Numerical analysis of unusual behavior of Zermanice dam

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Keywords: marly shale, monitoring, calibration, 3D FEM, viscoplastic flow

ABSTRACT: A three-dimensional FEM model simulating the unusual performance of Zermanice Concrete Dam and its foundation according to the long-term monitoring results was elaborated. To get proper input data, a series of two-dimensional local and regional models allowing for step-by-step calibration of mechanical, hydraulic, thermal and rheological parameters of bedrock according to the measurement results was applied. The numerical models fitting the measurement results discovered that viscoplastic flow of weak, disturbed marly shale in dam foundation is most likely responsible for the observed behaviour of the dam.

1 Introduction
The 36 m high Zermanice Concrete Dam, built in difficult geological conditions in the middle of the last century, reveals long-term uneven heave, tilting and horizontal displacements with constant or increasing velocity. In order to assess the safety of the dam and predict its further behaviour, a stability study consisting of extensive archival survey (Svancara, 2002, 2003) and modelling work was carried out (Dolezalova et al. 2003, 2004). A large 3D FEM model simulating the observed performance of the dam and its foundation according to the long-term monitoring results was elaborated. To solve the data limited problem, a step-by-step modelling procedure – a series of 2D local and regional FEM models – allowing for calibration of mechanical, hydraulic, thermal and rheological parameters of rock according to the measurement results was applied. The numerical models fitting the measurement results discovered that viscoplastic flow of weak, disturbed shale in the dam foundation is most likely responsible for the observed behaviour of the dam. Two flows, one from geological time with a recent slip surface in the right abutment and another induced by the reservoir filling were found and simulated by the models.

The dam and the dam site, field measurement results, the modelling procedure and selected results of 2D and 3D models are described in the following.

2 The dam and the dam site
2.1 Description of the dam
The Zermanice Concrete Dam (length: 311 m, storage capacity: 60.10^6 m^3) was built in Northwest Moravia for flood control and water supplement of Ostrava industrial region in 1952-1958. The dam is located in flysch rock of the Carpathian region, which is a sedimentary rock formation consisting of alternating strata of hard sandstone and weak marly shale. The dam site is formed by Cretaceous marly shale to a depth 650 m, which was originally covered by a competent, 28 m thick teschenite sill of volcanic origin. The marly shale is composed from quartz, kaolinite, illite and calcite, swelling minerals are not present. It is a competent rock in undisturbed state but disintegrating on the air and then softening and slushy in water.
Erosion of the valley in the geological time brought about bulging and sliding of the underlying marly shale on the right bank of the valley, which resulted in disturbing the shale in the bottom of the valley and breaking the overlying teschenite sill into separate blocks (Zaruba, 1956). The geological conditions in the longitudinal section of the dam, the initial values of material properties (\(E\) - deformation modulus, \(\nu\) - Poisson ratio, \(c\) - cohesion, \(\varphi\) - friction angle, \(\gamma\) – unit weight) and two characteristic cross sections of the dam 1-1' and 2-2' are shown in Figure 1.

Figure 1. Geological conditions in the longitudinal section of Zermanice Dam; Cross sections of the dam on marly shale (1-1') and competent teschenite (2-2').

The intricate geological conditions required three specific dam sections: a) conventional gravity dam section (2-2') on competent teschenite on the left bank b) extended gravity dam section founded on reinforced concrete slab and supported by embankment (1-1') on disturbed shale in the centre of the valley and c) a complex structure (concrete diaphragm wall+ retaining wall + embankment) founded on disintegrated, permeable teschenite on the right bank of the valley.

Improvement of the bedrock by extensive grouting was carried out. Along with a three-line type grout curtain, the bedrock was fortified (depth from 10 m to 30 m) along the whole base of the dam. However, inadmissible grouting pressure, causing lift of some dam sections on disturbed shale, resulted in highly uneven compressibility of the bedrock in the centre of the valley.

2.2 Measurements

Though the dam is in operation and so far no leakage or other warning event has occurred, the long-term monitoring results (1958-2004) carried out by optical levelling, trigonometric levelling and pendulum show unusual performance questioning the safety. Instead of settlement, the central part of the dam on disturbed shale manifests a heave (20 mm), tilting (44 mm) and large horizontal movements in downstream direction (60 mm). On the other hand, the dam sections founded on teschenite show no tilting and limited horizontal movements: 20 mm on the left side and 8 mm on the right side of the valley. All movements reveal constant or increasing velocity.

3 Modelling procedure

3.1 Modelling concept

To solve the above complex and data limited problem, a modelling concept consisting of eight linked up 2D models and a synthesizing 3D model of the dam and its foundation was suggested:

- 1a: local 2D model of the cross section dam on marly shale simulating in situ stress state, construction of the dam, first filling of the reservoir and operation from 1959 to 2000; calibration of mechanical and hydraulic parameters via a trial and error fitting procedure assuming only primary consolidation of the shale;
- 1b: 2D stability analysis using the 1a model and strength reduction method;
3.2 Solution procedure

For solving the above mechanical-hydraulic and rheological problems, a modified version of CRISP FEM Code (Britto & Gun, 1987) called CRISPATH with a highly effective iterative solver (Hladík et al., 1997), efficient iterative solution strategy and different constitutive models - here path dependent, elastic-plastic constitutive model (Doležalová, 1991, 1994) and multiface viscoplastic model - was applied. The path dependent model distinguishes zones with deformational response to loading and unloading in shear and normal stress, which proved to be crucial for matching the measurement results. The trial and error fitting procedure consisted of the following steps:

- Set up initial values of the deformation and strength parameters according to the archive files;
- Select the decisive loading and monitoring stages with characteristic measurement results;
- Simulate the whole process of the construction and operation of the dam;
- Compare the computational and measurement results for the selected monitoring stages;
- Keep the strength parameters and tune the deformation parameters by repeating the steps 3 and 4 as long as the set of parameters yields acceptable agreement for all monitoring stages.

5 alternatives with a total number of trial solutions 17 (5/17) were necessary to solve the 1a task, but only 3/6 to perform the 2a task. A simplified benchmark problem with 7 alternatives helped to determine the viscoplastic parameters for the 3a task and as many as 3/11 and 6/22 alternatives and trial solutions were necessary to deal with the 3b and 3c tasks. No special parameter identification procedure like (Swoboda et al., 1995) was applied, since these approaches do not follow the complex loading sequences and the corresponding stress paths, which are necessary to match the measurement results. Selected results of the 1b, 3a and 4 models are described below.

4 Stability assessment of the central section of the dam on weak marly shale

To discover the possible failure mechanisms and get the first information on the safety of the dam, the 1b task following the calibrated 1a model was solved. Step–by-step reduction of shear and tensile strength parameters allowed for simulating the process of strain localization in the dam foundation, which resulted in a continuous slip surface and failure of the dam. The corresponding reduction factor \( F \) \((F>1)\) determined the degree of safety of the dam. The reduced strength parameters \( c_F, \phi_F \), and \( \sigma_{\text{f}} \) are defined as \( c_F = c / F \), \( \tan \phi_F = \tan \phi / F \) and \( \sigma_{\text{f}} = \sigma_{\text{f}} / F \).

Using this method, the influence of different factors like altitude of the reservoir level, magnitude of pore pressures depending on dewatering of the dam foundation and progressive failure of the shale caused by strength reduction after exceeding the peak strength was investigated.

In Figure 2 the slip failure of the dam corresponding to an extreme loading stage assuming maximum reservoir level, out of work dewatering and progressive failure of the shale is displayed. A minimum safety factor of the dam \( F = 2.5 \) was found for this case, while a safety factor \( F = 3 \) was calculated for the normal operational stage.
Concerning the failure mechanism, no overturning due to tilting or slip along the dam-foundation contact, but a deeply located slip surface was found. The depth and shape of this slip surface are influenced by the extent of both the fortification of bedrock and the embankment on the downstream side. The solution proved that these measures increase the safety of the dam. The geometry of the slip surface is rather insensitive to mesh refinement.

5 Regional model of formation of the valley in geological time

In order to calculate the in situ stresses of the valley and estimate the rheological parameters of the marly shale, the process of formation of the valley in geological time (from \(-300,000\) to \(-10,000\) years) was simulated. The 3a model was based on scientific recovery of the geological events accompanying the formation of the valley published by Academic Q. Zaruba in 1956 and completed by Svancara & Bilek, 2003; Novosad, 2003. According to this concept, erosion of the valley to a depth 25 m in the last postglacial period brought about differential loading of the bottom and banks of the valley resulting in viscoplastic flow, sliding and bulging of the soft marly shale on the right bank of the valley. Avulsion of the disturbed marly shale at the bottom of the valley caused breaking of the overlying teschenite sill into separate blocks and their subsidence up to 13 m. This recent landslide spanning about 15,000 years was simulated using Perzyna's theory of viscoplasticity.

5.1 Formulation of multiface viscoplastic flow

Time-dependent plastic deformations of materials can be simulated by means of the theory of viscoplastic flow. According to this theory, the total strain at a point is the sum of its elastic and plastic components:

\[
\varepsilon = \varepsilon^e + \varepsilon^p
\]  

(1)

and the plastic strain rate in time \(\dot{\varepsilon}^p\) is defined by the following viscoplastic flow rule:

\[
\dot{\varepsilon}^p = \Gamma \Phi(F) \frac{\partial \Phi}{\partial \sigma}
\]  

(2)

Here, \(\Gamma\) is a fluidity parameter \(\Gamma = 1/\eta\) (where \(\eta\) is a viscosity coefficient), which may in general depend on time \(t\) or on the stress tensor \(\sigma\); further, \(F(\sigma)\) is the threshold of viscoplasticity, \(Q(\sigma)\) is the viscoplastic potential function and \(\Phi(F)\) is a function of \(F\) such that \(\Phi(F) = 0\) for \(F \leq 0\) and \(\Phi(F)\) is monotone increasing for \(F > 0\).

According to the theory of viscoplastic flow, the instantaneous deformation at \(t = 0\) is elastic and at \(t = \infty\) plastic, which is governed by the flow theory of plasticity. Viscoplastic deformation arises
for each stress $\sigma$, which is above the viscoplastic threshold (function) $F$. The finite element discretization of initial and boundary value problems with the elasto-viscoplastic constitutive model can be found e.g. in Owen & Hinton (1980), Reed (1992), Desai (2001).

The theory of multilaminate viscoplastic flow described in Zienkiewicz & Pande (1977) assumes that for a given material we can introduce more than one threshold of viscoplasticity $F_y$ and one viscoplastic potential function $Q_y$, say $k_y$, such functions $F_{y}, Q_{y}, y = 1, ... , k$, the effects of which are summed as follows:

$$\dot{\varepsilon}^P = \sum_{y=1}^{k} \dot{\varepsilon}^P_y = \sum_{y=1}^{k} \Gamma_y \Phi_y (F_y) \frac{\partial Q_y}{\partial \sigma}$$

Here for each $\dot{\varepsilon}^P_y$, $y = 1, ... , k$, the elasto-viscoplastic constitutive law with the threshold of viscoplasticity $F_y$, and the plastic potential function $Q_y$ and with the other parameters described above for the 'one-face' viscoplasticity holds true.

The formulation of viscoplastic flow with two surfaces was employed to simulate the observed behaviour of shale. It is outlined in Figure 3 where the Mohr-Coulomb yield condition with a tension cut-off is used. The first function $F_1$ represents a macro-structural threshold of plasticity, the second function $F_2$ the micro-structural threshold of plasticity. Function $F_1$ allows for the 'instantaneous' plastic deformations and can be determined from common laboratory tests. An artificially large value of the fluidity parameter $\Gamma_1$ ensures the instantaneous reduction of stress on $F_1$. The second function $F_2$ allows for the observed time-dependent plastic deformations. Non-associated flow rule with $\psi = 0$ applied to function $F_1$, associated flow rule to $F_2$ and a linear relation to viscoplastic flow rate $\Phi = F_2 / F_0$.

5.2 Solution procedure and results

The model for simulating the landslide due to viscoplastic flow is described in more detail in (Dolezalova, Hladik, 2005), here only the basic idea and the main results are given. Since at time $t = \infty$ the results of elasto-plastic solution and viscoplastic solution are the same, the problem could be solved in two phases. The first phase was an elasto-plastic stability analysis using the strength reduction method. The strength parameters of the shale were step-by-step reduced in such a way that erosion of the valley (replaced by excavation) resulted in sliding and subsidence of teschenite blocks according to the back prediction. In this way, the viscoplastic threshold of shale could be estimated as a fraction of strength. Then considering the duration of the event and the maximum shear strain increment induced by stress relaxation in selected points of the slip surface, the fluidity parameter could be estimated. The second phase was a viscoplastic solution of the same problem using the estimated values of the rheological parameters. A very low viscoplastic threshold amounting to only one tenth of strength and a fluidity parameter close to the parameter of rock salt were found for the disturbed marly shale. This clears up why the disturbed shale was ready to flow at any loading change induced by excavation or reservoir filling.

The calculated viscoplastic flow and bulging of shale across the valley and sliding and subsidence of the overlying teschenite blocks (back predicted subsidence 10 - 13 m, calculated 10 m) are shown in Figure 4. Here the deformed mesh and the maximum shear strain contours are plotted.
The corresponding in situ stress state of the valley demonstrates considerable redistribution of the horizontal stresses. The components along the valley are 1.4 times higher than the overburden weight, while the components across the valley show large tensile zones in teschenite on the right bank. This would explain, why the teschenite is disintegrated just in this part of the valley.

Figure 4. Viscoplastic flow and bulging of marly shale and subsidence of teschenite blocks due to erosion of the valley in geological time - deformed mesh and maximum shear strain contours.

6 3D regional model of the dam and its foundation

The 3D regional model of the dam and its foundation aimed at synthesising all 2D findings and simulating the 3D phenomena like co-operation of the dam and the embankment with the right abutment disturbed by sliding and super-position of viscoplastic flow in two directions: across the valley induced by geological events and along the valley due to reservoir filling.

To capture the complex geological conditions of the site and complicated structure of the dam, a large 3D model (DOF = 389 656) consisting of 66 cross sections was elaborated and the whole process of construction and operation including the stability assessment and prediction of the dam performance up to 2060 was simulated.

A 3D geological model reflecting the complex geological conditions of the site worked out by Svancara & Bilek, 2003 and Novosad, 2003 and the output of 2D models made up the input for the 3D model. Using the results of 2D models,
the in situ stress state according to the 3a model was generated and the deformational parameters of the shale were significantly refined. The in situ response of the shale fitting the monitoring results showed to be much more stiffer (deformation modulus $E = 100 - 150$ MPa) than the initial parameters based on the laboratory tests. Using the calibrated parameters, the uneven horizontal displacements of the dam due to the first filling of the reservoir were successfully simulated by the 3D model as shown in Figure 5. Comparison of calculated displacements / measured displacements yielded acceptable results: 18.5 mm / 15.5 mm for Section 1-1’ and 8.6 mm / 10.5 mm for Section 2-2’.

Along with the good results approving the above modelling concept, the simulation of the 3D viscoplastic flow of the marly shale brought about a number of new problems as well. The properties of the shale and disintegrated teschenite forming the right slope of the valley, the proper modelling of the dilatation joints, diaphragm wall and grout curtain belonged to the issues, which complicated the simulation of viscoplastic deformation of the dam. When the same parameters were introduced for both the shale forming the slope and the shale forming the bottom of the valley, unrealistic post-construction displacements of the dam were calculated (~10 mm instead of 44 mm for the period 1959-2000). The reason was that owing to the incorrect parameters, the viscoplastic flow across the valley suppressed the viscoplastic flow along the valley.

Extensive 3D parametric studies and fitting procedures (68 alternatives) involving also a simplified 3D model of the dam and its foundation (DOF = 265 959) were necessary to solve the problem. It turned out that a) the strength and viscoplastic threshold of the marly shale in the slope might be higher than those at the bottom of the valley (guess: $c = 50$ kPa, $\phi = 24^\circ$, threshold: one third of strength), b) the same is valid for the strength of the disintegrated teschenite in the slope ($c = 250$ kPa, $\phi = 33^\circ$), c) the dilatation joints of the dam display deterioration and strain softening with time and their stiffness is close to the stiffness of clay, d) the grout curtain should be modelled as a set of jointed panels, which is more flexible in upstream–downstream direction.

Using the 3D model with calibrated parameters, super-position of viscoplastic flow of shale across
the valley and along the valley produced the right magnitude and direction of displacement vectors (Figure 6), which corresponded to the measurement results. Comparison for the dam sections with highly different horizontal displacements in Figure 7 shows good agreement of the measured and calculated values for the first filling of the reservoir and the whole operational period of the dam from 1958 to 2000. The same is valid for the maximum shear displacement (calculated: 44 mm, measured: 40 mm) and acceptable results were obtained for the maximum tilting of the dam as well (calculated: 26.6 mm, measured: 31.7 mm). On the other hand, the calculated maximum heave is less (13.4 mm) than the measured one (19 mm).

7 Conclusions

Due to the limited space, the stability assessment and prediction made by the calibrated 3D model are not included in the paper and here only the main conclusions are given.

- The model discovered that the observed heave, tilting and uneven displacements of Zermanice Dam is caused by the viscoplastic flow of weak marly shale along the valley induced by filling of the reservoir and the viscoplastic flow of shale across the valley originated by sliding in geological time and recovered by the construction of the dam.
- Concerning the expected performance of the dam, slightly decreasing velocity of viscoplastic deformations and slightly increasing factor of safety of the dam was predicted by the 3D model fitting the long-term monitoring results. This is due to stress relaxation accompanying the viscoplastic flow of marly shale and the geometry and boundary conditions of the problem.
- Except the deteriorated dilatation joints, the safety of the dam does not require immediate measures. However, extension of the embankment in the future would slow down the viscoplastic flow of marly shale.

8 Acknowledgements

The authors are grateful to the Grant Agency of the Czech Republic for support of this work through Project No 103/04/0672.

9 References