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Performance review and safety evaluation of the Botonega Reservoire and Dam

Analyse de comportement et évaluation de sécurité de la retenue et du barrage à Botonega

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ABSTRACT
The Botonega Dam built in the Istria peninsula in the Adriatic Sea is intended to control flood water waves from watercourses upstream and to regulate downstream watercourses of the Botonega and Mirna Rivers. Due to increasing demand for water supply, especially during summer, intention of the local authorities is to expand its function as a reservoir and supply water to the Butoniga Buzet Istrian Water–Supply System. In order to reconstruct previous consolidation process and to predict future ones, a numerical model was worked out. In the paper, analyses of settlements and stability analyses are presented. Settlement values obtained by computing and geodetic monitoring correspond well. Stability analyses proved that the Botonega dam will be able to satisfy the criteria for its future use as a multi purpose construction.

RÉSUMÉ
Le barrage de Botonega, situé à la péninsule d'Istrie sur la côte Adriatique a été bâti afin de contrôler les eaux de crue provenant des cours d'eau situés à l'amont, et pour régler les cours d'eau à l'aval des rivières de Botonega et Mirna. Compte tenu de l'augmentation croissante des besoins en eau, notamment pendant les mois d'été, les autorités locales ont décidé d'étendre l'utilisation de ce barrage et de l'utiliser aussi comme retenue et source d'eau pour le système Butoniga - Buzet, faisant partie du Système istrien d'alimentation en eau. Le modèle numérique a été élaboré afin de reconstruire le processus de consolidation préalable et afin de prévoir ce processus dans l'avenir. Les analyses de tassement et de stabilité sont présentées. Une bonne correspondance entre les valeurs de tassement obtenues par calculs et ceux mesurées au cours des relevés géodésiques a été constatée. Les analyses de stabilité démontrent que le barrage de Botonega est capable de satisfaire aux critères de son utilisation future, c'est-à-dire de son utilisation comme un ouvrage à usages multiples.

1 INTRODUCTION

The Botonega dam is a multipurpose structure intended to: control flood water from watercourses upstream of the dam; regulate downstream watercourses of the Botonega and Mirna Rivers; and supply water to the *Butoniga Buzet, Istrian Water-Supply System.* The drainage area of the reservoir is 73 hm^2, and the reservoir volume at normal reservoir level is 19.7 million cubic meters. The dam is constructed of filled and compacted earth material so as to have a central clay core; the upstream part is constructed of limestone fill, and the downstream part of marl fill. The original ground line is at the elevation of 25.00 m above sea level and the crest of the dam is at the elevation of 44.70 m a.s.l. in order that it be 2 m higher than maximum water level in the reservoir, viz. 42.70 m a.s.l. The dam has a volume of $0.51 \cdot 10^6$ m^3, a length and width at the crest of 576 m and 6.0 m respectively. It has an upstream slope of 1:2, a downstream slope of 1:2.5, and a toe of 95.0 m width, on average. In cross-section, the dam is designed as asymmetric with inclined clay core and appropriate filter and drainage layers. The upstream face is covered with stone lining (Fig. 1).

The dam foundation consists of low-plasticity and medium-plasticity clays having 20-m average thickness; clayey gravel up to maximum depths of 31.5 m; and base rock (marl) whose abutments are lined with clayey liner (the right abutment) and limestone breccia (the left abutment).

2 PREVIOUS OBSERVATIONS AND MEASUREMENTS OF THE DAM CONSOLIDATION

As the dam abuts on the compressible base ground of low permeability, due to consolidation its stability increases in time. In order to ensure the dam stability at an initial stage, a rulebook on water regime lays down a criterion according to which the highest permitted water level is limited to 38.0 m a.s.l. until the results of measurements confirm that the degree of consolidation in the foundation has reached 90%; after that the limit of the permitted water level is increased to 39.0 m a.s.l., which is design water level in the reservoir.

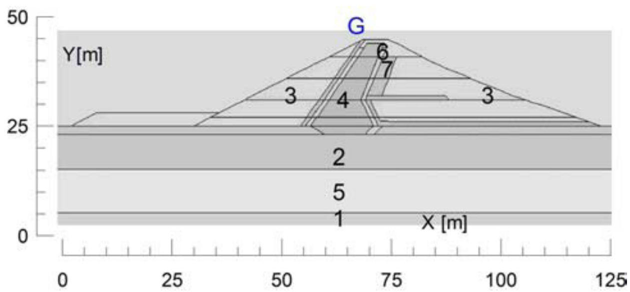

Figure 1. The characteristic cross-section of the Botonega dam used in numerical model (description of soil indicated by numbers are given in Tab. 1 and 2).

Periodical geodetic measurements are made at 41 measuring points spread over 12 profiles. The said measurements were started only after five years following the beginning of construction or two years after its completion. For this reason neither survey of the unloaded ground (foundation) nor the data on behaviour of both the dam and the foundation over the construction stages, when most settlement occurred, are available.

In addition to displacement measuring sensors, sensors measuring pore pressures were installed in two observation profiles located in both the core and the foundation. These sensors failed during the reservoir operation and consequently distribution of pore pressures cannot be determined.

The reference measurement was made at the measuring points located at the dam on 15 July 1986 (initial measurement).

Measurements are usually carried out twice a year. The analysis of the displacement of the measuring points shows that the manner in which the behaviour of the dam has been monitored to date - by the number of measuring points and time intervals - is satisfactory to make numerical models and evaluate the condition of the dam. The results of geodetic measurement (Vugrinec and Matešić, 2002) indicate that the dam displacement is decreasing with time.

Horizontal displacements of the top of inclinometer's pipes, measured by inclinometer, correspond to horizontal displacements obtained by geodetic measurement. The dam displacements determined by inclinometers have not been analysed in detail as yet.

Below the dam toe, in the foundation two double piezometers of the Casagrande type were placed, but the water in piezometers that were located at greater depths overflows and, therefore, these piezometers should be extended in order that the water column in piezometers could be determined. This is one of the reasons why data on piezometer height obtained by piezometer measurements are not included in the numerical model.

3 THE CONSOLIDATION CALCULATION

On the basis of geotechnical researches and geotechnical profiles made it can be concluded that the foundation is layered horizontally and is, therefore, modelled in the calculation with three horizontal layers. The first clay layer extends from the surface to the depth of 10 m and the second one extends to the depth of 20 m. The second, bottom layer is softer and of lower permeability. Below these two layers, a layer of clayey gravel extends to the depth of 25 m.

According to design documents and the 'as built' report (Vugrinec and Matešić, 2002a; Vugrinec and Matešić, 2002b) contours of the dam and zones within the dam embankment, having various properties, were determined. Although the upstream and downstream support zones were constructed of different materials (limestone and marl respectively), in the analyses of water flows and dam settlements they were assigned the same properties. In the cross-section around a clay core additional two filter zones were modelled.

In the analyses only one cross-section was considered which represents about 85% of the dam (dam abutments were not analysed). The finite element model of the dam is presented in Fig. 1.

A numerical analysis of the foundation and dam consolidation was made using a commercial computer program ***Plaxis 2D–V8*** (Bringgreve, 2002), which enables simultaneous analyses of water flow, dam settlements (consolidation) and stability analysis. Stages of loading and consolidation of the numerical model were adjusted to the stages of the dam construction and to water levels in the reservoir at the time when geodetic measurement of measuring points was made.

Mechanical and hydraulic parameters of the foundation and the materials placed in the dam embankment were determined on the basis of both the analysis of the available data obtained from field and laboratory investigations and on the data found in research papers on similar materials.

In order to analyse the consolidation of the Botonega dam, a hardening-soil constitutive model was used for simulating the behaviours of clayey elements including soft soil and stiff layers. Although Plaxis also offers a more sophisticated strain softening model, this model was not used for lack of required data. A basic feature of the present hardening-soil model is the dependence of soil stiffness on stress. For oedometer conditions of stress and strain, the model additionally provides the relationship of power law for E_{oed},

$$E_{oed} = E_{oed}^{ref}\left(\frac{\sigma}{p^{ref}}\right)^{m},$$

where E_{oed}^{ref} is oedemeter modulus for reference pressure p^{ref} and m input exponent. In this equation a tangent oedometer modulus was considered at a particular reference pressure p^{ref} (usually 100 kPa). For plastic deviatoric straining due to primary compression a well-known relationship was used between Young's modulus, an oedometer modulus and Poisson's coefficient (taking into account the conditions of lateral constraints in oedometer):

$$E_{50}^{ref} = E_{oed}^{ref}\frac{(1+\nu)(1-2\nu)}{(1-\nu)},$$

According to the back analysis the elastic unloading /reloading parameter was found to be:

$$E_{ur}^{ref} \approx 1.8 \cdot E_{50}^{ref},$$

For the coefficient of horizontal stress a default relationship was used:

$$K_0^{nc} = 1 - \sin\phi \cdot$$

Mechanical and hydraulic parameters of the materials used in the numerical model are given in Tab. 1 and 2. An example of deriving E_{oed}^{ref} and m from results of oedometer test is presented in Fig. 2a. Calculated and design (average) values for E_{oed}^{ref} and m through the dam core and the foundation soil profile are presented in Fig. 2b and c.

The loading and unloading of the dam are stages in calculation. Throughout all the stages of calculation, vertical displacements at a number of points at the dam embankment were calculated. This paper describes the displacements of the point G located at the dam crest (Fig. 1.).

The results of the calculation were primarily compared with geodetic measurements and other field observations. Fig. 3 shows comparative diagrams of horizontal displacements in transversal direction and settlements of the point G and those at geodetic measuring points 4 and 6 at the crest. As geodetic measurements had not been made since the start of the dam construction, they were related with numerical consolidation so as to begin as late as 1800 days after the commencement of construction.

Table 1: Mechanical and hydraulic parameters of the materials for elements described by Mohr-Coulomb constitutive model

ID	soil	k_x m/day	k_y m/day	E_{oed} kPa	E_{ink} kPa/m'	ν	c kPa	ϕ °	γ_{dry} kN/m^3	γ_{wet} kN/m^3
1	clayey gravel	1.0E-02	1.0E-02	22000	0	0.3	10	20	18	20.5
3	support zone	1.0E-01	1.0E-01	22000	0	0.3	2	34	18	20.5
6	filter 1	1.0E-03	1.0E-03	22000	0	0.3	0,2	34	18	20.5
7	filter 2	1.0E-02	1.0E-02	22000	0	0.3	0,2	34	18	20.5

Table 2: Mechanical and hydraulic parameters of the materials for elements described by Hardening-Soil constitutive model (p_{ref}=100 kPa).

ID	soil	k_x m/day	k_y m/day	E_{oed}^{ref} kPa	m	E_{50}^{ref} kPa	E_{ur}^{ref} kPa	ν	c kPa	ϕ °	γ_{dry} kN/m^3	γ_{wet} kN/m^3
2	clay	2.0E-04	1.0E-04	4320	0.49	3210	7600	0.3	20	20	17	20
4	core	1.0E-04	1.0E-04	5280	0.44	3922	7845	0.3	100	20	16	20
5	soft clay	2.0E-05	1.0E-05	3390	0.56	2520	6200	0.3	14	18	14	19

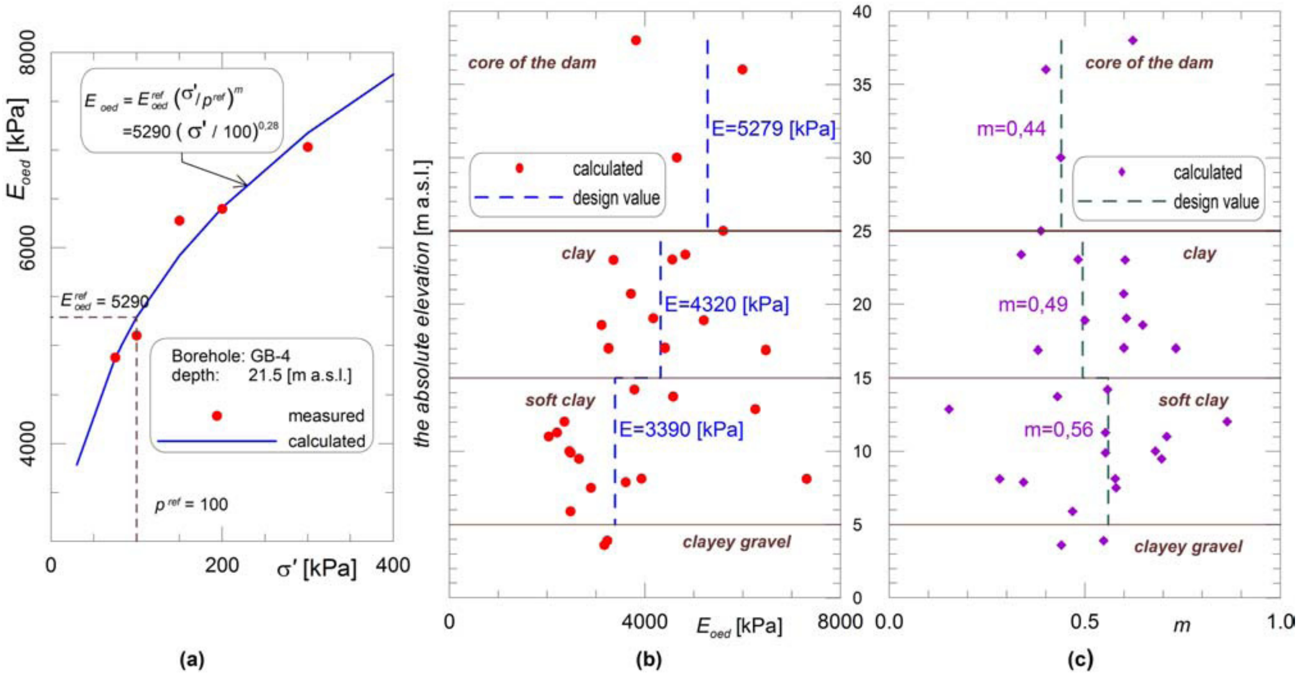

Figure 2. Deriving E_{oed}^{ref} and m from an oedometer test (calculated values) (a); Calculated and design (average) values for E_{oed}^{ref} (b), and m (c) through the dam core and the foundation soil profile.

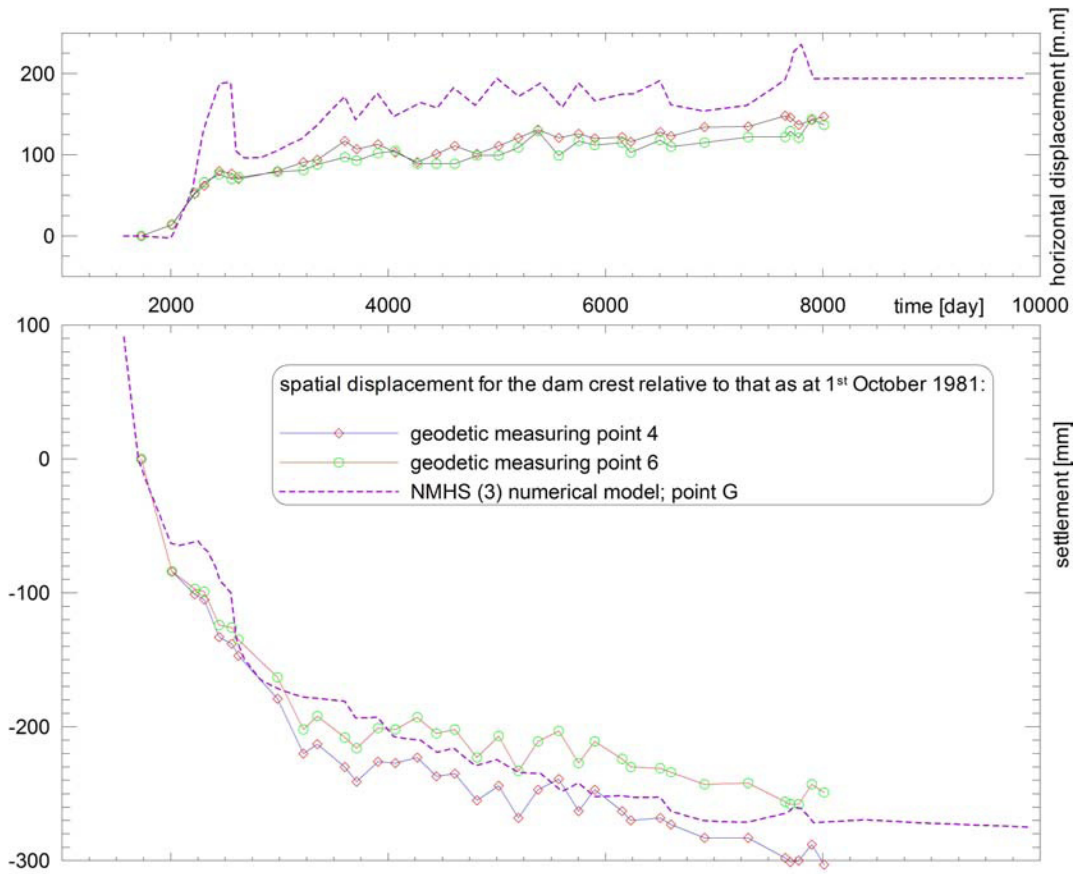

Figure 3 Comparative diagrams of horizontal displacements in transversal direction and settlements of the point G and of those at geodetic measuring points 4 and 6 at the crest

4 STABILITY ANALYSIS

When the construction of the dam began, an accident happened. Shortly after the third layer was filled and compacted (from height 6 to 11 m), the downstream portion of the dam failed due to the foundation soil failure. It is most likely that the failure occurred because designer's instructions as to time intervals between the layer placement were not followed. This resulted in pore pressure build-up and a decrease in shear strength in the foundation soil. The authors used this accident as a benchmark model in the stability analysis. The parameters available for the numerical model were the results of the tests carried out before the dam construction and those carried out five years later, when the dam was already built. Those tests involved SPTs and laboratory tests, i.e. direct shear, triaxial and unconstrained tests. The test results were verified by calculating undrained shear strength c_u. The results of the calculation are plotted on a graph showed in Fig. 4. c_u was calculated approximately from direct shear and triaxial tests by using normal stress and shear strength parameters, as half of unconfined compression strength and on the basis of correlations with SPTs according to Bowles (1982). It may be concluded that the test results are compatible in lower layers, with grater discrepancies in upper layers. On the graph distinction is made between undrained strength measured before and after the dam construction. On the basis of average values of c_u design shear parameters for clay core and foundation soil were selected.

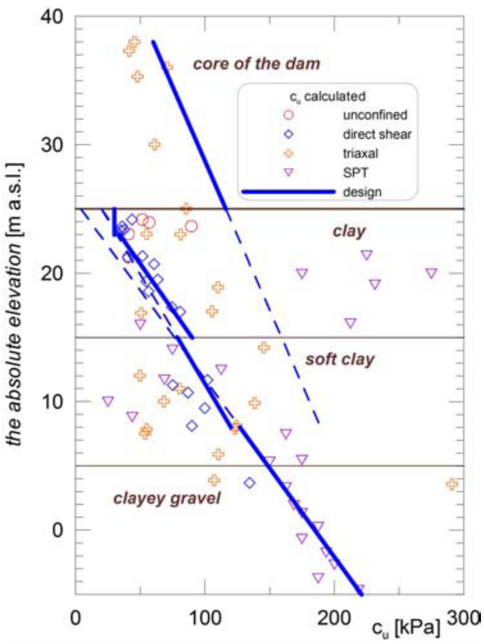

Figure 4. Comparative diagrams of in situ and laboratory tests for determination of undrained strength.

On the basis of the said parameters, a calculation using Plaxis was carried out taking into consideration the foundation soil consolidation and the time of the placement of the third soil layer. A back-analysis of the case when the third layer was placed in less then 30 days showed that the soil failure would occur (Fig 5). The results of the stability analysis that assumed different time intervals between the placement of the first two fill layers and the third layer are given in Fig. 6. It may be concluded that the third fill layer should be placed after at least 120 days when Fs is greater than 1.3.

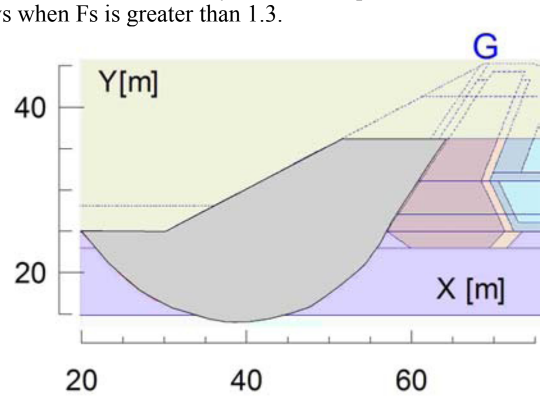

Figure 5. Results of the back-analysis of soil failure

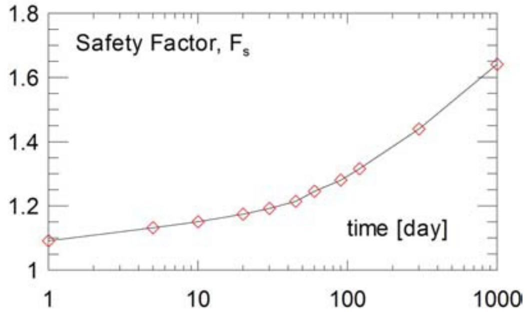

Figure 6. An increase in factor of safety depending on time of placement of the third fill layer.

In addition, these parameters were used in computations made to establish two things. One is the influence of a change in water level on dam stability and the other the current degree of stability in comparison to ultimate stability. The calculation indicated that factor of safety (Fs) varies up to 2% with the change in water level in the reservoir (for 36 m a.s.l. Fs=1.43 and for 40 m a.s.l. Fs=1.46). Changes in the current degree of stability in comparison to ultimate stability, for all water levels considered, were negligible (up to 0.5%).

5 CONCLUSION

The comparison of the behaviour of the constructed dam and the numerical model indicates that deformation and shear parameters of materials, determined on the basis of the results obtained from field investigations and laboratory testing, were estimated correctly (see also Matešić et al. 2003a and Matešić et al. 2003b). Regular visual observations showed no evidence of possible occurrence of significant dam deformation, leakage or any other manifestations that may pose a risk to its stability. This demonstrates that the behaviour of the dam is as designed.

Creep behaviour of the foundation soil was not analysed because of the lack of proper material parameters. Consequently, further soil and laboratory investigations should be carried out.

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