

The Deformation Mechanism of a High Rockfill Dam during the Construction and First Impounding

A. Akhtarpour^{a,1} and M. Salari^b

+989153143978

a. Assistant Professor of Geotechnical Engineering, Faculty of Engineering, Ferdowsi University of Mashhad, Mashhad, Iran

b. Ph.D. Student of Geotechnical Engineering, Faculty of Engineering, Ferdowsi University of Mashhad, Mashhad, Iran

Abstract: The Masjed-e-Soleyman dam is a high rockfill dam with clay core, located in Iran. During construction and first impounding, a considerably high excess pore water pressure has been developed inside the core and has been being dissipated with a very slow rate, so the consolidation deformations have been insignificant. However, there have been reports of noticeable internal deformations in the dam, the crest has also exhibited quick settlements during the first impounding. The main objective of this paper was to identify the deformation mechanism of this dam. For this purpose, the data recorded by its instruments were carefully studied and then a three-dimensional numerical model of the dam was developed. The mechanical behavior of materials was idealized by a hardening strain constitutive model. A numerical method was proposed, based on this constitutive model and Rowe's stress-dilatancy theory, to simulate the deformation behavior of coarse-grained materials, like rockfills, due to particle size distribution, particle breakage, rotation, and rearrangement under shearing. The results show that significant development of pore pressure in the core and its insignificant dissipation, plastic shear deformations inside the core and extensive collapse settlements of the upstream shell are the main causes influencing the deformation mechanism.

Keywords: high rockfill dam, hardening strain constitutive model, particle breakage, collapse settlement.

1. Introduction

Rockfill materials consist of sharp-edged aggregates obtained by blasting in rock borrow areas or rounded or sub-rounded aggregates excavated from river beds. These materials have had extensive use in the body of rockfill dams [e.g. 1-7], the subgrade of roads as well as a wide variety of other engineering structures. The high shear strength of these materials enables the embankments to maintain steeper slopes. The strength and deformation behaviors of rockfill materials are conventionally investigated through large-scale triaxial tests [e.g. 8-15]. The mechanical behavior of granular materials can be simulated via a variety of constitutive models, such as non-linear elastic model [16], elasto-plastic hardening model [17-19], and strain softening elasto-plastic model [20]. It used to be a common practice to simulate the behavior of rockfill materials by linear or non-linear elastic models [21,22], but there are now a variety of advanced constitutive elasto-plastic models based on the disturbed state concept [11,23,24] and critical state concept [25, 26] that can be used for modeling these materials. Previous laboratory researches have revealed that particle size distribution (PSD), particle breakage, rotation, and rearrangement has significant effects on the strength and deformation behaviors of coarse-grained material [e.g. 10, 14, 27-31]. There have been several creditable efforts to incorporate the impacts of these phenomena into mechanical constitutive models; these include the works carried out based on disturbed state concept [24] and Rowe's stress-dilatancy theory [25,26,32].

Laboratory investigations and field observations have detected the collapse behavior in a wide range of geotechnical materials [e.g. 33-39]. During the first impounding of central core rockfill dams, the collapse settlement inside the upstream shell can increase the settlement of rockfill shell relative to the core. For instance, during the first impounding of the Cherry Valley dam, the settlement of the upstream shell was four times greater than that of central core, which consequently led to formation of longitudinal cracks on the crest along the shell-core interface [40,41]. Nobari and Duncan [42] proposed a technique for modeling this phenomenon. This technique relies on hyperbolic model presented by Duncan and Chang [16] and is closely tied to the direct use of triaxial test results. Naylor et al. [43] proposed another method, which combined the technique of Nobari and Duncan [42] with critical state elastoplastic model. This method was used to simulate the collapse settlement of the rockfill shell of the Beliche dam, located in Portugal [43-46]. Other researchers used the porous media mechanics and unsaturated soil frameworks to develop other methods with similar objectives [47-49]. Recently, Mahinroosta and Alizadeh [50] used a concept called "stress reduction factor" to develop a practical technique for modeling the collapse settlement incorporated with a hardening/softening constitutive model [51], in the framework of saturated soil mechanics. This technique was used to simulate the collapse settlement of the rockfill shell of the Gotvand Dam, the highest rockfill dam located in Iran [52].

The Masjed-e-Soleyman (MES) dam is currently the second highest rock fill dam in Iran. This dam has exhibited a deformation behavior that is unique in comparison with other known high rockfill dams in the world. During construction and first impounding, a considerably high excess pore water pressure has been developed inside the core and it has been being dissipated with a very slow rate, so the consolidation deformations have been negligible. However, there have been reports of noticeable internal deformations in the different parts of the dam -especially inside the core- during the construction. Moreover, the crest has exhibited significant quick settlements during the first impounding. The main purpose of this research is to identify the deformation mechanism exhibited by this dam during the construction and first filling. To achieve this objective, first, the data recorded by the instruments installed inside the dam were studied to identify the main causes influencing its behavior. And, then a three-dimensional numerical model of the dam was developed to identify its deformation mechanism and determine the role of each effective cause influencing this mechanism. The calculations made simultaneous use of flow and mechanical formulations and also incorporated the water/soil interactions (consolidation phenomenon). The mechanical behavior of all materials of embankment was simulated by an elasto-plastic hardening/softening constitutive model built in finite difference Flac3D software [51]. This model was based on Mohr-Coulomb model. The difference, however, lies in the possibility that the cohesion, friction, dilation and tensile strength may harden or soften after the onset of plastic yield. In this paper, Authors proposed a numerical method, based on this constitutive model and Rowe's stress-dilatancy theory, to simulate the deformation behavior of coarse-grained materials, like rockfills, due to particle size distribution (PSD), particle breakage, rotation, and rearrangement under shearing. All parameters of the constitutive model for the coarse-grained zones were derived by numerical simulation of the large scale triaxial tests. The collapse settlement of the upstream rockfill shell during the first impounding was simulated by the technique proposed by Mahin Roosta and Alizadeh [50].

2. Masjed-e-Soleyman dam (MES Dam)

¹ Corresponding author, Tel: +985138805170
Box: 91775-1111, Mashhad, Iran
E-mail addresses: akhtarpour@um.ac.ir (A.Akhtarpour)
Salari_m@alumin.um.ac.ir (M.Salari)

The Masjed-e-Soleyman Dam previously named “Godar-e-Landar” is a 177 meters high rockfill dam with central core, located in southwest of Iran on the Karun river. It has an installed capacity of 2000 MW, and its reservoir holds 261,000,000 cubic metres of water. The crest width and crest length of this dam are 15 and 500 meters, respectively. The main cross-section of this dam is shown in **Fig. 1**. The rock mass surrounding the dam is part of Aghajari and Bakhtyari formations and consists of alternating layers of conglomerate and siltstone. This dam has been constructed on a narrow valley 18 meters wide at the river bed and with average wall slope of 36° to the horizon.

The geotechnical site investigations had identified the Simband borrow area as the best potential source of material for the core (see region 1 in **Fig.1**). This borrow area consists of irregular alternating layers of clay (CL) and clayey gravel (GC), so the core is a mixture of clay and gravel materials. The lower two-thirds part of the core has been compacted with higher than standard optimum water content and its upper one-third part has been compacted with lower than standard optimum water content. The shells consist of conglomerate aggregates obtained by blasting and have been compacted without adding the required moisture. The aggregates of regions 3A and 3C are coarser than 3B. The filter regions (2A, 2B and 2C) also consist of conglomerate aggregates with appropriate grading and have been placed with traffic compaction [53].

To monitor the stress, pore pressure, and settlement inside the dam, a number of instruments had been installed at cross chainages (CH.) 160, 260, 360 and 420 meters from the end of the crest (the left side), but a large number of these instruments, including the Electrical Piezometers (EP) and the Total Pressure Cells (TPC) installed in the core adjacent to the upstream shell and the magnetic plates (MP) (for measuring internal settlements) installed at the lower levels of the core, were damaged by the extreme deformations occurred during the construction of final 50 meters of embankment. **Fig.1** shows the arrangement of undamaged instruments in the main cross-section (CH. 260) of the dam. In this figure, each Cluster (C) includes three TPCs and one EP. These TPCs have been installed at horizontal, 45 degrees upward and 45 degrees downward directions. About one month after the start of first impounding, when the water reservoir level had risen about 57 meters, a system of survey measurement points (SMP) was installed on the dam to measure surface displacements (see **Fig. 2**).

3. Dam Monitoring

The fill load generated through the course of construction increases the total earth stress, leading to development of excess pore water pressure and deformation in those parts. According to Nobari and Duncan[42], the water entering the upstream shell during the first impounding has four effects: E1) water pressure acting on the core, E2) water pressure acting on the foundation, E3) buoyancy in upstream shell, and E4) saturation collapse in upstream shell (see **Fig. 3**). The evaluation of data recorded by the instruments revealed the following:

Pore water pressure: the pore pressure ratios of the core, r_u (the excess pore pressure, u , divided by the overburden, γh) at the instrumentation chainages for different levels are shown in **Table 1**. A number of causes like the fact that lower parts of the core have been compacted wet-of-optimum (high degree of saturation), the very low permeability of the core, and the relatively high rate of construction have led to significant development of excess pore water pressure in the core during the construction. As this table shows, at the lower levels of each chainage, almost constant pore pressure ratio is prevailing between the center of the core and at the downstream locations of the core. This is indicative of very low excess pore pressure dissipation in the core. The very low permeability of the compacted core material is one of the causes behind this behavior. Moreover, as presented in subsequent sections of this paper, as excess pore pressure dissipates and vertical effective stress increases, the permeability decreases exponentially. Reduced permeability of regions around the core and adjacent to the filters leads to slower rate (speed) of dissipation. During the first impounding, the effects E1 and E4 have increased the pore pressure ratio in the core.

Total stress: the arching ratios of the core, r_A (the vertical total stress, σ_y , divided by the overburden, γh), at the instrumentation chainages for different levels are shown in **Table 2**. During the first impounding, the effect E1 and the vertical component of the effect E4 have increased the arching ratio in the core. The reduction of these ratios at the downstream filter and shell during the first impounding can be attributed to rotational displacements of the TPCs installed at horizontal direction due to effects of the rotational component of the effect E4.

The real pore pressure ratios, R_u (the excess pore pressure, u , divided by the vertical total stress, σ_y), are shown in **Table 3**. Also, the ratios of vertical total stress to horizontal total stress, r_K , are shown in **Table 4**. At lower parts of the core, the R_u and r_K ratios are unity or very close to unity (especially in the middle chainages). These near-unity ratios can signify the presence of a near-zero effective stress zone with isotropic stress condition at the lower levels of the core. So, it seems that the core exhibits an almost incompressible behavior (quasi-fluid) in this zone; a behavior that has not been previously reported in the core of any zoned dam in the world. The significant development of excess pore water pressure in the lower parts of the core and very low rate of its dissipation are the main causes of forming this near-zero effective stress zone.

As mentioned, the three-component TPCs were installed in the dam. This is, therefore, possible to determine the magnitude and orientation of principal stresses σ_1 and σ_3 . The determined σ_1 and σ_3 are acting in the plane perpendicular to the dam axis. **Fig.4(a)** depicts the relevant results for main-cross section (CH.260) in terms of stress bars for the measured total stress and circles for the measured pore pressures at the end of first impounding. In the center of the core, the measuring points were omitted, as these points consistently show almost isotropic earth pressure meaning that orientation of principal stresses becomes random. Instead, these zones are marked with a general signature. The areas in which the measurements yielded zero stresses are especially indicated (Level: 270). Examination of **Fig.4 (a)** shows that a substantial rotation in the principal stress direction has been monitored. At level 270 and above, an anti-clockwise rotation is generally occurring in the core whereas at lower levels (level: 230) clockwise rotations have been monitored. This re-arrangement of principal stress directions is of systematic nature. It is indicative of a shear zone, which is progressing in inclined direction downward as systematically sketched in **Fig.4(b)**. In the core, such shear movement would be associated with a volume decrease (contractive behavior) and would more likely lead to a further rise rather than a relief of excess pore water pressures.

Deformation: **Fig.5** shows the time variations of data recorded by the MPs (installed along the central axis inside the core) and the SMP21 (positioned at the upstream of the crest) in the main cross section. At the end of construction, the maximum recorded settlement in the core has been 3.7 meters (about 2% of the dam height), and has occurred at the level +314 (see MP-2312 in **Fig.5**). Apparently, this significant settlement recorded during the construction is not consistent with the significantly slow consolidation process in the core. The oblique shear zones in the core may play a major role in this inconsistency, as presented in subsequent sections.

The effects of first impounding have caused a 2.2 meters settlement (about 1.23% of the dam height) in the crest (see MP-2319 in **Fig.5**). Moreover, the increased excess pore pressure in the core in this period points toward negligible consolidation settlements. As mentioned, one month after the start of first impounding, when the water reservoir level had risen about 57 meters, a system of survey measurement points (SMP) was installed on the dam to measure surface displacements. Comparing the settlement recorded by MP-2319 magnet plate and the settlement measured at survey point SMP21 (see **Fig. 5**) indicates that almost half of the total settlement due to the first impounding has occurred during the first month after the start of impounding. The reason behind this observation may be that before setting the system of SMPs a vast flood entered in the reservoir, passing over the cofferdam (the crest level of the coffer dam is +301) and flooding a considerable volume of the upstream shell.

The upstream rockfill shell that has been compacted without sufficient moisture is highly susceptible to collapse settlements. So, the collapse settlements of the upstream rockfill shell are likely to be the main cause influencing the deformation behavior of the dam during the first impounding.

4. Numerical modeling

Given the relatively low crest length-to-dam height ratio (=2.82), the behavior of the dam was simulated by a three-dimensional numerical model. The idealized three-dimensional geometry of the dam, its abutments and their corresponding mesh are presented in **Fig.6**. Numerical modeling was performed by finite difference method in the FLAC3D program. The calculations utilized the flow and mechanical formulations simultaneously and also incorporated the water/soil interactions (consolidation phenomenon).

4.1. Mechanical modeling

The mechanical behavior of the different materials of the dam was simulated by an elasto-plastic hardening/softening constitutive model built in the FLAC3D software [51]. This model has been developed based on Mohr-Coulomb constitutive model. In this model, the initial elastic behavior of the materials is simulated with Young's modulus (E) and Poisson's ratio (ν), and after the first yield, to simulate hardening/softening and dilative behaviors of material, the friction angle, cohesion, dilation angle and tensile strength are mobilized as functions of plastic shear strain and tensile strain and reach their peak values.

4.1.1 Yield envelopes and potential functions

Yield surface functions of this constitutive model were defined by Eqs.(1) and (2).

$$f^s = \sigma_1 - \sigma_3 \frac{1 - \sin \varphi_m}{1 + \sin \varphi_m} + 2c_m \sqrt{\frac{1 - \sin \varphi_m}{1 + \sin \varphi_m}} \quad (1)$$

$$f^t = \sigma_m^t - \sigma_3 \quad (2)$$

In these equations, f^s and f^t are the shear and tensile yield functions and σ_1 and σ_3 are the maximum and minimum principal stresses.

Also φ_m , c_m and σ_m^t are, respectively, the mobilized friction angle, mobilized cohesion, and mobilized tensile strength of the model.

This model used the non-associated flow rule for shearing and the associated flow rule for tension, and defined the plastic potential functions as follows:

$$Q^s = \sigma_1 - \sigma_3 \frac{1 - \sin \psi_m}{1 + \sin \psi_m} \quad (3)$$

$$Q^t = -\sigma_3 \quad (4)$$

In these equations, Q^s and Q^t are the shear and tensile plastic potential functions and ψ_m is the mobilized dilation angle.

4.1.2 Stress-dependent elastic modulus

In almost all geotechnical materials, the elastic modulus depends on the confining stress. Hence, Eq.(5) which was proposed in hyperbolic (nonlinear elastic) model by Duncan and Chang [16] is applied in the constitutive model for definition of this dependency. A multitude of researchers have confirmed the validity of this equation for a variety of soils and rocks under both laboratory and field conditions [e.g 22, 52,54,55].

$$E = KP_a \left(\frac{\sigma_3}{P_a} \right)^n \quad (5)$$

In this equation, E is the Young's modulus, K is the elastic modulus, P_a is the atmospheric pressure and n is the exponent of elastic modulus dependence on confining stress. Poisson's ratio (ν) is considered as a constant value, which associated with above mentioned elasticity parameters results in appropriate response in the elastic domain of the constitutive model.

4.1.3. Frictional hardening

Vermeer and de Borst [56] have proposed a relation for frictional hardening behavior of the geotechnical material. In this relation the mobilized friction angle, φ_m , depends on plastic shear strain and gradually increases to its peak value, φ_p . The authors modified this relation by introducing initial mobilized friction angle, φ_0 . The resulting modified equation is shown below:

$$\sin \varphi_m = \begin{cases} \sin \varphi_0 + 2 \frac{\sqrt{\varepsilon_s^p \times \varepsilon_s^f}}{\varepsilon_s^p + \varepsilon_s^f} (\sin \varphi_p - \sin \varphi_0) & \text{for } \varepsilon_s^p \leq \varepsilon_s^f \\ \sin \varphi_p & \text{for } \varepsilon_s^p > \varepsilon_s^f \end{cases} \quad (6)$$

In the above equation, ε_s^p is the plastic shear strain and ε_s^f is the plastic shear strain at peak friction angle. The parameter φ_0 controls the domain of elastic behavior to be exhibited by materials. This parameter can be defined as a function of confining stress.

4.1.4. Dilatancy behavior

Rowe [57] proposed Eq.(7) for dilatancy hardening behavior of geotechnical material.

$$\sin \psi_m = \frac{\sin \varphi_m - \sin \varphi_{cv}}{1 - \sin \varphi_m \sin \varphi_{cv}} \quad (7)$$

Where ψ_m is the mobilized dilation angle, φ_{cv} is the critical state friction angle or friction angle of constant volume, which can be obtained from Eq.(8)

$$\sin \varphi_{cv} = \frac{\sin \varphi_p - \sin \psi_p}{1 - \sin \varphi_p \sin \psi_p} \quad (8)$$

Where ψ_p is the peak dilation angle.

Xu and Song [25] reported that Eq. (7) must be modified prior to modeling the stress–dilatancy behaviors of rockfill materials. The main reasons for this requirement are summarized as follows:

- (1) The particle shape of rockfill material is irregular, and Rowe’s stress–dilatancy equation is based on the irregular packing of uniform rods.
- (2) Particle breakage, rotation, and rearrangement are the primary causes of deformation of rockfill materials; however, particle slip is the primary mechanism in Rowe’s stress– dilatancy theory.
- (3) The particle size distribution (PSD) in Rowe’s stress– dilatancy theory is significantly different from that in rockfill materials.
- (4) The validation of Rowe’s stress–dilatancy theory is primarily based on testing with sands and steel balls, which are more difficult to crush than rockfill materials.

Consequently, Xu and Song (2009) proposed a simple revised form of Rowe’s stress–dilatancy equation for rockfill material as follows:

$$\sin \psi_m = R_d \frac{\sin \varphi_m - \sin \varphi_{cv}}{1 - \sin \varphi_m \sin \varphi_{cv}} \quad (9)$$

In the above equation, R_d is introduced as a reduction factor that is less than unity and reflects the effects of PSD, particle breakage, rotation, and rearrangement on the deformation of rockfill material.

Although Xu and Song [25] have very well recognized the weaknesses of Rowe’s mobilized dilation equation (Eq.(7)), as it will be presented following, the constitutive model modified by their equation lacks the necessary comprehensiveness for the estimation of contractive volumetric strains of some rockfill materials, like MES rockfill material, with high potential of particle breakage.

Under shearing such as the triaxial test, dense granular materials first contract in the elastic range and then undergo an irreversible dilation. But when materials have a low density or have a potential for the particle breakage, rotation, and rearrangement, they experience irreversible contractive volumetric strains (contraction phase) before going into dilation phase. The magnitude of these contractive volumetric strains depends on the intensity of the effects of mentioned phenomena on the deformation mechanism of materials. On occasions, the magnitude of contractive volumetric strains may be great enough to prevent or significantly mitigate the dilation.

By definition, dilation (d) is:

$$d = \frac{d\varepsilon_v^p}{d\varepsilon_s^p} \quad (10)$$

Where ε_v^p and ε_s^p are plastic volumetric strain and plastic shear strain, respectively. Ignoring dilatational elastic strain under triaxial condition gives Eq.(11):

$$d = \frac{d\varepsilon_v^p}{d\varepsilon_s^p} = \frac{d\varepsilon_v^p}{d\varepsilon_1^p - \frac{1}{3}d\varepsilon_v^p} = \frac{\frac{d\varepsilon_v}{d\varepsilon_1}}{1 - \frac{1}{3} \frac{d\varepsilon_v}{d\varepsilon_1}} \quad (11)$$

Where ε_v , ε_s and ε_1 are, respectively, volumetric strain, shear strain, and axial strain. On the other hand, according to Rowe’s theory, the relationship between dilation, d , and mobilized dilation angle, ψ_m , is in form of Eq.(12):

$$d = -\frac{6 \sin \psi_m}{3 - \sin \psi_m} \quad (12)$$

According to Eqs.(11) and (12) when $d\varepsilon_v/d\varepsilon_1 > 0$, so $d > 0$ and $-90^\circ \leq \psi_m < 0$; this means that materials that are under shearing, are contracting. But when $d\varepsilon_v/d\varepsilon_1 < 0$, so $d < 0$ and $90^\circ \geq \psi_m > 0$; this is means that materials are dilating. With this description, it is clear that greater initial contractive volumetric strains correspond to steeper initial slope of $\varepsilon_v - \varepsilon_1$ curve ($d\varepsilon_v/d\varepsilon_1 < 0$) and lead to larger (more positive) values for dilation, d , and consequently lower (more negative) mobilized dilation angles, ψ_m .

According to Xu and Song [25], In Eq. (9), the reduction factor R_d is positive and less than unity. So, under shearing, in the early stages of loading (contraction phase), the values obtained for mobilized dilation angle, ψ_m , from Rowe’s equation (Eq.(7)) are smaller (more negative) than values obtained from Eq.(9). As a result, in this contraction phase, the contractive volumetric strains predicted by the hardening constitutive model modified by Eq. (9) will be lower than the ones predicted by the model modified by Eq.(7). Some rockfill materials such as ones used in the shell of MES dam have a high potential for the particle breakage rotation, and rearrangement phenomena. In other words, these materials exhibit significant contractive volumetric strains in the early stages of shear loading. Therefore, the constitutive model modified by Eq.(9) cannot adequately predict the contractive volumetric strains formed in these materials. In addition, in this research (which will be presented later) the results of numerical simulation of large-scale triaxial tests on rockfill materials constituting the shell of MES dam show that in the early stages of applying shear load, even the constitutive model modified by Rowe’s equation (Eq.(7)) cannot adequately predict the contractive volumetric strains of these materials. In conclusion, as mentioned, it can be stated that although Xu and Song [25] have very well recognized the weaknesses of Rowe’s mobilized dilation equation (Eq. (7)), it appears that their equation lacks the necessary comprehensiveness for the estimation of contractive volumetric strains, and their fourth reason of Xu and Song [25] for the modification of Rowe’s equation is not well reflected in their proposed equation (Eq.(9)). Xu and Song [25] validated their equation (Eq. (9)) based on the laboratory results of large-scale triaxial tests conducted by Varadarajan et al. [11]. These tests had been conducted on sharp-edged limestone rockfill materials (obtained by blasting) and rounded to sub-rounded rockfill materials collected from river bed. The results of these tests showed that contractive volumetric strains of these materials at confining stress of 1200 kPa is at most 1.5 percent, but because of great particle breakage potential of conglomerate rockfill materials such as the ones constituting the shell of MES dam, the magnitude of contractive volumetric strains of these materials in the same confining stress (1200 kPa) is up to several times greater than that value (about 5%). However, in the last stages of loading (dilation phase), the constitutive model modified by Eq. (9) can predict lower dilatational volumetric strains in comparison with the model modified by Eq. (7). Therefore, less than-unity-values of reduction factor R_d can very well reflects the effects of PSD, particle breakage, rotation, and rearrangement on the deformation of rockfill material in the dilation stage. In view of these results, the authors propose Eq.(13) as a more comprehensive alternative to Eqs.(7) and (9).

$$\sin \psi_m = \begin{cases} \sin(I_c \psi_{mR}) & -90^\circ \leq I_c \psi_{mR} \leq 0 \\ \sin(R_d \psi_{mR}) & 0 < R_d \psi_{mR} \leq 90^\circ \end{cases} \quad (13)$$

In the above equation, ψ_{mR} is the mobilized dilation angle obtained from the Rowe's equation (in degrees). I_c is the contraction factor, which is greater than zero and can be greater than unity. This factor is a suitable measure for quantitative assessment of severity of the effects of PSD, particle breakage, rotation, and rearrangement on the deformation of rockfill materials during the contraction phase. The greater values of this factor lead to smaller (more negative) mobilized dilation angles and steeper initial slope of and the result is the greater value of contractive volumetric strains caused by mentioned deformation phenomena. As mentioned, R_d is the dilation reduction factor and is always positive and smaller than unity. This factor acts as a measure for quantitative assessment of severity of the effects of PSD, particle breakage, rotation, and rearrangement on the deformation of rockfill materials during the dilation phase. The lower the value of this factor, the greater is the effects of mentioned deformation phenomena on the dilation of materials. In Eq.(13), these factors (I_c and R_d) are applied directly to the Rowe's mobilized dilation angle, ψ_{mR} , (not to the sine of the angle), so they can have a more precise effect on the changes of mobilized dilation angle. **Fig.7** shows the curve expressing the changes of mobilized dilation angle for different I_c values. As this figure shows, in the early stages of shear loading (contraction phase) as I_c increases, the mobilized dilation angle, ψ_m , becomes smaller (more negative). Obviously, materials with greater contractive volumetric strains will have a greater I_c value. These factors (I_c and R_d) will be obtained from the back-analyses on the triaxial test results. It should be mentioned that to incorporate the confining pressure on the mentioned deformation phenomena (especially the particle breakage phenomenon) the factors I_c and R_d can be considered as functions of confining stress.

4.1.5. Stress dependent peak friction and dilation angle

Laboratory tests indicated that the maximum friction angles depend on confining pressure. As a result, this dependency is accounted for in the constitutive model by the application of Eq.(14). This stress-dependent friction angle is used as maximum friction in Eq. (8)

$$\varphi_p = \varphi_s - \Delta\varphi \log\left(\frac{\sigma_3}{P_a}\right) \quad (14)$$

In the above equation, φ_p is the maximum friction angle, σ_3 is the minor principal stress, and φ_s is the angle of internal friction at $\sigma_3 = 100kPa$, P_a is the atmospheric pressure, $\Delta\varphi$ is the reduction in friction angle in respect to confining pressure for each 10 fold increase in σ_3 .

Also, for geotechnical materials the maximum dilation angle, ψ_p , depends on confining pressure. Generally the peak dilation angle, ψ_p , decreases with the increase of confining pressure, σ_3 .

4.1.6. Large-scale triaxial tests simulation

A series of conventional triaxial tests were conducted on the samples taken from the materials constituting the core; these tests were performed on samples with CL: GC ratios of 40:60, 60: 40 and 80:20. Moreover, a series of large-scale triaxial tests have been conducted in Karlsruhe University [58] on the coarse-grained materials of the shell, filter and drainage regions of this dam. As mentioned, these materials consist of crushed particles obtained by blasting in calcareous conglomerate borrow area of MES Dam. There are several methods for scaling down the size of the materials, e.g., scalping (cut-off), replacement of materials, generation of quadratic size distribution curve and parallel gradation techniques [11,59] The last method was found to be more appropriate for this materials. The large scale triaxial tests were performed on specimens with a height of 80 centimeters and a diameter of 80 centimeters. Other researches have shown that the values of marshal particle breakage Index [8] for these kind of rockfill materials are significantly high [54,55]. So, this material has a high potential for crushing under shearing.

To extract some parameters of this modified constitutive model for the materials of the dam body, the conventional and large scale triaxial tests on the body materials were simulated in FLAC software environment. All functions and equations described in the previous sections were coded into the constitutive model by FISH language embedded in the software. Simulations were conducted on a single element with axial symmetry and appropriate boundary condition (See Fig.8). In each step of analysis, the coded program computed the plastic shear strain and updated the model parameters based on the defined functions and equations. For instance, **Figs. 9 and 10** show the simulations conducted for rockfill materials used in regions 3A and 3C under initially dry and saturated conditions. These figures demonstrate the better simulated volume change behavior attained by the mobilized dilation angle equation proposed in this paper (Eq.(13)) rather than Rowe's Equation (Eq.(7)). The final parameters of this modified constitutive model for the different regions of the dam are shown in **Table 5**. In this analysis, the cohesion C of materials was not considered to be mobilized, and was set equal to the intercept of the tangent to the Mohr's circles resulted by triaxial tests. In this table, n and K parameters, in Eq. (5), were determined by Duncan and Chang' proposed method [16] using the laboratory results of triaxial tests. Poisson's ratio (ν) is considered as a constant value, determined from initial slope of $\varepsilon_v - \varepsilon_1$ curve of the laboratory results. Initial mobilized friction, ϕ_0 , in Eq. (6), determined from initial part of $q - \varepsilon_1$ curve, where q is deviator stress. This parameter can be defined as a function of confining stress. The response of material in the elastic domain of constitutive model is controlled by these five parameters. The peak friction angle, φ_p , and peak dilation angle, ψ_p , in Eqs. (6) and (7) defined as logarithmic functions of confining stress. These functions determined from plotting these parameters versus logarithm of confining stress. As mentioned in previous section, contraction factor, I_c , and reduction factor, R_d , in Eq. (13) control the shape of $\varepsilon_v - \varepsilon_1$ curve in the contraction and dilation phases under shearing, respectively. As shown in **Fig .7**, these two parameters were extracted for each confining stress through trial-analyses, and they can be defined as a function of confining stress.

As shown in **Table 5**, the exponent (n) of elastic modulus dependence on confining stress for regions 3A and 3C under initially dry conditions is negative. This means that as confining stress increases, the elastic component of Young's modulus of these materials decreases (see Eq. (5)). This phenomenon can be attributed to more intense particle breakage of the material. The maximum values of I_c were also obtained for regions 3A and 3C, which again points to more intense breakage than other regions. Considering the fine-grained nature of core materials which makes the breakage next to impossible, the I_c of these materials were considered to be unity.

Comparing the volumetric strains of initially dry and saturated specimens at a same confining stress (see **Figs 9a and 10a**) shows that initial saturation can significantly increase the volumetric strain, as it can facilitate the sliding of particles over each other. This indicates that during the

construction, the rockfill materials of the shell should have been extensively wetted with high pressure water jets to prevent significant settlements of the upstream shell during the first impounding. Given the noticeable stiffness of surrounding rock mass, it was simulated with a linear elastic constitutive model. **Table 6** shows the parameters of this material.

4.2. Flow modeling

Application of coupled mechanical-flow analysis necessitates the introduction of formulation of fluid flow. The formulation implemented in the FLAC3D software [51] is presented in Appendix A. In this formulation, the permeability tensor \mathbf{K} is split into an intrinsic part \mathbf{K}_{int} and a relative part k_r :

$$\mathbf{K} = \mathbf{K}_{int}(n)k_r(S_w) \quad (14)$$

The intrinsic permeability \mathbf{K}_{int} reflects the influence of the porous matrix on the water transfers. That is to say, intrinsic permeability is the saturated soil water permeability and should depend on the total porosity n . The relative permeability reflects only the fluid properties. Therefore, it is assumed to be a function of the saturation degree S_w , which may be computed by Eq. (15):

$$k_r(S_w) = (2 - 3S_w)S_w^2 \quad (15)$$

Based on some simplifying assumptions made in FLAC3D, **Fig. 11** is employed as the predefined retention curve of the model to explain the relation between saturation degree and interstitial fluid pressure. In this figure, σ_T is the fluid tension limit (negative number) and physically it is a tension which would be sustained by pore water. As the fluid pressure tries to drop below the set limit, cavitation occurs, and the interstitial pressure is then set to zero. Depending on conditions, the degree of saturation may also fall below one. If the saturation degree remains equal to one, further expansion of the pore volume could cause negative pore pressures to increase again [51].

The value of intrinsic permeability \mathbf{K}_{int} or the saturated soil permeability in each part of the numerical model was determined based on the results of permeability and consolidation tests. For simplicity's sake, we assumed the \mathbf{K}_{int} of all regions of the numerical model, except the core, to be a constant value (based on the initially estimated porosity). A series of consolidation tests were conducted on the core materials, and the results of these tests show that vertical permeability of the core increases with the increase of vertical effective stress. Statistical operations performed on the test results yielded a basic power function for vertical permeability of the core materials. The only variable of this function is the vertical effective stress, σ'_y . Ultimately, the exact values of the coefficients of this basic function and the horizontal-to-vertical permeability ratio of the core materials were determined through trial-analyses:

$$K_{int(y)} = \begin{cases} 2 \times 10^{-8} & \sigma'_y \leq 150 \\ 1.82 \times 10^{-6} \left(\frac{\sigma'_y}{P_a}\right)^{-0.48} & \sigma'_y \geq 150 \end{cases} \quad \frac{K_{int(x)}}{K_{int(y)}} = 2.5 \quad (16)$$

In the above equation $K_{int(x)}$ and $K_{int(y)}$ are the horizontal and vertical saturated permeability coefficients (in cm/s), and σ'_y is the vertical effective stress (in kPa). This function, also, was coded by the FISH programming language. **Tables 5 and 6** present the intrinsic permeability of the porous media for other regions.

4.3 Sequences of modeling

The dam construction was finished after about five years in November 2000. In this analysis, forty two layers were used in simulation of construction. A month after the end of construction, the first filling started. At the beginning of the first filling, the reservoir water level was +255. Due to heavy rains and floods, the reservoir water level climbed by an average of 2 m/day (with peak increase of 6 m/day) to about 57 meters in 28 days; then it climbed another 18 meters in the next 30 days (by an average of 0.6 m/day) and after about another 16 months, the reservoir water level reached its normal level (+371). In this model, the four impounding effects were idealized in six stages (see **Fig.3**). The first three impounding effects (E1 to E3) were simulated by applying the pore pressure proportionate to water level (at each stage of impounding) on the nodes of upstream shell, foundations, and filter elements and applying a water pressure on the outer surface of the upstream shell and foundation. The collapse settlement was simulated using the practical technique proposed by Mahinroosta and Alizadeh [50]. This technique has two steps: i) reducing all component of effective compressive stress of recently saturated layer with multiplying them by stress reduction coefficient (C_c), and ii) allocating the saturation parameters to recently saturated layer. The stress reduction coefficient (C_c) is less than unity. The lower the value of this coefficient, the greater is the collapse settlement. This coefficient should be determined by a special triaxial tests on the materials. **Fig.12** shows an idealized triaxial test of this type. According to **Fig.12** which is a hypothetical stress path, when a dry rockfill medium is submerged, internal stresses decrease vertically from its initial value σ_f to the collapse stress σ_c , therefore, this coefficient, C_c equals the ratio σ_c / σ_f . Dependency of the collapse phenomenon to confining pressure has been observed by a number of researchers [50,52,60- 62]. So, this coefficient was defined as function of confining stress [50]. Unfortunately, this kind of special triaxial tests was not conducted in this project. So, this coefficient was determined from trial-analyses for each stage of impounding (see **Table 7**). As can be seen in Table 7, this coefficient is smaller in the early stages of impounding. This is because the lower parts of the upstream shell are under greater confining stresses, and this naturally intensifies the particle breakage, ultimately causing the collapse deformations of these early stages to be greater than those of later stages.

4.4. Calibration of the 3D numerical model

The formulation of analytical model was carried out within the continuum media mechanic framework. So, According to a research by Pogano et al. [63] the back-analyses must be based upon quantities having a higher degree of representativeness. They showed that the displacements and pore water pressure (assuming a hydraulic equilibrium in porous media) have a higher representativeness than other quantities (e.g., stress, strain, etc.); so in this paper, the calibration of analytical model was performed accordingly. **Fig.13** show the variation of measured and calculated values of pore pressure at the main cross chainage (+260m) during the construction and first impounding. **Figs. 14 and 15** show the variations of measured and calculated value of deformation through the course of analysis. These Figs show a good agreement between measured and computed values which in turn points to highly accurate calibration of three-dimensional analytic model.

5. Numerical Results and discussion

Figs. 16 and 17 show the contour distribution of pore pressure in the main cross-section (CH.260) at the end of construction and first impounding, respectively. As mentioned, the upper part of the core has been placed on the dry side of standard optimum moisture content, so during the construction the excess pore pressure could not develop in this part. But a very high excess pore pressure was developed at the lower parts of the core. As mentioned in section 3, a number of causes like the fact that lower parts of the core have been compacted wet-of-optimum (high degree of saturation), the very low permeability of the core, and the relatively high rate of construction have led to significant development of excess pore water pressure in the core during the construction.

According to Eq. (16), as excess pore pressure dissipates and vertical effective stress increases, the permeability decreases exponentially. **Fig.18** shows the contour distribution of vertical permeability in the main cross-section (CH.260) at the end of construction. Reduced permeability of regions around the core and adjacent to the filters leads to slower rate (speed) of dissipation and consolidation. In the core, from the trial-analyses, horizontal-to-vertical permeability coefficients ratio is 2.5 (see Eq. (16)). So, the contours of excess pore pressure have an ellipse shape. At the lower levels of the core, almost constant excess pore pressure is prevailing between the center of the core and at the downstream or upstream locations of it. These results were confirmed by the data obtained from EPs installed at those locations (see section 3). During the first impounding, due to above causes, the core is almost under undrained condition. The water pressure acting on the core and saturation collapse in upstream shell (see Fig.3) can be important factors contributing to the increase of excess pore water pressure in the core

Figs. 19 to 20 show the contour distribution of vertical effective stress in the main cross-section at the end of construction and first impounding. The middle regions of the core at the lower parts have (approximately) zero-effective stress and exhibit an almost incompressible (quasi-fluid) behavior. As mentioned in section 3, the significant development of excess pore water pressure in the lower parts of the core and very low rate of its dissipation are the main causes of forming this near-zero effective stress zone. This result is consistent with the observations (see fig.4 (a)).

During the impounding, the effect of buoyancy forces reduces the vertical effective stresses in the upstream shell. On the other hand, the core is almost under undrained condition, so the water pressure acting on the upstream side of the core increases the total stress. As a result, the size (or breadth) of regions with zero-effective stress and positioned next to (or near) the upstream side of the core decreases.

Figs. 21 show the contour distribution of shear strain in the main cross-section at the end of first impounding. In this figure, the oblique shear zones are clearly visible. As mentioned, because of high excess pore water pressure, the lower parts of the core exhibit an almost incompressible behavior. During the construction, the stage loading due to the placement of the upper parts of embankment is the main cause of creating these oblique shear zones. During the impounding, the combined effects of shear forces caused by the differential deformations of the upstream shell relative to the core and water pressure acting on the core lead to further development of plastic shear deformations in the core. Increasing shear strains at the upstream core can provide stress conditions for forming hydraulic cracks. Moreover, in the core, such shear movement would be associated with a volume decrease (contractive behavior) and would more likely lead to a further rise rather than a relief of excess pore water pressures

The direction of principal stresses σ_1 and σ_3 acting in the plane perpendicular to the dam axis in the main cross-section at the end of first impounding is shown in **Fig. 22**. At lower levels of embankment, the identical magnitude of principal stresses in the central regions of the core and the rotation of directions of principal stresses in the vicinity of the filters are clearly evident. This result is consistent with the observations (see Fig.4 (a)). As mentioned earlier, rotation of directions of principal stresses is caused by the oblique shear zones (see Fig.4 (b)).

Figs.23 show the contour distribution of tensile stress parallel to dam axis in the 3D model surface at the end of first impounding. This figure shows that those upper parts of the dam body that are positioned next to abutments have a tensile stress condition. These tensile regions have the greatest potential for developing transverse tensile cracks. The tensile region next to the right abutment is more extensive than one next to the left abutment. These results correspond to field observations. At the end of first impounding, Based on the results of this three-dimensional model, the maximum depth of the tensile region in the dam body adjacent to the right and left abutments is almost 2 meters.

Figs.24and 25 show the contour distribution of vertical and horizontal deformations in the main cross-section at the end of construction. Also, deformed mesh of the main cross-section at the end of construction is shown in **Fig.26**. As mentioned, during the construction, because of developing very large excess pore pressure in the lower levels of the core and dissipating with very low rate, the consolidation settlements have been insignificant. Settlements of the shells are mainly due to the effects of PSD, particle breakage, rotation, and rearrangement phenomena; but those of the core are caused by internal oblique plastic shear movements. These oblique shear zones also lead to lateral deformations at the lower part of the core (bulging). The stage loading due to the placement of upper part of the dam puts the lower part of the core (which exhibits a quasi-fluid behavior) under vertical pressure. With time, the lateral deformation of lower parts of the core causes the upper part of the dam to settle. In the shells, the region 3B has lower compressibility properties than other regions. So, the settlements of this zone are larger than those occurred in other zones. Schematically, **Fig.27** shows a sketch of deformation mechanism of this dam during the construction.

The contour distribution of settlement the deformed mesh of the main cross-section and horizontal deformation in the main cross-section at the end of the first impounding are shown in **Figs.28 to 29**. Also, **Figs.30** show the contour distribution of settlement on the 3D model surface at the end of first impounding. The maximum settlements recorded inside the dam and on its crest are 5.93 and 2.2 meters, respectively. The upper parts of the upstream shell-core interface exhibit significant differential settlements. These differential settlements can cause longitudinal cracks in the crest during the operation. The lateral deformations in the lower parts of the core, especially in its upstream side, are smaller in comparison with the end of construction. This because of applying water pressure on the upstream face of the core. **Fig.31** shows deformed mesh of the main cross-section at the end of first impounding. This figure shows berm-like subsidence in the upper parts of the upstream side of the body. This feature was consistent with the field observations. The collapse settlements of the upstream rockfill shell are the main cause of this deformation behavior during the first impounding. **Fig. 32** shows the schematic sketch of the deformation mechanism of the dam during first impounding.

Now, 14 years after the first impounding, the excess pore pressure of the dam shows surprisingly little dissipation (at most 10% in some piezometers), so consolidation settlements of the core are still negligible; but deformations of the dam surface are progressing with a significant rate. This progress is such that the crest now exhibits an approximately 5-meter settlement. Visibly deep transverse tensile cracks have appeared

on the crest situated next to the abutments, and an unusual berm-like longitudinal subsidence is observed on the upstream surface. So, the long-term deformation mechanism of this dam can be the subject of further researches.

6. Conclusions

The results of this study can be summarized in two parts:

In part one, this paper proposed a numerical method, based on a well-known elasto-plastic hardening/softening constitutive model and Rowe's stress-dilatancy theory, to simulate the deformation behavior of coarse-grained materials, like rockfills, due to particle size distribution (PSD), particle breakage, rotation, and rearrangement under shearing. This method suggested a new mobilized dilation angle function that is a new version of Rowe's mobilized dilation angle function. This function has two factors, contraction factor, I_c , and reduction factor, R_c . These two parameters control the variation of plastic volumetric strain in the contraction and dilation phases under shearing, respectively. The results of large scale triaxial test conducted on the rockfill shell of MES dam shown this material has a high potential of particle breakage which leads to high amount of volumetric plastic strains. The numerical simulation of these triaxial tests indicated that the constitutive model modified with new dilation mobilized dilation angle function, can estimate deformation behavior of MES rockfill materials accurately.

In part two, To achieve the main objective of this study, first the data recorded by the instruments installed inside the dam body were carefully studied and then a three-dimensional numerical model of the dam was developed to simulate its deformation behavior. The calculations were carried out via simultaneous use of flow and mechanical formulations. The mechanical behavior of embankment materials was idealized by the modified hardening / softening constitutive model. The main results of this part of research are as below:

- 1) A number of factors like the fact that lower parts of the core have been compacted wet-of-optimum, the very low permeability of the core, and the relatively high rate of construction have led to significant development of excess pore water pressure in the core during the construction. Considerable pore pressure of the core has led to formation of a region with near-zero effective stress in the lower parts of the core. This region has an isotropic stress condition, so that part of the core which is within this region is expected to exhibit an incompressible or quasi fluid behavior. In addition, the reducing permeability of the core with further dissipation of excess pore pressure next to the filters has led to slowing rate (speed) of dissipation of excess pore pressure. The insignificant dissipation of excess pore pressure in the core has led to unexpectedly small consolidation settlements during the construction.
- 2) The stage loading due to placement of the upper layers of the dam has led to formation of oblique plastic shear zones inside the lower parts of the core. These plastic shear movements have led to settlement of the upper part of the embankment and lateral deformation (bulging) of the lower part of the core. The increased pore pressure of the core can be another outcome of these plastic shear deformations. Fairly significant deformations observed in the shell are mainly due to the effects of PSD, rearrangement, rotation, and especially particle breakage phenomena on the deformation of the shells, drainages and filters. To simulate the volumetric deformations caused by these phenomena, in part one, authors modified the Row's mobilized dilation function and incorporated it into the hardening/softening constitutive model.
- 3) During the first impounding, water presser acting on the core and most likely the oblique plastic shear movements inside the core have increased its excess pore pressure. On the other hand, the water presser acting on the core (E1) has increased its effective stress in the upstream side of the core and therefore reduced the extension of the zero-stress region, making it smaller than what it was at the end of construction.
- 4) During the first impounding, collapse settlements of the upstream rockfill shell have been the most important factor influencing the deformation behavior of the dam during the first impounding. This study utilized a concept known as "stress reduction factor" to simulate the collapse settlement phenomenon. The collapse settlements of the upstream rockfill shell have caused significant differential settlements in the upper levels of the upstream filter-core interface and have also led to further development of plastic shear deformations inside the core.
- 5) At the end of first impounding, those upper parts of the dam body that are positioned next to abutments have tensile stress condition. These tensile regions have the greatest potential for developing transverse tensile cracks. The tensile region next to the right abutment is more extensive than one next to the left abutment. Based on the results of this three-dimensional model, the maximum depth of the tensile region in the dam body adjacent to the right and left abutments is almost 2 meters.

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Appendix A: Formulation of Fluid Analysis in FLAC software

The flow formulation implemented in the FLAC3D software [51] is presented as below:

Water transport is assumed to be governed by Darcy's law:

$$q = -K \cdot \nabla (u_w - \gamma_w Z) \quad (A.1)$$

Where q is the vector of water flow, u_w is the pore water pressure and K is the permeability.

Mass conservation equation of water phase can be written as:

$$\frac{\partial \xi}{\partial t} + \text{div} q = q_v \quad (A.2)$$

Where ξ is the variation of fluid content (variation of fluid volume per unit volume of porous material), and q_v is a scalar quantity related to the volumetric fluid source intensity.

Following the formulation of deformation–diffusion processes based on quasi-static Biot theory, Eq.(A.3) is implemented in FLAC3D.

$$\frac{\partial u_w}{\partial t} = M \left(\frac{\partial \varepsilon}{\partial t} - \alpha \frac{\partial \varepsilon_v}{\partial t} \right) \quad (A.3)$$

Where ε_v is the volumetric strain and M , α are the Biot modulus and the Biot coefficient respectively. Ignoring the compressibility of solid matrix (that is, grains) compared to that of the deformable skeleton, we have:

$$M = \frac{K_w}{n} \quad \alpha = 1 \quad (A.4)$$

Where K_w is the fluid bulk modulus.

The list of symbols and their description

Symbol	Description	Symbol	Description
f^s & f^t	Shear and tensile yield functions	ε_s^p	Plastic shear strain
σ_1 & σ_3	Maximum and minimum principal stresses	ε_f^s	Plastic shear strain at peak friction angle
σ'_y	Vertical effective stress	ε_1	Axial strain
φ_m	Mobilized friction angle	ε_v	Volumetric strain
φ_0	Initial mobilized friction angle	R_d	Reduction factor
φ_p	Peak friction angle	I_c	Contraction factor
φ_{cv}	Critical state friction angle	E	Young's modulus
φ_s	Angle of internal friction at $\sigma_3 = 100kPa$	n	Exponent of elastic modulus and porosity
$\Delta\varphi$	Reduction in friction angle in respect to confining pressure for each 10 fold increase in σ_3	K	Elastic modulus and permeability tensor
Q^S & Q^t	Shear and tensile plastic potential functions	v	Poisson
ψ_m	Mobilized dilation angle	P_a	Atmospheric pressure
ψ_p	Peak dilation angle	K_{int}	Intrinsic permeability
ψ_{mR}	Mobilized dilation angle obtained from the Rowe's equation	k_r	Relative permeability
C	Cohesion	S_w	Saturation degree
C_m	Mobilized cohesion	q	Deviator stress
σ_m^t	Mobilized tensile strength	d	Dilation
ε_v^p	Plastic volumetric strain	ρ_d	Dry density

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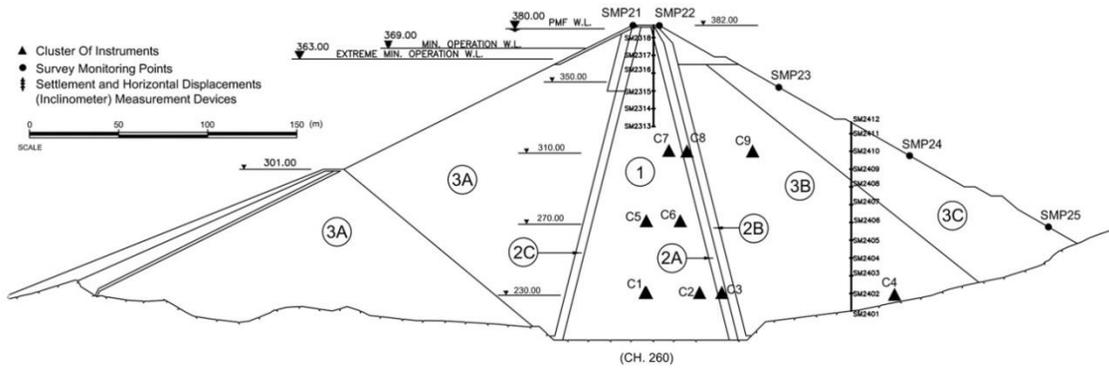


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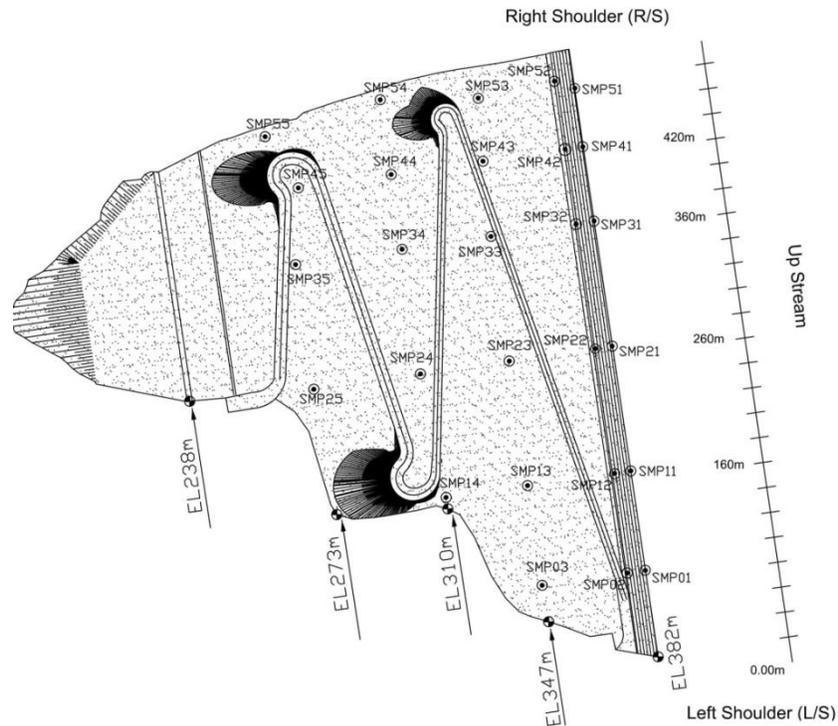


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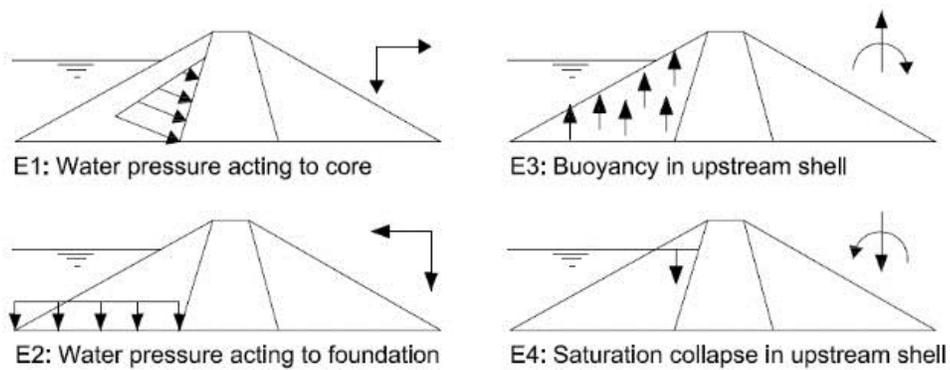


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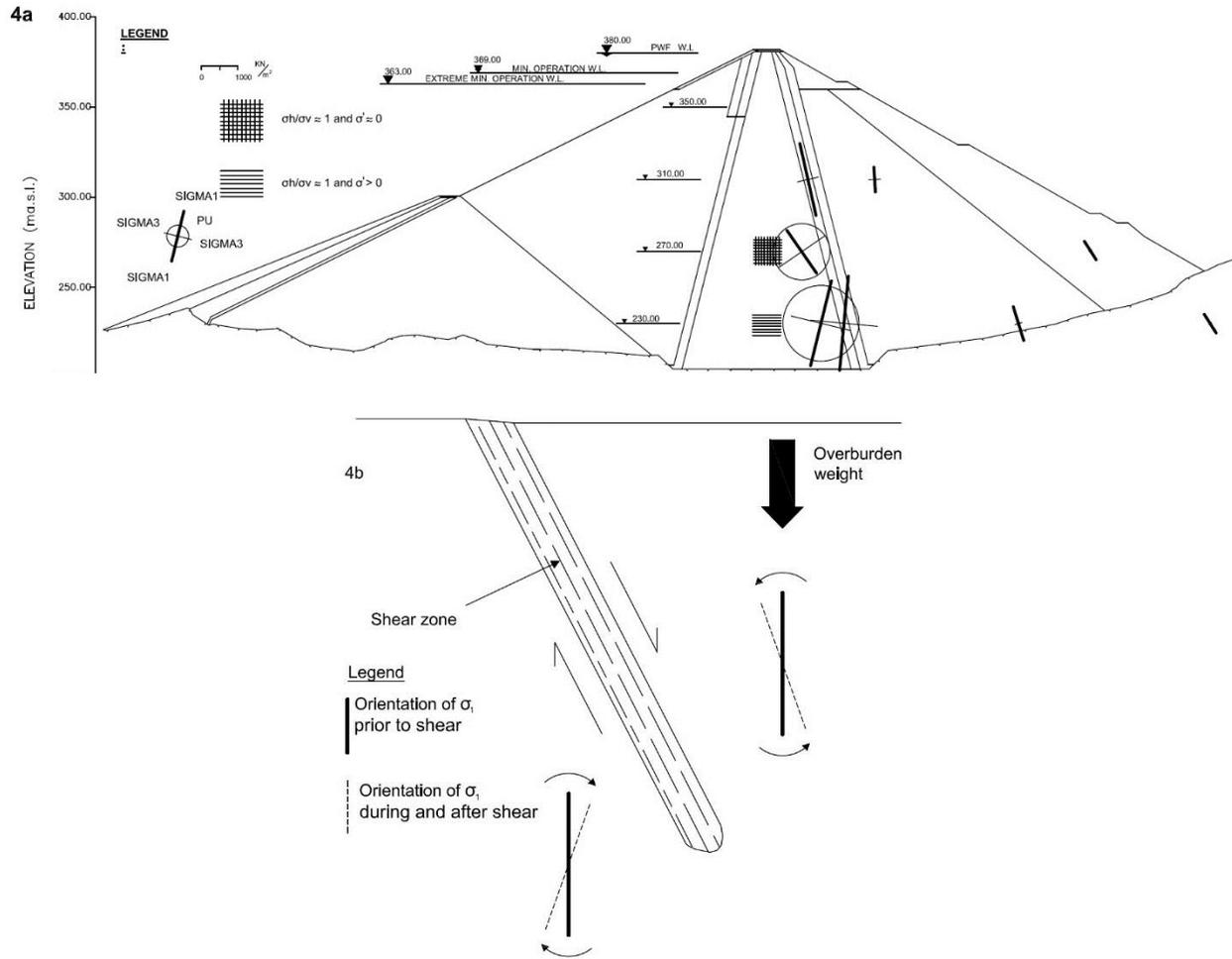


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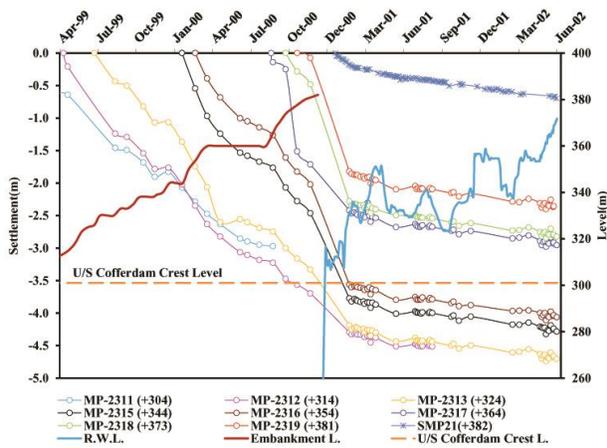


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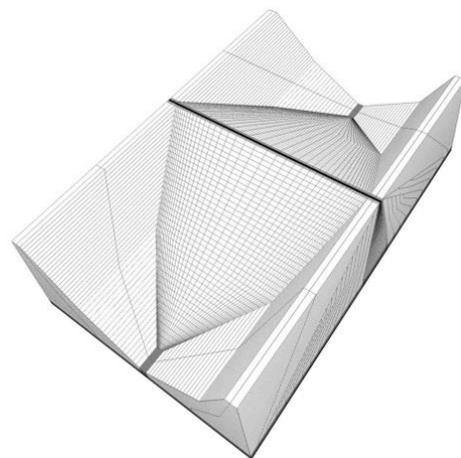


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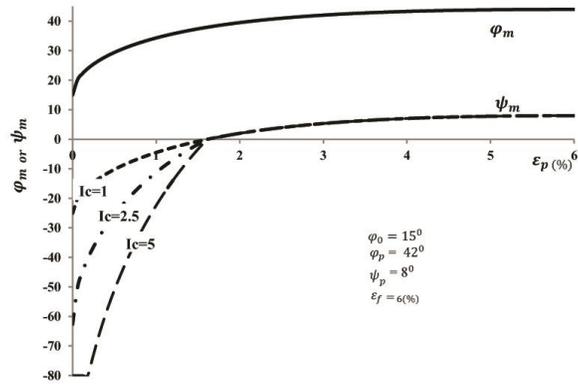


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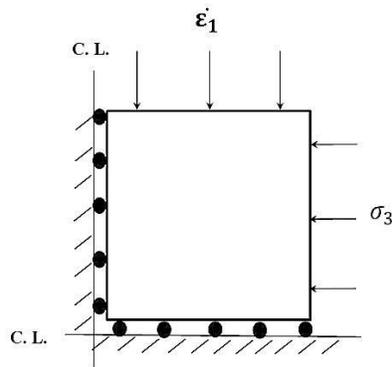


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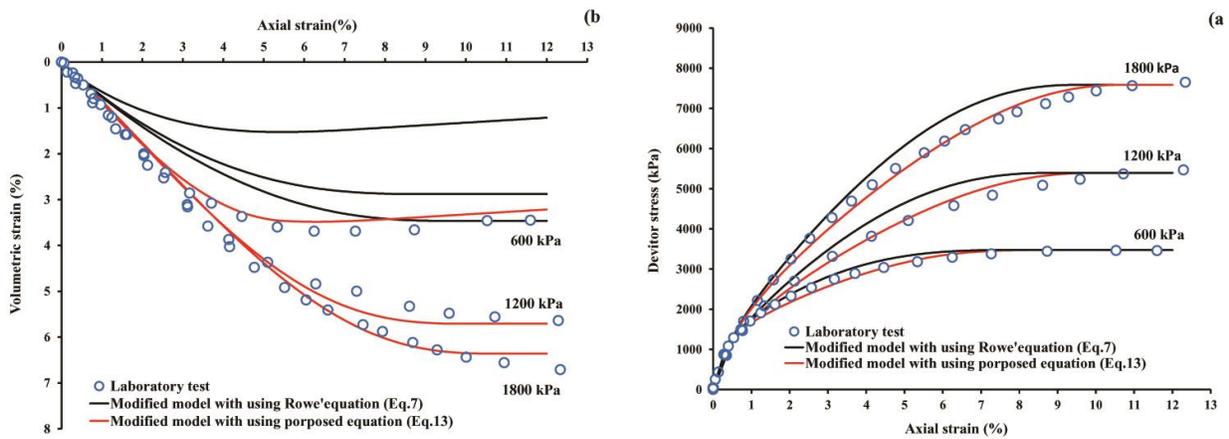


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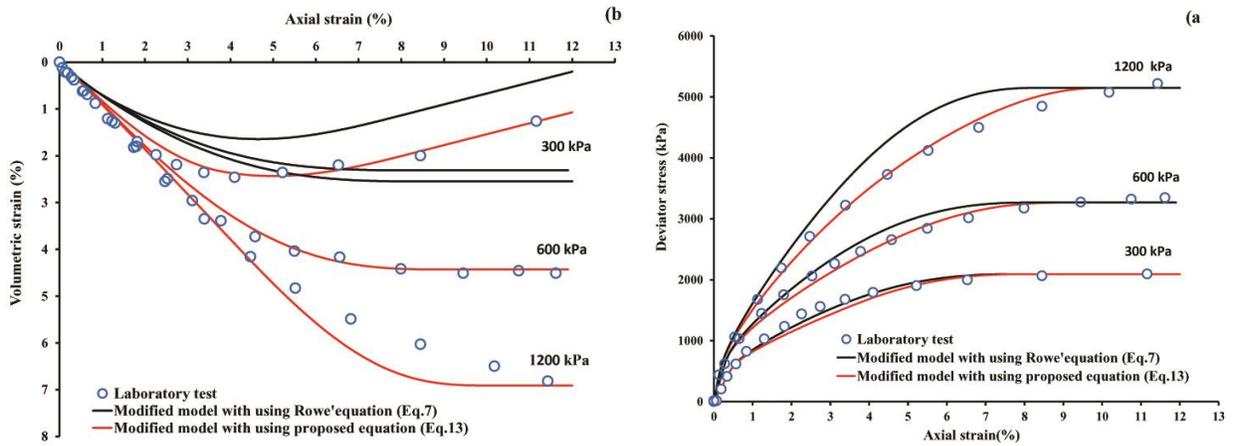


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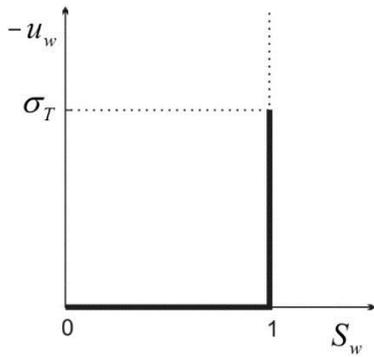


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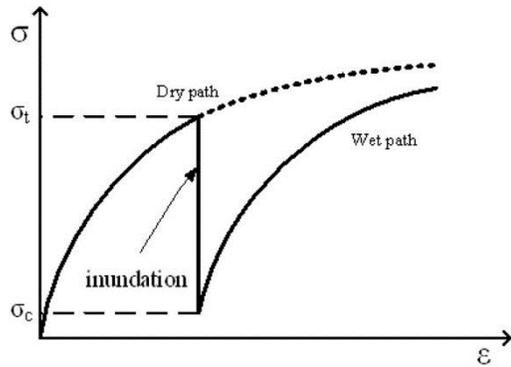


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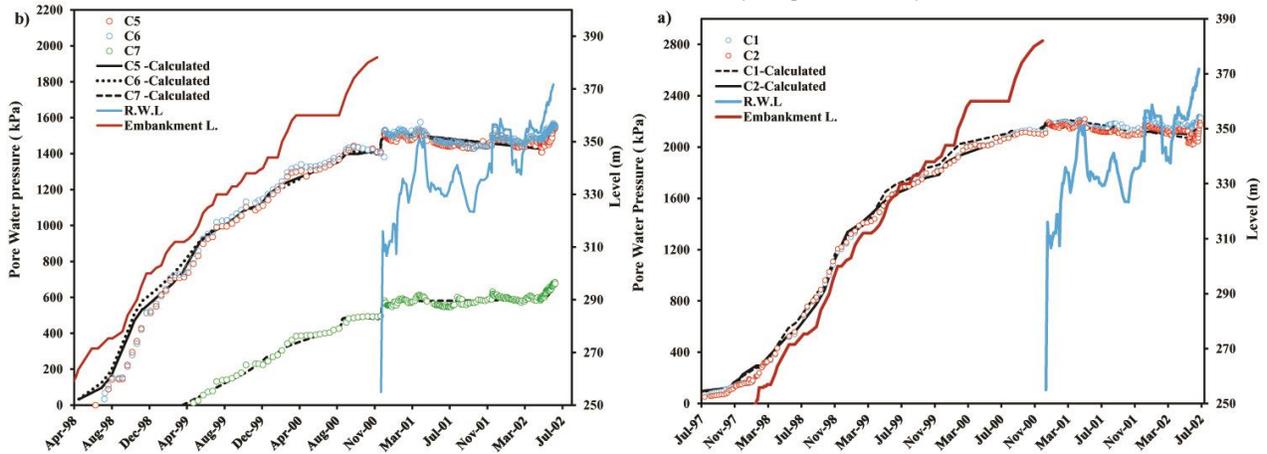


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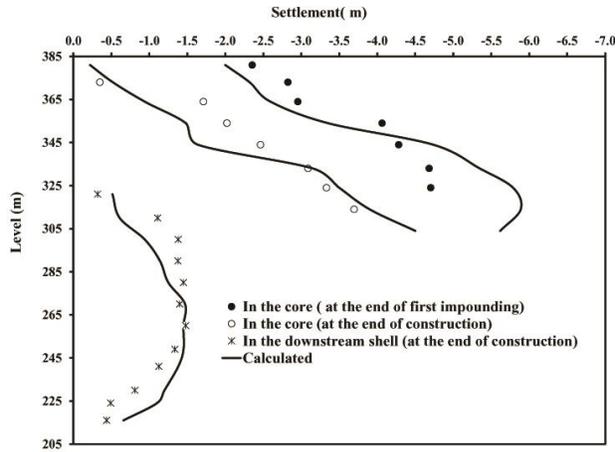


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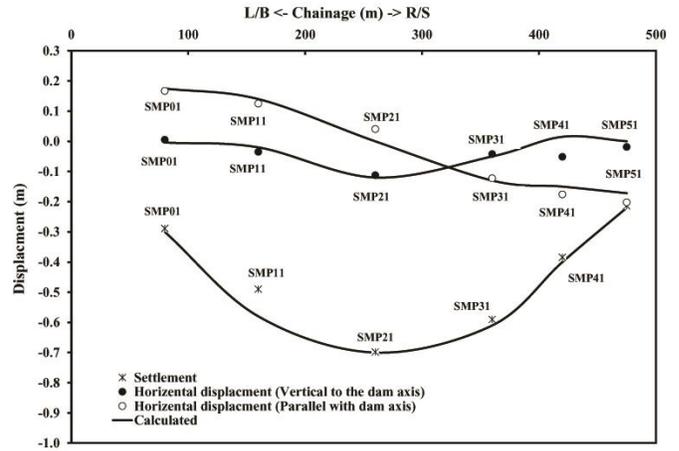


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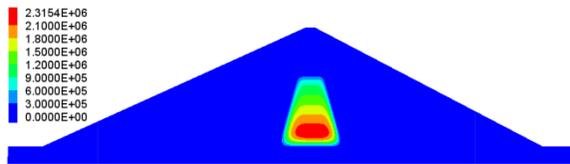


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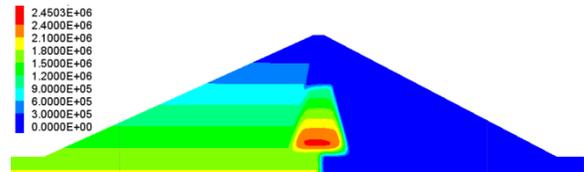


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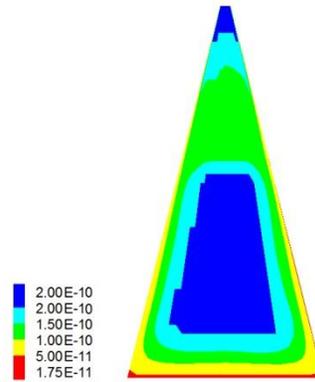


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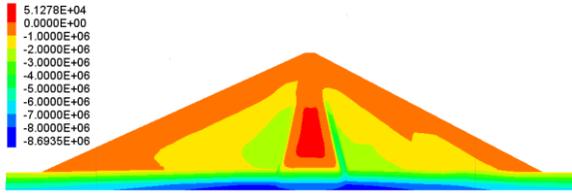


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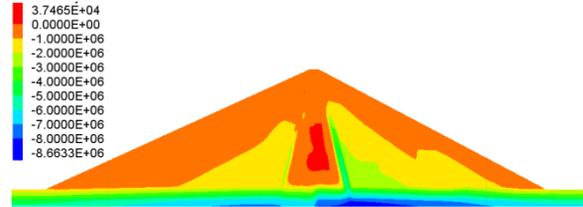


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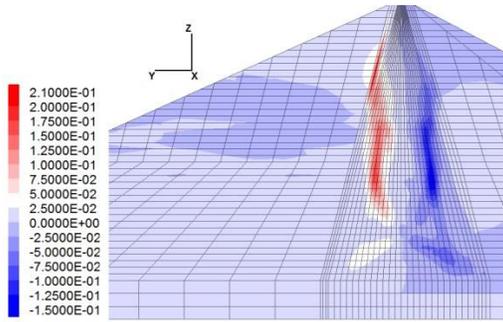


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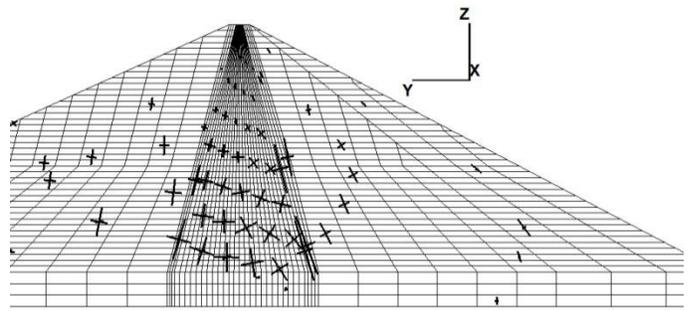


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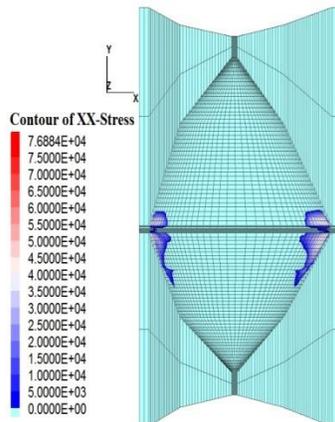


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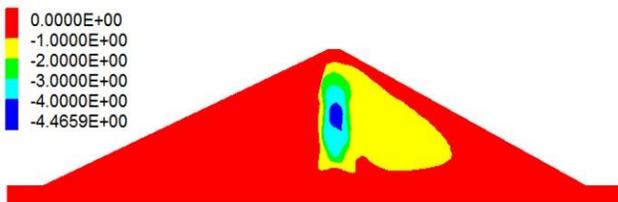


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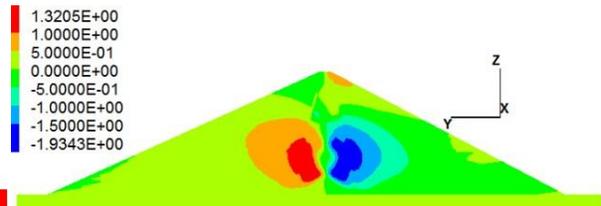


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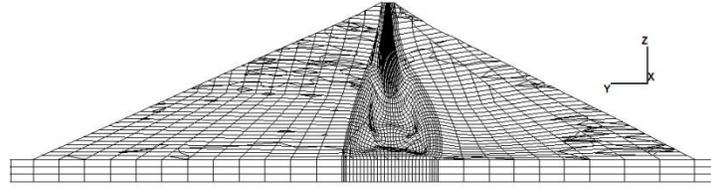


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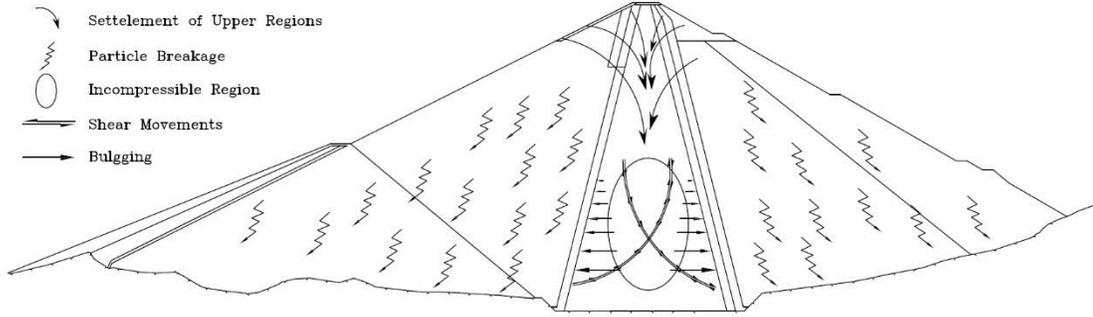


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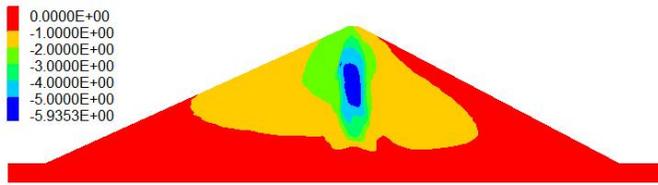


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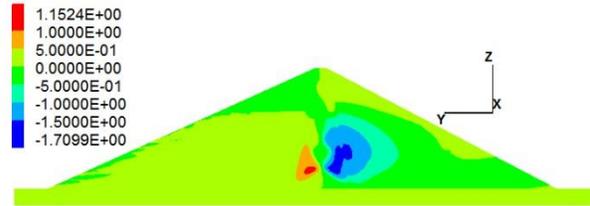


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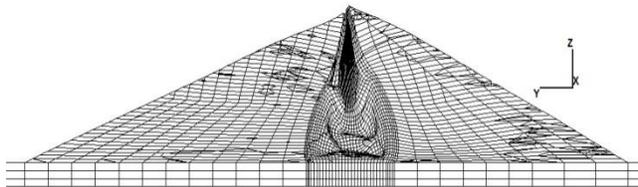


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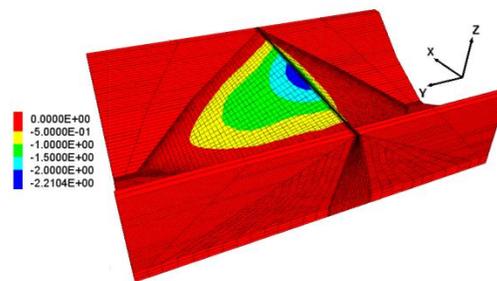


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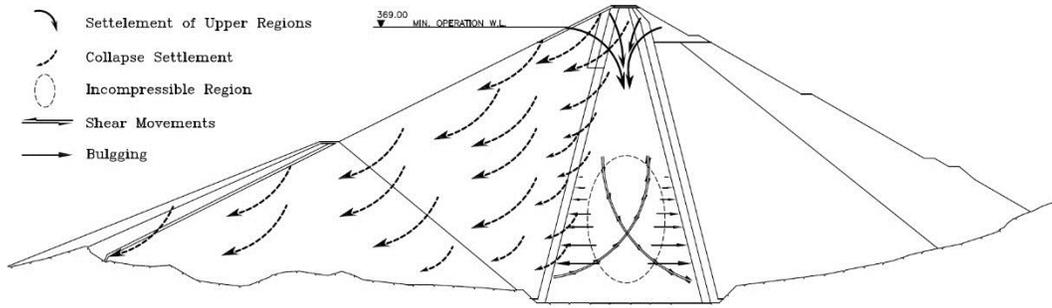


Fig.32. Schematic sketch of the deformation mechanism of the dam during the first impounding

Table 1. Pore pressure ratio, r_u , in the core

Chainage	Installation Level	End of Construction		End of First Filling	
		Center line	Downstream	Center line	Downstream
+160	+280	0.62	0.6	0.65	0.63
+160	+310	0.35	-	0.43	-
+260	+230	0.7	0.65	0.7	0.67
+260	+270	0.6	0.58	0.65	0.62
+260	+310	0.32	-	-	-
+360	+280	0.58	0.55	0.62	0.59
+360	+310	0.41	-	0.45	-
+430	+310	0.33	-	0.32	-

Table 2. Arching ratio, r_A , in the dam body

		End of Construction				End of First Filling			
		Core		Filter	Shell	Core		Filter	Shell
		Center line	Downstream	2A	3B	Center line	Downstream	2A	3B
+160	280+	0.66	0.66	-	-	0.68	0.68	-	-
+160	+310	-	0.58	1.35	1.08	-	0.62	1.29	1.36
+260	+230	0.68	0.77	1.38	0.49	0.7	0.78	0.99	0.47
+260	+270	0.59	0.61	-	-	0.61	0.63	-	-
+260	+310	-	0.43	1.45	0.81	-	-	1.37	0.68
+360	+280	0.59	0.65	-	1.01	0.6	0.65	-	0.94
+360	+310	-	0.58	1.59	0.68	-	0.66	1.49	0.53
+430	+310	-	0.61	1.22	1	-	0.61	1.22	0.85

Table 3. Real pore pressure ratio, R_u , in the core

Chainage	Installation Level	End of Construction		End of First Filling	
		Downstream	Center line	Downstream	Center line
+160	+280	0.91	0.94	0.93	0.95
+160	+310	-	0.6	-	0.68
+260	+230	0.95	0.9	0.96	0.89
+260	+270	0.96	0.99	1	1
+260	+310	-	0.7	-	-
+360	+280	0.94	0.89	0.99	0.93
+360	+310	-	0.68	-	0.74
+430	+310	-	0.55	-	0.52

Table 4. Ratio of vertical total stress to horizontal total stress, r_K , in the core

Chainage	Installation Level	End of Construction		End of First Filling	
		Downstream	Center line	Downstream	Center line
+160	+280	0.97	1	-	1.02
+160	+310	-	0.7	-	0.85
+260	+230	1.04	0.85	1	0.73
+260	+270	0.91	0.97	1.02	0.98
+260	+310	-	0.91	-	-
+360	+280	0.99	0.88	1.01	0.95
+360	+310	-	0.65	-	0.69
+430	+310	-	0.64	-	0.62

Table 5. The mechanical and flow parameters of the different regions of the embankment

Zone	3A,3C (Saturated)	3A,3C(dry)	3B	2A	2B,2C	Core
$\rho_d (\frac{gr}{cm^3})$	2	2	2	1.9	2.1	1.89
n	0.74	-0.12	0.05	1	0	0.8
K	556	3672	422	138	1663	120
ν	0.1	0.1	0.2	0.15	0.1	0.15
$c (\frac{kN}{m^2})$	25	65	40	10	20	30
$\varepsilon_f (\%)$	6	6	7	8.5	8.5	10
I_d	$0.77 \cdot (\frac{\sigma_3}{P_a}) + 0.25$	5	2.5	3.5	1	1
R_d	1	1	1	1	1	1
φ_p	$55 - 11 \log(\frac{\sigma_3}{P_a})$	$51.3 - 7 \log(\frac{\sigma_3}{P_a})$	$42.2 - 0.64 \log(\frac{\sigma_3}{P_a})$	$45 - 6.64 \log(\frac{\sigma_3}{P_a})$	$46.4 - 4.8 \log(\frac{\sigma_3}{P_a})$	30
ψ_p	$\begin{cases} 5.5 & \frac{\sigma_3}{P_a} \leq 3 \\ 0.0 & \frac{\sigma_3}{P_a} > 3 \end{cases}$	$\begin{cases} 1.5 & \frac{\sigma_3}{P_a} \leq 6 \\ 0.0 & \frac{\sigma_3}{P_a} > 6 \end{cases}$	0	0	$1.15 - 0.83 \log(\frac{\sigma_3}{P_a})$	0
φ_0	$42 - 30 \log(\frac{\sigma_3}{P_a})$	$33.6 - 16.6 \log(\frac{\sigma_3}{P_a})$	$4.2 - 20 \log(\frac{\sigma_3}{P_a})$	$40 - 2.7 \log(\frac{\sigma_3}{P_a})$	$30 - 2.6 \log(\frac{\sigma_3}{P_a})$	0
$K_{int} (cm/s)$	1×10^{-1}	1×10^{-1}	1×10^{-4}	2×10^{-2}	1×10^{-2}	Eq.(21)

Table 6. the mechanical and flow parameters of the rock mass

Model	$\rho_d (\frac{gr}{cm^3})$	E(GPa)	ν	$K_{int} (cm/s)$
Linear elastic	2.4	2	0.25	5×10^{-8}

Table 7. The range of reservoir water level, duration and stress reduction coefficient for each stage of impounding

Stage of impounding	1	2	3	4	5	6
Range of reservoir water level (m)	255-288	288-312	312-328	328-335	335-357	357-371
Duration (day)	1	9	49	12	446	34
C_c	0.1	0.1	0.8	0.8	0.8	0.8

The list of symbols and their description

Symbol	Description	Symbol	Description
f^s & f^t	Shear and tensile yield functions	ϵ_s^p	Plastic shear strain
σ_1 & σ_3	Maximum and minimum principal stresses	ϵ_f^s	Plastic shear strain at peak friction angle
σ_y'	Vertical effective stress	ϵ_1	Axial strain
φ_m	Mobilized friction angle	ϵ_v	Volumetric strain
φ_0	Initial mobilized friction angle	R_d	Reduction factor
φ_p	Peak friction angle	I_c	Contraction factor
φ_{cv}	Critical state friction angle	E	Young's modulus
φ_s	Angle of internal friction at $\sigma_3 = 100kPa$	n	Exponent of elastic modulus and porosity
$\Delta\varphi$	Reduction in friction angle in respect to confining pressure for each 10 fold increase in σ_3	K	Elastic modulus and permeability tensor
Q^s & Q^t	Shear and tensile plastic potential functions	v	Poisson
ψ_m	Mobilized dilation angle	P_a	Atmospheric pressure
ψ_p	Peak dilation angle	K_{int}	Intrinsic permeability
ψ_{mR}	Mobilized dilation angle obtained from the Rowe's equation	k_r	Relative permeability
C	Cohesion	S_w	Saturation degree
c_m	Mobilized cohesion	q	Deviator stress
σ_m^t	Mobilized tensile strength	d	Dilation
ϵ_v^p	Plastic volumetric strain	ρ_d	Dry density

In the name of God

A brief biography:

First Author is Corresponding Author: Dr Ali Akhtarpour

Dr Ali Akhtarpour was graduated from polytechnic university of Tehran in BSc of civil engineering, MSc (Under supervision of Dr Abbas Soroush) and PhD (Under supervision of Dr Ali Khodaii) of geotechnical engineering in 2000, 2003 and 2011 respectively. Now he is a staff member (Assistant Prof.) of engineering faculty in the Ferdowsi University of Mashhad. Also he has some experiences on the embankment dam design, monitoring and numerical modelling for more than 18 years. He is the technical head of a consulting engineering company in Mashhad. You can find more about publications and interesting subject area in his personal homepage as below:

<http://akhtarpour.profems.um.ac.ir/>

Second Author: Morteza salari

Morteza Salari has graduated in MSc of Geotechnical Engineering from Ferdowsi university of Mashhad in 2009. He is now an Expert in Abpooy Consulting Engineering Company in Mashhad. Also he has some experiences on the field of embankment dam engineering for the 8 years. Also he is a PhD student in Ferdowsi university of Mashhad.