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MODELLING OF DEFORMATIONS DURING CONSTRUCTION OF A LARGE EARTH DAM IN THE LA GRANDE COMPLEX, CANADA

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Key words: Deformation and stress modelling, earth dam, geotechnical parameters.

Abstract

Deformation of an earth dam is a complex process in which one should consider the nonlinear behaviour of the construction material, interaction between the structure and the underlying soil and rock strata, influence of water load on the structure and on the foundation bedrock, and the effects of water saturation. The deformation process can be simulated using, for example, the finite element method with the hyperbolic model of the nonlinear behaviour of the material. Due to the uncertainty of the model parameters, careful monitoring of the dam and its surroundings are required in order to verify and enhance the model. The deformations and stresses that develop in embankment dams during the construction phase are presented and illustrated by analyzing the behaviour of one of the most important structures of the La Grande Hydroelectric Complex (phase I).

MODELOWANIE DEFORMACJI W CZASIE BUDOWY DUŻEJ ZAPORY ZIEMNEJ W KOMPLEKSIE LA GRANDE, KANADA

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Słowa kluczowe: modelowanie deformacji zapory ziemnej, parametry geotechniczne naprężeń.

Streszczenie

Modelowanie deformacji ziemnych zapór wodnych jest skomplikowanym procesem. Modelowanie powinno uwzględnić nieliniowe zachowanie sie materiału, z którego jest zbudowana zapora, współdziałanie między zaporą i podłożem (ziemnym lub skalistym), wpływ ciśnienia wody w zbiorniku na zaporę i na podłoże oraz skutek nawodnienia części zapory. Modelowanie deformacji może być symulowane przez stosowanie np. metody elementów skończonych z hiperbolicznym modelem nieliniowego zachowania się materiału. Pomiary odkształceń dużych zapór wodnych dostarczają informacji nie tylko o zachowaniu się zapory, lecz również mogą służyć do weryfikacji parametrów geotechnicznych materiału użytego do budowy zapory. Metoda modelowania deformacji oraz naprężeń powstałych w czasie budowy zapory ziemnej jest przedstawiona na przykładzie analizy jednej z najważniejszych zapór kompleksu La Grande Hydroelectric Complex (phase I) w Kanadzie.

Introduction

The most common causes of failure of the embankment dams are internal erosion of fine-grained soils from the embankments, erosion under the foundation or abutment, stability problems resulting from the high pore pressures, hydraulic gradients, and overtopping of the dam or spillway. A less common cause of failure is the development of high water pressures and possible liquefaction either in the foundation or embankment during earthquakes.

Safety of earth dams depends on the proper design, construction, and monitoring of actual behaviour during the construction and during the operation of the structure. Monitoring also is important for a better and safer design of the future dams through the verification of the design parameters where the geotechnical parameters are of the highest importance (SZOSTAK-CHRZANOWSKI et al. 2003). The determination of geotechnical parameters may be done in situ or in the laboratory. In laboratory testing the selected samples may differ from one location to another, they may be disturbed during the collection, or the laboratory loading conditions may differ from natural conditions. Therefore, the comparison of the monitored data with the predicted data obtained during the design may give very important information concerning the geotechnical parameters (SZOSTAK-CHRZANOWSKI et al. 2002).

In the design of the earth dams, the finite element method (FEM) is used very often. The FEM is used in the analyses of expected displacements, strains, and stresses in the structure caused by changeable loading or boundary conditions. The values calculated from FEM may be compared with measured values giving additional information on the actual behaviour of the structure, boundary conditions and unexpected loads.

This paper presents the results of numerical analysis of the deformations during construction of an earth dam of La Grande Hydroelectric Complex (phase I) located in northern Quebec, Canada.

Deformations and Stresses in Embankment Dams

Deformations of an embankment dam start occurring during the construction of the dam. These deformations are caused by the increase of effective stresses during the construction by the consecutive layers of earth material and also by effects of creep of material. Deformations are also influenced by the deformations of the foundation (not discussed in this article), the transfer of stresses between the various zones of the dam and the other factors. After the end of the construction of a dam, the considerable movements of the crest and the body of the dam can develop during the first completion of the reservoir. Later, the rate of deformations decreases generally in time, with the exception of variations associated to the periodic variations of the level of the reservoir and, in seismic zones, to the earthquakes. Intensity, rate and direction of movements, in a specific point of the body of the dam or its crest, can vary during the various phases of the construction and the operation of the reservoir.

At the different elevations and in different zone types may occur the variations of stresses what can be caused by differential settlements between the core and the upstream and downstream filter zones. If the core is more compressible than the upstream and downstream filter zones, it settles more under its weight than the filter zone and, by the effect of arching, core mass leans on stiffer filter zones. This causes the reduction of vertical stresses and consequently the lateral stresses develop towards the base of the core. This phenomena can cause a hydraulic fracturing and a risk of erosion of the fine particles of the core.

Monitoring

The purpose of monitoring is to observe and verify the behaviour of embankment dams. Type, number and distribution of monitoring equipment depend on characteristics of the site of the dam (narrow valley with steep banks, rough variation of the geometry of foundations, soft or permeable deposits in the bed of the river or on supports, etc.). The number and the distribution of measuring instruments depend on specific problems foreseen in the training of the conception which, sometimes, control the schedule of due dates of construction. DUNNICLIFF (1988, 1989) presented in detail the various measuring instruments used in embankments and in earth and rockfill dams.

Monitoring of the embankment dams may be divided into following groups: environmental, geotechnical, geodetic, and visual inspection. The measurements of environmental effects causing the deformations and changes in the structure may be in the area of: hydrology (rainfall and snowfall), meteorology (temperatures of air and water, and external pressure), earthquake (seismic activity, natural and induced), and temperature within dam mass.

The geotechnical monitoring may be divided into two groups; physical and geometric measurements.
The physical measurements are:
1) pore pressure measurements in critical areas of the dam and in foundation using piezometers,
2) measurements of seepage through the dam, the foundation, and the abutments, using V-notch weirs,
3) measurement of stresses within the selected locations in the dam like steep abutments or narrow gorges and at earthfill - concrete interfaces using earth pressure cells.

The geometric measurements are:
1) tilt monitoring using plumblines or inclinometers,
2) foundation displacements using rod extensometers,
3) foundation movements using inclinometers.

Geodetic monitoring with terrestial and space positioning techniques (e.g. GPS) determines vertical and horizontal displacements of selected surface points from which rotations and strains can be derived. Recently, fully automated geodetic method (DUFFY et al. 2001) based on robotic total stations provide a powerful and reliable tool for monitoring. The visual inspection checks the developments of cracks, water seepage losses, local material degradation, and extension of wet spots.

Hyperbolic Model of the Behaviour of the Earth Dam

The behaviour of the earth material may be determined using hyperbolic non-linear model describing behaviour 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 σ_1–σ_3 versus axial strain plane. The relationship takes the form modified by DUNCAN and CHANG (1970):

$$\begin{pmatrix} \Delta\sigma_x \\ \Delta\sigma_y \\ \Delta\tau_{xy} \end{pmatrix} = \frac{3B}{9B-E} \begin{bmatrix} (3B+E) & (3B-E) & 0 \\ (3B-E) & (3B+E) & 0 \\ 0 & 0 & E \end{bmatrix} \cdot \begin{pmatrix} \Delta\varepsilon_x \\ \Delta\varepsilon_y \\ \Delta\gamma_{xy} \end{pmatrix} \tag{1}$$

where $\Delta\sigma$ and $\Delta\tau$ are stress increments and $\Delta\varepsilon$ and $\Delta\gamma$ are strain increments, E is Young modulus, and B is bulk modulus.

The relation of initial tangent modulus E_i and confining stress σ'_3 is given by JANBU (1963):

$$E_i = KP_a \left(\frac{\sigma'_3}{P_a} \right)^n \tag{2}$$

Similarly the relation between bulk modulus B and confining stress σ'_3 can be determined (DUNCAN et al. 1980):

$$B = K_b\, P_a \left(\frac{\sigma'_3}{P_a} \right)^m \tag{3}$$

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

LG–4 Main Dam of the La Grande Complex

La Grande 4 (LG-4) main dam, is the second largest structure of the La Grande Complex of James Bay hydroelectric development located in northern Quebec, Canada. The La Grande Complex covers 176 000 km^2 (Fig. 1). LG-4 main dam has maximum height of 125 m, crest length is 3800 m and has fill volume of about 19 millions m^3 (Société d'énergie de la Baie James, 1987). LG-4 main dam is a zoned embankment with central till core protected by sand and gravel filter and transition zones. The dam is constructed almost entirely on bedrock composed of granite and gneiss of Precambrian age. The typical cross-section of LG-4 main dam is shown on Figure 2. The soils used for various embankment dams of the La Grande Complex, phase I (PARÉ et al. 1982) were relatively homogeneous.

During the construction of the La Grande Complex, the main instruments which were installed in dikes and embankment dams to measure deformations were: inclinometers with tubes with telescopic joints, the indicators of settlements, and the linear extensometers. It allowed to follow the behaviour of the embankment during the construction and the filling up the reservoir.

A first analysis of these data at the end of construction was presented by GARNEAU et al. (1982), a more exhaustive review of these data, including those of the filling up the reservoir, was made by VERMA et al. (1985). These analyses put in evidence that studies made at the design stage (PARÉ et al. 1984) underestimated the modulus of the till core or of the granular filters (MASSIÉRA et al. 1989). A review of the data of deformations obtained in situ (BONCOMPAIN and MASSIÉRA 1991) showed that settlements during the construction of the LG-4 main dam had been underestimated, because the geotechnical parameters had been determined from triaxial compression tests realized on saturated samples. A new evaluation of parameters based, in particular on the oedometric compression tests for the till core and for the sand and gravel of filters and transitions on partially saturated samples was done by MASSIÉRA et al. (1999).

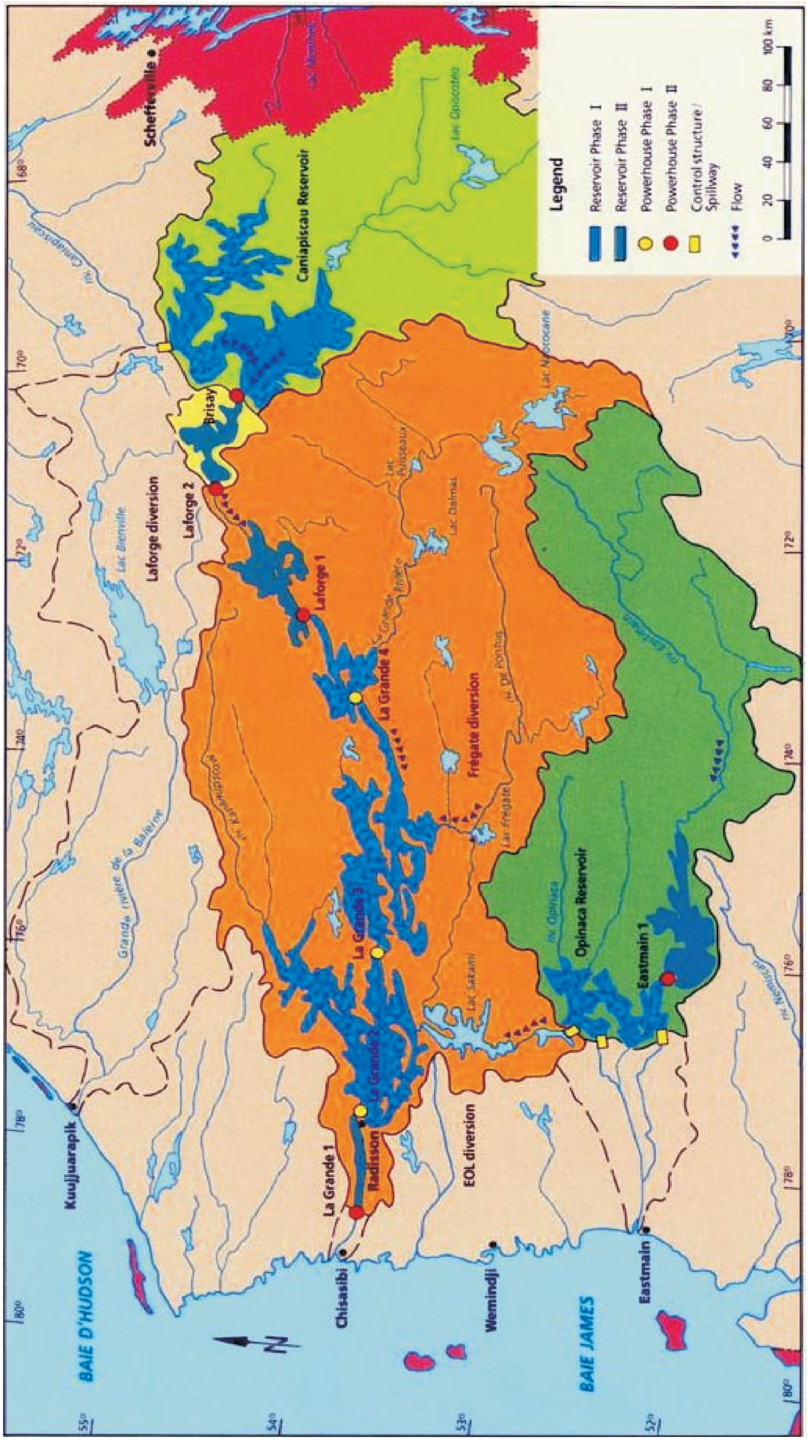

Fig. 1. Phases I and II of the La Grande Hydroelectric Complex

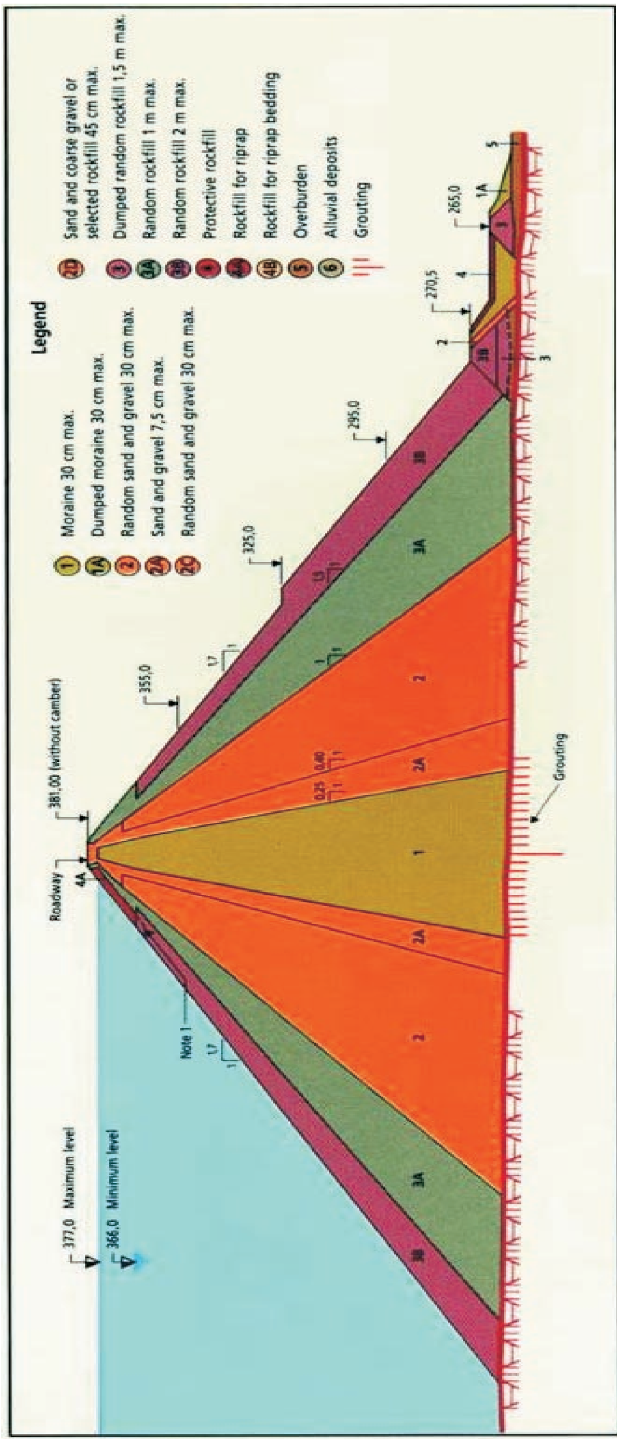

Fig. 2. General cross-section of the LG-4 main dam

Numerical Modelling of Deformations and Stresses

Simulation and analysis of the behavior of the LG-4 main dam aimed to confirm the values of geotechnical parameters (hyperbolic model) of materials used for the till core and for the filters and the transitions (sand and gravel).

The analysis was performed using finite element method with the hyperbolic model of the earth dam material. The dam was divided into 30 construction layers each 4 m high giving the total height of 120 m (Fig. 3). The analysed dam rested on the non-deformable bedrock. The modeled dam was divided into four zones: the till core (zone 1), upstream and downstream

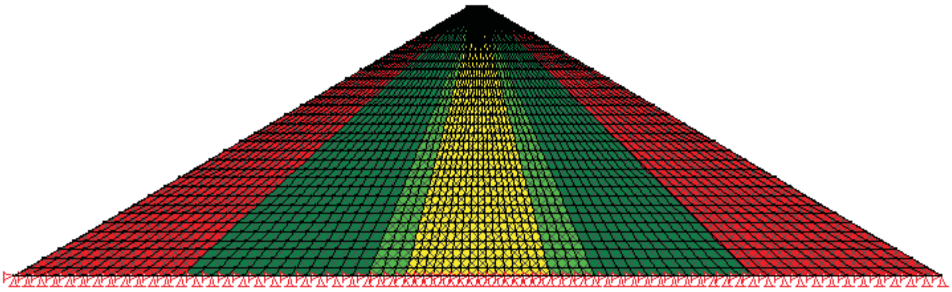

Fig. 3. LG-4 main dam finite element mesh

filters (zone 2A), upstream and downstream transitions (zone 2) and upstream and downstream rockfill shells (zones 3A and 3B). In the analysis using hyperbolic model, pore water pressures during the construction were neglected and the stresses calculated at the end of the construction correspond to actual stresses. This was confirmed by the values of pore pressures measured during the construction of the LG-4 main dam and the construction of the other similar dams of the La Grande Complex.

For the till core and the sand and gravel filters and transitions, hyperbolic parameters K, n, Kb, and m were determined from oedometric consolidation tests according to the method presented by Massiéra et al. (1999).

The values of parameters K, n, K_b and m of rock were selected from the results of the triaxial compression and of oedometric consolidation tests presented by the Norwegian Geotechnical Institute (NGI) in 1987. The values of K and n initially used in analyses were respectively 900 and 0.45, whereas they of K_b and m were respectively 300 and 0.20. Several parametric analyses allowed to correct the values of the exponents n and m in 0.8. An angle of friction f equal to 45° was adopted according to the study of the NGI (1987).

The values of R_f (Failure Ratio) were obtained from triaxial consolidation tests. Values of R_f were determined as being 0.5 for the till core and 0.6 for the sand and gravel (Massiéra et al. 1989). Value of R_f for the rock retained equal to 0.35. Table 1 presents the values of parameters used in analysis.

Table 1

Geotechnical parameters (LG-4)

Parameters	Core	Filters and transition	Shell
K	1670	4500	1000
K_b	1030	2850	(300)
			800
n	0,5	0.4	0.8
m	0,5	0.4	0.8
R_f	0,5	0.6	0.35
ϕ	37°	42°	45°
K_0	0.35	0.33	0.29
g	21.44	22.60	19.62

Calculated ssettlements were compared with those measured by the inclinometer INC-1 situated in the core and of the inclinometer INC-2 placed at the downstream location. Locations of INC-1 and INC-2 are 4 m and 35 m from the axis of the dam respectively. In the upper part in the dam at the location of INC-2 the calculated displacements are higher than measured. This gave an indication that the initial value of 300 for the geotechnical parameter K_b for rock fill was too low. The parameter K_b value was verified and assumed to be 800 (Table 1). The new analysis with the verified value for the geotechnical parameter K_b was performed. In the figure 4 (inclinometer INC-1), the settlements calculated in the core are compared with those observed when the dam reaches a height of 80 m and the end of the construction (height of 120 m). The figure 5 presents the comparison of calculated and measured settlements, in the filter, the transition and the rockfill shell (inclinometer INC-2). For the full height of the dam (120 m), maximum settlements measured and calculated in the core are of the same order of value, respectively 191 mm and 191 mm (Fig. 4). At the end of the construction, the maximum settlement measured in the downstream transition is 110 mm, compared with a calculated settlement of 132 mm (Fig. 5).

Measured and calculated horizontal displacements during the construction of the dam are small because of the zoning which is almost symmetric. The figure 6 present the horizontal displacements calculated in the end of the construction. Maximum displacements are observed in the downstream rockfill shell which is a little wider than the upstream rockfill shell.

The vertical stresses calculated at the end of the construction of the dam are presented in the figure 7. Results confirm the effect of arching observed. The measured and calculated vertical stresses of the dam during the construction are presented in figures 8. In the cell C1, situated in the axis of the base in till core, vertical stress calculated throughout the construction of the dam are close to those measured (Fig. 8).

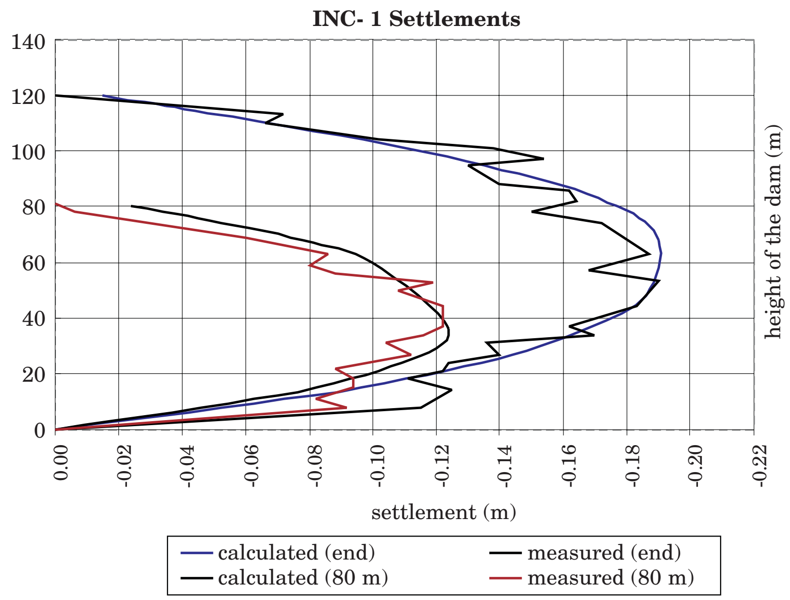

Fig. 4. Measured and calculated settlements at the location of INC-1 during construction

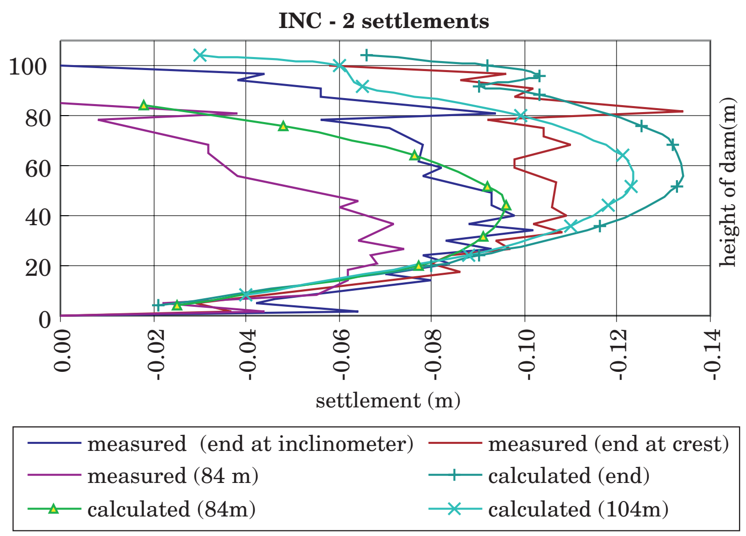

Fig. 5. Measured and calculated settlements at the location of INC-2 during construction

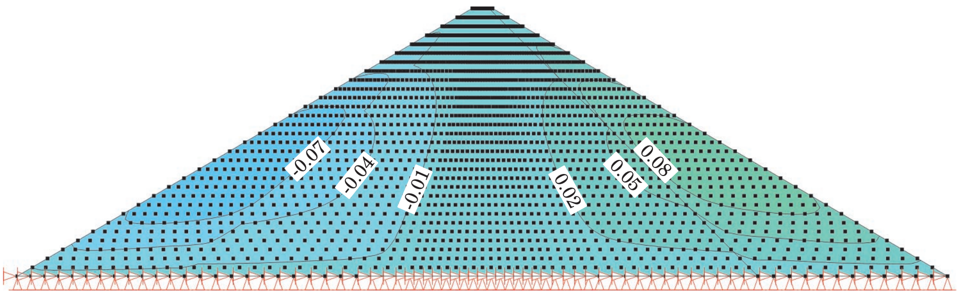

Fig. 6. Calculated horizontal displacements (end of construction) (m)

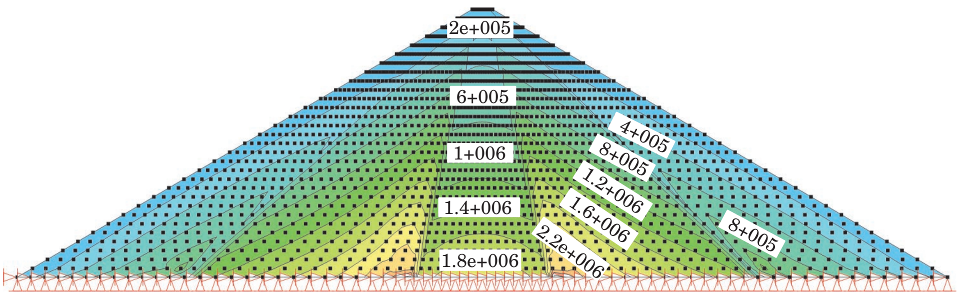

Fig. 7. Calculated vertical stresses (end of construction) (Pa)

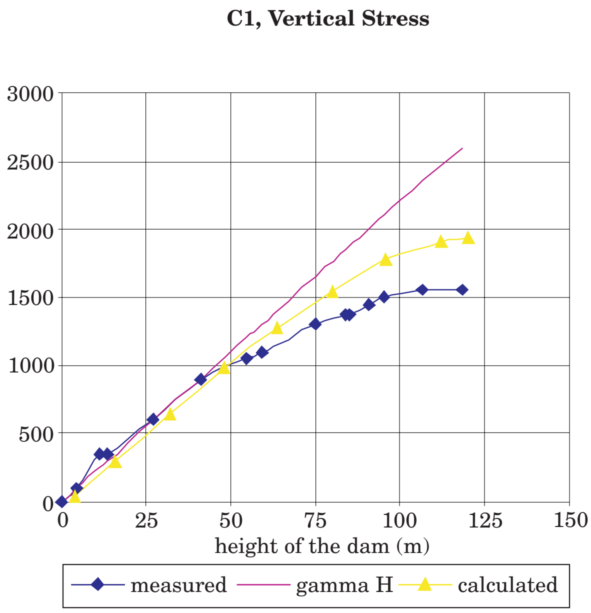

Fig. 8. Vertical stresses during construction in the axis of the base of the till core

Conclusions

Stresses and deformations can be relatively well estimated during the construction of embankment dams by using a hyperbolic model. For it, it is necessary that geotechnical materials parameters used in the various zones were correctly determined by means of laboratory tests for the till of the core and, for the sand and gravel of filter and transition zones.

A good understanding of the deformations, which occur in embankment dams, will allow to minimize the effects such as: transverse cracking, longitudinal fissuring, arching effect and stress concentration, hydraulic fracturing, development of plastic zones, damages in the instrumentation.

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