Seismic Deformation Analysis of Embankment Dams: A Comparison between Simplified and Non-Linear Numerical Methods

Analyse de la déformation d'une digue suite à un évènement sismique: comparaison entre la méthode simplifiée et l'analyse numérique non linéaire

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ABSTRACT: The principal seismic concern for design of an embankment dam is the development of displacement patterns that could lead to failure and uncontrolled release of water and/or tailings in the event of a significant earthquake. The prediction of such movements is dependent on the accuracy and complexity of the analysis procedure used. Despite their limitations, equivalent linear approaches coupled with simplified deformation analysis methods (such as Newmark) have been widely used to model seismic non-linear behaviour of soils. The most commonly used computer code for this purpose is SHAKE which works in a frequency domain and has been shown by previous studies to not necessarily predict a conservative estimate of dynamic response and deformations. In order to illustrate how variations in the adopted analysis method could affect the computed deformations a number of case studies are discussed and the predicted earthquake induced deformations by various methods ranging from empirical and Newmark based schemes to nonlinear numerical analyses, are compared.

RÉSUMÉ : La source principale d'inquiétude concernant la conception d'une digue est le développement d'un mode de déformation pouvant aboutir à une rupture ainsi qu'un déversement non contrôlé d'eau et/ou de boue à la suite d'un épisode sismique important. La prédiction de tels mouvements dépend de la précision et de la procédure d'analyse utilisée. Malgré leurs limitations, les analyses linéaires équivalentes couplées à l'analyse des déformations simplifiés tel que développée par Newmark ont étés largement utilisées afin de modéliser la déformation non linéaire des sols lors d'un évènement sismique. SHAKE est le logiciel informatique le plus utilisé qui opère dans le domaine des fréquences. Il a été démontré lors d'études antérieures qu'il n'a pas toujours permis une estimation prudente des réponses dynamiques et des déformations. Un certain nombre de cas d'études sont discutés afin de montrer comment les variations liées aux différentes méthodes d'analyses peuvent affecter la prédiction des déformations. Une comparaison est réalisée entre les différentes méthodes présentées dans cette étude.

KEYWORDS: Deformation analysis, Embankment dams, FLAC, Numerical simulation, SHAKE, and Simplified methods.

1 INTRODUCTION

Evaluation of the effects of earthquakes on embankment dams is one of the important design issues. There are a large number of cases where earthquakes have resulted in sliding and lateral spreading of embankments and settlement of their crests. The magnitude of the crest settlement of an embankment dam must be less than the freeboard of the dam to prevent overtopping and breach.

Different approaches have been proposed to predict the crest settlement of earth dams under earthquake loading, ranging from simplified Newmark (1965) based methods to complex stress–deformation analyses. Recent studies show that the simplified methods do not always give a conservative estimate of deformation of dams under earthquake loading (e.g. Rathje and Bray 2000, Ghahreman-Nejad et al. 2011, Meehan and Vahedifard 2013). Kan et al. (2016) proposed a framework to assess the reliability of simplified methods in calculation of the seismic deformation of dams based on the height and type of the embankment, and the seismic activity of the site.

In this paper, a number of case studies are discussed and the predicted earthquake induced deformations by various methods are compared. Consideration is given to the Newmark based simplified methods and their performance compared with the more complex stress–deformation analyses.

2 DESCRIPTION OF THE PROBLEM

Three rockfill embankment dams with different applications are selected here to study their seismic behviour, including:-

- **Dam1:** A Tailings/Water Dam, 226 m high (GFRD)
- **Dam2:** A Tailings Dam, 90 m high (ECRD)
- **Dam3:** A Water Storage Dam, 84 m high (ACRD)

All of the above dams are currently operational. A short description of each dam is provided in the ensuing sections.

2.1 *Dam1*

Dam1, a Geomembrane Faced Rockfill Dam (GFRD), is a tailings/water retaining dam which is to be raised in stages using the downstream construction technique to reach a maximum height of 226m. The starter embankment (Stage 1 analysed here) has a height of approximately 96m.

Figure 1 shows a typical cross section of the dam as a simplified FLAC (Itasca, 2011) model. Note that the tailings level was considered to be at approximately half of the embankment height with water level at full supply level (5m below crest level). The embankment upstream and downstream slopes are 1.7:1 (H:V) and 1.75:1 (H:V) respectively. The embankment has a crest width of 15m.

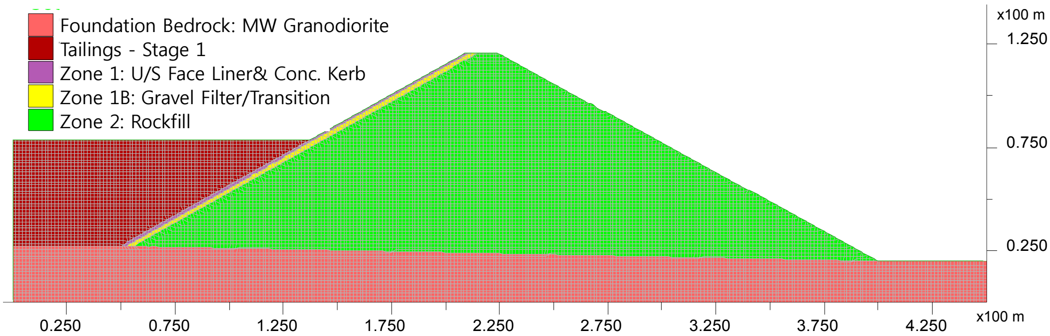

Figure 1. Typical cross section of Dam1 presented as a simplified FLAC mesh.

2.2 *Dam2*

Dam2, an Earth Core Rockfill Dam (ECRD), is also a tailings retaining structure, which is to be raised in stages using the downstream construction method to reach a final height of approximately 90m. In its first stage of development (analyzed here) is 63 m high with a crest width of around 40m. The final crest width (Stage 3) will be 10m. The upstream and downstream slopes are 1.5:1 (H:V) and 1.7:1 (H:V), respectively. The embankment consists of an inclined upstream clayey core, filters and transition zones on either side of the core, transitioning to rockfill shoulders.

2.3 *Dam3*

Dam3, an 84 m high Asphaltic Concrete Core Rockfill Dam (ACRD), is a water retaining structure with a storage capacity of 34 Mm^3 at the full supply level (4 m below the crest). The dam is founded on volcanic bedrock formed by inter-bedded zones of andesite and tuff. The upstream and downstream slopes are 1.75:1 (H:V) and overall 1.8:1 (H:V), respectively. The embankment consists of a 0.6m wide central asphalt core, transition zones on either side of the core, free draining rockfill and coarse rockfill.

3 SEISMIC DEFORMATION ANALYSIS

3.1 *Selection of Design Earthquakes*

All three dams are located in regions of high seismicity and consequently the seismic action was considered to be the governing load case. A suite of appropriate earthquake time histories were produced based on the design uniform hazard spectrum (UHS) for each site. The acceleration time histories were spectrally matched or scaled to the sites' UHS for Dam1 and Dam2/Dam3, respectively. A summary of main characteristics of earthquake motions for all projects is given in Table 1. The time history of Peru Coast (1974) earthquake, Record #2, is presented in Figure 2.

Table 1. Main characteristics of earthquake motions for all projects.

Project		Dam1	Dam2	Dam3
PGA (g)		0.74g	0.64g	0.8g
Magnitude		7.6-8.2	6.7-7.1	6.9-7.5
Selected Time Histories Matched and/or Scaled	#1	Peru Coast, IGP (1974)	Loma Prieta (1989)	Loma Prieta (1989)
	#2	Iquique Chusmiza, Chile (2014)	Cape Mendocino (1992)	Cape Mendocino (1992)
	#3	-	Northridge (1994)	Nahanni (1985)

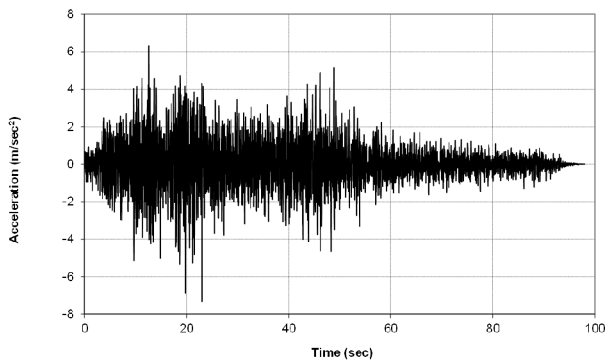

Figure 2. Time history of Peru Coast (1974) earthquake, recorded at IGP Station, applied in numerical simulations of Dam1.

3.2 *Empirical Methods*

A number of empirical and semi-empirical models have been proposed in the literature in order to simplify the calculation of seismic deformation of embankments. Two of the most relied on methods in practice are those proposed by Bray and Travasarou (2007), and Swaisgood (2003). In the former model the seismic displacement can be calculated based on the sliding mass yield acceleration (k_y) and its fundamental period (T_s), and the ground motion's spectral acceleration at a degraded period equal to 1.5T_s. Swaisgood's model (2003) is based on analysis of the observed crest settlements of 69 embankment dams during past earthquakes and provides a mathematical relationship to predict the extent of earthquake induced settlement. This relationship only requires the earthquake magnitude and peak ground acceleration (PGA).

The above models were used to predict the displacement of all three dams under the design earthquake loading. The adopted parameters and corresponding deformations are listed in Table 2.

Table 2. Predicted Maximum Crest Settlement from Bray and Travasarou (2007), and Swaisgood (2003) empirical models

Parameter	Dam1	Dam2	Dam3
y/h (sliding mass depth from crest/ emb. height)	2/3	1	1
T_s (sec)	0.23	0.411	0.392
k_y (g)	0.420	0.30	0.23
S_a (1.5 T_s) (g)	1.1	0.695	1.322
PGA (g)	0.74	0.64	0.8
M_w	7.6-8.2	6.7-7.1	6.9-7.5
Displacement(m) Bray&Trav.	**0.06-0.08**	**0.04-0.05**	**0.33-0.39**
Settlement(m) Swaisgood	**2.19-3.08**	**0.47-0.59**	**1.75-2.07**

3.3 *Simplified Newmark-based Methods*

Newmark (1965) proposed a method for evaluation of permanent crest settlement of embankments subjected to earthquake loading. This method is based on sliding of soil mass along an inclined failure surface due to the inertia forces. Sliding would be initiated when the inertia forces exceed the resistance of the shearing sliding mass and would stop once the inertia forces are reversed. Makdisi and Seed (1978) extended the Newmark method by including the dynamic response of the embankment in the analysis process.

A series of seismic deformation analyses were conducted for Dam3 based on the Makdisi and Seed (1978) approach using SHAKE (2003) and the design earthquake time history. The analyses for Dam 1 and Dam2 are currently underway but could not be completed in time for presentation in this paper.

A series of failure surfaces were considered for Dam3 with the exit point of the failure surface gradually increasing in depth, commencing in the upper quarter of the upstream embankment face and gradually increasing in depth until the toe of the upstream embankment slope was reached.

The deformation predicted for Dam3 using this method was estimated to be between 0.24m and 0.49m for slip surfaces with a depth/embankment height ratio (y/h) of between 1 and 0.2, respectively.

3.4 *Elasto-plastic Dynamic Analysis*

For numerical simulation of each dam, a two dimensional, plane strain, dynamic deformation analysis of the embankment dam was performed using the software FLAC 2D (Itasca, 2011) and its built-in elastic-plastic Mohr-Coulomb model. In the analyses, the embankment model (mesh) is generated and material properties are assigned so that the initial state of stress within the embankment and its foundation can be established. Then dynamic parameters are assigned and seismic calculations are performed. A number of functions were developed using FLAC's built-in programming language, FISH, in order to model variations in properties of the embankment materials with depth/confining stress (i.e. friction angle, undrained shear strength and shear modulus, etc.).

The earthquake motions were de-convoluted for use in the FLAC model as "within ground" motion. For each bedrock motion, a Fourier amplitude spectrum was derived in order to identify the dominant range of frequencies. It was found that most of the energy in the input motions were below certain frequencies and hence seismic records were filtered to remove frequencies greater than these frequencies. After filtering, the waves were corrected for a base line drift and applied as a stress boundary in order to establish quiet boundary conditions along the same boundary as the dynamic input. The results of the analysis for the Peru Coast IGP record are graphically presented in Figures 3 to 5.

A summary of all numerical simulations for different dams and earthquake records are summarized in Table 3.

It should be noted that the maximum relative displacement in Table 3 is provided for the mid crest. Hence, the predicted horizontal or vertical (settlement) displacements might be different from those closer to the upstream or downstream edges. This is particularly important for Dam2 which has a relatively wide crest.

Table 3. Summary of dam crest settlements (m) from FLAC analyses.

Dam	EQ Record	Predicted Peak Crest Acceleration (g)	Maximum Deformation (m) @ Location	Mid Crest Maximum Relative[3] Displacement (m)	
				Horizontal[1]	Vertical[2]
Dam1	Peru Coast	0.78	4.43 @D/S upper quartile	3.2	-2.12
	Iquique	0.57	2.76 @D/S upper quartile	1.56	-1.1
Dam2	Cape Mendocino	0.47	1.16 @U/S upper third	0.05	-0.02
	Loma Prieta	0.77	1.41 @U/S upper third	0.13	-0.16
	Northridge	0.73	1.66 @U/S upper third	0.48	-0.19
Dam3	Loma Prieta	1.1	1.39 @U/S upper quartile	-0.85	-1.44
	Nahanni	1.4	-	-1.0	-1.47
	Cape Mendocino	1.7	-	-0.6	-0.85

U/S and D/S are upstream slope and downstream slope respectively.
[1] Positive horizontal displacement indicates movement in the downstream direction and vice versa.
[2] Negative vertical displacement indicates settlement and vice versa.
[3] Relative to the top of rock foundation.

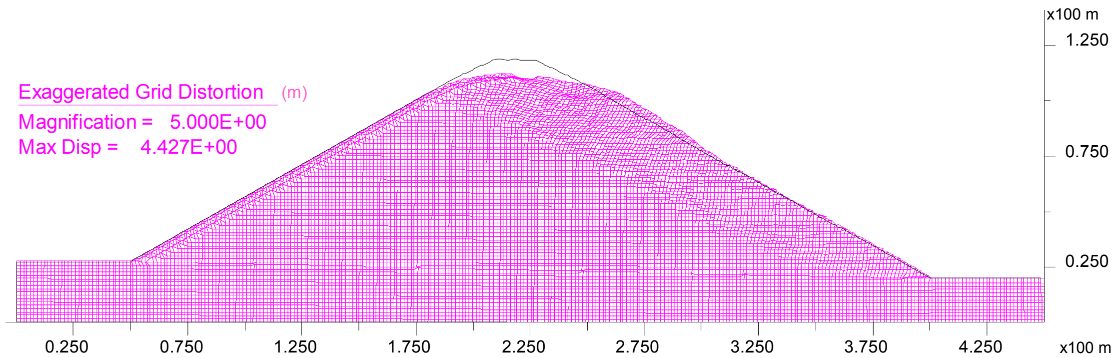

Figure 3. Deformation Pattern Resulted from Dynamic Analysis of Dam1 under Peru Coast Earthquake.

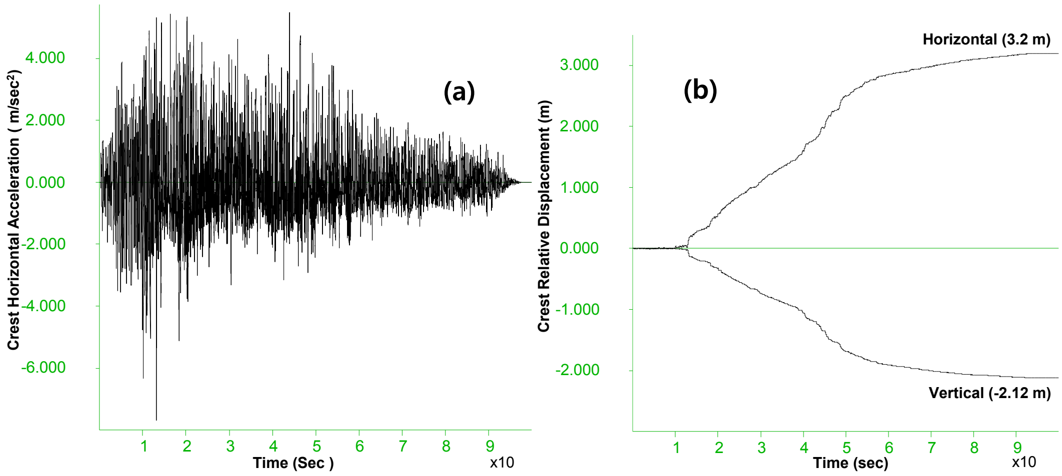

Figure 4. Crest acceleration (a), and variation of mid crest relative vertical and horizontal displacements (b) with time under Peru Coast Earthquake.

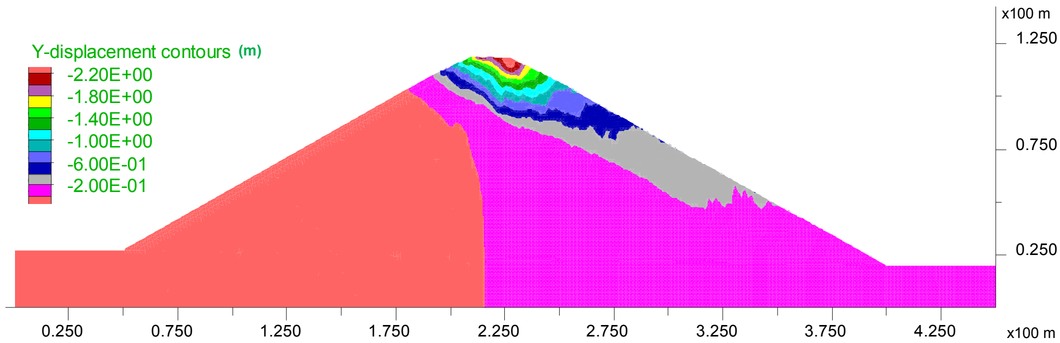

Figure 5. Vertical displacement contours for Dam1 under Peru Coast Earthquake.

4 DISCUSSIONS AND CONCLUSION

The results of these analyses clearly indicate that the semi-empirical methods based on Bray and Travasarou (2007) or Makdisi and Seed (1978) may not necessarily result in conservative estimates of the deformation for rockfill embankment dams as is currently expected in the profession. Nonetheless, Swaisgood's empirical formulation has produced an upper boundary to the predicted deformations for all cases presented in this paper. The magnitude of the crest settlements predicted by the Swaigood's empirical method and dynamic elasto-plastic modelling are between approximately 2 and 30 times those predicted by either Makdisi and Seed (1978) or Bray and Travasarou (2007) methods. This is in agreement with the findings of some previous studies such as Rathje and Bray 2000, Ghahreman-Nejad et al. 2011, and Meehan and Vahedifard 2013. The concept of a unique failure plane, as adopted by some of the simplified methods, may be misleading, as deformation would be more likely to be characterised by the deformation pattern predicted using a non-linear elasto-plastic model and numerical modelling.

The simplified methods are currently proposed in a number of guidelines around the world (e.g. ANCOLD) as the first step screening tool before initiating a more sophisticated numerical analysis. Based on the results of analysis presented in the preceding sections and in the authors' opinion the simplified empirical methods may not necessarily trigger the need for the next stage of analysis and hence may impose significant risks if adequate measures are not employed in design of embankment dams, particularly, in the regions with moderate to high seismicity.

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