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LOAD-CARRYING CAPACITY OF UNBRACED HISTORIC CONCRETE TIED-ARCH BRIDGES

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ABSTRACT

The stability assessment of concrete arch bridges is considered for 4 heritage examples, having no upper wind bracing. The relevance of this stability problem is due to the need of evaluating the load carrying capacity of older structures and the increase of design loads, or in case of possible defects. The simplified method for verifying second order effects of Eurocode 2, based on limiting values of the slenderness factor, suggests that all 4 cases show nonlinear behavior. The latter has been taken into account by applying the reduced stiffness method, which seems appropriate for solid structures as concrete arches. This has been combined with geometric nonlinear simulations, also including imperfections. However, the relation between the critical load factor and the arch slenderness fails to render clear results. Although the number of bridge cases is limited, it appears that the slenderness may not be an adequate factor for determining the arch stability and failure. The effect of imperfections, corresponding to the fundamental mode shape, has been found to be similar for at least 2 completely different bridges. A low value of the imperfection amplitude decreases the failure load in an identical manner, suggesting that the effect may be defined in a general way, reducing failure loads by a constant factor.

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The stability assessment of concrete arch bridges is considered for 4 heritage examples, having no upper wind bracing. The relevance of this stability problem is due to the need of evaluating the load carrying capacity of older structures and the increase of design loads, or in case of possible defects. The simplified method for verifying second order effects of Eurocode 2, based on limiting values of the slenderness factor, suggests that all 4 cases show nonlinear behavior. The latter has been taken into account by applying the reduced stiffness method, which seems appropriate for solid structures as concrete arches. This has been combined with geometric nonlinear simulations, also including imperfections. However, the relation between the critical load factor and the arch slenderness fails to render clear results. Although the number of bridge cases is limited, it appears that the slenderness may not be an adequate factor for determining the arch stability and failure. The effect of imperfections, corresponding to the fundamental mode shape, has been found to be similar for at least 2 completely different bridges. A low value of the imperfection amplitude decreases the failure load in an identical manner, suggesting that the effect may be defined in a general way, reducing failure loads by a constant factor.

Introduction

In the first half of the past century, concrete tied arch bridges have been built regularly to cross small rivers or roads. This type of structure was used for crossing spans, varying from 30 to 50 m. The examples being presented more extensively in the following section, demonstrate that these structures can no longer be built in an economical manner, but also their heritage value. Some of these arch bridges completely consist of reinforced concrete, while others have steel or iron hangers connecting the arches to the lower chord. In the former case, the hangers may show sufficient stiffness to obtain framework or Vierendeel-behaviour. Due to the aging of these concrete structures, and because traffic loads are increasing constantly, the question about the load carrying capacity arises. Many bridge owners have doubts about the functionality of these valuable bridges, even if no major damage has been detected so far. Obviously, the main concern is about the effect of higher loads, the issue of material characteristics being of secondary concern.

During structural assessment of arch bridges, lateral stability of the arches is of major importance. The upper bracing is not so much needed to resist wind actions, but mainly to avoid lateral arch buckling. Due to the rather solid character of arch cross-sections, structural second-order effects are expected to be low. However, the use of bracings, connecting two parallel arches, apparently was found necessary for various bridges, although

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it influences aesthetics badly. There are already many references about nonlinear behaviour of concrete arches. Nonlinear material effects have been thoroughly assessed [1] and the stability problem in particular loading conditions is well known [2]. However, these results can mostly not be applied directly to tied arch bridges. In addition, most of the recent research has been concentrated on concrete filled steel arch bridges [3]. It is felt that the assessment of heritage structures may need an alternative approach. Eurocode 2 [4] provides guidance for assessing stability conditions of concrete members, such as columns or slender beams. Unfortunately, The code sometimes renders conservative results, especially for curved or arch members as demonstrated in [5]. Therefore, a more detailed assessment of concrete arch stability may add to clearer understanding..

Historic concrete tied arch bridges

In the following, 4 particular small concrete tied arch bridges are presented.the aim being to demonstrate that this type of bridge may be valuable and probably can continue to carry present traffic loads and to determine to which extent an upper bracing would effectively be needed to ensure the latter. More precisely, the buckling resistance of unbraced arches is to be determined by using various strategies.

The first example of an older concrete tied arch bridge crosses the small river Lys upstream of the city of Ghent (Belgium). It was built before the widening and recalibration of the river, allowing international 1350 tons vessels. Fig. 1 shows the Pontweg bridge in Drongen close to Ghent, built in 1926. Because of its rare appearance the structure has been listed as a monument. This is a typical RC Vierendeel bridge with rigid nodes, which were carefully detailed. The Pontweg bridge was seriously damaged at one lower chord member and a central longitudinal beam. A refurbishment procedure is now being implemented [6].

Figure 1. Pontweg bridge across river Lys

Another fine example example is the Albert bridge near Brussels, also called Rampe du Lion, built in 1923, consisting of 2 series of 3 continuous RC tied arch bridges. A photograph is shown in Fig. 2. From this angle, the repetition of arches becomes quite clear. The span length equals 34.42 m, the arch rise being limited to 5.5 m. The Rampe du Lion bridge has been refurbished recently. The octagonal cross-section of the arch members are a particular characteristic of this bridge. A third example is a bridge across the Emu-river in Australia, and will also be referred to. This bridge formerly had a steel bracing, which was taken out because of the vertical clearance. More recently, the lack of an upper bracing was felt to be critical. The bridge span is limited to 27 m and the structure gives the impression having short side spans, which are actually long and massive arch springs. The last example is a particular structure, located in the town of Dendermonde and crossing a railway station. The single span equals 60 m, the rise being 10.75 m. The Dendermonde bridge was built in 1932 and, as shown in fig. 3, given its age, it is in a perfect condition. It may be seen from Fig. 3 that the arches have a particular arrangement, since they consist of two separate members, connected at the hanger nodes.

Figure 2. Rampe du Lion bridge crossing railway tracks at Schaerbeek

Figure 3. Bridge Dendermonde crossing various railway tracks

All of these bridges have no upper bracing, connecting the RC arches. The question arises whether the lateral stability of the arches might compromise the load carrying capacity, or if second order effects should be taken into account during structural verification. The latter certainly complicates assessment of the load carrying capacity of these historic bridges.

Concrete arch stability

A first verification of the importance of the arch stability is given by member 5.8.3 of Eurocode 2, through the use of the simplified criteria for second order effects. This is based on the verification whether the slenderness of an individual member does not exceed a limit value, given by

$$\lambda_{lim} = 20\ A\ B\ C\ /\ \sqrt{;n} \qquad (1)$$

n being the relative normal force in the member

$$n = N_{Ed}\ /\ A_c\ f_{cd} \qquad (2)$$

The values of A, B and C are given by the code and approximations may be used. Obviously, the member slenderness has to be determined, the appropriate formulas being mainly intended for straight columns. The procedure is similar to the one of steel members and in view of this the slenderness may be obtained from elastic buckling analysis. The latter results in buckling modes as well as critical values of the arch compression force N_{crit}. From this, the slenderness may be determined as

λ_{red} = **Error!** (3)

λ = $\pi\,\lambda_{red}$ **Error!**
(4)

Equations (3) and (4) are merely a transcription of the procedure used for steel members. The method may be discussed or even criticised, although the expressions allow practical verification of the criteria of eqs. (1) and (2). This has been verified for Pontweg Bridge, Rampe du Lion, River Emu Bridge and the Dendermonde bridge. Figures 4 and 5 respectively show the lowest buckling modes of Pontweg and Emu bridges.

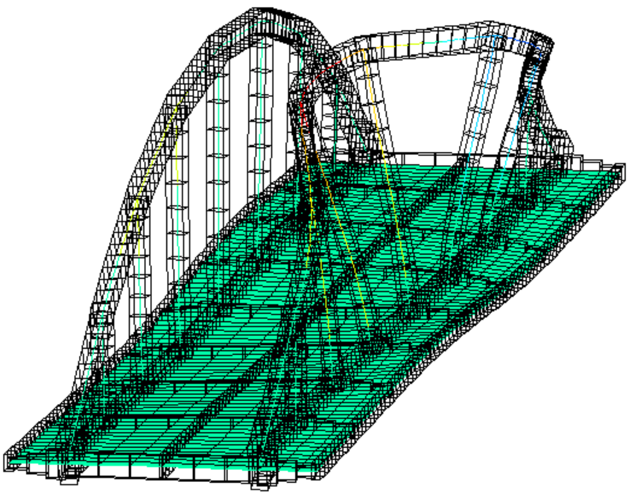

Figure 4. Fundamental mode Pontweg bridge

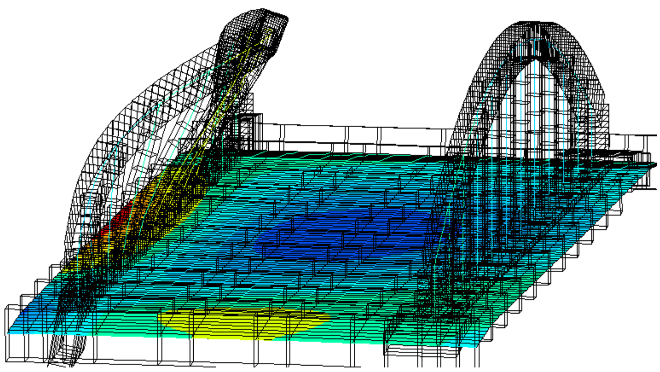

Figure 5. Fundamental mode Emu-bridge

In the case of Dendermonde Bridge, having a larger rise to span ratio, the fundamental mode has two waves, whereas the more stocky arch of Emu River bridge, shows a single wave fundamental mode. The latter is due to heavy end crossbeams and the stiff arch springs, which cause large clamping effects, thus preventing lateral displacement in those parts near the springs. This concepts adds considerably in avoiding any arch buckling effects.

Table 1 summarizes the results of this type of analysis. Clearly, for all 4 structures, the actual slenderness exceeds the limit value according to eq. (1). Hence, the code, as well as insight requires further verification of possible stability failure. It is important to mention that first order verification of concrete compression at ULS shows that concrete failure is irrelevant. In addition, lower values of the rise to span ratio render lower slenderness, the limiting value not being significantly different.

Table 1. Criterion for second order effects.

Slenderness	**EMU-bridge**	**Pontweg bridge**	**Rampe du Lion**	**Dendermonde**
λ_{lim}	15.61	17.18	17.45	12.58
λ	42.35	58.17	39.83	35.52

Geometric nonlinear and reduced stiffness approach

There is much reason to accept that concrete strength is not a real issue in the 3 cases being considered. This may suggest that the most important cause of nonlinear effects resides in large deformations (geometric nonlinearity) rather than in nonlinear material behaviour. Evidently, this assumption, yet to be demonstrated, may limit the analysis to consideration of geometric nonlinearity.

In these conditions, Eurocode 2 then suggests to use the reduced stiffness method, consisting of the use of a reduced member bending stiffness. The latter must be found from the expression

$$E\,I = K_c\,E_{cd}\,I_c + K_s\,E_s\,I_s \qquad (5)$$

Expression (5) reduces the contribution of the concrete through a factor including the effects of cracking, creep, concrete compression and strength, as well as slenderness. The contribution of steel is generally not reduced. The practical application of eq. (5) is also related with the moment of inertia used by the software code being considered. Hence, the expression (5) will result in the use of a lower value of the concrete modulus. The application of expression (5) to the 4 aforementioned concrete arches results in a wide variety of the reduced concrete modulus. Table 2 summarizes the results, showing that for Rampe du Lion bridge the modulus is reduced to 24 % of its normal value, whereas for the Dendermonde bridge it is reduced to 32%. This is due to a factor depending on the normal compression force and the arch slenderness, as well as to the low reinforcement section.

Table 2. Reduced concrete moduli

	EMU-bridge	**Pontweg bridge**	**Rampe du Lion**	**Dendermonde**
Ecd	23123.65	21467.69	24571.05	24571.05
Kc	0.1264	0.1102	0.0979	0.1501
Ic	0.0109	0.0017	0.0118	0.0125
E	23705.28	12637.66	5749.26	7930.62

The nonlinear calculations show various deformations as the total load is being increased during an incremental type of analysis. Depending on the arch slenderness, stiffness and clamping of the springs, failure may occur, either near to the arch top or at quarter span. In all cases the lateral deflection is the largest displacement and finally causes collapse.

Failure loads

The arch compression force and bending moments may be plotted versus the load increase. Especially bending moments are critical for the arch failure. The evolution with load increase for the Dendermonde bridge is shown in Figure 6. Similar graphs are found for normal compression force and for out-of-plane bending moment. However, the normal force and out-of-plane bending are increasing linearly with growing load until failure, whereas the in-plane bending moment is increasing fast when the failure load is approached. As the compression force increases linearly, without sudden modification, both bending moments are the best way to determine exactly the arch failure. They may be used in a simple formulation of the interaction N-M for reinforced concrete sections. Obviously, material characteristics, especially of reinforcing steel needed to be assessed. There are sufficient data to obtain a fair estimate of these characteristics [6].

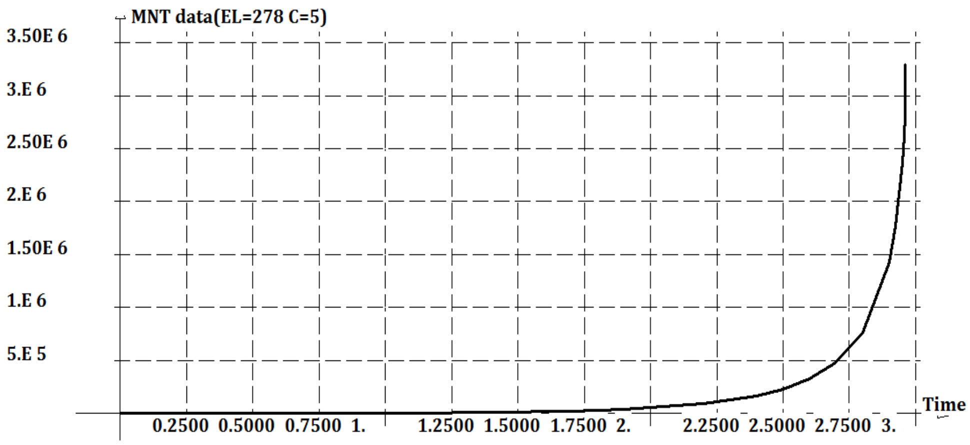

Figure 6. Nonlinear evolution of in-plane bending moment with load increase

The results for failure loads have been summarized in Table 3 as a function of the arch slenderness, calculated according to expression (4). The 4 bridges are sorted by decreasing values of λ and represent a variety of this factor. It should be remembered that the load factor of Table 3 is an multiplication factor to multiply the sum of all loads, including dead weight and LM1, for failure conditions.

Table 3. Load factor and slenderness

	EMU-bridge	**Pontweg bridge**	**Rampe du Lion**	**Dendermonde**
λ	58.17	42.35	39.83	35.52
λ_{red}	0.585	0.435	0.401	0.438
Load factor	6.80	6.10	3.61	2.37

Surprisingly the most slender arches have the largest load factor, or the highest load carrying capacity. In addition, whether the arches are stocky, such as River Emu bridge and Rampe du Lion, or more slender, the load factor does not seem to vary accordingly. In fact, the bridge with lowest slenderness has the lowest load carrying capacity. This also stands if the reduced slenderness is being considered, However, the reduced slenderness may be recalculated, taking into account the reduced modulus. Nevertheless, the relation between the failure load and slenderness does not seem to fit into a simple expression at present. Hence, it must be clear that the slenderness as defined by expression (4) is probably inadequate. In all 4 cases, the load or is well beyond $\gamma = 1.35$ for ultimate strength. Consequently, the load carrying capacity of all 4 bridges is larger than required by the code.

Imperfections

Nonlinear calculations should also consider the effect of member imperfections. The type of imperfection is taken mostly identical to the elastic buckling mode. This generally results in more negative effects of imperfections. Adversely to steel arches, concrete members cannot show effects from residual stress. Hence, the imperfections must be nearer to effective values, instead of being equivalent imperfections.

For the Dendermonde and Pontweg bridges, the effect of variable imperfection amplitudes for shapes corresponding to the fundamental mode shapes has been calculated. The graph of fig. 7 shows the results. Although the characteristics of both structures are completely different and the slenderness of Pontweg bridge is almost double, both curves are completely similar. A minor imperfection already decreases the failure load to 75% of the perfect value.

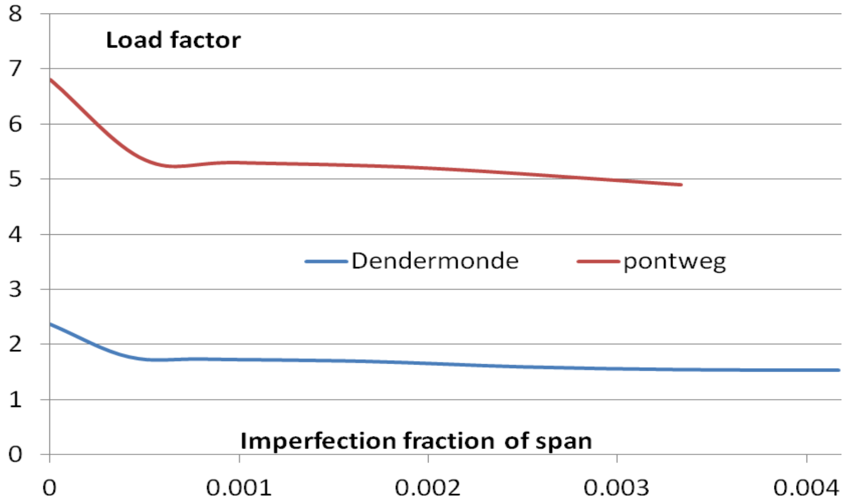

Figure 7. Effect of imperfections on arch ultimate load

The curves than continue to descend flatly to reach 65% of the initial value. The effect of imperfections can thus be identified in a general manner. Unfortunately, this also means that Table 3 needs no particular correction and the effect of the slenderness continues to exist as calculated.

These results demonstrate that the effect of imperfections may be a constant factor, allowing multiplication of the perfect load factor, without any other type of parameter. This may very well be due to the fact that there are no effects of residual stress, the imperfection being exclusively geometrical.

Load-carrying capacities

So far, 4 concrete bridges having no upper wind bracing have been considered, the Dendermonde bridge being a particular case of double arch, connected by short bars in the alignment of the hangers. The results prove that ore showcases should be examined to establish an alternative relation to eqs (1) to (4). The definition of the latter equations is based on the normal compression of the arch. The alternative may be to consider the in-plane bending moment. Unfortunately these moments are rather small, the reinforcement of the arches being based on minimum requirements or local bending at the nodes.

However, it should be remembered that the load-carrying capacity is not expressed in a correct manner by a load factor and should be calculated as the reliability index or the failure probability according to [7]. The bridges being considered were originally designed for much lower loads. Today, they are subjected to normal road traffic, although this does not necessarily include the heavy vehicles that were used in the past to determine LM 1.

From the load factor γ the reliability index β may be calculated, by eq (6), provided the relative variation coefficient V and the influence factor is known quantities. Both may be

β = **Error!** (6)

quantified as mentioned in [6], since an acceptable value of α =0.7 and the variation coefficient frequently varies around 0.15. In the case of the 4 bridge considered above, this results in the reliability indices and failure probabilities P_f of Table 3.

Table 3. Reliability index and failure probabilities

	EMU-bridge	Pontweg bridge	Rampe du Lion	Dendermonde
β	55.24	48.57	24.86	13.05
P_f	0.00E+00	1.11E-253	4.13E-68	3.32E-20

Table 3 demonstrates that all 4 bridges have abundant load-carrying capacity with respect to arch stability.

Conclusions

Four older concrete tied arch bridges of small to moderate span have been used to assess whether the limitation of slenderness, according to Eurocode 2 is valid to exclude nonlinear effects. The reduced stiffness method, combined with geometric nonlinearity, conducted to clear results in terms of deformations and internal force resultants. Critical sections may thus be determined, thus enabling to determine the failure load factor.

However, the relation between the critical load factor and the arch slenderness fails to render clear results. Although the number of bridge cases is limited, it appears that the slenderness may not be an adequate factor for determining the arch stability and failure. For all showcases, the reliability index and thus the load-carrying capacity is well above any normal requirement. The effect of imperfections is found to be similar for at least 2 completely different bridges. Hence, it may be defined in a general manner, reducing the failure loads by a constant factor.

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