SUSCEPTIBILITY OF ARCHES TO DEGRADATION UNDER SERVICE LOADING CONDITIONS

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SUMMARY

Being true that bridges do not collapse under the current traffic loads, many of them present a large number of damage which structural impact is not entirely known. These damages are associated with the service loading conditions as well as to the structure stiffness. As consequence, UIC experts have considered necessary to study deeply the behaviour of these bridges under service loading conditions. This paper presents a study of the current train loads that circulate by the different railway lines, comparing them with the load patterns given by the different existing standards, reaching to a load pattern for passenger and freight situations that are representative of the traffic in service conditions. Moreover, an analysis of the different bridge types based on geometrical and mechanical parameters has been undertaken in order to determine which are the most vulnerable to present damages as well as which parameters have a greater influence in the structural behaviour.

Keywords: *Masonry arch bridges, UIC, damaged bridges, service loading conditions, train traffic, load patterns, geometrical and mechanical parameters, 2D and 3D behaviour, stiffness phenomena.*

1. INTRODUCTION

This work forms part of the UIC project P/0314. Assessment of masonry arch bridges, carried out during 2012-2015.

It is true that 99% of the existing bridges have some kind of damage. It is also true, that these damages, in many occasions, have been there for many years without causing further problems.

Generally, masonry arch bridges are being assessed from the point of view of their failure (ULS), featuring comfortable results indicating the high adaptability of these structures to the new operation conditions (loads and speeds). Moreover, despite obtaining high values for their safety factor, numerous damage is found (longitudinal and transversal cracks in the vaults, spandrel separation, etc..), which structural impact is not entirely known.

Therefore, some research is needed to understand these damages and to avoid this normalization of uncertainties.

Following that purpose a susceptibility to deterioration study focused on masonry arch bridges was undertaken as first step and is now presented in this paper. Such study consisted in determining those bridge types most sensitive to present damage; conducting a parametric study of their behavior under ultimate and service loading conditions.

The first approach performed consisted in studying the traffic currently circulating and compare it with the load patterns given by the different Standards in order to establish if such load patterns are representative of the service loading conditions.

A second study was focused in determining, which are the geometrical and mechanical parameters that have a greater influence in the structural behavior of these bridges, under the loads currently operating. The geometries analyzed include bridges which span is between 4 and 20 m, arch depression conditioned by rise/span ratio between 1/2 and 1/6 and with the slenderness determined by the ring thickness and its span represented by a ring thickness/span ratio from 1/10 to 1/20. In all cases, a bridge of a single span and loaded on a single track has been studied.

These considerations encompass approximately 80% of the existing railway masonry arch bridges in the world network.

The mechanical parameters considered are those corresponding to the masonry, to the granular infill and to the backfill, being summarized in compressive strength, Young Modulus, Poisson’s coefficient and Friction angle.

The study is completed analyzing the influence of the consideration of the boundary conditions.

The overall objective was to try shedding light on some of the following questions:

- Which variables must be set to control behaviour under service loading conditions? What should we look at?
- Which bridge types based on geometrical parameters are more susceptible under service loading?
- Which load patterns should be used as representative for conducting an assessment under service loading conditions?
- Is it necessary to consider the buttresses and the soil beneath when analysing masonry arch bridges under service loading conditions?
- Can the behaviour of these bridges under service loading be represented with a 2-D analysis or should a 3-D analysis be performed?
- Which is the quantified contribution of each of the structural elements of these bridges?

2. DELIMITATION AND DEFINITION OF THE LOADING CONDITIONS

2.1. Contextualization

As dealing with masonry bridges implies dealing with structures which have been in service more than 100 years, the first step was to look for the evolution of the load patterns given by the different standards among this period of time.

This evolution shows that Axle Load has doubled along the XXth century as consequence of two factors. On one side, the increase of the load corresponding to the material that circulates through the railway lines, and on the other hand because of the evolution of the codes philosophy, that from prescribing real trains starts to prescribe fictitious trains that are based in the real rail compositions envelope.

On the other hand, locomotive's total weight has increased in the same proportion but not in such a constant way along time.

This implies that masonry bridges could be currently working practically with 100% more load than for which they were designed. Regarding these results, and being the Masonry Arch bridges not comparable to the bridges that are constructed nowadays from which everybody has got in mind behaviour ratios; the influence of the live loads against the self-weight, basing the analysis in four different span lengths (5 m, 10 m, 15 m and 20 m) which were considered that could reflect a representative result has been undertaken. Such study has been done using the current traffic loads.

Table 1. *Weight and load relation.*

Span	**W_{tota} [kN]**	**$IQ_2{\cdot}Q_1I_{max}$ [kN]**	**Q_{max} over the bridge [kN]**	**Q_{max} per axle [kN]**	**$IQ_2{\cdot}Q_1I_{max}/W_{total}$**	**Q_{max} over the bridge / W_{total}**	**Q_{max} per axle/ W_{total}**
5	1023.88	460	675	230	0.45	0.66	0.22
10	2165.78	800	1200	225	0.37	0.55	0.10
15	3688.44	1000	1350	230	0.27	0.37	0.06
20	5354.36	1200	1800	230	0.22	0.34	0.04

As the previous table shows, the types most susceptible to live loads are those bridges which span is between 5 m and 10 m, where the train loads imply approximately 50% of the bridge self-weight.

Once having this in mind, the existing traffic loads were studied.

2.2. Comparison of current traffic loads with Standard defined load patterns

The traffic load comparison has been done using the traffic loads currently circulating along more than 7 countries and therefore, corresponding to over 7 different railway administrations, all belonging to the UIC group. All traffic loads information had in common the differentiation between passenger and freight trains, differentiation that could also be seen in certain Standards, when defining the load patterns.

Mainly three different sources were used to obtain load patterns, which were: UIC, Eurocode, and those used by the MAV Hungarian Railways. For the particular case of obtaining passenger load patterns, UNIFE (Union des Industries Ferroviaires Européennes) was also consulted and their load patterns were also included in the analysis. Other Standards corresponding to countries belonging to UIC were reviewed, discovering that their load patterns were directly related with one of the exposed ones, mainly being a multiple of the UIC-71 as it was the case of Spain or either a multiple of the Eurocode ones as it was for example the case of Switzerland.

Therefore, the passenger traffic loads currently circulating of over 7 countries was compared with the Eurocode, the MAV and the UNIFE corresponding load patterns; and the freight traffic loads were compared with the UIC-71 convoy as well as with the Eurocode and MAV corresponding combinations.

The methodology followed for comparing the different train loads consisted in once defined the representative geometries of existing bridges, conduct a geometrical and load analysis involving three load variables that were commuted with the bridge geometries defined. These variables were: the maximum total weight, the maximum axle load and the maximum antifunicular load that each traffic load could apply on the different bridge geometries. Thus, this analysis did not took only into account the different values of the loads but the configuration of the wagons, highlighting the importance of the distance between the bogies and the overhang they have got between two wagons.

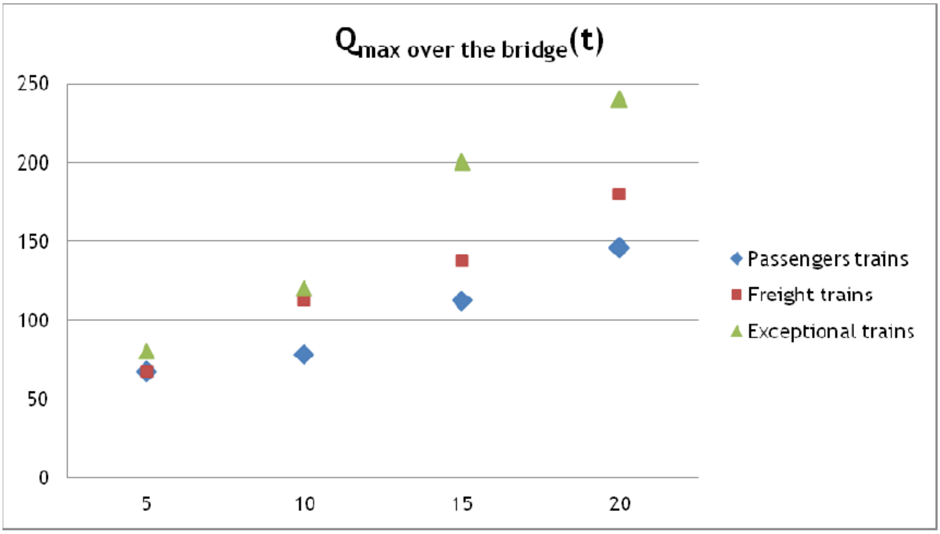

Fig. 1. *$Q_{max\ over\ the\ bridge}$ values depending on the train typology and the bridge span.*

The study undertaken concluded that for the passenger train loads, the Eurocode Type 1 load pattern can be taken as envelope of loads, being close to the maximum real train loads currently circulating along the railway lines. Regarding the freight trains, Eurocode Type 5 is the load pattern that best defines the real train loads that are currently circulating. In relation with this comparison it is important to highlight that for passenger trains, one Irish train happen to be slightly more unfavourable than the one defined from Eurocode and something similar happened with the freight loads, having found a Spanish convoy which exceeds slightly the weights of Eurocode Type 5.

Another important conclusion from this comparison was determining that the UIC-71 convoy cannot be used for analysing the service loading conditions because of being far beyond the weight ratios that are currently circulating through the railway lines and therefore not being representative.

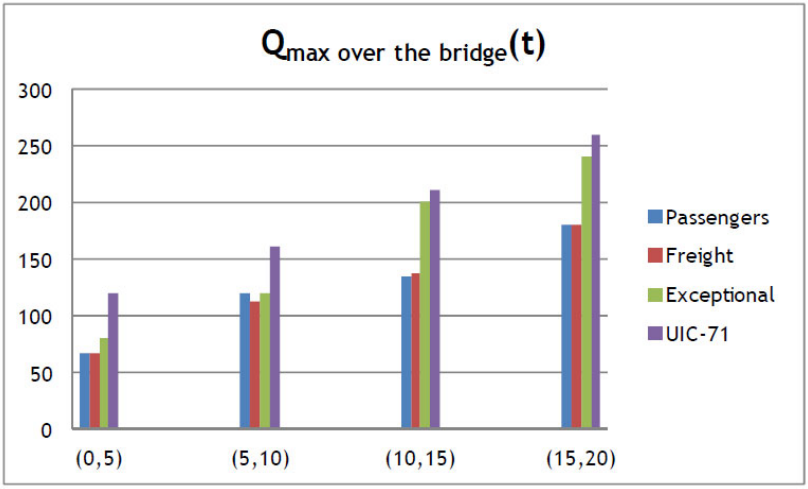

Fig. 2. *$Q_{max\ over\ the\ bridge}$. of real trains and UIC-71 comparison.*

3. DEFINITION OF REPRESENTATIVE BRIDGE GEOMETRIES TO BE STUDIED UNDER SERVICE LOADING CONDITIONS

Once the loads that need to be used for undertaking an analysis under service conditions were defined, the next step was defining which bridge types were representative of the existing masonry bridges within the railway network, and in other hand to determine which is the relation between the geometrical parameters that define a bridge and its behaviour under service loading conditions.

3.1. Parametric study based in the bridge geometry

After a study of over 500 bridges belonging to different countries and railway administrations it was concluded that over 80% of them could be geometrically defined considering a span variation between 4 and 20, an arch depression conditioned by rise/span ratio between 1/2 and 1/6 and with the slenderness determined by the ring thickness and its span represented by a ring thickness/span ratio from 1/10 to 1/20.

As these geometries imply a very huge amount of possibilities, in order to have a first impression of the vulnerability depending on the bridge geometry a simple study based in Ultimate Limit State analysis was undertaken, obtaining the different safety factors for the different geometries under the loads defined in the previous stage and listed next:

Passenger train loads:	Freight train loads:	Exceptional loads
Eurocode Type 1	Eurocode Type 5	32C5
201 Class + MK4	SERIES 251+MA5	

In this first analysis, the only mechanical property taken in consideration was the compressive strength of the masonry. The values under which the analysis was performed was 5 MPa, 10 MPa and 20 MPa.

The conclusions of this first analysis performed are:

1) The maximum load over the bridge is not necessarily the most unfavourable situation. There exist positions of the train loads different from the one that implies the maximum weight over the structure which imply worse ultimate behaviour.
2) The variation of the rise / span ratio implies a uniform variation of the bridge safety factor if the compressive strength is kept constant, therefore the change of geometry does not influence in a significant way on the selection of the most unfavourable load patterns.
3) The load patterns influence is higher for slenderer values of the raise/ span ratio of the bridge as the compressive strength is increased. Such variation is mostly homogeneous and the failure factor for the different rise/span ratios can be obtained by vertical translation.
4) The analysis performed seems to indicate a valley boundary in those bridges spanning 8 to 12 m, but it is not concluding.

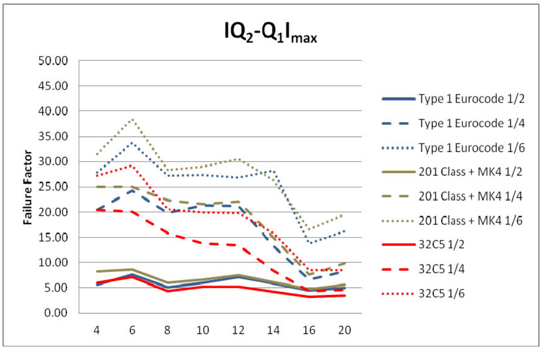

Fig. 3. *Failure factor values for 5 MPa compressive strength cases.*

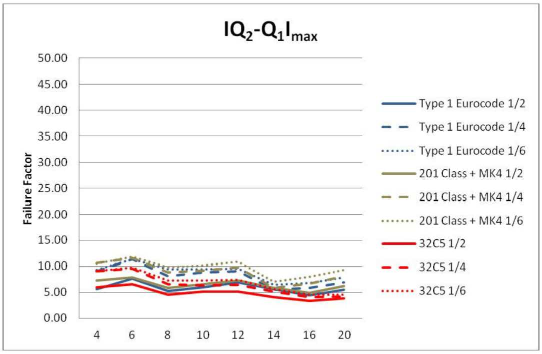

Fig. 4. *Failure factor values for 20 MPa compressive strength cases.*

3.2. Parametric study based in the bridge material mechanical properties

As it was mentioned, the geometrical analysis undertaken only included the variation of one mechanical variable, the masonry compressive strength in order to try representing the different masonry types that are commonly used in these structures.

This study is motivated by the general lack of knowledge when undertaking a structural assessment of a masonry arch bridge, about the mechanical properties of its different structural elements. The objective was to determine the influence of these mechanical variables on the structural behaviour of these bridges under service loading conditions.

The bridge elements having a structural contribution are either built of masonry or of some granular material. Therefore the mechanical parameters that are considered of interest are related with Young Modulus, Poisson's coefficients and Friction angles.

For performing this analysis, the methodology followed was based in performing 2D analyses where a parametric variation of the mechanical properties was conducted. Besides the different mechanical properties studied, as no full conclusion could be reached regarding the most unfavourable geometries, the same bridge types analysed in the previous stage were studied in this one.

The variation defined for the mechanical properties was governed in a first moment by the masonry compressive strength, followed by a comparison between three possible values for the granular infill and two possible values for the back filling material. Such comparison consisted in keeping two variables fix and varying the remaining one in order to see its impact in the bridge structural behaviour.

The conclusions reached from this analysis can be summarized in:

1) The granular infill properties turned out to be of great influence in the service and failure behaviour of arch masonry bridges.

2) The peak stress values increase greatly in the cases in which a medium-low granular infill exists (with a high deformability compared with the vault) compared to the stresses obtained for a very good granular infill. This variation can reach up to a 200% - 250% in some control sections.

3) Logically, the mean stresses values remain constant for the same load combinations. However, when the infill stiffness is low, higher bending moments appear at the vault; so the peak stresses increase and, for service loading conditions, cracking zones appear (springings).

4) On the other hand, the properties of the backfill have no influence as long as it can be considered stiff.

5) The geometries which have resulted more unfavourable certify what was advanced in the previous study, which are those with 8 m and 12 m span. In this case as a stress analysis was performed, those geometries which present slenderer rise / span ratio are presenting higher peak values.

For further contrasting the obtained results, a 3D analysis was undertaken of those geometries that were concluded to be having a worse behaviour under service loading conditions.

This 3D study allowed not only ratifying the tendencies in the behaviour obtained previously but showing that if the transverse section is considered, the stress level of the bridge structural elements decreases as consequence of the following phenomena:

Firstly, the spandrels stiffen the structure and allow the vault to be predominantly compressed. This might be checked comparing the differences in the values of peak and mean stresses in steep and plain vaults. It can be noticed that the decrease is greater for peak values and for steep vaults.

Secondly, the spandrels also carry the loads to the abutments; this is, they do not only act as stiffeners but also as elements which carry thrust.

Lastly, the 2D models had a fixed vault width of 3.00 m. With the 3D models it could be checked that the vault width mobilized depends on the cross section (being different at crown and springing) and is generally greater than the 3m defined for the 2D model. This causes an important decrease of the peak stresses at the skewbacks even bigger than the one at the crown.

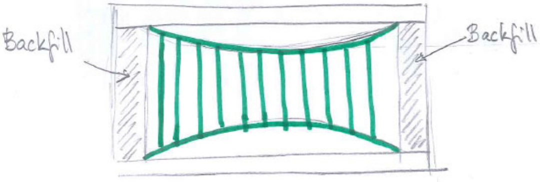

Fig. 5. *Variation of the vault's effective width.*

4. BOUNDARY CONDITIONS REPERCUSSION IN THE BRIDGE ASSESSMENT UNDER SERVICE LOADING CONDITIONS

Finally, to conclude with the study of susceptibility to deterioration of masonry arch bridges, the real boundary conditions of the vault were studied. For this, two different types of analysis were compared: a first one in which the vault is fixed at the skewbacks (no displacements) and a second in which the vault, abutments and foundation were taken in consideration. In the second case, the real displacements depend on the vault-abutments-foundation interaction.

To analyse and understand the importance of considering the foundation in the service behaviour of masonry arch bridges, a study to understand the influence of the geotechnical properties of the foundation's layer was undertaken. This study focused on the development of differential settlements and, hence, in the stress distribution of the vault.

For performing this study, the methodology followed was based in representing the abutments with the geometries defined by the engineers of the time of construction of these bridges. In order to evaluate the influence of the foundation, different Young modulus were compared at the time that the depth of the foundation layer was varied from 8 m to 24 m.

The conclusions reached from this analysis are:

1) If the analysis includes the abutments and the foundation, displacements and rotations are allowed resulting in an increase of the peak stresses at the vault.

2) Such increase of the peak stresses is never higher than a 20% of what was obtained in the analysis without abutments and foundation. The reason for this is that, for the studied live loads, small displacements and rotations are expected.
3) The peak stress increase is mainly due to rotations of the abutments. It should never be forgotten that these structures have been already in service for approximately 100 years, what implies that the soil beneath them is necessarily well consolidated. Therefore, the foundation is logically less determinant.
4) Mean stresses remain constant, since the change only affects the point where the resultant is applied.

5. CONCLUSIONS

- Which load patterns should be used as representative for conducting an assessment under service loading conditions?

  Eurocode Type 1 and Eurocode Type 5 can be used as load patterns to study the behaviour of masonry arch bridges under passenger and freight train loads respectively.

  The geometrical configuration of the train can be more important than the axle load.

  The maximum load over the bridge is not necessarily the most unfavourable situation.

- Which bridge types based on geometrical parameters are more susceptible under service loading?

  Bridges spanning 8 m and 12 m are the bridge types which present a lower safety factor and at the same time are more susceptible to low mechanical properties of their structural elements. Especially high peak stresses appear in slender rise / span ratios (1/6).

- Is it necessary to consider the buttresses and the soil beneath when analysing masonry arch bridges under service loading conditions?

  The soil beneath the bridge does not need to be considered for undertaking an analysis under service loading conditions.

  Considering the abutments implies adding settlements and rotations to the analysis, obtaining an increase of the peak stresses which can reach a maximum of 20 % of the peak stress obtained for an ideal support.

- Can the behaviour of these bridges under service loading be represented with a 2-D analysis or should a 3-D analysis be performed?

  Using 2-D disregards the transverse behaviour of the bridge and the contribution of the spandrels, that is, missis evaluating following stiffness criteria, which should be taken in consideration if the behaviour under service loading conditions wants to be known.

3-D analysis will be mandatory when the original bridge load path is violated, that is, when cracks appear in the vault.

- Which is the quantified contribution of each of the structural elements of these bridges?

  The granular infill properties result essential in the bridges behaviour. The difference between a bad and a good material implies variation of over 200 % in the vault registered stresses.

  Backfill mechanical properties are not so important as its height.

  Spandrels stiffen the structure and allow the vault to be predominantly compressed at the same time they carry the loads to the abutments. Both performances allow having the vault subjected to a lower load. Therefore, it can be concluded that cracks between the vault and the spandrels require special attention.

These conclusions have been extracted from 2-D and 3-D numerical analyses. No monitoring analyses have been performed to contrast them. This could be an interesting future line of investigation.

REFERENCES

[1] MARTIN-CARO J.A., LOPEZ D. Stage of serviceability assessment: Selection of bridges typologies that suffer more damage under the new exploitation conditions (new loads and speeds). WP 1.1: Susceptibility of arches to degradation under service loading conditions. UIC Project P/0314.

[2] MARTIN-CARO J.A., LOPEZ D. Stage of serviceability assessment: Selection of bridges typologies that suffer more damage under the new exploitation conditions (new loads and speeds). WP 2.1: Assessment of Damaged Arches. UIC Project P/0314.

[3] UIC. MASONRY ARCH BRIDGES GROUP. Recommendations for the inspection, assessment and maintenance of masonry arch bridges. 2007.

[4] OZAETA, R., MARTIN-CARO J.A. Catalogue of Damages on Masonry Arch Bridges. *Optimised inspection and monitoring of masonry arch bridges*. UIC Project I/03/U/285.

[5] Bień J., Kamiński T., Rawa P. Improving Assessment, Optimisation of Maintenance and Development of Database for Masonry Arch Bridges. *Methodology for the Condition Assessment of Masonry Arch Bridges by Means of Expert Tools*, UIC Project I/03/U/285.

[6] Gilbert, M., Melbourne, C., Rigid-block analysis of masonry structures, *The Structural Engineer*, 72, 356-361,1994.