Evaluation of long-term settlement of ghoocham dam by numerical analysis

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Abstract

Ghoocham dam is 45 m height ECRFD type with a vertical clay core and soft alluvial and rock foundation under construction in west of Iran. To obtain the settlement distribution in dam body and foundation during different stages of construction, end of construction, first impounding and steady state, numerical analysis was conducted using a material behavior model including a combination of non-linear elastic Duncan and Chang model and elastic- perfectly plastic Mohr-coulomb. Numerical analysis was carried out by FLAC^{2D} which is explicit finite difference software. The material behavior model is implemented in numerical finite difference code, FLAC^{2D}, using its built-in fish language and plane strain analysis carried out to obtain deformations and stresses within dam body and foundation. The results show that the distribution of settlements of dam body and foundation are in acceptable ranges.

Keywords: rockfill dam, hyperbolic model, non-linear behavior, soft foundation, numerical analysis.

1. INTRODUCTION

Dams play a significant role in fulfilling the increasing demand of water for municipal and agricultural purposes. Embankment dams have become very popular among dam engineers since available materials of different types at the site could be used in appropriate zones of dam. Safety of rockfill dams depends on the proper analysis, design, construction and monitoring of actual behavior during the construction and the operation of the structure. The deformations in dam body and foundation are a crucial consideration in the design and construction of earth dams. Geotechnical analysis using numerical methods requires the implementation of material non-linear models for better prediction of structure behavior. In numerical analysis constitutive modeling of soil and rock mass is an essential component. Elastic- perfect plastic Mohr-coulomb and hyperbolic model are the most common constitutive models for predicting the behavior of foundation and dam body. Ghoocham dam is an ECRFD type which is under construction in west of Iran. In this paper, to obtain the deformation in dam body and foundation, numerical analysis was conducted by a combination of non-linear elastic Duncan and Chang model and elastic-perfectly plastic Mohr-coulomb constitutive model.

2. GEOMETRY

Ghoocham dam is an ECRD with vertical clay core material with 45 m height. The length of dam crest is 1820 m at the elevation 1856 m.a.s.l. The slopes of both dam abutments are gentle, around 12%-15% and 600 m of the central part of dam foundation is almost flat. Upstream and downstream slopes of rockfill shell are 1:1.7 and 1:1.5 (V:H), respectively. The slope of central core is 1:0.35(V:H).

Due to weak rock foundation, two stabilizing berms at upstream and downstream of dam with 70 m width have been designed in central part of dam. At maximum height section of dam, two 3 m and 2 m width fine filter and transition zone are considered at both upstream and downstream of core with the same slope of central core. Figure 1 shows the typical cross section of Ghoocham dam. Shell materials are obtained from two limestone and andesite-bazalt quarries close to the dam site. Materials for zones 4, 5A, filter and transition are obtained from limestone quarry and material for zones 4A and 4B are from andesite-bazalt quarry. Zone 7 material is from mandatory excavation of fine alluvial and rock foundation of dam body. Foundation of dam consists of 4-14 m thickness alluvial material and weak rock of tertiary quaternary unit of sanandaj-sirjan zone underneath. Fig. 1 shows the typical cross section of Ghoocham dam.

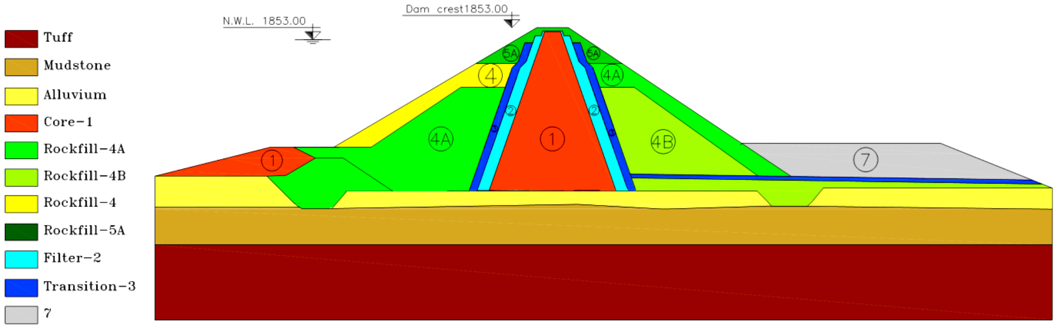

Fig 1. Typical cross section of Ghoocham dam

3. Foundation and Dam Body Material

3.1. Overburden and Foundation

The overburden of Ghoocham site with 4-14 m thickness consists of alluvial, slopewash and residual soils. The type of soil is majority silt, clay with thin layer of sand and gravel. The Consistency of overburden is classified as stiff to hard based on standard penetration test (SPT). A complete set of site and laboratory tests have been performed on these materials. Summary of overburden material parameters is shown in table 1.

Table 1. Geomechanical parameters of overburden.

Materials	γd (kg/m3)	E (MPa)	υ	C (KPa)	φ	k (cm/sec)
Alluvium	1800	10 – 40	0.25	34	19	3×10^{-7}

Rock types of foundation at Ghoocham dam site are mudstone, tuff and weak conglomerate. The average of rock quality designation (RQD) of rock is 81. The different types of foundation rock parameters are shown in table 2. Based on this table, the cohesion of rock is 40-50 KPa, internal friction angle is 22-25 degree and deformation modulus ranges between 470-800 MPa. The permeability of rock is around 3×10^{-7} cm/s.

Table 2. Geomechanical parameters of dam foundation.

Materials	γd (kg/m3)	E (MPa)	υ	C (KPa)	φ	k (cm/sec)
Tuff	2400	470	0.3	40	22	3×10^{-7}
Mudstone	2180	800	0.3	50	25	3×10^{-7}

3.2. Dam Body Material

Clay material of central core has been obtained from 0.5-1.5 Km upstream borrow areas in dam reservoir. The Core material is classified as CL based on unified classification, average PI of material is 22 and compacted with 2-3 percent moisture more than optimum water content. The maximum size of core material aggregates is 25 mm. fine and coarse filters at both upstream and downstream of core zone are processed from limestone quarry. D_{15} of fine filter is 0.45 mm with 10 mm maximum size aggregate (MSA). The MSA of coarse filter is 50 mm. The upstream outer shell zone (zone 4) and upper elevation shell (5A) are from limestone quarry. The other shell zones (4A & 4B) are andesite-bazalt type rock from basalt quarry. The layer thickness of zones 4, 4A and 4B is 60 cm. The maximum percent of passing sieve no.200 for zones 4 and 4A materials is 5 percent and for zone 4B material is 15 percent. The filling of shell zones has been done with wet procedure. A complete series of laboratory tests including large scale triaxial, shear and index tests performed on shell materials.

4. NUMERICAL ANALYSIS

The numerical analysis for different stages of during construction, end of construction, first impounding and steady state were carried out through the finite difference code FLAC^{2D}. The ratio of crest length to dam height is high enough to permit a two dimensional plane strain analysis. The analysis is performed in a layer by layer construction in effective stress condition. The seepage analysis in parallel to deformation analysis simulates the real condition of saturated soil materials behavior and consolidation process. Many researchers have proposed various constitutive models to simulate the behavior of rock and soil [1-5]. The Studies have shown that most of soil materials show non-linear behavior under different stages of loading [3,6-9]. Regarding the non-linear behavior of soil material, it's necessary to use a non-linear constitutive model in numerical analysis. One of the most prevalent non-linear constitutive models is Duncan & Chang. The hyperbolic model proposed by Duncan & Chang (1970), has been vastly used to predict non-linear behavior of soil materials [3]. In a hyperbolic constitutive model, the elastic modulus (E) changes relates to confining pressure and has an increasing trend with increasing confining pressure. The relationship between the deviatoric stress and axial strain for a constant confining pressure is described as:

$$\sigma_1 - \sigma_3 = \frac{\varepsilon}{\frac{1}{E_i} + \frac{\varepsilon}{(\sigma_1 - \sigma_3)_{ult}}} \tag{1}$$

E_i = initial tangent modulus, ε = axial strain, $(\sigma 1 - \sigma 3)_{ult}$ = ultimate deviatoric stress at large strain. σ_1 and σ_3 are the maximum and minimum principal stresses, respectively. Based on above equation, the young modulus (E) is defined as non-linear function of stress level and confining pressure. Therefore, the tangential Young modulus of elasticity (E_t) can be determined as:

$$E_t = [1 - R_f S_l]^2 \, K_{pl} P_a \left(\frac{\sigma_3}{P_a}\right)^n \tag{2}$$

Where R_f, S_l, P_a, σ_3 are failure ratio, stress level, atmospheric pressure (Kpa), confining pressure and K_{pl}, n are non-dimensional parameters.

Stress level (S_l) is a ratio between the deviatoric stress level and the failure stress level. The failure ratio and the stress level are defined as:

$$S_l = \frac{\sigma_1 - \sigma_3}{(\sigma_1 - \sigma_3)_f} \tag{3}$$

$$R_f = \frac{(\sigma_1 - \sigma_3)_f}{(\sigma_1 - \sigma_3)_{ult}} \tag{4}$$

Where $(\sigma_1 - \sigma_3)_f$ is deviatoric failure and it is expressed by the Mohr-Coulomb failure envelope as function of the friction angle (φ) and the cohesion (C).

$$(\sigma_1 - \sigma_3)_f = \sigma_3(N_\phi - 1) - 2c\sqrt{N_\phi} \tag{5}$$

$$N_\phi = (1 + \sin\phi)/(1 - \sin\phi) \tag{6}$$

$$\phi = \phi_0 - \Delta\phi \, . \mathrm{Log}\left(\frac{\sigma_3}{Pa}\right) \tag{7}$$

Where φ is the friction angle and φ_0 is the friction angle of soil at atmospheric pressure [3,10,11].

The Duncan & Chang hyperbolic model couldn't predict elasto-Plastic behavior of soil [10], therefore the Mohr-Coulomb constitutive model is implemented to predict plastic behavior of soil and calculate plastic strains. Thus, the constant elastic modulus in the Mohr-Coulomb model is modified based on hyperbolic model and is replaced with the tangential modulus presented in Duncan & Chang model. Equations of the proposed model have been implemented in the numerical finite difference code FLAC^{2D} using its built-in FISH language for the constitutive model. Then the model used for soil materials behavior of the dam body to calculate the deformation during different stages of loading. Due to stress distribution, the dimension of geometrical model has been considered 320×82 m (L×H). Construction stage analysis was conducted layer by layer with a thickness around 2 meters for each layer. Aanalysis was performed based on effective stress approach along with water flow analysis to determine pore pressure within core and foundation zones. The analysis duration for each layer is set according to the dam construction time schedule.

In the analysis, the parameters of constitutive model have been assumed as table 1 to 3. These parameters have been obtained by in-situ and laboratory tests. The constitutive models of dam body and foundation are the combination model and the Elasto-plastic Mohr coulomb model, respectively.

Table 3. Material parameters of dam body for the analysis

Material/Zone	γ_d (kg/m^3)	K_p	N	R_f	υ	C (KPa)	φ	K(cm/sec)
Core	1650	125	0.56	0.87	0.35	41	28.8	$^{7-}10 \times 4$
Filter	1900	250	0.7	0.8	0.25	0	38	$^{3-}10 \times 1$
Rockfill (4)	2150	147	0.88	0.83	0.25	38	43	$^{5-}10 \times 1$
Rockfill (4A)	2150	631	0.4	0.86	0.25	0	43.2	$^{5-}10 \times 1$
Rockfill (4B)	2150	891	0.73	0.81	0.25	0	43.2	$^{5-}10 \times 1$
Rockfill (5A)	2150	147	0.88	0.83	0.25	38	43	$^{5-}10 \times 1$
Zone 7	1900	170	0.8	0.7	0.25	30	30	$^{5-}10 \times 1$

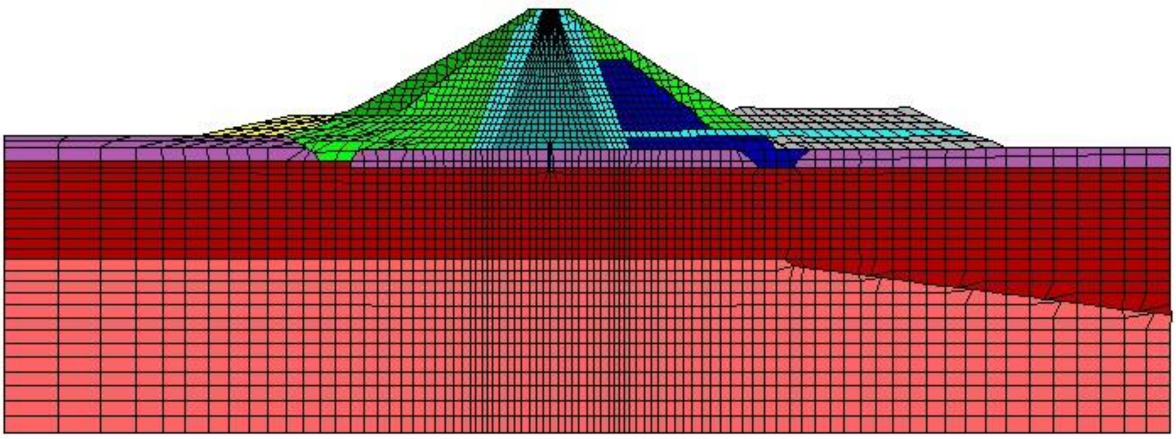

Fig 2. Finite difference mesh of Ghoocham dam body and foundation

4.1. CONSTRUCTION STAGE ANALYSIS

At first stage, the stress- deformation analysis has been conducted up to elevation 1849 (m.a.s.l.). By comparison the measured settlement and instrumentation data with analysis results, the numerical model has been calibrated. To verify the calibrated model, the settlement of diversion system under the dam body has been measured at different time intervals and compared with the numerical results. The maximum settlement has been reported 40 cm at embankment elevation of 1849 (m.a.s.l.) equal to 38 m dam height. At next step, the stress- deformation analysis for end of construction stage has been done up to dam crest elevation (1856 m.a.s.l.), then the first impounding and steady state analysis have been carried out and stress and settlement distribution have been computed.

The vertical and horizontal displacements are illustrated in Fig. 1 and Fig. 2. The maximum settlement and horizontal displacement in dam body are 73 cm and 17 cm. Fig. 6 shows foundation settlements profile along pipes of diversion system, in the vicinity of pipes the maximum settlement happened in upstream and it's equal to 41 cm. Compared the numerical results with reported settlements shows the suitability of calibrated model to predict dam body and foundation deformations. Due to the cut off wall the settlement in the axial of dam is lower than another area.

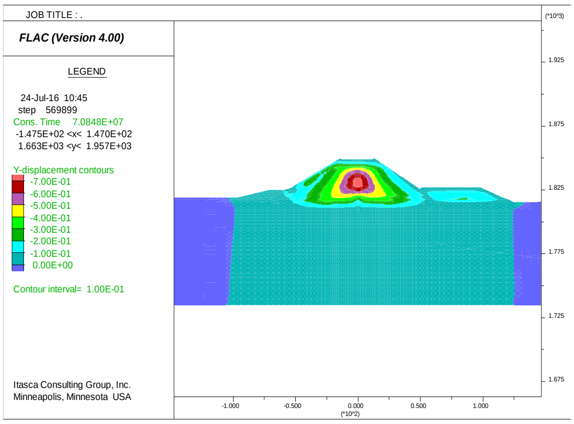

Fig 3. Vertical displacement contours (m)

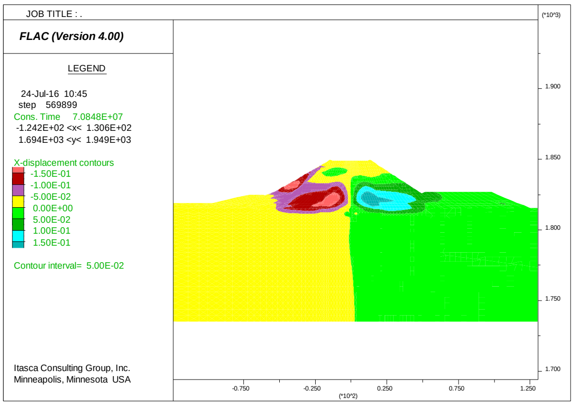

Fig 4. Horizontal displacement contours (m)

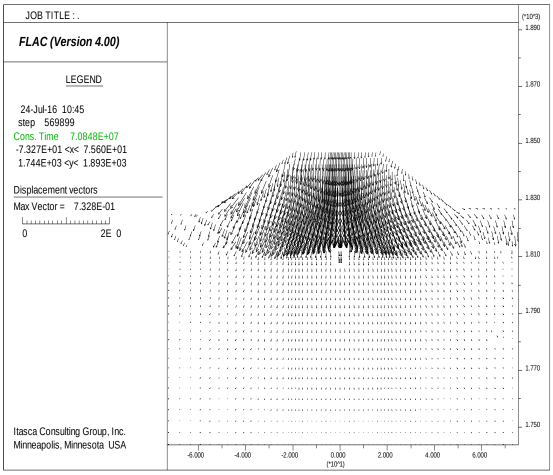

Fig 5. Displacements vectors (m)

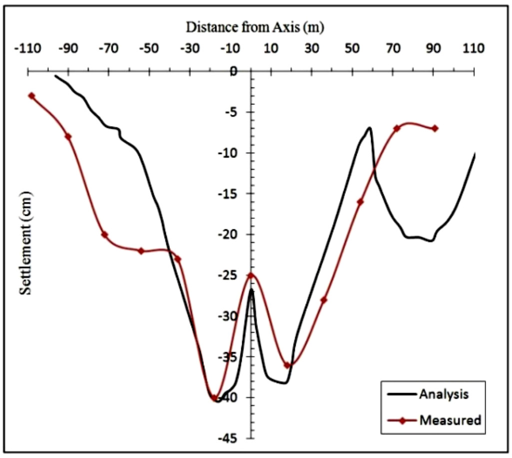

Fig 6. Settlements profile along pipes of diversion system (cm)

4.2. END OF CONSTRUCTION ANALYSIS

Construction stage analysis was conducted layer by layer. The settlement and the horizontal deformation contours and vector are presented in Fig. 7 and Fig. 8, respectively. It can be seen that the maximum settlement in dam body is 97.5 cm occurs in middle height of core zone and maximum horizontal displacement is 22 cm. Fig. 9 shows the displacement vectors, as shown with increase embankment elevation the shells moved toward upstream and downstream. Fig. 10 shows foundation settlements profile along pipes of diversion system. As shown, the maximum settlement of foundation along the pipes is equal to 45 cm at end of construction stage.

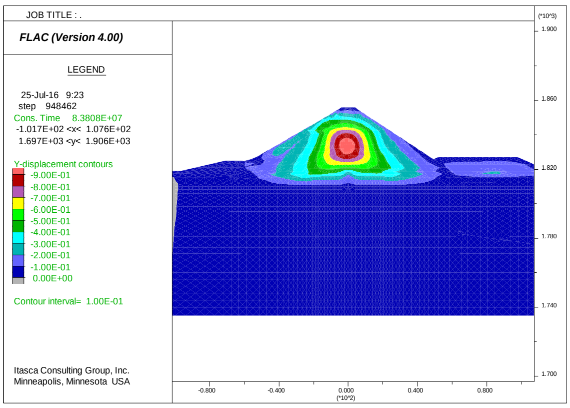

Fig 7. Vertical displacement contours (m)

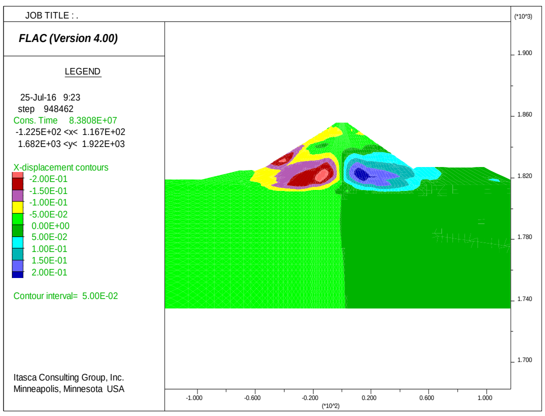

Fig 8. Horizontal displacement contours (m)

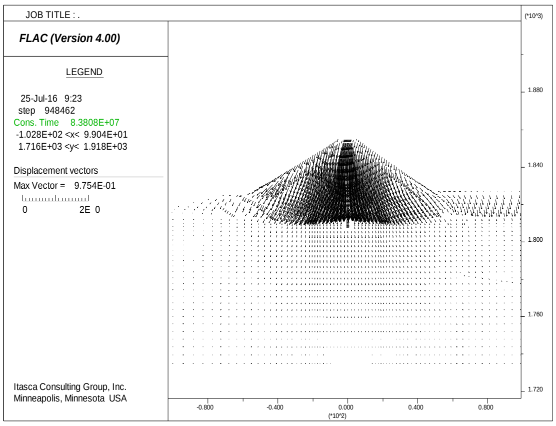

Fig 9. Displacements vectors (m)

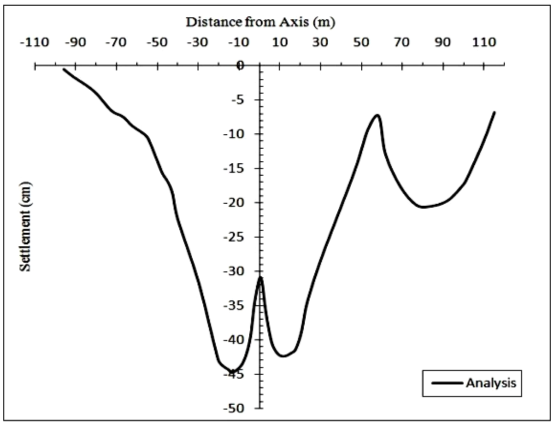

Fig 10. Settlements profile along pipes of diversion system (cm)

4.3. FIRST IMPOUNDING STAGE

The first impounding and steady state are the most important stages of dam construction. Increasing the water elevation in reservoir leads to horizontal and vertical displacements in dam body and foundation. In order to predict the dam behavior, the numerical analysis has been conducted using the parameters same as construction stage. Impounding duration up to normal water elevation (1853 m.a.s.l.) has been assumed 2 months. The vertical and horizontal displacements contours are illustrated in Fig. 11 and Fig. 12. The maximum settlement and horizontal deformation of dam body at this stage are 108 cm and 46 cm respectively. As shown in Fig. 13 the reservoir water pressure leads to horizontal deformation toward the downstream and settlement in dam body and foundation. Fig. 14 shows the foundation settlements profile along pipes of diversion system. The maximum settlement along the diversion system is 52 cm.

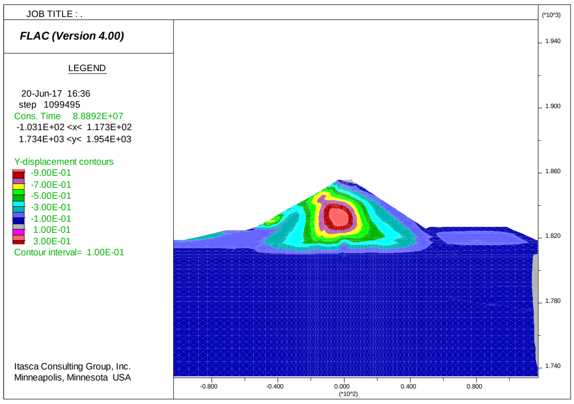

Fig 11. Vertical displacement contours (m)

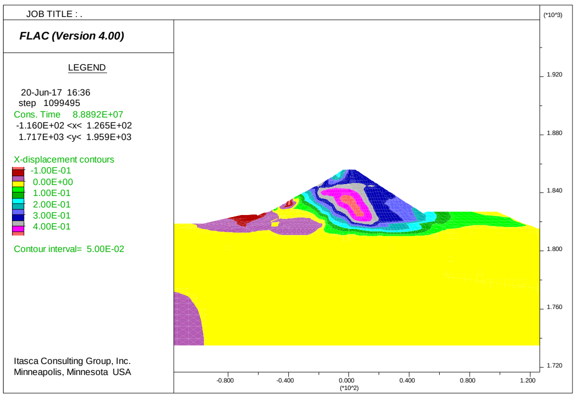

Fig 12. Horizontal displacement contours (m)

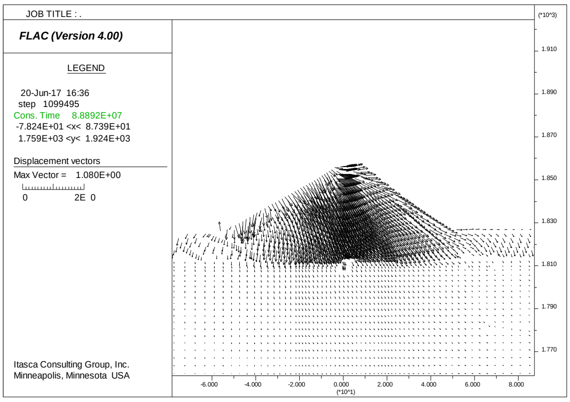

Fig 13. Displacements vectors (m)

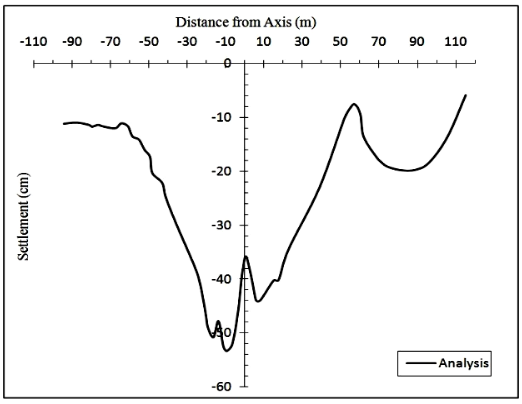

Fig 14. Settlements profile along pipes of diversion system (cm)

4.4. STEADY STATE STAGE

At the steady state stage, the pore pressure in low permeability zones begins to dissipate and long term deformations and consolidation settlements occur. The duration of this stage in numerical analysis assumed 180 months. The vertical and horizontal displacements are illustrated in Fig. 15 and 16. The displacement vectors of dam body s illustrated in Fig. 17. The maximum settlement and horizontal displacement include immediate and consolidation settlements in dam body are 110 cm and 49 cm, respectively. Fig. 18 shows the foundation settlements profile along pipes of diversion system. The maximum settlement in foundation along the diversion system is 52 cm.

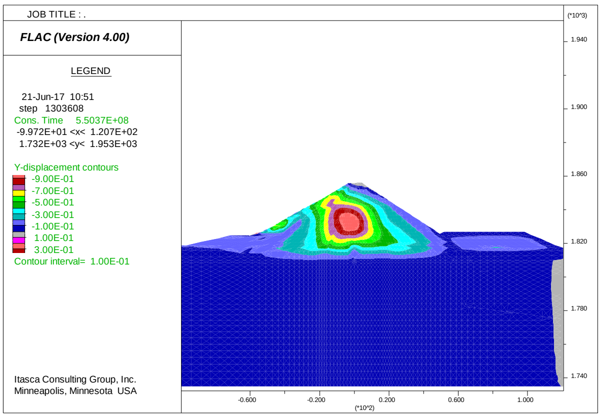

Fig 15. Vertical displacement contours (m)

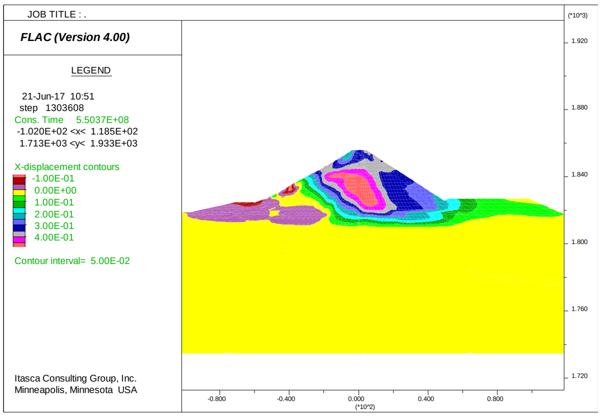

Fig16. Horizontal displacement contours (m)

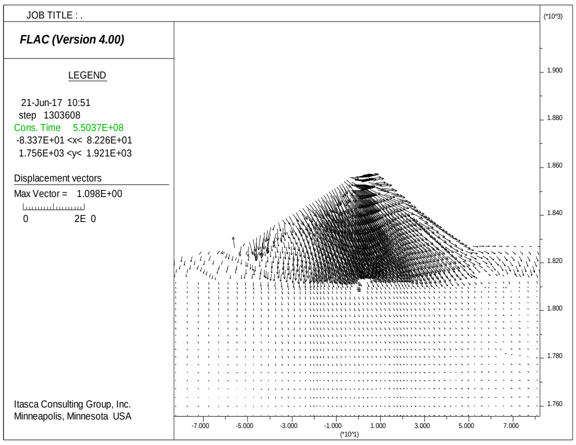

Fig 17. Displacements vectors (m)

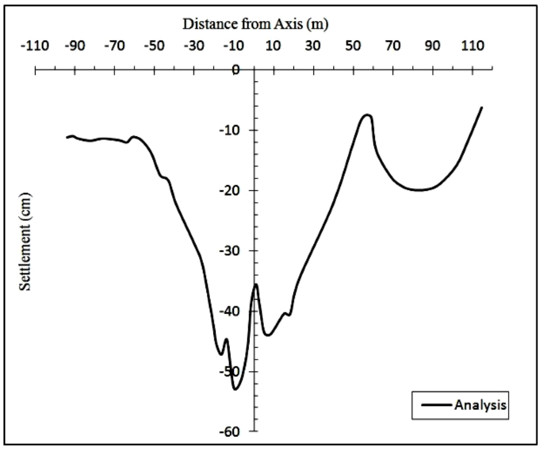

Fig 18. Settlements profile along pipes of diversion system (cm)

5. CONCLUSIONS

To obtain the long term settlements of dam and foundation, numerical analysis has been conducted. Based on experience and laboratory test results, a combination of non-linear elastic Duncan and Chang model and elastic perfectly plastic Mohr-coulomb model was used in the analysis. The model is implemented in numerical finite difference code FLAC2D using its built in FISH language and then used to analyze dam. The results show the maximum settlement of dam body and diversion system up to elevation 1849 (m.a.s.l.) that is equal 37 m height of embankment is 73 cm and 41 cm, respectively. The last measured settlement of diversion system is 40 cm, the comparison between measured settlements and numerical results confirm the suitability of assumed constitutive model and parameters to predict the dam behavior. The numerical results show the maximum settlement of dam body and diversion system at end of construction are 97.5 cm and 45 cm. Also, the maximum settlements of dam body at first impounding and steady state are 108 cm and 110 cm and the maximum settlements of diversion system are 46 and 49 cm, respectively. Based on some engineering codes such as ICOLD, USBR and USSD which have suggested allowable deformation of dam body and foundation can be said, the distribution of settlements is acceptable.

6. REFERENCES

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