation is to retrieve samples for a percentage of the total amount of foundation piles below the structure. Per pile, one sample is retrieved at 1 m depth below pile head and these results are subsequently translated to the whole pile. This is a conservative approach given the variability of the wood material due to the growth process of the tree, but also due to biological degradation.

The Municipality of Amsterdam has set up a large-scale scientific study of wooden foundation piles with the aim of developing more reliable assessment methods to test wooden piles for structural safety, where Royal HaskoningDHV is involved. This research is divided into four themes: (1) alternative measuring instruments for foundation research, (2) semi-probabilistic model, (3) the (remaining) structural capacity and the associated failure behaviour and (4) the remaining geotechnical capacity.

The public results of this research have been used in the assessment of structures on wooden foundation piles in other projects within Royal HaskoningDHV. This allowed the existing pile foundation to be reused. From these assessments we see that the governing failure mechanism of wooden pile foundations is overloading of the geotechnical bearing capacity due to increased traffic load with the consequence of continuous vertical displacements. Often, vertical or diagonal cracks are then induced. Other issues are buckling of a wooden pile in case of decay of the cross section, lack of horizontal support due to rinse out and a critical vertical load. The degradation of the longitudinal wood, which is situated on top of the wooden piles, results often in cracking of the masonry in the abutment and, in cases of failure of a wooden beam, local collapses.

## STRUCTURAL RATING OF A LARGER GROUP OF ARCHES

A large part of the public road network in the old Dutch cities is still formed by masonry arch bridges. Assessment of all these bridges is time and cost consuming. To assess a large group of masonry arches, a quick scan is introduced. The MEXE assessment method introduced in Christchurch 1952 and implemented by the Department of Transport HMSO London in 1993, is an empirical method for the classification of masonry arches that derives the maximum load ‘W’ that could be supported by the arch (equation 1). By making some assumptions for the shape of the arch and the boundary conditions, only six input parameters are needed for the calculation of the failure load.

$$W = (256fhdl) + 128\sigma lh(a28d - 121 - h + d4a)(25a + 42d) \quad [1]$$

Where l = span length, d = arch thickness, h = fill thickness at the top of the arch, a = the height of the rise, f = compressive strength of the masonry and σ = the weight of the fill material.

The result of the MEXE theory and the failure load derived by NLFEA are compared for three objects. Graph 1 shows the MEXE result in blue and the NLFEA in red for masonry arch bridges with spans of 5.4 m, 5.6 m and 6.14 m. The three examples show a higher failure load for the NLFEA. In general, the failure load derived with MEXE is more conservative in the case of smaller fill depth on top of the arch (<0.4 m).

A prioritisation can be made by first insight in the failure load by MEXE, together with the intended load the bridge should withstand. The result of the quick scan gives insight into the most critical arches which are eligible for further assessment. This method of approach meets the assessment process from the guideline CUR124: Structural safety of existing bridges and viaducts from decentralised authorities.

## DISCUSSION

The design traffic load is applied for a 1-meter-wide strip to assess the masonry arch. This is done by dividing the load of the design tandem by a norm-based load width of 3.0 m perpendicular to the driving direction for global assessment. However, this traffic load will spread through the abutments with an angle of approximately 45º toward the bottom of the shallow foundation. Here the traffic load per meter is significantly lower than at the arch. In case premature failure occurs for the shallow foundation, the cohesion of the soil layer at the foundation level can be increased. Now the bearing capacity of the shallow foundation is omitted, and the arch behaviour is to be assessed. Perhaps a 3D model of the bridge can help the problems with premature failure of the shallow foundation. However, modelling effort and calculation time significantly increase in this case.

The substantiation for choosing the plane strain condition is the constrains by spandrel walls including transverse anchorage, or by an adjacent arch. Destructive research could exclude whether the plane strain stress condition is well-founded. Masonry arch bridges with a thin fill on top of the arch can be prone to punching shear. This failure mechanism is to be assessed including crushing of the masonry material and are often triggered by a horizontal displacement of an abutment.

This paper comprises an assessment method for masonry arch structures, whether or not founded on pile foundations by using 2D non-linear plane strain FEA analysis. A guideline for this assessment method has been set up commissioned by the municipality of ‘s-Hertogenbosch.

## CONCLUSIONS

The typical Dutch masonry arch bridge requires nonlinear finite element analysis including soil-structure interaction for assessment. The Dutch masonry arch bridges are durable structures based on age and the evident residual life. The structures are often constructed with a certain amount of overcapacity and have a rather ductile failure behaviour in terms of cracking. The MEXE-theory allows for prioritisation based on the main dimensions of a large group of arch bridges. With the prevention of material decay and deferred maintenance, masonry arch bridges can often be safely part of the local infrastructure and withstand motorised traffic. In the case of a slender and or flat arch, little soil fill, or decaying masonry, the bearing capacity can be critical compared to the heavy use of the bridge. The option of strengthening or replacement of the superstructure makes reuse of the foundation including the abutments possible.

## ACKNOWLEDGEMENTS

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