# Shear Sliding of Gravity Dams for Maximum Design Earthquake Analysis



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#### SUMMARY:

The safety analysis of a concrete gravity dam requires the study of a shear sliding failure scenario through the concrete/rock foundation interface for the maximum design earthquake. The shear sliding analysis of a gravity dam with 100.0 m height is performed. A bi-dimensional explicit model is adopted, where the concrete dam and its foundation are represented by adjacent elastic blocks that interact with each other through the concrete/rock interface. Different models for the concrete/rock interface are adopted based on the finite element method, in small displacements, and on the discrete element methodology, allowing large displacements at the interface. Several parametric studies are presented where the peak ground acceleration, the damping model and, the foundation stiffness influences are assessed.

Keywords: Gravity dam, Numerical modelling, Shear sliding, Earthquake engineering

## **1. INTRODUCTION**

The seismic evaluation of concrete gravity dams should consider the possibility of sliding on the horizontal foundation joint for the maximum design earthquake, MDE. This analysis should be performed not only for new dams but also for existing ones given that in the past simplified pseudo-static models were usually adopted. The reassessment of existing gravity dams should also be considered when the existing design norms are updated leading to higher maximum loads and more demanding safety factors.

Before considering linear or nonlinear advanced computational models, it is possible to perform an initial pseudo-dynamic analysis, Fenves and Chopra (1987). More recently, Basili and Nuti (2011), have enriched the latter simplified mechanical model by including a nonlinear frictional and a cohesive threshold shear resistant value. Léger et. al (1996) presented an overview of the procedures required to perform finite element analysis of concrete dams. Several computer programs allow this kind of analysis, Singhal (1991). In order to be able to perform nonlinear advanced shear sliding analysis, the adopted analysis package should consider joint or gap elements with a non-linear behavior, connecting the dam to a rigid or flexible foundation and they should also consider preferably a time domain procedure.

Chavez and Fenves (1995) have proposed a hybrid frequency-time domain procedure that allows for base sliding on an elastic half-plane. Several examples can be found in the literature that deal with the application of finite element models to dam with a special emphasis in shear sliding analysis, namely in 2D, Aidi and Hall (1989), Léger and Katsouli (1989), Mir and Taylor (1996), and in 3D Arabshahi and Lofti (2008). Some of the works also include nonlinear models for the concrete dam. A discrete element approach to shear sliding scenario can also be adopted, Lemos (1999). Programs based on discrete element technology have the ability to model large displacements at the joint interfaces.

In this work two structural analysis programs are adopted, *Parmac2D* and *Dec-Dam*. Both adopt an explicit time domain solution algorithm based on the centred-difference algorithm, the main difference between them are the models adopted for the interface. The program *Parmac2D* can adopt either, zero thickness interface elements, small displacement analysis, or a discrete element model based on a simplified circular particle/edge interaction, Azevedo and Lemos (2006). The program *Dec-Dam* adopts for the interface sophisticated discrete element edge/edge contact models, Bretas (2012).

The shear sliding analysis of a gravity dam with 100.0 m height is performed. A bi-dimensional plane strain model is adopted, where the concrete dam and its foundation are represented by adjacent elastic blocks that interact with each other through the concrete/foundation interface. Several parametric studies are presented where the peak ground acceleration, the damping model and, the foundation stiffness influences are evaluated. The performance of the different structural programs adopted is compared, namely the interface models.

#### 2. NUMERICAL MODELS

#### **2.1. Introduction**

The shear sliding structural analysis was performed with two structural analysis programs that follow an explicit time domain solution algorithm. In both programs, a 4 node plane strain finite element is adopted in the dam and foundation discretization. The programs are similar in all aspects except in the models adopted for nonlinear interfaces.

The program *Parmac2D* can adopt either, zero thickness interface elements Hohberg (1992), small displacement analysis, following a two point Lobatto integration rule, Fig. 1a, or a discrete element model based on a simplified circular particle/edge interaction, Azevedo and Lemos (2006), Fig. 1b, that allows large structural displacement at the interface. In a simplified manner the finite elements edges representing the dam are discretized with smaller inner circular particles that are allowed to interact with the edges of the finite elements representing the foundation boundary. The contact length of each particle/edge contact is directly given by the particle circular radius, but the contact can be transferred from one finite element edge to the adjacent as the calculation progresses and the contact displacement increases.

The program *Dec-Dam* adopts an elaborated edge/edge contact following a discrete element methodology, full implementation details can be found in Bretas (2012). As the calculation evolves it is able to recognize new possible finite element edge to finite element edge contacts. It also updates the contact geometry, including the contact length, of the existing contacts, Fig. 1c.



Figure 1. Contact models for dam/foundation interface joint

When the structural program predicts significant shear displacements at the concrete/foundation

interface, a contact methodology that allows large displacements should be considered. A discrete element or a finite element approach that allows large displacement should be adopted for the interface model as it will be shown.

In the examples here presented a linear elastic model was adopted for the dam and for the foundation plane elements. For the dam/foundation interface a nonlinear Mohr-Coulomb without cohesion was adopted in all contact models, following the Portuguese dam design recommendations.

# 2.2. Loading considerations

On a shear sliding scenario analysis the self-weight of the dam, the hydrostatic pressure acting upstream, corresponding to a reservoir level at its crest and the uplift pressure at the dam/foundation interface are the main static forces to be considered.

When a drainage system is installed, a bilinear diagram along the dam/foundation interface is adopted. A 2/3 reduction of the hydraulic head at the drain location is usually considered. At the upstream and at the downstream faces the hydraulic head is given by the corresponding water level. More elaborated models that apply a hydromechanical analysis through a one-way coupling can be adopted if required, Lemos (1999) and Bretas (2012).

Prior to the seismic analysis, a seismological study for the dam location should be performed in order to characterize the peak ground horizontal acceleration and the corresponding time records. In this work, in a simplified manner, the vertical seismic component is given by scaling with a factor of 2/3 the horizontal acceleration records. The vertical component of the seismic action can increase the shear sliding values that are predicted, for this reason it is important to adopt a model that allows the consideration of both components.

Earthquake loading is represented by an upward moving plane wave as shown in Fig. 2, Lemos (1999). At the bottom boundary quiet boundary conditions are adopted and the dynamic input is applied as a stress wave. The viscous boundary formulation proposed by Lysmer and Kuhlemeyer (1969) is adopted. At some distance from the dam, free-field conditions occur. Numerically this is accomplished by supplying to the model lateral boundaries where the free-field conditions are established, a thorough description of the free-field implementation can be found in Lemos (1999).

Several models are available for the dynamic interaction between dam and water reservoir, namely the Westergaard added mass approach, Westergaard (1933), an Euler formulation for the fluid, Cervera et al. (1995), or a Lagrangian fluid formulation, Wilson and Khalvati (1983). In this work the simplified added mass concept is followed, with this purpose the nodal points at the dam upstream face have their mass increased in the normal direction to the upstream face.

## 2.3. Rayleigh damping

In structural dynamics Rayleigh damping is commonly adopted give its simplicity and the control it allows over modal damping ratios, namely for linear elastic analysis. In Rayleigh damping, the damping matrix, [C], is given by:

$$[C] = \alpha[M] + k[K] \tag{2.1}$$

where,  $\alpha$  and k are damping coefficients associated to the mass matrix, [M], and to the stiffness matrix [K], respectively. The mass proportional term of the Rayleigh damping corresponds physically to linear viscous dampers connecting the structure nodal points to external supports, whereas the stiffness proportional term corresponds physically to linear viscous dampers that interconnect the structural nodal points, Hall (2006). The coefficients  $\alpha$  e k can be defined using:

$$\alpha = \xi_{t \operatorname{arg} et} \, \omega_{t \operatorname{arg} et}$$

$$k = \xi_{t \operatorname{arg} et} / \omega_{t \operatorname{arg} et}$$

$$(2.2)$$

where,  $\xi_{target}$  is the fraction of the critical damping for the target mode and  $\omega_{target}$  is the corresponding vibration mode frequency with the lowest modal damp. As pointed out in Hall (2005) the mass proportional term is not a real physical mechanism. When the damping formulation is in global coordinates, rigid body motion can lead to erroneous damping forces if the mass proportional term is present. This can occur in a shear sliding scenario. For this reason, in Hall (2005) it is proposed that only the stiffness term should be adopted. Hall (2005) also proposed a maximum threshold value of the damping forces associated with the stiffness term in order not to introduce erroneous extremely high stiffness damping forces.

In Fig. 3 it is shown the modal damping percentage in terms of the vibration frequency, for a 5% damping ratio for a target frequency of 1.76 Hz. The mass proportional term highly damps modal responses with small frequencies, as referred before this can lead to erroneous damping for rigid body sliding modes. In high frequencies, the mass proportional term has a small contribution, this as it will be shown in the case studies here presented, can lead to higher shear sliding displacement values when compared to the examples where Rayleigh damping is adopted, or when only the stiffness term is included.

In the analysis carried in this work, for the reasons mentioned above, the effect of the stiffness and mass viscous damping are analyzed in separate. One of the Rayleigh damping terms may erroneously overdamp the mechanism that triggers the failure mode.



Figure 2. Boundary conditions for seismic analysis

Figure 3. Rayleigh damping

#### 2.4. Modelling sequence

In the initial stage the dam self-weight and the hydrostatic pressure at the upstream face are applied adopting a linear behavior for the dam/foundation interface. Next, the nonlinear interface model is adopted and a static analysis starting from the linear elastic equilibrium is carried out. Finally, the uplift pressure at the interface is considered and a static analysis is performed. During the static loading, the base and the lateral boundaries of the foundation have zero displacements in the normal direction to the corresponding faces.

At the end of the static loading, the supports are replaced by the equivalent static reactions and the Westergaard equivalent hydrodynamic masses are added to the upstream face nodal points in the direction normal to the upstream face. In a second stage a time domain dynamic analysis is performed, where the earthquake loading is represented by an upward moving plane wave in the horizontal and in the vertical direction. In this work, in a simplified manner, the vertical seismic component is given by scaling with a factor of 2/3 the horizontal acceleration record.

# **3. CASE STUDY**

## **3.1. Introduction**

The shear sliding of a gravity dam with 100.0 m height is carried out, Fig. 4. The numerical model consists of two different blocks, dam and foundation, that interact between them through the interface zone, Fig. 5. The foundation block is represented by 817 plane 4 node strain finite elements, and the dam block is discretized with 435 plane 4 node strain finite elements. Table 1 shows the material properties adopted for the dam and the foundation. As indicated, three different scenarios are adopted for the foundation elastic stiffness. Both the foundation block and the dam block follow a linear elastic model.

 Table 3.1. Material properties

	Dam	Foundation	Foundation	Foundation
		Z1	Z2	Z3
Young Modulus [GPa]	24,0	10,0	17,0	24,0
Poisson Coefficient	0,18	0,175	0,175	0,175
Density [kg/m <sup>3</sup> ]	2,30	2,65	2,65	2,65
Density [kg/m]	2,30	2,03	2,03	2,03



Figure 4. Dam monolith geometry Figure 5. Mathematical models for non-linear response history analysis

For the dam/foundation interface a Mohr-Coulomb constitutive law without cohesion was adopted for all the interface models under analysis. Table 2 presents the mechanical properties that were adopted for the nonlinear interface.

	Foundations
	Z1, Z2 & Z3
Normal stiffness [GPa/m]	24,0
Shear stiffness [GPa/m]	8,0
Friction angle [°]	45,0
Cohesion [MPa]	0.0
Tensile strength [MPa]	0.0

Table 3.2. Interface Joint Elastic and Strength Properties

The static loads considered where the self-weight of the dam, the hydrostatic pressure at the upstream face corresponding to a reservoir level at the crest and the uplift pressure. For the uplift pressure it was considered a bilinear diagram with a 2/3 reduction of the hydraulic head at a distance of 7,5 m from the dam upstream heel. This is the zone where the drainage system is considered to be installed.

For the maximum design earthquake, MDE, two accelerations records were adopted for the vertical and for the horizontal components, Fig. 6. They were defined based on a fault model, Carvalho (2007), for a peak ground acceleration of 0,17  $\mathbf{g}$ , Fig. 6. The records were then scaled for the several peak ground acceleration values analyzed.



Figure 6. Accelerograms for maximum design earthquake (horizontal & vertical components)

## 3.2. Modal analysis

In the dynamic analysis of gravity dams Rayleigh damping centred in the fundamental modal response frequency is commonly adopted, Bureau and Keller (2005). For this reason it is required to perform a modal analysis in order to find the fundamental frequency of the model. This was performed considering a massless foundation and adopting the Westergaard added mass at the upstream face degrees of freedom in the direction normal to the face, reservoir level at the crest. Table 4 presents the fundamental frequencies for each type of foundation stiffness under analysis.

Mode 1Foundation<br/>Foundation<br/>Z1Foundation<br/>Z2Foundation<br/>Z3Vibration frequency [Hz]1,762,082,28

Table 3.3. Fundamental frequency - Full reservoir

## 3.3. Peak ground acceleration effect

In order to evaluate the peak ground acceleration, PGA, effect and to assess the performance of the interface models several PGA values were adopted: 0,34 g, 0,68 g and 1,02 g. For the vertical component a 2/3 reduction factor was considered. The acceleration records presented in Fig. 6, were then scaled in order to reproduced the desired acceleration. Only the foundation Z1 was studied.

Three possibilities of damping were considered, Rayleigh damping (MK), stiffness proportional damping only (K), and mass proportional damping only (M). The mass term contribution was only considered in the dam block. The stiffness term contribution was considered at the dam block and at the dam/foundation interface. In all cases the damping constants were defined in order to have a critical damping ratio of 5% at the fundamental response frequency, Table 3.3.

Fig. 7 shows the normal effective stress at the dam/foundation interface, prior to dynamic loading, for the several contact interface models under analysis: zero thickness interface finite element, Parmac2D-FE, simple discrete element approach based on particle/edge contact, Parmac2D-DE and complex discrete element approach based on edge/edge contact, Dec-Dam.

From Fig. 7, it can be verified that for small displacement analysis and under static response the Parmac2D-FE and the Dec-Dam predict similar diagrams. This is related with the fact that on the finite element implementation a two point Lobatto integration rule is adopted, whereas in the edge/edge discrete element interaction two springs at the edge ends are adopted. It can also be verified that the Parmac2D-DE response adopting around 25 particles with 0,1 m radius on each dam finite element edge, Fig. 1b) has a tendency to predict higher/lower stress values being more influenced by the stress singularities known to occur at the dam heel and the dam toe.

The shear sliding displacement predicted at the dam heel for the several PGA values, for the different interface models and for the different damping approaches, is presented in Fig. 8. It can be seen that the different contact models lead to similar behavior for lower PGA values. For higher PGA values it can be verified that there is a difference in the response of the interface models that follow a discrete element approach and the response of the finite element approach. For significant shear displacement values, displacements higher than 15,0 mm which correspond to 3% of the plane element edge length

at the interface, 5 m, it is mandatory to adopt an interface model that allows large displacements.



Figure 7. Normal effective stress distribution at the dam/foundation interface before to seismic analysis



Figure 8. Time history of shear displacement at the dam heel during seismic analysis – Foundation Z1

For PGA values higher than 0,34 g, the damping term proportional to the mass, M, leads to higher shear displacement values when compared with Rayleigh damping, MK, and stiffness proportional damping only, K, for all the different interface models. It would be expectable that higher shear displacement would be related with higher rigid body sliding motions, and that consequently the mass damping term would include erroneous damping values without any physical meaning. The fact that this does not occur may be related to the fact that mass proportional damping provides smaller damping ratios for higher vibration modes. These high frequencies response can then trigger a rocking mechanism that is followed by shear sliding.

Fig. 9 shows the time history of shear sliding and normal effective stress at the dam heel for the Parmac2D-FE interface, and for two values of PGA, 0,34 g and 1.02 g. Only the stiffness damping and the mass proportional damping are showed. It can be verified that sliding increases after a peak in the normal effective stress at the dam heel, and also that mass proportional damping predicts higher peak normal stress values which are obviously followed by a decompression or interface opening at the zone in question. For higher PGA values this effect is amplified which then leads to higher shear sliding displacements when mass proportional damping is adopted.

Note that, the shear sliding predicted at the dam heel is not fully related with rigid body motion of the

dam as it also includes some of the transversal deformation related to the fact that the dam heel is decompressed for the initial static loading, Fig. 7. It is important to control the shear displacements at the dam toe and at the drainage area. Damage at the drainage area due to shear sliding has a positive effect in the increase of the uplift pressure, which will lead to an increase in the total value of shear displacement during the MDE.

From the results here presented one can conclude that at least two different damping analyses should be performed, proportional mass damping only and proportional stiffness damping only. Beforehand it is not possible to exclude one of them as the damping approach that will predict higher shear displacement values. It is also shown that after a certain level of shear sliding it is important to adopt an interface model that allows large displacements, Parmac2D-DE and Dec-Dam.



Figure 9. Time history of shear displacement and normal stress at the dam heel – Foundation Z1

## **3.4.** Foundation stiffness effect

The foundation stiffness effect on the shear sliding response was only evaluated for a PGA value of 0,68 g. All the interface models and all the damping approaches were analyzed. In all cases the damping constants were defined in order to have a critical damping ratio of 5% at the fundamental response frequency, Table 3.3. The same normal and shear stiffnesses were adopted on all cases, Table 3.3.

Fig. 10 shows the normal stress distribution at the dam/foundation interface, prior to the MDE loading, for different foundation stiffness values. As mentioned before the Parmac2D-FE and the Dec-Dam interface model lead to similar static distributions, whereas the interface model adopted in Parmac2D-DE predicts slightly different diagrams given its tendency to follow the singularity values that occur at the dam heel and dam toe.



Figure 10. Normal effective stress distribution at the dam/foundation interface before to seismic analysis – Foundation stiffness

From the static loading distribution it is possible to verify that as the foundation stiffness increases there is a tendency to increase the stress at the dam toe and to decrease the stress value at the dam heel, Fig. 10. The shear sliding displacements at the dam heel for mass proportional damping, stiffness proportional damping and Rayleigh damping for the MDE are presented in Fig. 11.

As shown in Fig. 11 an increase in the foundation stiffness does favour an increase in the shear sliding displacement value. This can be related to the fact that as the foundation stiffness increases the decompression zone at dam/foundation interface increases.



Figura 13. Time history of shear displacement at the dam heel during seismic analysis – Foundation stiffness effect – Mass, Stiffness and Rayleigh damping

## 4. CONCLUSION

The shear sliding analysis of a concrete gravity dam is presented. For lower peak ground acceleration, PGA, values, a finite element joint interface model, Parmac2D-FE, formulated in small displacements, predicts similar shear displacement values than an interface model adopting a discrete element particle/edge methodology, and an interface model adopting a complex edge/edge interaction following a discrete element methodology, both allowing large displacements.

For shear displacements higher that 3% of the plane element edge, it is shown that it is important to adopt a contact interface formulation that handles large displacements at the interfaces, either based on discrete or finite element technology. Note that the interface model adopted in Parmac2D-FE is computationally less demanding than the models based on discrete element methodology given the required geometrical updated.

The results here presented also show that it is important to study separately the effect of mass proportional damping and stiffness proportional damping. For the dam geometry studied and for PGA values lower than 0,34  $\mathbf{g}$ , it is shown that mass proportional damping leads to lower shear sliding displacements as expected because mass proportional damping may include erroneous damping forces when a structure is under rigid body motion.

For PGA values higher than 0,68 g it is shown that the mass proportional damping leads to higher shear sliding values. This can be related to the fact that mass proportional damping does not filter properly high frequencies. These high frequencies are the trigger for a failure mechanism based more on rocking. When applying design guidelines it is important to obtain conservative responses.

It is also shown that foundations with higher stiffness favor an increase of the shear sliding displacement due to the maximum design earthquake, MDE. The nonlinear Mohr-Coulomb model without cohesion adopted at the dam/foundation interface is very sensitive to the normal effective stress installed, and foundations with higher stiffness have the tendency to decrease the stress at the dam heel.

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