CONSTRUCTION DEFORMATION OF THE GELEVARD
CONCRETE FACE ROCK FILL DAM–IRAN

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Abstract. The Gelevard dam which is located in northern part of Iran, is a concrete face rock fill embankment type (CFRD), with maximum height of 110 m and 280 m crest length. The rock fill dam body had recently finished and contractor has commenced construction of concrete face by starter slabs. The performance of dam during construction was monitored though monitoring instruments that were installed in three sections, representing the right abutment, at the maximum height section in the river bed and last section in the left abutment. For vertical and horizontal displacement of the rock fill body, 2 vertical inclinometers with mounted circular magnet plates on them, were installed in dam body, one is at mid part of dam, and another in downstream dam shell, in each chosen cross sections. Also hydraulic settlement devices were installed in two elevations of dam body. The largest vertical displacement registered in the central part of dam body, have reached about 120 cm. Interpretation and analysis of the results shows some deviations from design values for which settlement modulus was considered about 100 Mpa, whereas by using in-situ rock fill this modulus is varying between 40 to 70 Mpa causing these deviations. This paper represents the results of dam body deformation during construction.

1 INTRODUCTION

The Gelevard dams is located in the northern part of Iran, Mazandaran province, about 45 km apart from Neka city, on the Neka river and its middle point situated on the coordination of 36.59 N and 53.61 E. The project belong to Mazadaran regional water authority. Engineering works including final and detailed design and construction to EV.Yol-Ab-Niru joint venture based on design and build (D.B) contract. In figure (1), general location of dam site is shown. The main dam is a concrete face rockfill dam with maximum height of 110 m, crest length of 280 m and rockfill volume of 2,250,000 m³.

2 TOPOGRAPHY AND GEOLOGY OF DAM SITE

The dam site is located in a unsymmetrical U shape and relative narrow valley, with a
river bed width of about 60 to 90 m and valley width for about 350 m. The rock in both bank is covered with 1~5 m top soil in which the jungle trees grown in their slopes. The left bank has approximately uniform slope of 42°. The slope of right bank under ele. 710 m is about 50° above this level about 35°. The river bed is covered by alluvial deposit (sand and gravel) with average thickness of 10-15 m, which was removed from all dam foundation. Foundation and abutments of dam are composed of rock mass limestone of Lar formation. The Lar formation include of two units of limestone, light and gray limestone (LW and LG). LW unit has outcrops in both slopes of the gorge and in the vicinity of dam axis, however the wood land trees cover prevents it from being seen. Therefore, its natural view and outcrop property could not be described appropriately.

These limestone include milky white, fine grained, crystallized and dense limestone with average to high compressive strength with highly weathered condition. LG unit is located in the lower elevation of the dam site gorge, and consists of the hard gray colored well-bedded limestone or dolomite limestone which is introduced as LG unit. An extensive karst area (KK unit) developed approximately at lower part of (LW). This zone has various thickness and slope in different part of dam site. In karst area many dissolution channels and joints are filled with fine material. Although numerous cavity developed in karst area, some cavity filled with compacted fine material having high plasticity such as fat and stiff clay, and marly clay. The LG unit is located under the karstic unit (KK unit) it can be seen at the lower slope and its bedding can be clearly identified. The Shemshak formation is located under LG unit of Jurassic Lar formation. This formation consist of interbedded layer of shale siltstone and sandstone with coal
intercalation.
Some thin layer of coal are reported in dark shale at this formation.
The outcrops of Shemshak formation has not been seen at any place of dam site.
Fig. 2 shows the valley shape and the general geology conditions of longitudinal section of dam.

3 DESCRIPTION OF DAM SECTION:
The initial maximum section of the dam is shown in fig (3), the dam fills is constituted by 8 zones.

(1A) : Non-cohesive silty material .
(1B) : Coarse- random fill material .
(2A) : Cushion layer with max size = 40~80 mm
(2B) : Transition layer with max size = 100~200 mm
(3A) : Main rockfill with max size = 200~800 mm
(3B) : Downstream rockfill with max size = 200~1000 mm
(3C) : Middle rockfill with max size =75~400 mm

The rockfill materials came form the rock borrow area in downstream side of dam about 800 m from of it. Due to environmental consideration and limitation the contractor could not change the location of borrow area to gain rockfill with good quality,. Because based on the results of two boreholes in rock borrow area, two layers of extensive karstic rock mass were laid down. It is anticipated that the excavated rock mass need to be processed for rockfill or the dam body zoning, should be revised.
Therefore, after commencing rock excavation for dam body construction the zoning of dam body adjusted to the real condition of borrow area, thus we confronted with rockfill of 15 to 25% fine material from sieve No. 4.
Fig (4), shows the final dam zones. The rockfill used in dam body are limestone from Lar formation.
Main average parameters of rockfill are presented below:
Bulk specify gravity of the rock solids = 2.72
Water absorption (%) = 2
Los Angeles (%) = 27
Soundness (%) = 4
Unconfined compressive strength, dry (MPa) = 40-100
Unconfined compressive strength, saturated (MPa) = 25-60

4 DISPLACEMENT MONITORING DESIGN

The performance of dam during construction due to rockfill quality, and other technical purpose were monitored by instruments that installed in three section, representing the right abutment (Ch. 0+190), at max. height section in the river bed (Ch. 0+130) and last section in the left abutment, shown in the fig (5). Vertical displacement (settlement) are being monitored through hydraulic cells settlement each at about 35 m and magnet plates which are mounted on vertical inclinometer at each 6 m distance. Also horizontal inclinometer with 60 m length are installed at El. 660 m of sections 0+130 and 0+190.
The horizontal displacements are measured in the same position of the settlement cells in the
maximum section and also section 0+190, both at El.680.
For measurement of displacement along dam axis a multiple rod extensometer is installed between two bank at El.680.
In this paper only settlement results are presented (for rockfill placing in 3 years).

Figure 5. Instrumented cross sections of dam body.

5 MONITORING OF SETTLEMENT

As above mentioned, construction settlement have been measured through hydraulic cells and magnet plates which are mounted on vertical inclinometers. According to the results from these instruments, maximum settlement during construction of 100.0 cm, occurred at El.685 at station of 130 based on hydraulic settlement cell results, maximum settlement is about 120 cm at El.680 in downstream shell about 20 m from dam axis at station 0+190.
In fig (6), the variation of measured settlement of magnet plates with depth on inclinometers installed at center line of instrumented stations (0+70, 0+130 and0+190) are shown.
Also in fig (7) variation of measured settlement in each magnet plate with reading time are shown.
Measured settlement through hydraulic settlement cell at El. 680 in station 0+130 and 0+190 are shown in fig (8).
Unfortunately due to some malfunction in horizontal inclinometer the measured settlement in the installation area are very small and not reliable. A 3D mathematical model as well as a 2D model were used to predict deformation and stresses during construction, and first impounding (due to topographic valley shape 3D behavior is prevailed). The deformation resulted by the mathematical model are quite far from measured ones, because of change in material quality and grading. Based on these analyses maximum settlement for 3D and 2D model are calculated of 51, 55 cm respectively. This was caused by the unconservative valves of deformation modulus used in model due to bad quality, (rock change quality and it grading size and fine content) especially grain size of 3C materials, as mentioned in previous section.
Figure 6. Measured settlement at magnet plates of instrumented sections.
Figure 7. Variation of measured settlement at magnet plates of instrumented sections with reading time.
Figure 8. Variation of measured settlement at hydraulic settlement cell of instrumented sections with reading time.
6 ESTIMATION OF DEFORMATION MODULUS

The deformation modulus can be derived from measurements of vertical settlement of during construction and the calculated vertical fill load above the settlement gage as follows:

\[ E_v = \frac{H \gamma_r h}{1000 S} \]  

(1)

\( E_v \) = vertical deformation modulus (MPa)
\( H \) = vertical depth rockfill above the settlement gage (m)
\( \gamma_r \) = unit weight of rockfill (KN/m³)
\( h \) = column of rockfill below the settlement gage (m)
\( s \) = settlement of the gage (m)

Using above formula and last settlement measurement the vertical deformation modulus can be calculated.

Instrumented sections show that most of the measuring settlement instruments are installed at middle rockfill (3C). Even for those instruments that are installed in main rockfill (3A) or downstream rockfill. The characteristic of (3C) material is governing the performance of those ones. Therefore the calculation vertical deformation modulus only should be related to middle rockfill (3C) materials. This materials have passing particles averagely form sieves No.200 and No.4 about 5~8 and 10~25 percent, respectively. The average vertical deformation modulus for all last measurement is about 40 to 70 Mpa. Three dimensional (3D) stress-strain analysis with vertical deformation modulus equal 100 Mpa result the maximum vertical settlement 51 cm which contrary to actual measurement shows 100 percent increase in deformation mainly because of fine content. The grading curves of 3C materials (middle rockfill) as well as 3A materials are shown in fig (9). It is clear that 3C material is a rockfill–soil mixtures, in which its mechanical properties such as shear strength, deformation modulus are strongly influenced by fine materials part.

The 3C materials has 10~25 fine material passing through sieve No.4, which increase compressibility of its as mentioned by Marsal, which “one-dimensional compression tests performed on specimens prepared by mixing rockfill with different percentage of silt or clay reveal that the compressibility of the mixture is generally higher than that of the uncontaminated rockfill”.

It should be noted that due to some problems financing the construction time was longer than anticipated time (3 years versus 2 years). So some of the time dependent settlement of the dam body, especially in middle rockfill (3C) had been occurred during construction.

Secondary settlement of clean rockfill such as (3A) material are smaller than (3C) materials, because this is initiated from creep behavior of clean rockfill but in (3C) material occurred due to compressibility characteristic of fine material part of soil–rock mixture.

Therefore the measured total settlement of dam body is higher than one predicted by 3D and 2D mathematical models. The new 3D and 2D analyses based on calculated deformation modulus of dam body materials are under preparation. Although the average calculated vertical deformation modulus shows the behavior of middle rockfill (3C) but
only for contrary that modulus are depicted on Pinto’s graph (fig10). The shape factor shows some deviation from general trend due to special gradation of middle rockfill material, and its fine content. Also the total displacement of face slab after first impounding based on 2D stress-strain analysis with \( E=100 \) MPa is about 30 cm. So due to decreasing the actual vertical deformation modulus from 100 MPa to about 50 MPa (average value), the maximum total displacement of face slab could be estimated by the following Formulas:

\[
D = \left( \frac{0.003}{e^{0.2(1+\frac{A}{H^2})}} \right) \frac{H^2}{E_{rc}}
\]

Which:
- \( D \) = face slab deflection (m)
- \( A \) = area of the upstream face slab (m²)
- \( H \) = embankment height (m)
- \( E_{rc} \) = deformation modulus during construction (MPa)

For \( E=50 \) Mpa maximum deflection is estimated about 0.36 m

7 CONCLUSIONS

In Gelevard Dam, due to environmental consideration and limitations, the contractor could not change the location of rock borrow area to gain rockfill with good quality. So the deformation modulus of most parts of the dam body decreased with respect to forecast deformation modulus and measured constructive settlement are greater than those predicted by mathematical model. Despite of this, field evaluation shows that the performance of dam body at end of construction is good.
Figure 10. Deformation modulus vs. Valley shape factor (after Pinto(2007))

REFERENCES

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