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Monitoring of Concrete Face Rock fill Dam

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ABSTRACT

The Shahr-e-Bijar dam is a 96.5 m high and 430 m long concrete faced rock-fill dam with reservoir volume of 105 million m^3. Construction of the dam was completed in April 2014. Throughout the dam construction, comprehensive monitoring had been carried out to verify the dam behavior so that stability and safety of the dam were ensured. Monitoring of the dam behavior was based on the measurement data of instruments which installed in the dam body and the foundation as well as daily visual inspection. In this study, a two dimensional finite element analysis of SEB dam is carried out and the computed displacements and internal stresses compared with those measured in situ by the instrumentation. The rock-fill material is represented by the hardening soil model which is a modified version of Duncan and Chang's hyperbolic model. The maximum recorded construction settlement is around 0.80 m at the maximum cross section and around 0.50 m at the sections on the abutments which correspond to 0.9% of the dam height. The deformations will further increase with first impounding and rising reservoir to full supply level and due to creeping of the rock-fill. The results of different instruments which are used for the settlement monitoring of the dam are shown that the accuracy of hydrostatic settlement cells is more than other instruments and the measured settlements by the magnetic plates around the inclinometer's tubes are usual below the actual settlement. The stress arching within the dam body and abutments is recorded as 50 % in the middle of dam body and 60 % in the near of the banks. Also, during the construction, the shrinkage is mobilized within the dam body along the axis of dam at the first stage of construction and then by increasing the overburden's height, the expansion is recorded and increased up to the end of construction. The results are shown 65 mm as the maximum expansion which is mobilized at the middle part of the dam body.

Key words: Instrument challenges, Monitoring, Concrete Faced Rock-fill Dam, Finite Element Analysis, Dam Behavior, Safety.

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1. INTRODUCTION

Today, concrete faced rock-fill dams (CFRD) are very popular all over the world, especially in regions, which receive heavy rain and where impervious soil reserves are insufficient. In the current state of the art, the design of a CFRD is based on experience and engineering judgment (1). CFRDs are gaining a worldwide recognition as the most economical type of dams to be constructed in extreme northern and sub-Antarctic regions. Use of the rock-fill material, which is not sensitive to the frost action and the construction technology allow lengthening the construction season. The total duration of the construction of CFRDs with regard to the total duration of construction of earth dams is on average reduced by one year. Since, these constructions are important structures; their behavior should be estimated realistically for both construction and reservoir filling stages. Finite element method is one of the available tools used in the prediction of structural behavior. Deformations of CFRD dams start occurring during the construction. 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 the material. Deformations may also be influenced by deformations of the foundation, by transfer of stresses between the various zones of the dam and by other factors. After the construction is completed, the considerable movements of the crest and of the body of the dam can develop due to pressure of water during the first filling of the reservoir. The load of water and defor- mations of the rock-fill will be forced on concrete slab to deform. The concrete slab acts as an impervious membrane and any development of cracks in the slab would allow for water to penetrate the rock-fill of the dam and cause the structure to weaken or even loose stability. According to the working state, force distribution and hydraulic features of CFRD, proper zoning of dam filling material is carried out to take a full utilization of the material from structure excavation and to reduce the investment under the condition that the safety of operation is ensured. After filling of the reservoir, the rate of movement in the dam and in the concrete face generally diminishes with time, except for variations associated with periodic raising and lowering the level of the reservoir. In classic CFRDs where the concrete face is constructed after the end of construction of the rock-fill embankment, it is very important to estimate the displacements of the concrete face during the filling of the reservoir and to verify whether these displacements are lower than displacements compatible with the structural integrity of the concrete face. Most of the constructed CFRDs rest on the bedrock. However, there are some CFRDs constructed on soil foundations. Foundation conditions of the planned constructions call for studies to determine the range of possible movements of the concrete face slab during the construction of the dam and especially, during the filling of the reservoir (2). 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 engineering profession, is the need for better understanding of basic design concepts, stress-deformation characteristics, and geotechnical characteristics of soil and rock fill. The development of prediction methods, which allow a determination of deformations and stress distribution and comparison of predicted values with observed, constitutes very valid tools to control safety. The key point in the analysis is down-to-earth modelling of the stress-strain relationships of rock-fill materials, preferably based on tri-axial test results. Considering the particle sizes of rock-fill material, up to 1.2 m diameter, the difficulty in obtaining experimental data, which in our case is lacking, becomes obvious. Although limited, the available tri-axial data in the literature indicate that rock-fill materials pos- sess highly stress dependent, inelastic and non-linear stress-strain relationships (3-5). Among the material mod- els used in the available studies carried out in recent years, Duncan and Chang's (1970) hyperbolic model is probably the most common (6). The hyperbolic model has been uti- lized in a number of similar research successfully (7-9). In the present study, the ''hardening soil model'', which is essentially a modified implementation of the hyperbolic model available in FLAC, is used to represent the rock-fill behavior (10, 11).

2. DAM CHARACTERISTICS

Shahr-e-Bijar (SEB) dam is a 96.5 m high and 430 m long concrete faced rock fill dam with reservoir volume of 105 million m^3. Construction of the dam was completed in April 2014 and impounding of the reservoir will be started during the next months. The dam is located in near of Rasht city, 40 Km, in the north of Iran. The main purpose of the dam is the water storage. Construction of the dam embankment was started in 2008 and finished in 2014. After the completion of rock-filling, the construction process had been given a halt for about 0.5 year until the major part of the settlement of rock-fill embankment was completed. The construction of concrete lining was continued after there. SEB dam is 96.5 m high from the river bed and the side slopes are 2.0:1 and 1.6:1 (H:V) in the upstream and downstream embankment faces, respectively. The crest extends 437 m. The valley is 165 m wide and abutment slopes, on the average are 30 and 45 for the left and right abutments, respectively. Figure 1 shows the typical cross-section with different zones indicated.

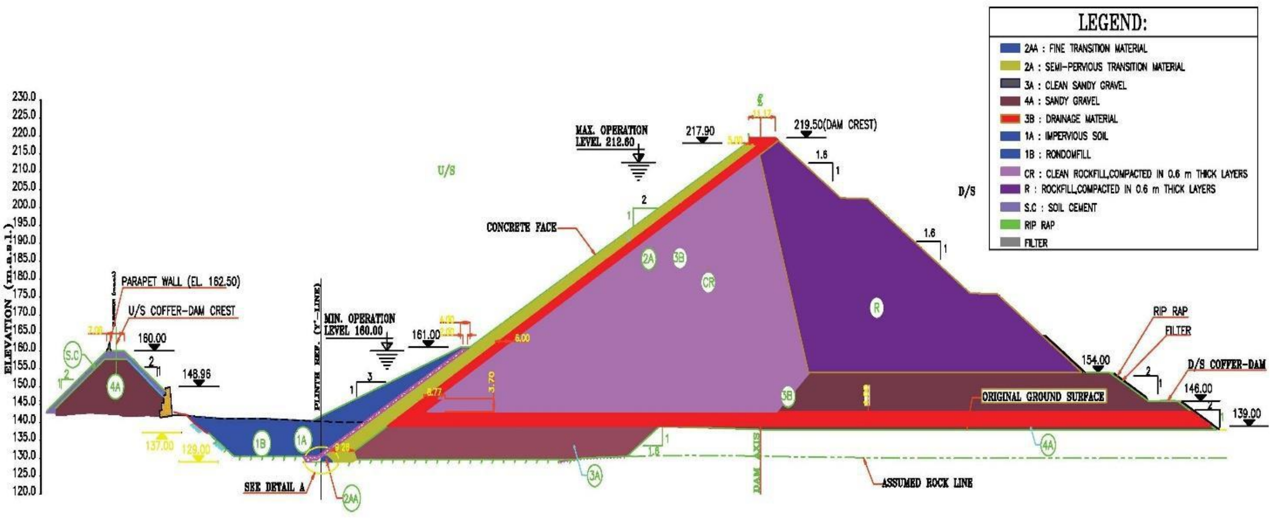

Figure 1. Typical cross section of SEB dam

Basic geological units in the dam site are sandstone, siltstone and limestone. Sandstone is the most common type of rock found in the region on which the dam rests. The details of zoning and construction are given in Table 1.

Table 1. Materials and construction details of SEB dam

	Material				Construction techniques	
		Particle Sizes			Compaction	
Zone	Type	D max (Mm)	Sand (%)	Fines (%)	Layers (m)	(12 & 15 t vibratory roller)
1A	Impervious fill	25	>50	>30	0.15	4 passes (static)
1B	alluvium	400		<20	0.3	4 passes
2A	Sieved rock (or alluvium)	75	35–55	3–10	0.15	6 passes (4 static + 6 dynamic)
2AA	Filter	40	38–60	5-12	0.15	6 passes
CR R	U/S & D/s Rock fill	300	16–35	<7	0.4	6 passes+150 lit/m^3 water
3B	Drainage	100	16-38	<3	0.25	6 passes

3. NUMERICAL SIMULATION

A modified version of Duncan–Chang model has been utilized in the analyses. Since no large-scale tri-axial test results are available, pertinent works from the literature (Saboya and Byrne, 1993) with other experience in Iran are referred in selecting the hardening soil model parameters (9). Table 2 shows the selected parameters used in the numerical simulations. In hardening model, the secant modulus for primary loading corresponding to 50% of shear strength q_f is defined by Schanz et al. (1999) (Figure 2) (11).

Table 2. Range of hardening soil parameters used in the numerical simulation.

Zone	γ [kN/m^3]	Φ_o	$\Delta\Phi$	K	n	R_f	K_{ur}	K_b	m
Transition	22	45	6.6	1000	0.40	0.85	2000	450	0.15
Main Rock fill	22	42	6.5	1100	0.35	0.82	2200	600	0.10
Secondary Rock fill	22	40	6.4	850	0.25	0.80	1700	400	0.05

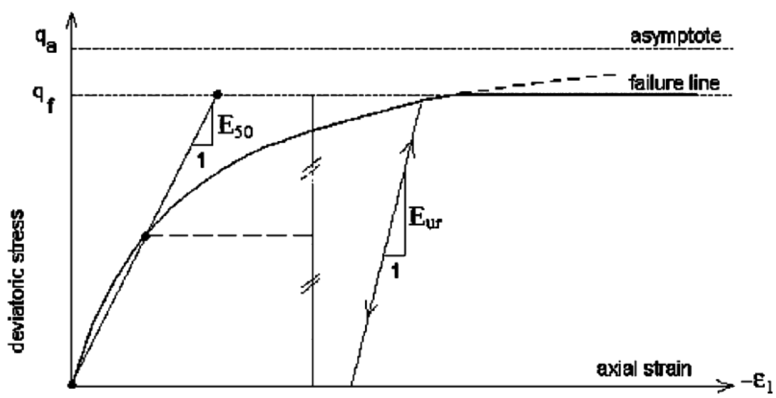

Figure 2. Hyperbolic stress-strain curve (11)

Zones 2A and 2AA shown in Figure 1 are not expected to have a significant effect on the overall dam behavior and have not been included in the finite element analysis. The rock foundation of the dam is simulated base on the geotechnical laboratory tests. The analyses are carried out considering end of construction (EoC) conditions and are compared with the measured data by the installed instruments within the embankment. The dam is simulated by assuming 2-D plane strain conditions. The finite element software, FLAC and GEOSLOPE-Sigma are employed. The rock fill embankment is modelled by 15- node triangular elements with the hardening soil as the material model. These elements have 12 interior stress points located at different coordinates from the element nodes where displacements are output. The concrete slab is model led by five nodded linear elastic beam elements. These elements have 50 cm thicknesses and 8 stress points. The material parameters of the concrete are shown in Table 3.

Table 3. Linear elastic model parameters of the concrete slab

Material	E (MPa)	d (m)	c (kN/m^3)	m
Concrete	28,500	0.50	23.50	0.20

For a preliminary analysis, each construction stage was represented by a 5 m thick layer. It was observed that reducing the layer thickness renders the simulation better while extending the runtime. Finally, a layer thickness of 5 m was decided as agreeable. The last 6 m's of the dam from the top was not included in the mesh, since the parapet wall unnecessarily complicated the geometry, never the less the weight of the wall was taken into account in the model as a surcharge of 100 kPa.

4. INSTRUMENTATION OF THE DAM

Because it was the first example of its type in north of Iran, SEB dam was extensively instrumented in order to monitor the performance. Several types of instruments were used for monitoring the behavior. The instruments are listed below which are used for monitoring the dam at EoC.

1. Hydraulic settlement cells located within the rock-fill (SC1-31)
2. Total pressure cells, located within the rock-fill embankment (PC1-7)
3. Vertical Inclinometers, located vertically in 3 main section of dam, (IS1-3)
4. Magnetic settlement plates with 3 meters space around the vertical inclinometers,
5. Electrical piezometers around the grout curtain and dam's foundation (EP 1-30)
6. Cross arm extensometers within the embankments along longitudinal in three different level (150, 175 and 200 m.a.s.l.)

It is notable that one in-place inclinometer, 37 Electrolevel cells and 12 three-dimensional joint-meters will be evaluated to monitor the rotation, displacement and movements of the concrete slab in the future.

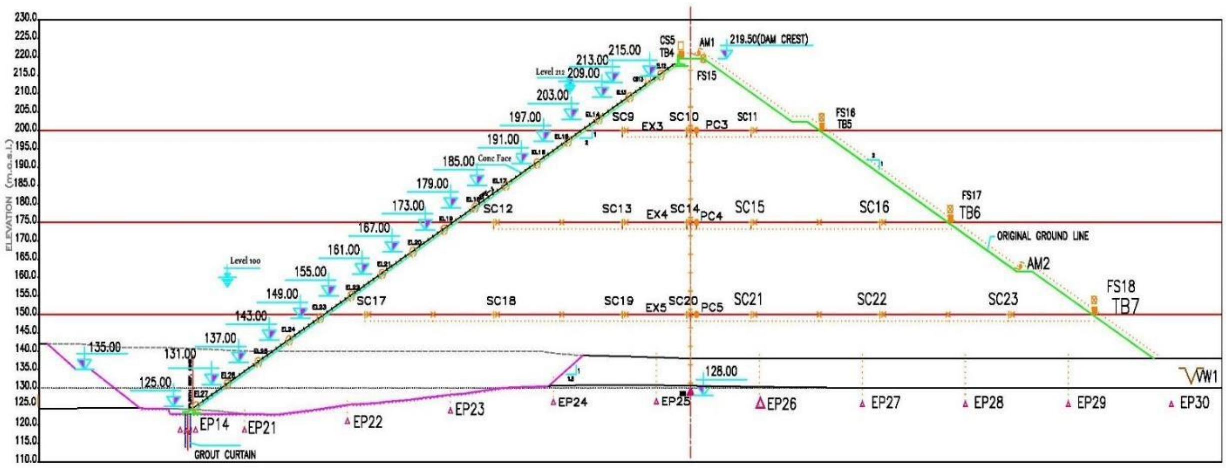

Figure 3. Instrumentation in the rock-fill embankment (Max. cross section)

As listed above, a total of 31 hydraulic settlement devices, 7 earth pressure cells and 3 inclinometers with magnetic settlement plates were installed at 3 cross-sections and three different elevations to control the behavior during the construction of dam. The locations of the devices at the maximum cross-section of the dam are shown in Figure 3. In the continue, the measured data of the instruments will be discussed and compared with the numerical simulation results at the end of construction for SEB dam. It is notable that this dam recently is going to first impounding.

5. SETTLEMENT

Dam deformations during construction were measured with hydrostatic settlement cells, fixed embankment extensometers and settlement gauges with combined inclinometer tubing. These instruments provided data of deformations along 3 sections within the dam body. The maximum recorded construction settlement is around 0.80 m at the maximum cross section and around 0.50 m at the sections on the abutments. Measured settlement at the different depth of highest cross section is shown as Figure 4 during EoC.

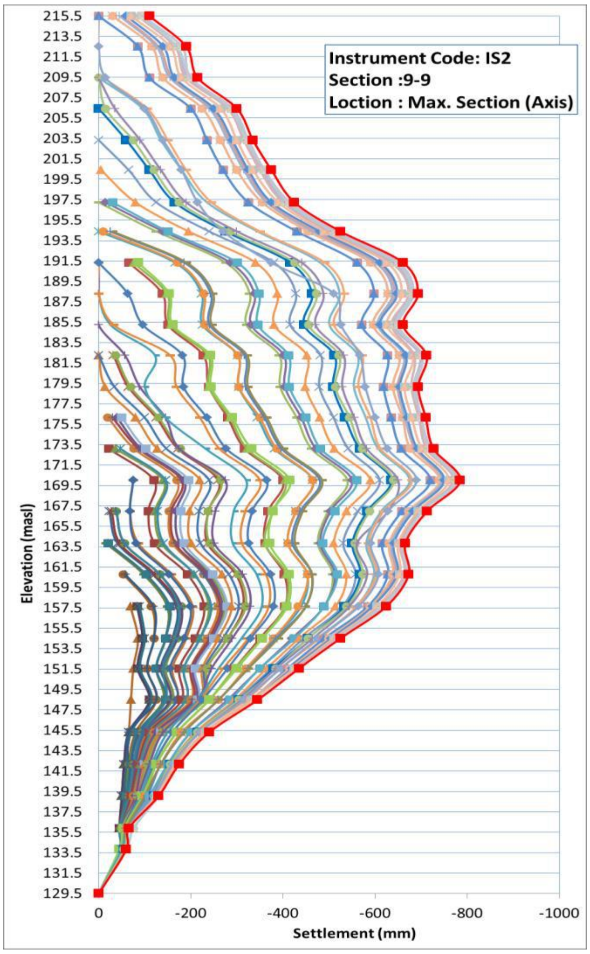

Figure 4. Measured settlement at the highest cross section during the construction

The construction settlements measured with the instruments were used for calculating the deformation moduli during construction E_{rc}:

$$E_{rc} = \gamma H d / s \qquad (1)$$

γ = unit weight of fill above settlement plate,
H = height of fill above settlement plate,
d = thickness of fill below settlement plate,
s = recorded settlement of the settlement plate.

The back-calculated deformation moduli during construction are in the range of 60 MPa (upstream side) to 45 MPa (downstream side). Using these deformation moduli the dam settlements during construction were back-modelled using an elasto-plastic material model. The results are in good agreement with the monitored data.

Figure 6. Vertical stress distribution in the highest cross section at EoC. The calculated dam settlements comply well with the measured settlement as shown on Figure 4. According to numerical simulation, total settlement at the end of construction is defined by the linear elastic model equal to 0.85 m and 1.2 m by the Duncan-change hyperbolic model (Figure 5).

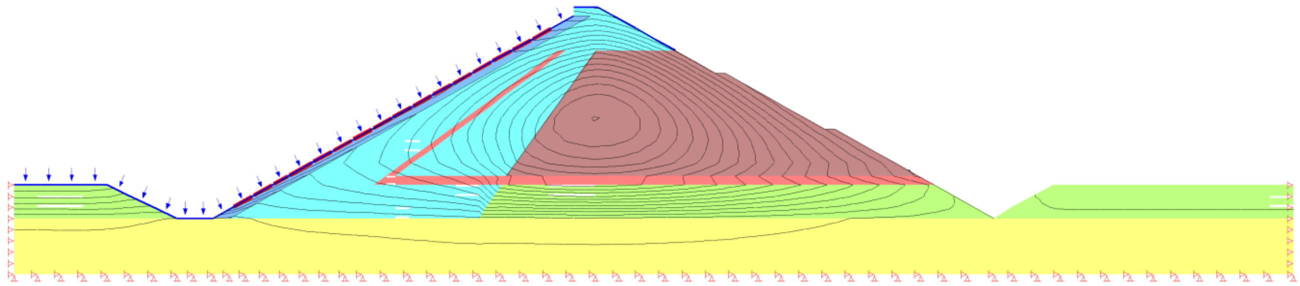

Figure 5. Calculated settlement at the highest cross section of SEB dam at EoC; (a) Linear elastic model (b)Hyperbolic Model

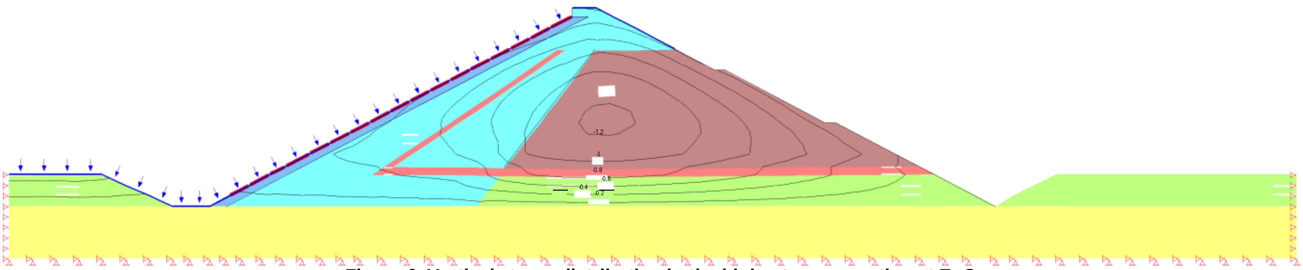

Figure 6. Vertical stress distribution in the highest cross section at EoC

In general view, the maximum dam settlement at the end of construction is about 0.8 m which corresponds to 0.9% of the dam height. Post-construction dam deformations generally occur due to the increased load from the reservoir, creep deformations, breakage of rock fill particles and softening of the rock fill due to wetting and saturation. The total maximum settlement at the dam crest was reported about 75 mm. During the SEB construction, three main parameters were effective on the dam settlement. Void ratio of the compacted rock fill, Shape factor and breakable or creep capacity of the used materials to embank of dam were very effect on the mobilized deformations. According to the monitoring monthly and inspection daily, it is found that the void ratio is the main effective parameter on the total settlement of the dam. The results of different instru- ments which are used for the settlement monitoring of the dam are shown that:

1. Accuracy of hydrostatic settlement cells is more than other instruments.
2. Measured settlements by the magnetic plates around the inclinometer's pipes are usual lower than the actual settlement.
3. Reading errors of the hydrostatic cells are more than other cell.

6. TOTAL STRESSES

The comparison of predicted (Figure 6) and observed (Figure 7) total stresses for SEB dam at EOC is shown that the simulations and readings are similar from the trend view as well. But the theoretical value (overburden weight) and the measured data are not close together. As shown in Figure 7, the similar value is recorded for the measured vertical stress and the theoretical pressure at the first stage of EoC, but the difference of these pressures is increasing by the completion of dam body. The inclination of total pressure cells (from horizontal line) due to the construction displacement or rock fill creep movements is the main reason. To define the correction coefficient of stresses for SEB dam, the engineer is proposed the physical model of the dam to measure laboratory stresses path and then back-calculate the stresses in the filed condition. The stress arching, transfer the stresses from the dam body to the neighboring materials such as the abutment's rock, is recorded as 50 % in the middle of dam body and 60 % in the near of the banks.

7. EXTENSOMETERS

To measure the lateral movements within the dam body along the valley, 3 longitudinal extensometers are installed at three levels (151, 175 and 200). Figure 8 illustrate the photo of a fix installed point of this instrument as an example and its recorded data. As can be seen from the figure, during the construction, the shrinkage is mobilized at the first and by increasing the overburden's height, the expan- sion is recorded and increased up to the end of construction. The results are shown that lateral displacement equal to 65 mm as the maximum expansion which is mobilized at middle monitored elevation (175 m.a.s.l.). At the upper elevation, maximum expansion movements are limited to 34 mm. The upstream view of SEB dam at the end of construction with the above mentioned elevations is shown in Figure 9. Generally, the lateral movements within the dam body are lower than the predicated values. Base on the simulations, lateral movement should be mobilized in the lower part (1/3 H) of the body more than other parts and the maximum movement is calculated about 100 mm. Comparison of the calculated and recorded data means that 65 percent of the predicted horizontal movement is actually mobilized.

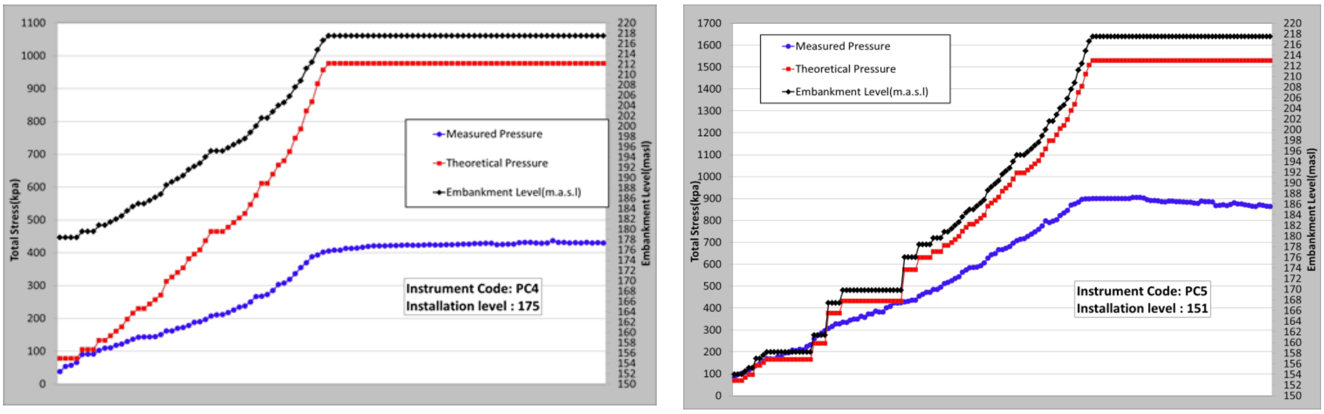

Figure 7. Measured total stresses at the highest cross section during the construction

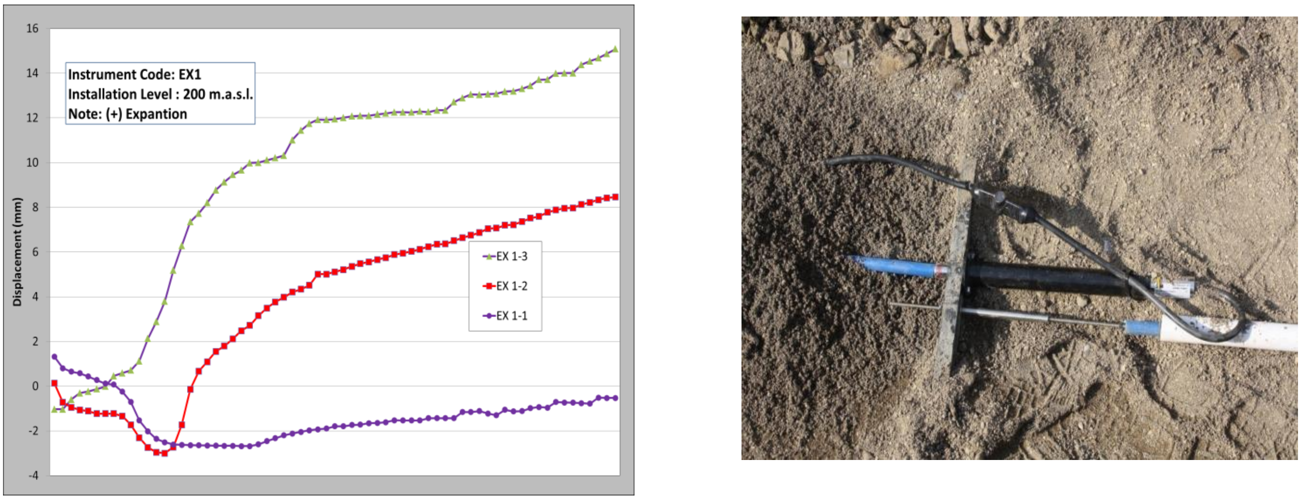

Figure 8. Measured lateral movement in the dam body at 200 m.a.s.l. by the extensometer

Figure 9. Upstream view of SEB dam

8. CONCLUSIONS

In this study, a two dimensional finite element analysis of Shahr-e-Bijar (SEB) dam is carried out and the computed displacements and internal stresses compared with those measured in situ by the instrumentation. The rock-fill material is represented by the hardening soil model which is a modified version of Duncan and Chang's hyperbolic model. The monitoring system of SEB dam has provided useful information on the deformation behavior of the dam during construction and the end of construction. The maximum recorded construction settlement is around 0.80 m at the maximum cross section and around 0.50 m at the sections on the abutments. According to numerical simulations, total settlement at the end of construction is defined by the linear elastic model equal to 0.85 m and 1.2 m by the Duncan-Change hyperbolic model. In general view, the maximum dam settlement at the end of construction is about 0.8 m which corresponds to 0.9% of the dam height. The deformations will further increase with first impounding and rising reservoir to full supply level and due to creeping of the rock fill. Based on deformation monitoring at the downstream part of the dam, still a high rate of creeping is noticed. The results of different instruments which are used for the settlement monitoring of the dam are shown that the accuracy of hydrostatic settlement cells is more than other instruments and the measured settlements by the magnetic plates around the inclinometer's tubes are usual below the actual settlement. The similar value is recorded for the measured vertical stress and the theoretical pressure at the first of end of construction, but the difference of these pressures is increasing by the completion of dam body. The inclination of total pressure cells (from horizontal line) due to the construction displacement or rock fill creep movements is the main reason. The stress arching within the dam body and abutments is recorded as 50 % in the middle of dam body and 60 % in the near of the banks. During the construction, the shrinkage is mobilized within the dam body at the first along the axis of dam and then by increasing the overburden's height, the expansion is recorded and increased up to the end of construction. The results are shown that 65 mm as the maximum expansion which is mobilized at the middle elevation of the dam body. At the upper part of dam, maximum expansion movements are limited to 34 mm. Comparison of the calculated and recorded data means that 65 percent of the predicted hori- zontal movement is actually mobilized.

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AUTHORS CONTRIBUTION

This work was carried out in collaboration among all authors.

CONFLICT OF INTEREST

The author (s) declared no potential conflicts of interests with respect to the authorship and/or publication of this paper.

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