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# NUMERICAL ANALYSIS OF THE LA SARCELLE POWER PLANT: A CASE STUDY

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***Abstract:*** **La Sarcelle is part of the Eastmain-1-A/La Sarcelle/Rupert project and it is the first bulb units power plant of Hydro-Québec commissioning in 2013. The power plant is made of 3 large bulb units of 50 MW with a large runner diameter of 7 meters. The owner and the lead consultant had to deal with a new kind of plant introducing strong technical challenges. This implies for the consultant a large R&D component to complete the work, not only to control the bulb plant knowledge, but also to apply this knowledge to a Nordic climate. This paper presents an overview of the structural analysis process: the team work, the finite element models involved for global and local analysis, the loads applied to the critical concrete parts and the tools available in post-processing to understand the structural behaviour of complex structures.**

***Keywords:*** **Bulb power plant, Finite element models, Graphical post-processing**

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## I. INTRODUCTION

The La Sarcelle regulator structure was commissioned in 1980 during the Phase I of the La Grande power plants. Thirty years later, the partial diversion of the Rupert River will significantly increase the water flow to the La Sarcelle site and the existing structure will be unable to correctly evacuate large floods. Hence, a second regulation structure is required.

The low head (11,7 m) and the large flood (1 380 $m^3$/sec) available will be efficiently harnessed using bulb-type turbines. After commissioning, La Sarcelle will become the first bulb turbine power plant in the Hydro-Québec facilities. This first experience is starting with 3 large bulb units of 50 MW using a 7 meters runner, which is one of the largest existing runner diameters. Moreover, it is one of the first applications of this technology in a Nordic climate.

The name ‘bulb’ comes from the shape of the upstream watertight casing which contains a generator located on a horizontal axis. Units are submerged in the water passage and are able to handle significant discharge variations. The straight water passage in the draft tube improves the hydraulic behaviour of the unit, and also results in a low submergence. This, in turn, allows for a reduction in size, cost and civil works requirements. Figure 1 presents a typical elevation view of a bulb unit powerhouse.

There are mainly 3 major technical challenges to solve. The first one is related to the hydraulic modeling of the inlet channel to avoid vortex and frazil ice. The second challenge is the development of new design approaches of all equipments and systems to increase the utilization factor as close as possible to 85% with minimum human intervention. Finally, the third challenge is associated to structural analysis and design.

This paper presents an overview of the structural analysis of the powerhouse. This overview includes: the team work, the finite element models for global and local analysis, the load cases, and, the tools developed to visualize stresses and compute forces and moments, on an arbitrary plane in space for complex structures.

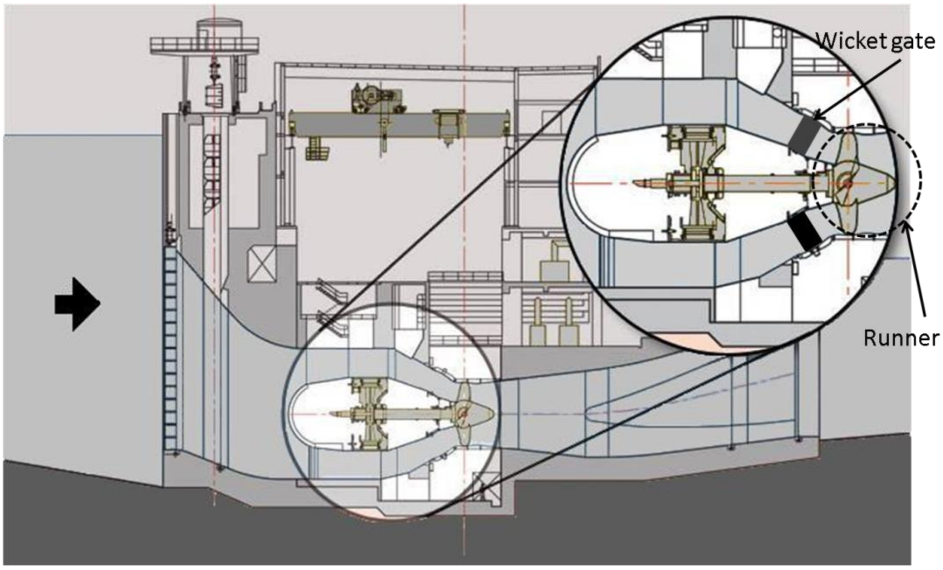


Figure 1 Typical elevation view of a bulb unit powerhouse

## II. TEAM WORK

The client of the La Sarcelle power plant is SEBJ (Société d'énergie de la Baie James) and the team work for the structural analysis is formed by the lead consultant AECOM Tecsult, Hydro-Québec (internal consultant) and the mechanical provider ALSTOM Hydro Canada. The internal consultant had the mandate to verify globally the work done by the lead consultant and to also perform some specific analysis. There is a strong interaction between the loads generated by the bulb unit and the way they are transferred to concrete. For this reason, the present work required a tight collaboration between all the members of the team. It should be noted that the present work has been presented, on a regular basis, to a board of experts.

## III. GLOBAL MODEL

The global model developed is related to one of the bulb units, as stand alone behaviour, and includes the intake, the power house, the draft tube and a consistent foundation as shown in Figure 2. The critical part of the model is the powerhouse and it was discretized using a fine mesh. A coarser mesh was used for the intake, the draft tube and the foundation. The intake and the draft tube modeling was not used to study their behaviour but only to correctly simulate their rigidity and transfer their loads to the power house without any simplifications or assumptions.

The power house is made of two parts: the concrete Phase I and Phase II. The concrete Phase II is the critical part of the present study as all the loads from the bulb unit are applied locally to it. Figure 3 presents the solid model of the power house including the most important components for the structural analysis. The bulb unit is attached to the concrete mainly using the upper and lower stay columns. However, the horizontal rigidity of the bulb unit is increased by the lateral supports orientation. The stay columns are embedded in concrete by extending the stay columns in the upper and lower housing ($\simeq$2 meters). The upstream beam creates a rigid frame in the upstream part of the powerhouse and it is highly solicited by hydrostatic loads. The concrete around the stay ring shroud sustained large tensile stresses from the hydrostatic loads with additional loads coming from the stay ring shroud that is connected by steel plates to the wicket gates (Figure 1). In the present case, the upstream concrete cover of the upper housing has a relatively small thickness and will influence the embedded steel components design. The influence of a steel liner, on the interior face of the Phase II concrete (neglecting the cantilever part), has also been studied.

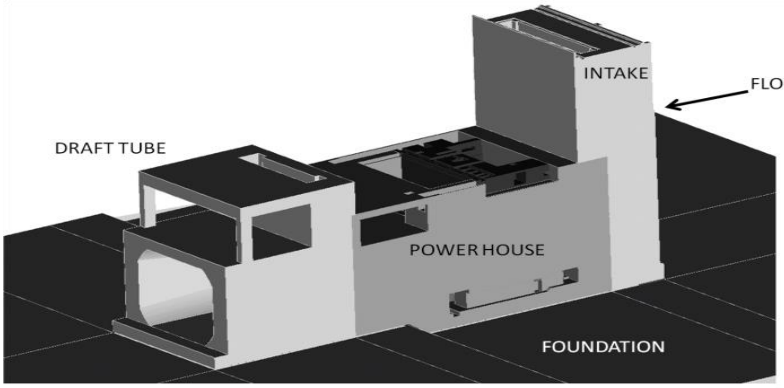


Figure 2 Global solid model

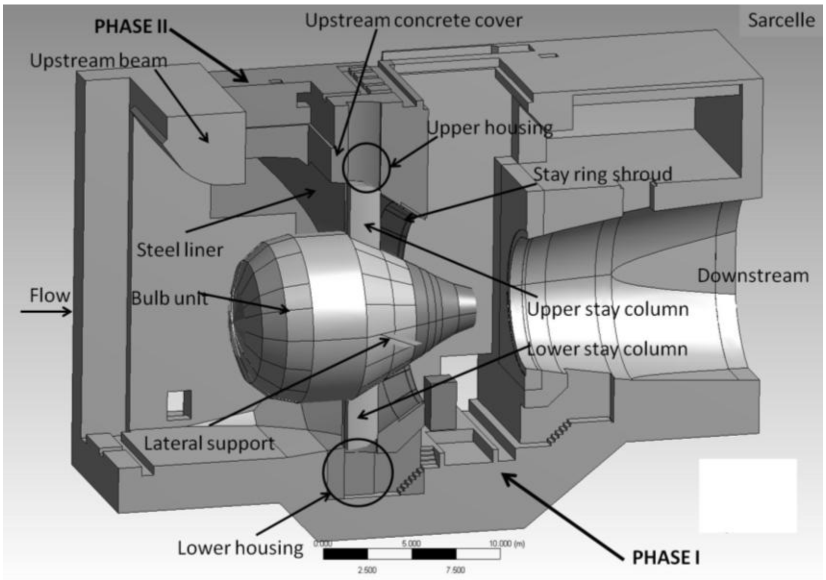


Figure 3 Powerhouse and structural components

The global model was done using linear tetrahedral elements except in the concrete Phase II where linear hexahedral elements were used. Two million nodes and 5.7 million elements are required for the global model: the fine mesh of the powerhouse’s Phase I concrete required 0.5 million nodes and 2.6 million elements and, finally, the critical Phase II concrete mobilizes 1.3 million nodes and 1.2 million hexahedral elements. All the members of the team conducted their structural analysis using the same finite element software (ANSYS) and the global model of the lead structural consultant can be run only on a 64 bits computer.

Some initial assumptions should be made on the load transfer from the bulb unit to the embedded steel parts (stay columns and horizontal supports). The first assumption is related to the load transfer from the bulb unit. The initial loads provided by the mechanical provider supposed fixed end supports of the stay column in contact with concrete (Figure 4). These loads are computed, for example, to the center of gravity of the stay columns ends and subsequently applied to the concrete as static loads. The stay columns are embedded in concrete (upper and lower housing) by extending the stay columns and were initially supposed perfectly encased in concrete. The stay ring shroud on the downstream part of the Phase II concrete was simulated perfectly encased using an equivalent thickness. A lining was also simulated on the surface of the Phase II concrete to connect the stay column and the stay ring shroud. Preliminary computations were made to verify some of these assumptions and were used as guidelines for local modeling.

## IV. LOADING CASES

The load cases applied to the global model should include the static and the dynamic loads. These load cases generate distributed loads (hydrostatic component) and local loads (equivalent static loads) applied at the ends (top and bottom) of the stay column and at the ends of the bulb unit horizontal supports. From the mechanical provider, the horizontal supports do not only stabilize the bulb unit laterally but also transfer shear loads thus modifying the end moments of the stay column (Figure 4).

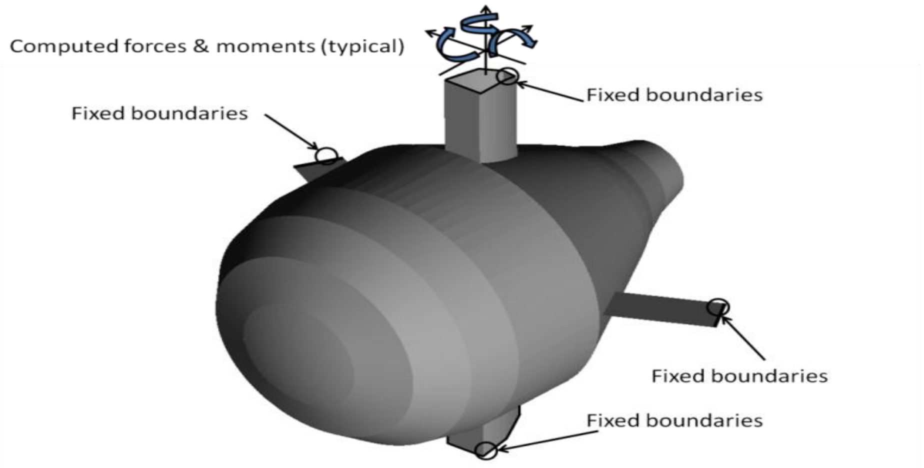


Figure 4 Initial boundaries of the bulb unit model

Usually the thermal loads, generated from contraction or expansion of the bulb unit and the stay columns are neglected. The validity of this approach has been checked using a combined analysis of the bulb unit model (Figure 4) and the global model. In the present case, the upstream plates of the stay columns in contact with the bulb unit are attached to a strong internal ring creating a nearly continuous column introducing large thermal axial loads (Figure 5). For the basic bulb unit load cases, the horizontal lateral supports reduce the end moments of the stay column. However, inverse behaviour appears from thermal loads. The definition of these loads and the results obtained are covered in details later, these specific studies were conducted by the internal consultant and the mechanical provider.

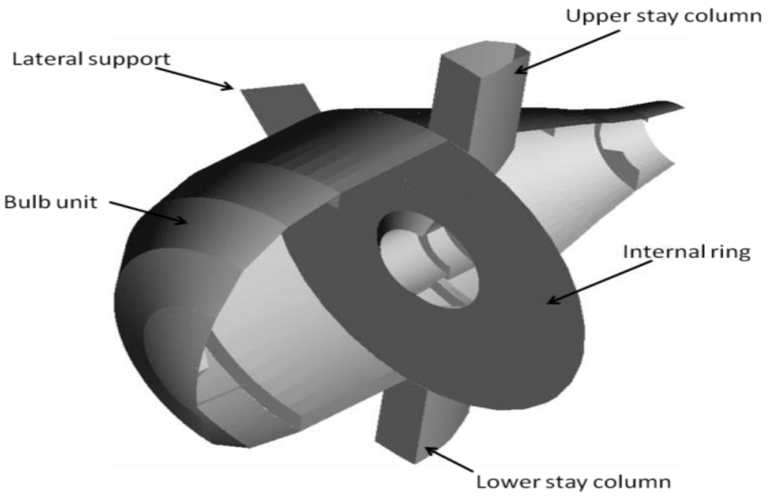


Figure 5 Stay columns and internal ring assembly

The dynamic structural behaviour includes two parts: seismic loads and dynamic interaction between the bulb unit and the surrounding structure. The peak ground acceleration at La Sarcelle is very small and the seismic component has been neglected, dynamic amplification is possible only if bulb unit frequencies are similar to the structural frequencies, this verification made by the internal consultant is covered in detail later.

Finally, 10 static load cases have been used to evaluate the critical behaviour of the powerhouse from the global model. These load cases include the 6 basic load cases combined with the thermal loads and the influence of the contact opening between the base of the powerhouse and the rock. A load case includes the usual hydrostatic loads combined with the loads transferred by the bulb unit.

- NORMAL OPERATION MODE
- NORMAL OPERATION MODE - 15°C
- SHORT CIRCUIT TORQUE OF THE GENERATOR
- RUN AWAY
- DOWNSTREAM GATE CLOSED
- DOWNSTREAM GATE CLOSED - 15°C
- WATER HAMMER
- WATER HAMMER (contact opening assumed)
- GATES CLOSED AND WATER PASSAGE EMPTY (contact opening)
- GATES CLOSED AND WATER PASSAGE EMPTY + 15°C (contact opening)

## V. PRELIMINARY COMPUTATIONS

Preliminary computations were required to check the influence of each load case and also to verify some initial assumptions before performing final load cases analysis and local modeling of the critical parts of the structure. The first initial assumption to be investigated was the magnitude of shears and end moments of the stay column computed by the mechanical provider assuming fixed ends.

Combining the Phase II concrete model and the initial bulb model from the mechanical provider has shown a 25% reduction of stay columns end monents. The forces and moments at ends of the stay columns are computed from the finite element nodal forces. The concrete flexibility is not only significant for usual bulb unit loads but also for thermal loads where the axial loads are smaller due to the vertical concrete flexibility associated mainly with the top of the stay column. From these analyses, it became clear that: The bulb unit model should include the surrounding concrete to reduce the stay columns end moments.

The load transfer at the top of the stay column, without the thermal loads, has shown large compression stresses reaching 14 MPa and also tensile stresses (5 MPa) exceeding the concrete cracking strength (Figure 6). This means that stiffeners are required to distribute bearing loads and that a no tension analysis should be used.

Initial simulation of the lining, stiffeners and the thick plate surrounding the stay columns (Figure 7) in contact with concrete, to control the upper housing compressions downstream (Figure 6), have shown the following conclusions: that the stiffeners and the plate surrounding the stay columns should have similar thicknesses than the stay columns (i.e., 80-90 mm) and that the use of the lining (19 mm) cannot be justified structurally if the stay ring shroud is assumed perfectly embedded in concrete. This means that the lining has a watertight function only and will be not be used for economical reasons.

Another important point is the efficiency of struts to sustain shears at the top of the stay column (upper housing). The struts connecting the plates to concrete are sometimes used from hand calculations to transfer direct or indirect shear coming from the end moments of the stay column. Neglecting the shear and tension between the steel components of the upper housing with concrete (Figure 7), and providing load transfer by compression only with a regular struts distribution, have shown that struts are not required. The rigidity of the struts is too small compare to the contact rigidity to be efficient. This means that shear and moments should be sustained using a combination of horizontal and vertical bearing plates only.

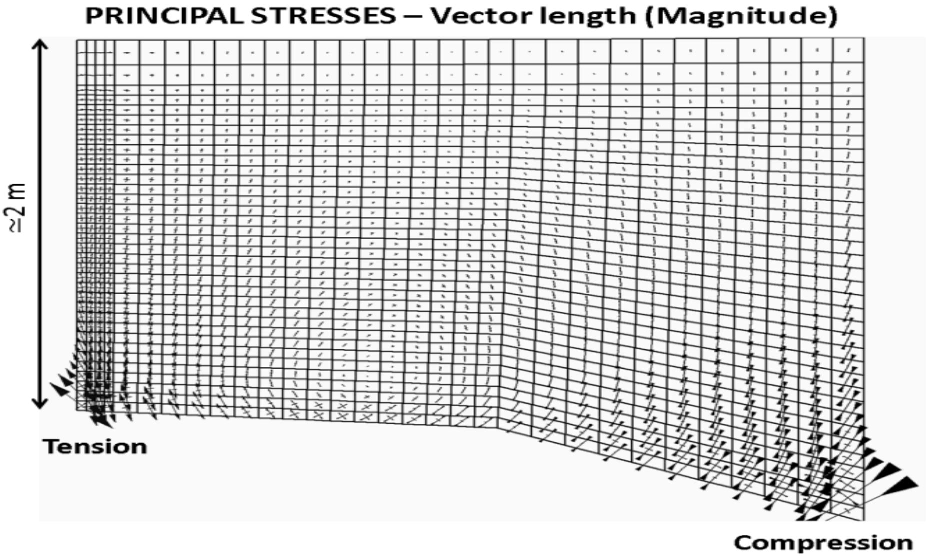


Figure 6 Principale stresses - Concrete faces - Right bank of the upper housing

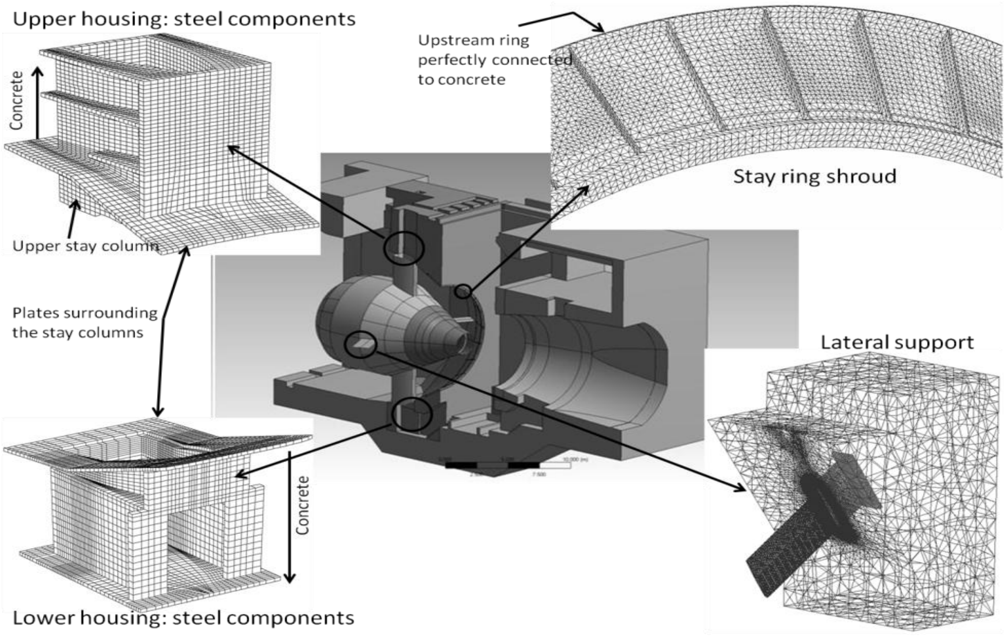


Fig 7. Local models developed

## VI. THERMAL ANALYSIS

For hydraulic bulb type power plants, embedment of the stay column ends in the concrete causes the structure to be hyper-static. This, together with possible temperature gradient between concrete elements and components of the steel bulb, as well as an attachment of the stay column to the strong internal ring of the bulb, causes thermal stresses to occur. In analysis of La Sarcelle power plant several scenarios were considered. These included, amongst others, seasonal variations of temperature as well as various temperature gradients corresponding to filling and emptying of the water passage.

The construction (i.e. placement of the concrete and installation of the equipment) was conducted at ambient temperature of approximately 17°C. The assumptions corresponding to most extreme cases are discussed below. First one involved filling the water passage at temperature of 2°C. In this case, it was considered that the equipment was in working condition directly after impoundment and without installation of heating or ventilation. It was assumed that the temperature of stay column and the housing (plates and stiffeners) was dropping quickly to 2°C when the concrete walls of the structure did not have time to cool down to the ambient temperature. This introduced a gradient of -15°C in the equipment but not in the structural concrete parts. All together, these conditions result in the highest thermal stresses and consequently to the most critical scenario, as long as the negative gradient of temperature is considered. The subsequent cooling of the concrete wall from 17°C to 2°C caused relaxation of some of the internal forces. However, at the end of this stage there are still some residual stresses, though significantly smaller than the ones discussed previously.

The second critical case involved the conditions of draining of the water passage in winter. Here, halting the equipment while cutting-off ventilation and heating prior to the draining, was considered. Specifically, it was assumed that the temperature of stay column and the bulb shell had already dropped to the magnitude of 2°C, the same temperature as the surrounding water and concrete walls. Draining the passage will result in a very quick rise of the steel elements temperature from 2°C to the ambient temperature of 17°C. While the temperature of the concrete walls remains the same. This situation results in 15°C positive temperature gradient in the steel. The case scenario proves to be most critical as long as positive temperature gradients are considered.

As before, filling the passage will slow rise the concrete temperature resulting in some internal forces relaxation.

The numerical analyses for both scenarios were conduced in two stages:

1) Evaluation of the thermal steady state
2) Coupled linear static and thermal analysis

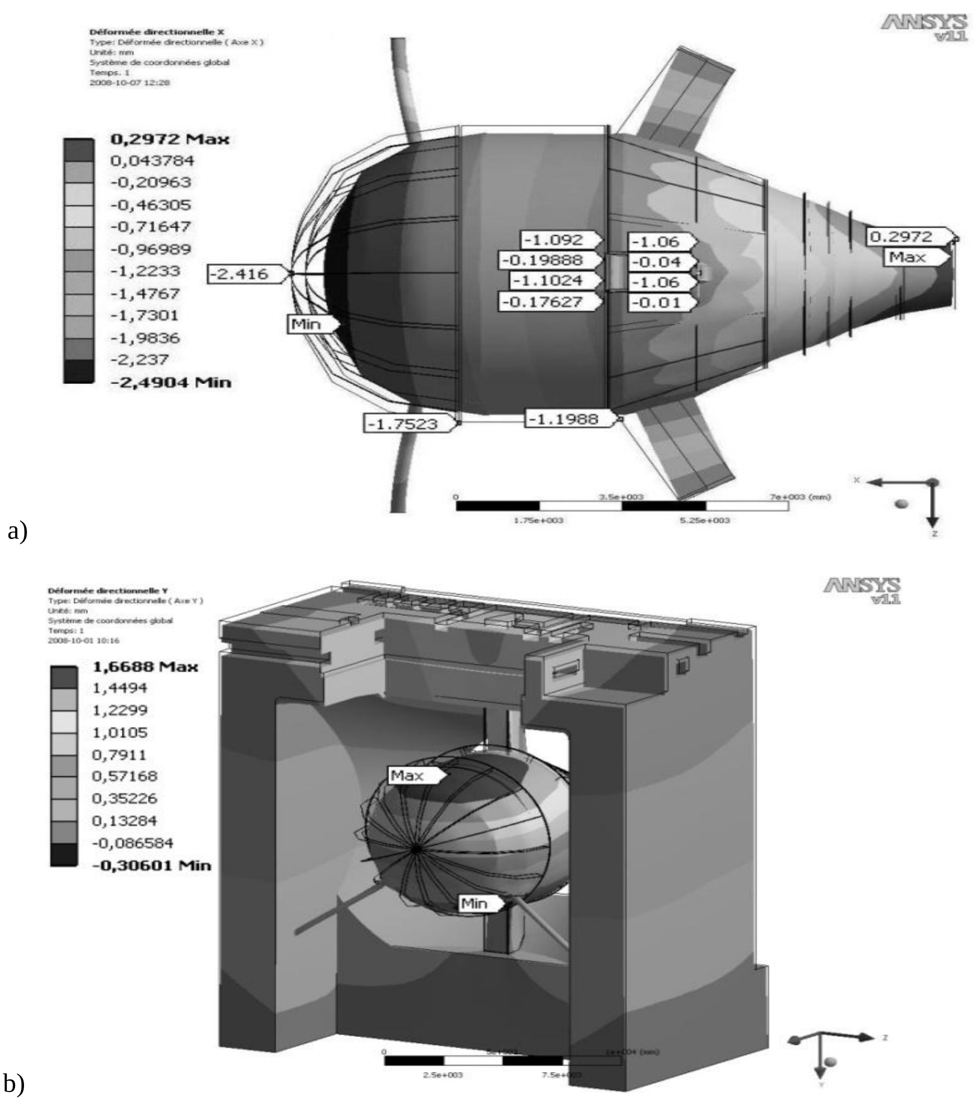
For each analyzed case, the resulting distributions of temperature, displacements and stresses were determined. Figure 8.a shows the deformation of the bulb in an upstream/downstream view. Figure 8.b shows vertical deformation of the whole mechanical unit together with the concrete structure. It should be stressed that the determined axial load reached approximately 10 000 kN in compression and tension and cannot be neglected under any circumstances.

## VII. LOCAL BEHAVIOUR

The global model cannot handle all the details of the embedded steel parts such as: the top and bottom of the stay column, the horizontal bulb support and the stay ring shroud. It is impossible for several reasons: the steel details can be initially unknown (shape, size, number, positioning), and the introduction of these details in the global model will increase the model size significantly, which is, in practice, an inefficient approach.

Figure 8 presents the four parts of the powerhouse studied using local models. We usually only show the steel part to simplify and have a better look of the resulting encased steel parts. The bulb supports (horizontal and top/bottom of the stay column) have been studied as compression structures only, where no tension is allowed and shear between steel and concrete is neglected. These three models have a local behaviour because the loads applied to these structures are mainly controlled by the bulb unit.

The stay ring shroud is different. In this case, the global behaviour can modify the response and we applied to the boundaries of the local model the displacement of the critical load case studied (WATER HAMMER). Moreover, tension is allowed at the upstream ring of the stay ring shroud (Figure 7) to be able to compute the steel rebars keeping the structure under elastic behaviour.



a)

b)

Fig 8. a) Upstream/downstream deformations (mm); b) Vertical deformations (mm) (bulb & concrete)

The top part of the stay column encased in concrete was the most complex in regard to the position, size and number of plates required to safely distribute the compression stresses. Several versions have been studied before to obtain this final version. The criteria used to create the chosen configuration are the following:

- Contact pressure below 4 MPa
- Small footprint to simplify rebars installation (upstream and downstream of the stay column)
- Potential load transfer to downstream

The contact pressure criteria of 4 MPa is adapted from hand calculation and it is usually applied over a resulting contact surface. From finite element results, it is sometimes difficult to limit the contact pressure to 4 MPa. For this reason, the maximum contact pressure can be greater than 4 MPa locally. Obviously, this criterion should be obtained over a relatively small area where local redistribution can easily happen.

The concrete cover upstream of the upper housing (Figure 3) is smaller than 1 meter and the internal consultant was concerned by the potential lost of the concrete strength in this region. For this reason, the configuration chosen is also able to adequately redistribute the loads to the structure in this case. Figure 9 presents an example of the resulting compression surface on the downstream face of the upper housing. The chosen bearing plates provide a good distribution of the compression loads to the concrete and are also able to distribute adequately these loads without the upstream concrete cover of the upper housing.

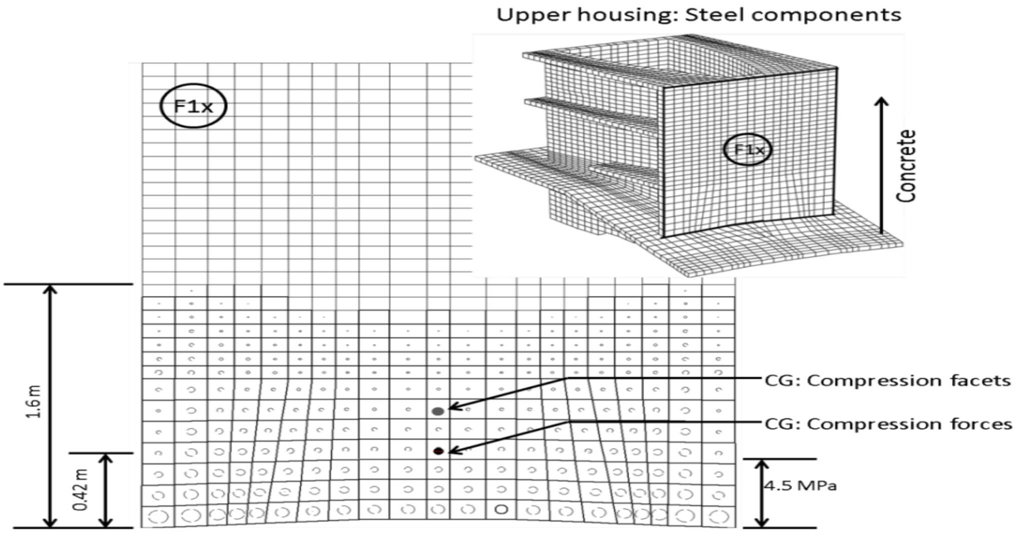


Fig 9. Upper housing - Downstream compression surface (example)

## VIII. DYNAMIC ANALYSIS

The cyclic vibration of turbines and their influence on the power plant concrete structure was analyzed by comparing the fundamental frequencies of their steel and concrete components (Table 1). For these analyses, the boundary conditions correspond to complete fixation at the base of the foundation. The Eigen vectors and Eigen values were determined using the Lanczos algorithm.

Two pass computations were conducted for condition with and without influence of water in the passage. The influence of the water was simulated indirectly including an equivalent concrete density accounting for tributary mass of the water:

$$\gamma' = (M_c + M_w)/Vol_c, \text{ in which,} \qquad (1)$$

$\gamma'$ = equivalent body density
$M_c$ = body mass concrete
$M_w$ = mass of the water tributary volume
$Vol_c$ = body volume

The first ten modes were calculated for the two models (with and without water). Figure 10 shows the first mode corresponding to the left-right movement of the water intake. The objective of the exercise is to compare the fundamental frequency of the structure to the rotational speed of the machine which is summarized in Table 1. The fundamental frequency of the structure is more than 2.26 times the machine nominal frequency. For the runaway loading case, the machine and the concrete structure frequencies are of the same order of magnitude.

**Table 1. Structural fundamental frequencies and machine rotational speeds**

| Generator | | | First structural frequency | |
|---|---|---|---|---|
| Condition | Speed | | Concrete only | Concrete+water |
| | rpm | Hz | Hz | Hz |
| Nominal | 85.7 | 1.428 | 4.039 | 3.228 |
| Runaway | 180 | 4.667 | | |

To evaluate the effect of vibration, the fluctuating forces generated by the rotating components to upper and lower housings were applied as loads to the model with water (figure 11). The loading frequencies are function of the natural frequencies of the bulb unit in each direction. The mean amplitude of the loading corresponds to the static load and the amplitude variation are chosen as maximum reasonable values.

The response was evaluated for selected sampling points located at intake head (I1,I2), turbine floor (P1,P2,P3) and downstream slab (D1,D2) (Figure 12). The transient responses are shown in Figure 13 for the 7 points selected and the 3 directions (X, Y, and Z). The mean response of each curve is related to the equivalent static load. A steady state response appears after 1 to 3 seconds and the maximum peak to peak in this region for both states was compared for the two studied frequencies to the classical Rathbone chart adapted from Dimarogonas &. Haddad - 1992 (Figure 14). The steady state response presents an extremely smooth to smooth behaviour for both frequencies (nominal, runaway). The short transient response varies between smooth to good behaviour.

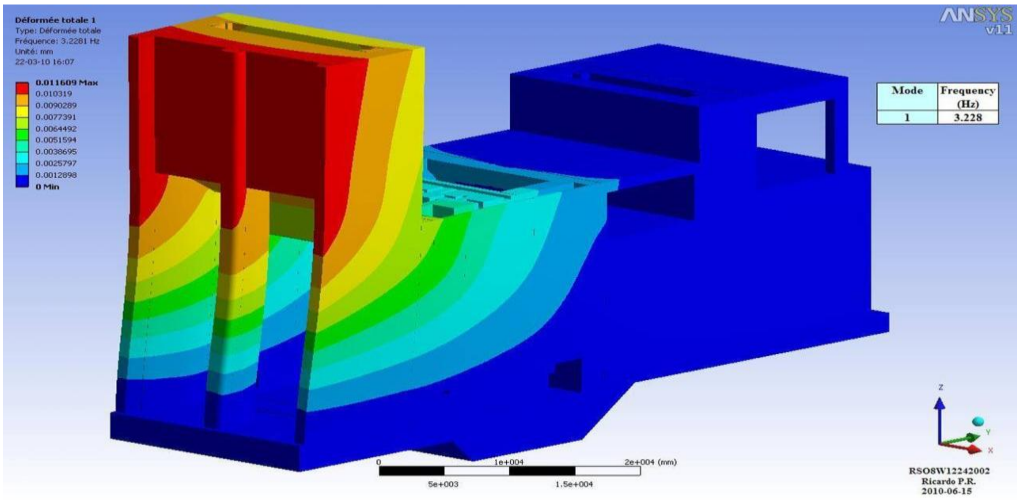


Fig 10. First mode shape - Left/right bank

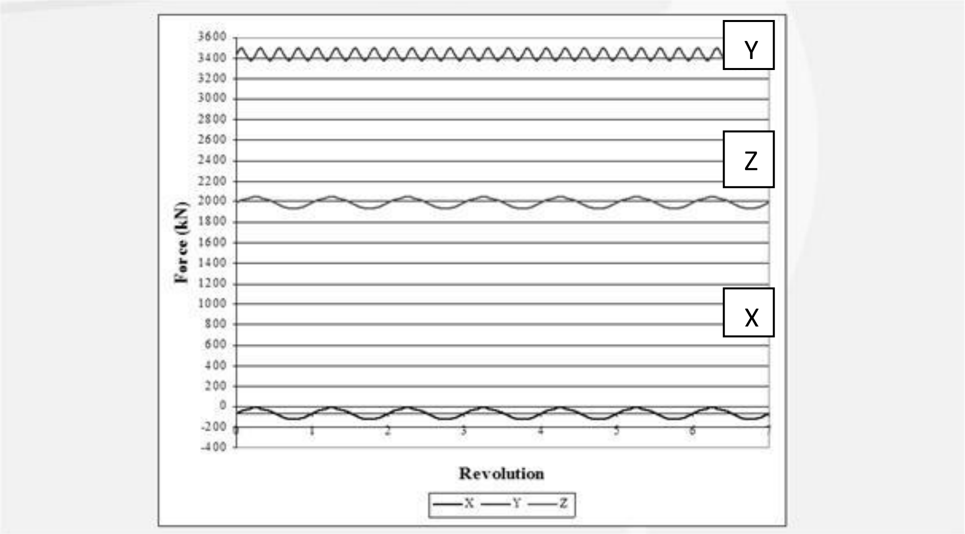


Fig 11. Fluctuating forces at the upper housing

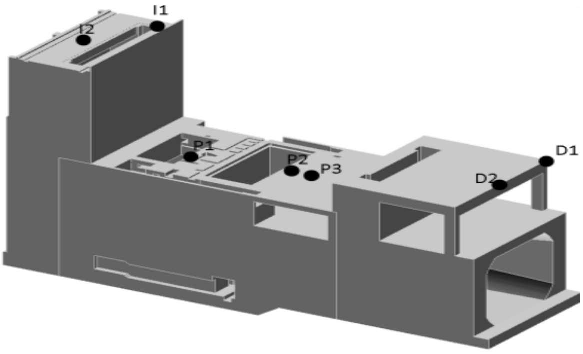


Fig 12. Selected points on the structure

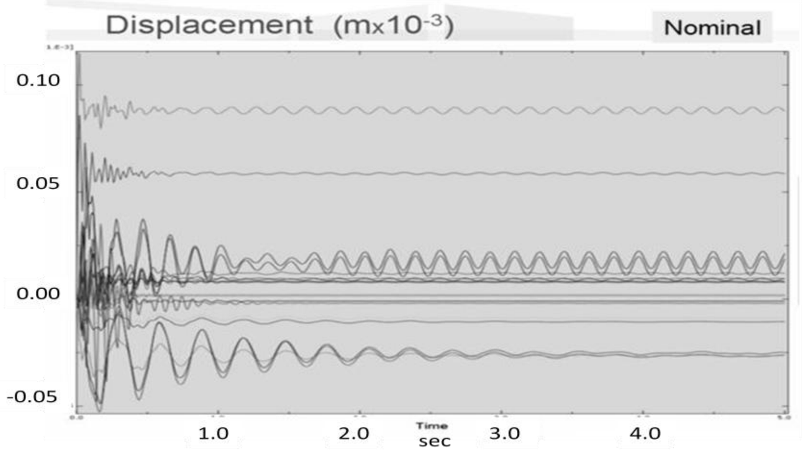


Fig 13. Displacement response for the normal loading case

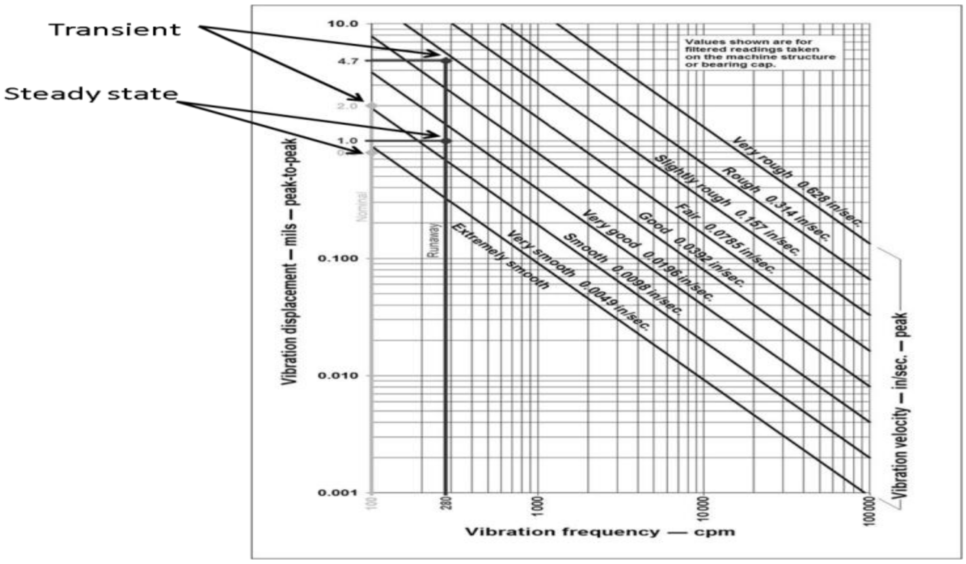


Fig 14. Displacement response comparison - adapted from Dimarogonas &. Haddad, 1992

## IX. POST-PROCESSING TOOLS

The post-processing tool for solid finite element analysis is the graphical postprocessor VCSFER (Visualization and Computation of Solid Finite Element Results) from the lead structural consultant including numerical add-on software for forces, moments and equilibrium computation. This software is strictly limited to visualization of stresses adapted for Civil Engineering Design purposes. Deformed shape and color rendering of displacements and stresses are already available in commercial software and are not included. Design purposes in Civil Engineering required appropriate stresses distribution in arbitrary planes according to geometry, field stresses and rebars positioning.

A general forces and moments procedure required the following steps:

Step A: Creation of a mesh contour and surface selection
Step B: Virtual mesh creation
Step C: Mesh field data interpolation
Step D: Surface boundaries
Step E: Tributary surfaces and local system of axis

The forces and moments procedure is described for the right bank wall at the upstream base of the powerhouse (Figure 15). Each step is now explicitly described:

Step A: a mesh contour is created using a cutting plane and the external facets of the Finite Element Model, thus generating a point where a line of the facet is cut by the plane. Several points are created and imported; from these points, we select a surface and simplify the selected region by reducing the number of points to a minimum, and if required, by creating additional points.

Step B: creation of a 2 dimensional mesh. The created mesh is a virtual mesh and it is independent of the original tridimensional mesh. The Cartesian stresses of the mesh nodes have to be interpolated from the original 3 dimensional field stresses.

Step C: Exact interpolation of the 3 dimensional stress field data to the 2 dimensional virtual mesh is conducted. In the present case, the interpolation is exact because it is based on the element shape functions. To interpolate exactly, we need to find the finite element including the selected point in space and the natural coordinates of the element shape functions.

Step D: The forces and moments over a given surface vary according to geometry (shape, thickness) and field stresses. In concrete, there is no need to design for the peak local stress and the equivalent stress computed over a given width, for example 1 m, will provide more realistic values. This is done by dividing the studied surface in several sub-surfaces by selecting some nodes of the 2 dimensional virtual mesh. Three kinds of points should be provided: 1) line boundaries, 2) breaking line boundaries and 3) additional sections. In the present case, 22 nodes were rapidly selected from the screen. According to the nodes selected on the line boundaries, the nodes linking these nodes to create tributary surfaces are automatically selected. It is now possible to compute forces and moments on the resulting sub-surfaces according to the selected load cases. The envelope of the critical combination (axial force and moment, shear) is provided for both sides of a sub-surface and results per unit width are also given for design purposes.

Step E: The validation of step D is done by plotting the resulting sub-surfaces computed including the corresponding system of axis and the sub-surface numbering. This picture and the corresponding numerical results are the basic input to the designer.

Sometimes, the structures are less complex and predefined planes are available for design purposes using existing finite element faces. This approach is faster and corresponds to steps D and E of Figure 15. The forces and moments software can combine elastic load cases with a dynamic load case from a spectral analysis. For the spectral load case, the software combines periodic modes using the CQC modal combination rule in each direction with an SRSS spatial combination. It also includes the influence of rigid modes and missing masses. Finally, it is also possible to compute the resultants from an arbitrary surface of a given plane to verify global equilibrium.

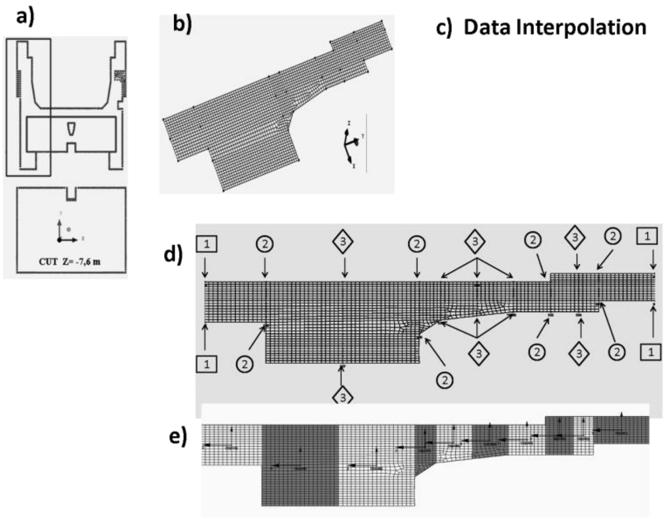


Fig 15 General forces and moments procedure: a) Cutting plane, b) Virtual mesh, c) Mesh field data interpolation, d) Surface boundaries, e) Tributary surfaces and local system of axis

The required stresses to compute forces and moments for design purposes are the normal and shear stresses in a given plane. The principal stresses are also significant to define the regions with high tensile stresses and to select the cutting planes providing critical forces and moments. Figure 16 shows a simplified example of the procedure adopted when the geometry is more complex. From an elevation view of the powerhouse (concrete Phase I), we want to look the tensile stresses at the top of the conduit 1 meter inside the concrete. The initial mesh in this region combined tetrahedral elements (Phase I) and hexahedral elements (Phase II). The resultant virtual mesh (cut A) and the corresponding tensile stresses (bold) are shown, the maximum values are in the upstream beam but they are also present around the upper housing. A second cut (cut B) is done in the middle of the upstream beam (initially tetrahedral elements) and the corresponding axial stresses are plotted using a circle to define their relative magnitude. The resultants (3) and moments (3) are computed about the center of gravity of the surface to help the design process. The advantage of the virtual mesh approach is clearly demonstrated with complex structures like the bulb powerhouse mixing different element types (tetrahedral, hexahedral).

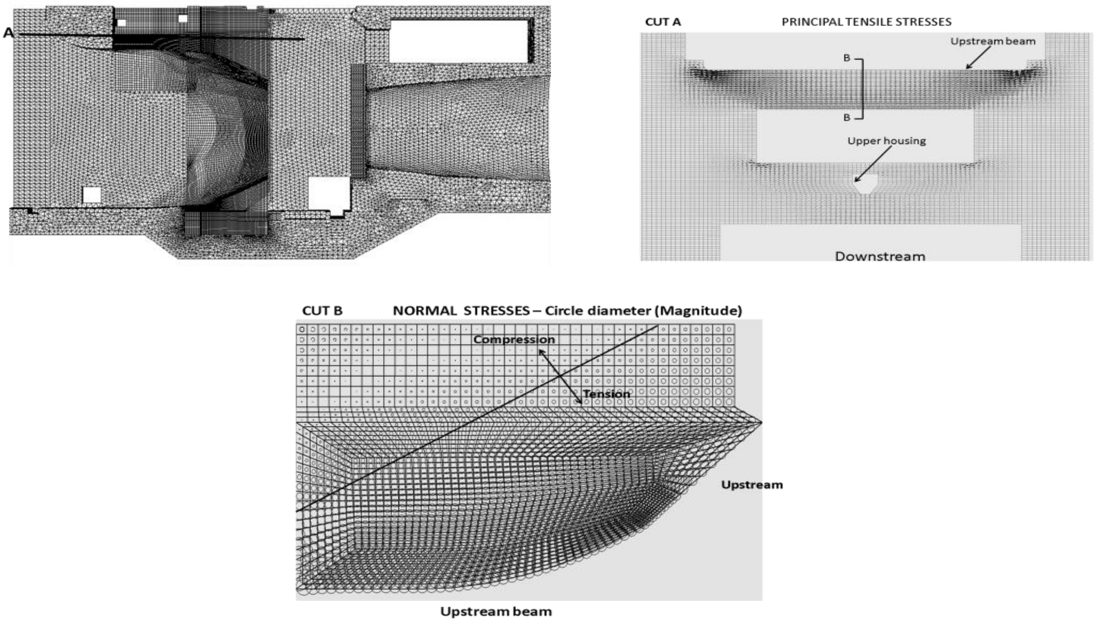


Fig 16. Example of principal and normal stresses from a virtual mesh

## X. CONCLUSIONS

The numerical analysis of the La Sarcelle power plant was an extremely powerful tool to understand the relative behaviour of each component (concrete phases and steel) and simulate the local behaviour of the embedded steel parts based on the global model behaviour.

Some important lessons from the numerical analysis can be summarized as follow:

- The bulb unit model should include the surrounding concrete to reduce the stay columns end moments;
- The embedded steel components of the stay column should have a similar thickness as the stay column including the surrounding plate at the wet concrete surface;
- The lining between the stay column and the stay ring shroud is not structural (water tightness function only, if the stay ring shroud is perfectly embedded in concrete);
- The struts are not efficient to transfer shears to concrete as regard to their small rigidity;
- The bulb thermal loads are significant in the present case and should be investigated in all cases;
- The loads at the top of the stay column (upper housing) are well transferred using appropriate horizontal and vertical bearing plates;

Additional comments can be summarized as follow:

- The harmonic loading have shown small displacements and do not present a human discomfort.
- The internal post processing tool is a valuable tool to process dedicated civil engineering stresses and compute local behaviour as forces and moments for design purposes or global equilibrium.
- The ability to present the stresses on arbitrary planes of complex structures is a significant improvement to help the design process.

## ACKNOWLEDGEMENT

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