Some Examples of Z_Soil Application to Dam Engineering

N. Nilipour and C. Cekerevac
Stucky Ltd
Rue du Lac 33, P. O. Box
CH – 1020 Renens, Switzerland

Keywords: Concrete dam, thermal analysis, tunnel junction, buckling, earth-fill dam, water loss.

1. Introduction
In this paper some applications of finite element method in the field of Dam Engineering are presented. Different modules of Z_Soil software were used based on the type of the calculation.

In Chapter 2, different types of thermal analysis of the dams are discussed and for each case examples are presented. In Chapter 3, an embankment dam is presented as an application for seepage problems. Finally in Chapter 4 an example of tunnel lining calculation is discussed.

2. Thermal Analysis of Concrete Dams
The type of analysis to be carried out nowadays in the domain of concrete dams can be categorised into two main groups: a) Existing dams, b) Design of new dams.

In the case of the existing dam, back analysis is usually performed to verify proper behaviour of the structure. An important part of the back analysis of dams deals with temperature variation inside the structure and related thermal stresses and displacements. This can be done by correlating the temperature measurement readings with the results of a numerical model simulating the variation of temperature on the boundaries and thermal characteristics of the dam.

Whereas for the new dams, the design of the structure should be carried out based on static and dynamic analyses. Temperature load case plays an important role in
determining maximum stresses in the dams. Special care should be given to the variation of water temperature and reservoir level variation. Usually two load cases should be considered taking into account maximum (summer) and minimum (winter) temperature load cases, causing usually maximum thermal compressive and tensile stresses on the dam faces, respectively.

Another important thermal analysis for new dams focuses on the construction details and temperature control measures. For the final design of the dam, hydration heat produced by the concrete setting process have to be considered in order to avoid thermal cracking of the new concrete. Additionally, the grouting of the vertical contraction joints has to be performed in a way to respect the design closure temperature as well as the project schedule, particularly for arch dams. In both cases construction schedule and temperature control measures are adapted so that all thermal aspects of design criteria are satisfied.

Obviously, the most complicated case is the combination of all issues above-mentioned, which is the case of heightening or important rehabilitation of existing dams. In the following chapters some examples of thermal analysis of dams are discussed and the results are presented.

2.1. Back Analysis

Thermal back analysis can be carried out using two different approaches: (a) Analytical solution to heat transfer equation using average temperature curves, (b) Finite Element transient thermal analysis using real measured temperature curve. For the following examples of thermal back analysis of dams Time-dependant (transient) thermal module of Z_Soil.PC 2003 was used.

The Spitallamm and Seeuferegg dams are parts of an important scheme on the upper course of the Aar River in the central part of Switzerland. The whole scheme includes seven dams, nine powerhouses with pumping-generating capacities, eleven reservoirs and many km of pressure tunnels and pressure shafts, resulting in a quite complicated and versatile hydroelectric system owned by Kraftwerke Oberhasli Ltd (KWO). The Spitallamm dam impounds, together with the Seeuferegg dam, the Grimsel reservoir. Whereas the Spitallamm dam consists of a circular arch-gravity structure with a maximum height of 114 m, a gravity type dam has been chosen for the 42 m high Seeuferegg dam. The main goal of the back analysis is to verify the thermal behaviour and parameters of the both dams in the framework of a heightening project of the Grimsel lake [1] and [2].

2.1.1. Spitallamm Arch-Gravity Dam

Constructed between 1928 and 1932, the dam, with its maximum height of 114 m, presents a cylindrical shape with a near vertical U/S face (10V:1H) and a 1V:0.5H D/S face (Figure 1). To insure the impermeability of the dam, the upstream
face was provided with higher cement content while the dam body was built using mass concrete.

![Figure 1. Situation layout and cross section of Spitallamm arch-gravity dam.](image)

Thermal calibration of Spitallamm dam has been performed by comparing the results of the transient analysis and the thermometers readings. Considering the shape and the orientation of the Spitallamm dam with respect to the sun and also being an arch dam, solar radiation heating effects vary with the different dam sections. Therefore, the thermal analysis has been carried out on three 2-D sections with different orientations, Figure 1. For each section a preliminary case is first analyzed without considering the effects of solar radiation. Then based on the differences between the calculated and thermometer reading values, the solar radiation can then be estimated. Since the temperature rise due to the solar radiation is different on the upstream and downstream faces and on the dam crest, a few iterations were required to reach the optimum boundary temperatures so as to properly simulate the temperature field within the dam. The results of the thermal back analysis for one of the 2-D sections are presented in Figure 2.

It should be mentioned that thermometers close to the surface could be affected by daily or weekly variations of ambient temperature. Additionally, it is possible that due to surface cracking some thermometers close to the upstream face of the dam are in contact with the reservoir water. Therefore, the adjustment of the boundary temperatures is carried out mostly based on the results for the most interior thermometers.
The temperature field of the dam calculated for different moments verifies the thermal properties of the concrete used for the calculation as well as the thermal loads and the solar radiation effect. Effect of the solar radiation was found to vary for different sections, ranging from 1 to 3 °C for summer and 3 to 5°C for winter.

2.1.2. Seeuferegg Gravity Dam

In the case of Seeuferegg dam, it is assumed that the effects of solar radiation can be considered constant over each face of the dam. Hence, only one 2-D section, Section 5a-6, was analysed. Figure 3 shows the selected section and position of the thermo-couples.

Figure 2. Back analysis results comparing the calculated and measured temperature in the dam.

Figure 3. Position of Thermometers in Block 5-6a of Seeuferegg Dam.
Since the terrain factor for Seeuferegg dam is negligible and the dam is exposed to the sun almost in the same manner throughout the year, the effects of solar radiation are assumed to be not distinguishable between summer and winter.

Figure 4 shows the comparison of the calculated temperature with thermocouples measurements. As it is usually the case for such analysis, the correlation for thermometers close to the dam surface is not very accurate, since they can be affected by some small superficial cracks. The rise of temperature due to solar effect is estimated 6 °C and 2°C for upstream and downstream faces, respectively.
2.2. Analysis During Construction

Thermal analysis during construction focuses on the development of temperature due to hydration heat of cement during setting process of the concrete. Two examples are briefly presented in this chapter.

2.2.1. Picada RCC gravity Dam

Picada Hydroelectric Power Plant Project dams the Peixe River flowing in the State of Minas Gerais in Brazil. The job site is located at approximately 40 km west from the city of Juiz de Fora. It takes advantage of a 128 m water head to yield a 51 MW energy production. The project enjoys an RCC gravity dam with height of 27 m, crest length of 97 m, RCC volume of 12’600 m³ and CVC volume of 4’400 m³.

The ambient temperature at site has been chosen variable through the year between 16°C (June to August) and 22°C (December to February). The construction begins in August over 45 days assuming one RCC layer per day. The adiabatic temperature rise of the cement assumed is 13.5°C/(100 kg cementitious content), corresponding to a Portland Blast Furnace Slag cement. The air convection surface coefficient has been selected as 20 N/(m.s.°C). This figure accounts for an average wind effect of 10 km/h.

Figure 5. Temperature distribution in Picada dam during and after construction.
The computation has been run with a selected time step of 0.1 day (2.4 hours) during construction and progressively from 1 to 5 days once the dam is completed. The overall time period considered in the computation is one year. The placement of two RCC layers per day has been considered, 6 days a week. This yields an overall dam construction time of 46 days.

With a placement temperature assumed at 25°C, the results show that in the central section of the dam, the temperature rise due to the hydration heat is almost adiabatic, and the temperature rises up to a maximal value of 44.0°C, Figure 5.

Considering a placement temperature of 16°C or even 30°C does not significantly alter the temperature development, Figure 6. The trend of the three curves is very similar, with peak values reaching 45.2°C, respectively 44.2°C and 42.1°C for a placement temperature of 30°C, respectively 25°C and 16°C. It is therefore inferred that the placement temperature does not play a significant role on the peak temperature value. Given its limited volume and thin depth (30 cm), the fresh and recently placed RCC layer is sensitive to external temperature conditions and thus cools down before the hydration heat fully mobilises, thanks to the low ambient air temperature prevailing in August (16°C).

It should be mentioned that in the mid-height of the dam, where temperature reaches almost 45 °C, a 9 m high test block performed for research reasons using high strength RCC was foreseen with a cementitious content varying between 200 and 240 kg/m³. Otherwise, such high temperature rise in a normal RCC gravity dam
is unusual. In the other parts of the dam a low cement RCC with 8 MPa strength was used.

2.2.2. Seeuferegg Gravity Dam

In the framework of KWO+ project, 23 m heightening of Seeuferegg dam has been foreseen. The heightening of this gravity dam is performed on the upstream side by adding a hollow gravity part to the existing dam, see Figure 7.

The evolution of temperature during construction of Seeuferegg Dam is studied by conducting transient thermal analysis considering hydration heat of the cement and ambient temperature variations. The results of this analysis are to be used to verify construction schedule with respect to the joint grouting schedule, necessity of pre- and post-cooling and to determine the potential of cracking by performing thermo-mechanical analysis.

![Figure 7. Heightening of Seeuferegg Dam, different variants.](image)

Construction schedule criteria are defined to respect minimum horizontal, vertical and horizontal time interval between concrete lifts, i.e. 3 days. The construction of Seeuferegg Dam lasts 14 working months during 3 years, assuming 5 working days per week and concreting two blocks per day. 3-D FE modelling is carried out simulating concrete hydration heat and construction schedule of the three central blocks.
In most parts of the dam, maximum temperature occurs after 3-4 days in the centre of the block. The temperature rise is almost the same in all elevations, between 24°C and 25.5°C, except in the beginning of the concreting seasons where the effect of boundary conditions is considerable. Variation of the vertical time interval in different elevations can affect the maximum temperature rise. Therefore, it can be said that having 3m-height concrete lifts, the temperature rise in the centre of the block is almost adiabatic.

Since the calculation is conducted assuming concreting against the existing dam, a local temperature rise can be seen in the existing concrete close to the interface. In the early days after concreting, the temperature increase can be more than 25°C. Temperature rise on the upstream surface is much less, maximum 10°C, and after few days follows the ambient temperature variations. Hence a noticeable thermal gradient occurs between the centre of the block and upstream surface as well as interface with the existing concrete. The maximum thermal gradient between the interior and the surface reaches 21.5°C few days after concreting while two years after construction in the beginning of the summer, the effect is reverse and thermal gradient can reach (–12.2°C) which means the temperature on the surface is higher than the interior of the dam due to the seasonal ambient temperature variations.

Figure 8. Evolution of temperature in the new part of the dam during construction.
In all elevations, rapid temperature rise followed by rather quick natural cooling from the top free surface can be observed during the first few days, before concreting of the upper concrete lift, Figure 8. This shows again the importance of the vertical time interval between concrete lifts. A slight temperature increase occurs due to upper concrete lift and then natural cooling continues, mostly through lateral sides. For the lower part of the dam, up to Elevation 1900 masl, natural cooling proceeds from the upstream face and through the cavities (if they are open), while for the upper part of dam (above the existing dam crest) cooling can be fulfilled from both upstream and downstream faces. Therefore, from thermal point of view, the critical zone during construction is between Elevations 1900 masl and 1911.84 masl. On the other hand, natural cooling in this zone lasts longer than the other parts of the dam. This can be improved by modifying the conduction schedule in order to increase the vertical time interval between lifts or by re-grouting the joints in the next season when the concrete is cooled down to the closure temperature. In this case, regroutable pipes should be installed in this zone.


The following example presents utilisation of Z_Soil for a back-analysis of underground flow of Wadi Meneghin dam. The dam is a 38 m high, concrete face rock-fill structure, located approximately 70 km South-West of Tripoli. It was constructed in 1972, for purpose of irrigation and flood control. The dam body is formed of rockfill and is made watertight by the provision of a reinforced concrete slab on the upstream face. The upstream concrete membrane connects to a concrete cut-off wall formed of “Benoto” piles. There is also a grout curtain extending below the concrete cut-off. The upstream face is formed in four planes with slightly different slopes ranging from 1 : 1.3 (V : H) at the toe to 1 : 1 at the crest. The downstream face is formed at a slope of 1 : 1.4 (V : H) but also incorporate three intermediate berms.

In February 2003, water was observed flowing in the Wadi, downstream of the dam. In order to identify cause and improve water tightness a series of piezometers, set in pairs a short one and a long one, have been placed in the dam profile and six of them have been used for back-analysis here presented (Figure 9).

<table>
<thead>
<tr>
<th></th>
<th>P5</th>
<th>P5.1</th>
<th>P2</th>
<th>P2.1</th>
<th>P8.1</th>
<th>P8</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Elevation (masl)</strong></td>
<td>228.23</td>
<td>241.48</td>
<td>227.22</td>
<td>240.67</td>
<td>223.00</td>
<td>223.00</td>
</tr>
<tr>
<td><strong>W.L. (measure)</strong></td>
<td>248.30</td>
<td>248.00</td>
<td>248.80</td>
<td>247.80</td>
<td>249.20</td>
<td>255.30</td>
</tr>
</tbody>
</table>

Table 1. Piezometers readings at low water level (May 2004).
A series of piezometers readings have been carried out starting from December 2003 up to the middle of 2004. Here presented results corresponds to the low water level (May 2004) for which piezometers readings are given in Table 1. Upstream water level was at 268.77 masl and downstream at 247.00 masl.

Figure 9. Wadi Megenin dam: geometry of the seepage domain, materials and position of piezometers.

Based on a preliminary laboratory results, the water permeability of all materials were known with some incertitude. Therefore, the first part of the study was to
calibrate the water permeability of the materials (Figure 9) based on above shown pore pressure measurements.

Figure 10 presents numerical results of the pore pressure distribution over the domain. The final results of the calculations in terms of difference between measurements and calculated water height in the considered piezoemeters are presented in Figure 11. Piezometers P5 and P5.1 and P2 and P2.1 are presented by mean values since they are constructed on the same section (Figure 9). It can be seen that there is very good correlation between measured and calculated values; the biggest difference is less than 0.5 m.

Based on such back-calculation, we find out water permeability of related materials and thus we find precisely water outflow below the dam. Integrating the outflow over the dam length we found total lost of 3.50 m³/day. This value is largely smaller than in-situ observed indicating that the most of water loosening do not happens below the dam.

![Graph showing difference between calculated and measured water height in the piezoemeters.](image)

Figure 11. Difference between calculated and measured water height in the piezoemeters.

The above study initiate an additional study concentrated on water outflow lateral of the dam. The study shows that most of the loosening happens laterally and consequently rehabilitation measures were proposed.

4. Numerical modelling of tunnel lining

In addition to the above presented thermal analysis of Seeuferegg and Spitallamm dams, will be presented an analysis of so-called appurtenant structures, which are part of KWO hydroelectric system. The whole system covers a huge catchments surface, seven dams and nine powerhouses with pumping-generating capacities
linked with many kilometres of tunnels under very high internal and external pressures.

Internal pressure acts during most of the time, i.e. operation of the tunnel while external pressure becomes very important only during emptying of the tunnel, when internal pressure is almost zero. In such loading case, the tunnel lining should satisfy the following design criteria. First, to prevent failure of the lining and water loosen due to internal pressure and second to prevent buckling of the lining due to an important external pressure.

In order to support high tensile stresses in the lining an internal steel lining is a very common solution. However, due to the external pressure and possibility of the lining buckling some additional measures must be taken into account. The following example presents a finite element analysis of a steel reinforcement of two galleries junction – called Kapf.

Design of the lining of the junction Kapf has been done by few iterations in order to optimise dimensions of the lining and stiffening rings. It should be noted that lining of existing tunnel has been designed and constructed as self-supporting. Moreover, according to technical specifications imposed by KWO (Table 2), lining of the new tunnel has to be designed as self-supporting as well.

<table>
<thead>
<tr>
<th>Diameter of existing gallery</th>
<th>3300 mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>Thickness of the existing gallery lining</td>
<td>8 mm</td>
</tr>
<tr>
<td>Diameter of new gallery</td>
<td>3600 mm</td>
</tr>
<tr>
<td>Diameter of junction existing-new gallery</td>
<td>3300 mm</td>
</tr>
<tr>
<td>Radius of the junction</td>
<td>9400 mm</td>
</tr>
<tr>
<td>Thickness of new gallery lining</td>
<td>12 mm</td>
</tr>
<tr>
<td>Maximal internal pressure</td>
<td>58 mCE</td>
</tr>
<tr>
<td>Dynamic pressure</td>
<td>70 mCE</td>
</tr>
<tr>
<td>Maximal external pressure</td>
<td>4 bar</td>
</tr>
</tbody>
</table>

Table 2. Technical specifications of two tunnel junction - Kapf.

Assuming a stiffening ring 150 x 15 mm spaced 1500 mm we calculate the critical external pressure for stiffener rings including effective width according to Jacobsen’s theory [3] to be 1.81 MPa. The critical external pressure for shell between rings has been calculated according to Mises to be 1.02 MPa. However, in
the zone of junction of two tunnels, the stiffening rings have been designed to resist to a part of internal pressure as well. In other words, the rings have to support a part of the internal forces that has not been tunnel lining [4].

![Figure 12. Finite element mesh.](image)

In addition to that, a 3D finite element analysis has been carried out by using Z_Soil and its beneficial pre and post-processing capabilities. As can be seen in Figure 12, a complex geometry of the tunnel junction lining has been simulated using 3D solid elements with three unknown in each node: displacement in three directions: $u_x$, $u_y$ and $u_z$. In order to simplify pre-processing and to diminish calculation time, we used the symmetry and a only a half of the domain has been modelled. For the analysis we assume elastic behaviour of the tunnel lining, with elastic limit of 355 GPa and theory of small deformations.

It should be noted that the analysis has been carried out for internal and external pressures. Since the internal pressure gives the most critical stresses and strains in the lining here will be presented only results of this analysis.

Principal stress $\sigma_1$ and principal strain $\varepsilon_1$, calculated for dynamic internal pressure of 700 kPa are presented in Figure 13. It is clear that in the zone of closed tube, the principal stress acts on the radial direction with a maximal value of 97 MPa. However, in the junction zone repartition of the principal stress changes and it increases considerably up to 160 MPa. Also, in a zone of cone introduction, the principal stress increase as well. The major principal strain is in the range of $5.5 \times 10^{-4}$.

The above presented analysis shows that stresses and strains of the designed tunnel lining are in acceptable range. The results clearly show zones of the stress and strain concentration, which do not influence the lining stability.
Figure 13. Principal stresses: a) the maximum principal stress, $\sigma_1$ and b) the minimum principal stress $\sigma_3$. 
5. Conclusions
Some examples of the application of Finite Element Method using Z_Soil software are presented and discussed.

User-friendly features and strong pre- and post-processors allow modelling more precisely the geometry, loads and other issues dealing with numerical modelling. However, engineering judgment and simple hypotheses based on the practical experience play a very important role in the domain of dam engineering and it should be always considered in the design and construction of such structures.

6. References