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# Concrete Gravity Dams: Earthquake-Induced Behaviour Analysis Based on the Concept of Non-Linear Response Spectra

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# Concrete Gravity Dams: Earthquake-Induced Behaviour Analysis Based on the Concept of Non-Linear Response Spectra

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In the present work, a new simplified analysis approach for the earthquake-induced behaviour of a concrete gravity dam is proposed. The proposed technique consists of the following steps: 1) A set of possible failure mechanisms for the particular dam is defined. Then, the dam structure is approximated by a SDOF system according to the sliding block method. 2) With respect to this simple system, correction of the input accelerogram is performed. 3) A non-linear incremental static pushover analysis if performed to obtain the integral SDOF system parameters. 4) By means of well established algorithms, the non-linear response spectra of the system are computed. Finaly, key system response parameters can be obtained by means of these response spectra. The described computational approach is illustrated by a numerical example solved with the parameters of a newly designed Bulgarian dam. The important conclusion is drawn that in general, the sliding block method should not be applied for concrete gravity dams in such form. In some cases its implementation can be even dangerous since some failure mechanisms are not identified by this method.

Keywords: seismic safety, concrete gravity dam, sliding block method, simplified

## 1 Introduction and problem statement

It is known that concrete dams belong to a specific class of structures whose response to strong earthquakes is quite different from the response of buildings. The purpose of the modern seismic codes is to develop reliable methods for the dams design. On the other hand, methods should be simple enough to allow for an easy application for the design purposes. It has been over the last decades an aim of the researchers to develop methods both safe and economical enough to enable designing large dams capable of resisting strong earthquakes.

The energy dissipation mechanisms for dams are quite different from the mechanisms related to other types of structures. The following important points should be taken into account concerning the seismic resistant design of dams:

- Dams are not ductile structures. Inelastic response is mainly due to cracking of the concrete continuum. The most essential reason for energy dissipation is the development and propagation of cracks. For 2D models, opening and sliding modes of crack opening are typical.

- Because of the low tension resistance of concrete, the dam body can be damaged even under a relatively small lateral loading. Increasing the lateral load leads to progressive crack propagation and brittle failure.

- Smeared cracks are found to be distributed in strip locations (so called blunt cracks). Uncracked concrete remains practically undeformed. Thus, the sources of inelastic deformations are mainly the crack areas. Inelastic strains due to cracks are dominating on the total strains being obtained as a sum of inelastic and elastic strains. It is worth considering the idea of approximate analysis assuming the uncracked areas as absolutely rigid and cracked area as the unique source of deformations – an approach widely used in the field of dam design.

- In general, three types of failure mechanisms can be identified: a) rocking mechanism caused by crack opening displacements; b) sliding mechanism being related to shear mode of failure; c) mixed mechanism incorporating both previous mechanisms. The latter mechanism is very complex and still needs more theoretical and experimental research.

Recently, several guidelines for seismic safety evaluation of concrete dams have been developed, for example Papazchev et.al (1996), Workshop Report (1998). A typical feature of these guidelines is to expect a generally linear elastic response under moderate intensity earthquake, and controlled damage under the maximum credible earthquake without endangering the ability of the dam to retain the full reservoir. Accordingly, all modern guidelines recommend the application of progressive approach starting with simple linear elastic models of the dam-foundation-reservoir system, followed by non-linear models accounting for concrete cracking if the design earthquake-induced loads exceed the elastic strength capacity of the dam.

A problem of particular importance for the dynamic analysis of a concrete gravity dam is the non-linear behaviour of the mass concrete under dynamic (sign-alternating) loading including the development and propagation of cracks. Cracks penetrating deep inside the dam body may considerably decrease the structural resistance, and thus may decrease the dam safety. It is expected that under normal load conditions (gravity, hydrostatic pressure and ambient temperature) concrete gravity dams experience no cracking of structural significance. In the case of the maximum design earthquake (MDE) and especially of the most severe seismic excitation possible at the given site – the maximum credible earthquake (MCE), potential cracking has to be taken into account.

The complete procedure for seismic risk assessment of a concrete dam is a complex process but its performance is already regarded as necessary for both existing and newly designed large dams. At the same time with the continuous improvement of the seismic risk analysis procedure and with the growing experience of its application, attempts always exist for developing of simplified approaches for fast approximate global safety assessment of a given dam.

In the present work, a simplified approach is proposed for overall approximate assessment of the resistance of concrete gravity dams to earthquake excitation presented here in the form of a design acceleration time history. It is based on the well known and established also in the field of Dam Engineering sliding block method firstly introduced by Newmark for slope stability analyses. At present, this method is widely used as a simple analytical technique in stability investigations predominantly of embankment dams when some parts of the dam body can be successfully modelled as rigid blocks with appropriate assumptions for the boundary conditions [Anandarajah, Sakamoto et.al].

In the field of concrete gravity dams, not only simplified but also more rigorous techniques deal with their seismic sliding stability. A numerical method developed by Chavez and Fenves (1995) has been implemented in a computer program (EAGD-SLIDE) to evaluate the earthquake response of concrete gravity dams, including sliding at the interface between the base of the dam and the foundation rock surface. The earthquake response of the dam is influenced by the interaction between the dam and the compressible water in the reservoir, by the interaction between the dam and foundation rock, and by the dam-foundation rock interface properties. However, the dam monolith is modelled by a finite element discretization with linear, elastic, orthotropic material behaviour. Base sliding is the only source of nonlinearity in the system.

A simplified method for the safety assessment of concrete dams subjected to the maximum credible earthquake (MCE) is proposed by Malla and Wieland (2003) based on the dynamic stability (rocking and sliding motion) of concrete blocks separated by cracks and/or lift joints. It is based on the above mentioned sliding block model. It is assumed that these rigid blocks are formed predominantly by the planes of contraction and lift joints where the crack/joint deformations also prevent the surrounding concrete from further cracking. Thus, it is expected that only few cracks will be caused in concrete dams due to severe ground shaking. Observed earthquake damages of the only few large concrete dams that have suffered significant earthquake damage so far are pointed to support this statement.

## 2 Short description of the proposed method

The present paper deals with the sliding mode of failure which can occur at most of the sections of a concrete dam, especially at construction joints. In the following, a short outline of the method is presented. It is illustrated by implementation to the newly designed 104.20 m Ardino concrete gravity dam.

The first step consists of defining a set of possible failure mechanisms for the particular dam. Then, the dam structure is approximated by a single-degree-of-freedom (SDOF) system according to the sliding block method. These failure mechanisms and rigid blocks, respectively, have to be determined as type and shape on the basis of practical experience with similar dams, engineering judgement, experimental investigations, and the results of conventional static and dynamic analyses for other structures of this type. The general assumption here is that the structural behaviour of the dam can be described by a single parameter when having reached a limit state. Since this state is difficult to be defined for concrete dams, it is suggested that some of the collapse mechanisms, shown in Figure 1, takes place. Each mechanism is related to some failure mode, for example such as crack opening or sliding typical for such type of structures.

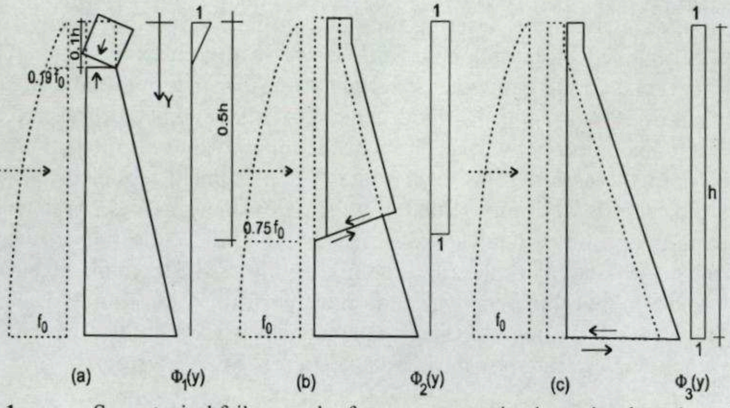


**Figure 1:** Some typical failure modes for concrete gravity dam related to: a) crack opening mode of fracture *Bhattacharjee and Leger (1992, 1993), Ghrib and Tinawi (1995)*; b) shear mode of fracture *Bhattacharjee and Leger (1992)*; and c) sliding between the dam and the foundation *Bury and Kreuzer (1985), Chavez and Fenves (1995)*

The governing system of equations written for multi-degree of freedom system is transformed into a single equation of the dynamic equilibrium of the SDOF system here:

$$m\ddot{v} + c\dot{v} + F(v) = -m\ddot{v}_g \cos\alpha - mg\sin\alpha \tag{1}$$

where: m is the mass of the sliding block, v is the displacement along the sliding surface, $\ddot{v}_g$ is the input accelerogram, and $\alpha$ is the slope angle of the sliding plane to the horizon.

In the second step of the analysis, correction (shift) of the input accelerogram is performed according to Eq. (1) with respect to the formulated simple SDOF system, Figure 1. In this particular case, the following further simplifying assumption is made: only the term ($g\sin\alpha$) has been added to the original accelerogram. On one hand, this retains the character of the model as SDOF system. On the other hand, retaining the original accelerogram instead of the necessary reduction of it by $\cos\alpha$ can be regarded as taking into account in such form the influence of the additionally arising excitation component normal to the defined sliding plane.

The third analysis step consists of performing an incremental static pushover analysis with a non-linear finite element model of the dam. The conditions of this pushover analysis are described in detail in Kisliakov and Petkov (2002). Here, the obtained structural behaviour curve is presented in Figure 2. The incremental load on the dam consists of pressure (constant over the dam height) steps (each of 10 kPa), and the displacement values have been measured at the dam crest. This curve delivers in the most adequate way the integral structural parameters of the assumed bi-linear force – displacement model of the SDOF system and is very well approximated by such bi-linear model. An interesting result from the pushover analysis is further that the failure mechanism activated by the pushover is actually the first one (a) on Figure 1. The main advantage of this approach is that this pushover analysis is performed only once. Then, with the already obtained integral system parameters, different failure modes of the system can be investigated by introducing different SDOF systems.

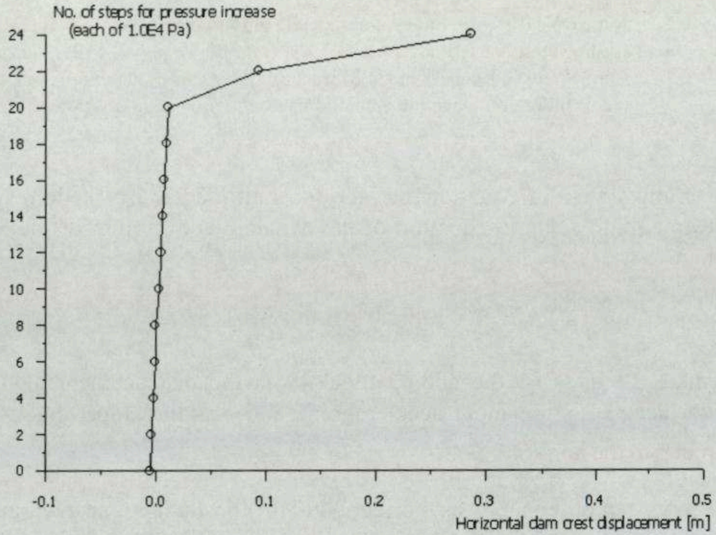


**Figure 2:** Load – displacement curve as result from the pushover analysis

The fourth step is then to perform inelastic time history analysis of the SDOF system. This step is performed by means of already well established algorithms for constructing non-linear response spectra and for inelastic time history analysis of a SDOF system, in this case – the ones incorporated in the computer program Bispec [Hachem (1999-2004)]. The input ground excitation consists of two generated accelerograms for the horizontal ground motion component at the dam site as a result of the carried out seismological site investigation and corresponding to MDE and MCE impact, respectively, Figure 3.

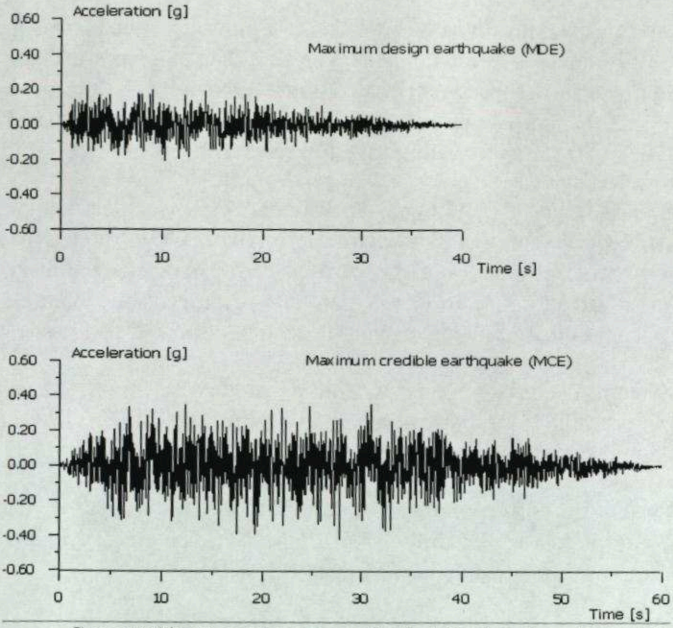


**Figure 3:** Generated input accelerograms of both defined earthquake models for the dam site – MDE and MCE

## 3 Analysis of the results and conclusions

For testing the validity of the proposed simplified safety assessment approach, comparative studies were performed with a sophisticated computer code allowing fully dynamic fracture analysis of the considered cases. The following results were obtained during these carried out comparative analyses:

1. Full non-linear dynamic fracture analysis with a 2D finite element program incorporating the smeared cracks approach for mass concrete structures (FRAC_DAM) – some results in this connection have been already reported in Kisliakov and Petkov (2002):

The MDE does not lead to crack development, i.e. the dynamic behaviour of the dam structure remains in the linear elastic range

The MCE leads to the break of the crest part after about 6 sec, i.e. the first failure mode (a) on Figure 1 is realized. However, the overall stability of the dam is yet retained formally during the whole excitation period although a crack develops and propagates up to 40% of the base interface plane. This fracturing process is fully developed already after the first 30 sec of the excitation. This can be regarded as a subsequent realization of the third failure mode (c) on Figure 1 since thus no more bearing capacity of the dam is available with respect to the hydrostatic pressure of the full reservoir.

2. Non-linear time history and linear / non-linear response spectrum analysis of the assumed equivalent SDOF system. Here, two SDOF models have been investigated according to both identified and above mentioned failure mechanisms – breaking of the crest part and sliding at the dam-foundation interface. The system parameters of both these models are as follows:

- System A corresponding to failure mode (a): mass: $151.2*10^3$ kg, period: 0.8367 s, stiffness: 8.53 MN/m, positive yield: 2.5 MN, damping 5%, positive hardening stiffness ratio: 0.125;
- System B: corresponding to failure mode (c): mass: $10.08*10^6$ kg, period: 2.415 s, stiffness: 68.26 MN/m, positive yield: 20 MN, damping 5%, positive hardening stiffness ratio: 0.212.

Three sliding surfaces for the dam crest block were considered – at angles $0^0$, $15^0$, and $30^0$ to the horizon, and one horizontal plane at the dam base.

The MDE has not activated the non-linear range of SDOF system behaviour for any of both systems considered.

The MCE activates the non-linear range of the SDOF system behaviour, i.e. the positive yield has been exceeded. However, the most important point is that this phenomenon was observed only for the case of System B, i.e. for sliding at the dam foundation plane.

The spectral acceleration values read for System B from the constructed non-linear response spectra for the input MDE and MCE accelerograms, are 46% and 40% of the corresponding elastic response spectrum values (0.187g and 0.347g), respectively. These values correspond quite well even with some safety margin to the reduction coefficient of 0.5 considering inelastic actual structural behaviour in the Bulgarian Codes, for example. However, any statement about systematic dependence in this connection would need a rigorous proof based on extensive research results.

The qualitative comparison between the results from the performed non-linear analyses of the SDOF models and the sophisticated dynamic fracture finite element model clearly shows that the implementation of this approach may not be recommended. On one hand, the sliding block method is a well established method often used as a simplified analytical approach in Dam Engineering. For the pure sliding failure mode (c) on Figure 1 it leads to realistic results and values for both MDE and MCE scenarios. On the other hand, the definitely initially to develop failure mode (a) – breaking of the dam crest – was not identified by this technique at all.

Thus, as a final conclusion, we would not recommend the use of the sliding block method as a simplified approach for the seismic safety assessment of concrete gravity dams, unless investigations are available about the applicability of the method with respect to the physics of each of the identified basic failure mechanisms, Figure 1, supported by a representative parameter study of the problem.

The range and the complicated character of such investigation formulate it as a problem for future research.

## 4 Acknowledgement

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