Seismic assessment methods for concrete gravity dams

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15 WCEE
LISBOA 2012

SUMMARY:
The International Commission On Large Dams (ICOLD) estimates there are 37626 dams in the world. Those structures offer a great contribution to electricity generation, floods control and agricultural productivity. Due to this reason, the seismic assessment of existing dams is a particularly relevant problem, confirmed also in the recent international codes. Each dam is a prototype which needs a specific and deep analyses. In order to have an overview of seismic safety of existing dams simplified analyses can be useful. The main scope of this article is to compare the latest simplified methods with the modern finite element methods for seismic safety assessment of existing concrete gravity dams. Advantages and disadvantages of these different approaches will be shown. The methods treated are applied to a case study.

Keywords: Concrete Gravity Dams, Seismic Assessment, Simplified Analysis

1. INTRODUCTION

The seismic safety assessment of concrete gravity dams represents a complex and multidisciplinary problem of civil engineering. Since the high risk associated to the dams, due to the catastrophic consequences of collapse and uncontrolled release of water, the best analysis procedures with the greatest reliability are expected. This can be obtained using both simplified and accurate methods. The former give us an overview of the vulnerability of the dam and useful results for rapid checks, screenings and comparison; the latter, based on the use of Finite Element Method, allows the evaluation of the real structural response. In this sense, step by step procedures, starting from simplified methods going through accurate one represent a good approach for seismic assessment studies. Another essential part is related to the past experiences. The usual behaviour of the dams is well known thanks to the extended monitoring, on the other part their seismic behaviour is still in the initial stages of knowledge.

1.1. Seismic effects on dams

Few dams collapsed cause of earthquakes. The International Commission On Large Dams (ICOLD) in the 120th bulletin lists the main effects produced by the past earthquakes to dams. Embankment dam seem to be the most vulnerable. The collapse of Fujinuma dam (a 16 m high earth-fill embankment dam) caused by the Tōhoku Japan earthquake (11 March 2011, Ms 9.0), has confirmed this. Others important informations came from the Wenchuan earthquake (12 May 2008, Ms 8.0). During this, four key hydropower projects of different types have suffered reparable damages. According to the experience collected till now the most important seismic effect observed during strong earthquake motions are: cracks in the dam body, slidings along weak surfaces and damages of appurtenant structures.

1.2. Seismic assessment and seismic codes

In the past, seismic actions were considered in an approximated way multiplying the mass of the structure by the acceleration a = α x g, where α is the seismic coefficient equal to 0.07 – 0.1 and g is the gravity acceleration. In those analyses dam body was considered rigid and hydrodynamic pressure taken into account using Westergaard distribution of pressure [Westergaard, 1931] present nowadays in various national codes.

A lot of developments and updating are done from that time. In some cases seismology studies change the actions expected on the ground. In a recent update of the ICOLD bulletin entitled “selecting seismic parameters for large dams” the probabilistic hazard assessment associated to periods of return up to 10000 years is introduced. In this scenario of regulatory updates seismic assessment of existing dams is becoming a pressing question. This is particularly true in Italy, where the main part of the 500 existing dams were built in the sixties with different seismic actions and obsolete standards. A new regulation is going to be published to reduce the gap in seismic assessments [CSLP, 2008]. In the following analyses the requests of the new Italian rules have been complied.

2. CASE STUDY

In order to focus the principal advantages and disadvantages of simplified or accurate analyses, the different approaches are applied to a real Italian dam. The main geometric and mechanical features of the case study are resumed in Figure 1 and Table 2.1. The dam is composed by 19 blocks large about 20 meters. The tallest block is 87.00 m high with an upstream and downstream slope of 0.03 and 0.70. Usual water lever is considered 2.00 meter under the top of the dam.

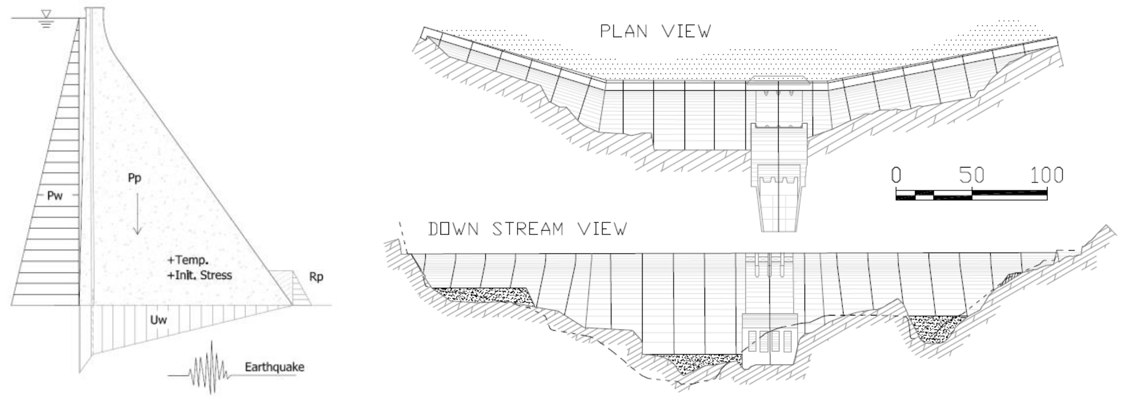

Figure 1. (Left) Section of the dam and main loads applied (right) plan and downstream view of the case study

Table 2.1. Mechanical properties of concrete and rock

Description	Symbol	Value	Unit
Concrete – specific weight	γ_c	23.90	KN/m^3
Concrete – Young modulus	E	23640	Mpa
Concrete – compressive strength	R_{ck}	34.910	Mpa
Concrete – tensile strength	f_{ct}	1.30	Mpa
Rock – specific weight	γ_f	27.30	KN/m^3
Rock – Young modulus	E_f	41550	Mpa
Concrete- Rock – friction angle (residual)	Φ_f	45	

Seismic action is considered by means of the target spectrum derived from Italian regulation and seven natural time histories selected and modified to be spectrum compatible. Different limit states are considered, in the following only the “collapse” one is reported. The Peak Ground Acceleration (PGA) associated with this (related to a probability of exceedance of 5% in 100 years) is equal to 2.53 m/sec^2.

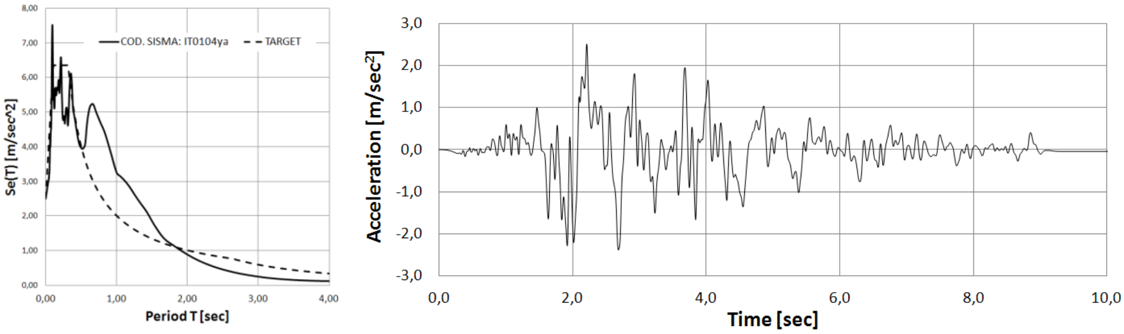

Figure 2. Seismic actions: Target and IT0104ya spectra, time history IT0104ya.

3. SIMPLIFIED ANALYSES

Simplified methods help to have an overview of the main problems that can affect a particular dam, and give us the reference values to ensure the reliability of the advanced results. For preliminary analyses, especially for gravity dams composed by vertical adjacent blocks, it's possible to use planar analysis. In the next paragraphs the study is conducted on the highest blocks that could be considered the most vulnerable.

The simplified methods described hereafter take into account the interactions reservoir –foundation – structure and the non-linearity caused by sliding at the base of the dam. Furthermore, three-dimensional effects (such as those caused by narrow valleys) are evaluated estimating the mutual forces that develop as a result of the interaction between adjacent segments.

3.1. Preliminary analysis using Fenves and Chopra method

If only the fundamental period of an empty gravity dam is considered, the structure can be associated to a SDOF system. Starting from this hypothesis, Fenves and Chopra have evaluated the interaction between the structure, the foundation and the reservoir arriving to a simplified method [Fenves and Chopra, 1986]. A modified SDOF, equivalent to the system structure – rock foundation – reservoir, can be represented using the following modified characteristics:

$$\tilde{T}_1 = R_r\, R_f\, T_1\,; \quad \tilde{\xi}_1 = \left(1/R_r\, R^3{}_f\right) \xi_1 + \xi_r + \xi_f \tag{3.1}$$

Where R_r, R_f, ξ_r, ξ_f can be evaluated from tables.

The main effects of these interactions are: the increase of mass due to water, the decrease of stiffness related to foundation deformability and the increase of damping caused by the double contribution of the radiation damping in the rock foundation and of the waves absorption into the reservoir deposits. The authors also provide a useful equivalent static distribution of forces reproducing the earthquake effects.

$$f_1\,(y) = \tilde{L}_1/\tilde{M}_1 \,*\, Sa(\tilde{T}_1, \tilde{\xi}\,)/g[w_s\,(y)\phi_1\,(y) + g\bar{p}_1\,(y, \tilde{T}_r\,)] \tag{3.2}$$

Where $\tilde{L}_1$and $\tilde{M}_1$are equivalent generalized quantity, w_s and $\bar{p}_1$ are related to inertial and hydrodynamic pressure and $\phi_1$is the fundamental modal function.

According to this method, the period of the dam considered empty and on rigid foundation T_1 is 0.215 sec. After the evaluation of $R_r = 1.344$ and $R_f = 1.099$, the equivalent period taking into account fluid and soil interaction becomes 0.317 sec. For $\xi_r = 1.3\%$ and $\xi_f = 3.5\%$ the damping of the dam pass from 5.0% to 7.6%.

In order to evaluate the seismic response of the dam, the static equivalent pressure (3.2) can now be applied to a mono-dimensional element like a cantilever or to a bi-dimensional FEM model (to catch better distribution of stress). For a rapid evaluation of the seismic safety of the dam the main results directly related to the possible kind of failures are: the upstream stress at the base $\sigma_m = 2.88\ MPa$ (> f_{ct}) , the Sliding Safety Factor $SSF = 0.703$ at the base and top displacement $u_{max} = 0.039\ m$. The results are obtained from a simple spreadsheet called S.I.M.DAM [Furgani et al., 2011].

According to these results, both cracks and base sliding can happen, that mean: accurate analyses are needed in these directions. In Table 3.1 results referred to different simplified approach applied to a cantilever beam are reported.

Table 3.1. Results from the simplest method to Fenves and Chopra simplified method

	σ_m[Mpa]	SSF [-]	u [m]
Rigid dam inertial action with Westergaard pressure (as done in the past)	1.09	0.91	0.015
Fluid – structure interaction (Fenves and Chopra with rigid foundation)	3.13	0.68	0.041
Soil - structure interaction (Fenves and Chopra using Westergaard pressure)	1.04	0.89	0.027
FLUID-SOIL-STRUCTURE INTERACTION (Fenves and Chopra)	2.88	0.70	0.039

3.2. Base sliding using Nuti and Basili Method

Concrete gravity dams are built on rock, for the sliding problem, the concrete-rock interface is reasonably the weakest link. Nuti and Basili have proposed a simplified method to evaluate the slip at the base of the dams using the results of non linear analysis [Basili and Nuti, 2009]. Base sliding of dams during seismic motion was evaluated through the object-oriented framework for finite element analysis Opensees. The dam was modelled with a SDOF system enriched with a non linear link at the base, as described in Figure 3.

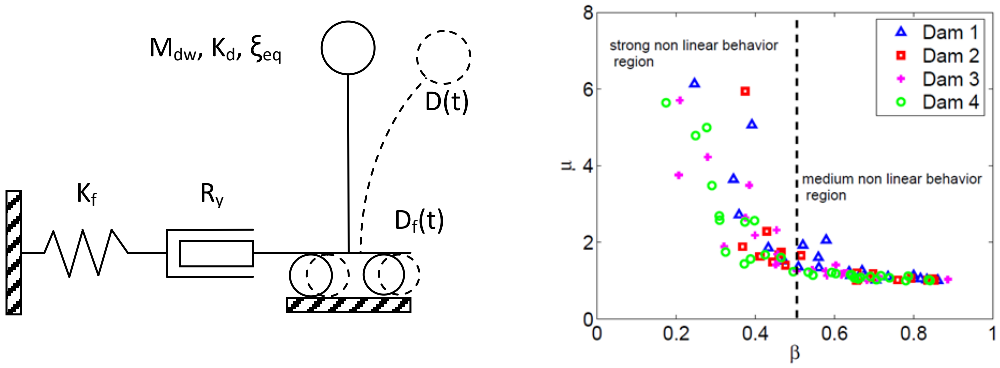

Figure 3. (Left) Model for non-linear sliding response analysis (Right) results of non linear analysis

Using a selection of 47 natural earthquake collected from Italian earthquake strong motion database (ITACA), the Pacific Earthquake Engineering Research Centre (PEER) and the European Strong Motion Database (ESD), non linear analyses are conducted using four case studies of concrete gravity dams. The main parameters of the analysis are: the sliding resistance R_y (due to frictional, cohesive and passive wedge resistance), the equivalent characteristics of the system M_{dw} , K (derived from Fenves and Chopra method), and the participation factor p that relates the equivalent displacement $D(t)$ with the real displacement $y(t) = p\ D(T)$.

From the results of these nonlinear dynamic analyses a law that correlates the parameter μ (the ratio between the maximum elastic displacement and the displacement producing sliding at the base) and the parameter β (ratio between sliding limit acceleration and the spectral acceleration) was built. The law, effective when β is in the range 0.5÷1.0, allows the assessment of the residual displacement.

Using the following method as showed in the Table 3.2 we obtain a residual displacement of 3.26 cm. This slip phenomenon has the important consequence of energy dissipation inherent to the acceleration limit definition. Thanks to its simplicity this method can be very powerful for the safety screening of existing dams.

Table 3.2. Main results from Nuti and Basili simplified method

	Formula	Value	Unit
Sliding resistance	$R_y = R_{yc} + R_{y\Phi} + R_{yPWR}$	54 653	kN
Equivalent mass	$M_{dw} = (\widetilde{\omega}_f^2/\widetilde{\omega}^2)\,\tilde{L}$	1660	ton
Acceleration limit	$a_L = (\tilde{R}_y - P_w)/M_{dw}$	4.04	m/sec^2
Parameter β	$\beta = a_L/a(\tilde{T})$	0.714	-
Parameter μ	$\mu = y_{max}/y_y = 1/\beta$	1.40	-
Participation factor	$p = (\widetilde{\omega}_f^2/\widetilde{\omega}^2)(\tilde{L}/\tilde{M})$	2.865	-
Sliding displacement	$y_y = p\,R_y/K_d$	0.0813	m
Residual displacement	$y_R = y_{max} - y_y = y_y(\mu - 1)$	0.0326	m

3.3. Three-dimensional effects with S.I.M.DAM

The vertical joints, made principally to reduce the self-tension due to heat of hydration of the concrete, divide the dam body in blocks. The contact between the blocks, which depends on their degree of connection (type of joint), the seasonal temperature variations (joints closed in summer and open in winter) and the topography of the valley, generates mutual forces and affects the structural response of the dam.

Three-dimensional effects can be evaluated in a simplified way using a “grid approach”. Dam is divided into vertical cantilever, connected horizontally using a specific constitutive law. In a previous study [Furgani et al., 2011] the mutual forces - relative displacements relationship, $R_{g_i} = k_{g_i}\delta_i$, is used. With a simple spreadsheet it’s possible to implement an iterative procedure to evaluate the mutual forces between adjacent blocks as described in Figure 4. From relative displacement between the cantilevers, preliminarily considered disconnected, it’s possible to evaluate the mutual forces that modify the relative displacement previously evaluated. This is repeated till convergence is reached.

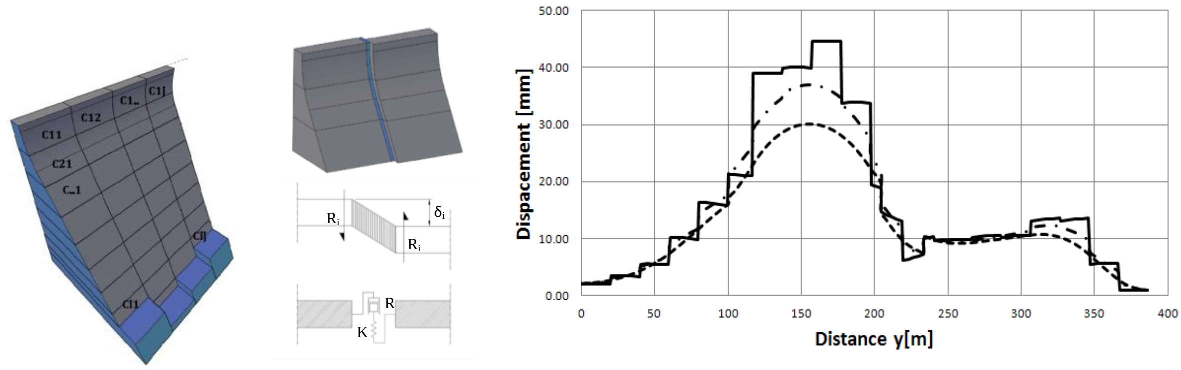

Figure 4. (Left) Discretization of a dam and scheme used to reproduce interactions between the blocks (elasto – plastic constitutive law) (Right) Top displacements of the blocks considering them not connected (solid line) perfectly connected (dashed) and using the modelling joint shear stiffness (dashed-dot line).

The dam was subjected to simplified analysis using S.I.M.DAM. This program considers all the 19 blocks by means of beam theory. Each of this is subjected to Fenves and Chopra seismic force distribution (3.2). The analysis is carried out in both linear and plastic field, considering base sliding as described before.

The influences of different scenarios such the possible not activation of wedge passive resistance or the decrease of elastic stiffness due to cracks are also investigated.

In absence of more information, the horizontal shear stiffness k_{gi} is assumed equal to 10 000 KN/m^3. The analysis of the effects of the mutual forces developed between blocks (considered connected or not) takes to the results listed in Table 3.3. As showed three-dimensional effects produce generally a decrease of the response of the highest block. This depend principally by the value of k_{gi} that, in absence of laboratory tests, can be evaluated through accurate numerical analyses taking into account the range of seasonal temperature.

Table 3.3. Three-dimensional effect evaluated through simplified S.I.M.DAM method

	σ_m[Mpa]	SSF [-]	u [m]
Fluid-Soil-Structure Interaction (Fenves and Chopra)	2.88	0.70	0.039
Interactions between the blocks (S.I.M.DAM)	2.62	0.75	0.036

This method, associated to the calibration of $\mathrm{k_{gi}}$, can be applied also to 3D FEM program using solid elements, characterized by only shear stiffness $\mathrm{k_g}$ (positioned between the vertical blocks) to model the joints. Some results are showed in Figure 4 using $\mathrm{k_g} = 100\ 000\ \mathrm{KN/m^3}$. As showed dam have an intermediate behaviour between an "independent blocks dam" and a "monolithic dam".

4. FEM ANALYSES

FEM analyses can reduce the gap between simple scheme used in the simplified analyses and reality. Seismic response of dams implies a lot of problems: dynamic interaction, non linearity, water pore pressure evolution, pre-seismic stress state and others [ICOLD, 1986]. A single analysis able to analyze in detail all the phenomena is quite impossible and too expensive in term of parameters. In this sense a good selection of FEM methods must be used. This means that some problems can be evaluated using less advanced method.

In the following paragraphs, accurate analyses are conducted using Abaqus 6.11. In order to catch sliding, cracking, fluid structure interaction and the soil structure interaction, two-dimensional analyses using plain strain elements are developed. Sliding and cracking problems are treated using respectively the "Coulomb friction model" and the "damaged plasticity model" present in Abaqus and briefly described in Figure 5 [Dassault, 1993]. The Rayleigh damping parameter α and β associated to mass and stiffness are assumed respectively 2.4166 and $6.43x10^{-4}$ (when cracking is considered $\alpha = \beta = 0$). The three-dimensional effects are evaluated using tetrahedral finite elements and "surface to surface contact" model using a "Coulomb friction model" (friction coefficient $\mu = 1$). In order to take into account the static forces, the self weight and the hydrostatic pressure, a fictitious step is defined before the seismic analysis. All the following analyses are conducted following the same starting hypothesis, simply adding new elements of complexity. In this sense the framework followed can be defined a "step by step" procedure starting from simple going through complex.

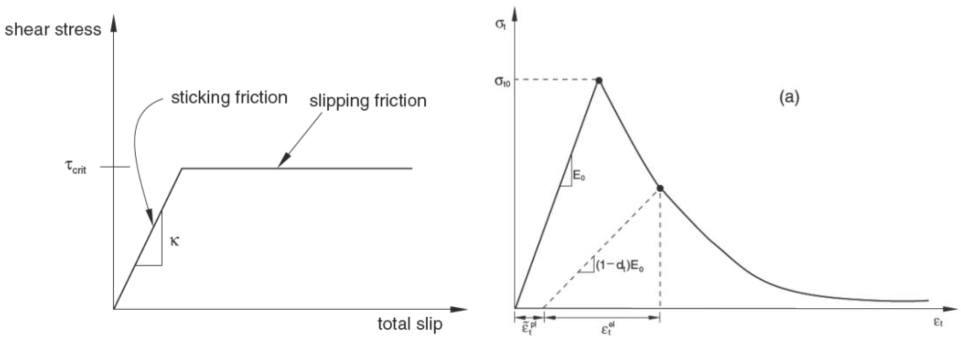

Figure 5. (Left) Coulomb friction model (Right) damaged plasticity model associated to tensile stress

4.1. Linear Dynamic Interactions

4.1.1. Fluid Structure Interaction

Fluid Structure Interaction (FSI) can be solved using different hypothesis. The dam may be considered rigid or flexible. In the first case only impulsive pressure is evaluated. In the flexible case, sloshing and interaction pressure is also evaluated. Convective pressure for dams is negligible, so the main problem is the evaluation of interaction between the deformation of the structure and the boundary changes of fluid medium. Different approaches exist to solve this problem, the main two are: “lagrangian approach” and “eulerian approach”. Different studies are done using both the approaches but the latter is nowadays the most used.

Despite of this, in the following analyses, to evaluate the hydrodynamic effect the added mass approach is used. Taking as reference the Italian standard [CSLP, 2008] the hydrodynamic effect of reservoir during earthquake can be evaluated using the formula:

$$\mathrm{p(y)} = \mathrm{a_g(t)}\ \rho_\mathrm{w}\ \mathrm{c(y)}\ \mathrm{y_0} \tag{4.1}$$

$$c(y) = \frac{c_m}{2}\left[\frac{y}{y_0}\left(2-\frac{y}{y_0}\right)+\sqrt{\frac{y}{y_0}\left(2-\frac{y}{y_0}\right)}\right] \tag{4.2}$$

Where y is the vertical coordinate from the free surface, y_0 is the height of the reservoir and $c(y)$ is the shape function of pressure. The added mass pressure formula is applied to the upstream wall of the dam as a pressure. It’s applied together with the acceleration at the base of the dam and varies over the time as the Peak Ground Motion $\mathrm{a_g(t)}$. The effects on the relative displacement of the fluid structure interaction evaluated in this simplified way are reported in Table 4.1.

4.1.2. Soil Structure Interaction

Soil structure interaction (SSI) is a problem well known for dams. Due to the dimension of the foundation and the weight of the dam, SSI effects are always relevant. The problem can be treated by mean of two main approaches: the “direct approach” and the “substructure approach” [Wolf, 1988]. In the former, a consistent part of the soil is modelled. In the latter, boundary condition obtained from dynamic impedance matrix can replace the half space under the dam base. In this work the direct approach is used.

SSI gives to the system more flexibility and additional damping. In the following analyses the first effect is implicitly reproduced modelling the foundation (see Figure 6) while the second is introduced thanks to Lysmer dampers positioned at the vertical boundary of the soil part. The damping associated to these dashpots is evaluated as $c = \rho_f\ A\ v_c$, where $v_c = \sqrt{(E_f/\rho_f)}$ is the velocity of compression waves and A is the afferent area. The dimensions of the soil are four time the height of the dam. A coarser mesh is used in order to reduce the computational effort. Dynamic analyses are conducted applying the time history at the base of the foundation. The effects of SSI interaction are described in Figure 6 and relative displacement reported in Table 4.1.

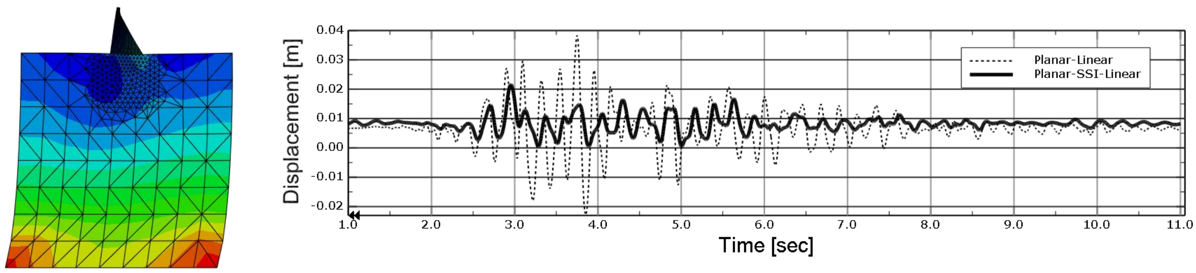

Figure 6. (Left) 2D FEM Model deformation and (Right) effects of SSI on the relative top displacements.

Table 4.1. Results obtained from linear dynamic analysis

	u_{max} [m]
Hydrodynamic added pressure NOT considered	0.025 (t=3.75)
Hydrodynamic added pressure considered	0.038 (t=3.75)
Soil Structure Interaction with added hydrodynamic pressure	0.022 (t=2.96)

4.2. Non Linear Dynamic Analyses

4.1.1. Sliding

In the simplified method sliding stability and residual displacement are evaluated using the resultant forces. Thanks to FEM programs it's possible to consider local effects. Using the "Coulomb friction model" (Figure 5) bi-dimensional analyses are done. In order to have the biggest value of slip the Lysmer dampers are not used.

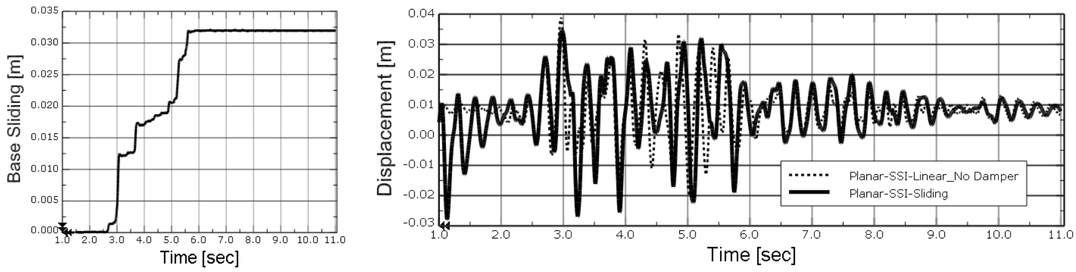

Figure 7. (Left) slip during earthquake and (Right) sliding effect on relative top displacement.

The maximum slip due to seismic action is 3.2 cm. Considering the differences between spectrum and time history, simplified analyses are in good agree with accurate one. As showed in Figure 7, base sliding produces also a reduction of maximum displacement (as a consequence a reduction in stresses).

4.1.2. Cracking

The correct evaluation of crack position and entity is fundamental for dam safety. Crack openings are the only measures that can say if the structure is safe. Two main approaches are nowadays present: the smeared and the discrete crack approach. The former approach is the one used in the following.

Non linear dynamic analysis that takes into account the damage of the concrete is done using the "damaged plasticity model" implemented in Abaqus. This model is based on the plasticity scheme that permits to evaluate the plastic strain produced when the yield (compressive and tensile) values are reached and on the damage theory that quantify during the cyclic load the damage of the material through the reduction of stiffness (see Figure 5).

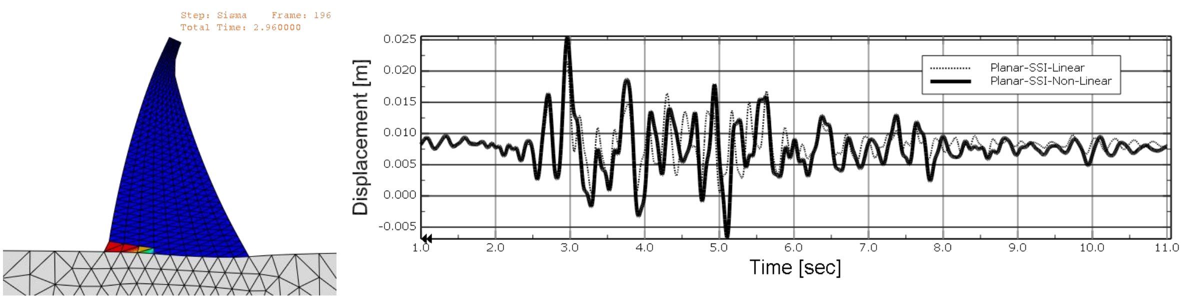

Figure 8. (Left) damage at the base of the dam and (Right) effect of cracks opening on relative top displacement

The maximum crack opening evaluated at the base of the dam is less than 1 mm. The crack opening is particularly important for uplift pressure evolution that can modify the equilibrium of the entire dam. As showed in the chart, cracks take to an increase of deformability; relative displacement varies from 0.022 m (Table 4.1) to 0.025 m.

4.3. Three-dimensional effects

When dams are founded on deep valleys, as in this case study, 3D analyses must be conducted. The special topography of the foundation and the connection between the opposite faces of the blocks can in fact produce mutual forces able to modify the seismic response of the structure.

Using the "surface to surface contact" and the "Coulomb friction model" of Abaqus the mutual forces are modelled. Dynamic analyses are conducted in the same way described in the previous paragraphs but using 3D finite elements and neglecting soil structure interaction.

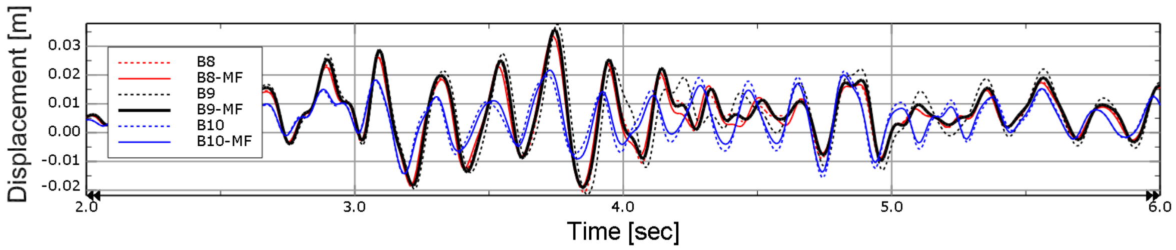

Figure 9. Effects of mutual forces (MF) on the linear response of the three highest blocks

As showed in Figure 9 three dimensional effects have little influence on the relative displacements of the highest blocks of the dam. This makes the results of planar analyses more reliable.

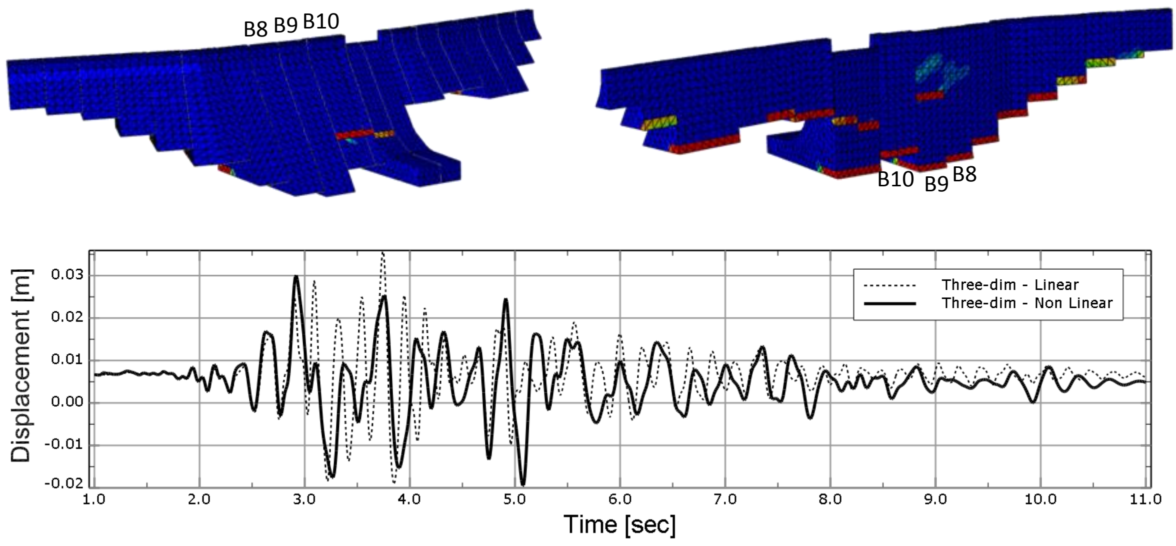

Figure 10. (Top) damages (stiffness reduction) evaluated when dam is full and (Bottom) effects on relative displacement histoy of three-dimensional and non linearity

To better show the possible damages on the entire dam, non linear analysis is also conducted. The maximum crack opening, obtained at the base of the dam, is 1.1 mm. From the distribution of damages, it seems that highest block is not, in general, the most vulnerable (see the downstream wall of B10). As shown in (Figure 10) the relative displacement of this is 0.030 (t=2.92) lower than the maximum linear displacement 0.038 (t= 3.75).

4.4. Conclusions

Simplified and accurate methods for the evaluation of seismic safety of concrete gravity dams are briefly described in this paper. Some of these are applied to a case study and the results compared. The problems treated are: linear dynamic structure-reservoir-foundation interactions, the non linearity due to slidings and cracks and the three-dimensional effects.

With the "step by step" procedure used in this work, based on a "simple to complex" approach it's possible to follow an organized framework to find the most important aspects of seismic safety of the dam, to address the more accurate analyses and to check the results obtained.

The results obtained with the simplified method are in agree with the accurate one. Thanks to this they can give us the order of greatness associated to the problem considered and allow to detect the vulnerabilities of the dam. Accurate analyses are essential especially for cracking analyses and to focus on the three-dimensional aspects of the problems.

ACKNOWLEDGMENT

The present work is carried out within the activities of AT2 – LR 2 – Task 2.2.1 of the ReLuis-DPC Executive Project 2010-2013, whose financial support is greatly appreciated.

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