

Numerical analysis of a load test on a skewed masonry arch bridge

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ABSTRACT: This paper presents the results of different numerical analysis of a masonry skewed arch bridge that was tested on February, 2005. The results point out firstly the great importance of the internal geometry of the bridge in the carrying capacity, specially the height of masonry backing. Secondly, that the piers of a skewed bridge tend to rotate on their vertical axis. Therefore, the stiffness of the piers may change the response of these bridges, from skewed to straight. The main objectives of this analysis is to deepen on the structural behaviour of the skewed masonry arch bridges and to develop an analytical model to be used as a reference to the displacements measured during the test.

1 INTRODUCTION

Due to the construction of the new railway to Barcelona a hundred years old masonry skewed bridge ought to be demolished. The owner of the bridge, the Spanish Railway Administrator (ADIF), decided to make a load test to collapse before demolition and therefore encouraged several tasks that should throw light upon structural behaviour of this type of bridges.

Ines Consultant Engineers were in charge of the structural analysis of the bridge. This paper describes the structural analysis that has been done and its results.

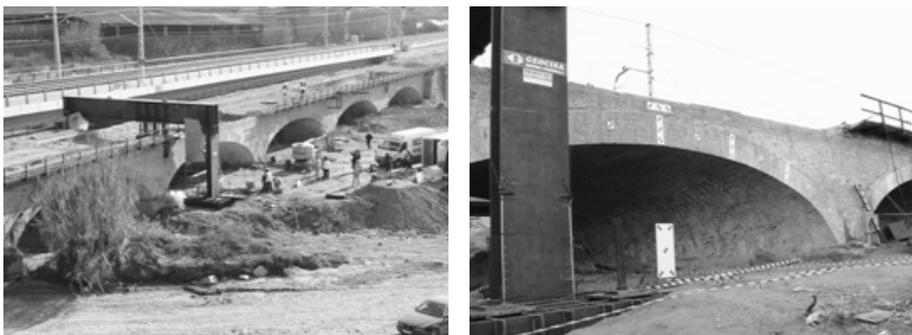


Figure 1 : Bridge over Rubí river

2 OBJECTIVES AND SCOPE

The structural analysis of the bridge intends to achieve two goals. The first one is to deepen on the structural behaviour of masonry skewed bridges in general. The second objective is to develop an analytical model so that the displacements measured during the load test could be easily explained.

3 STRUCTURAL ANALYSIS

Two different approaches to the structural analysis have been developed. The first one is a simplified analysis in 2D, which does not consider the 3D effects of the skewed bridges. Later a complete analysis with a 3D finite element model was made.

3.1 2D analysis

The need of the structural analysis arose when the jacks for the collapse load test had to be dimensioned.

For the load test a steel frame was disposed with its horizontal beam over the vault and its vertical piers on both sides of the bridge. The bridge was to be loaded with jacks situated between the frame and the vault. In order to avoid punching and to distribute the load uniformly, a steel beam of 0.6m width and 3m long was disposed under the jacks.

The load test scheme is presented in Fig. 1, with the load on the skewed direction. According to the most common theories of the structural behaviour of skewed bridges, the stresses follow the shortest way from the load acting on the vault to the piers and abutments, as shown in Fig. 1.

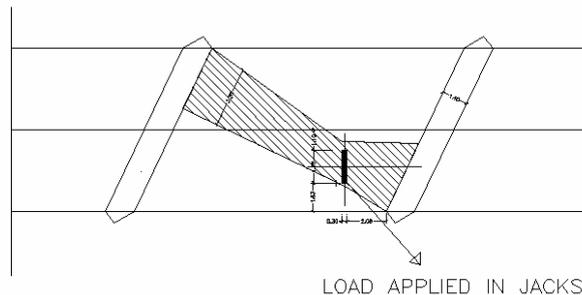


Figure 1 : Scheme of the load test.

In order to estimate the ultimate load of the bridge, the following hypotheses were made:

- The bridge works on its square span.
- Failure of foundation and rotation of piers do not occur. Therefore, the failure mechanism is of the monoarch type.
- The compression strength is unknown. In order to study its influence on the ultimate load, it is taken as 4.0, 6.0 and 8.0MPa.
- The height of masonry backfill is taken as 1.25, 2.00, 2.50, 2.85 and 3.00m over the arch springing.
- The height of granular filling over the crown of the vault is 0.5m
- The analysis is made without taking into consideration the effects of the arch rings. This simplification is made because ring separation could not be seen on the bridge.

Following results were obtained:

Table 1 : Ultimate load [kN] for a 3.00m width vault.

Height of masonry backfill	Compressive strength		
	4MPa	6MPa	8MPa
0.00m	1110	1250	1330
1.25m	1390	1660	1830
2.00m	2190	3030	3690
2.50m	-	4450	5840
2.85m	-	4720	6380
3.00m	-	4710	6490

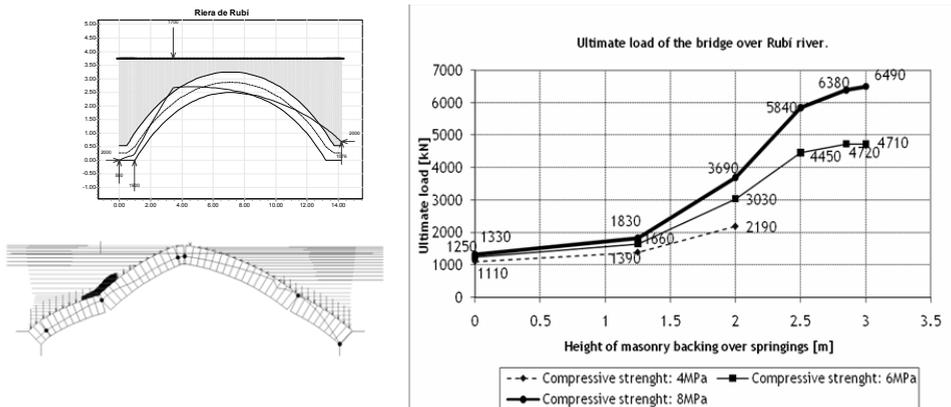


Figure 2 : Evolution of the ultimate load.

For the 2D analysis of the bridge two different programs have been used: RING and VLASTA. RING program was developed by the Sheffield University and calculates the failure mechanism through the rigid block method. VLASTA is a program made on the Universidad Politécnica de Madrid. Its output is the thrust line on the vault.

Due to the ignorance in regard the masonry backfill height and the compressive strength of masonry, they were assumed to be 2.00m and 6MPa, respectively. Therefore, the ultimate load ought to be close to 3000kN. The jacks were dimensioned for a 3500kN load.

As it was discovered after the demolition of the bridge, the height of masonry backfill was of 2.85m over springings. The mechanical analysis made after the demolition gave values of compressive strength bigger than 8MPa. For these reasons, the true ultimate load could be twice the one estimated for the load test. Hence, on the load test collapse was not reached.

3.2 3D analysis

In order to describe the structural behaviour of the bridge, several FEM models have been developed. These are 3D models that were analyzed with the FEM program SOFiSTiK. This program allows calculations with non linear materials and under second and third order theories. In this case, second and third order theories have not been used because the allowed strains are of 0.1% in tension, being acceptable the calculations over the undeformed structure.

For the calculations a square mesh of volumetric elements has been disposed. Vault, masonry backing, fill and spandrels have a common mesh, in which one node may belong at the same time to two different elements.

3.2.1 Model description

The material law of the masonry vault is non linear. The maximum compression stress is 8MPa, as determined on the masonry tests made after the demolition of the bridge. The material is supposed to be linear until the strain reaches 0.25% in compression. The maximum allowed stresses are 8MPa in compression and 1MPa in tension.

The material from the filling and masonry backing has been modelled according to Drucker-Prager theory. The material properties are taken from Table 2.

Table 2 : Material properties of fill and masonry backing.

	Elasticity modulus MPa	Poisson's ratio	Unit weight kN/m ³	Friction angle °	Cohesion MPa
Fill	22	0.2	18	30	0.02
Masonry backing	30	0.2	20	45	0.08



Figure 3 : View of the elements of the bridge: vault, spandrels, masonry backing and fill.

The boundary conditions are different between abutments and piers. On abutments, all the nodes of the lower horizontal plane are fixed, while those belonging to the extreme vertical planes have their vertical displacement free. The boundary conditions of piers vary between the different models. The nodes of the piers are always fixed on vertical direction, but have spring on both horizontal directions. The spring constant varies between the different models. On the first one, the constant is 1000kN/m, 10000kN/m for the second and infinite for the last one.

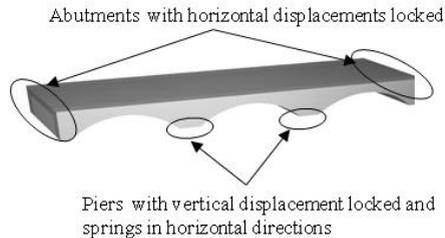


Figure 4 : Boundary conditions

The load cases analyzed are: self weight alone and self weight with the load from the jack that has been increased from 1000kN to 7000kN in six steps.

3.2.2 Results

Several variables have been obtained in the model, in order to contrast with the ones measured on the load test. At the same time, the variables obtained are significant of the structural behaviour of the bridge and help to understand how this type of bridges work.

3.2.2.1 Vertical displacements on the ring course depending on the load applied

Seven load cases have been analyzed on the 3D model. For every load, the vertical displacement has been measured. The selected node lays on the lowest side of the ring course which is as close as possible to the load introduced.

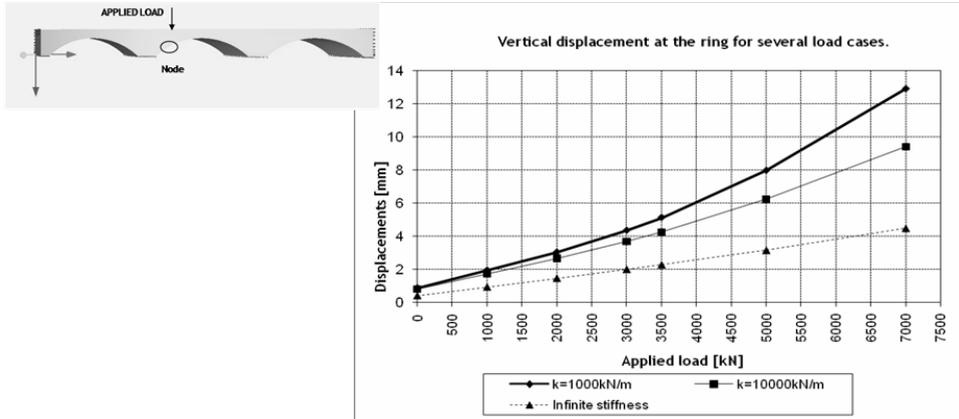


Figure 5 : Vertical displacements on the ring course charged.

For the load introduced by the jacks (approximately 3500kN) the vertical displacement should be between 2 and 5mm. It is remarkable that for stiff piers the response is linear, in spite of the non linear analysis, while the model with flexible piers shows a non linear response when the load applied is higher than 3500kN.

3.2.2.2 Horizontal displacements and reactions on the pier, depending on the stiffness of the piers

In order to determine the influence of the stiffness of the piers in the general behaviour of the bridge, displacements and reactions on the piers have been obtained.

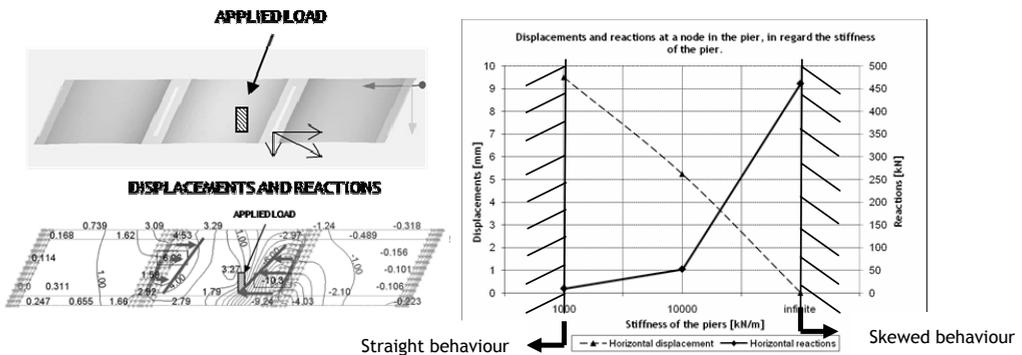


Figure 6 : Displacements and reactions on the piers.

As seen in the Fig. 6, while the stiffness of the piers is low the displacements are small and the reactions on the piers are big, as it was expected. It will be later demonstrated that the stiffness of the piers is of great importance in the general structural behaviour of the bridge. The great reactions that appear on the stiff piers cause their rotation on vertical axis, proving the theory of rotation of piers around vertical axis on skewed bridges

Also worth mentioning is that displacements show a linear evolution with a constant slope, while reactions increase exponentially.

3.2.2.3 *Principal stresses on the vault, depending on the stiffness of the piers*

Two different directions of the principal stresses are seen on the vault. The first one lays on the straight direction of the vault, while the second represents the skewed direction.

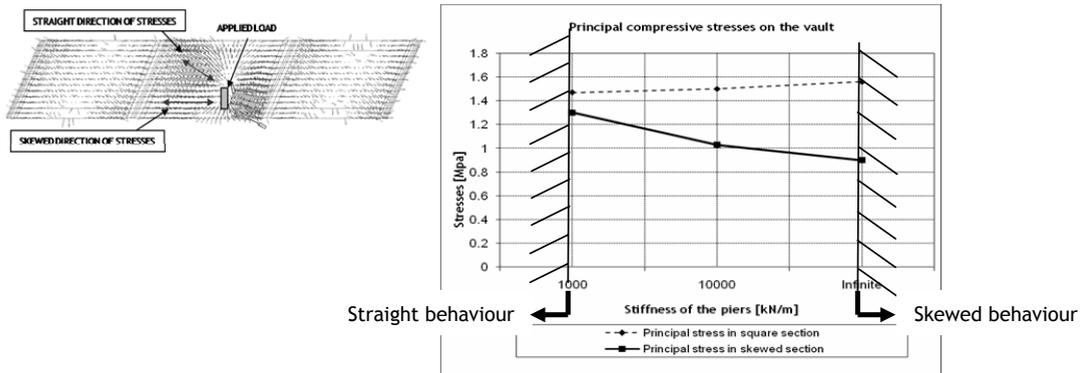


Figure 7 : Principal stresses on the vault.

Results make clear that stresses tend to follow the skewed direction when displacements of the piers are allowed. On the other hand, for stiff piers the stresses appear mainly on the straight direction.

If the piers are flexible, the stresses are of similar value. In the case of stiff piers the stress on the straight direction is much higher.

3.2.2.4 *Normal stresses on the vault*

It has already been mentioned that the calculations were made under a non linear analysis, limiting the maximal stress tension of the masonry to 1MPa. As shown in Fig. 8, the maximal tension stresses are reached by the load of 7000kN, which means the formation of a hinge. The rest of the hinges that should be found according to Melbourne (1995) can already be seen.

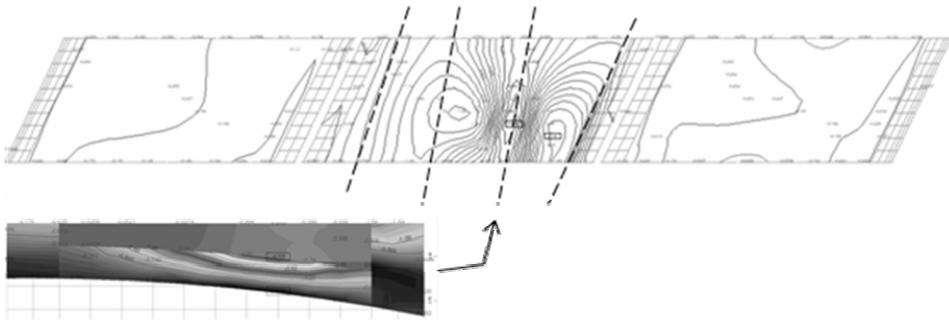


Figure 8 : Hinges formation on the vault and transversal section.

4 CONCLUSIONS

In order to increase the knowledge of the structural behaviour of skewed bridges is of great importance to program load tests on existing bridges. The performance of every test made (non destructive tests, collapse test and controlled demolitions) may give information about the structural components and behaviour of the bridge.

The load test of the bridge over the Rubí river evidences the significance of the masonry backing height and the stiffness of the piers on the global behaviour of skewed bridges. The latter 3D analysis proves that to assure the usual theories about skewed bridges, it is necessary for the piers and foundations to resist the torsion efforts as shown in Fig. 9.

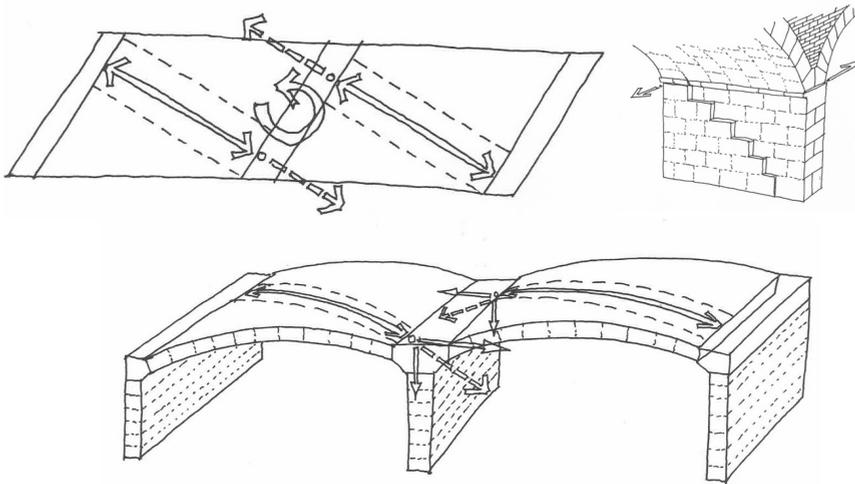


Figure 9 : Schemes of the structural behaviour of the skewed bridges.

Last but not least is the need to analyze skewed bridges on 3D models. The great influence of the skew does not allow its analysis on simplified 2D models.

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