Non-linear behaviour of the piers of the Millau viaduct

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Summary

The Millau viaduct is an exceptional structure in terms of its length, but above all in terms of the level of its deck, 270 m above the river Tarn. The height of the piers varies from 78 m (P7) to 245 m (P2)

The dimensions of the piers are essentially determined by:

- the effects of the wind for piers P2 to P6
- the effects of the wind but also by the extreme temperature variations for the end piers P1 and P7

The calculations have enabled two sources of non-linearity to be taken into account:

- second order effects
- the non-linearity of the behaviour of reinforced concrete (cracking and plastification of the concrete)

The non-linear calculations allowed the size of deflections in the piers to be estimated and confirmed the stability of the structure as a whole when subject to the most unfavourable stress conditions due to the effects of wind or thermal variations.

Keywords: piers; prestressing; non-linear calculation; temperature; wind load; displacement.

1. Introduction

The Millau viaduct is an exceptional structure in terms of its length, but above all in terms of the level of its deck, 270 m above the river Tarn. The height of the piers varies from 78 m for the shortest (P7) to 245 m for the highest (P2).

The required useful life of the Millau viaduct of 120 years led the project owner to consider the behaviour of the reinforced concrete piers.

Four types of loads produce stresses in the piers:

- the weight of the structure itself and its equipment
- the traffic
- the effects of variations in temperature of the deck
- the wind

Examination of these stresses shows that the dimensions of the piers are essentially determined:

- by the effects of the wind for piers P2 to P6
- the effects of the wind but also by the extreme temperature variations for the end piers P1 and P7, because the deck is bound to the piers by fixed bearings

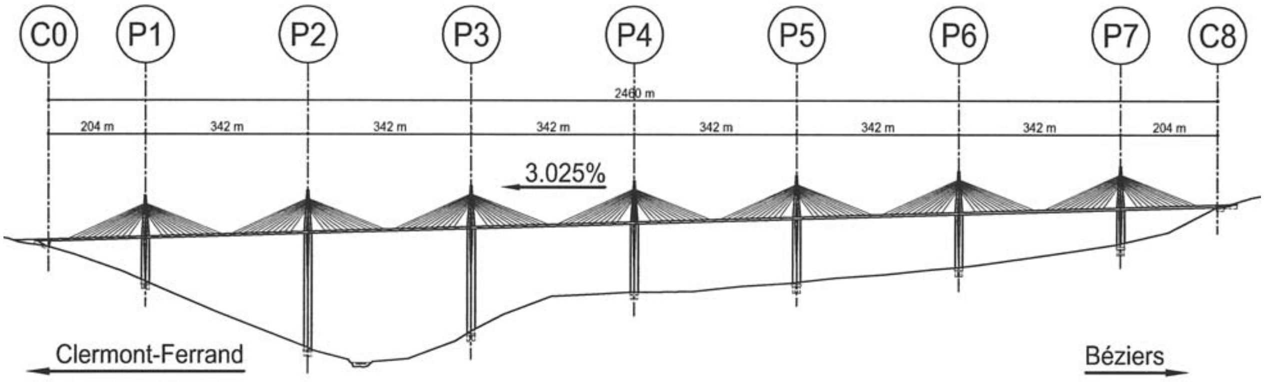


Figure 1: Elevation

When the wind is from the west or from oblique directions, north-west or south-east, the stresses induced are relatively significant and the resulting deflections are of the order of several tens of centimetres.

The calculations enabled two sources of non-linearity to be taken into account:

- second order effects
- the non-linearity of the behaviour of reinforced concrete (cracking and plastification of the concrete)

Examination of the static model allows the piers to be grouped in terms of their sensitivity to non-linear effects:

- Piers P2 to P6: under the effect of the wind, the transversal displacement of the deck can attain 0.58 m (SLS). The exceptional height of these piers (from 245 m for P2 to 112 m for P6) gives them a significant slenderness ratio and thus a significant sensitivity to geometrically non-linear effects.

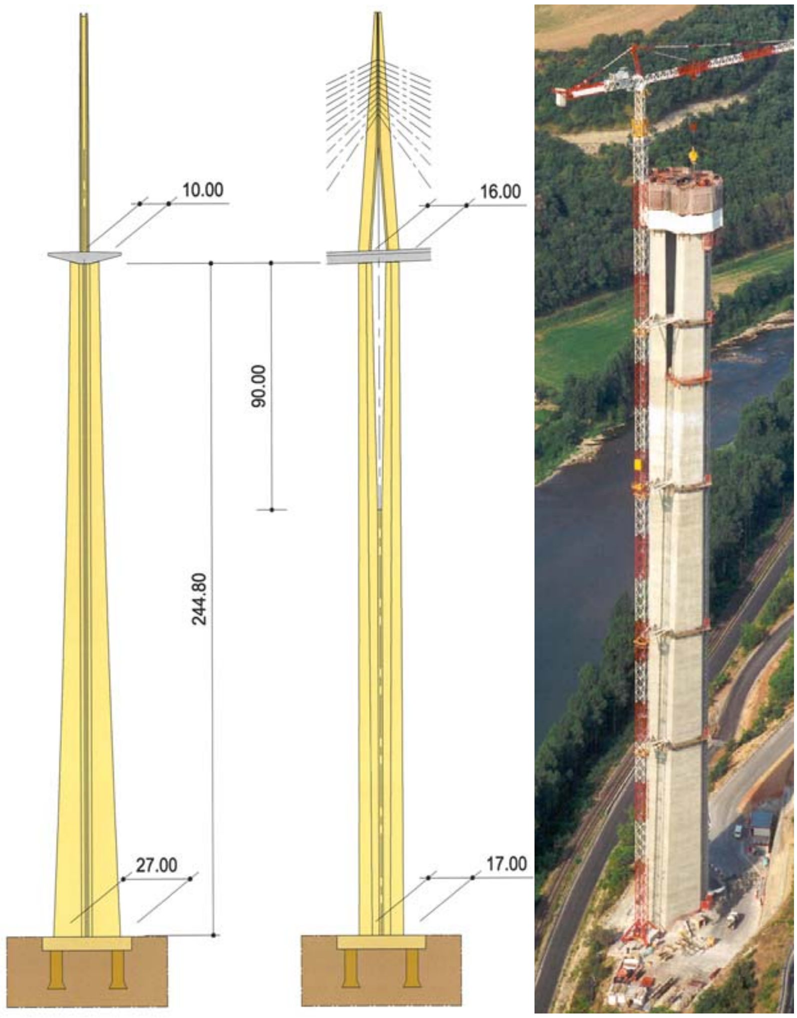


Figure 2

- Piers P1 and P7: under the action of temperature variation in the deck, the end piers will undergo big longitudinal displacements although they are the shortest and thus the most rigid. At the frequent SLS (ΔT = 24°) and the rare SLS (ΔT = 40°), these displacements equal respectively 0.25 m and 0.42 m. The resulting longitudinal moment is likely to crack the section of the piers and thus amplify their sensitivity to non-linear effects.

2. Calculations made

Two types of simulation were made taking into account non-linear effects:

- a study of the behaviour of piers P1 and P7 under the combined effect of variations in temperature and side wind, and the sensitivity of that behaviour to the influence of different parameters such as the distribution of the passive reinforcing, the law of the behaviour of concrete under tension and the presence of pre-stressing.
- an overall study of the behaviour of the structure under the action of vertical loads and of the wind

In the first place Greisch consultancy undertook the study of the behaviour of piers P1 and P7 and thus showed the value of pre-stressing the whole height of the split shafts of the piers, pre-stressing which Michel Virlogeux had recommended. The second study, undertaken by the design office of Eiffage TP, Arcadis ESG and Geonumeric, allowed the dimensions of the piers to be validated, at the same time confirming the overall stability of the structure.

3. Modelling and the laws of materials behaviour

The calculation models were undertaken by means of classic finished elements of the beam type for the piers as well as for the pylons and the deck. The constitutive laws adopted were as follows:

- elastic for the metal deck and pylons
- elasto-plastic, taking into account cracking, for the piers

In order to model its non-linear behaviour, each section of concrete is modelled using the technique known as multi-fibre, each fibre is attributed the constitutive law either for concrete, for passive reinforcement or for pre-stressing cables.

In the program developed by Geonumeric, the results for each section are given in terms of the level of strain in the fibres of steel and concrete as well as the position of the extreme values of this strain (figure 3).

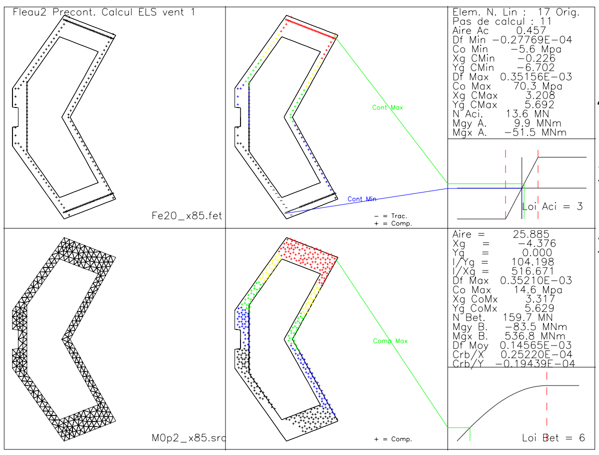


Figure 3

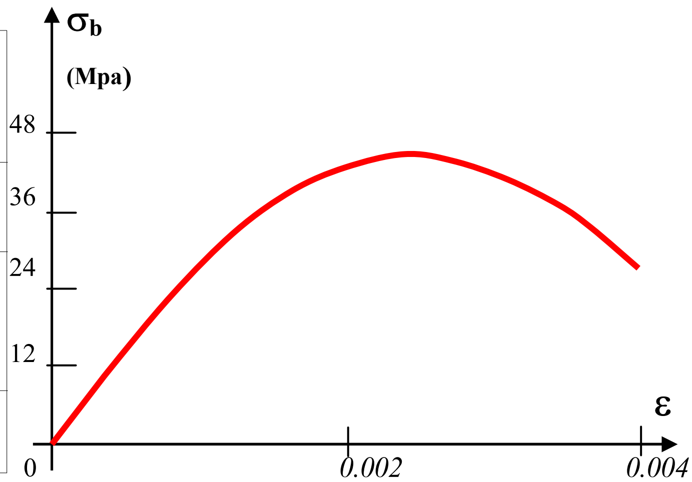


Figure 4

For concrete fibres in compression, Sargin's law is used and in tension an elastic law is used limited to one constraint, either nil or equal to $f_{t28}/2$. At the ULS, the slope at the origin of the curve should be equal to E_{i28}/γ_b, avec γ_b=1,35; this leads to E_{i28}=31 900 MPa for a concrete of type B60. The deformation at the peak of compression is given by:

$$\varepsilon b0 := .62 \cdot 10^{-3} \cdot \sqrt[3]{fc28 \cdot MPa^{-1}}$$

The deformation/constraint curve is shown in figure 4.
For the calculations at the SLS, Sargin's law is used without a reducing coefficient; in this case the modulus of deformation at the origin is 43,060 MPa (figure 4).

The constitutive law for the passive steel fibres is modelled by a classic bi-linear law.

For the pre-stressing cables, the law of behaviour defined by the BPEL is adopted, as shown in figure 5.

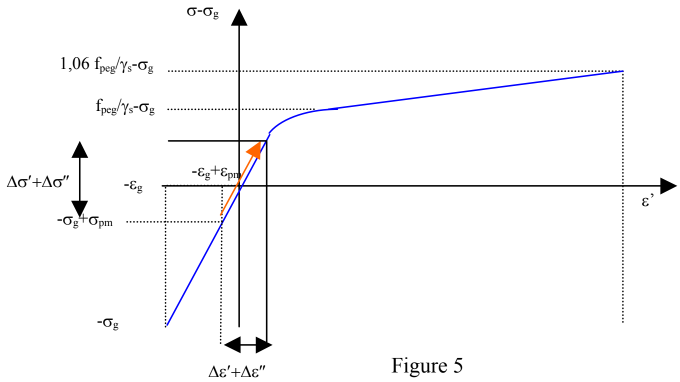


Figure 5

4. Main results of the non-linear calculations

4.1. Preliminary studies on the effect of non-linearity in the concrete

The expansion of the deck due to variations in temperature produces a longitudinal moment in the piers which is likely to crack their shafts. To this must be added the dynamic effects of cross-winds which will cause stresses which are variable both in sign and in value:

- the drag effects of the wind will produce transverse moments in the pier
- lift effects of the wind on the deck, longitudinal moments in the pier or normal stresses in each shaft

It was important to try and describe the behaviour of piers P1 and P7 under the combined effects of these loads.

Three types of calculation were made to try and find an answer:

- a study of the whole structure under cross-winds with the state of cracking of the piers as the parameter
- a study of piers P1 and P7, with the non-linear behaviour of the concrete, and the installation of prestressing as parameters
  - considering only thermal variations,
  - under cross-winds after cracking of the pier under the effects of temperature.

4.1.1. Spectral analysis of the whole structure under wind load

The calculation under turbulent wind conditions is made by spectral analysis by Greisch, and is thus a linear calculation. The state of cracking of the pier is taken into account by replacing its stiffness by a secant stiffness equivalent pro-rata to the stresses generated by the variations of temperature in the deck.

Table 1 shows the displacements of the tops of the piers under this load condition as well as the hypotheses adopted for the calculations under wind load.

Table 1

<table>
<tr><td colspan="2"></td><td>P1</td><td>P2</td><td>P3</td><td>P4</td><td>P5</td><td>P6</td><td>P7</td></tr>
<tr><td colspan="2">Displacement at ΔT°= 40°</td><td>0.419 m</td><td>0.297 m</td><td>0.174 m</td><td>0.053 m</td><td>-0.066 m</td><td>-0.185 m</td><td>-0.310 m</td></tr>
<tr><td rowspan="3">% of the elastic rigidity</td><td>Hypothesis 1</td><td colspan="7">100 %</td></tr>
<tr><td>Hypothesis 2</td><td>50 %</td><td colspan="5">100 %</td><td>50 %</td></tr>
<tr><td>Hypothesis 3</td><td>50 %</td><td colspan="2">70 % of the height of the shafts</td><td colspan="3">100 %</td><td>50 %</td></tr>
</table>

Table 2

	Hypothesis 1	Hypothesis 2	Hypothesis 3
Frequency			
1st transverse mode	0.194 Hz	0.192 Hz	0.188 Hz
1st vertical mode	0.212 Hz	0.188 Hz	0.187 Hz
Displacement of the deck			
Longitudinal sense	0.003 ± 0.082 m	Not calculated	0.003 ± 0.094 m
Transverse sense	0.181 ± 0.609 m	Not calculated	0.189 ± 0.633 m

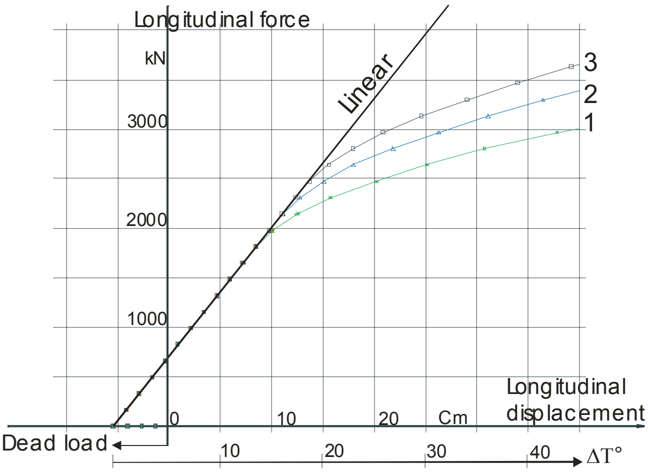
Table 2 shows that the cracking leads to a very slight reduction in natural frequencies. The displacements under the effects of wind also change very little and the structure shows a more than satisfactory overall behaviour. The local losses of rigidity are in fact compensated for by the overall behaviour of the structure.

4.1.2. Non-linear parametric analysis of piers P1 and P7

These simulations were carried out using the FINELG program, developed by Greisch and were based on the doctoral studies of H. Somja

4.1.2.1 Behaviour of piers under thermal variations

The hypotheses and parameters adopted are as follows:



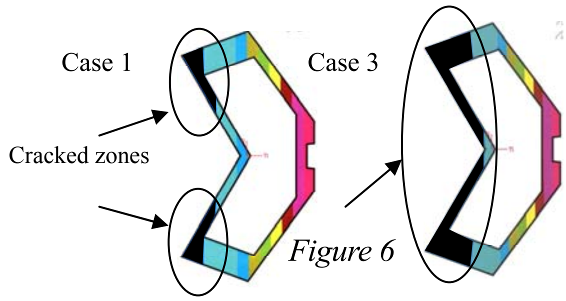


Figure 6

- modelling of pier P1 with an isostatic beam fixed at the base and free at the top. The reactions of the deck are applied at the top of the model and the stresses distributed throughout the length of the shaft. Only the southernmost split shaft of pier P1 is discretised
- application of permanent loads and of longitudinal displacement equivalent to the thermal expansion of the deck
- use of an elasto-plastic law of B60 concrete and resistance under zero tension
- percentage of passive reinforcement 0.8 % at the base and 1.6 % at the top
  - case 1: without pre-stressing,
  - case 2: with a constant pre-stressing of 20000 kN,
  - case 3: with variable pre-stressing of 20000 kN at the top, to 40000 kN at the base of the split shafts.

Under the effect of temperature variation, pre-stressing leads to a reduction of 50 % in the cracked zones despite the fact that the stresses in the heads of the shafts are approximately 20 % higher. Complementary studies have shown that taking into account an elastic law with tension limited to 4.21 MPa for the concrete leads to an additional 10% increase in this stress.

4.1.2.2 Behaviour under the effects of wind

The objective of this study was to check the behaviour of the same piers under the alternating stresses caused by the wind when it is more than possible that the concrete sections have already been cracked due to temperature variations.

The dynamic load of the wind was replaced by equivalent static loads on the basis that this change would allow the reproduction of the maximal effects obtained by spectral analysis. In the case of piers P1 and P7, the maximal wind effects to be reproduced were the normal stress in the head of the pier and the transverse moment at the level of the foundation. The evaluation of these equivalent loads from the spectral analysis, in other words from a linear analysis which does not take account of cracking, was considered to be valid because paragraph 4.1.1 shows that the non-linear behaviour of the concrete does not modify the overall behaviour of the structure under wind load.

The following loads were added sequentially and incrementally:

1: the weight of the structure itself and its equipment
2: increase in the temperature of the deck (+40°) followed by a reduction (-30°) to provoke cracking in the piers. At this stage, the pier is loaded by its own weight plus the effect of a temperature increase of 10°.
3: application of the effects of an EAST wind, combining stationary and turbulent effects. At this stage, the pier is loaded by its own weight plus the effect of a temperature increase of 10° plus the effects of an east wind.
4: suppression of the turbulent element of the effects of the EAST wind. At this stage, the pier is loaded by its own weight plus the effect of a temperature increase of 10° plus the stationary effects of an east wind.

The parameters of the study were as follows:

- Case 1: linear elastic law of concrete
- Case 2: elasto-plastic law of concrete, resistance under tension (f_{tj}) nil and pre-stressing absent
- Case 3: elasto-plastic law of concrete under compression, linear elastic law under tension, limited to 4.21 MPa and pre-stressing absent
- Case 4: elasto-plastic law of concrete, resistance under tension nil and constant pre-stressing over the entire height of the pier (20,000 kN)
- Case 5: elasto-plastic law of concrete, resistance under tension nil, pre-stressing force of 20,000 kN over the upper half of the pier and 40,000 kN over the lower half.

These studies allow the following conclusions to be drawn (figure 7):

- there is a three-fold difference in the maximum displacement between case 1 (8 cm) and case 2 (24 cm)
- taking into account the resistance under tension of the concrete tends to reduce the displacement by a factor of 2 (cases 2 and 3)
- the inclusion of pre-stressing in the piers reduces the displacement to 12 cm (case 5).

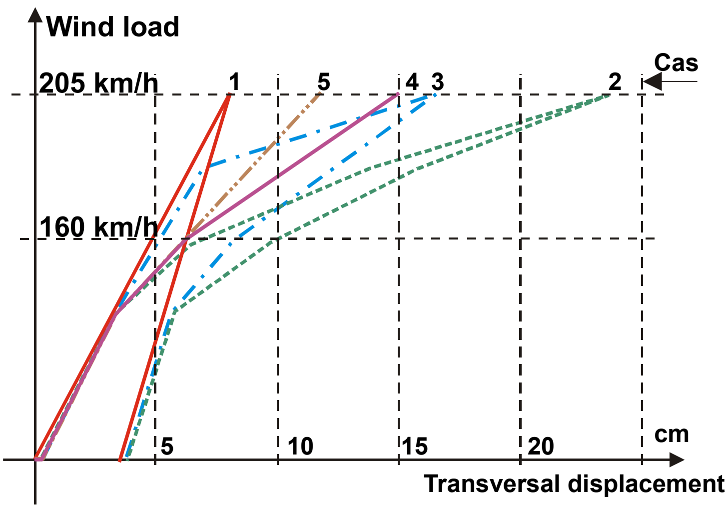


Figure 7

It must however be noted that for wind speeds of 160 km/h, which are frequently encountered at the site, the behaviour of the pier is almost identical whichever hypothesis is adopted, which shows that the design of the piers is satisfactory. In order to ensure the required useful life of the structure of 120 years, it was nevertheless decided to pre-stress the split shafts as in case 5.

4.2. Overall stability of the viaduct

Based on the conclusions from section 4.1, Geonumeric undertook checks on the overall stability of the viaduct.

The reduction in the stresses in piers P1 and P7 under the thermal effects due to cracking is of the order of 30% under the hypothesis which ignores the rigidity of stressed concrete (figure 8). This result accords well with that obtained during the analysis of an isolated pier with Sétra's Pylostab program:

- Taking into account the stiffness of stressed concrete (step at $f_{tj}/2$) leads to an increase of approximately 20 % in the moment of longitudinal bending at the base of the shafts.
- It was confirmed that in the extreme case of wind where one of the split shafts is totally cracked, the other shaft, highly compressed, is able to take up on its own the concomitant shear and torsion forces acting on the pier.
- The displacements obtained under extreme wind conditions at the ULS at the top of the highest pier (P2) with a constitutive law for concrete recommended for the ULS (Ei28 = 31,900 MPa and with $\sigma_t = 0$ are given in table 3 below:

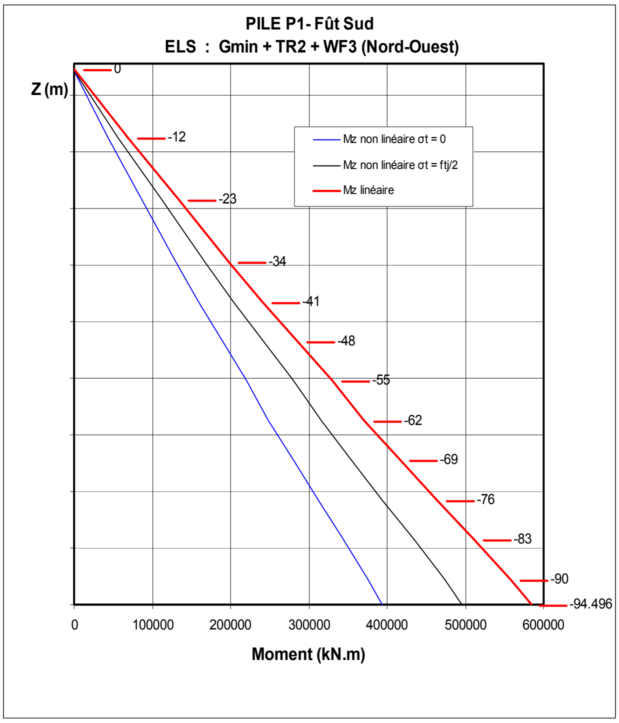


Figure 8 : Pier P1

Longitudinal moment: linear / non-linear comparison

Table 3

Type of analysis	U_X – north shaft (m)	U_Y – north shaft (m)	U_X – south shaft (m)	U_Y – south shaft (m)
Linear (ULS)	0.0375	0.857	0.0379	0.870
Non-linear (ULS)	0.0567	1.451	0.0575	1.472

A significant increase should be noted in the transverse deflection, UY, between the linear and non-linear calculations.

In the split shafts of piers P2 to P6, the percentage of passive reinforcement used is between 0.8% and 0.9%. In the single shafts of these piers, the percentage is 0.8%. For the most rigid piers, P1 and P7, the percentage of passive reinforcement in the lower part of the split shafts is significantly higher at around 1.75%

5. Conclusion

The non-linear calculations undertaken for the study of the behaviour of the piers of the Millau viaduct allowed measurements to be made of the influence of pre-stressing on the cracking of the piers, which is an important factor for their longevity and hence for that of the structure as a whole, and also to confirm the stability of the structure as a whole when subject to the most unfavourable stress conditions due to the effects of wind or thermal variations and to estimate the size of the deflections.

6. References

H. SOMJA: doctoral thesis to be submitted in October 2004 at the University of Liège, Belgium. "Contribution to the modelling of the phasing of the construction of civil engineering structures taking into account material and geometrical non-linearity".