Long-term dynamic monitoring of arch dams. The case of Cabril dam, Portugal

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SUMMARY:
In the scope of LNEC research program on dam safety control, late 2008, a long-term dynamic monitoring system was installed in the highest Portuguese arch dam - Cabril dam (60 years old; 132 m high; horizontal cracking near the crest). The acceleration records that have been continuously measured (on 16 points over the dam body and on 2 points near the insertion) are automatically processed using modal identification techniques, in order to obtain experimental information about the evolution of natural frequencies, modal damping, and mode shapes (stationary or non-stationary modes).
These dynamic monitoring systems are expected to cover the current lack of experimental reliable data about the dynamic response of such structures, which is a fundamental step for addressing the remaining questions about the dynamic modelling of dam-reservoir-foundation systems, particularly as refers to the hypotheses related with the simulation of water-structure interaction and damping effects.

Keywords: Long-term dam dynamic monitoring, modal identification, deterioration, seismic monitoring

1. INTRODUCTION

The safety control of large dams under static and dynamic loads, involving observation data and numerical modelling, is now one of the challenges being faced by structural engineering (Oliveira 2002, 2011). The complexity of the dam-reservoir-foundation geometry, the presence of different types of discontinuities, the water-structure interaction (Oliveira et al. 2006, Lemos et al. 2008), the influence of thermal and water level variations, the development of deterioration processes over time and the occurrence of exceptional events such as major floods or earthquakes (Chen et al. 2003, Wieland 2008, 2009, Chen 2007,2009), makes the structural safety control of large dams an activity that requires continuous updating, both in terms of the equipment for measurement, transmission, and storage of the observation data, and in terms of computer applications to support the automation process of collecting, processing, analysing and managing all information required for the safety control.

As regards the dynamic behaviour of concrete dams, mention should be made of the fact that the latter has been observed using forced vibration tests, ambient vibration tests (Svern 2007) - of which the results are analysed using modal identification methods extensively described in literature (Peeters 2000, De Roeck et al. 2000, Juang et al. 2001, Rodrigues 2004, Magalhães 2010) - and long-term vibration-measuring tests (Ghanaat et al. 2000, Oliveira et al. 2011), which are also carried out on other civil engineering structures as bridges (Cunha and Caetano 2006). The main objective of these tests is the health monitoring and the seismic monitoring (Fig. 1), but they have also been used for obtaining experimental information to calibrate and update existing numerical models.

Nowadays, in dam monitoring, the use of Automatic Data Acquisition Systems (ADAS) tends to be extended from static to dynamic measurements using the concept of the online systems presented for continuous long-term dynamic monitoring (Darbre and Proulx 2002, Oliveira et al. 2011). With these systems, the collected data (acceleration records at high sampling rates ~50Hz) should be automatically analysed by modal identification techniques (Peeters 2000, Juang et al. 2001, Magalhães 2010), considering time periods of about one hour. Then, the evolution of the identified modal parameters (natural frequencies, mode shapes and modal damping) can be compared with numerical results from finite element models (Oliveira et al. 2006) and/or discrete element models (Lemos et al. 2008) (Fig. 2).

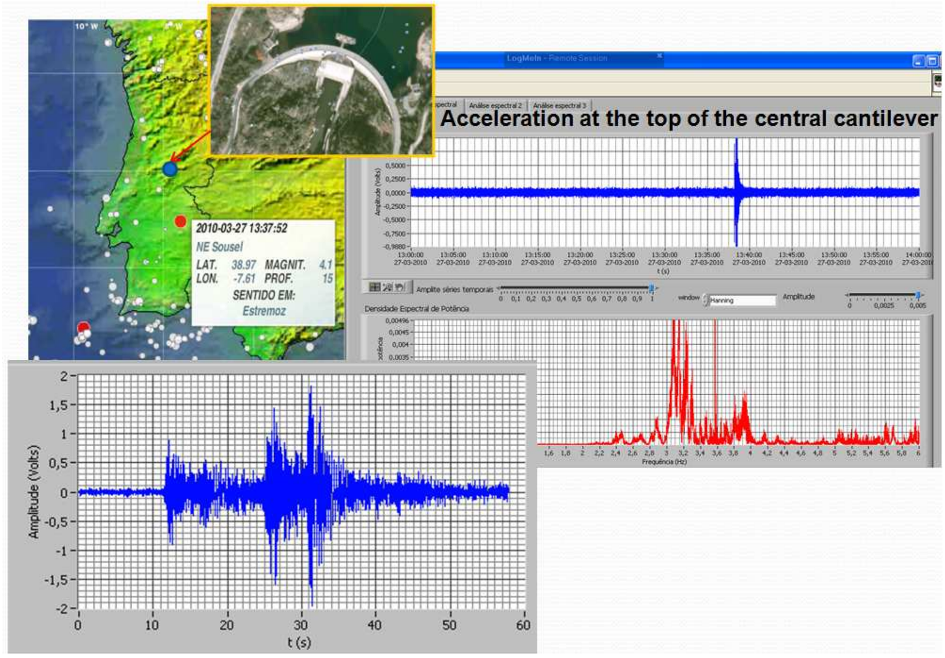


Figure 1. Measurement of Sousel earthquake (27Mar2010) at Cabril dam.

The use of ADAS, for the long-term dynamic monitoring of large dams, is the best way to get useful experimental information to: i) study the evolution in the main modal parameters; ii) gather information about the correlation between changes in the modal parameters and structural changes due to deterioration processes; iii) measure the dynamic dam response under ambient excitation and under seismic loads; iv) study the influence of the reservoir on the structural dynamic behaviour of the dam-foundation-reservoir system.

2. DYNAMIC BEHAVIOUR OF DAM-RESERVOIR-FOUNDATION SYSTEMS. MODELLING AND MONITORING

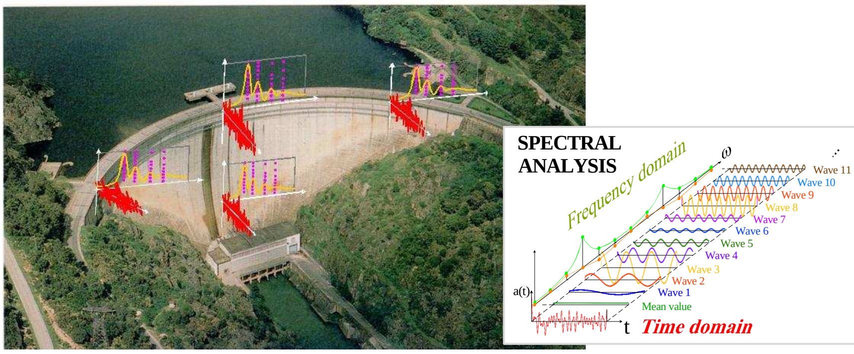
The interest of ADAS for the continuous monitoring of dam dynamic behaviour not only depends on the reliability of hardware components (which is fundamental to assure the quality of collected data) but also depends greatly on the potentialities of the software used for the automatic data analysis.

Hence, it is fundamental to develop software for data analysis automation, which allows generating and/or updating, every hour, synthetic information under graphical form to be used by engineers and other technicians of the staff responsible for the dam safety control. With that graphical information, it should be easy to analyse the evolution in the main parameters and variables that characterise the dam dynamic response: modal parameters, water level, maximum accelerations, acceleration spectra, etc.

This software is to include computational modules that make it possible to detect special events (earthquakes, civil works nearby the dam site, hydraulic discharges, etc.) and to detect abnormal structural changes by comparison between the dynamic monitored response (analysed by modal identification) and the dam response predicted by numerical modelling (FE/DE) and statistical models for effect separation.

DYNAMIC BEHAVIOR OF DAM-RESERVOIR-FOUNDATION SYSTEMS

Continuous monitoring and automatic modal identification



FINITE ELEMENT MODELS

a) Models with Westergaard added water masses and proportional damping

b) Water-structure interaction models with non-proportional damping

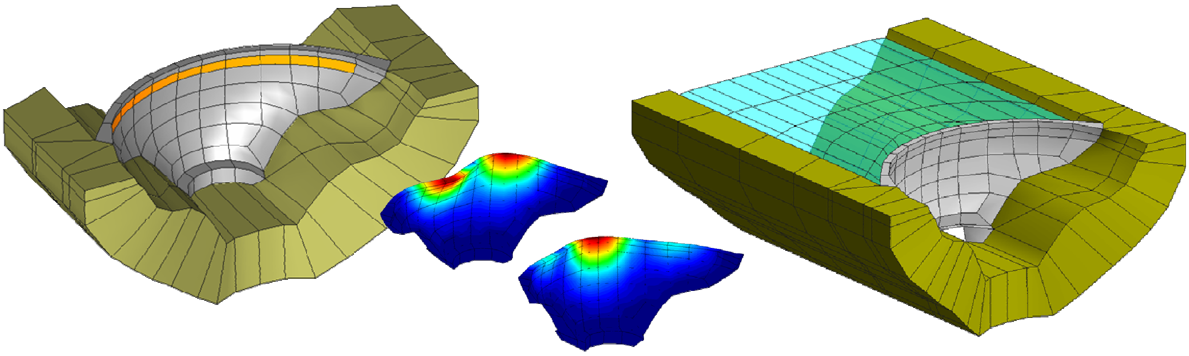


Figure 2. Integrated use of dynamic monitoring data, modal identification methodologies and numerical results from FE models with different hypotheses for simulating the hydrodynamic pressure and damping.

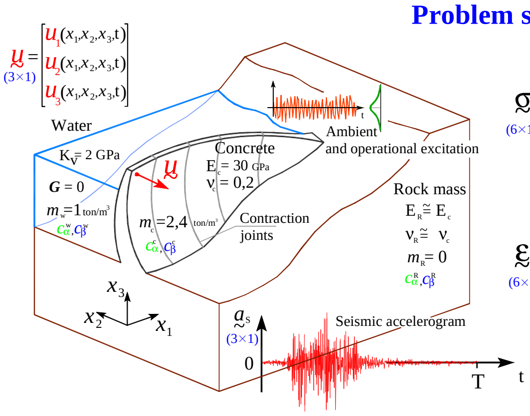
In the development of the above mentioned software, the models/formulations/algorithms/ interfaces to be used should be specifically adapted to the simulation and analysis of the dynamic behaviour of dam-foundation-reservoir systems (Fig. 2). In particular, the models to be used should allow the analysis of the observed dynamic behaviour by taking into account the water level variations, the temperature variations and the time effects that may be related to deterioration processes (i.e. progressive cracking due to swelling reactions or induced by exceptional actions like floods or earthquakes).

For choosing the adequate models/hypotheses (Pedro et al. 1996) to be used in the software development for supporting ADAS for dam dynamic monitoring, one important feature that should be taken into account is related to the fact that these structures may present non-stationary vibration modes, which, as well known, cannot be simulated by simple elastic models based on the hypothesis of classical damping of Rayleigh type (proportional to the global mass and/or stiffness). This hypothesis of classical damping is generally used as a good approximation in civil engineering structures with no dynamic water-structure interaction phenomena.

In order to take into account the existence of non-stationary vibration modes, the models to be used should enable the consideration of the generalized damping hypothesis (non-classical damping, i.e., non-proportional to the mass and stiffness). From among these models, the simplest ones can be formulated using state formulations as indicated in Fig. 3.

Dynamic Behavior of Dam-Reservoir-Foundation Systems

Problem statement. Linear elastic approach



$$\underset{(6\times1)}{\sigma}=\begin{bmatrix}\sigma_{11}\\\sigma_{22}\\\sigma_{33}\\\sigma_{23}\\\sigma_{31}\\\sigma_{12}\end{bmatrix}\qquad \underset{(6\times1)}{\varepsilon}=\begin{bmatrix}\varepsilon_{11}\\\varepsilon_{22}\\\varepsilon_{33}\\2\varepsilon_{23}\\2\varepsilon_{31}\\2\varepsilon_{12}\end{bmatrix}$$

Elasticity matrix (isotropic material)

$$\underset{(6\times6)}{\boldsymbol{D}}=\begin{bmatrix} K_v+\frac{4}{3}G & K_v-\frac{2}{3}G & K_v-\frac{2}{3}G & & & \\ & K_v+\frac{4}{3}G & K_v-\frac{2}{3}G & & \underline{0} & \\ \text{sim.} & & K_v+\frac{4}{3}G & & & \\ & & & G & 0 & \\ & \underline{0} & & & G & \\ & & & 0 & & G \end{bmatrix}$$

$$K_v=\frac{E}{3(1-2\nu)}\qquad G=\frac{E}{2(1+\nu)}$$

[Chen; Saleeb 1994]

Differential operator

$$\underset{(6\times3)}{\boldsymbol{L}}=\begin{bmatrix}\frac{\partial}{\partial x_1} & 0 & 0\\ 0 & \frac{\partial}{\partial x_2} & 0\\ 0 & 0 & \frac{\partial}{\partial x_3}\\ 0 & \frac{\partial}{\partial x_3} & \frac{\partial}{\partial x_2}\\ \frac{\partial}{\partial x_3} & 0 & \frac{\partial}{\partial x_1}\\ \frac{\partial}{\partial x_2} & \frac{\partial}{\partial x_1} & 0\end{bmatrix}$$

State Space Formulation. Finite Element Modelling

Compatibility

$$\varepsilon = \boldsymbol{L}\,u$$

Constitutive equation

$$\sigma = \boldsymbol{D}\varepsilon + c_\beta \boldsymbol{D}\dot{\varepsilon}\quad\text{(Kelvin body)}$$

Equilibrium

$$\boldsymbol{L}^T\sigma + f = 0$$

Equilibrium of forces at a point

Differential form or strong form ($c_\beta = 0$)

$$\left\{\begin{array}{l}\boldsymbol{L}^T(\boldsymbol{D}\boldsymbol{L}u) + f = 0\ ,\ \text{in } V\\ \text{Initial and boundary conditions}\end{array}\right.$$

$$f = mg - m(\ddot{u} + a_s) - c_\alpha \dot{u}$$

FLVC

Green-Gauss Theorem

Equilibrium of energy to be verified in the volume V **(VWP)**

Integral form or weak form ($c_\beta = 0$)

$$\left\{\begin{array}{l}\int_V (\boldsymbol{L}u)^T \boldsymbol{D}\,(\boldsymbol{L}\varphi)\, dV = \int_V f\,\varphi\, dV\ ,\ \forall\varphi\in\mathscr{D}\\ \text{Initial and boundary conditions}\end{array}\right.$$

Form that is used in FEM

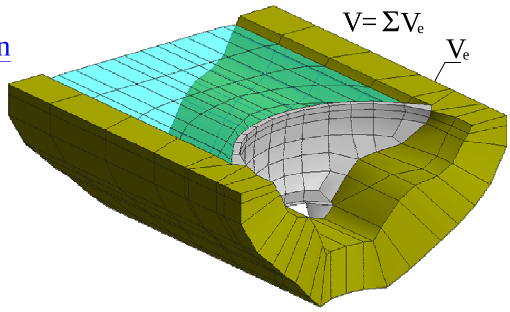
Numerical solution (FEM)

1st Step. Spatial integration

$$m = \sum m^e \qquad c = \sum c^e$$

$$k = \sum k^e \qquad f_G = \sum f^e$$

If $c \neq \alpha\, m + \beta\, k$ the system is said to be non-classically damped and the vibration modes are non-stationary (with complex components) [Veletsos; Ventura 1986]



The displacement field and the test functions φ (or virtual displac.) are approached by interpolation: $u = \mathbf{N}\, u^e$ and $\varphi = \mathbf{N}\,\varphi^e$. For each FE we obtain:

$$m^e = \int_{V_e} m\mathbf{N}^T\mathbf{N}\, dV \qquad c^e = c_\alpha\, m^e/m + c_\beta\, k^e$$

$$k^e = \int_{V_e} \boldsymbol{B}^T\boldsymbol{D}\,\boldsymbol{B}\, dV \qquad f^e = -m^e\, a^e_S$$

(seismic load)

$$\boldsymbol{B} = \boldsymbol{L}\,\mathbf{N}\qquad \text{[Zienkiewicz 1967]}$$

Classical representation (in displacements)

$$\left\{\begin{array}{l} m\,\ddot{u}(t) + c\,\dot{u}(t) + k\,u(t) = \underset{(N_{DF}\times1)}{f_G(t)}\\ \text{Initial conditions}\end{array}\right.$$

System of $\mathbf{N}_{DF}$ differential 2nd order eq.

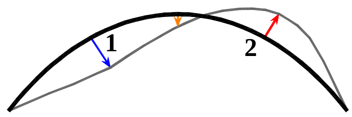
Representation in the **state space** (in displacements u and velocities v)

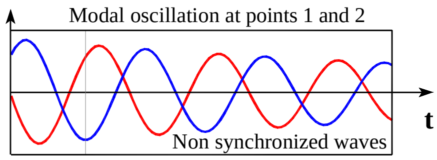
$$\left\{\begin{array}{l}\begin{bmatrix}\dot{u}(t)\\\dot{v}(t)\end{bmatrix} = \begin{bmatrix}0 & I\\ -m^{-1}k & -m^{-1}c\end{bmatrix}\begin{bmatrix}u(t)\\v(t)\end{bmatrix} + \begin{bmatrix}0\\m^{-1}\end{bmatrix} f_G(t)\\ \text{Initial conditions}\end{array}\right. \Leftrightarrow \left\{\begin{array}{l}\dot{x}(t) = \underset{(mck)}{A}\,x(t) + \underset{(ms)}{B}\,f_G(t)\\ \text{Initial conditions}\end{array}\right.$$

x state vector $(2N_{DF}\times1)$; state matrix $(2N_{GL}\times 2N_{GL})$

System of $\mathbf{2N_{DF}}$ differential equations of 1st order

From the eigen values (complex) $\lambda_n = -\xi_n\omega_n + i\omega_n\sqrt{1-\xi_n^2}$ of the state matrix $\underset{(mck)}{A}$ we can obtain the natural frequencies $\omega_n = |\lambda_n|$ and the modal damping $\xi_n = -\mathrm{Re}(\lambda_n)/\omega_n$. The vibration modes are given by the corresponding eigen vectors with complex components, that can describe **non-stationary vibration modes**:





2nd Step. Time integration

The time integration can be performed using modal or structural coordinates. Generally the time is discretized in equal time steps $t_k = k\,\Delta t$ and recursive solutions are derived. For a linear force variation in each time step, and using modal coordinates x^* (being $x = \Phi_E\, x^*$, where Φ_E is the eigenvectors matrix of A, and λ_E the corresponding eigenvalues diagonal matrix, $\lambda_E = \Phi_E^{-1}\,\underset{(mck)}{A}\,\Phi_E$) we obtain $2N_{DF}$ independent equations $\dot{x}^* = \lambda_E\, x^* + f^*$, whose solutions are

$$x^*(t_{i+1}) = e^{\lambda_E\Delta t}\, x^*(t_i) + \frac{(\Delta t-1)(e^{\lambda_E\Delta t}-1) - \lambda_E}{\lambda_E/\Delta t}\, f^*(t_i) + \frac{e^{\lambda_E\Delta t} + (\Delta t-1)}{\lambda_E/\Delta t}\, f^*(t_{i+1})\ ,\qquad f^* = \Phi_E^{-1}\,\underset{(ms)}{B}\, f_G(t)$$

Figure 3. Modelling and measuring the dynamic behaviour of concrete dams. State space formulations and non-proportional damping in the development of FE models for dynamic analysis. Concept of non-stationary vibration modes (usually measured on arch dams).

Fig. 3 emphasises the interest of the state space formulations for structural dynamics. Using the state matrix, the natural frequencies, modal damping and modal configurations can be easily computed through their eigenvalues and eigenvectors.

For generalized damping, the eigenvalues λ_n of the state matrix are complex numbers, as well as the j components of the corresponding eigenvectors ϕ_{jn}. Physically, these complex values correspond to the existence of the mentioned non-stationary modes that are graphically represented in Fig. 3. So, for the generalized damping hypothesis we can say that for a given mode n, the vibration at each DOF j is described by an harmonic decreasing wave (Fig.3), which is completely defined by four parameters that can be "extracted" from the complex value λ_n and from the complex modal component ϕ_{jn}: i) natural frequency $\omega_n = |\lambda_n|$; ii) modal damping, $\xi_n = -\mathrm{Re}(\lambda_n)/\omega_n$; iii) amplitude $|\phi_{jn}|$; and iv) phase angle $\mathrm{atan}\left(\mathrm{Im}(\phi_{jn})/\mathrm{Re}(\phi_{jn})\right)$.

3. CABRIL DAM

Cabril dam (Zêzere river) is a sixty-year old double curvature arch dam (see Fig. 4), which is the highest Portuguese dam (132 m). In this dam, significant horizontal cracking occurred near the crest since the first filling of the reservoir, and a concrete swelling process has been recently detected; the water level presents important variations along the year. As regards the monitoring system, although quite complete, it should be mentioned that only the dynamic monitoring component has been automated, since 2008.

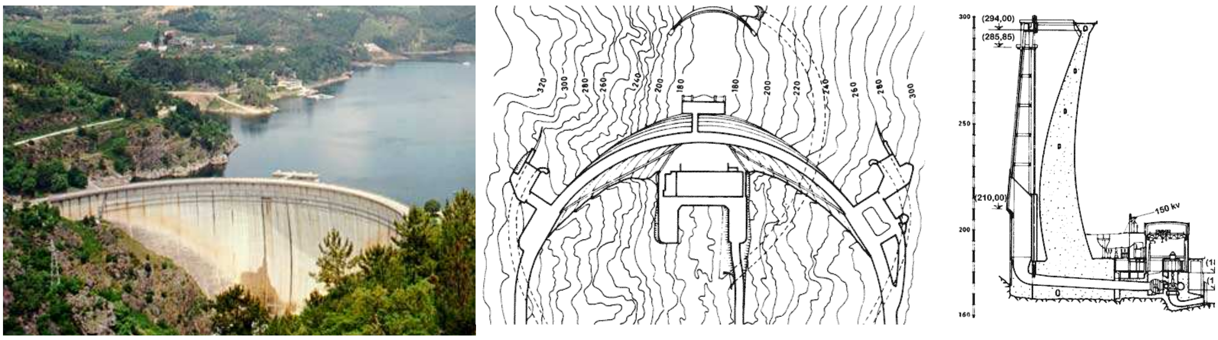


Figure 4. Cabril dam. Plan and central cantilever (with intake tower).

4. CONTINUOUS DYNAMIC MONITORING SYSTEM IN CABRIL DAM

The continuous dynamic monitoring system of Cabril dam was designed (Oliveira et al. 2011) for measuring accelerations (sampling rate of 1000 Hz) at the upper zone of the dam and insertion, with 16 uniaxial accelerometers and three triaxial (equipment installed thanks to previous funding from FCT, REEQ/815/ECM/2005) as can be seen in Fig. 5. The main configuration parameters are defined so as to have a system with a high dynamic range, capable of measuring continuously and accurately the dam response to the several actions: ambient excitation, operational excitation and seismic actions of different magnitudes.

The accelerometers are linked to a modular system composed by units for acquisition and digitalization, which are controlled by 4 data concentrators (Fig.5) that receive the data that is sent trough an optical fibre local network (intranet) (Fig.5), to a computer located at the dam power station. The collected data is stored and processed continuously using automatic modal identification procedures, for storing the main natural frequencies, mode shapes (amplitude and phase at each measuring point) and modal damping.

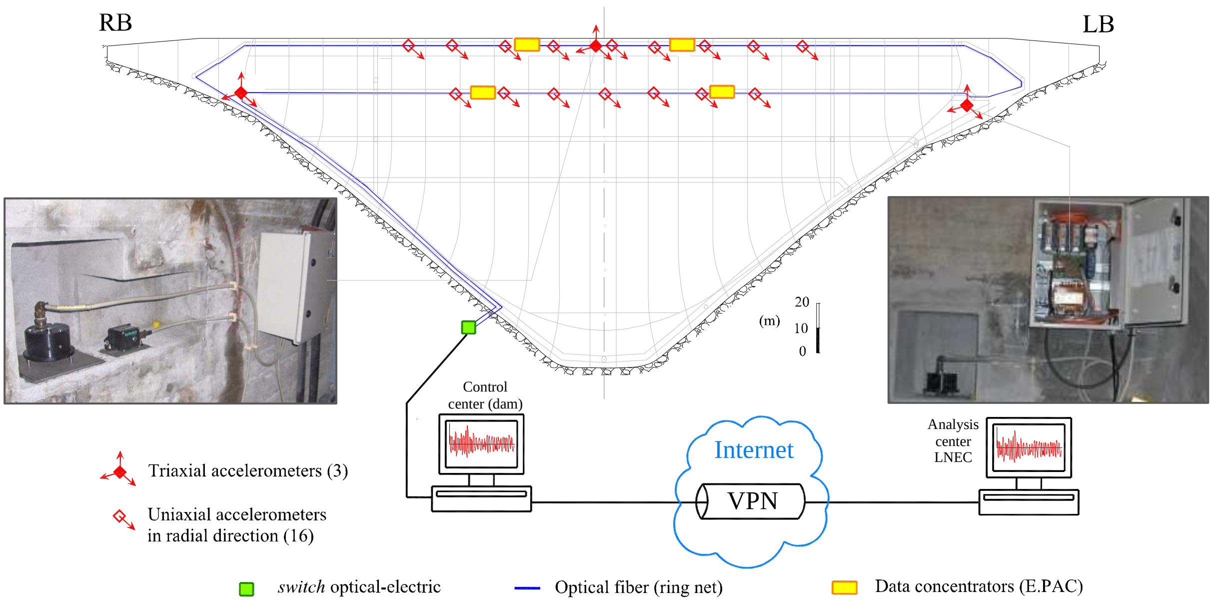


Figure 5. Main components of the continuous dynamic monitoring system (Oliveira et al. 2011).

5. DYNAMIC MONITORING OF CABRIL DAM. DATA ANALYSIS

Despite the Cabril dam dynamic monitoring system being in use since late 2008, the software *MoniDam* for automatic management and data analysis (installed on the PC at the dam's central office) is still under development. At the moment, *MoniDam* (developed in FORTRAN, with a VB.net module for scheduled email sending) is being tested on Cabril dam in order to achieve a more efficient processing of the huge amount of data gathered by the system (using the installed 25 channels of acceleration, the system is recording 25000 real numbers of single precision per second!).

MoniDam is intended to analyse automatically the recorded acceleration histories (stored in files of 1 hour length) in order to generate .DXF drawing files with the main spectra and the vibration modes corresponding to the spectral peaks identified during each hour. At the present trial stage of *MoniDam*, a significant variability in the spectra computed at each hour has been detected. This can be due to the high variations in the excitation amplitude: when the power groups are in operation the amplitude of dam vibrations can be 10 or 20 times greater than when the power groups are off (ambient excitation).

As shown below, we also observed that a simple FE model, using the Westergaard added water mass hypothesis and the classical damping (see Fig. 2a), might not be the most adequate means to understand the real monitored dynamic dam behaviour. Using this simple model, the two first vibration stationary modes typically computed for empty reservoir, of frequencies 2.6 and 2.8 Hz (symmetric and anti-symmetric), are the same as those obtained for non-empty reservoir, with frequencies decreasing up to 2.1 and 2.2 Hz for full reservoir. According to the experimental results presented afterwards, for non-empty reservoir situations, more than two spectral peaks arise in the frequency range of the first vibration modes, which points to the existence of coupled modes due to the water-structure interaction (possibly non-stationary, as suggested by the mode shape analysis). These results could be explained by the use of new FE models taking into account the water-structure interaction and the generalised damping hypothesis (see Fig. 2b).

Fig. 6 shows the spectra corresponding to the measured accelerograms on 3Nov2011, between 10.00 p.m. and 11.00 p.m., with the water level at 264.3 m (power groups on). These spectra were computed for each hour (over acceleration records of 3600 s, originally with a sample frequency

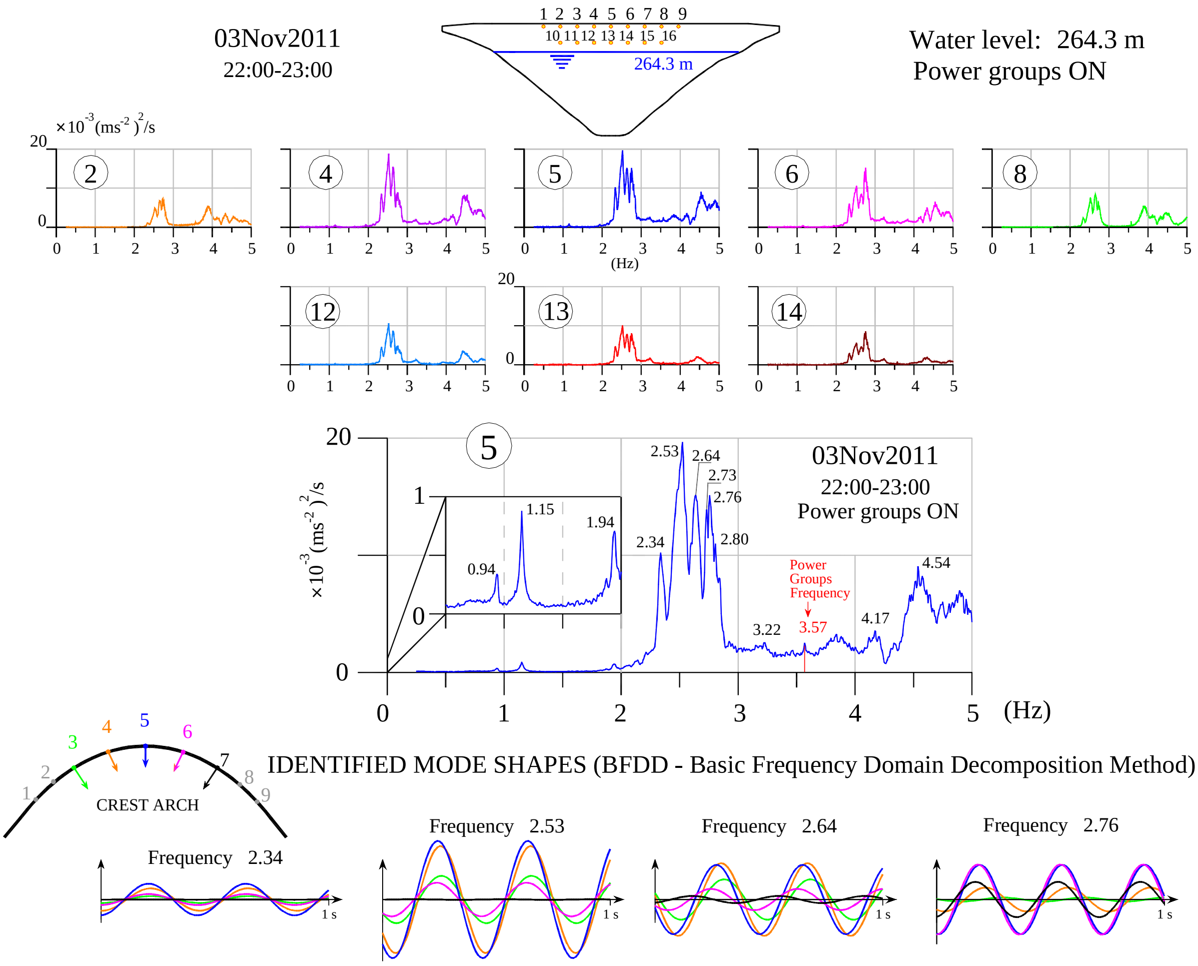


Figure 6. Spectral analysis results of the acceleration records from Cabril dam dynamic monitoring system: day 03Nov2011; water level 264.3 m; power groups on. Identified mode shapes from *MoniDam*.

of 1000 Hz; the analysis was performed after decimation to 50 Hz, taking average values on decimation intervals) by averaging using time windows of 160 s length (Hanning) superposed at 2/3 (which gives a frequency accuracy of $\Delta f = 1/160 = 0.00625$ Hz). From these spectra, we can see that high amplitude vibrations occur near the centre of the upper gallery (points 4, 5 and 6). At the frequency range 2.0 - 3.0 Hz, we can identify clearly 5 important peaks at frequencies: 2.34, 2.53, 2.64, 2.73 and 2.76 Hz. In this case, a symmetric mode for the highest amplitude peak (2.53 Hz) has been identified. Taking a zoom over the frequency range 0.50 – 2.00 it was possible to distinguish spectral peaks linked to the intake tower vibration modes (Espada et al. 2011).

Fig. 7 shows the results of *MoniDam* concerning the analysis of the data collected between 3 and 17Nov2011 (14 days). In this period, the water level rose from 264 m to 269 m (almost linearly), which is a non-significant water level regarding the dam's height (base level: 165 m; crest level: 297 m). This figure also presents the identified frequencies at each hour. As can be seen, there is a peak at frequency range 1.15 - 1.10, which is assumed to represent the 2nd vibration mode (downstream-upstream direction) of the intake tower (Espada et al. 2011). This vibration mode of the intake tower has been identified on site (28 April 2009) from an ambient vibration test (Espada et al. 2011); the frequency identified in April 2009 was of about 0.9 Hz, a value less than the ~1.15 Hz identified in November 2011 because, in April 2009, the reservoir water level was higher (~281.9 m). It is also possible to identify a peak in the frequency range 2.35-2.40 Hz and several other peaks in the frequency range, in which resonances are expected regarding the main vibration modes of the system dam-reservoir-foundation. In the range 2.5 - 2.8 Hz, four main peaks can be identified at the frequencies mentioned above (2.53, 2.64, 2.73 and 2.76 Hz). The variability in the excitation conditions (power groups on or off) can induce different dam dynamic responses which are influenced by the water-structure interaction that can be responsible for the occurrence of coupled vibration modes (as identified from experimental data, these modes seem to be non-stationary, which may indicate a real non-proportional damping). The peaks matching these coupled modes present relative amplitudes that may change hourly due to the changes in excitation conditions. It should be noted that the amplitude of the vibrations under ambient excitation is about 10 to 20 times less than the registered amplitude with the power groups on, so, large variations in damping parameters (as is known, damping can greatly depend on the amplitude of vibrations) are expectable.

Finally, Fig. 8 shows a comparison between the peak frequencies that are automatically identified with *MoniDam* in the test period (3 to 17Nov2011) and the frequencies numerically computed with a simple FE model (with added water masses and proportional damping). In this period (14 days), it is possible to notice a slight decrease in the main peak frequencies automatically identified, which is due to the slight increase in the water level (from 264 to 269 m). This decrease in the main peak frequencies is of about 0.05 Hz and can be observed for several peaks including the peak linked to the 2nd vibration mode (1.15 to 1.10 Hz) of the intake tower.

The detection of this very small frequency decrease shows the great efficiency of this dynamic monitoring system.

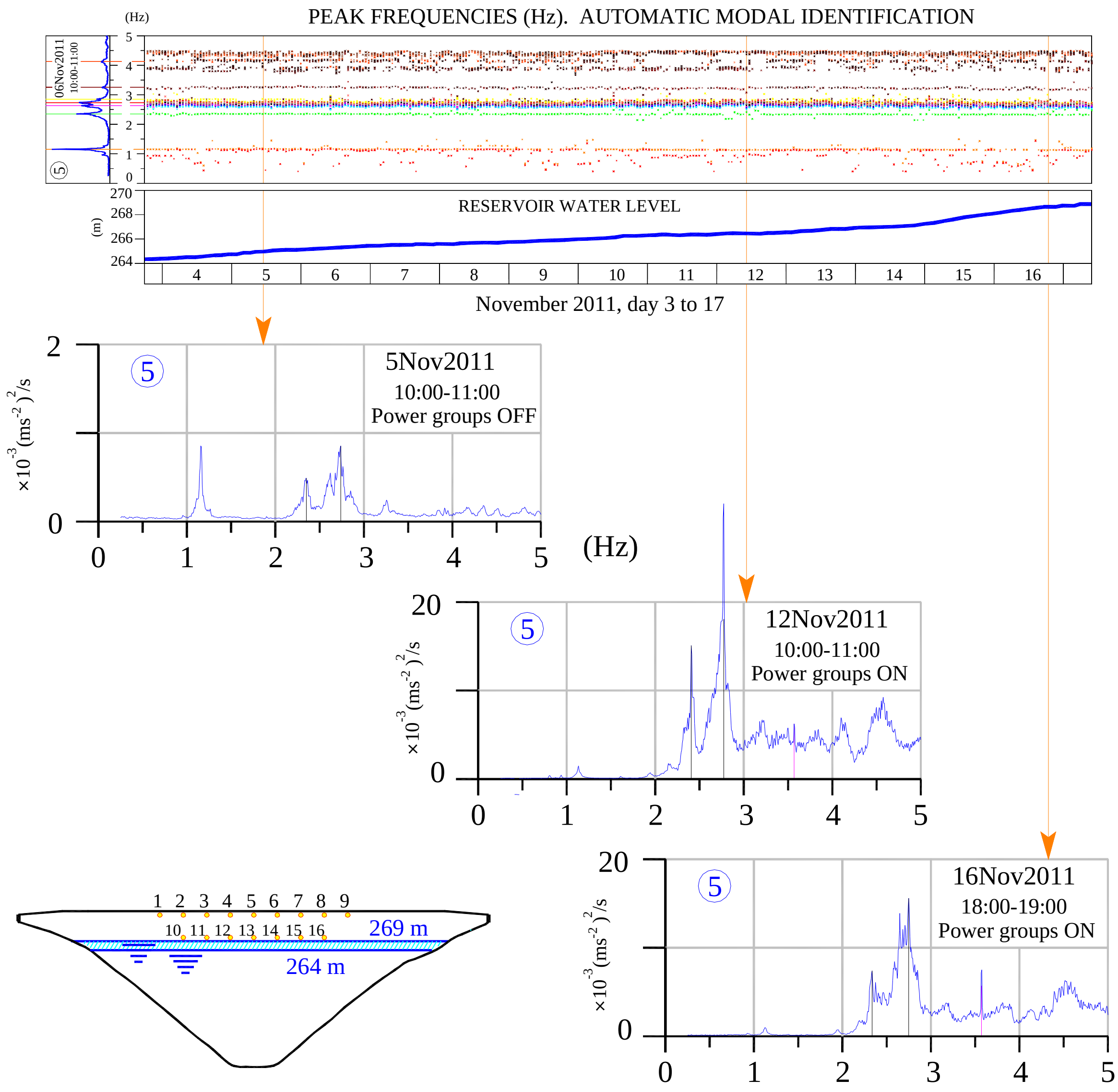


Figure 7. Results of *MoniDam*. Track of peak frequencies automatically identified between 3 and 17Nov2011. Water level evolution and some spectra at point 5 (top at the central cantilever).

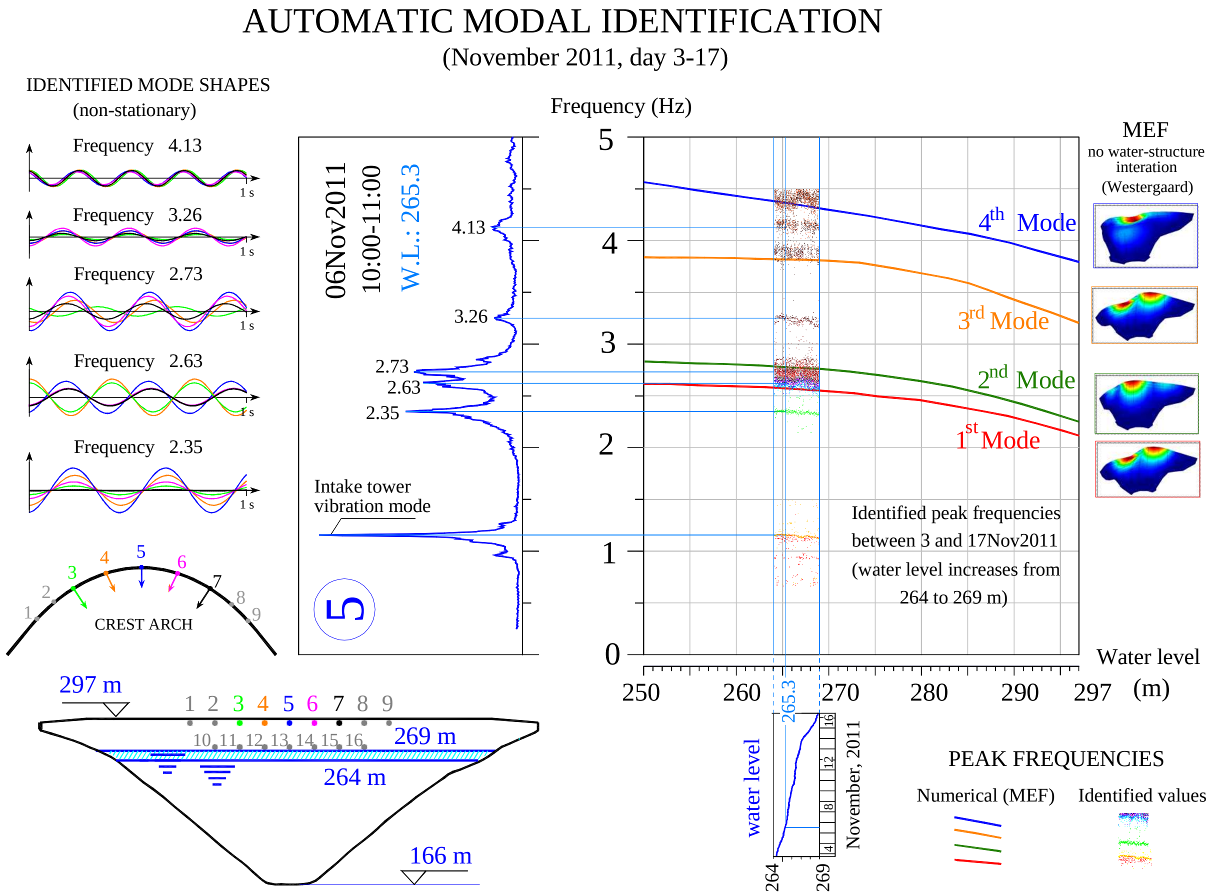


Figure 8. Comparison between the peak frequencies automatically identified with *MoniDam* in the test period (3 to 17Nov2011) and the frequencies numerically computed with a simple FE model. Identified non-stationary mode shapes and numerical mode shapes (stationary).

6. CONCLUSIONS

The presented results have shown that the use of Automatic Data Acquisition Systems (ADAS) for Continuous Dynamic Monitoring of Large Arch Dams makes it possible to gather very relevant information for the characterisation of the dynamic behaviour of dam-reservoir-foundation systems.

In particular, with this kind of systems it is possible to identify with great accuracy the time variation in the spectral peak frequencies corresponding to the main vibration modes of the dam-reservoir-foundation system as well as in the mode shapes. The presented results, regarding a 14 day period (3 to 17 Nov2011), show that the mentioned spectral peaks have relative amplitudes, which can vary significantly with the excitation conditions and the water level.

The installed system - acquisition software (*CabrilAquis*), management and analysis software (*MoniDam*) and hardware (accelerometers, digitizers, data gatherers and computer) - makes it possible to automatically detect small variations in peak frequencies, regardless of their amplitude, and also gives the possibility of identifying the vibration mode shapes.

The results of *MoniDam* indicate that the main spectral peaks identified can correspond to non-stationary vibration modes, of which the simulation through FE models requires the consideration of the water-structure dynamic interaction and the hypothesis of generalised damping. Hence, a new 3D numerical model to simulate the dynamic behaviour of Cabril dam is under development. This model, based on a state formulation, considers the hypothesis of non-proportional damping and uses water FE to discretise the reservoir.

This new model is expected to enable a better understanding of the observed results, which may be very important to extend our knowledge about the dynamic behaviour of these large structures.

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