Predicting Hydraulic Fracturing in Hyttejuvet Dam

S. Mohsen Haeri
Sharif University of Technology, Tehran, Iran

Danial Faghihi
Sharif University of Technology, Tehran, Iran

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S .Mohsen. Haeri
Professor, Civil Engineering Department,
Sharif University of Technology, Tehran, Iran,
smhaeri@sharif.edu

Danial Faghihi
MSc Student, Civil Engineering Department,
Sharif University of Technology, Tehran, Iran,
d_faghihi@civil.sharif.edu

ABSTRACT

Hydraulic fracturing can occur in the clay core of earth and rockfill dams if the vertical effective stress in the core is reduced to the levels that are small enough to allow a tensile fracture to occur due to hydraulic pressure of the seeping water. This situation may arise if the total stress in the core is reduced by the “arching effect” where the core settles relative to the filter or rock-fill shell of the dam. Water pressure increase in the core which occurs on first impounding of water, may reduce effective stresses further, and if they reach low enough values, a fracture will occur. The design of earth dams (especially those with thin vertical central cores) to resist hydraulic fracture is therefore of great importance, as there have been several dam failures in the past that have been attributed to the hydraulic fracture.

In this paper, the behavior of Hyttejuvet Dam, which was thought to have failed due to hydraulic fracturing, is studied. 2D coupled consolidation finite element analysis of the construction and first impounding of the rockfill dam was carried out with elasto-plastic model (Drucker-Preger/Cap model) using *ABAQUS* software. The result of the analysis with respect to the pore pressure and settlement in some parts of the dam are compared with the measured data from the instruments in the dam. According to the result of the comparison, the appropriate model for predicting the behavior of Hyttejuvet Dam is obtained. Also different criteria are used to predict the hydraulic fracturing of the dam. By comparing the results of the study using these criteria, one may be able to predict the hydraulic fracturing mechanism in the clay core of the studied dam.

INTRODUCTION

Hydraulic fracturing can occur in soils when the normal stress is reduced to zero or below zero by an applied hydraulic pressure, a phenomenon well known to the grouting industry in cases where grout is lost in boreholes due to excessive grouting pressure. In embankment dams, hydraulic fracturing can occur through the clay core of the dam if conditions are such that the pore pressures due to the stored water are high enough to reduce the normal stress in the core to zero or below zero. If such a fracture does occur transversely through the core, a rapid and catastrophic failure will result, and so it is of interest to be able to check any potential core design for the likelihood of a fracture occurring.

One way of overcoming problems associated with such a fracture is to construct a sloping core so that the weight of the upstream rock-fill shell keeps the core in compression and thus reduces the likelihood of a tensile fracture. However, this option is not always possible. For example, in regions of high rainfall where the clay cannot be economically dried and has to be placed wet of the optimum moisture content, the stability of the upstream face may be a problem. A slope failure may occur through the wet core material during construction unless the upstream face has a very shallow and uneconomic slope. A vertical core may then have to be used.
A vertical core, especially if it is thin, can be vulnerable to fracture, as the clay in the core is more compressible than the rock-fill shoulders. The clay will tend to settle more than the rock fill, and can “hang up” on the rock-fill shoulders, lowering the stresses in the core. An increase in water pressure during first filling of the reservoir can then lead to a fracture, as the water pressures lower the effective stresses in the core even further.

There are few published field studies related to hydraulic fracturing in embankment dams (Kjærnsli and Torblaa 1968; Kjærnsli 1973; Vestad 1976; Vaughan et al. 1970; Penman and Charles 1981; Sherard 1986; Ng and small 1999), such as that which was carried out for the Hyttejuvet Dam in Valdalen, Norway. During the first impounding, leakage occurred suddenly when the water level in the reservoir reached a few meters below the regulated water level. An investigation was established which included sinking a series of boreholes at the crest of the dam (Kjærnsli and Torblaa 1968). The investigators also examined the deformation and stresses in the dam monitored during construction and filling of the reservoir. They concluded that the primary cause of the leakage was arching of the core, which eventually led to hydraulic fracturing when the reservoir was filled.

For predicting the hydraulic fracturing, first pore pressure and effective stress were determined in the medium and then the occurrence of crack was anticipated using a threshold criterion. The existence criteria are divided in three main groups: 1) Fracture Mechanics Criteria (Griffith 1921, 1924; Irwin 1960; J-Integral;…), 2) Yield and Failure Criteria (Komak panah 1989, Lo and Kaniare 1990…), 3) Experimental investigations on hydraulic fracturing (Bjerrum 1974; Jaworski 1981; Mordoch 1993; ……..).

However the fracture criteria for geotechnical material are limited. In addition the parameters needed for satisfying such criteria are not easily obtainable.

In this paper, a finite element analysis is performed to investigate the cause of hydraulic fracturing in Hyttejuvet Dam using *ABAQUS* software. The mathematical framework adopted in the analysis is based on a formulation that couples the equations of flow of pore water and deformation for a porous media. Furthermore, to investigate the hydraulic fracturing occurrences in the core, some criteria were used. By comparing the results of the study using these criteria, the advantages and disadvantages of each criterion in predicting the hydraulic fracturing in the clay core of the studied dam is obtained.

The analysis simulates the behavior of the dam during construction and filling of the reservoir. The measured and calculated pore water pressures were compared during construction of the dam and first filling. The results showed that hydraulic fracturing had probably occurred during the first filling of the reservoir, as concluded by the previous investigators (Kjærnsli and Torblaa 1968 ; Ng and small 1999). In addition, the analysis provides some insight into the cause of fracturing and mechanisms involved.

BACKGROUND

To put the analysis of Hyttejuvet Dam into context, some background information from the literature (Kjærnsli and Torblaa 1968; Kjærnsli 1973) is provided.

As part of the Röldal-suldal Power Project in Valdalen, Norway, Hyttejuvet Dam was constructed between 1964 and 1965. The dam is an earth and rock-fill dam and has a maximum height of 93 m. Figures 1 and 2 show the dam in plan and section, respectively. The dam has a narrow and almost vertical core, flanked by shoulders mainly constructed with gravel and quarried rock. The fill material for the core was a well-graded moraine.

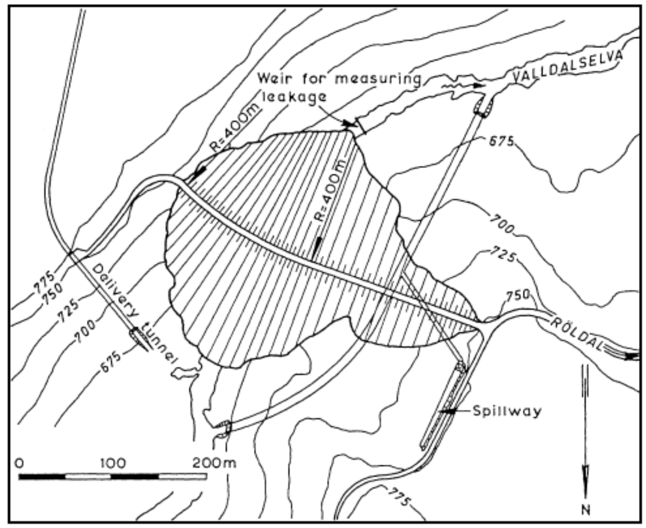

Fig. 1. Plan of Hyttejuvet Dam (reported from Kjærnsli and Torblaa 1968)

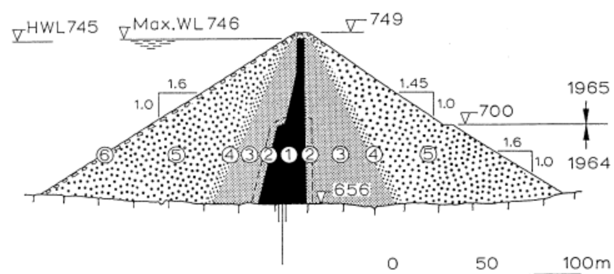

Fig. 2. Section of Hyttejuvet Dam (reproduced from Kjærnsli and Torblaa 1968). The elevation of 700 m, which was reached after the first construction season in 1964, is indicated. 1) moraine; 2) sandy gravel; 3) gravel; 4) tunnel spoil; 5) quarried rock; 6) quarried rock> 0.25 m3.

The grain-size distributions of the various materials used in the construction of the dam are given in Fig..3. The morainic material for the core was placed with the moisture content wetter than the Proctor optimum and in 0.25 m layers. The material was compacted by five passes of a bulldozer weighing no less than 15.2 t. The average degree of saturation as compacted was 0.90. The gravel fill for the shoulders was also placed wet of the optimum. Each layer of gravel deposited was about 0.5 m thick and sluiced with water at moderate pressure. No special compaction was applied to the gravel except normal construction traffic. The rockfill forming the main part of the shoulders was placed in lifts of 3–5 m and sluiced with water from pressure jets.

The construction of the entire dam took approximately 17 months and was completed in two construction seasons. Figure 4 shows the variation of the constructed height of the dam with time during construction.

After the first construction season in 1964, it was found that the pore water pressures measured on site were higher than anticipated. Subsequently, the design was revised to allow more rapid dissipation of pore-water pressures in the core. As a result, the width of the core in the upper part of the dam was reduced in the following construction season. This is indicated by a sharp change of the core thickness at the elevation of 700 m as shown in Fig.. 2.

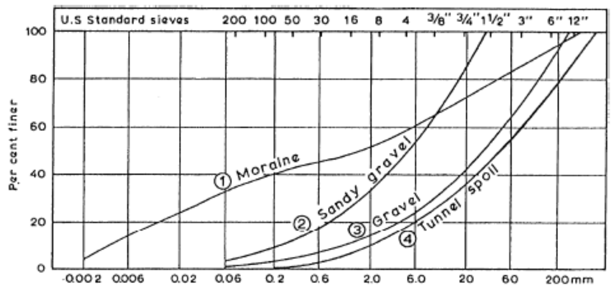

Fig. 3. Grading curves for the fill materials in Hyttejuvet Dam (reproduced from Kjærnsli and Torblaa 1968).

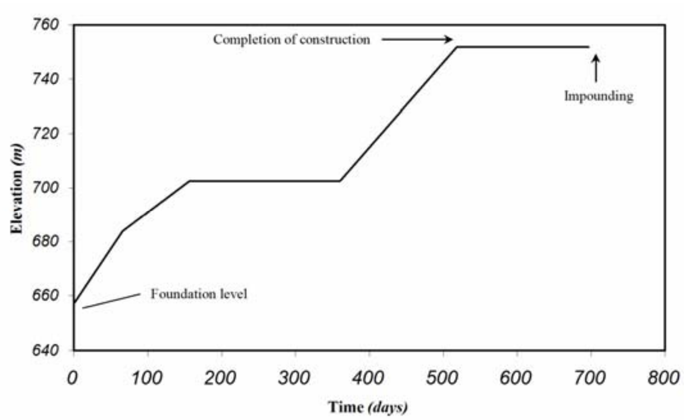

Fig. 4. Variation of the height of Hyttejuvet Dam with time during construction (0 days = 4 June 1964).

To monitor the performance of the dam at different stages, various types of instruments were installed. Pressure cells were installed in the core to monitor the pore water pressures and earth pressures. In addition, settlement bolts were placed along the crest of the dam and on the upstream slope face. An overflow weir was constructed downstream of the dam to measure the leakage through the dam when the reservoir was filled (see Fig.. 1).

Records and field investigation of leakage

According to Kjærnsli and Torblaa (1968), the first filling of the reservoir began in May 1966, about 6 months after completion of the dam. The measured leakage of the dam from the downstream overflow weir was initially negligible (1–2 L/s). However, when the water level in the reservoir reached a level about 7 m below the regulated elevation of 745 m, severe leakage was observed. Figure 5 shows the variation of the water level in the reservoir and leakage measured in relation to time. As indicated, the leakage increased considerably when the water level reached an elevation of 738 m. It was then necessary to reduce the rate at which the water level rose in the reservoir.

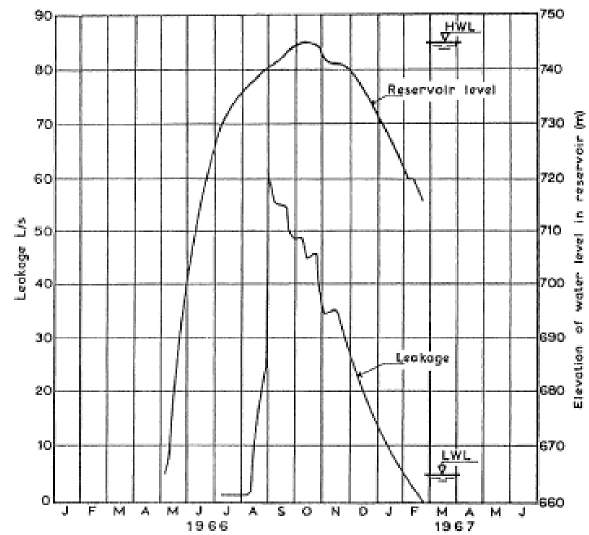

Fig. 5. Leakage record during first filling of the reservoir (reproduced from Kjærnsli and Torblaa 1968).

To investigate the leakage, boreholes were sunk at the crest of the dam and into the core. The boreholes were up to about 18 m deep, and percolation tests were carried out in the boreholes. It was clear that water either flowed out of or into the hole at certain depths in the core. The amount of grout required to fill several of these boreholes after the percolation tests were also considerable. While the tests were being performed, the water level in the reservoir continued to rise at a slower rate. The leakage recorded during this period also dropped. When the highest regulated water level was reached in October 1966, the water in the reservoir could be used for power generation. The water level in the reservoir was subsequently lowered and the leakage was also reduced further.

The remedial works began in the summer of 1967, some time after the first filling of the reservoir. Grout holes were drilled in the core down to the bedrock or to a maximum depth of 30 m. The maximum intake of grout was 53.7 m^3/m.
Kjærnsli and Torblaa (1968) investigated the cause of the leakage. They pointed out that the results of the percolation tests and the grouting record should not be considered as evidence of existence of hydraulic fracturing in the dam, as the pressures applied in these tests might be large enough to create hydraulic fracturing themselves. Kjærnsli and Torblaa concluded that the primary cause of leakage was the arching of the core, which led to hydraulic fracturing during the first filling of the reservoir, and hypothesized that hydraulic fracturing, could occur in the form of horizontal cracks across the core.

Kjærnsli and Torblaa (1968) based their conclusions on the monitored settlements, pore water pressures, and, more importantly, the earth pressure registered in a pressure cell installed within the core. The measured earth pressure in the core showed that at some stage during the filling of the reservoir the total vertical stress was less than the water pressure in the reservoir, as shown in Fig.. 6.

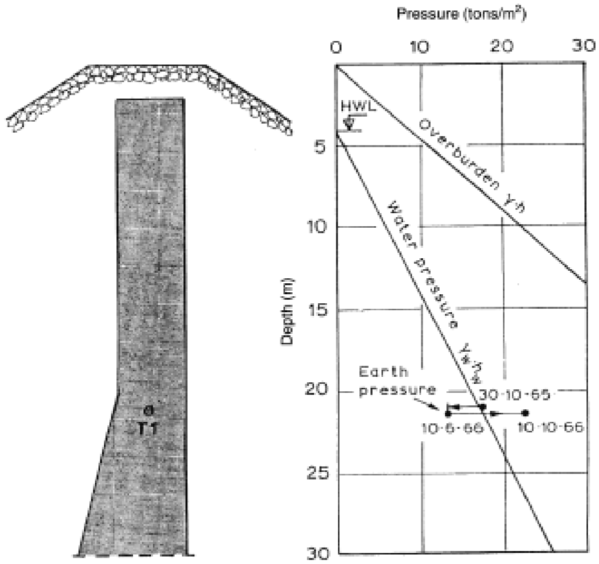

Fig. 6. Measured earth pressures (1 ton/m2) in relation to water pressure and weight of fill (reproduced from Kjærnsli and Torblaa 1968). γ *unit weight of soil;* h *depth;* γ_w *unit weight of water;* h_w *water depth.*

FINITE ELEMENT ANALYSIS AND FRACTURE CRITERIA

Introduction

The finite element method has been widely used in analyzing earth and rock-fill dams in recent years. Most of the analyses were used to study deformation and seepage in dams.

In this case study, consideration was given to the pore water pressures and effective stresses in Hyttejuvet Dam at the time of reservoir filling to investigate the possibility of fracturing. The well-known consolidation theory provides a basic framework for modeling both the deformation and the diffusion of the pore water in porous media, and thereby allows the transient pore water pressures and deformation in the dam to be modeled. Finite element methods based on this theory have been used by some investigators in analyzing dam problems (Cavounidis and Höeg 1977; Ghaboussi and Kim 1982; Naylor et al. 1988; Alonso et al. 1988; Kohgo and Yamashita 1988; Ng and Small 1995). The coupled formulation for stress, deformation and pore pressure of ABAQUS (2006) is used here to model the dam in question that has an unsaturated core before first impounding.

Fracture criteria

By performing several experimental works on hollow cylindrical specimens Komakpanah (1989, 1990) suggested the minimum hydraulic fracture pressure. If hydrostatic pressure on the surface of the core proved to be large enough to overcome the minimum hydraulic pressure, the occurrence of hydraulic fracture will be predictable. Komakpanah suggested the minimum hydraulic fracture pressure in the shear mode of cracking base on Mohr-Coloumb Criterion and in the tension mode base on tensile strength as follow:

$$P_{fs} = P_0(1+\sin\varphi) + C_u(\cos\varphi) \tag{1}$$

$$P_{ft} = 2P_0 + \sigma_t \tag{2}$$

In these equations, P_{fs} is defined as the minimum hydraulic fracture pressure in shear mode, P_{ft} is the minimum hydraulic fracture pressure in the tensile mode, C and φ are shear strength parameters of the soil, P_0 is the minimum principal stress and σ_t is the tensile strength of the soil.
Lo and Kaniare (1990) presented similar criterion while considering tensile strength in shear mode as follows :

$$P_{fs} = P_0(1+\sin\varphi) + C_u(\cos\varphi) + (1+\sin\varphi)\sigma_t \tag{3}$$

These criteria have some flaws based on the assumption that reaching the stress state of the material on Mohr-Coloumb surface can represent the occurrence of cracking in the medium.

In addition to these criteria, a different approach given in Equation 4 is suggested by the authors to predict hydraulic fracturing in Hyttejuvet Dam based on the fact that if the water pressure exceeds the minimum principal stress plus the tensile strength of the soil, the crack initiation and propagation is possible. If this fact does not occur, one may examine an assumption that shear mode can initiate the crack in the medium while tensile mode can propagate this crack (Chang 2004) and finally hydraulic fracturing can occur in the core.

$$\sigma_3 + \sigma_t \le P \tag{4}$$

In this equation P is hydraulic pressure on upstream surface of the core, σ_3 is the minimum principal stress. Based on this approach, regions of the core that shear plastic strain can occurred, are identified and then with a tensile criterion the possibility of hydraulic fracturing is investigated:

Finite element analyses for Hyttejuvet Dam

A coupled finite element analysis using ABAQUS was performed to simulate the deepest section of the dam during construction and the first filling of the reservoir. The idealized layout of the dam section is shown in Fig. 7, which is based on the actual section shown in Fig. 2. As there were no comprehensive data on the stiffness of the individual materials in the gravel and rock-fill shoulders, the shoulders of the dam were assumed to be homogeneous for the sake of simplicity. The finite element mesh used in the analysis, consists of 556 eight-nodded isoperimetric continuum elements and each element has four degrees of freedom for pore pressure (Fig. 8).

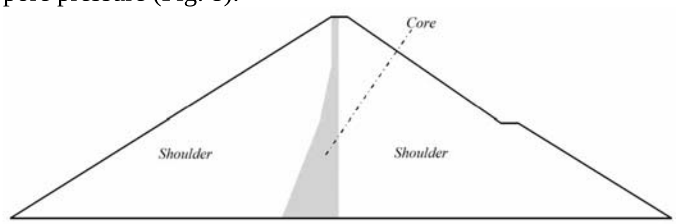

Fig. 7. Idealized section of Hyttejuvet Dam.

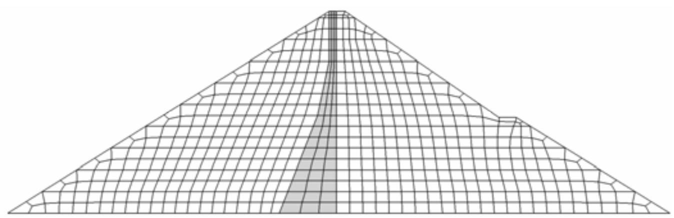

Fig. 8. Finite element mesh for Hyttejuvet Dam.

Four layers of elements were used in simulating the construction described earlier (Fig. 4). The body force of each new layer of elements to be placed was applied gradually in eight increments. Simulation of the filling of the reservoir started 177 days after completion of construction, and the water level was allowed to rise to the maximum value according to the recorded rates (Fig. 5) in 78 time steps. The finite element mesh was restrained at the foundation at all times. It was anticipated that the gravel and rockfill in the shoulders would be highly permeable and no pore-water pressures would build up in the shoulders. Therefore, the shoulders were assumed to have the same saturation during construction process as is in the beginning of construction ($S_{r_0} = 0.6$) and the pore-water pressures along these sections were assumed to be fixed. However, when the reservoir is filled, the boundary conditions for the water heads have to be adjusted accordingly. The total water heads along the immersed part of the upstream slope were set to the total head of the water level in the reservoir. A surface traction was also applied to that surface to simulate the applied weight of water.

In the finite element analysis, the core moraine material, which had more than 30% fines was modeled with Elastic - Modified Drucker- Prager / Cap constitutive laws for modeling the elasto-plastic behavior of the soil. The gravel and rock-fill shoulders, on the other hand, were assumed to behave elastic for simplicity. Table 1 summarizes the material properties used in the analysis. Soil parameters were taken from the literature on Hyttejuvet Dam wherever possible (Kjærnsli and Torblaa 1968; Wood et al. 1976). However, some of the parameters were derived based on other given parameters. Some of the soil parameters had to be assumed. The parameters in Table 1 are as follows:

β, d, α, R, K and ε_v^p are Modified Drucker-Prager/Cap parameters; ν is Poisson's ratio; k_x and k_y are the horizontal and vertical permabilities, respectively; γ_s is the unit weight; S_{r_0} is the degree of saturation of the compacted soil; and E and n_0 are the Young's modulus and initial porosity, respectively.

Soil	*Parameter*
Shoulder	$E = 50000\ (kPa)$
	$\nu = 0.3$
	$n_0 = 0.33$
	$k_x = k_y = 0.0225 (m/s)$
	$\gamma_s = 22\ (kN/m^3)$
Core	$d = 51.24 (kPa)$
	$\beta = 44.9^\circ$
	$R = 1$
	$(\varepsilon_{vol}^p) = 0$
	$\alpha = 0.01$
	$K = 0.8$
	$\lambda = 0.035$
	$\kappa = 0.0015$
	$\nu = 0.3$
	$k_x = k_y = 7e-10\ (m/s)$
	$\gamma_s = 22.66\ (kN/m^3)$
	$S_{r_0} = 0.9$

Table 1. Soil properties of Hyttejuvet Dam.

The effective Young's modulus of the shoulder was previously estimated by Covarrubias (1969) for analyzing the cracking of the dam immediately after construction. The same value was adopted here. The permeability of the shoulders was estimated based on the grading of the gravel in the shoulders using the empirical formula by Hazen (1892) for sand. The Poisson's ratio of all materials was assumed to be 0.3.

ABAQUS does not consider the air pressure (*Pa=0*) and modeling the unsaturated behavior of the soil as it uses the curve of Saturation instead of negative pore pressure (sorption curve). It also assumes $S_r = 1.0$ for the regions which has positive pore pressure. The percentage of the clay, silt, sand and coarse grain of the shoulders were determined from Fig. 3, then SWCC of the material was determined based on these percentages using unique SWCCs for pure clay, silt, sand and coarse material (Fig. 9). Finally by dividing the amount of volumetric water content (θ) that of the saturated volumetric water content (θ_s), the sorption curve of the material was obtained (Fig. 10).

Fig. 10 shows the sorption curve of the core material which is based on the experimental results for variation of degree of saturation with suction by Kohgo (1992) for a till. Having Fig

10 in mind, keeping the saturation of shoulders as it is during the construction period, a constant value for the pore pressure of the shoulders of the dam was considered.

The permeability of the unsaturated parts of the dam was estimated based on the formula by Nguyen and Dorso (1983):

$$k_{unsaturate} = S_r^{\,3}.k_{saturate}. \qquad (9)$$

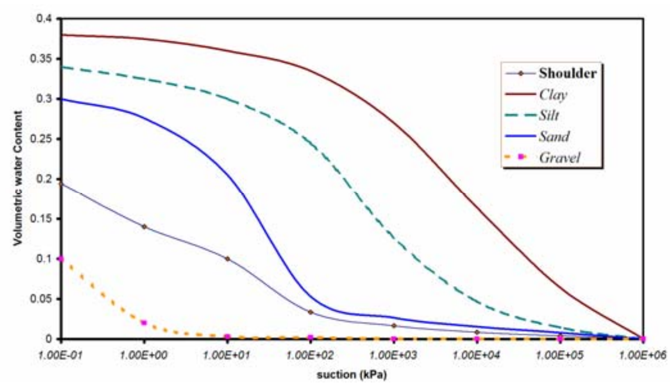

Fig. 9. SWCC for Pure clay, silt, sand and gravel and shoulder.

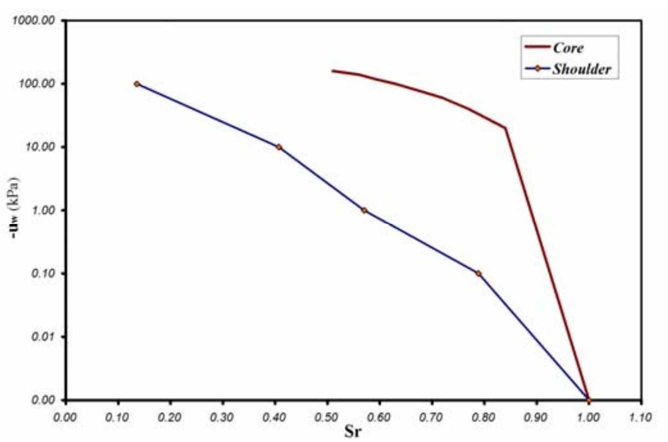

Fig. 10. Sorption curve of the material of core and shoulder.

Numerical results and comparison with measured data

The deformed meshes for the dam during construction and water impounding are given in Figs. 11 and 12. The core settled considerably more than the supporting shoulders. The differential settlement also increased as construction proceeded. The differential settlement would create an arching effect in the core and reduce the vertical stresses in the core during construction. Arching effect and reduction of vertical stress in the core can be seen in Fig. 13 which shows the vertical effective stress in the Dam.

Contours of the computed pore-water pressures during construction and first impounding are given in Figs. 14 and 15. During construction, pore-water pressure builds up in relatively impermeable core. When the water level in the reservoir rises, the pore-water pressures in the core is affected by the external hydraulic pressure and increases, as shown in Fig. 14.

Before the possibility of fracturing is examined, the measured and calculated pore water pressures are compared, as they play an important role in assessing the stresses in the dam. The pore-water pressures in the core of the dam had been monitored on site during construction and first impounding. The location of pizometers to measure porewater pressure in the core where fracturing would be possible are reported by previous investigators (Kjærnsli and Torblaa 1968, NG and Small 1999) and are shown in Fig. 16. Comparison of the measured pore-water pressures and those computed in this study is presented in Figs.17–19. Although the predicted values are generally lower than the measured values, they are in good agreement with the measured values.

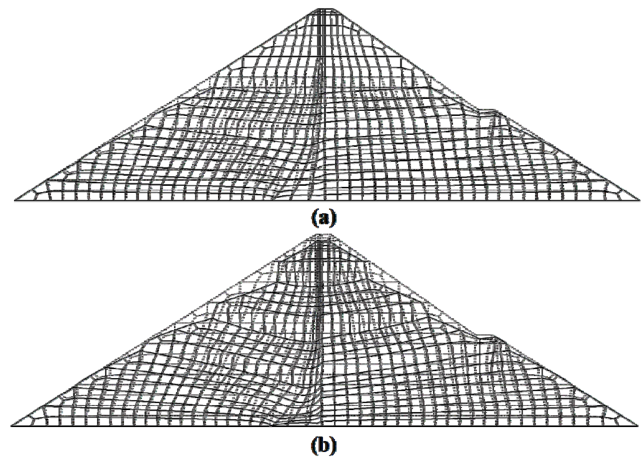

Fig. 11. Deformation mesh during construction (a) 406 days (b) 696.5 days. (scale=30)

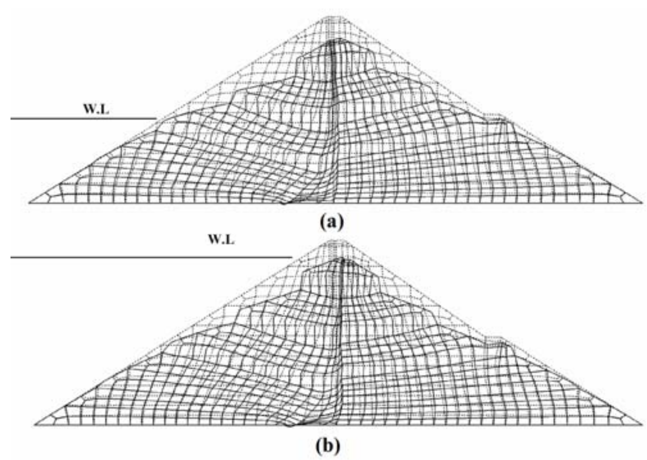

Fig. 12. Deformed mesh during construction. (a) 726 days (b) 832.8 days. (scale=30)

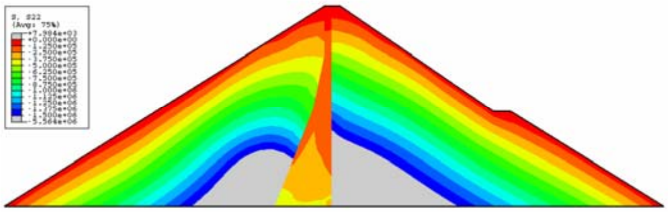

Fig. 13. Vertical stress in completion of construction (t=696.5 day).

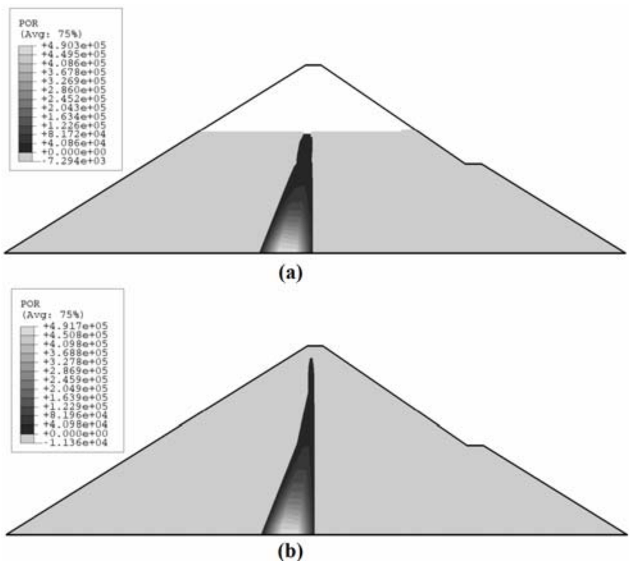

Fig. 14. Pore water pressure during construction (a) 406 days (b) 696.5 (days).

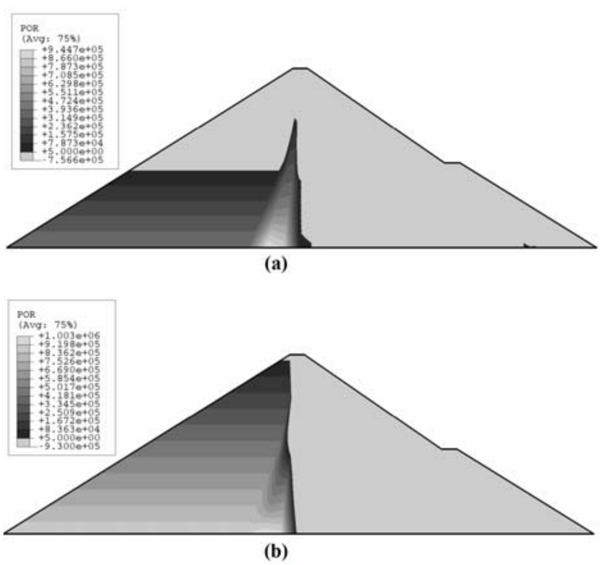

Fig. 15. Pore water pressure during impounding (a) 726 days (b) 832.8 days.

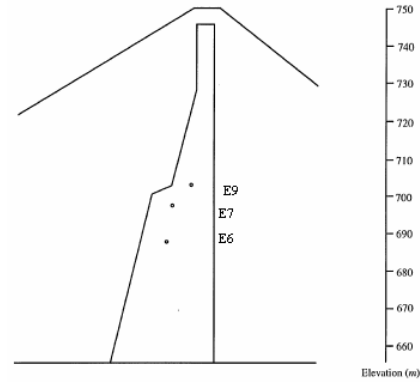

Fig. 16. Pore water pressure monitoring locations selected for comparison with the analysis (after Kjærnsli and Torblaa 1968).

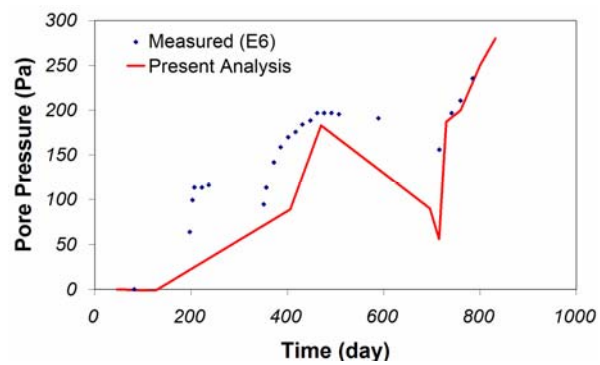

Fig. 17. Computed and measured pore water pressures in the core at location E6.

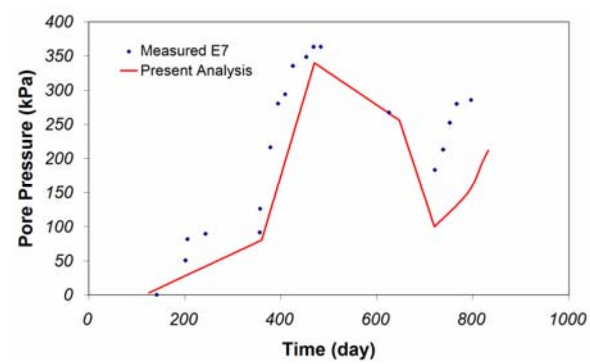

Fig. 18. Computed and measured pore water pressures in the core at location E7.

PREDICTING HYDRAULIC FRACTURING

Criteria

For investigation of the hydraulic fracturing in the core, different criteria given in Equations 1, 2 and 3 are checked in all elements of the core. With assumption that if crack initiates in the upstream surface of the core the hydraulic pressure on the crack strip is the same as the hydrostatic pressure induced on the core, quality of *P* is assumed to be the same in all elements in a specific elevation. Contours of *P* are shown in Fig. 20 for 40m high impounding and for the complete impounding processes.

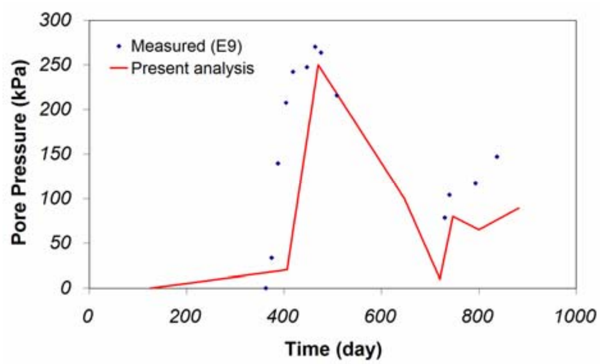

Fig. 19. Computed and measured pore water pressures in the core at location E9.

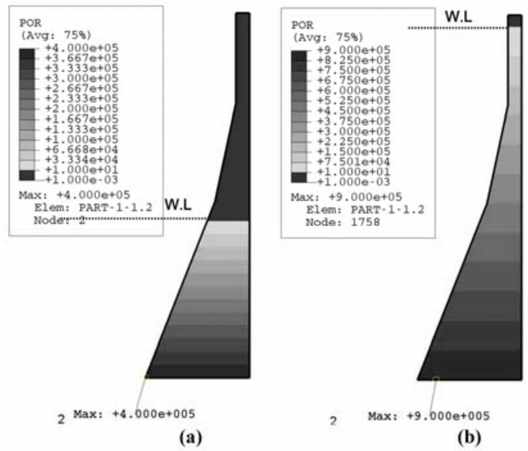

Fig. 20. Contour of P at (a) 40m impounding (b) 90 m impounding.

Criterion which is given in Eqs. 1 - 3 are modified to get safety factor as follow:

$$\frac{P_0(1+\sin\varphi)+C_u(\cos\varphi)}{P}<1 \tag{5}$$

$$\frac{2P_0+\sigma_t}{P}<1 \tag{6}$$

$$\frac{P_0(1+\sin\varphi)+C_u(\cos\varphi)+(1+\sin\varphi)\sigma_t}{P}<1 \tag{7}$$

Contours of Equation 7 for the last two stages of impounding is presented in Fig. 21. It can be seen in this Figure that minimum factor of safety takes place at the location of sharp change in the core thickness at elevation of 700 m (factor of safety at this point is *0.075*). The contours for Equation 5 are also shown in Fig. 22 in which the minimum factor of safety is 0.042 and takes place at the location of sharp change in the core thickness. Tension criterion (Equation 6) also indicates that the region of sharp change of the core thickness has the highest possibility for initiation of the crack (Fig. 23), with safety factors of 0.056 and 0.042 respectively for 80 and 90 meters impoundings.

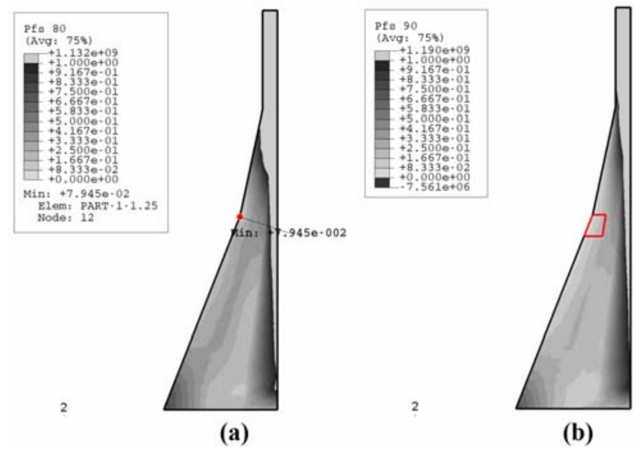

Fig. 21. Contour of Eq. 7 in (a) 80m impounding ;(b) 90m impounding.

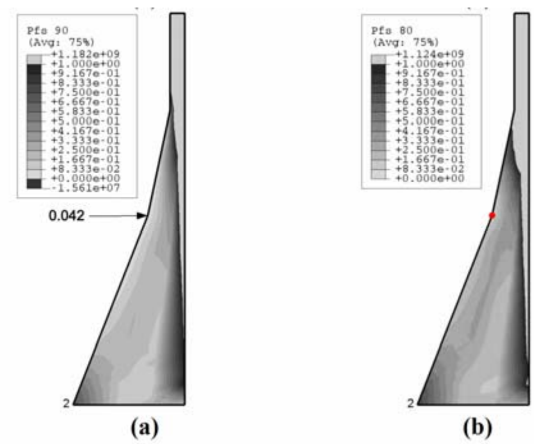

Fig. 22. Contour of Eq. 5 in (a) 80m impounding ;(b) 90m impounding.

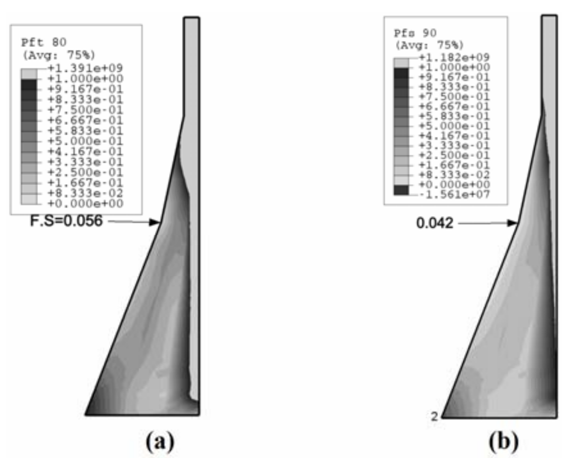

Fig .23. Contour of Eq.6 in (a) 80m AND (b) 90m impounding.

Considering Figures 21 – 23 we can conclude that these criteria (Equations 1-3) have capability to predict initiation of crack in the place that Hyttejuvet Dam began to fail as reported by Kjærnsli and Torblaa 1968 and Ng and Small 1999. However, the factor of safety against cracking criteria is not the same along a horizontal section of the core as shown in Fig. 24 in more details for Equation 6. The factor of safety as seen in this figure increases from 0.04 at upstream face to 2.0 at the down stream face of the core at the critical section. As it is clear crack initiates from upstream face of the core and propagates towards downstream to the midway of the core thicken and stops. Therefore a complete upstream-downstream crack would not occur with any of the mentioned criteria and thus a new approach should be considered to examine theoretically the complete hydraulic fracturing.

<u>New approach</u>

Fig. 25 shows the plastic shear strain (on Drucker-Prager/Cap surface) in the last two stages of impounding. It can be seen in this Figure that with the rising of water level in the reservoir, the plastic region becomes wider in upstream face due to the increase in hydraulic pressure on that face. Fig.25 also shows that the maximum plastic strain takes place in the region of sharp change in the core thickness at elevation 700 m and the amount of strains are *7.1e-3* and *1.33e-2* respectively for 80m and 90m impounding. The region pointed out in Fig.25 is the region of high possibility for crack initiation.Counters of Equation 8 which is restated form of Equation 4 in a safety factor mode, is shown in Fig. 26.

$$\frac{\sigma_3 + \sigma_t}{P} \leq 1 \qquad (8)$$

The tension criterion (Fig. 26) shows that this crack can propagate to the rest of the core thickness and hydraulic fracturing in the core is predictable. Factors of safety in the tip of the crack and downstream face of the core in tensile mode are *0.2* and *0.81* respectively at 90m impounding (Fig 27). This approach, therefore, can show the hydraulic fracturing at elevation 700 m of the Hyttejuvet dam which is about 50m above the dam foundation.

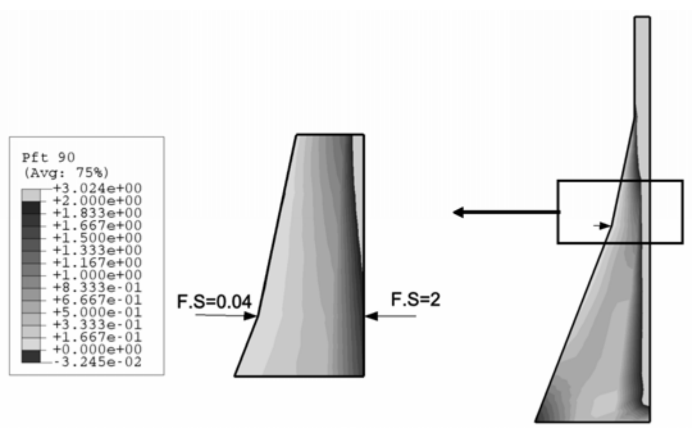

Fig. 24. Contour of Eq. 6 at 90m impounding in dangerous region.

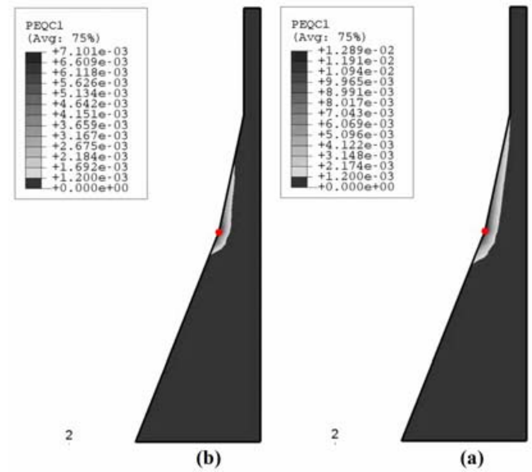

Fig. 25. Shear plastic strain in core at (a) 80m impounding; (b) 90m impounding.

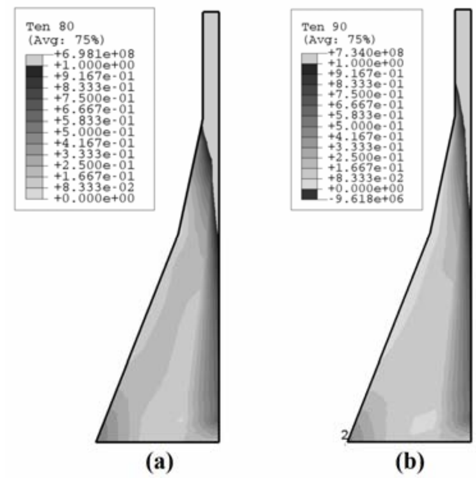

Fig. 26. Contour of Eq. 8 in (a) 80m impounding; (b) 90m impounding.

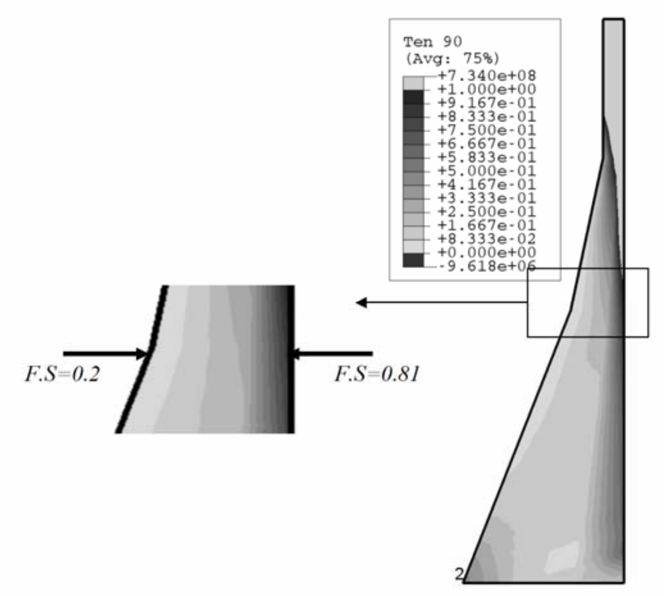

Fig. 27. Contour of Eq. 8 in 90m impounding in dangerous region

CONCLUSION

A case study of Hyttejuvet Dam using finite element methods has been presented. A finite element analysis was carried out to simulate the behavior of the dam during construction and first filling of the reservoir. The results of the analysis indicated that significant load transfer probably occurred during construction due to differential settlement between the gravel and rock-fill shoulders and the core of the dam. The odd shape of the core could accelerate this phenomenon. The results of the numerical analysis are supported by the good agreement between the computed and measured pore water pressures which play an important role in assessing the effective stress conditions in the dam.

Although the results of the criteria presented by previous investigators show that the initiation of crack could have occurred at various locations in the core during first impounding, they could not predict a complete hydraulic fracturing in the core of the Hyttejuvet dam. The results of the analysis show that with the present criteria the initiation of hydraulic fracturing could be predictable; however, none can predict the complete upstream-down stream hydraulic fracturing. However, using the new approach proposed by the authors of this paper a complete hydraulic fracturing is predictable in the core of Hyttejuvet Dam. The major and complete fracture predicted by this analysis which can lead to severe leakage was estimated to be at about 50 m above the dam foundation, at elevation 700.

The findings support the observations and previous conclusions drawn by Kjærnsli and Torblaa (1968) and Ng and Small (1999).

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