Numerical Analysis of El-Agrem Concrete Face Rockfill Dam

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Abstract. For concrete face rockfill dams, the behaviour of the concrete slab which provides the watertightness of the dam is a major concern. The deformation of the dam embankment during construction and operation leads to concrete slab deformation. This deformation should be compatible with the structural integrity of the concrete slab. This paper presents the numerical analysis of El-Agrem concrete face rockfill dam during reservoir filling using the finite element code Plaxis 2D-2010. Two models are used in this study, the Mohr Coulomb model and the Hardening Soil model to emphasize the impact of constitutive model on the dam deformation. The material stiffness parameters used in constitutive models were calibrated based on the available measured crest settlement. The analysis results indicate that the face slab deformation can be numerically predicted based on monitoring data at the dam crest and the Hardening Soil model gives a maximum slab deformation less than that obtained using Mohr Coulomb model.

Keywords: concrete face rockfill dam, stress-strain analysis, concrete slab deformation, rockfill behaviour.

1. Introduction

In recent years, the construction of concrete face rockfill dams has known a considerable interest in many parts of the world. The speed of construction, steep slopes, stabilizing effect of water load and resistance to seismic loading are some advantages of this type of dams compared to other types of embankment dams. However, the problems we may encounter with concrete face rockfill dams are the vulnerability of perimetric joints, deformations, cracks and aging of the concrete face slab.

The performance of a concrete face rockfill dam depends mainly on the performance of its concrete face slab. Won & Kim (2008) studied the post-construction deformation of concrete face rockfill dams based on data published in the literature and showed that the crest and slab deformations depend mainly on the intact strength of the rockfill. They noticed that about 80% of the total long-term (after at least 10 years) deformation normal to the face slab occurs during the first filling and it is smaller than 0.5% of the dam height.

For concrete face rockfill dams, the most significant parameters to monitor are the deformation of the crest, the deformation of the concrete slab and the quantity of seepage through it. Crest deformation is usually obtained using surveying methods whereas the deformation of the concrete slab can be monitored by instruments such as inclinometers or settlement gauges. These instruments placed underwater are difficult to check or maintain and may fail to work. Therefore, in many dams, slab deformation measurements are not available such as Dix river dam, Kangaroo Creek dam, Little Para dam, Mangrove Creek dam, Serpentine dam, Wishon dam, etc. (Hunter *et al.*, 2003).

The objective of this study is the prediction of the face slab deformation of El-Agrem concrete face rockfill dam based on available monitoring data at the dam crest using the finite element code Plaxis 2D-2010. The analyses were performed using two constitutive models, linear elastic perfectly plastic model (Mohr Coulomb model) and non-linear stress-strain constitutive model (Hardening Soil model).

2. Constitutive Models

The Mohr Coulomb model is a linear elastic perfectly plastic model with Mohr-Coulomb failure criterion.The model requires five input parameters namely Young's modulus (E), Poission's ratio (ν), cohesion (c), friction angle (ϕ) and dilatancy angle (ψ).

The hardening soil model is an advanced model applied for all types of soils and is based on shear and compression hardening. This model supersedes the hyperbolic model of Duncan & Chang (1970) by using the theory of plasticity rather than the theory of elasticity and by including soil dilatancy and yield cap (Schanz *et al.*, 1999). In contrast to Mohr-Coulomb model, the yield surface of a hardening plasticity model is not fixed in principal stress space but it can expand due to plastic straining (Brinkgreve *et al.*, 2010).

The hyperbolic relationships for standard drained triaxial tests tend to yield curves, which can be described by:

$$-\varepsilon_1 = \frac{1}{E_i}\frac{q}{1-q/q_a} \quad \text{for } q < q_f \tag{1}$$

$$R_f = \frac{q_f}{q_a} \tag{2}$$

where E_i is the initial tangent Young's modulus, q_a is the asymptotic value of the shear strength, q is deviatoric stress, ε_1 is vertical strain, q_f is the ultimate deviatoric stress derived from the Mohr-Coulomb failure criterion and R_f is the failure ratio.

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E_i is related to E_{50} by:

$$E_i = \frac{2E_{50}}{2-R_f} \tag{3}$$

The confining stress dependent stiffness modulus for primary loading (E_{50}), for unloading and reloading (E_{ur}) and for oedometer stress-strain conditions (E_{oed}) are given by the following equations:

$$E_{50} = E_{50}^{ref}\left(\frac{c\cos\varphi - \sigma'_3 \sin\varphi}{c\cos\varphi + p^{ref}\sin\varphi}\right)^m \tag{4}$$

$$E_{ur} = E_{ur}^{ref}\left(\frac{c\cos\varphi - \sigma'_3 \sin\varphi}{c\cos\varphi + p^{ref}\sin\varphi}\right)^m \tag{5}$$

$$E_{oed} = E_{oed}^{ref}\left(\frac{c\cos\varphi - \frac{\sigma'_3}{K_0^{nc}}\sin\varphi}{c\cos\varphi + p^{ref}\sin\varphi}\right)^m \tag{6}$$

where E_{50}^{ref}, E_{ur}^{ref}, E_{oed}^{ref} are reference stiffness modulus corresponding to the reference confining pressure p^{ref}, m is the amount of stress dependency, K_0^{nc} is the K_0 value for normal consolidation, c and φ are strength parameters.

The triaxial modulus controls the shear yield surface and the oedometer modulus controls the cap yield surface.

3. Calibration Procedure of Rockfill Material Parameters

The finite element method is a useful tool, widely used in recent years to analyze the concrete face rockfill dams behaviour, but the accuracy of the finite element analysis depends on the selection of appropriate constitutive models and material parameters. The selection of the strength and stiffness parameters for dam materials needs much experience and engineering judgment, especially for rockfill materials. The rockfill testing data are usually limited or of low quality due to the large size of the rockfill elements which is difficult to test using conventional experimental equipments. Therefore, the parameters of rockfill materials used in the FEM analysis are often adopted from published data for similar materials.

The material stiffness modulus is essential for dam deformation prediction and the range of this parameter is very large, which make its estimation very approximate compared to other model parameters. In this study, the stiffness parameters of the rockfill materials are back analyzed based on the available crest monitoring data as follow:

- The Young's modulus (E) of the MC model is calibrated by minimizing the deviation between the field observation and numerically computed results. The stiffness modulus obtained corresponds to the secant Young's modulus E_{50}. Loupasakis *et al.* (2009) evaluated the Mohr Coulomb model for rockfill shell materials taken from an embankment dam and concluded that, the model theoretical curves that best match the triaxial experimental curves, are those having the secant stiffness modulus values $E_{50.}$
- The stiffness modulus E_{50}^{ref} used in HS model is then evaluated based on the stiffness modulus E_{50} obtained from MC model using the Eq. 4. This stiffness modulus can also be estimated using the expression:

$$E_{50}^{ref} = 50(2-R_f)K \tag{7}$$

This expression was derived from the combination of Eq. 3 and Janbu (1963) equation $E_i = KP_a\left(\sigma'_3/P_a\right)^n$, where K is the modulus number, P_a is the atmospheric pressure, σ'_3 is the minor principal stress and n is the modulus exponent.

- E_{ur}^{ref} is set equal to 3 times E_{50}^{ref} as suggested in Plaxis manual and the oedometer stiffness parameter E_{oed}^{ref} is then optimized by back analysis.

4. El-Agrem Dam Characteristics

The El-Agrem concrete face rockfill dam as shown in Fig. 1a, is located 15 km south - east of Jijel city (Algeria). The dam reservoir can store approximately 35 million cubic meters of water and provides drinking water to the surrounding region.

The dam is 64 m high and 395 m long with a crest width of 10 m. The embankment volume is 1.6 x 10^6 m^3 and consists of two zones of granitic rockfill materials as shown in Fig. 1b. Zone E1 comprises the majority of the embankment section with a maximum particle size of 0.6 m and compacted in layers of 0.8 m thick. Zone E2 is a 1 m thick transition zone consists of smaller-size rock, provided between the concrete face slab and the rockfill embankment.

The rockfill is compacted to a high density in order to minimize deformations and concrete slab cracks and leakage. The upstream and downstream slopes of the dam were established at 1.7H:1V.

The concrete slab on the upstream side, provides the watertightness of the dam above the ground together with the perimetric joints and the plinth. The concrete slab has a total area of 31000 m^2 with a variable thickness of 0.5 m at the base and 0.35 m at the top of the dam. The dam foundation consists of granite gneiss rock on the right bank and of marly rock on the left bank.

The dam embankment was almost completed in October 1993 (the embankment level reached was 140.5 m), after that the construction activities were stopped for economic raisons. Construction activities resumed during the year 1999 and the embankment was raised to its final level 143 m. The face slab was concreted during the year 2000, about 7 years after the end of embankment construction.

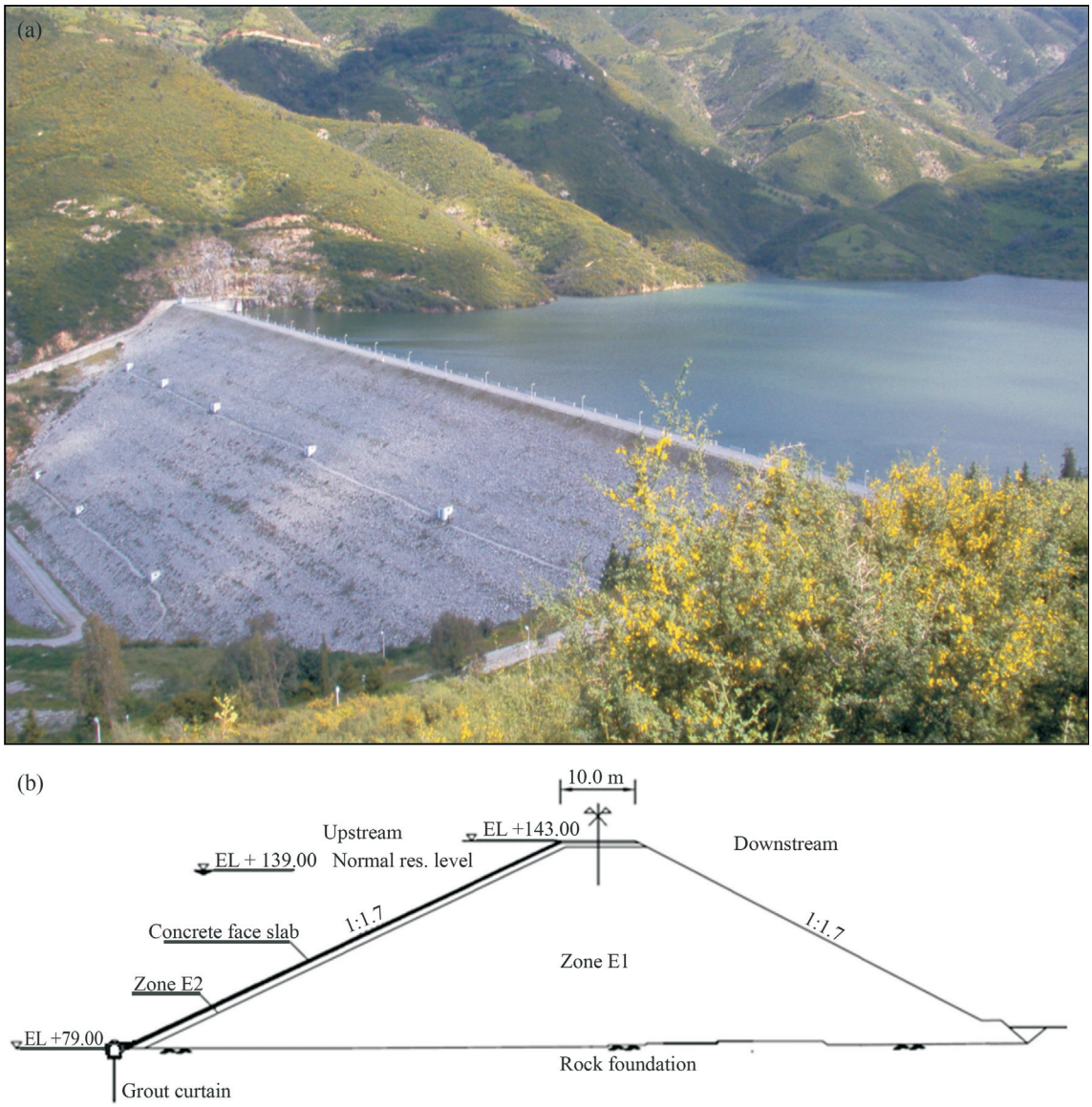

Figure 1 - a: General view of El-Agrem concrete face rockfill dam. b: Typical cross-section of El-Agrem concrete face rockfill dam.

Reservoir filling was started in 17 February 2001 (about three months after the completion of the face slab in November 2000) and the water level reached 99.21 m in 26 January 2002 and stabilized at this level until October 2002. From this date, the water level continued to increase and reached the normal pool level of 139 m in April 2003. Figure 2 shows the reservoir water level with respect to time.

The instrumentation system installed for monitoring the dam behavior includes geodetic monitoring system, inclinometers (no longer work), piezometers and flow measurement weir. The available geodetic monitoring data shows that the maximum crest settlement is about 0.035 m. This value was obtained during the period from February 2001, which corresponds to the beginning of reservoir filling (that is more than 7 years after embankment construction and 3 months after concrete slab completion) to September 2004 (that is 17 months after the end of the first reservoir filling).

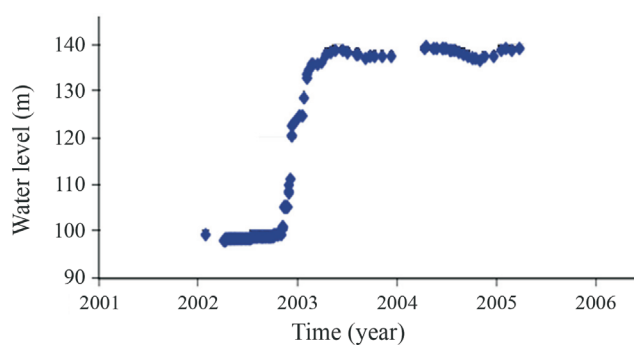

Figure 2 - Reservoir water level with respect to time.

The numerical study was conducted on the maximum cross - section of the dam resting on rock foundation using

Table 1 - Rockfill parameters.

Layer	Zone E1	Zone E2	Zone E1	Zone E2
Model	Hardening Soil	Hardening Soil	Mohr Coulomb	Mohr Coulomb
Unit weight: γ/γ_{sat} (kN/m^3)	19.5/20.5	20/21	19.5/20.5	20/21
Cohesion: c' (kPa)	0	0	0	0
Friction angle: ϕ'°	45	42	45	42
Dilatancy angle ψ°	15	12	15	12
Poisson's ratio: ν	-	-	0.15	0.15
Young's modulus: E (MPa)	-	-	34*	80
Stiffness modulus: E_{50}^{ref} (MPa)	29*	80	-	-
Stiffness modulus: E_{oed}^{ref} (MPa)	21*	80	-	-
Stiffness modulus: E_{ur}^{ref} (MPa)	87	240	-	-
Poisson's ratio: ν_{ur}	0.2	0.2	-	-
Stress dependent stiffness: m	0.25	0.25	-	-
Failure ratio: R_f	0.75	0.75	-	-

*Obtained by back analysis.

Plaxis 2D-2010 software. The embankment was modelled using fifteen noded plane strain elements. Figure 3 shows the finite element model having 2228 elements and 18217 nodes.

The deformation of the dam foundation is considered negligible; therefore the model sets fixed displacements along the dam base in contact with the rock foundation. Furthermore, the dam is located in a wide valley (crest length / dam height equals to 6.1), thus three dimensional effect is negligible.

The concrete slab is modelled as linear elastic with Young's modulus equals to 30000 MPa and Poisson ratio equals to 0.2. Rockfill materials are modelled with Mohr Coulomb and Hardening Soil models. Table 1 lists the parameters of the dam materials used in the numerical analysis. The dilation angle was estimated by the relation $\psi = \phi$ - 30 (Bolton, 1986).

The water pressure was applied as a triangular load distribution acting perpendicular to the face slab.

5. Results and Discussion

Only the stiffness parameters for the main rockfill zone (E_1) were optimized by back analysis based on the available monitoring data while other parameters remained the same. The estimated values of the remainder models (MC and HS models) input parameters based on published data for similar materials are judged to be more accurate than stiffness parameters. The transition zone material (E_2) beneath the concrete face has no significant effect on the prediction of the dam deformation.

The Young's modulus for the main rockfill zone (E_1) used in MC model was calibrated based on the available measured crest settlement in situ (35 mm) and the best result was obtained when E = 34 MPa. This value agrees well

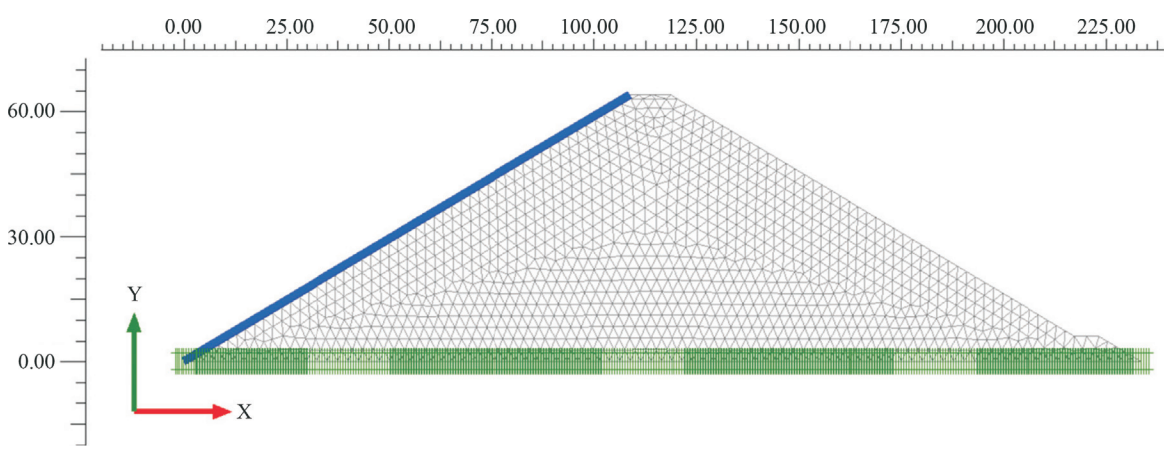

Figure 3 - Two dimensional finite element model.

with the stiffness modulus of Xingo dam rockfill material measured at the end of construction (E = 30 to 39 MPa). This granite gneiss rockfill material has a maximum particle size of 650 mm and compacted in layers of 1 m thick (Pinto & Marques, 1998).

Using Eq. 4 with $E_{50} = E = 34$ MPa , $m = 0.25$ and an average confining pressure of the dam embankment of about 190 kPa, the computed E_{50}^{ref} value used in HS model was found equals to 29 MPa. This value can be verified by introducing it into the Eq. 7 and calculating the number K used in hyperbolic model (Duncan & Chang, 1970). This number was found equals to 464, which agree with the typical values given by many authors for this parameter such as Duncan *et al.*, 1980; Saboya *et al.*, 1993; Liu *et al.*, 2002; Massiera *et al.*, 2006.

The oedometric stiffness modulus E_{oed}^{ref} is then back analyzed based on the available measured crest settlement in situ (35 mm) and the best results was obtained with $E_{oed}^{ref} = 21$ kPa, *i.e.*, $E_{oed}^{ref} = 0.72\,E_{50}^{ref}$. This result agrees fairly well with that obtained by Pramthawee *et al.* (2011) for a concrete face rockfill dam main rockfill material with a maximum particle size of 800 mm ($E_{oed}^{ref} = 0.8\,E_{50}^{ref}$).

Figure 4 shows the variation of the total displacement with respect to dam height for the concrete slab, the downstream face and along the dam axis at the end of reservoir filling (water level at 139 m). This figure indicates that, the maximum displacement of the concrete slab obtained using MC model is 0.26 m and takes place at a point about 44% of the dam height whereas HS model gives a maximum displacement of the concrete slab of 0.23 m and takes place at a point about 38% of the dam height.

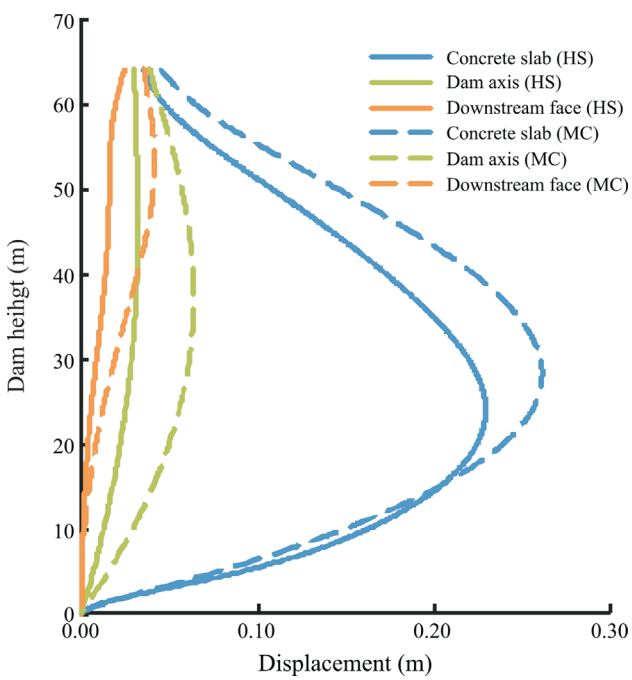

Figure 4 - Total displacement with respect to dam height at the end of reservoir filling.

The HS model resulted in concrete slab deformation less than MC model. This is attributable to the unloading behaviour of the upstream side of the dam where water load is applied, which is not taken into account by MC model.

Figure 5 indicates that the two models resulted in similar face slab deformation until the reservoir water depth reached about 40% of the dam height, after that the discrepancy between the two models increases with the increase of water depth. The maximum deformation normal to the face slab at the end of reservoir filling is less than 0.5% of the dam height and agreed with the results presented by Won & Kim (2008).

Figure 6 shows that the crest settlement is not affected by water load when reservoir water depth is less than about 40%H (H is the dam height) for HS model and 60%H for MC model. This can be explained by the fact that when the location of water load was far from the dam crest, no significant crest settlement was observed. With the increase of reservoir water level, the location of water load becomes

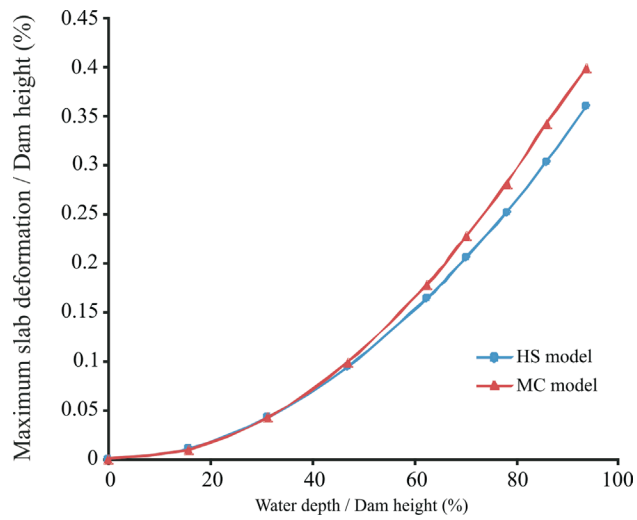

Figure 5 - Maximum deformation normal to the face slab *vs.* reservoir water depth expressed as a ratio of the dam height.

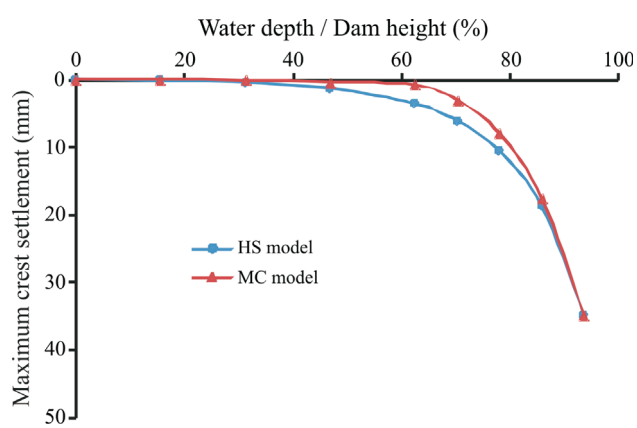

Figure 6 - Maximum crest settlement *vs.* reservoir water depth expressed as a ratio of the dam height.

close to the crest, and then significant increase of crest settlement appears.

6. Conclusions

Since the reservoir filling in April 2003, the dam has performed well. The measured flow is less than 250 L/min. In the absence of monitoring data of the face slab deformation, numerical analysis was conducted to predict the face slab deflection. The parameters of the dam materials used in the numerical analysis were calibrated based on the available monitoring data of the crest settlement.

The numerical results indicate that the magnitude and location of the face slab maximum deflection are in good agreement with the results presented in the literature for concrete face rockfill dams (Hunter *et al.*, 2003).

This study shows that from the measurement of the crest settlement which is easy to get from geodetic surveys, we can predict the face slab deformation using an advanced model (HS model) to simulate the behaviour of the rockfill materials. The HS model captures many important features of rockfill behaviour than MC model such as the non-linearity of stress-strain relationship and stress dependency of stiffness.

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