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DEFORMATIONS OF CONCRETE FACE ROCKFILL DAMS (CFRDs) RESTING ON SOIL FOUNDATION

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K e y w o r d s: conrete face, rockfill dams, diformations.

A b s t r a k t

Concrete Face Rockfill Dams (CFRDs) undergo deformations during their construction and during reservoir filling. During construction, the rockfill settles and compacts until the end-of-construction stage. During impounding, the rockfill deforms under the water pressure. The upstream concrete face deforms as a result of the rockfill deformation. The type of foundation also has an influence on the deformation of the rockfill and the concrete face. The displacements of the concrete face during the reservoir filling should not exceed the maximum values allowed in order to maintain its structural integrity.

The paper presents a study on the behavior of the upstream concrete face and internal displacements that develop in the rockfill embankments and its foundation during the construction phase and the reservoir filling in the case when the dam is built on till foundation or on dense granular alluvium deposits. The research is based on geotechnical parameters of the Toulnustouc dam located in Northern Quebec. The results of the study are supposed to serve also as a basis for designing deformation monitoring schemes for dams built on till foundation or on dense granular alluvium deposits during their construction and during the filling of the reservoir.

DEFORMACJE TAM Z OSŁONĄ BETONOWĄ POSADOWIONYCH NA ZIEMI

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Słowa kluczowe: betonowy ekran, zapory ziemne, deformacje.

Streszczenie

Zapory ziemne z betonowym ekranem ulegają deformacjom w czasie budowy i napełniania zbiornika wodą. W czasie budowy następują osiadanie i kompakcja materiału ziemnego, aż do osiągnięcia stanu pokonstrukcyjnego. Podczas wypełniania zbiornika wodą materiał ziemny zapory ulega deformacji na skutek działania ciśnienia wody. Zachowanie się betonowego ekranu jest zależne od deformacji materiału ziemnego, na którym ekran spoczywa. Rodzaj materiału fundamentów, na których jest zbudowana zapora, ma także wpływ na deformację zapory ziemnej i ekranu betonowego. W celu zapewnienia jednolitej struktury zapory deformacje ekranu betonowego nie powinny przekroczyć maksymalnych wartości krytycznych.

Przedstawiono badania nad zachowaniem się betonowego ekranu oraz deformacjami zapory ziemnej i fundamentów w czasie budowy i wypełniania zbiornika wodą, gdy zapora jest budowana na fundamentach składających się z moreny. Wykorzystano dane konstrukcyjne i wartości parametrów geotechnicznych otrzymane z projektu zapory Toulnustouc znajdującej się w Północnym Quebeku. Wyniki badan powinny być wykorzystane w projektowaniu monitorowania deformacji zapory w czasie budowy i wypełniania zbiornika wodą.

Introduction

Concrete face rockfill dams (CFRDs) are gaining a worldwide recognition as the most economical type of dams to be constructed in extreme northern and sub-Antarctic regions. Heights of CFRDs may exceed 200 m. The use of rockfill that is less or not sensitive to the frost action and construction technologies allows lengthening of the construction season. The total duration of the construction of CFRDs with regards to the total duration of construction of earth dams is, on average, reduced by one year. The reduced construction time reduces largely the costs of construction and makes hydroelectric projects more economical. COOKE (1984) indicated that the use of this type of dam seems inevitable in the regions of the world which have extreme climates.

Most of the constructed CFRDs rest on the bedrock. However, there are some CFRDs constructed on soil foundations. Examples are: Potrerillos dam, 116 m in height, Pichi-Picun Leufu dam, 54 m in height, and Los Caracoles dam, all in Argentina (PUJOL 1999); Santa Juana dam, 103 m in height (ASTE-

TE and al. 1992) and Puclaro dam, 83 m in height in Chile (NOGUERA and al. 1999); West Seti dam, 190 m in height in Nepal (KENNEALLY and al. 2001;) and Morro de Arica dam, 215 m in height in Peru. The thickness of alluvium deposits of the foundation of CFRDs generally does not exceed 70 m, with the exception of Puclero's dam (NOGUERA and al. 1999) which rests on alluvium deposits 113 m thick.

The main concern for the safety of CFRDs is the deformation of the concrete face. During the reservoir filling, the load of the water and deformations of the rockfill forces the concrete slab to deform. The concrete slab acts as an impervious membrane and any development of cracks in the slab would allow water to penetrate the rockfill of the dam and cause the structure to weaken or even lose its stability.

During the construction of a rockfill dam, internal deformations take place due to change in effective stresses, and due to creep and secondary time effects. During the first filling of the reservoir, considerable movements can develop in the dam and in the concrete face. Thereafter, the rate of movement generally diminishes with time, except for variations associated with periodic raising and lowering of the reservoir and as a result of earthquakes or tectonic plate movements. In a classic CFRD where the concrete face is constructed after the construction of the rockfill embankment had been completed, it is very important to estimate the displacements of the concrete face during the filling of the reservoir. Furthermore, it is important to verify if these displacements are lower than the displacements compatible with the structural integrity of the concrete face.

Safety is the most important reason for observing the deformations of dams. Too large or unexpected deformations can be the only indication of potential problems of the dam or its foundation. Another reason for observing the deformations of dams, of less immediate concern but of potentially great long-range significance to the engineering profession, is the need for a better understanding of basic design concepts, and stress-deformation characteristics and geotechnical characteristics of soil and rockfill. The development of prediction methods, which allows the determination of deformations, stress distribution and a comparison of the predicted with the observed values, constitutes a valid tool to control safety.

The condition of foundations is an important factor influencing the stability of a dam. Marques FILHO and MACHADO (2000) reported that the concrete face of the dam Pichu-Picun Leufu 54 m in height, resting on deposits of gravels 25 m in thickness, moved maximum 1.2 m during the filling of the reservoir.

In the cases where the thickness of the granular deposits is lower than 30 m, it is possible to improve the compactness of the deposits by densification by vibro compaction to decrease displacements during the construction of the dam and the filling of the reservoir.

The construction of dams and dykes in the regions of Northern Canada, where the foundation conditions vary from bedrock to dense till or granular deposits, generates wide interest. Foundation conditions of the planned constructions call for studies to determine the range of possible movements of the concrete face during the construction of the dam and especially during the filling of the reservoir. The objective of this study was to determine if it is possible to build CFRDs on a medium to dense soil foundation. One of the CFRD projects is the Toulnustouc dam, located north of Baie-Comeau on the Toulnustouc River in Northern Quebec. The existing dam is 75 m in height and 575 m in length, and it is built on bedrock foundation. There are plans to construct additional dams and dykes in the region to increase the installed hydroelectric capacity. The foundation conditions vary from bedrock to dense till or dense granular alluvium deposits. Data on the geotechnical parameters from this site were used in the analysis.

The study was performed for heights of dams varying from 75 m to 150 m, resting on a till foundation or on granular deposits varying from 10 m to 140 m in thickness. Results of the study were compared with the calculated displacements of the concrete face of CFRD resting on the bedrock.

Description of the analyzed cfrds

Four cases of CFRD's were analyzed. The heights of the analyzed CFRDs were 75 m, 100 m, 125 m and 150 m. The cross-sections of theses dams are based on the cross-section of the Toulnustouc dam. The width of the crest of the analyzed dams was 7 m. The upstream and downstream slopes were equal to 1.3 H : 1V. The thickness of the concrete faces was 0,3 m for the dams which have a hydraulic head H of 50 m to 100 m and of 0.3 m + 0.002 H to 0.3 m + 0.004 H for the dams whose hydraulic head H exceeds 100 m (ICOLD, 2002). The analyzed dams were composed of 4 zones (7B, 8A, 8B, and 8C). Zone 7B, 3.5 m in width, composed of crushed stone. The zone 7B acted as a support material under the concrete slab. Next to zone 7B, there was zone 8A, 3.5 m in width, built with crushed stone. Zone 8A acted as a transition zone between zone 7B and the main rockfill zone 8B. Zone 8B was built of compacted rockfill. Zone 8B formed the body of the dam, together with zone 8C. Zone 8C was built of compacted rockfill (Figure 1).

In the analyses, the CFRDs rested on a foundation of dense till or granular alluvium deposits. The thickness of foundation varied from 10 m to 140 m.

Hyperbolic model of soil material

The behavior of the soil material may be determined using a non-linear hyperbolic model, describing the behavior of soil before failure, developed by KONDNER (1963) and KONDNER and ZELASKO (1963). In the hyperbolic model, the non-linear stress- strain curve is a hyperbola in s1-s3 versus axial strain plan.

The relation of the initial tangent modulus E_i and the confining stress s'3 is given by JANBU (1963):

$$E_i = KP_a \left(\frac{\sigma'_3}{P_a} \right)^n \tag{1}$$

Similarly the relation between the bulk modulus B and the confining stress s'3 can be determined (DUNCAN et al. 1980):

$$B = K_b \, P_a \left(\frac{\sigma'_3}{P_a} \right)^m \tag{2}$$

where P_a is the atmospheric pressure, K is the loading modulus number, and n is the exponent for loading behaviour. K^b is the bulk modulus number and m is the bulk modulus exponent. Parameters K, n, K_b and m, (DUNCAN et al. 1980), are generally determined from triaxial compression tests.

The hyperbolic model was modified by DUNCAN and CHANG (1970) with tangent Young modulus defined as:

$$E_t = \left[1 - \frac{R_f \, (\sigma_1 - \sigma_3)\,(1 - \sin\phi)}{2c\,(\cos\phi) + 2\,\sigma'_3 \sin\phi} \right]^2 E_i \tag{3}$$

where ϕ is the angle of friction, c cohesion, σ_1 and σ_3 are principal stresses. R_f is the failure ratio:

$$R_f = \frac{(\sigma_1 - \sigma_3)_f}{(\sigma_1 - \sigma_3)_{ult}} \tag{4}$$

The presented model of the non linear behavior of rockfill material is incorporated in the SIGMA/W software (KRAHN 2004). The model allows modeling of the rock embankment resting on the bedrock or on a foundation whose behavior is time independent.

Geotechnical parameters of CFRD

Rockfill

There are a very few data from triaxial compression tests and oedometric compression tests performed on a similar rockfill to Toulnustouc dam materials. The values of geotechnical parameters published in a geotechnical literature are very diverse. For example, the Norwegian Geotechnical Institute (NGI, 1987) uses respectively values of K = 1800 and n = 0.25 with R_f = 0.65 for the rockfill, with grain size maximum 200 mm and K = 900 and n = 0.45 with R_f = 0.70 for the rockfill with maximum grain size 800 mm. The rockfill comes from granite gneiss or a diorite massif. According to BARTON and KJAERNSLI (1981), these types of rockfill have an angle of effective friction of 45o under an effective normal stress of 1000 kPa. SABOYA and BYRNE (1993) recommends to use for a preliminary analysis, values of varying from 250 to 500, n varying from 0.25 to 0.50 and R_f equal to 0.60 or 0.80 according to the diameter of the rockfill (maximum 500 mm or 1000 mm) for rockfill resulting from a massive basalt and from a basalt breccia.

The values of the geotechnical parameters used in the analysis are presented in Table 1 (RSW INC 2001). The following zones are specified: zone of the crushed stone with maximum grain size 80 mm (zone 7B), zone of the crushed stone with maximum grain size 200 mm (zone 8A), rockfill with maximum grain size 900 mm (zone 8B), and zone of rockfill with maximum grain size 1800 mm (zone 8C).

Table 1

Values of geotechnical parameters

Parameters	Rockfill Zones 7B and 8A	Rockfill Zone 8B	Rockfill Zone 8C	Foundation of Till (Moraine)	Foundation of granular alluviums		
					0–30 m	30–60 m	>60 m
K	1000	500	400	1670	800	1000	1200
K_b	800	240	240	1030	400	500	600
n	0.5	0.5	0.5	0.5	0.5	0.5	0.5
m	0.2	0.2	0.2	0.5	0.2	0.2	0.2
R_f	0.35	0.35	0.35	0.5	0.7	0.7	0.7
K_{ur}	1200	600	480	2004	0	0	0
ϕ	45^o	45^o	45^o	37^o	32^o	32^o	32^o
γ (KN/m^3)	19.5	19.5	19.5	21.44	18.8	19.0	19.5

Types of foundations: till (moraine) and granular alluvium deposits

Two types of foundations were investigated. In the first phase of the analysis, it was assumed that a very dense till (moraine) constitutes the foundation of CFRDs. Values of the geotechnical parameters used in the hyperbolic model of the moraine are presented in Table 1 (MASSIÉRA and SZOSTAK-CHRZANOWSKI 2003). In the analysis, the groundwater was assumed to reach the surface of soil foundation.

In the second phase of the analysis, it was assumed that the foundation was formed by granular alluvium. The foundation was subdivided into three layers varying in depth:
- between 0 and 30 m – a layer built of compact fine to coarse sand,
- between 30 m and 60 m – a layer built of dense medium to coarse sand, and
- more than 60 m – a layer built of dense coarse gravely sand with the presence of blocks.

The values of the granular deposits parameters are presented in Table 1. The parameters were determined on the bases of the results of triaxial drained compression tests made on similar granular soils, during the realization of the La Grande Complex, phase 1 and later adjusted according to the gradation and the compaction of granular materials.

Results of parametric analyses

Analytical method

The analysis was performed using finite element method and SIGMA/W software (KRAHN 2004). The analyzed CFRDs rested on soil foundation (moraine or granular deposits). The investigated dams were respectively 75 m, 100 m, 125 m and 150 m in height, and were divided into 5 m thick layers (Figure 1). In the phase of the filling of the reservoir, the water pressure was applied step by step, increasing the water level by 5 m until the maximum level at the end of the filling of the reservoir. A horizontal pressure of water was considered on the vertical line crossing by the upstream foot of the CFRDs resting on soil foundation to take into account the presence of a concrete cutoff (Figure 1). In this phase, the internal stresses calculated at the end of the phase of construction of CFRD and the soil foundation, were taken as initial conditions. In the analysis, the groundwater level was assumed to be at the level of the initial surface of the soil foundation.

The base of soil foundation at the contact with bedrock was considered as non-deformable. The vertical borders of the soil foundation were situated at two times the half width of the dam and were constrained in horizontal direction (Figure 1).

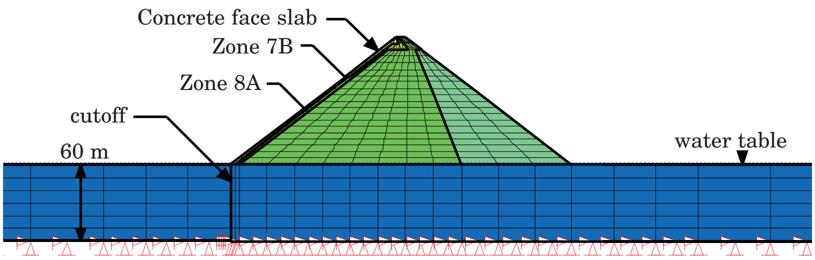

Fig. 1. CFRD of 100 m in height resting on 60 m of soil foundation

Displacements calculated during the construction

Figures 2 and Figure 3 present settlement isolines and horizontal displacements isolines of a CFRD cross-section 100 m in height resting on a deposit of 60 m thick till (moraine) respectively.

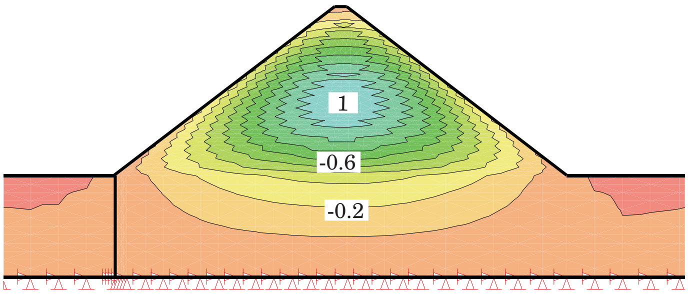

Fig. 2. Calculated settlements (m) at the end of the construction for a CFRD 100 m in height resting on 60 m of till (moraine)

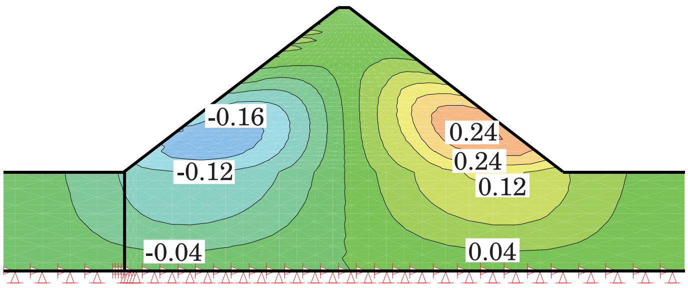

Fig. 3. Calculated horizontal displacements (m) at the end of the construction for a CFRD 100 m in height resting on 60 m of till (moraine)

The maximum settlements of the upstream concrete face, horizontal displacements, and the maximum displacements of the upstream slope increase not only with the height of the dam, but also with the thickness of the soil foundation. The level where the total maximum displacements of the concrete face develop decreases from 0.75 H (height of the dam) for a CFRD of 100 m in height resting on the bedrock, to 0.7 H for an identical dam resting on 60 m of till (moraine).

During the construction, the level where the maximum internal settlements developed decreases from 0.5 H (height) for a CFRD 100 m in height resting on the bedrock, to 0.4 H for an identical dam resting on 60 m of till (moraine) (Figure 2). Maximum horizontal displacements occur in the downstream shell, because in the upstream shell, zones 7B and 8A restrict side movements (Figure 3). The downstream face maximum horizontal displacements decreased from 0.3 H (height) from the base for a CFRD 100 m in height resting on the bedrock, to 0.2 H for an identical dam resting on 60 m of till (moraine).

Displacements of the upstream concrete face calculated during the filling of the reservoir

Figures 4 and 5 show settlements and horizontal displacements isolines calculated during the filling of the reservoir of a 100 m in height CFRD resting on a deposit of moraine 60 m thick. The settlements, horizontal displacements and maximum total displacements of the concrete face increased not only with the height of the dam but also with the thickness of soil foundation (Table 2, Figure 6 and Figure 7). The maximum displacements of the concrete face were function also of the maximum displacements of the dam during the filling of the reservoir. The level where these maximum displacements occurred decreased from 0.47 H (height) from the base for a CFRD 100 m in height resting on the bedrock, to 0.37 H for an identical

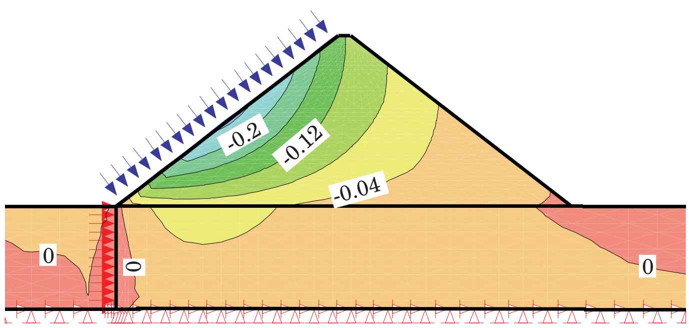

Fig. 4. Calculated settlements (m) at the end of the filling of the reservoir for a CFRD 100 m in height resting on 60 m of till (moraine)

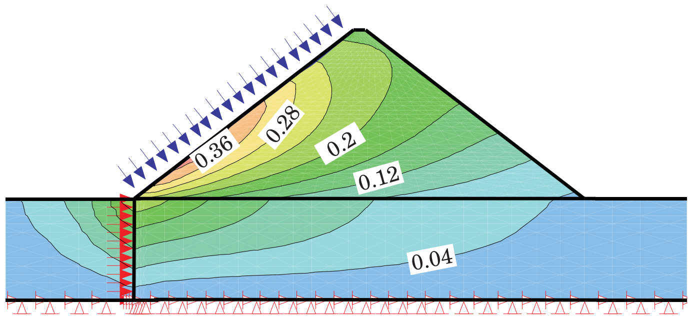

Fig. 5. Calculated horizontal displacements (m) at the end of the filling of the reservoir for a CFRD 100 m in height resting on 60 m of till (moraine)

Table 2

Maximum displacements of the concrete face of CFRD during the phase of reservoir filling

Maximum displacements of the concrete face							
Height of dam (m)	thickness of soil foundation (m)	foundation of till (moraine)			foundation of granular deposits		
		X (m)	*Y* (m)	*X-Y* (m)	*X* (m)	*Y* (m)	*X-Y* (m)
75	0	0.191	-0.141	0.234	0.191	-0.141	0.234
	20	0.202	-0.144	0.243	0.354	-0.167	0.375
	60	0.264	-0.156	0.297	0.706	-0.250	0.721
	100	0.329	-0.163	0.355	0.963	-0.329	0.976
	140	0.376	-0.164	0.399	1.174	-0.412	1.186
100	0	0.303	-0.230	0.374	0.303	-0.230	0.374
	20	0.312	-0.232	0.380	0.488	-0.256	0.522
	60	0.377	-0.246	0.435	0.899	-0.357	0.926
	100	0.455	-0.255	0.503	1.233	-0.426	1.248
	140	0.529	-0.261	0.567	1.538	-0.493	1.555
	0	0.434	-0.334	0.536	0.434	-0.334	0.536
	20	0.439	-0.335	0.539	0.632	-0.359	0.688
125	60	0.507	-0.349	0.596	1.104	-0.472	1.133
	100	0.591	-0.362	0.667	1.486	-0.555	1.515
	140	0.678	-0.370	0.739	1.864	-0.603	1.889
150	0	0.581	-0.453	0.720	0.581	-0.453	0.720
	20	0.583	-0.452	0.718	0.788	-0.476	0.873
	60	0.673	-0.473	0.780	1.305	-0.600	1.362
	100	0.740	-0.481	0.849	1.742	-0.702	1.812
	140	0.836	-0.492	0.929	2.162	-0.787	2.248

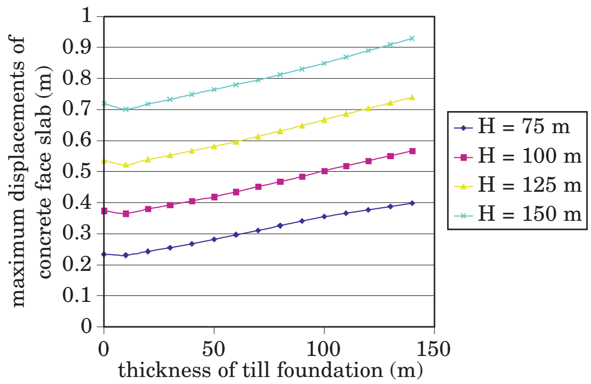

Fig. 6. Maximum total displacements (m) of the concrete face for CFRDs of various heights H (m) resting on the bedrock or on a till foundation, at the end of the filling of the reservoir

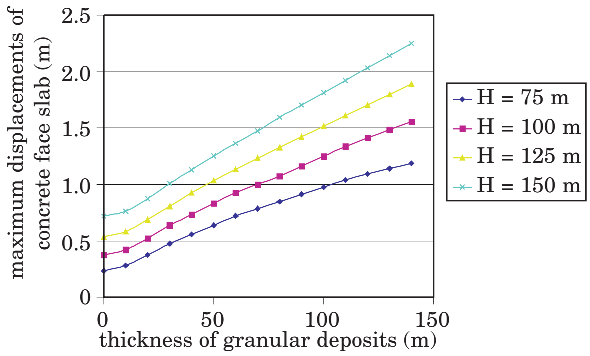

Fig. 7. Maximum total displacements (m) of the concrete face for CFRDs of various heights resting on the bedrock or on granular deposits, at the end of the filling of the reservoir

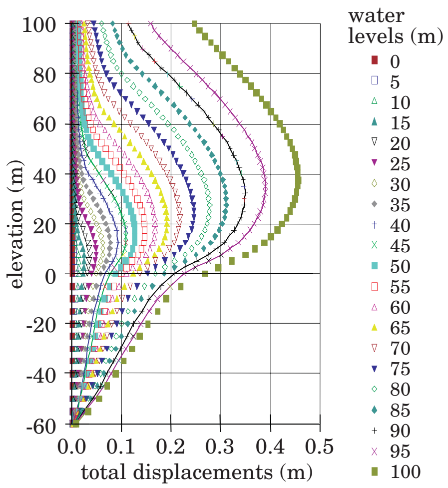

Fig. 8. Total displacements (m) of the concrete face at the end of the filling of the reservoir for a CFRD 100 m in height resting on 60 m of till (moraine)

dam resting on 60 m of till (moraine) (Figure 8) and at 0.2 H for an identical dam resting on 60 m of granular deposits (Figure 9). The maximum total displacements of the concrete face increased considerably when CFRD rested on granular deposits of higher thickness.

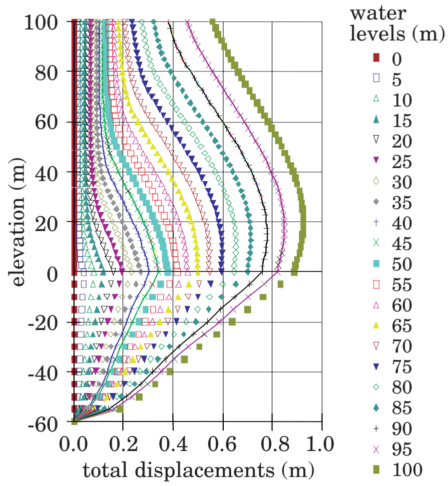

Fig. 9. Total displacements (m) of the concrete face at the end of the filling of the reservoir for a CFRD 100 m in height on resting on 60 m of granular deposits

Considerations regarding the design of deformation monitoring schemes

The purpose of installing geotechnical or geodetic instrumentation is to observe the behavior of dams. Type, number, and distribution of instrumentation depend on characteristics of the site of a dam (narrow valley with steep banks, rough variation of the geometry of foundations, more or less permeable deposits in the riverbed or on the abutments, etc.). The number and the distribution of monitoring instruments depend on specific problems foreseen in the design stage. Since the construction of CFRDs on till foundations and on granular alluvium deposits is still a novelty, the presented results of the study must be verified during the construction of any new dam and during the filling of the reservoir by monitoring measurements.

To have an overview of the deformations which occur in the body of a dam it is necessary to have, at least, a distribution of measuring instruments in two main cross-sections. The cross-section, where a plane strain state may be assumed, generally, coincides with the transverse section where the dam is the highest. The second cross-section is the plane corresponding to the axis of the dam. Monitoring data allows to make comparisons with bi-dimensional analyses and to verify the behavior of the dam.

ASCE Task Committee (2000) presented in detail various measuring instruments used to evaluate the behavior of dams. CFRDs contain typically two layers of crushed stone of transition of at least 3 m in width at once under the concrete face. These layers of transition are generally in crushed stone 0-80 mm for the first layer, and in crushed stone 0–200 mm for the second layer of transition. It is possible to install, in the first layer, inclinometers with telescopic joints parallel to the concrete face. Furthermore in the surface of the concrete face, movements in three directions can be measured punctually with submersible tilt meters connected with a reading post.

In the CFRDs, there are crack meters across the vertical joints in the slab, close to the abutments, to measure any opening of the construction joints. In most of CFRDs, they are installed at the top of the dam to measure the movements of the joint in the base of the rail. It is also recommended to use different types of instruments for the same type of deformation. For example, settlements can be measured with tilt meters and using inclinometer with telescopic joints.

The geodetic monitoring should be combined with geotechnical monitoring. There are new fully automated techniques for monitoring structural stability (DUFFY et al. 2001) of the dams. The techniques combine the use of Robotic Total Stations (RTS) and Global Positioning System (GPS). The monitoring systems are supported by ALERT software (WILKINS et al. 2003) developed at the Canadian Center for Geodetic Engineering (see http://ccge.unb.ca). ALERT provides for fully automated data collection, data processing, and graphical presentation of displacements. The fully automatic systems may also include geotechnical instrumentation. In the case of CFRDs, the monitoring scheme must be capable of monitoring the concrete face, which after completing the reservoir filling will be submerged under water. One should note that the maximum displacements are expected to be below the crest of a dam, therefore new monitoring geodetic techniques will have to be developed to detect the expected displacements at various elevations of the submerged concrete face.

Conclusion

For two dams of identical height resting on the bedrock or on the till foundation 140 m in thickness, the difference between the displacements of the concrete face during construction and during the filling of the reservoir of these dams was relatively small. The study showed that it is possible to build CFRDs on a very dense till foundation or very dense granular alluvium deposits. The CFRD which are planned to be built on compact to dense granular alluvium deposits should be the object of more studies to make sure that the anticipated displacements of the concrete face during the filling of the reservoir are compatible with the structural integrity of the concrete of slab.

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