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BEHAVIOUR OF TWO GATE-STRUCTURE DAMS UNDER SEISMIC LOAD

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ABSTRACT

This paper presents the behaviour's analysis of two gate-structure dams under seismic load. Because of the height of the piers, compare to their thickness, and also because of the mecanicals equipements on top of the piers, where the amplification of the groundmotions is the most important, seismic behaviour of these kinds of dams has to be correctly evaluated. This paper will present the seismic assements realized for both dams : a linear analysis in the case of Tuilieres and the differents reflexions that leaded to design a new bracing of the piers, and a non-linear analysis in the case of Cize-Bolozon dam.

INTRODUCTION

Generally, the main part of the research about behavior of concrete dams under seismic load focus on high arch dams, subject to high level of ground motions. This is not the target of this paper. Gate-structure dams, generally between 20 and 40 m high, are a lot less known for their seismic behavior: built between 1900 and 1950, they were designed with slender piers on top of which stay a system of winches to raise up the gates. Despite of low level of ground motion, but because of their shape, these gate-structure dams seem particularly vulnerable to earthquake.

The seismic assessments of two dams: Tuilieres and Cize-Bolozon dams are presented in this paper. For the first one, an improvement of the seismic behavior of the dam has been realized as part of a global project of rehabilitation of the dam. For the second, only the dam analysis, considering non-linear behavior, is proposed.

PRESENTATION OF TUILIERES DAM

The Tuilières dam (Fig.1), built in 1908 on the Dordogne River, 20 km upstream of Bergerac, south-west of France, is composed by:

- A masonry gate-structure dam (subject of this paper), adjacent to the left bank,
- The powerhouse with 8 Kaplan units with an installed capacity of 38 MW, between the gate structure and the right bank.

The gate-structure is made of 9 masonry piers. The first sluice on the right side is 7 m wide whereas the others are 10 m wide. sluices are closed by Stoney gates, vertically moved by a system of winches located on a platform on the top of the piers at elevation 49.00, and counterweights. A reinforced concrete footbridge at elevation 36.00 is used to cross the dam and serves furthermore as a bracing.

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The elevation of the full supply level is 31.10 NGF while the normal tailwater level is about 19.00 NGF and the concrete apron 17.73 NGF. The storage volume of the reservoir is 5 millions of m^3.

Figure 1. Global view of the Tuilières dam, from downstream.

The main geometric characteristics of the piers are (Fig.2):

- Thickness: 3 m (except pier n°2 : 4 m),
- Height: 31.30 m above the concrete apron,
- Length upstream-downstream, at base level: 16.65 m.

The piers are built on a relatively thin apron made of concrete with a cut off. There is no drainage system. The dam is built on quality limestones which form good rock foundation.

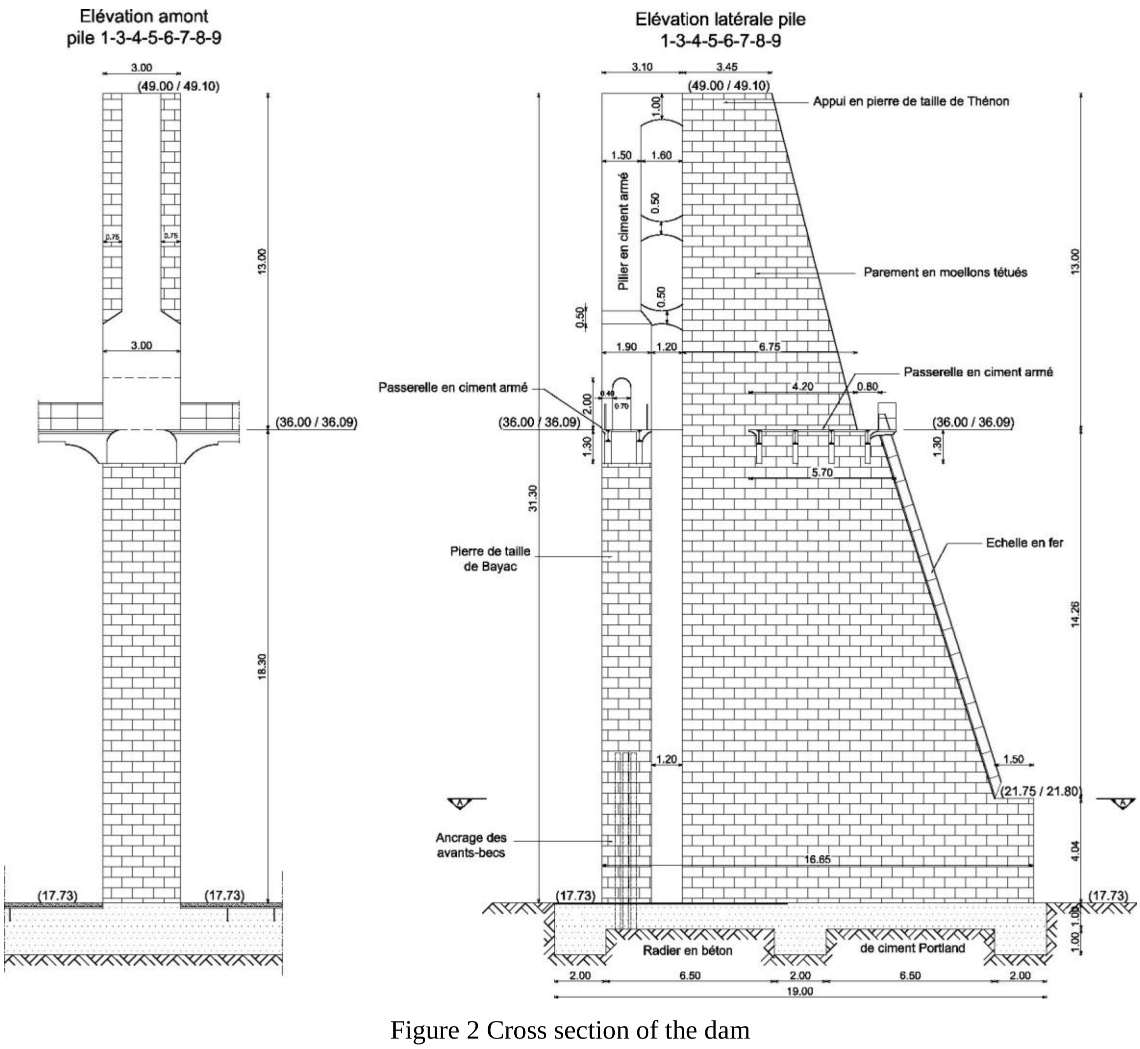

Figure 2 Cross section of the dam

ANALYSIS OF TUILIERES DAM

As the dam was under a important project of rehabilitation, a finite element analysis of the whole gate-structure dam has then been performed in order to understand the behaviour under earthquake (Robbe 2010).

The finite element model represents the whole gate-structure dam with the 9 piers and their bracing, the apron and the foundation with about 14,000 solid twenty-noded brick elements and 70,000 nodes. The gates have not been modeled : the water pressure will be directly reported on the piers. The abutments on the left side and on the powerhouse are also taken into consideration. On the top of the pier, elements with adequate density represent the weight of the top-platform, the winches and the counterweights.

The seismic analysis is realized with Code_Aster using a modal recombination method :

- computation of the elastic eigenmodes,
- selection of the eigenmodes (only the most representatives are take in account),
- computation of the dynamic response in the eigenmodes basis,
- post-processing of the displacements, acceleration, stress on nodes,
- superposition of the stress fields computed from this dynamic analysis and a static analysis (gravity, water load applied on the piers, uplift).

The choice has been made to represent the hydrodynamic load with the Westergaard method (Westergaard, 1930) : as we are looking for the side-to-side behaviour of the piers, the influence of the hydrodynamic effect is considered weak.

A seismic risk study has been realized in order to determine the earthquake that can affect the dam : a deep earthquake, intensity VI-VII MSK and a peak ground acceleration of 0.04 g, a shallow earthquake, intensity V-VI MSK and peak ground acceleration of 0.01 g. The last one is confirmed for the analysis as the stronger in the frequencies concerning the dam. From this spectrum, 3 accelerations signals are generated and combined in order to produce the most unfavorable ground motion for the structure.

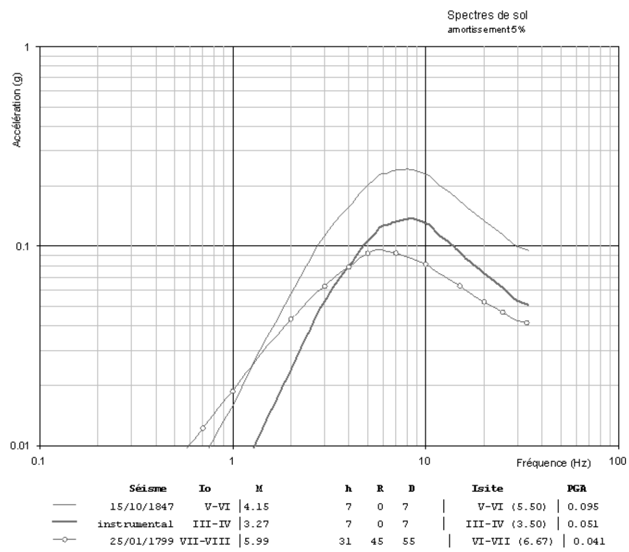

Figure 3. Ground spectrums for Tuilieres dam

The modal analysis computes 100 eigenmodes between 0 and 30 Hz. Only 24 are really involving a significant percentage of weight. The first one is obviously a side-to side and synchronized flexure of all the piers, at 2.77 Hz. The first flexure mode in the upstream-downstream direction occurs for 8.32 Hz, at the peak of the spectrum. Both of them affect about 40% of the weight of the structure in their direction.

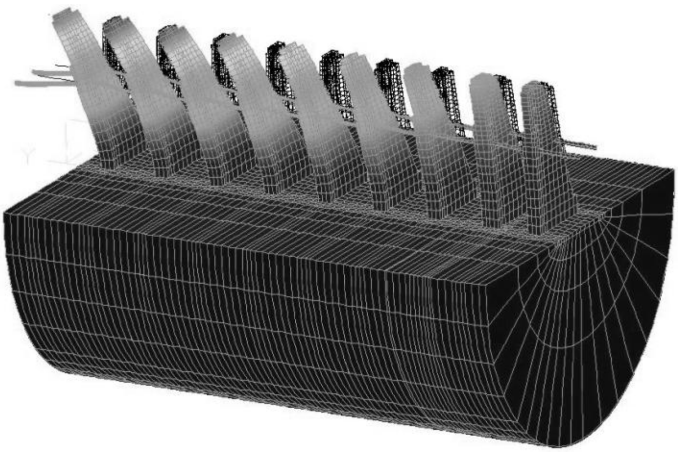

Figure 4. Deformed shape for the first mode

The dynamic analysis shows that a synchronized and side-to-side oscillation movement of flexure of the piers could probably occur during earthquake. Displacements about 1.2 cm are computed on top of the piers. On the contact pier-apron, tensile stresses of 0.7 MPa are observed, that could lead to a full degradation of the contact.

In such event, the bracing is very important to prevent an increase of the displacement between two pears that can lead to block the gate. But high values of tensile stress (from 2 to 4 MPa) are also computed at the contact between bracing and piers. The masonry won't probably accept such values and the link between bracing and piers will probably break.

One of the restriction of the project was that the rehabilitated dam as to be as close as possible of the original design. The purpose is then to improve as best as possible the behaviour under seismic load with the slightest modifications. The rehabilitation project brings also some transformation of the gate-structure dam according to others needs :

- the noses of the piers are extended upstream in order to improve the way to install stoplogs,
- replacement of the whole system of gates, winches, counterweights and the upper-platform modify the repartition of mass on top of the piers.

This leads to a modification of the finite-elements model and have an influence on the behaviour presented on the followings chapters.

In order to reduce the side-to-side flexure of the piers under seismic load, a continuous reinforced bracing with strong abutments on the powerhouse and on the left side has been imagined. Unfortunately, such solution leads to an increase of the first eigenmode in frequency. As the peak of the spectrum is around 8 Hz, under this value, an increase of the eigenfrequency leads to an increase of the acceleration. In consequences, such bracing doesn't reduce the displacements on top of piers ; moreover it increases the force on the abutment.

Others analysis show that the release of the abutment on the side of the gate-structure leads to a decrease of the first eigenfrequency and a reduction of displacements on top of piers.

Finally, a new bracing is designed 3 m above the level on the old one. This raising contributes also to lower the first eigenfrequency. The first eigenmode of the gate-structure is now about 1.93 Hz with the followings effects on the dynamic results:

- side to side max displacements on top of piers : 0.6 cm (instead of 1.2 cm before),
- effective tensile stress at the contact pier-apron : 0.17 MPa (instead of 0.7 MPa),
- tensile stress at the contact pier-bracing : 0.6 MPa (instead of 2.7 MPa before).

This study confirms the sensibility of this type of dam composed of slender piers with heavy mass at top, under seismic load. In this case, at top of the piers, accelerations of 0,4 to 0,5 g have been computed : about 5 times the peak ground acceleration.

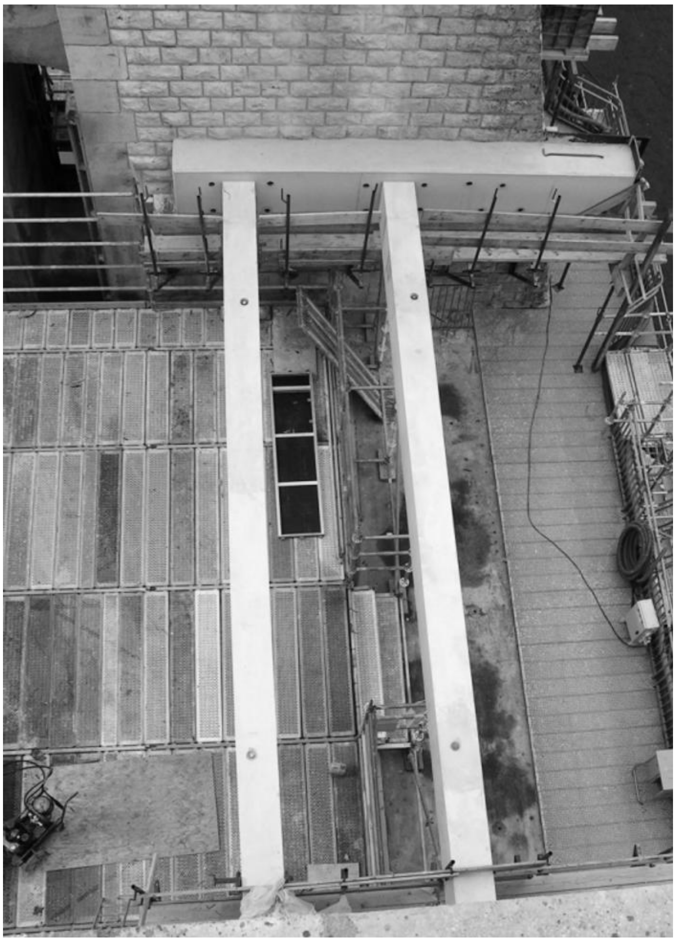

Figure 5 The new bracing

CIZE-BOLOZON DAM

Figure 6 downstream view of Cize-Bolozon dam

The Cize-Bolozon dam (Figure 6), built in 1931 on the Ain river, 80 km upstream of Lyon, east of France, is composed by concrete gate-structure, designed with 3 sluices closed by Stoney gates. An important but not reinforced superstructure stands on the top of the dam to support winches and a hoist.

Considering the slender of the piers, a finite-element analysis of this part of the dam is requiered to evaluate not only the level of stress in the whole structure but also the sollicitation on the top of the pier in order to assess the mecanical equipement used to open the gates.

As different bridges between the piers are not designed as bracing, the study consideres there are effort’s transfert between piers ; accordingly finite-element mesh of the structure only represent one pier as shown on fig.7. As for Tuilieres’s dam analysis, the following assumptions are considered :

- fundation is modeled but considered massless
- added mass with Westergaard formulation are taken into account for hydrodynamic effect of the water
- gates are not modeled but the proper water pressure (static and dynamic) is applied on the pier.

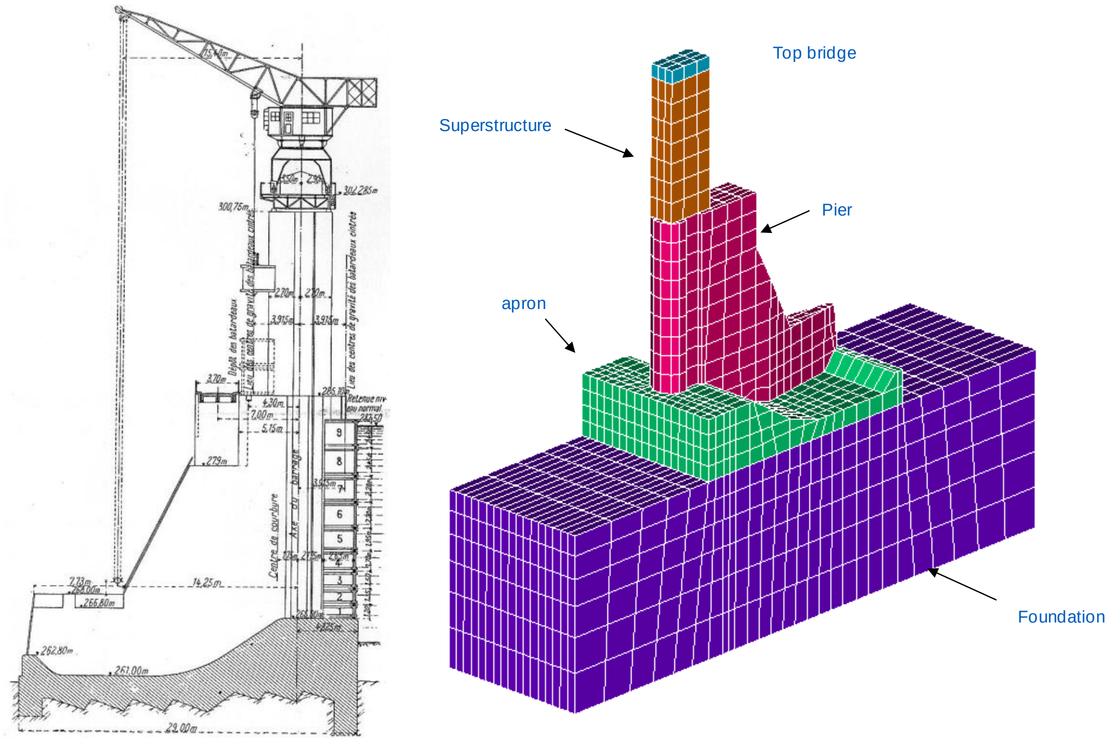

Figure 7

First, a linear analysis with a modal recombination method (similar to the method used for Tuilières dam) has been realized. A seismic risk study determines the earthquake that can affect the dam : an earthquake with intensity VII-VIII MSK and a peak ground acceleration of 0.14 g is selected. From this spectrum, 3 accelerations signals are generated and combined in order to produce the most unfavorable ground motion for the structure.

The first eigenmode of the structure are presented on fig.8 : the first one is a side-to-side mode but it is also important to realized that the 2nd and 3rd eigenfrequencies are closed to the peak of the chosen spectrum (about 8 Hz).

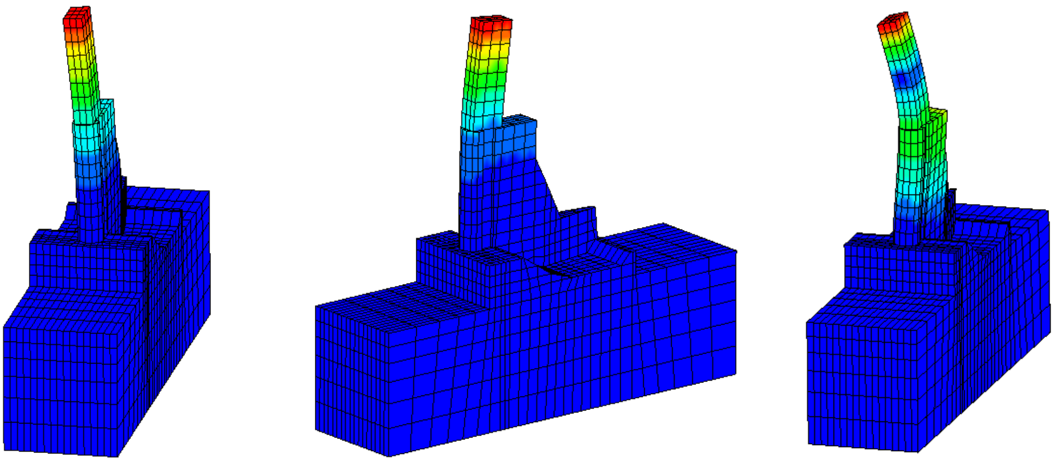

Figure 8 first eigenmode of the structure : 2.29 Hz, 6.43 Hz, 7.63 Hz

Time-analysis shows that the main behaviour of the structure during earthquake is a side-to-side displacement of the top of the pier : the following maximum displacements are computed on top of the superstructure : 18.1 mm in the side direction, 5 mm in the upstream/downstream direction and 1 mm in the vertical direction. This leads to tensile stresses, whose maximum during the whole simulation are represented on fig. 9. High values of verticale tensile stresses are calculated : about 2-3 MPa at the feet of the pier (considered acceptable because of the reinforcement inside the concrete for this part of the dam), and 5 MPa at the link between the pier and the superstructure. Without reinforcement at this level of the dam, this value will likely leads to cracks.

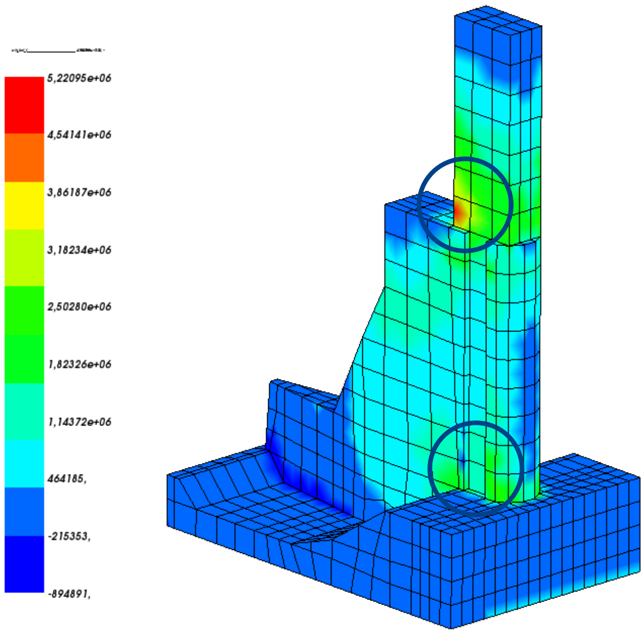

Figure 9 Maximum verticale tensile stresses during the earthquake

In order to evaluate the spread of such cracks and its impact on the behaviour of the whole structure, a non-linear analysis has been realized using the introduction of joint-elements (Laverne et al., 2011) between the pier and the superstructure (fig.10). The behaviour law for these elements is elastic in compression with a limited tensile strength (here 1 MPa).

Fig.10 also shows the spread of the crack with differents views of damage’s increase during the first important side-to-side rocking of the superstructure : corners are successively damaged and after only 2 seconds of ground motions, no more tensile strength left on this area.

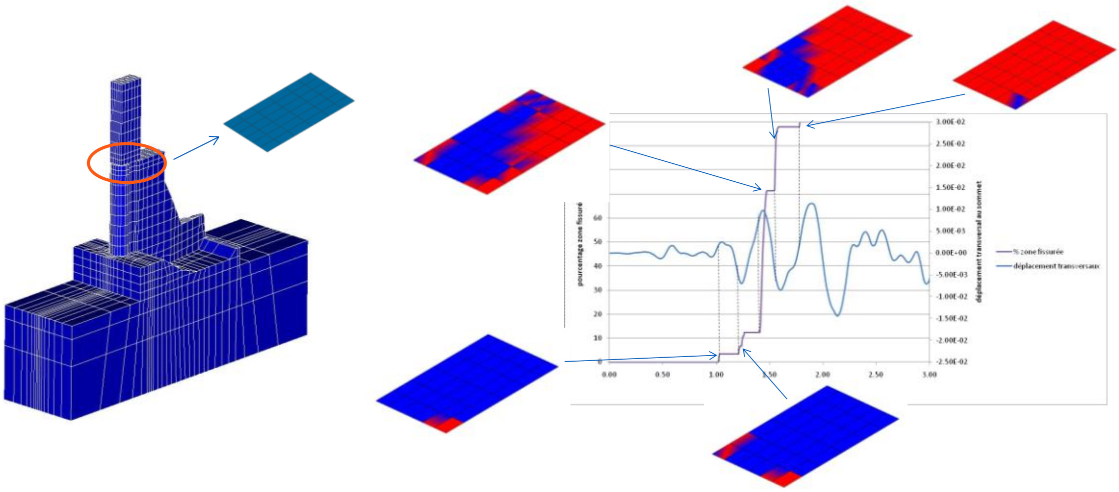

Figure 10 position of the joint-elements and spread of the crack during the ground motion

The influence of the lost of tensile strength between the pier and the superstructure can be observed on the comparison of the displacement on top of the superstructure between the linear and non-linear analysis (fig.11) :

- during the first seconds of the earthquake, displacements are similar,
- from 1.5 s to 4 s, the opening of the joints elements leads to desynchronized the mouvement of the pier and the superstructure, which reduce the top’s displacements
- from 4 s to 6 s, an important increase of the side-to-side displacements is observed (23 mm compare to 18 mm with the linear analysis) : the opening of the joints accentuate the response of the superstructure
- then the values are in the same range.

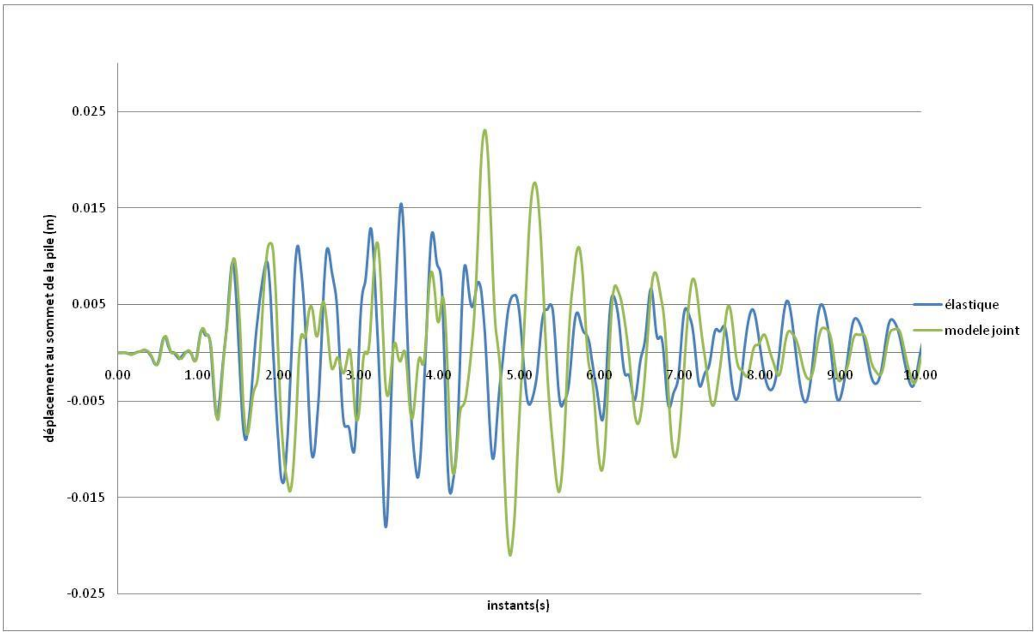

Figure 11comparison of the side-to-side displacements on the top of the superstructure with the linear and non linear analysis

In addition, stresses analysis shows that opening of the joint-elements leads to reduce stress level in the rest of the structure. This finite-element study show that a failover of the superstructure is not going to happen, even whith a full crack between the pier and the superstructure. But the finite-element model does not take into account the likely damaged of the concrete in the feet's corner of the superstructure due to high verticale compression : if that's happen successively at each rocking, a reduction of the support section is likely to occur with strong influence on the behaviour. That's why a reinforcement of this part can be proposed: not to prevent horizontal cracks but to be sure that the feet section of the superstructure remain whole.

Finally, as observed for Tuilieres dam, a important increase of the acceleration on the top of the superstructure has been computed. In Cize-Bolozon's case, an amplification of 6.5 is computed between the peak ground acceleration (0.14 g) and the acceleration for high frequencies values on top of the superstructure (1 g in the side direction)

CONCLUSIONS

This paper summarized the safety evaluation of two gate-structure dams under seismic load. The finite-elements analysis show that this kind of dam are particularly weak considering this kind of events : rocking movements lead to verticale tensile strength in the concrete but also great amplification of the acceleration on the top of the dam, where stay mechanical equipments important for the safety of the dam. In the case of Tuilières dam, an new bracing has been realized in order to improve the behavior of the dam under such events, while in the case of Cize-Bolozon dam, studies are still on their way to find the better solution.

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