

A review of Permanent control of embankment dam stability during construction

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Abstract— Permanent control of embankment dam stability during construction, the first impounding and also during utilization of the project, is a significant issue. Instrumentation of such dams which monitor the dam behavior, plays an important role in the dam stability. Regarding that an earth dam is a continuous structure with non-linear behavior, finite element method and appropriate soil modeling could be employed for non-linear analysis. 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. Also 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.

Keywords— Embankment dam, geotechnical parameters, deformation and stress modeling, stability of dam

I. INTRODUCTION

According to seismic damage statistics, slope failure is a common failure mode for earth-rock dams. In the Wenchuan earthquake, 69 dams were severely damaged. Among them, 16 had landslides including the Fengshou, Minle and Yuejing dams [1,2]. Therefore, the analysis of the seismic stability of dam slopes is extremely important. Many researchers are developing new calculation methods to analyse seismic stability problems. However, the conventional pseudo-static approach is still widely used in engineering design because of its simplicity. In a pseudo-static analysis, the seismic loading is modelled as statically applied inertial forces. The pseudo-static approach has traditionally been implemented with limit equilibrium methods to assess the seismic stability of slopes [3–7]. However, neither static nor kinematic admissibility is necessarily satisfied in limit equilibrium.

Gikas et al in 2008 were investigated on Settlement analysis of the Mornos earth dam in Greece. The paper studies the long-term (>30 years) settlement behavior of the Mornos dam on the basis of finite element analysis and vertical displacement data. It compares actually measured deformations resulting from a continuous geodetic monitoring record of the dam behavior with a numerical back analysis. Their aim is to explain the actual deformation history on the

basis of the mechanical behavior of the dam. The deformation monitoring record consists of precise leveling data of a large number of control stations established along the crest and the inspection gallery of the dam, as well as settlements derived by magnetic extensometers placed inside the dam body. Overall, the available data cover the phases of construction, first filling of the reservoir and most of the operational time of the dam. The numerical modeling assumes 2D plane-strain conditions to obtain the displacement and the stress-strain characteristics of the abutments and the dam at eleven equally-spaced cross sections. Comparative evaluation of the results of the geodetic monitoring analysis against the findings from the finite element back analysis simulating characteristic stages in the lifetime of the structure, shows a very good agreement (on average 0.03 m) between the measured and computed displacements, which testifies to the correctness of the geotechnical parameters and loads used in the analysis [8].

Kim et al in 2014 were investigated on Deformation characteristics during construction and after impoundment of the concrete-faced rock-fill dam (CFRD)-type Daegok Dam in Korea. The monitoring record obtained during the dam construction and initial impoundment is presented. The vertical displacements during dam embankment are compared with other CFRD cases in Korea. Two-dimensional finite element analyses were performed to assess the displacements during both the construction and initial reservoir filling stages, using the model parameters estimated based on large-scale triaxial tests. A nonlinear and inelastic constitutive soil model was used to represent the behavior of the rock materials. In addition, the monitoring results for 29 CFRDs were analyzed. The observed and predicted values on the normalized maximum displacement and the location of maximum values during the dam construction stage agree well. Also, the limitations of numerical analysis on the reservoir filling stage are addressed [9].

Xu et al in 2014 were studied on Three-dimensional simulation of the construction process of the Zipingpu concrete face rockfill dam based on a generalized plasticity model. Concrete face rockfill dams (CFRDs) are becoming a widely used type of rockfill dam in China. In many cases, the design and construction of CFRDs are based primarily on precedent and engineering judgments. Few numerical or analytical methods have been developed to properly evaluate the deformation of CFRDs, which is important for dam safety and for subsequent evaluation of seismic performance. In this study, a finite element procedure was developed to

simulate the construction process of a CFRD, using the Zipingpu CFRD in China as an illustrative example. The Zipingpu dam was subjected to a strong earthquake in 2008, and can be used as a benchmark problem to assess the safety of CFRD. It is thus important to the engineering community to establish the state of stress-strain in the dam prior to the earthquake. A generalized plasticity model was modified to better model the rockfill materials, and the interfaces between face slabs and cushions are modeled using zero-thickness interface elements that follow a hyperbolic stress-strain model in the tangential direction. The model parameters were calibrated by large-scale triaxial tests and direct shear tests performed on materials used in the dam. The step-by-step construction followed by subsequent impounding of the reservoir was simulated in the numerical procedure. The numerical results agree well with in situ monitoring records of dam settlements, indicating that a three-dimensional finite element procedure based on a modified generalized plasticity model and a hyperbolic interface model can be used to evaluate the deformation of CFRDs [10].

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) [11]. 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

predicted data obtained during the design may give very important information concerning the geotechnical parameters (SZOSTAK-CHRZANOWSKI et al. 2002) [12].

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 behavior of the structure, boundary conditions and unexpected loads.

II. 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 km². LG-4 main dam has maximum height of 125 m, crest length is 3800 m and has fill volume of about 19 millions m³ (James, 1987) [13]. 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 1. The soils used for various embankment dams of the La Grande Complex, phase I (PARÉ et al. 1982) [14] 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) [15], a more exhaustive review of these data, including those of the filling up the reservoir, was made by VERMA et al. (1985) [16]. These analyses put in evidence that studies made at the design stage (PARÉ et al.

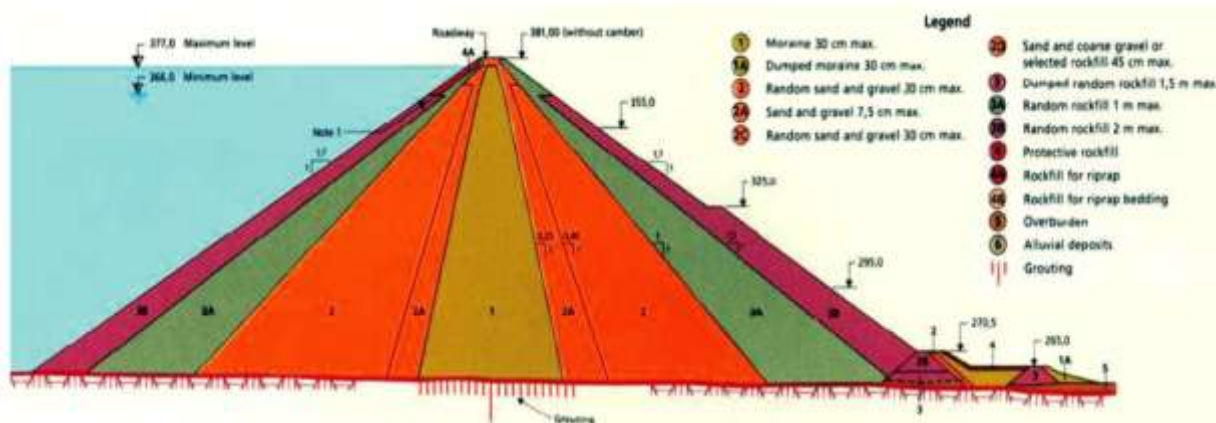


Fig. 1. General cross-section of the LG-4 main dam

loading conditions may be used. Therefore, the comparison

of the modulus of the till core or of the filter material (PARÉ et al. 1989) [18]. A review of

the data of deformations obtained in situ (BONCOMPAIN and MASSIÉRA 1991) [19] 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) [20].

due to shortcomings of the experimental data, empirical curves have been used for estimation of dynamic parameters of the materials of dam body and foundation [21].

One of the proposed formulas for estimation of G_{max} for granular materials known as Seed & Idriss, (1986) equation is Eq. (1) [22].

$$G_{max} = 21.7 P_a k_{2max} \left(\frac{\sigma'_0}{P_a} \right)^{1/2} \quad (KPa) \quad (1)$$

The only unknown parameter in the above equation is k_{2max} which can be defined by using proposed empirical curves. One of these diagrams is shown in Fig. (2) as an example. The value of k_{2max} is taken 140 for shells and 50 for filter and 40 for mixed core, according to the experimental results carried out on Karkheh earth dam materials [23]. Since the major part of the material consist of grains, Seed & Idriss (1986), [22], relation seems suitable. Meanwhile the value of G_{max} for foundation material has been derived from the results of measured shear wave velocity and regarding equation (2) [22].

$$G_{max} = \frac{\gamma}{g} v_s^2 \quad (2)$$

It should be noted that shear wave velocity in foundation material is also derived from the in situ tests as 1000 m/s [24].

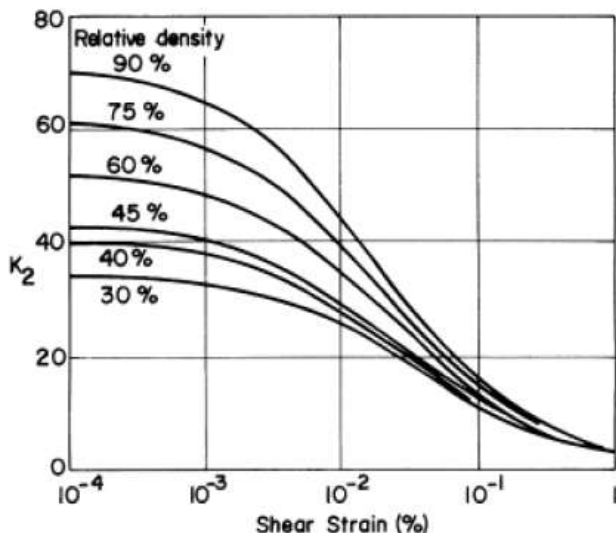


Fig.2. The different values of k_2 for different relative density in sand soils [22]

III. 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. Also Seepage and stress-strain analysis used to investigate the mechanical performance of cut off wall-core connecting systems in earth dams.

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