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COMPARING THE GEODETICAL AND GEOTECHNICAL METHODS IN INVESTIGATING THE DEFORMATION OF EARTHFILL DAMS; A CASE STUDY OF MAHABAD EARTHFILL DAM, IRAN

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Abstract

The monitoring of an earthfill dam during the various phases of the construction and service is a crucial process to investigate the performance of the dam from the safety point of view. Monitoring is carried out through installing the instruments at the critical locations. In the present paper, instrumentation data and the numerical analyses have been used for the monitoring of Mahabad earthfill dam, Iran. Numerical analyses were carried out using Plaxis and Sigma-W. Considering the good agreement between the measured data and the values predicted by Plaxis and Sigma-W, it can be concluded that the numerical models developed in the present paper are accurate enough to be used for the analysis of earthfill dams.

Keywords: Monitoring, FE modeling, Mahabad earthfill dam, Plaxis, Sigma-W.

1. Introduction

Using the results of experiments carried out on the soil samples is not a perfectly reliable method to determine the properties and parameters of the soil which is suitable for constructing an earthfill dam. Hence, the perfect agreement between the behaviour predicted based on the experimental results/theoretical relations and real in-situ behaviour of structure is not expected. Accordingly, it is necessary to find a way that through which the observed and predicted behaviours can be compared. Thus, the geotechnical designs should be accompanied with the instrumentation in order to control the in-service structural behaviour and compare it with the predicted behaviour. In this way, it is possible to be sure about the safety of structural performance and to verify the theoretical approaches and design assumptions.

Nomenclatures

FE	Finite Elements
ICOLD	International Commission on Large Dams

Using the Instrumentation in an earthfill dam can not heal the existing defects but can be used as an alarm system which warns about the potential and dangerous defects. An effective instrumentation and monitoring system can identify the abnormal and sudden changes in specified parameters due to potential problems [1]. The main objectives of a monitoring process can be categorized as follows:

- To investigate the performance of the dam in order to be sure about its safety.
- To compare the real in-situ behaviour with the one predicted by the designer.

As stated in ICOLD Bulletin 68 [2], the type of monitoring, time step between two successive records, and the instrumentation method depend on the dam type, the age of the dam, the level of damage, and also importance and size of the dam.

One of the most important stages of the monitoring process is the selection of quantities considered for the monitoring. ICOLD Bulletin 68 [2] divided these quantities into two categories: primary and dependent. For example, the level of reservoir water surface is a primary quantity and the pore water pressure is a dependent one. A list of these quantities is presented in Table 1. It must be noted that some of them are not of great importance in earthfill dams.

Yamaguchi et al. [3] discussed the safety inspections and seismic behaviour of embankment dams during the 2011 off the Pacific Coast of Tohoku earthquake. Xenaki and Athanasopoulos [4] presented the results of laboratory resonant column and cyclic triaxial tests on specimens of two compacted soils (a sandy-silty clay and a sand-gravel mixture), planned to be used in the core and the shells, respectively, of a proposed earthfill dam. Meehan and Vahedifard [5] provided a review and comparison of existing simplified displacement-based sliding block models. Analyses were performed to evaluate the relative accuracy of fifteen of these simplified models for predicting earthquake-induced displacements in earth dams and embankments. Davoodi et al. [6] investigated the seismic response of embankment dams under near-fault and far-field ground motion excitation.

Zhu et al. [7] discussed the safety inspection strategy for earth embankment dams using fully distributed sensing. Total risk rating and stability analysis of embankment dams in the Kachchh region, Gujarat, India were performed by Srivastava and Sivakumar Babu [8]. Noorzad and Omidvar [9] carried out a seismic displacement analysis on embankment dams with reinforced cohesive shell. Lizarraga and Lai [10] studied the effects of spatial variability of soil properties on the seismic response of an embankment dam. Day et al. [11] discussed coupled pore pressure and stability analysis of embankment dam construction. Akhtarpour and Khodaii [12] experimentally studied the asphaltic concrete dynamic properties as an impervious core in embankment dams.

Table 1. Quantities usually considered for monitoring.

Primary quantities	Dependent quantities
Reservoir's water level	Seepage
Precipitation	Crown settlement
Temperature	Deflection
Seismic activities	Pore water pressure
	Global stress
	Earthquake induced acceleration

2. Mahabad Earthfill Dam; Specifications of Monitoring System

Mahabad dam was built with the aim of storing and using the water of Mahabad River in the southwest of Iran. It is located at the distance of 120 km from Urmia. The river has a high potential of flooding and its average annual discharge is 205 m^3/s. Mahabad earthfill dam has a thin impenetrable clay core protected by two layers of filter at its up- and down-streams [13].

After observing the abnormal behavior of the dam, most of its monitoring systems have been installed or revised by Mahab Ghods Co. in 1993. The objectives of this monitoring system include measuring: (a) the surface motions, (b) the internal deformations, (c) the piezometric pressure, (d) the level of the reservoir water surface, and (e) the volume of seepage [14].

2.1. Measuring the surface motions

In order to measure the dam motions, instruments were initially installed at the points B1-B7 on the down-stream edge of the dam (Fig. 1). Thereafter, in order to raise the level of control, points B8-B13 on the down-stream berm were also instrumented in 1972 (Figs. 2-3). Thirty additional points named T series were also instrumented on the down-stream of the dam crown. Since no considerable changes were observed in the records of T series, data recording at these 30 points has been stopped. In addition to the mentioned points, 41 additional points were instrumented on the up- and down-stream concrete kerbs of the crown. The recording at these 41 points has also been stopped. Since none of the above-mentioned points has a root inside the dam's body, they can only be used for measuring the surface motions of the crown.

2.2. Measuring the internal deformations

During the construction phase, two settlement measuring tubes were installed in the central part of the alluvial base at the down-stream in order to measure the settlement of the foundation. Installation sections of the tubes were located at 0+220 and 0+300 km (Fig. 5). These tubes did not have sleeves, so due to the friction among the tubes and the shell materials, the measured settlements are expected to be larger than the real values. It must be noted that the overload at the central section of the dam, which is equivalent to the weight of 47 m high dam body materials, is larger compared with the overload at the installation location of the tubes which is equivalent to the weight of 30 m high dam body materials. Furthermore, because of two reasons, the overload at the down-stream sections is lower than the central and up-stream sections: (1) the soil of this region is not fully saturated and (2) the water flows through the soil of this region. However, considering the settlement of the dam, the data obtained form these two locations is vitally important.

Considering the previously mentioned shortcomings of the B1-B7 instrumentations, in order to improve the control of dam motions, four shafts named B14, B14a, B15, and B16 were considered on the core crown (Fig. 4). The shafts are 40 cm wide steel tubes surrounded by concrete. The shafts penetrate 1.5 m into the clay core and their upper ends are located at the height of 0.5 m above the core crown. These shafts are used for controlling the vertical and horizontal motions of the clay core's upper segment.

2.3. Data recorded by monitoring systems

In this section, the data obtained from the monitoring systems including the piezometers, settlement measuring tubes, pressure breaking sinks, and instrumentation used to measure the deformations and reservoir water level are investigated (Figs. 1-7). Measurement systems can be divided into two main groups: (1) displacement measurements, (2) the measurement of water level in piezometers. The measurement of displacements was carried out by geodetic approach. In this method, a number of fixed points with global coordinates are considered around the dam. In order to determine the displacement of a specified point, coordinates of the considered point are determined trough these fixed points. Thereafter, by comparing these coordinates with the initial values, vertical and horizontal displacements are calculated.

About the fixed installation points on the dam body, it is worth mentioning here that:

a) The points controlling the crown of the dam are located on the integrated concrete berms at the up- and down-streams of the dam. Hence, due to the integrity and the lack of independence between the point and its connection to the concrete slap, it is not possible to separately determine the settlement of a specified point. Consequently, displacement of these points may not be a suitable representative of the region beneath the points.

b) The points corresponding to the monitoring of the clay core were installed four years after completing the construction of dam. These fixed points are located on the core as a concrete plate and can appropriately represent the displacement of the clay core.

c) The fixed points which are used to measure the settlement of the foundation, overestimate the magnitude of the settlement. The reason is the friction between the rod and the down-stream materials due to the lack of sleeves.

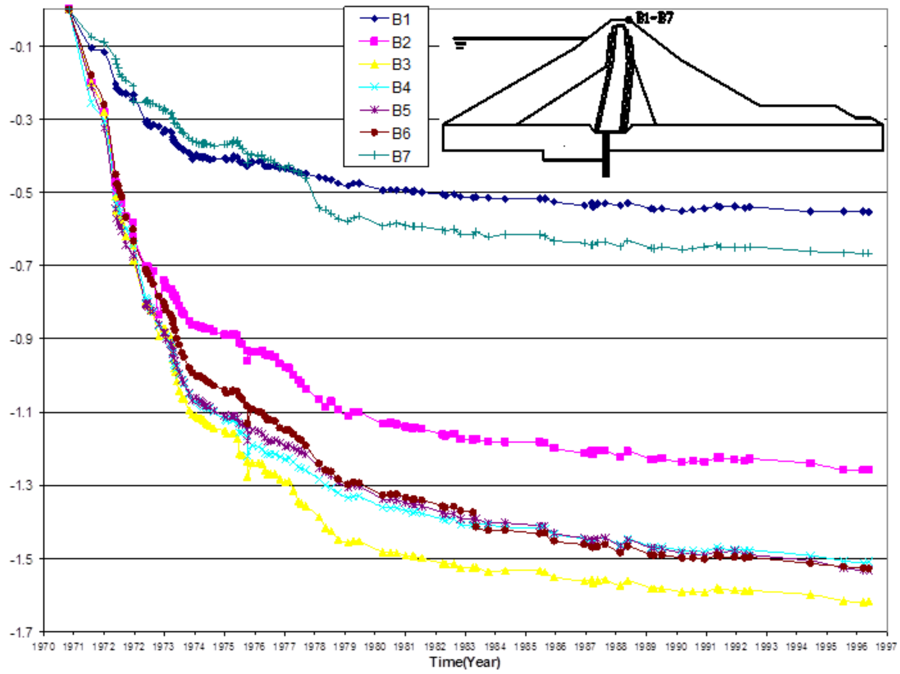


Fig. 1. Settlement of the points located on the down-stream edge of the dam's crown from kilometer 0+100 to 0+485.

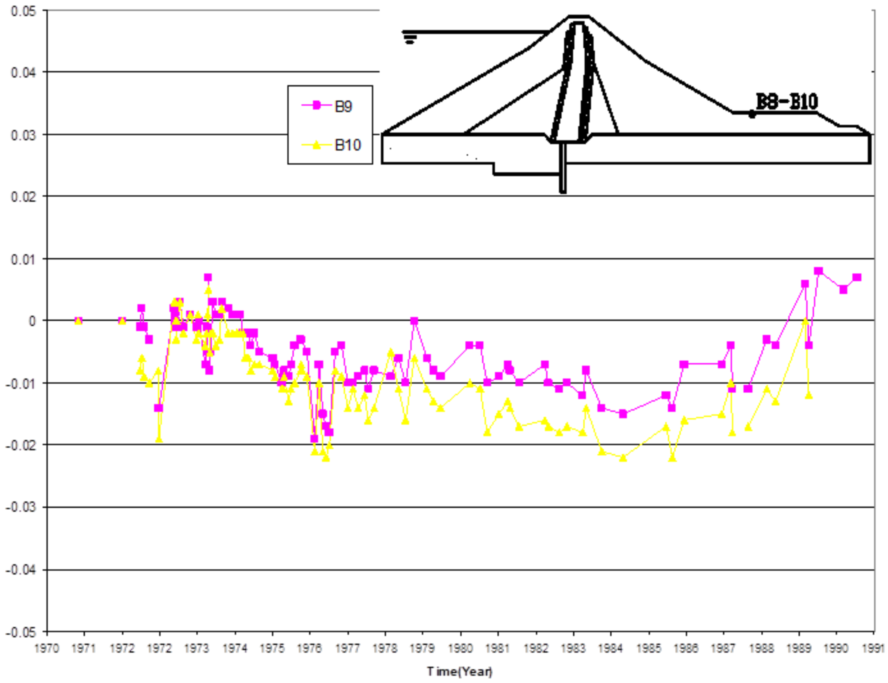


Fig. 2. Settlement of the points located on the down-stream berm.

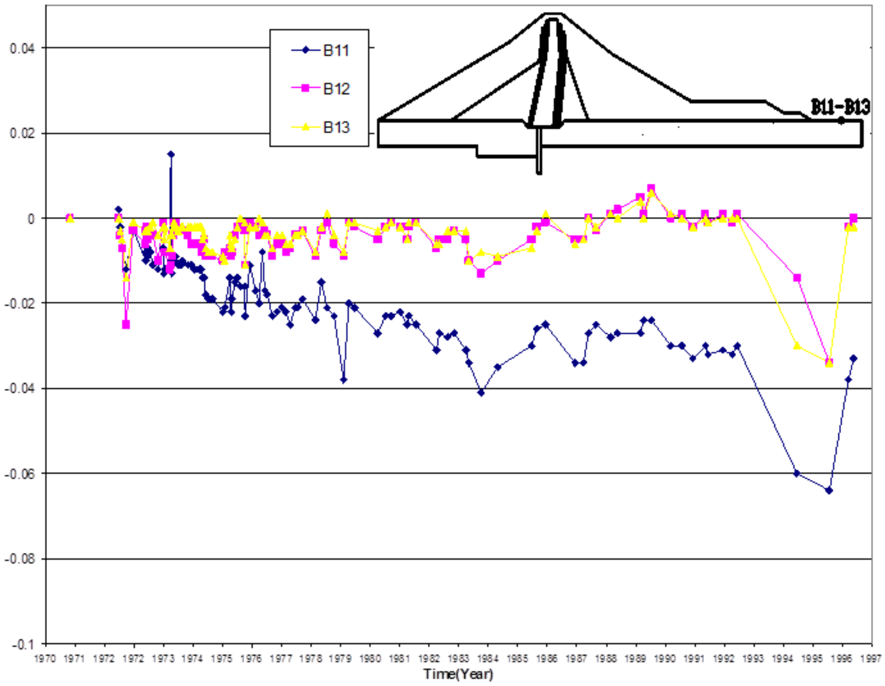


Fig. 3. Settlement of three points at the distance of 15 cm located on the down-stream of the dam.

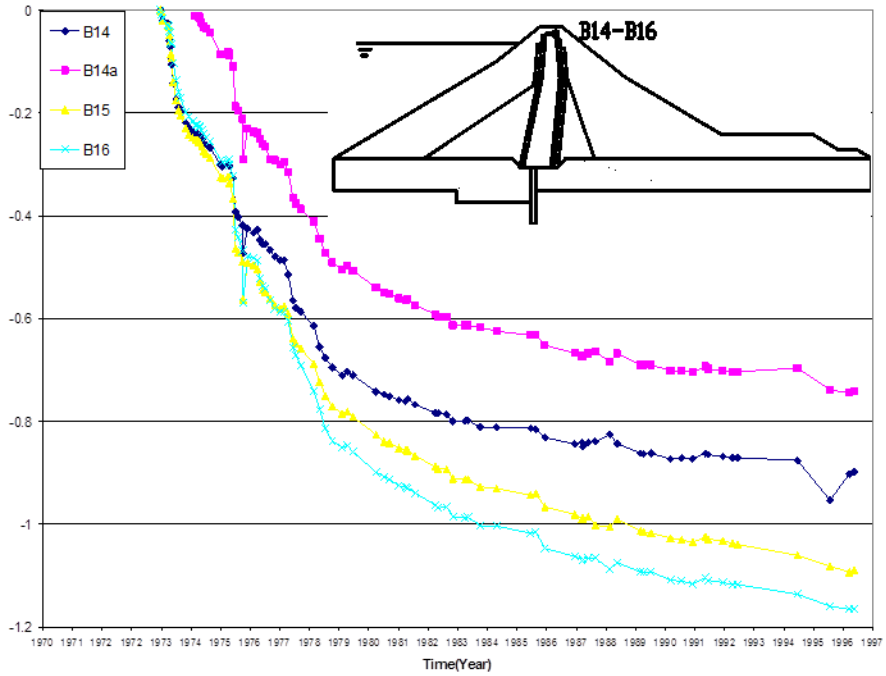


Fig. 4. Settlement of the points located on the clay core.

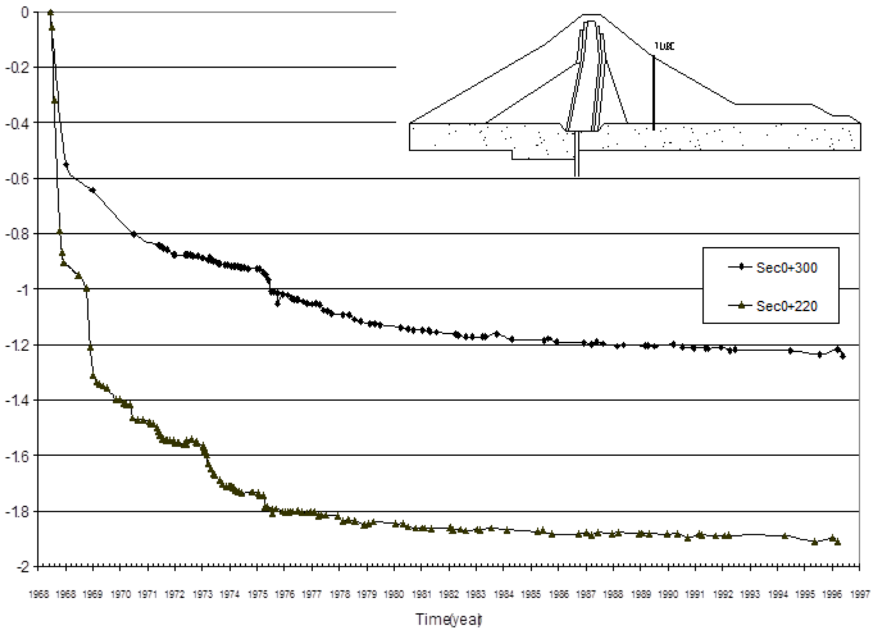


Fig. 5. Settlement of the foundation at kilometers 0+220 and 0+300 measured by settlement measuring tubes.

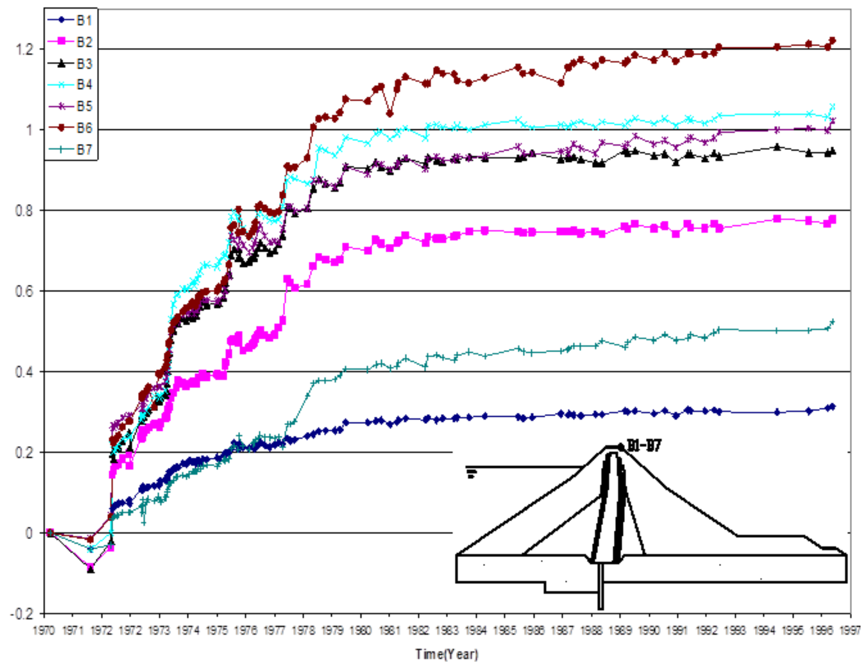


Fig. 6. Horizontal displacement of the points located on the down-stream edge of the dam's crown.

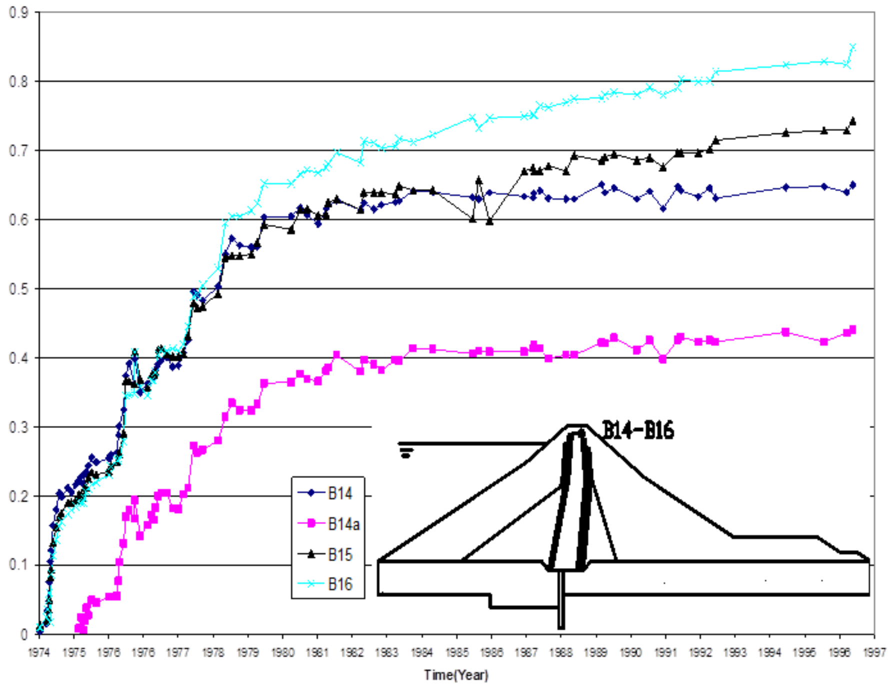


Fig. 7. Horizontal displacement of the points located on the clay core.

3. Numerical Modelling of Mahabad Earthfill Dam

3.1. FE software packages

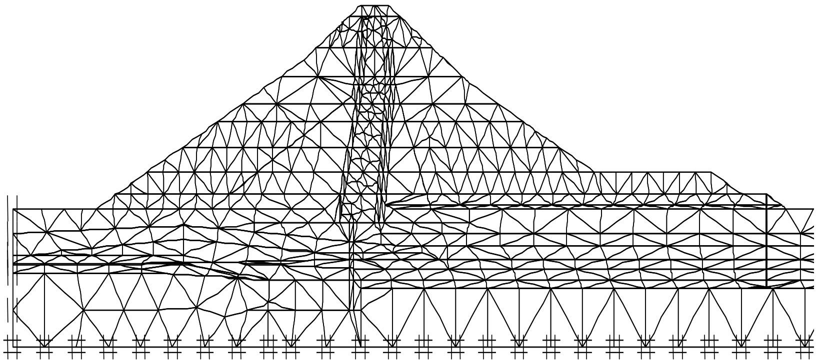
Plaxis is used for the analysis of deformation and stability of earthfill dams. It is capable of modelling the nonlinear and time-dependent behavior of the soil. Using the Plaxis, it is possible to model the interaction between the soil and the adjacent structure [15]. Sigma-W can be used for deformation and stress analysis in soil structures. The production and decay of pore water pressure can be modeled through linking the Sigma-W and Seep/W packages. Seep/W is used to model the pore water pressure distribution inside the soil and rock. It is useful for the analysis and design in not only geotechnical projects but also hydrological and mining projects. Seep/W is a comprehensive computer tool for seepage analysis which is capable of modeling both the saturated and unsaturated soils.

3.2. Modelling the construction phase

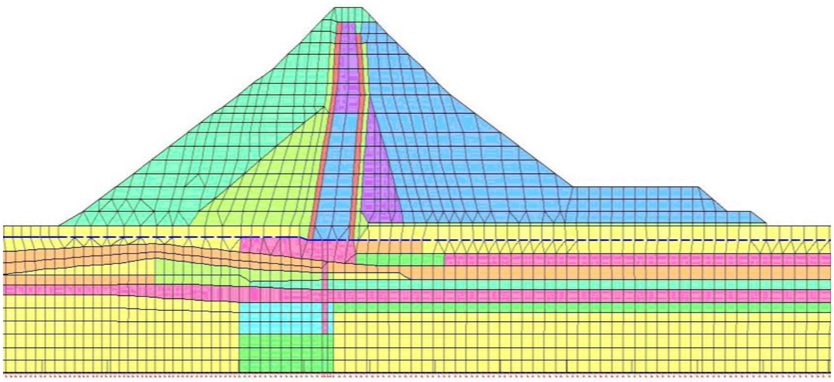
Primary section of the dam is shown in Fig. 8 and the computational stages are summarized in Table 2. When the construction was finished, all the construction induced deformations were eliminated and finally the design level codes were reached. It must be noted that this process eliminates only the deformations and it does not affect the existing stresses in the elements. The FE mesh generated by Plaxis and Sigma-W are shown in Fig. 9. As can be seen in this figure, triangular and four-nodded elements have been used in Plaxis and Sigma-W, respectively.

Table 2. Computational stages for construction phase of Mahabad dam.

Stage No.	Computation type	Drainage condition	Construction level	Water level	Construction time (days)
1	Plastic	Drained	1320	1314.5	29
2	Plastic	Drained	1325	1314.5	28
3	Plastic	Drained	1330.5	1314.5	22
4	Plastic	Drained	1336	1314.5	22
5	Plastic	Drained	1340	1314.5	18
6	Plastic	Drained	1346	1314.5	17
7	Plastic	Drained	1346	1314.5	249
8	Plastic	Drained	1352	1314.5	82
9	Plastic	Drained	1361.5	1314.5	94
10	Consolidation	-	1361.5	1314.5	256



(a)



(b)

Fig. 9. Finite element mesh generated in (a) Plaxis, (b) Sigma-W.

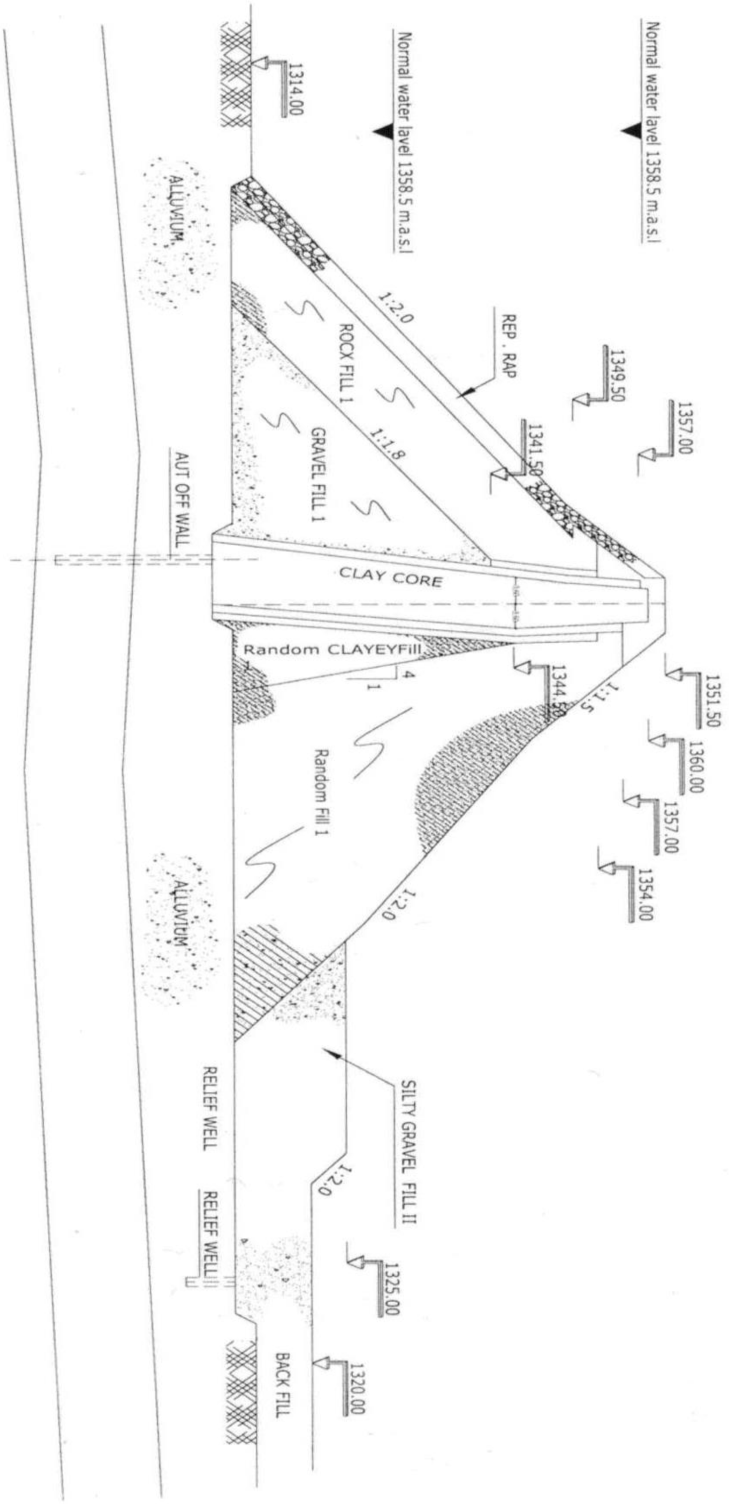


Fig. 8 Body section of the Mahabad dam at kilometre 0+300.

Selected behaviour models in Plaxis and Sigma-W are as follows:

In Plaxis, in order to model the rock materials and cut-off wall, linearly elastic model was used. Granular materials were modeled using Mohr-Coulomb behavior model. Soft-Soil-Creep behavior model was used for modeling the fine materials and clay core.

In Sigma-W, the rock materials and cut-off wall were modeled using the linearly elastic behavior model and Mohr-Coulomb behavior model was used for granular materials. Clay core and fine materials were modelled using Modified-Cam-Clay behaviour model.

The properties and parameters defined in both above-mentioned FE packages can be categorized as follows:

a) Parameters such as density, permeability, coherency, internal friction angle, modulus of elasticity, etc. which had been obtained through the experiments carried out during the construction of the dam.

b) Parameters required for each behaviour model which have not been determined experimentally. These parameters were initially assigned according to tentative relations and repeatedly revised based on the obtained results.

The diagrams showing the settlement of the dam versus the elevation of construction are of great importance during the construction. These diagrams were obtained separately by both Plaxis and Sigma-W and the results are shown in Figs. 10 and 11, respectively. As can be seen in these figures, the location of maximum settlement in all graphs is above the 50% of constructed segment (middle line) height. As the height of the dam increases, the distance from the middle line is increased. The first reason is inadequate lateral restriction which leads to stress release and increase of horizontal stress. Two of the major concerns about the geometry of dam are its steep slopes and relatively narrow crown. These problems have led to the decrease of lateral restriction. The second reason is the relatively low strength and stiffness of the down-stream shell materials.

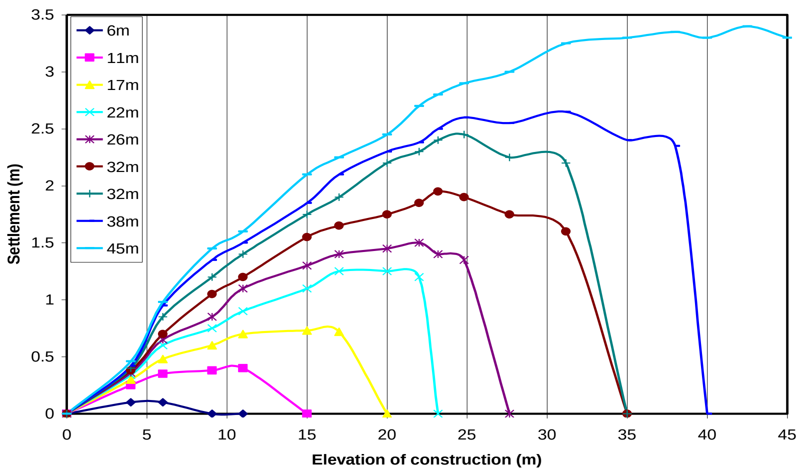


Fig. 10. Settlement profiles corresponding to the construction phase (Extracted by Plaxis).

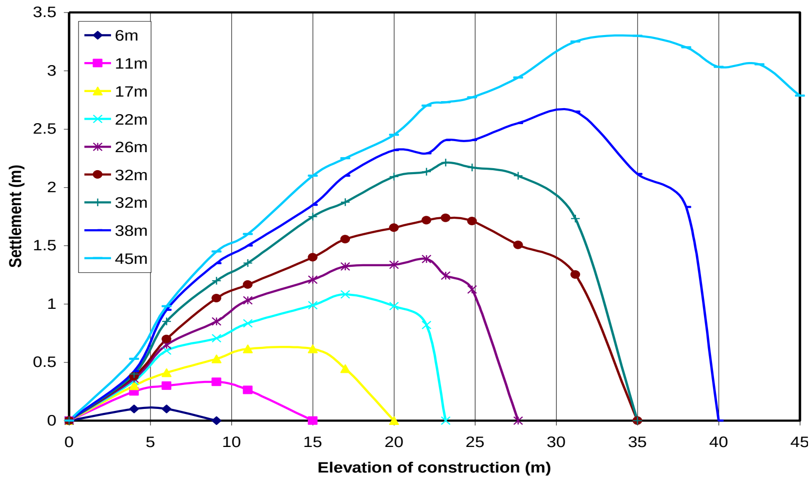


Fig. 11. Settlement profiles corresponding to the construction phase (Extracted by Sigma-W).

When the dam reached to the height of 30 m, the construction process was paused for 249 days. The increase in the expansion of the graphs in Figs. 10 and 11 is due to the consolidation settlement occurred during the days of construction pause. It can be concluded from the investigation of settlements that the maximum deflections during the construction have occurred at clay core and down-stream shell.

The magnitude of the dam deflections during the construction process indicates the inadequacy of the stiffness of gravel shell protecting the core, specially the down-stream shell. This leads to the release of horizontal stresses and consequently to inappropriate load transfer from the core to the shells. Shape of the deflections during the construction of dam shows that its geometrical shape is not very suitable.

3.3. Foundation settlement

As stated in Section 2, two settlement measuring tubes were installed in the foundation at kilometres 0+200 and 0+300. The settlement values predicted by Plaxis and Sigma-W are compared with the measured values at Table 3. Considering the complexity of the modelling process in Sigma-W, only 0+300 section was modelled by this software. As can be seen in Table 3, the difference between the measured and predicted values before the flooding is approximately 13%; while this difference is increased after the initiation of flooding. The reason is the lack of sleeves for settlement measuring tubes. In fact, the friction between the rod and the materials of the dam induces higher settlement in the rod [16]. It must be noted that although these settlement measuring tools are accompanied with a certain amount of error, but they represent very valuable information. As can be seen in Table 3, 58-72% of the overall foundation settlement occurred before the initiation of flooding. The corresponding percentage computed using Plaxis is 85%. Considering this value and according to Table 3, it can be concluded that after the beginning of flooding, only secondary consolidation settlement (creep) has occurred. The reason is the increase of pore water pressure due to flooding which leads to the decrease of effective stress inside the foundation. Hence, consolidation settlement does not take place, but the materials experience an amount of heave due to the decrease of effective stress. The decrease of effective stress leads to pre-consolidation of the foundation. The decrease in the pore water pressure during the discharge of reservoir leads to the increase of effective stress. But these effective stresses are smaller that the corresponding stresses before the beginning of flooding. Hence, no consolidation occurs and only the previously induced heave will be removed. The change of maximum effective stress inside the foundation is shown in Fig. 12.

3.4. Behaviour of the dam during the service

In order to control the surface settlements of the dam, three instrumentation points were considered. The first one was on the crown, the second one was on the dam's body at the down-stream and the last one was outside of the dam's body at the down-stream. Four years after the completeness of dam construction, observing the large amounts of settlement led to installing a system to control the settlement of clay core.

Table 3. Measured and predicted values for settlement of dam foundation.

Time spent after construction	Measured Settlement (m)	The percentage of total measured settlement	Predicted settlement (m)		The percentage of total measured settlement		Difference between the measured and predicted values (cm)	
			Plaxis	Sigma-W	Plaxis	Sigma-W	Plaxis	Sigma-W
Kilometre 0+200								
219	0.55	44	0.65	-	71	-	10	-
385	0.62	50	0.685	-	74	-	6.5	-
561	0.642	52	0.74	-	80	-	9.8	-
817	0.72	58	0.78	-	85	-	6.0	-
1137	0.8	64	0.8	-	87	-	0	-
2784	0.928	75	0.85	-	92	-	-7.8	-
7641	1.2	97	0.91	-	99	-	-29	-
10499	1.217	98	0.92	-	100	-	-29.7	-
Kilometre 0+300								
49	0.32	17	0.4	0.35	28	23	8	3
181	0.9	47	1.01	0.92	71	56	11	2
385	0.955	50	1.094	1.02	77	66	13.9	6.5
561	1.27	66	1.17	1.13	82	72	-10	-14
817	1.37	72	1.22	1.21	85	77	-15	-16
1889	1.54	81	1.28	1.30	90	87	-26	-24
2710	1.73	91	1.31	1.34	92	91	-42	-39
7567	1.88	98	1.4	1.423	98	96	-48	-46
10425	1.9	99	1.43	1.49	100	100	-47	-41

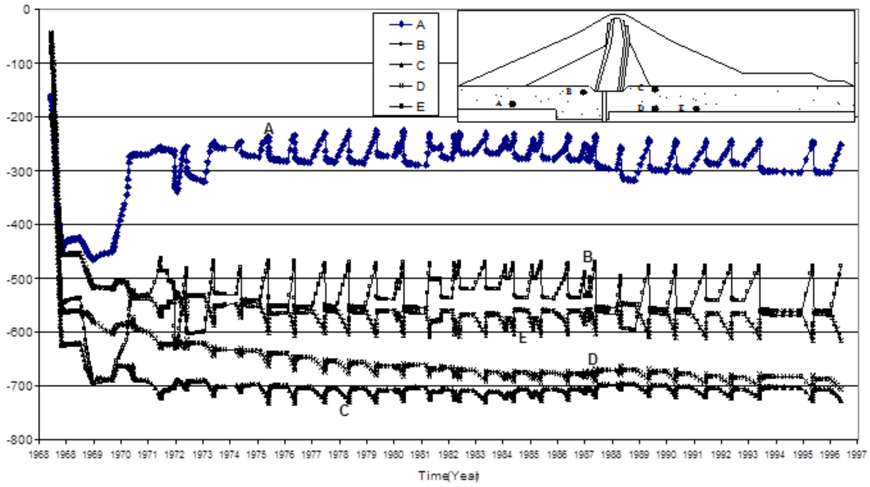


Fig. 12. Change of the maximum effective stress at the various points inside the foundation by time.

In Figs. 13-16, the values measured by this system and also by instrumentation installed on the crown are compared with the corresponding values predicted by Plaxis. As can be seen in Fig. 13, the difference between the measured and predicted values is relatively high. The main reason is the installation time of these points. The graphs of measured values are drawn assuming that the initial settlement is zero. Since the recording at these points has started four years after the completeness of dam construction, hence an initial error had been accompanied by the measured values and consequently led to a considerable cumulative error. However, the rates of the changes in the observed and predicted values are in reasonable agreement.

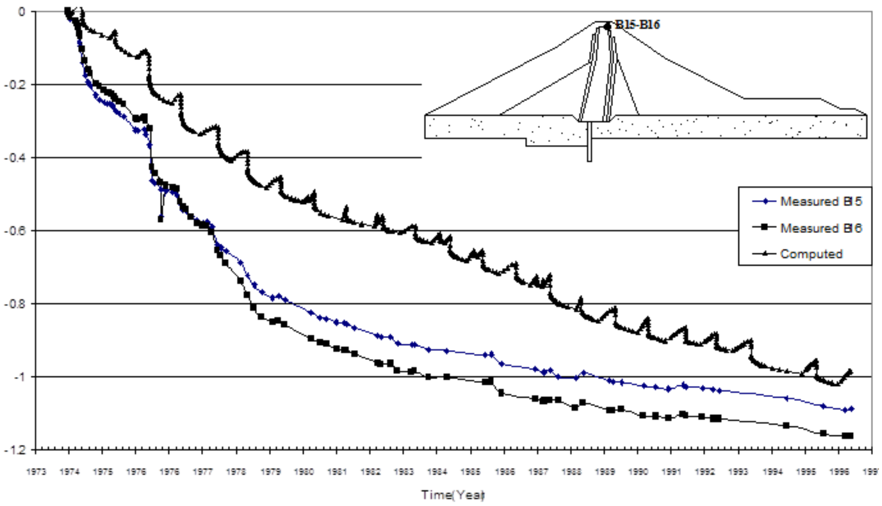


Fig. 13. Measured and predicted values for the settlement of clay core at kilometer 0+200.

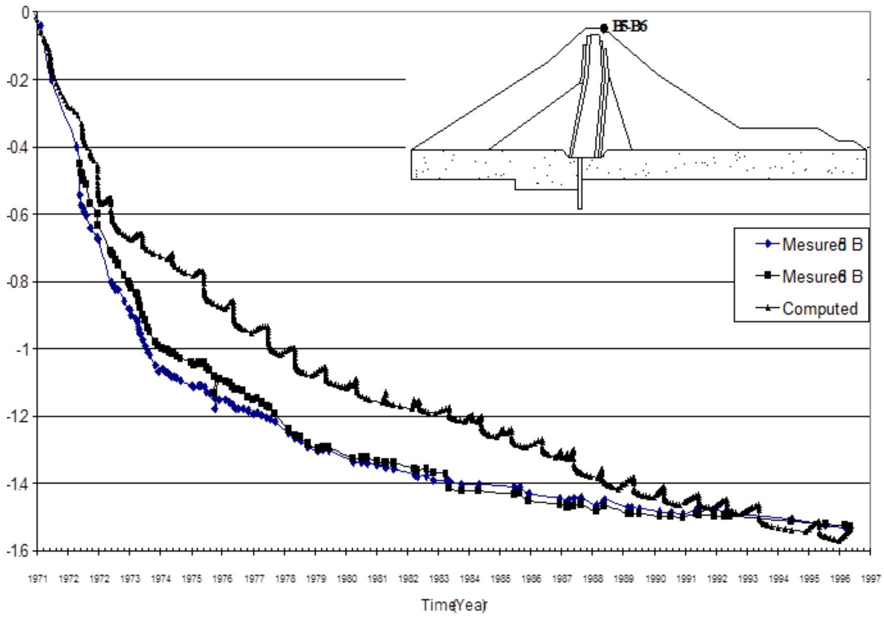


Fig. 14. Measured and predicted values for the settlement of dam's crown at kilometer 0+200.

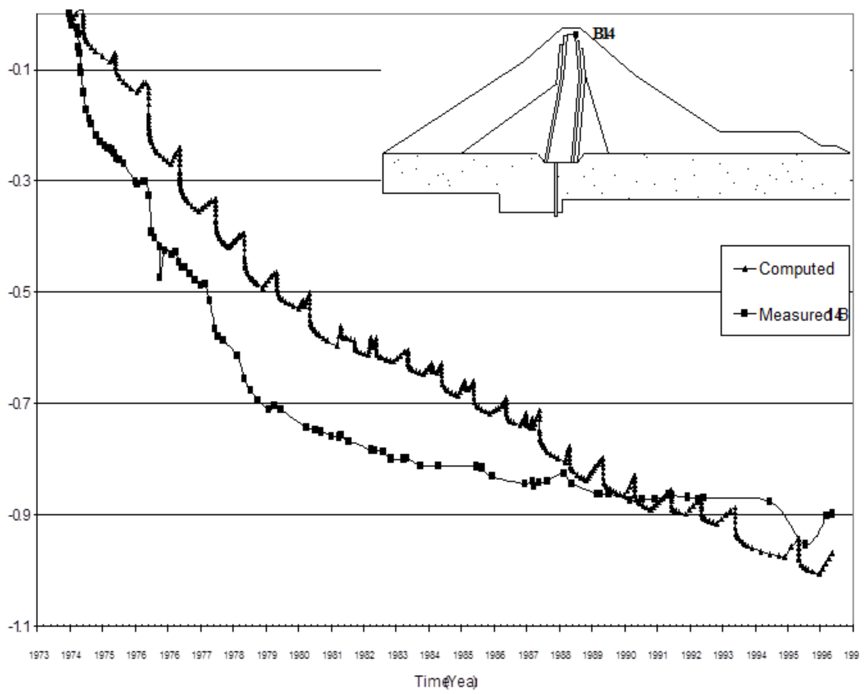


Fig. 15. Measured and predicted values for the settlement of clay core at kilometer 0+300.

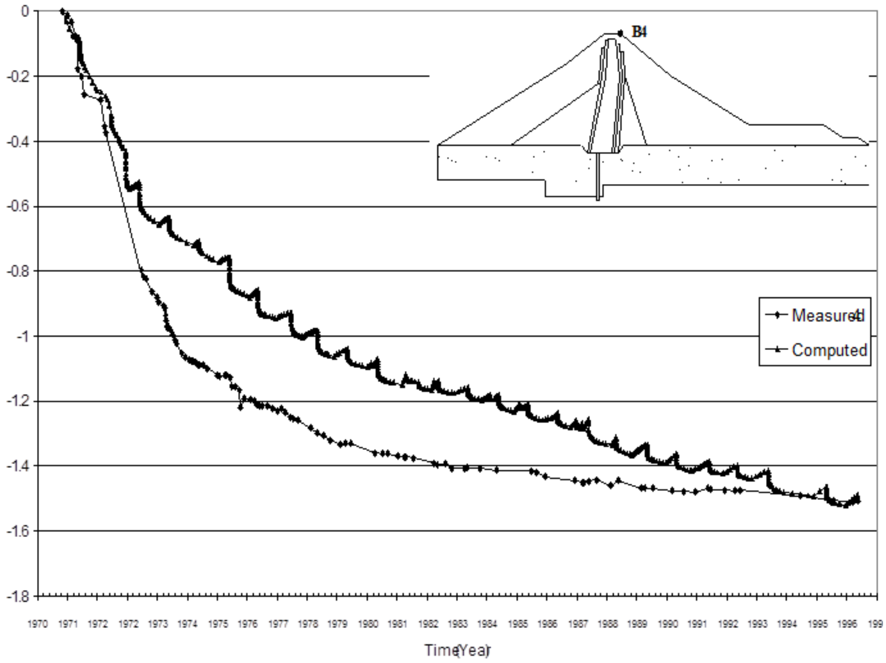


Fig. 16. Measured and predicted values for the settlement of dam's crown at kilometer 0+300.

It can be concluded from the figures showing the settlement of the dam's crown and core that the magnitude of displacements in the last years is very small relative to the initial values. However, a settlement of 10 cm (25 years after the construction of dam) is a considerable value.

It must be noted that the values of the settlement of the Mahabad dam are considerably high in comparison with the values corresponding to the similar typical earthfill dams.

In order to control the horizontal displacements of the dam, three points were instrumented at kilometres 0+200 and 0+300. In Figs. 17-18, the measured values at two of these points are compared with the values predicted by Plaxis. Gravel collapse analysis was not considered in numerical modelling. Hence, initial horizontal displacement of the dam toward the up-stream is not indicated in the analysis results. However, since the volume of the gravel is small, the corresponding deformation is also small and is inconsiderable in comparison with the following changes.

As can be in Figs. 17 and 18, there is a good agreement between the measured and predicted values. Quick downfall of the graphs for predicted results is due to the plastic analysis during the sudden downfall of reservoir's water level (discharge). It is assumed that when the water level is suddenly downfallen, the governing condition is un-drained.

As can be seen in Figs. 17 and 18, after 1979, the rate of the change in the horizontal displacements is substantially reduced and reached a stable condition.

It must be noted that the values of the horizontal displacement of the Mahabad dam are considerably high in comparison with the values corresponding to the similar typical earthfill dams.

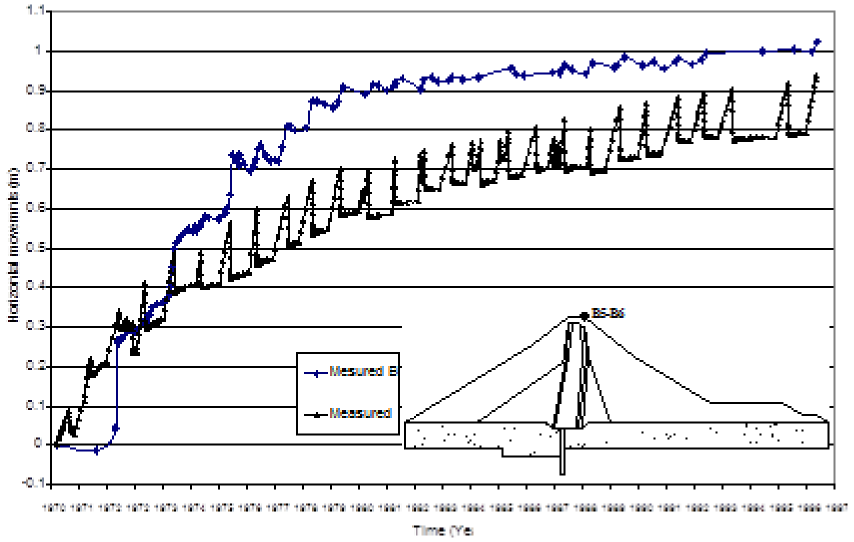


Fig. 17. Measured and predicted values for the horizontal displacement of dam's crown at kilometer 0+200.

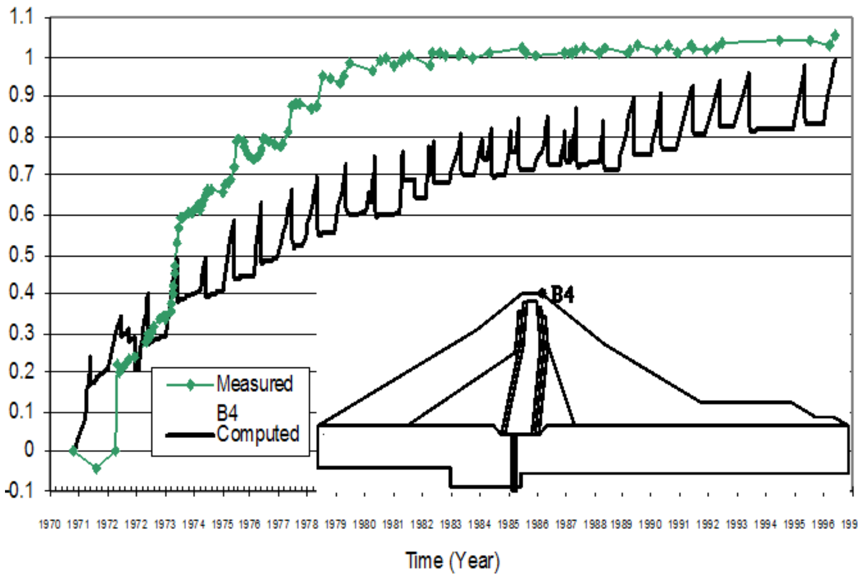


Fig. 18. Measured and predicted values for the horizontal displacement of dam's crown at kilometer 0+300.

4. Conclusions

In the present paper, instrumentation data and the numerical analyses were used for the monitoring of Mahabad earthfill dam, Iran. Numerical analyses were carried out using Plaxis and Sigma-W. Results can be summarized as follows:

- Considering the good agreement between the measured values and the values predicted by Plaxis and Sigma-W, it can be concluded that the numerical models developed in the present paper are accurate enough to be used for the analysis of earthfill dams.
- The maximum settlements in the clay core occur at the middle sections, and the minimum settlements occur at the edge sections corresponding to the rock restraints. This is because of the higher embankment at the middle sections.
- Most of the settlements in the different time intervals occur at the level of 1.2-1.3 of embankment depth. The reason is that the lower layers become consolidated as the embankment process progresses; and their corresponding settlement is continuously reduced. The upper layers are subjected to lower overload pressures. Hence the maximum settlement in these layers occurs at the middle (maximum) sections.
- The values of the Mahabad dam settlement are considerably high in comparison with the values corresponding to the similar typical earthfill dams.
- The magnitude of the dam deflections during the construction process indicates the inadequacy of the stiffness of gravel shell protecting the core (specially the down-stream shell). This leads to the release of horizontal stresses and consequently to inappropriate load transfer from the core to the shells. The shape of deflections during the dam construction shows that its geometrical shape is not very suitable.
- According to the obtained results, after nearly 30 years of service, most of the constructing materials of the dam experienced their consolidation settlement. Hence, the settlement of the fine grained materials of the dam body due to the change of reservoir's water level will be small.
- Parameters having an effective role in the occurrence of high settlements in Mahabad dam are: (a) the type of materials used for the down-stream shell and their low stiffness, (b) inappropriate geometrical shape of the dam, (c) low compaction of the up-stream shell and fine grained core and consequently its high consolidation settlements.

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