The Optimization of the Design of a Concrete Faced Rockfill Dam by the Application of Sand-Gravel Fill Zones

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ABSTRACT: The design of the dam case study presented was changed during different design and planning stages, switching from an ordinary CFRD type to a quite sophisticated CFSGD with deep foundations. Whilst obtaining more detailed information on the geological conditions and the available fill materials the engineers responsible decided to use the alluvial deposits close to the dam area for the fill. Originally, the dam was to have shallow foundations on the alluvial deposits present before clay layers with considerable thicknesses were encountered within the dam foundation. Therefore, the complete foundation concept had to be changed to a deep foundation in a highly permeable alluvium. Additionally, the left bank suffered from landslides during initial excavation works for the energy tunnel inlet so that the stability and design adjacent to the left bank has to be re-evaluated and re-designed. After agreeing on the detailed design principles and a general dam design material, investigations are ongoing and the optimization of the dam design is expected to be completed before starting dam fill works in mid of 2011.

Keywords: CFRD, CFSGD, Sand-Gravel Fill, Rockfill, Meta-Flysch

1 INTRODUCTION

In Turkey a huge number of hydropower projects are under construction. Frequently, these hydropower projects are for dams of considerable heights of more than 50 m (Haselsteiner et al., 2009a, b). Due to the threat of energy scarcity and thanks to the privatization of the energy sector at the beginning of this millennium, Turkish hydropower projects face a remarkable pressure in regard to project costs and realization periods exerted by the private sector. While previously, the projects were mainly financed and coordinated by governmental organizations the private investors and owners go for profit. One of the dominant newcomer private energy companies in Turkey is EnerjiSA, a joint venture of the Sabanci Group (Turkey) and Verbund (Austria). The goal is to reach an installed capacity of P = 5,000 MW by 2015 with a portfolio which increases by half the existing capacity of hydropower projects. Sabanci as newcomer to the energy sector grows tremendously, while relying on the support of Verbund with its long experience and considerable knowledge in construction and operation of hydropower plants in Austria. Both partners split the shares and the investment equally.

One main development area of EnerjiSA and also Turkey is South-East Anatolia where mountainous regions give rise to some of the large rivers of Turkey such as the Euphrates, Tigris, Seyhan and Ceyhan. On the latter rivers some well-known dam Turkish projects have already been constructed. Among them are the Aslantas, Sir, Berke and Menzelet dams (Figure 1). Berke Dam was the highest double arch curvature dam in Turkey with a height of 201 m before Ermenek Dam with a height of 210 m edged it out by only a few metres. Deriner Dam is now already underway and shall be the highest dam in Turkey when measured from foundation to crest, reaching H = 247 m (Wieland et al., 2007).

With these dam heights one should not forget that 100 m high dams reflect a considerable challenge and risk, particularly when geology does not act as anticipated and foundation as well as dam fill material create challenging engineering tasks. One of those projects is EnerjiSA's Sarigüzel Dam and HEPP which is located on the Ceyhan River approximately 1.5 h drive away from Kahramanmaras not far from Adana in South-East Anatolia. Sarigüzel is EnerjiSA's second project on the Ceyhan River, upstream of Hacininoglu and downstream of Kandil (Figure 1) - another two projects owned by EnerjiSA. The cascade which belongs to EnerjiSA further consists of Dagdalen upstream of Kandil. At a contributory downstream of Hacininoglu, EnerjiSA overtook the Sucati project; a smaller project with 7 MW installed capacity which is also the first high RCC dam in Turkey. An overview of the development of the project of EnerjiSA is given by Kaya et al. (2010) for the reference years 2009/10.

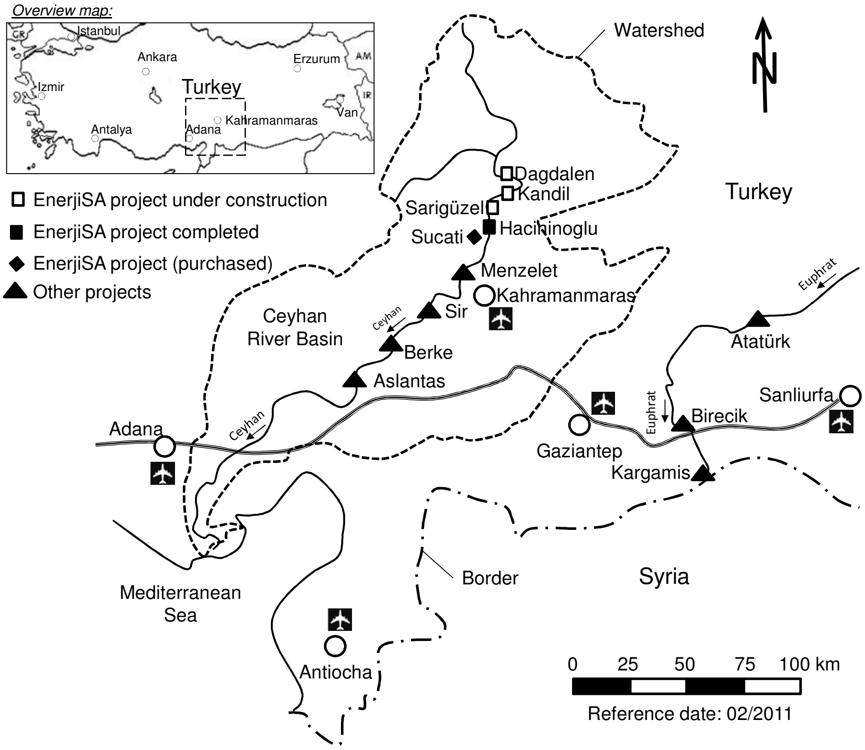

Figure 1. Ceyhan River Basin in South-East Turkey including EnerjiSA HEPPs and some large governmental projects

Although, the Turkish government initiated the "Güneydoğu Anadolu Projesi" (GAP) project consisting of over more than 19 large dams on the rivers Euphrates and Tigris decades ago, a number of large projects are still undeveloped in these regions, especially on the tributaries of these large rivers. One of the largest projects on a main stream which is part of the GAP project and is now under construction is the Ilisu project on the Tigris which has an installed capacity of P = 1,200 MW and a dam height of H = 130 m. This project faced much national and international resistance due to social and environmental impacts. Two of the most critical issues are the interference with important cultural heritage within the district town Hasankeyf and the large number of affected civilians. The project is being developed by the government.

Although, the Sarigüzel project is also a large dam the social and environmental impact assessment did not show serious impacts as the region does not have large settlements and/or cultural heritage within the project and reservoir area. Instead, the Seyhan and Ceyhan basins accommodate mountain torrents at high elevations. These are inhospitable conditions for big farms or settlements or for intensive agriculture. These conditions led only to small settlements which did not develop large cultural heritage and the people living there made their livings by extensive farming and livestock.

2 GENERAL CONDITIONS & DAM FILL MATERIALS

2.1 *General conditions*

The Sarigüzel Dam and HEPP has an installed capacity of approx. P = 104 MW including also an environmental powerhouse with an installed capacity of P = 3 MW. The total annual energy generation is assumed to reach A = 280-290 GWh/a (Haselsteiner & Ersoy; 2011a). The project consists of a dam and a headrace tunnel with a length of approximately six kilometres, two powerhouses and an ogee overflow spillway with six bays. Optimization measures may decrease the number of bays. The dam type and the general dam layout are discussed in the sections below.

In Table 1 the main data of the Sarigüzel project are summarized. Whereas some of the figures are taken from existing studies, some are still under discussion, particularly for the dam. These figures may be changed before detailed design is completed or even during the construction for optimization.

Table 1. Main basic data of the Sarigüzel project (see also Kaya et al., 2010; Haselsteiner & Ersoy, 2011a)

Item	Figure/data
Installed Capacity Main Plant P [MW]	101
Annual Generation Main Plant A [GWh/a]	257
Design Discharge Main Plant Q_D [m³/s]	111
Type of Units Main Plant	2 x Francis
Headrace Tunnel Main Plant L [km] / D [m]	5.5 – 6.0 / 6
Installed Capacity Env. Plant P_{Env} [MW]	3
Annual Generation Env. Plant A_{Env} [GWh/a]	25
Environmental Flow Q_{Env} [m³/s]	5.4
Catchment Area E_0 [km²]	6,500
Mean Annual Discharge Q_m [m³/s]	49
Maximum Probable Flood Q_{PMF} [m³/s]	4,863
Diversion Design Flood Q_{Div} [m³/s] / T [a]	546 / 25
Reservoir Volume $V_{R,tot}$ [Mio. m³]	≈ 50
Active Reservoir Volume $V_{R,act}$ [Mio. m³]	≈ 25-30
Normal Operation Reservoir Level [masl]	860
Minimum Reservoir Level [masl]	840
Dam type	CFSGD (former: CFRD)
Dam height H_D [m]*	≈ 80
Crest length L_C [m]	≈ 540
Dam Fill Volume V_D [Mio. m³]	≈ 3.1
Crest elevation [masl]	865
Upstream slope [1:V]	1.4 to 1.7
Downstream slope [1:V]	1.4 to 1.7

* Referring to thalweg elevation

2.2 *Geological and geotechnical conditions*

Special attention had to be paid to the occurrence of meta-flysch at the dam axis and along the headrace tunnel. The bedrock of both abutments consists of these metamorphic sedimentary rocks which show, particularly locally, poor rock properties and strengths. Additionally, the colluvium at the left bank suffered from a landslide caused by initial excavation works. Therefore, the left bank stability is being re-investigated and additional stability works will be executed. Fortunately, at the bottom of the left bank the quality of the meta-flysch is in better condition with regard to weathering, discontinuities and strength so that the occurrence of a global failure is unlikely to occur. Investigations are still ongoing for this subject.

Generally, two types of meta-flysch have been distinguished. Poor meta-flysch was characterized by an uniaxial compression strength of UCS = 10-15 MPa or smaller, whereas the stronger meta-flysch showed values of UCS > 15 MPa. Several specimens reached UCS > 30-50 MPa. Generally, the weaker meta-flysch is overlaying the stronger units at the left bank. Corresponding to the genesis of this formation some parts were classified as schist. The RMR values comprised a range of RMR = 30-40 on average (rough estimation) leading to values of GSI = 25-35.

Before the construction works started, a drilling programme was undertaken that showed partly very poor TCR and RQD values. This led to a general underestimation of the geotechnical parameters of the rocks present. A second extensive drilling programme resulted in TCR values throughout of more than 90%, also comprising very weak, completely decomposed shear zones. This second drilling programme particularly contributed to a better understanding of the geology present. During the extensive drilling programmes the very weak shear zones were of major concern. Generally, these shear zones lead to decreasing shear strength between the potential shear surfaces which should not be affected by more than 10-20 % by the weak shear zones. The present metamorphic sedimentary rock types sand-, silt- and claystones are dominant.

The global orientation of the discontinuities, if valid for this kind of metamorphic rock formation, is considered to be favorable in terms of the global slope stability. Still, investigations and discussions are ongoing to determine the actual rock stress-strength and stress-shear behavior.

The local stability of the colluviums consisting of sandy-clay material with interbedded limestone blocks is dominated by an interface layer between the colluvium and the meta-flysch which showed very weak shear strength. After the landslide occurred at the left bank at the beginning of 2010, slickenside shear surfaces could be observed. Laboratory tests confirmed that the residual shear parameters correspond to typical slickenside surfaces showing φ_R = 12-15° and c_R = 0 kN/m². The colluvium itself shows peak strength parameters of approx. φ' = 25-30° and c' = 15-20 kN/m².

2.3 *Foundation conditions*

The Ceyhan river shows considerable alluvial deposits of more than 30-50 m thickness within the riverbed. Interbedded clay layers of several metres depth indicate large settlement if loaded. This was also the reason for the adjustment of the foundation design, since large settlements could not be excluded. The decision to change to deep excavation was taken reaching down to a depth of approximately 30 m corresponding to the depth that the clay layers were encountered.

Generally the alluvium shows permeable characteristics with $k \geq 10^{-4}$ m/s. Highly permeable coarse gravel layers are considered to be able to allow large seepage flow during the excavation of the foundation. This was the reason for applying cut-off walls up- and downstream of the excavated foundation area. Conversely, the abutments consisting of poor to medium classified meta-flysch show only very low permeability characteristics. The Lugeon values obtained do not exceed the value of approximately four Lugeon at an investigation depth down to 60 m below ground surface. The strength parameters are already described within the previous section.

2.4 *Available dam fill materials*

The availability of dam fill materials influenced the dam design considerably. Counter to initial expectations, when a rockfill dam design was favored, strong and decent rockfill material with an adequate volume was not encountered within a reasonable distance. Alternatively, the present alluvium was taken into consideration as dam fill material. In Figure 2 the site map is given of the Sarigüzel dam area. The alluvial deposits are close to the dam area mainly downstream of the dam. Still the volume of available materials is under investigation. Most of the areas are downstream of the dam axis. For materials in areas B, C and D field trial compaction tests were carried out.

During the first stages when no site testing had been performed and the borrow areas were not finally defined several sieve curves were prepared from some of the proposed borrow areas. In Figure 3 the sieve curve envelope for the main dam fill zone 3B is shown. Compared to benchmark curves the area 3B comprises sandy-gravels to gravelly cobbles/stones showing a maximum of over 50% of grains with block size. Benchmark data for sand-gravel fills are described in Cruz et al. (2009), Fell et al. (2005), Kutzner (1996) and Noguera et al. (1999). Generally, the dam fill material is considered to be very suitable as fill material if the technical specifications are defined appropriately.

The next step was to investigate all the borrow areas and test them in detail. The range of the sieve curves obtained is given in Figure 4 which more or less corresponds to the results of the first program. To be accurate, the material has to be classified as sandy gravel with low percentage of fines (< 10 %). Further processing cannot be excluded since some areas show very coarse material which may not give the predicted deformation requirements. The extraction and mixing process shall guarantee that no local alluviums with unfavorable gradations will be used.

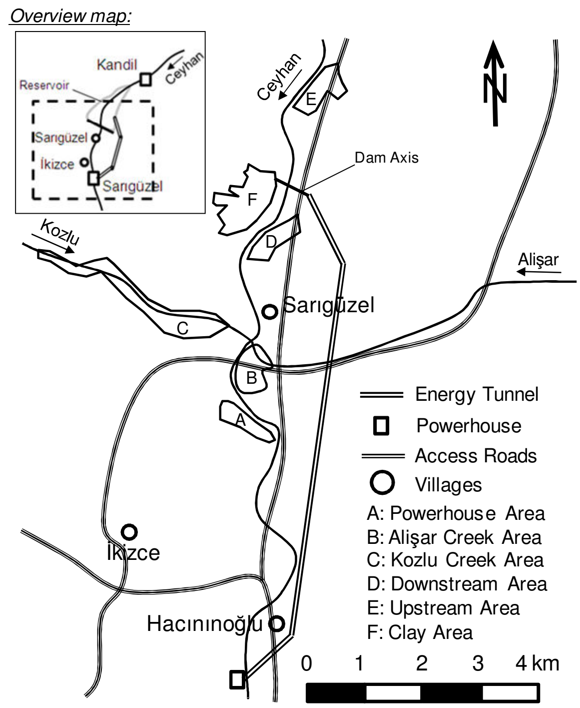

Figure 2. Sand-gravel fill (and clay) borrow areas A-E within the Sarigüzel project area

Laboratory and field trial compaction tests were carried out in order to determine the alluvium's parameters for construction conditions. Initial results are very promising, indicating a dry density of the compacted material of γ_d = 2.50 kg/cm³. For the time being, the layer height for zone 3B is most probably

60 cm applying four to eight passes, most probably six. Recent tests already indicate a deformation modulus of E_{V2} = 200 MPa on average reaching peaks of 300 MPa for a 100 cm layer. Still higher values are expected which shall be confirmed by ongoing tests. Benchmark values (Fell et al., 2005; Cruz et al., 2009) confirm these high values for sand-gravel fill materials. The elasticity moduli of construction are approximately 3-4 times higher than for typical rockfill materials (Haselsteiner & Ersoy, 2011).

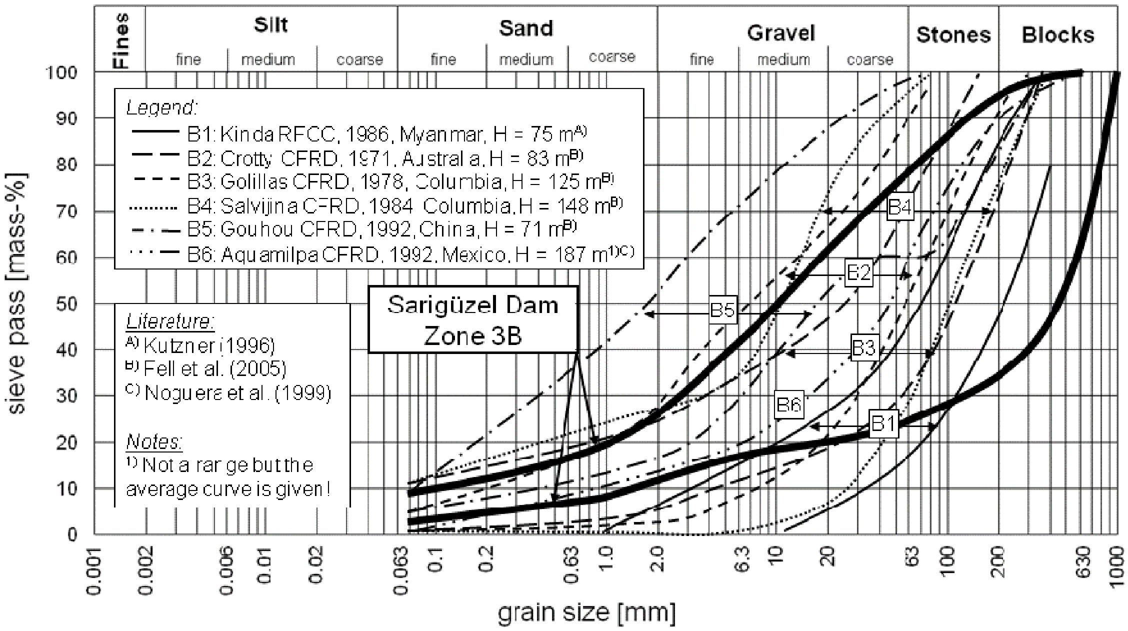

Figure 3. Sieve curve envelope for the sand-gravel fill from several locations around the dam axis (1st investigation stage)

The resulting sieve curves during the 2nd investigation stage (Figure 4) do approximately correspond to the range of the sieve curves of the 1st investigation stage (Figure 3). Additionally, the field trial test embankments resulted in good material properties. Therefore, no concerns are directed to the applicability of the available material, particularly regarding the deformation characteristics.

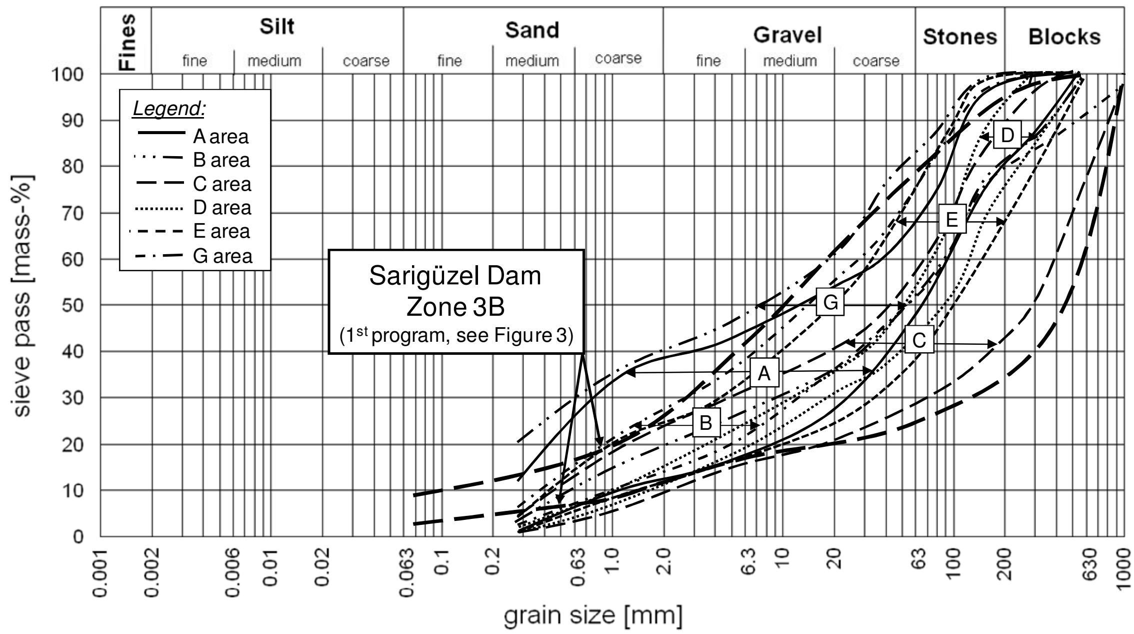

Figure 4. Sieve curve envelope for the sand-gravel fill taken from locations A, B, C, D, E and G (2nd stage investigations)

The optimum water content for compaction is still under investigation. Since the material will be extracted also underwater the natural moisture content maybe too high for immediate placement and compaction. The processing of the material will be adjusted for the results of the ongoing site trial tests which shall shed light on the range of applicable moisture content and on the deformation behavior. For the time being, the placement and compaction shall be done applying the natural moisture content of the material.

Currently, large scale triaxial tests have been agreed on and material was sent to Karlsruhe (Germany) which has a triaxial cell with a diameter of 80 cm (Bettzieche & Bieberstein, 2009). After receiving the test results the dam design will be revised again after consideration of stability and deformation aspects. Optimization is envisaged but is not guaranteed in advance, since sand-gravel fill materials show less favorable shear strength at low overburden pressures compared to rockfill materials.

Area G is not shown in Figure 2. Area F is not included in Figure 4 since its use for dam fill material is not decided.

3 DAM DESIGN DEVELOPMENT

3.1 *Clay Core Rockfill Dam*

During the different project phases, the dam type and design was changed several times and is still undergoing optimization. During the master plan the typical DSI (Turkish: Devlet Su Isleri; English: State Hydraulic Works) design for dams on moderate strong foundation was taken into consideration, which is a clay core rockfill dam (CCRF). These kind of dams are widespread in Turkey and are quite conservatively designed with slopes of V:H = 1:2.2-3.0. Since Turkey is a region prone to earthquakes this may be justified for single locations close to the Northern and Eastern Anatolia fault zone. With a conservative design the dam volumes show high values which is directly affecting costs and construction periods. For example, Akköprü Dam on the river Dalaman was finished recently after 15 years construction showing a volume of 13 Mio. m³ with a dam height of over 100 m and conservative flat slopes. This is not manageable for private companies which are dependent on profit. The second difficulty for a typical clay core rockfill dam is the availability of sufficient clay material. Since Sarigüzel is located in a mountainous region the clay borrow areas are limited.

3.2 *Concrete Face(d) Rockfill Dam*

Frequently, after private companies takeover a project from a governmental authority the feasibility studies are prepared, or if already available, are revised. The dam type is frequently changed to more efficient and economic dam type compared to CCRF, such as Concrete Faced Rockfill Dams (CFRD) and Roller Compacted Concrete Dams (RCC). Less frequently arch dams or other dam types are discussed or agreed on.

Sarigüzel Dam was also considered to be a CFRD dam assuming that enough rockfill material will be available within an economic distance of the dam axis. In this project phase, the possibility of using the large amount of alluvial deposits for the dam fill had still not been discussed. The CFRD should be partly founded on alluvial deposits. The plinth should be founded on bedrock after excavation of a limited depth of alluvium in the river bed.

3.3 *Concrete Face(d) Sand-Gravel Fill Dam*

Within a revision of the feasibility study the dam type was changed to a Concrete Face Sand-Gravel Fill Dam (CFSGD) due to the presence of a large volume of alluvium. The slopes were designed with H:V = 1.0:1.6-1.7 which is appropriate with regard to literature regarding stability and seepage control (Cruz et al., 2009). The dam volume increased compared to the former CFRD type. Seepage control was considered to be guaranteed by a L-shaped central drain layer (see Figure 5).

3.4 *Concrete Face(d) Sand-Gravel Fill Dam with Deep Excavation*

During the investigation in the course of the final design clay layers were detected within the river bed. The clay layers reach a thickness of several meters and are located deepest at 30 m depth below the river bed according to the available drilling data. This led to the decision for a deep excavation and backfilling with sand-gravel fill material as shown in Figure 6. In consideration of the expected seepage flow, upstream and downstream cut-off walls are to be used. Upstream, another cut-off wall shall connect the surface slab sealing to the underground sealing at the plinth.

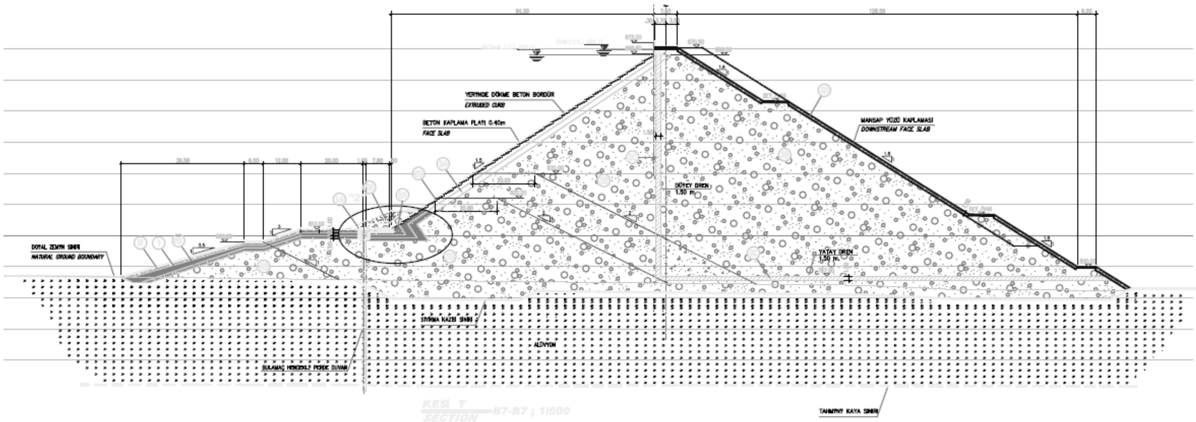

Figure 5. CFSGD with L-shaped drain founded on shallow alluvium

Since the design presented in Figure 6 still left a few critical questions open, such as the deformation of the concrete slab and the seepage control in case of cracks within the concrete face (see also Haselsteiner & Ersoy, 2011) the design is still under revision. The application of high friction rockfill at the upstream slope should have strengthened the slope in case of rapid drawdown, combining high friction at low stresses and high permeability. But, rockfill is, as mentioned above, more compressible than compacted sand-gravel fill and allows higher deformations and, therefore, is likely to lead to cracks in the slab.

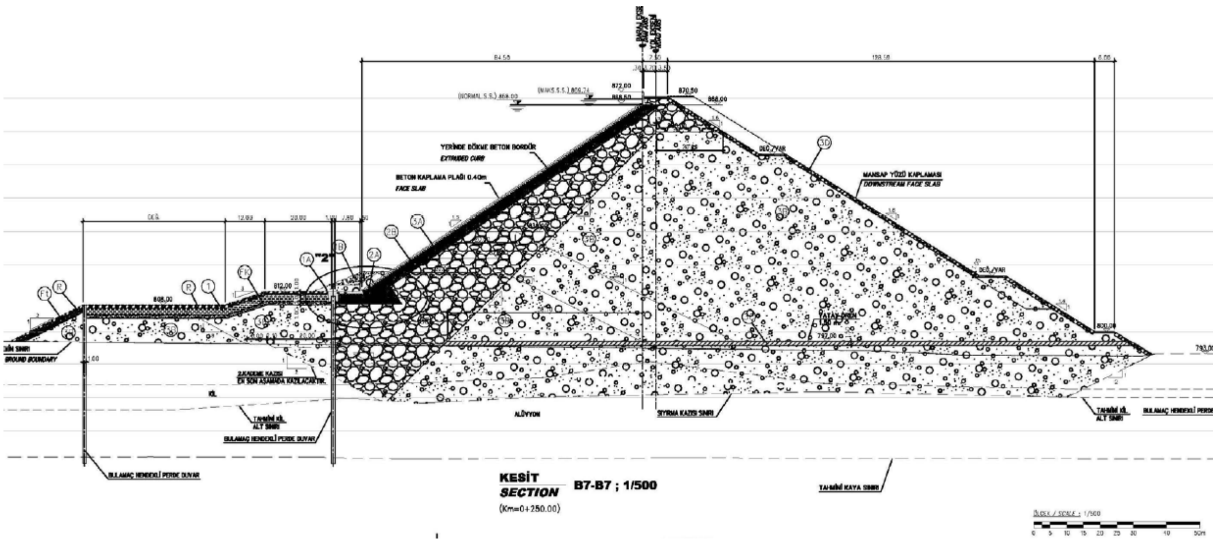

Figure 6. CFSGD with basement drain and rockfill beneath the surface slab and deep foundation

Due to the ongoing investigations and testing, a guideline dam design was prepared. This involves considering the typical dam design principles regarding stability, seepage and deformation (Figure 7). Under and close to the elements which are sensitive to deformations, low compressive and strongly compacted rockfill/sand-gravel fill shall be placed in consideration of the deformation requirements which are estimated by a 2D FEM deformation analysis.

After the field trial compaction tests are completed and the large scale triaxial test results are evaluated the design has to be reviewed and, most likely, revised. Other design options such as the application of a membrane on the surface slab are currently being discussed. The construction of the deep cut-off walls and the quality control are another issues within further discussions. Still the seepage behavior of the dam in case of cracks in the surface (Haselsteiner & Ersoy, 2011) is under investigation. For this purpose a drain toe consisting of selected rockfill material is considered to be valuable, improving both the seepage control function of the dam and the stability in terms of potential superficial sliding surfaces along the downstream slope.

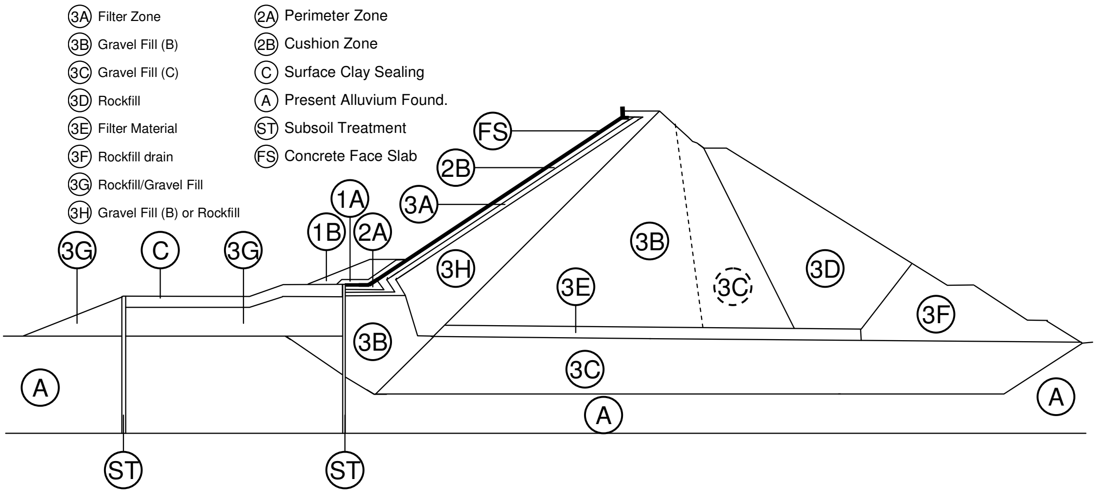

Figure 7. Guideline design for the CFSGD with basement drain, special zoning and deep foundation (adjusted after ANCOLD, 1991; ICOLD, 2005)

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