USE OF GEODETIC MONITORING SURVEYS IN VERIFYING DESIGN PARAMETERS OF LARGE EARTHEN DAMS AT THE STAGE OF FILLING THE RESERVOIR

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Abstract

Deformation monitoring of large dams and their surroundings supplies information on the behaviour of the structure and its interaction with the bedrock. Monitoring results may also be used in verifying design parameters where the geotechnical parameters are of the highest importance. 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 important information concerning the quality of the accepted geotechnical parameters. This paper discusses potentials and limitations of using geodetic monitoring surveys in verifying geotechnical parameters at the post-construction wetting stage during the filling of the reservoir. Large earthen dams of the Diamond Valley Lake (DVL) project in Southern California are used as an example of the discussion. A fully automated geodetic monitoring system has been established at the site of the project. A simulated analysis of loading effect during filling the reservoir (water loading and wetting), has been performed using the finite element method. The results of the FEM analysis of a partially saturated dam are comparable with the observed vertical displacements.

1. Introduction

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. The most common causes of failure of 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. Active tectonic faults in the area of the dam may be a source of extensive deformation of the dam and foundations and may lead to a loss of stability of the structure.

Monitoring of embankment dams, besides the visual inspection (qualitative data) and collection of environmental data (rainfall, seismic activity, and meteorological conditions), is usually grouped into geotechnical and geodetic surveying methods. Geotechnical monitoring may be divided into physical and deformation (geometric) measurements. The physical measurements include pore pressure measurements in critical areas of the dam and in its foundation; measurements of seepage through the dam and abutments; and measurements of stresses, using, e.g., earth pressure cells, within the selected locations in the dam. The deformation measurements typically involve settlement measurements, structural tilt monitoring and foundation displacements and deformations using e.g., extensometers and inclinometers.

Geodetic monitoring determines vertical and horizontal absolute displacements of selected (targeted) points at the crest and on the surface faces of the dam with respect to reference points located in a stable area using terrestrial and satellite positioning techniques. With the recent technological advancement in terrestrial surveying methods, robotic total stations with the automatic target recognition are commonly used to provide 3-D positioning information at pre-selected time intervals with the telemetric data acquisition. The satellite Global Positioning System (GPS) can provide continuous updating of positions of GPS receivers with a few millimetres accuracy with respect to stable points located up to several kilometres away.

Both geotechnical measurements and geodetic surveys complement each other in providing internal and surface information on the dam stability. Certainly, both types of instrumentation, if properly calibrated and installed, can serve as a warning system in case of an abnormal behaviour of the dam.

The goal of the authors' research is to optimise the use of the monitoring data for verification of the designed geotechnical parameters through a back (inverse) analysis (Gioda and Sakurai, 1987; Szostak-Chrzanowski et al., 1994). This paper is based on the data taken from the currently built Diamond Valley Lake Reservoir Project (DVL) in Southern California that includes three large earth/rock filled dams. The DVL project includes a fully automated geodetic monitoring scheme (Whitaker et al., 1999, Duffy et al., 2001) with a very dense distribution of frequently observed targets on the dam faces.

A preliminary study on the use of geodetic monitoring data at the stage of dam construction in dry condition was given in Szostak-Chrzanowski et al., (2000). This paper presents a study on a potential application of the geodetic monitoring surveys at the stage of filling the reservoir when wetting of the dam material may cause significant changes to the material properties resulting in a significant dam deformation.

2. Diamond Valley Lake Project

2.1. Description of the Diamond Valley Lake Reservoir Project

In 1996, the Metropolitan Water District (MWD) of Southern California, started a $2 billion project of constructing the Southern California's largest water storage reservoir Diamond Valley Lake (DVL) located about 160 km south-east of Los Angeles. It is being created by enclosing the Domenigoni/Diamond valleys at an elevation of about 500 m by three earth/rock filled dams. The reservoir, about 7.2 km long and more than 3 km wide, will cover over 1800 hectares of land.

This largest in the United States earthfill dam project consists of:
•West Dam, about 87 m high and 2.7 km long,
•East Dam, 56 m high and 3.2 km long, and
•Saddle Dam, 39 m high and 0.8 km long (Duffy et al., 2001).

In December 1999, the construction of the three dams was completed and filling of the reservoir begun. The filling of the reservoir may take between two to four years depending on water availability.

2.2. Designed Threshold Values of Deformation

At the design stage of the DVL Project, MWD performed a thorough analysis of the expected dam deformations during and after the construction including, among others, analyses of gravitational consolidation and settlement, effects of seepage, and earthquake- induced deformations. As a result of the analysis, post-construction threshold values of observed deformation have been established for the normal behaviour of the structures as listed in (MWD, 2000). Based on those values, the overall accuracy of the monitoring surveys has been designed to detect displacements with the accuracy of 0.01m at the 95% confidence level.

2.3. Monitoring of Dam Deformations

A fully automated monitoring scheme with a telemetric data acquisition has been designed using both geotechnical and geodetic instrumentation. The geotechnical instrumentation array includes 189 piezometers, 15 strong motion accelerographs, 3 inclinometers, 74 settlement sensors, 4 fixed embankment extensometers and 16 weirs. The instruments have been grouped mainly along selected cross-sections of the three dams.

The geodetic monitoring network consists of over 300 marked (targeted) points installed along the crests of the dams and on their upstream and downstream faces. The targets, on massive concrete blocks, have been installed successively with the progress of the dam construction with the first surveys initiated as soon as the first berm was completed. The accuracy of the geodetic measurements was designed to detect displacements larger than 10 mm at 95% confidence level (Whitaker et al., 1999).

Until October 2000, the monitoring surveys were performed manually, using geodetic levelling and four roving GPS receivers. In October 2000 a fully automated system with a capability of the continuous monitoring of the behaviour of the dams was implemented. The automated system consists of 8 robotic total stations (Leica TCA1800S) with the automatic target recognition and electronic measurements of angles and distances. In addition, 5 continuously working GPS receivers, linked to CORS have been permanently installed on the crests of the dams to provide a warning system that will “wake up” the robotic total stations in case of abnormally large displacements. The monitoring data is automatically collected at preselected time intervals and is controlled by an office computer located about 100 km away. All the data collection and automatic data processing are controlled by DIMONS software developed at the University of New Brunswick (Duffy et al., 2001; Lutes, et al. 2001).

3. Methodology of Deformation Analysis Due to Wetting

The analysis was performed with water reaching the West Dam about 27 m (beginning of December 2000) above its toe and the East Dam about 20 m (end of January 2001). Effects of wetting have been analysed and compared with monitoring data at both West and East Dams during the filling of the reservoir.

At the stage of filling the reservoir, the main two effects were considered: pressure of water and effect of wetting. During the process of wetting, the values of geotechnical material parameters and the derived values of Young modulus (E) decrease. Young modulus of the material in the submerged sections of the structure becomes smaller and buoyancy force is developed producing dam deformation.

The determination of deformation of the dams due to wetting was performed assuming the behaviour of earth dam material and the bedrock as linear elastic materials. In the analysis, the Young modulus of the sections of the earth dam were obtained from the non linear construction analysis (Szostak-Chrzanowski et al, 2000). The analysis was performed using the finite element method (FEM). The determined values of Young modulus vary through the structure. The values used in the analysis were averaged over selected zones.

In the process of the calculation of displacements, one has to determine the change of E between dry and wet conditions. Thus, the analysis of the effects of wetting has been performed in the following steps.
1. Determination of Young modulus for dry conditions (E_{dry}) of the dam and foundation using non-linear analysis of the dam construction.
2. Determination of Young modulus for partially wet conditions (E_{sat}) of the dam structure using non-linear analysis. The wet conditions are function of the reservoir water level.
3. The determination of the displacements of the dam structure caused by the difference between E_{dry} and E_{sat} using linear elastic analysis.

In the analysis of construction process (step 1 and step 2), the dams were assumed to have non-linear material characteristics and were modelled using the hyperbolic model (Duncan and Chang, 1970). In the hyperbolic model, the non-linear stress- strain curve is a hyperbola in $\sigma_1-\sigma_3$ versus axial strain plane. The relationship takes the form:

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

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.

In the non-linear hyperbolic model, the initial Young modulus (Janbu, 1963) is given as

$$E_i = K\ P_a \left(\frac{\sigma_3}{P_a} \right)^n$$

where E_i is the initial in situ Young modulus as a function of confining stress σ_3, K is loading modulus number, P_a is atmospheric pressure, and n is exponent for loading behavior. The bulk modulus is computed using formula (Duncan et al. 1980):

$$B = K_b P_a \left(\frac{\sigma_3}{P_a} \right)^n$$

where K_b is modulus number and m is bulk modulus exponent.

During the construction of the dam, the non-linear behaviour of the soil was modelled by successive increments of loading. Within each increment of the load, the soil behaviour was assumed to be linear, with the re-evaluated values of Young modulus. After achieving full compaction, the material of the embankment dam was modelled as a linear-elastic material.

The rock mass on which the embankment dam is located has been assumed to behave as a linear-elastic material under the load of the weight of the dam and the weight of water in the reservoir.

4. Geotechnical Parameters of the Dam Material

The DVL dams have been constructed from soil and rock. The core materials are silty and clayey sandy alluviums obtained from the floor of the reservoir and the rock fill was obtained from bedrock hills of the reservoir. Fig. 1 shows as an example a typical cross-section of the West Dam. Table 1 lists geotechnical parameters used in the analysis.

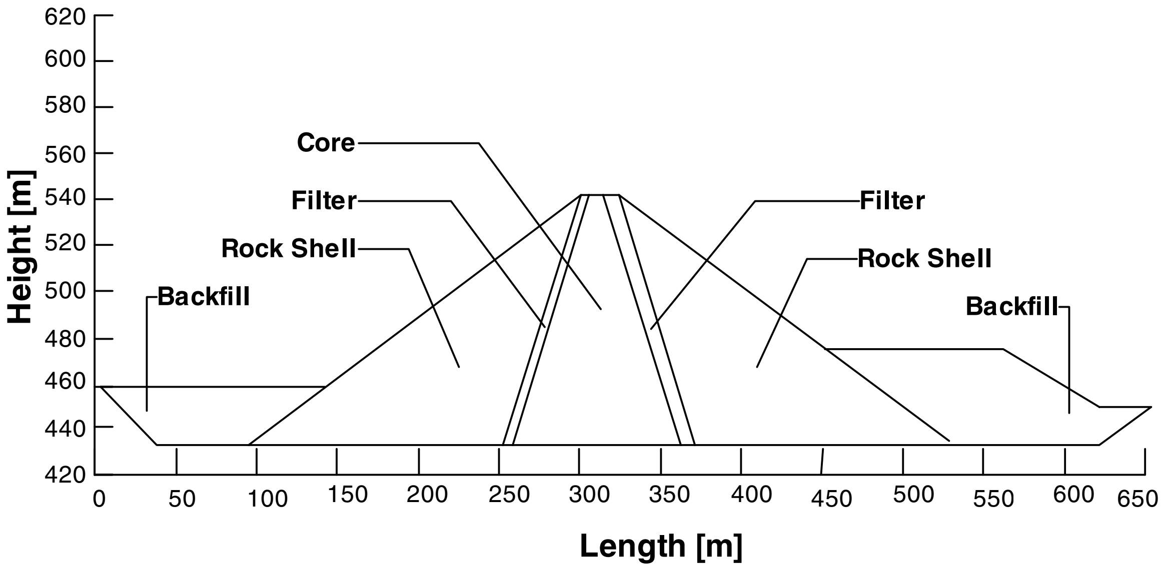

Fig.1 General cross-section schematic of the West Dam

Table 1. Geotechnical Parameters.

Parameters	DVL Parameters		
	Core	Filter	Rockfill shell
γ [kN/m^3]	22.12	20.54	20.00
ϕ	38	47	45
K	500	560	500
K_b	210	330	300
m	0.4	0.33	0.35
n	0.55	0.48	0.50

In Table 1, ϕ is an angle of friction, γ is unit weight, K is loading modulus number, and n is exponent for loading behaviour. K_b is bulk modulus number and m is bulk modulus exponent.
The wet parameters are smaller than dry parameters for the same soil. The wet parameters were obtained (Table 2) using following relations for core, shell and filter (Touileb et al. 2000):
Core: $K_{sat} = 0.5\ K_{dry,}$

Filter: $K_{sat} = 0.85\ K_{dry,}$

Rockfill: $K_{sat} = 0.6\ K_{dry}$

Table 2.Geotechnical parameters for saturated conditions at East and West Dams.

Material	K_{sat}	$K_{b\ sat}$
Rockfill shell	336	198
Filter	476	280.5
Core	250	105

5. Determination of Young Modulus

The values of Young modulus were obtained from the construction analysis using geotechnical parameters for dry and wet conditions. Figure 2 shows the investigated cross-section of the West Dam with the delineated wet (saturated) zones 5, 6, 7, and 8 when water level reached the height of 27 m above the toe of the dam**.** The obtained values of Young modulus significantly vary through the structure. Some selected values of E for dry conditions for the core in the West and East dams, are shown in Table 3 as a function of elevation. Table 4 lists average values of E_{dry} and E_{sat} for different zones of dry and wet material.

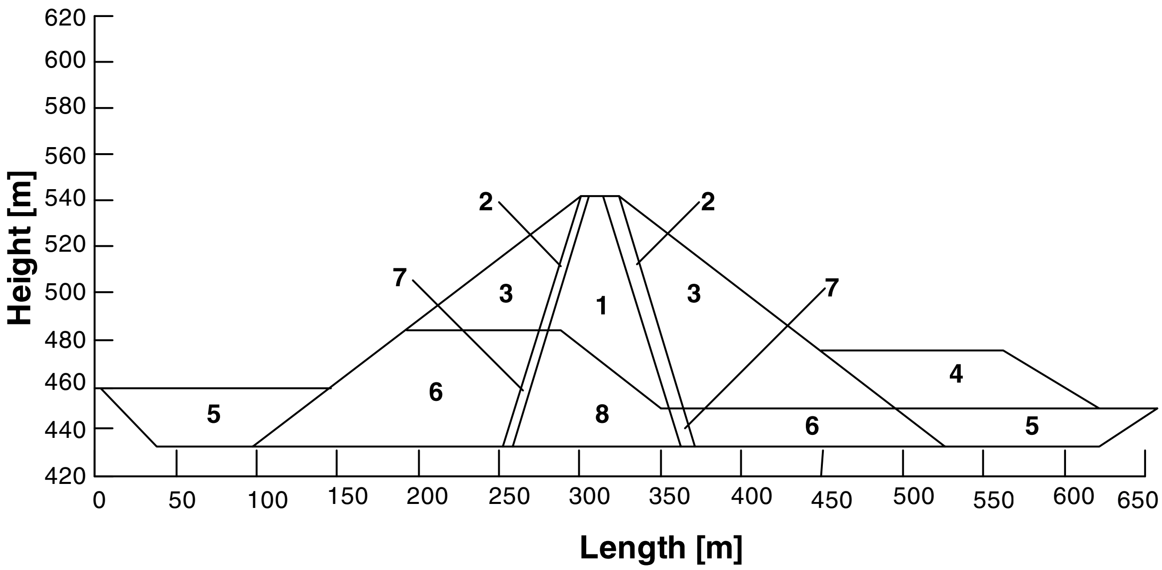

Fig 2. Schematic cross-section of the West Dam with dry and wet zones (zones 5,6,7, and 8)

Table 3. Calculated values of E for the core at the selected heights for dry conditions.

	East Dam	West Dam
Elevation [m]	E [MPa]	E [MPa]
530	25	42
505	40	39
475	44	37

Table 4. Average values of E for dry and wet conditions.

Zone number	Dry conditions	Saturated conditions
	E [MPa]	E [MPa]
1	38	
2	58	
3	38	
4	35	
5	40	25
6	50	40
7	90	70
8	38	28

6. Modelling of vertical displacements

6.1. Determination of Vertical Displacements at the West Dam.

The investigated cross-section of West Dam is located in the middle of the dam. At this location the dam is 83 m high. The cross-section of the model is shown in Fig. 2. The calculated values of Young modulus for dry and wet conditions are given in Table 4. The analysis was performed for the water level reaching 27 m above the toe of the dam. The calculated vertical displacements are shown in Table 5.

Table 5. West Dam - vertical displacements [m]

Elevation [m]	West Dam vertical displacements		
	Down stream face FEM	Down stream face measured	Centre of the core FEM
539 (crest)	-0.135	-0.144	-0.116
528	-0.163		-0.123
516	-0.146		-0.132
505	-0.108		-0.144
494	-0.068		-0.152
482	-0.043		-0.165
472	-0.026		-0.173

The FEM calculated displacements could be compared only with one measured value obtained to a point on the crest of the dam because the manual surveys (prior to the commencement of automated measurements in October 2000) did not include points on the downstream face. This one comparison gave a very good agreement.

6.2. Modelling and Prediction of Vertical Displacements at the East Dam

The investigated cross-section of East Dam is located in the middle of the dam. At this location the dam is 52 m high and at the time of the analysis the water level was 20 m above the toe. Following the same procedure as for the West dam, predicted vertical displacements were determined for a saturated model with acting buoyancy force. The results are listed in Table 6

Table 6. Vertical displacements on the downstream face of the East dam (water level 20 m)

Elevation [m]	East Dam Vertical Displacements [m]		
	FEM Saturated conditions	FEM Partially saturated conditions	Measured
539 (crest)	-0.043	-0.018	-0.026
529	-0.079	-0.030	
518	-0.092	-0.029	
508	-0.073	-0.026	
497	-0.021	-0.002	
487	0.044	0.004	

Similar to the West dam, the only comparison between the FEM determined and observed vertical displacements could be made at the crest of the dam. The difference indicated that the material was not yet fully saturated. A new model was considered with only partial saturation. In this model it was assumed that water did not yet penetrate through the second filter and therefore, the downstream sections 5 and 6 were not saturated. The observed -26mm displacement agreed with the calculated displacement of -18mm within the accuracy of the geodetic measurements.

7. CONCLUSIONS

Due to possible uncertainties of the values of geotechnical parameters of the construction material, one may expect large uncertainties in the determined post-construction Young modulus. This affects the determination of the expected deformations of the embankment dams. The developed method for modelling effects of the saturation of the dam material gives displacement results comparable to the observed values. The presented examples of modeling the dam deformation due to wetting shows that the predicted displacements, at the crest and at the downstream face, are of the magnitude that can easily be detected by geodetic measurements. The research is in progress and additional modeling of the DVL dams will be performed when filling of the reservoir will be completed and when more geodetic data will become available for the verification of the results.

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