Efficient Design, Construction and Maintenance of Hydropower Plants in Turkey

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Introduction

The Turkish hydropower sector is booming. A huge number of hydro projects are under development and construction. For the design and construction of new dams and Hydroelectric Power Plants (HEPP), Turkish engineers are able to rely on the experience gained from the construction of world famous hydro projects such as the Atatürk, Berke, Deriner, Karakaya or Altinkaya Dams. But, simple and standard design prevails for small and medium hydropower projects in order to minimize costs for construction and maintenance. It is also common practice to apply a “design-as-you-go” philosophy to accelerate realization. This paper includes general information about the Turkish hydropower market and useful details for the conceptual and structural design of HEPP in Turkey with a focus on small to medium hydro projects.

1. Hydropower in Turkey

The threat of an energy crisis motivated the Turkish government to privatize the energy market in order to attract investors and owners of all kinds of businesses, particularly of Turkish origin, but also from abroad. Since Turkey is the country with the second largest hydroelectrical energy potential in Europe many private investors and owners aim for small to medium sized hydroelectric power plants. Large projects are mostly still developed by the federal water management authority DSI.

Referring to the year 2000 10,538 MW of installed capacity was provided by hydropower in Turkey. This represented a share of 40 % of the overall installed capacity and for 30 % of the whole energy production of 36.7 10^3 GWh/a. Within five years of the privatisation of the energy market installed capacity increased by 23 % to 12,941 MW and the power generation reached 42 10^3 GWh/a. This represented an increase of 13 % (Haselsteiner et al, 2008).

The Economically Feasible Electric Energy Potential (EFEP) of Turkey is estimated to range from 125 to 130 10^3 GWh/a, depending on which author is being quoted (Yüksek, 2008; Bayazit & Avci, 1997). Some authors even estimate an EFEP of 188 GWh/a (Eroglu, 2006). The technically feasible hydropower potential is considered to be over 200 10^3 GWh/a. Despite the increase in electricity generation from the present hydroelectric power sector its share in the total installed capacity is forecast to decrease from 35 % to 25 % by the year 2020. By comparison, the overall renewable energy sector is expected to hold a share of 60 % of the total energy generation (Kuzu & Ercin, 2004).

In the past the major rivers in Turkey were utilized for realizing huge hydropower projects, e.g. for the GAP-project 22 large dams (Tosun, 2008) were erected on the Euphrates and Tigris rivers in South-East Anatolia. Due to privatization recent projects tend to be located on tributary and smaller rivers in mountainous areas. It is estimated that there are more than 2,000 hydropower projects currently under development. A definitive number cannot be given as every day new projects are filed with governmental authorities. In order to utilize the huge hydroelectric potential before 2020 when critical energy supply deficits are predicted, all of the involved parties must act in concert to realize the projects as quickly as possible. Legal processes and requirements constituted by recently passed laws and regulations must be considered simultaneously. This hand-in-hand approach of authorities, privatized energy trade companies, owners, investors, consultants and designers will pave the way to the realization of hydropower projects in Turkey.

2. Design and Layout Considerations

2.1 General

The layout of typical small to medium HEPPs in Turkey is kept quite simple and reliable in order to optimize construction costs and maintenance efforts, considering functionality and durability. For small and medium hydropower plants in particular, “design as you go” is both common and accredited practice.

The common layout of small and medium HEPPs in Turkey contains the following structures:

- Diversion weir or dam with spillway (and reservoir)
- Sedimentation basin / facilities
- Conveyance channels and tunnels
- Inlet structure, penstock and/or surge tank
- Penstock
- Powerhouse
- Downstream channel

Major hydropower plants, such as Atatürk Dam (earth-/rockfill), Berke Dam (concrete arch dam) or Keban Dam (concrete gravity) (see Fig. 1) will always require an individual and unique design and layout which is not easy to categorize or to “design as you go” as can be done with small to medium power plants. For example, the plunge pool at Berke spillway consists of another double-curvature arch dam of 50 m height which definitively does not count to engineering standard solutions.

In order to optimize energy generation and selling benefits reservoirs are placed upstream of anticipated hydropower projects. Large reservoirs upstream of HEPPs are usually built with a capacity of 0.5 to 1.0 of the average annual flow volume. By contrast, reservoirs for smaller downstream projects are constructed in order to balance the inflow of sub catchments. The ability to balance inflows is usually very limited at the downstream reservoirs due to their limited capacity. Mostly, small regulator weirs divert the inflow to the water conveyance structures. Where only small structures are anticipated sedimentation entry has to be expected and appropriate sedimentation traps/basins have to be installed. At huge reservoirs usually the sedimentation is covered by the dead storage volume. However, problems can arise as soon as outlet facilities such as bottom outlets get blocked.

As is shown in Fig. 1 the dam structures of small and medium hydropower plants can reach considerable heights, e.g. Kemer gravity dam with a dam height of 180 m and a capacity of 48 MW installed. Therefore, requirements regarding design and stability aspects have to be equal for small and large hydropower plants. Naturally the costs for investigations and engineering take a higher percentage of the total project cost if the project volume is smaller. Consequently, for smaller HEPPs the engineering work is often done by applying standard layouts, design and assumptions, although the anticipated structures are often as challenging as major hydropower projects.

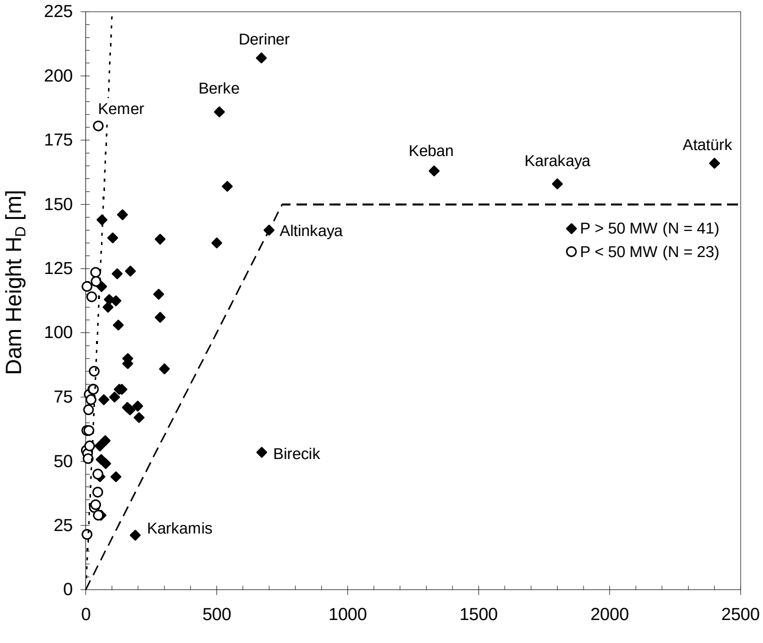

Fig. 1: Installed capacity vs. dam height for 64 selected major dams in Turkey (Source: DSI, 2002)

2.2 Earthquake threat in Turkey

Of major concern to civil and geotechnical engineering is the inherent earthquake threat. Although losses of life are always related to the failure of buildings, dams also suffer serious damage from earthquakes as reported in Tosun (2008). Both the fault zone and the earthquake risk in Turkey belong to the best understood in the world since many (hydropower related) studies have been carried out. Complete earthquake risk maps have been prepared and are provided by governmental organizations. These maps provide a first indication of applicable Peak Ground Acceleration (PGA) values. The accuracy of these earthquake maps is considered to be sufficient to get a first estimation for the PGA values for projects in the pre-feasibility or feasibility stages. Nevertheless, a seismic risk assessment is often prepared for larger projects in order to determine the design load values for the dam and the powerhouse more precisely in order to confirm and optimize the initial design.

For small and medium hydropower plants and related structures the earthquake impact is often considered by applying according PGA values within a pseudo-static stability analysis. For higher dams and dams located in Earthquake Hazard Zone I dynamic FEM analyses are sometimes performed to check the deformation behaviour with respect to the design criteria. Common practice in Turkey is to apply full horizontal PGA values within the pseudo-static analysis.

Just recently Turkish engineers have started to prepare an earthquake risk assessment for some major dams, introducing four hazard classes. This risk classification is the result of evaluating several aspects including the PGA, the dam height, the reservoir capacity, evacuation options and potential damage (Tosun, 2008).

2.3 Site Investigations and Laboratory Testing, Design Work and Stability Analysis

For large hydropower plants the geotechnical investigation programme is carried out precisely to reduce the risk for involved project partners. Site investigations are usually performed including an adequate number of boreholes with an adequate depth, water pressure tests and sometimes other methods, e.g., geophysical or in-borehole tests. For small to medium hydro projects, laboratory tests are mostly performed only for the detailed design phase. Preliminary design stages usually apply carefully estimated literature values as design parameters which are subsequently confirmed by site investigations, laboratory tests or site trial tests. If the all design and construction work is done in-house, the contracting of an independent checking consultant is recommended in order to optimize and confirm the design assumptions and the design itself.

Experience shows that the complexity of the stability analysis performed including seepage, static and stress-deformation analysis mainly increases with the project size. Also, the participation of foreign investors or shareholders usually increases the outline of the required stability analysis and design work in order to give transparency to the design work and assumptions. For small to medium hydropower plants a more or less practice-orientated approach according to the requirements of the General Directorate of State Hydraulic Works (DSI), with only compulsory and required works carried out and filed at the responsible organizations. In this case the involvement of foreign shareholder or other foreign investors increases the effort required to satisfy the additional requirements requested by lender’s engineers or checking consultants.

Usually the design and stability analyses and according requirements set by DSI are based on international regulations in combination with Turkish experience and design habits. State-of-the-art in Turkey is mainly defined by publications released and standards applied within the United States (USACE, USBR, ASTM, …) and ICOLD papers. Most of these regulations, standards and manuals are available freely via the internet or can be purchased from the corresponding organizations.

2.4 Load Factor for HEPP in Turkey, Energy Generation and Financial Aspects

The load factor in Turkey (5 to 20 % being equal to 20 to70 days) is chosen relatively high compared to Middle and Western European conditions. In Germany for example a load factor of 25 % (90 days) is used for the preliminary design and layout of a small HEPP. For large HEPP and projects an accurate and iterative cost-benefit-analysis is usually carried out in order to find the economically optimal load factor. For small to medium HEPP the optimization process is carried out within the usual design process (feasibility design => final design => detail design). In this case the application of a different load factor is critically dependent on the bids of the E&M equipment supplier, the whole bid of the contractor and whether the application of one unit more or less is economically justified. In the past such a consideration has led to the complete design change of a structure such as a power tunnel, penstock and powerhouse on many projects. Mostly, the feasibility studies were outdated and new boundary conditions had to be considered.

The annual energy generation is dependent on the head, the (design) discharge and the factual degree of efficiency of the whole HEPP. In Fig. 2 values for the annual energy generation of 62 HEPP are shown in relation to the values for the installed capacity. As a rule of thumb the annual energy generation can be taken as $A = \alpha \cdot P$

with $\alpha \approx 3.75$. The annual energy generation has to be considered in [GWh/a] and the installed capacity in [MW]. As Fig. 2 shows the factor α ranges between 2.3 and 4.5.

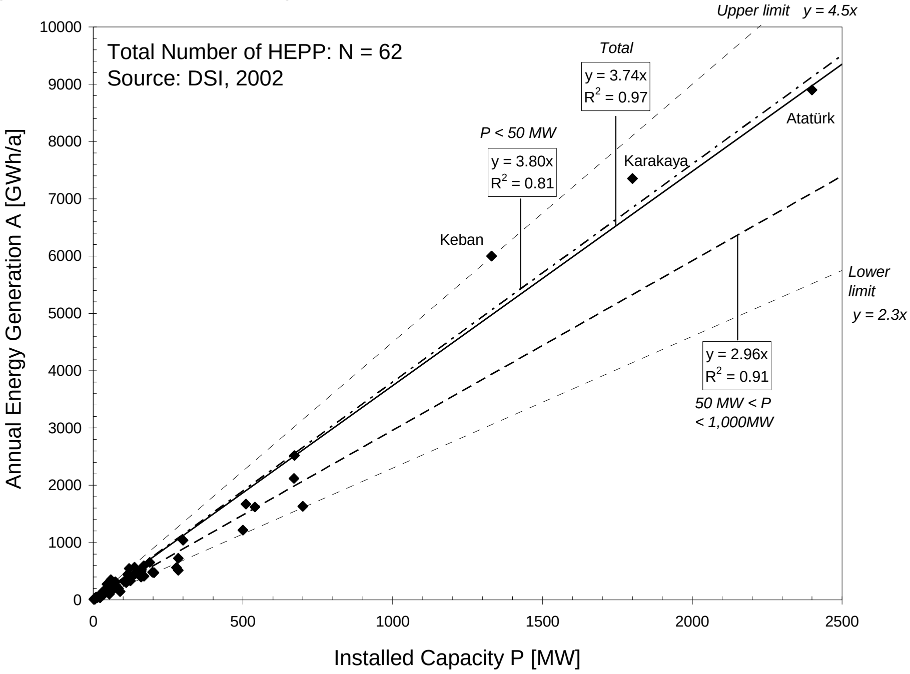

Fig. 2: Installed capacity vs. annual energy generation for 62 selected major dams in Turkey (Source: DSI, 2002)

Fig. 3 illustrates the effects of the load factor in respect of the annual energy generation. The data were derived from a typical load curve of a HEPP in the South-East of Turkey (Fig. 3, left). The annual production is sensitive to the applied hydrological load curves as can also be observed in Fig. 3 (left diagram). With increasing load factor the relative variation of power generation decreases. If the load factor reaches 100 % the variation of the load curve has no influence since the design discharge corresponds to the long-term mean low water discharge which is probabilistically guaranteed throughout the whole year.

In consideration of the relative annual energy production and the relative design discharge of the example project it is clearly shown that a decrease of the load factor and design discharge leads to an increase of the annual energy production. This increase is strong for low load factors and high design discharges. As indicated in Fig. 3 (right) a doubling of the design discharge leads to an increase in annual energy generation of 16 %. An increase of the design discharge Q_D up to three times the mean discharge increases the annual energy production by 18 %. An increase from $Q_D = 2 \cdot Q_{Mean}$ to $Q_D = 3 \cdot Q_{Mean}$ increases the generation only by 2 %. This increase has to be compared to the necessary costs to alter the turbines, the powerhouse, the channels, the tunnels and the penstock to accommodate the increased design discharge. For the given example, the increase of the design discharge from $Q_D = 1.0 \cdot Q_{Mean}$ to $Q_D = 2.0 \cdot Q_{Mean}$ and $Q_D = 3.0 \cdot Q_{Mean}$ corresponds to a decrease of the load factor LF = 31.5 % to LF = 5.5 % and LF = 1.0 %.

The choice of the load factor has to be assessed when considering the O&M costs and the whole capital expenditure, including financing and construction costs. On average, the construction costs for small to medium hydropower plants can be estimated by applying 1.0 million € per installed MW (see Haselsteiner et al, 2009). This is also a good indication for large hydropower plants, although the variation of costs is considerable dependent on the particular projects. For example, the costs for Atatürk Dam Project cost 1,000 million € for an installed capacity of 2,400 MW. Thus, the investment costs were 0.420 Mio. € per installed MW. The reader may keep in mind that the source of data for the overall costs of the Atatürk Dam have been neither checked nor approved.

Due to the current financial crisis contractors have been recently forced to grant discounts of 25% or more which can considerably decrease the construction cost. For O&M costs (including staff) 2.5% to 3.0% of the total investment is a rough but nevertheless reliable value for estimating costs or preparing cost calculations at the feasibility stage (Haselsteiner et al, 2009). Financial models and prognoses critically depend on the fluctuation of the energy selling price which is linked to the global economy and particularly to the oil price. Although the applied financial models consider several parameters such as the global oil price trend, inflation tendency, interest rate trend and regular expenditures for rehabilitation, it is well-known that some of these parameters are almost unpredictable. For example the oil price reacts sensitively to global market trends influenced by economical aspects and political decisions. Therefore, models using “only” simple analytical trends verified by benchmark data from the past provide reliable results which do not differ considerably from those obtained by high-end, elaborate financial models. The sensitivity and vulnerability of the global economy as revealed to the world in the form of the global finance crisis in 2009 makes it quite difficult to predict the energy selling price in Turkey with any confidence. On the other hand, a rising demand and the anticipated insufficiency of energy supply in Turkey within the next few years are facts that may suggest an increase of the selling price. Owners and investors should not worry about the profitability of their hydro projects in Turkey now and in the future.

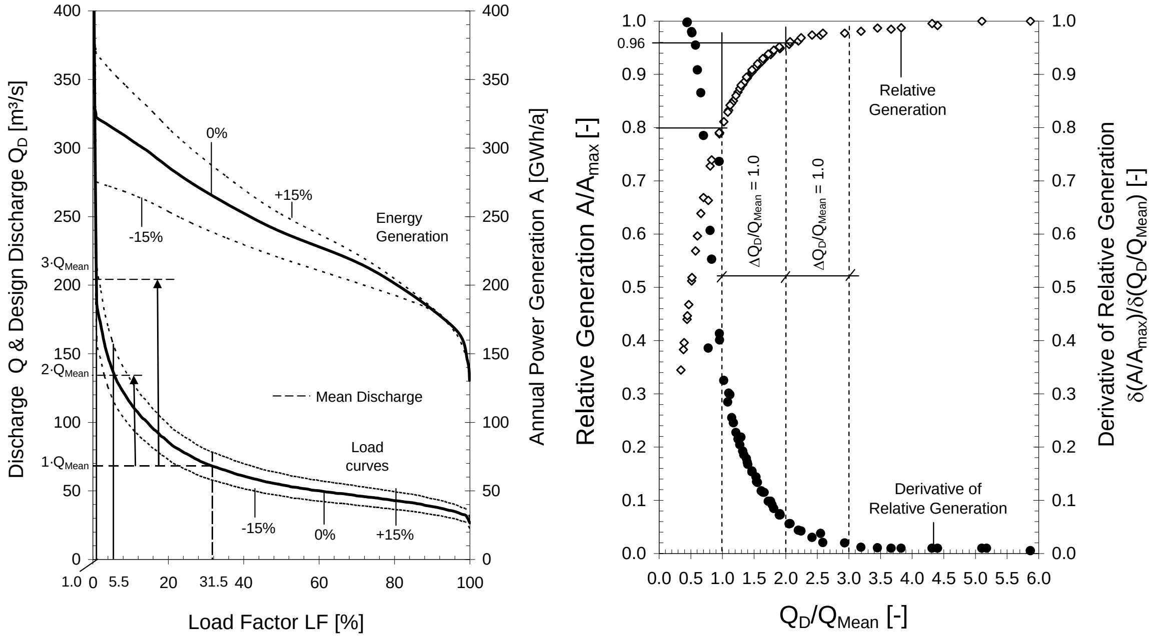

Fig. 3: Loading curves and annual energy generation (left) and relative annual energy production in relation to the relative discharge (right) (conditions taken from a Turkish HEPP under development)

2.5 Hydraulic Structures

Hydraulic conveyance structures such as open (or covered) channels or tunnels are constructed by using cut & cover, side-cut or typical tunnelling techniques. Such structures are usually designed to minimize the effort to operate and maintain the structure by constructing access roads and also minimizing the construction costs. If possible valley crossings and the application of siphon culverts are avoided as maintenance and construction are relatively difficult compared to open channels.

Open channels are usually possible where relatively low discharges are to be diverted to the powerhouse. In such a case the excavation works for channels are more economical than the excavation of tunnels. In areas where rockfall or landslides are expected the channels are covered by concrete slabs and/or protected by retaining walls. In order to guarantee easy access to the structures for rehabilitation and maintenance purposes access roads are usually located along the channels.

Geological investigations, particularly for long tunnels, are most commonly limited to the entrance and exit areas of the main tunnel. Usually a geological analysis in form of a desk study is performed for the feasibility design. Also for smaller open cut channels geological investigations are usually limited to a geological desk study and the design is done “as you go”, including additional protection measures such as shotcrete or anchoring for affected slopes.

Based on Turkish experience and according to international practice in time scheduling, the excavation progress is usually estimated at 3.0-6.0 m/d and 4.5 m/d on average per work face. Tunnel Boring Machines (TBMs) are rarely used for HEPP projects due to the limited tunnel length required and excavation area. Normally, the excavation is done by blasting. For cases where weak schists or similar rocks are encountered, mechanical breaking technology applying pneumatic hammers is preferred. Rock tunnels are usually lined to minimise head loss. The rate of lining achieved is usually considered to be 4 m/d per work face.

Similar to the applied treatment of channel construction, tunnel design and construction as well is changed “as you go” with respect to the actual rock conditions encountered. According to the expected variation in rock conditions different classes of protection measures were designed, mainly applying the NATM technique. For very weak and fractured rock conditions steel beams, reinforcement and shotcrete are designed according to international accredited design criteria (Bieniawski, 1979, Grimstad & Barton, 1993) which consider very low span widths and relatively thick shotcrete protection layers.

Common dam engineering practice is to combine weirs and spillways with concrete dams and to locate them separately for embankment dams. Where it is possible, aerated chutes are combined with flip buckets which concentrate the water jet to the centre of the former river bed. Experience concerning flip buckets and aerated chutes has evolved so far that physical models are only applied for cases where particular design criteria needs to be investigated and determined, e.g. the quality of energy dissipation or expected scouring. According to international practice the discharge capacity of spillways at high embankment dams is designed for the Peak Mean Flow (PMF). Regulator weirs have a capacity which is equal to the 100 year flood event as a minimum. Concrete gravity dams, including RCC dams, are designed for an approx. 10,000 year flood event accepting that overtopping will occur but assuming that a complete failure will not occur.

Elaborate sedimentation analyses are usually not carried out, particularly for small to medium HEPPs. Sedimentation basins or traps are usually designed according to the requirements of the turbine supplier to limit the maximum transported grain size based on of rough assumptions of the potential bed load. An estimation of the annual sedimentation volume is mostly performed by applying average annual bed loads per square kilometre of catchment area. The efficiency and functionality of sedimentation basins depends on the hydraulic conditions which can still only be evaluated correctly by setting up expensive physical models, maybe in combination with hydraulic mathematical 3D modelling. These structures are usually designed according to international engineering practice adjusting the length of the basin and placing flush gates at the end. A verification of the functionality is usually not part of the scope of design; therefore, excessive sedimentation is often the subject of ongoing maintenance and of regular remediation works in case damage at concrete structures or hydro-mechanical equipment occurs.

Surge impacts in pressure tunnels and penstocks are dealt with by adequate surge tanks or head ponds. At smaller HEPPs, a conveyance channel usually leads to the head pond which doubles as the intake structure to the penstock. A spillway mostly designed as an ogee-shaped overflow section is usually constructed at the head pond that relieves higher water levels generated by surge in the case of rapid shutdowns. In general, for small HEPPs the theoretical surge height is limited to a few meters and consequently the deduction of limited discharges via the head pond's spillway is technically and economically favourable.

2.6 Dam Structures and Weirs

Due to the considerable earthquake threat in Turkey the preferred dam types are embankment dams due to their favorable deformation behavior during earthquakes. Over 90% of the studied dam projects in Fig. 4 are embankment fill dams, of which 1/3 are rockfill and 2/3 earthfill dams according to data provided by DSI (dated 2002). Very few dams in Turkey are designed as concrete dams. But recently mainly smaller dams with a height of less than approximately 50 m are designed as concrete dams preferring the RCC technique due to the rapid construction progress and the cheap unit price for roller compacted concrete.

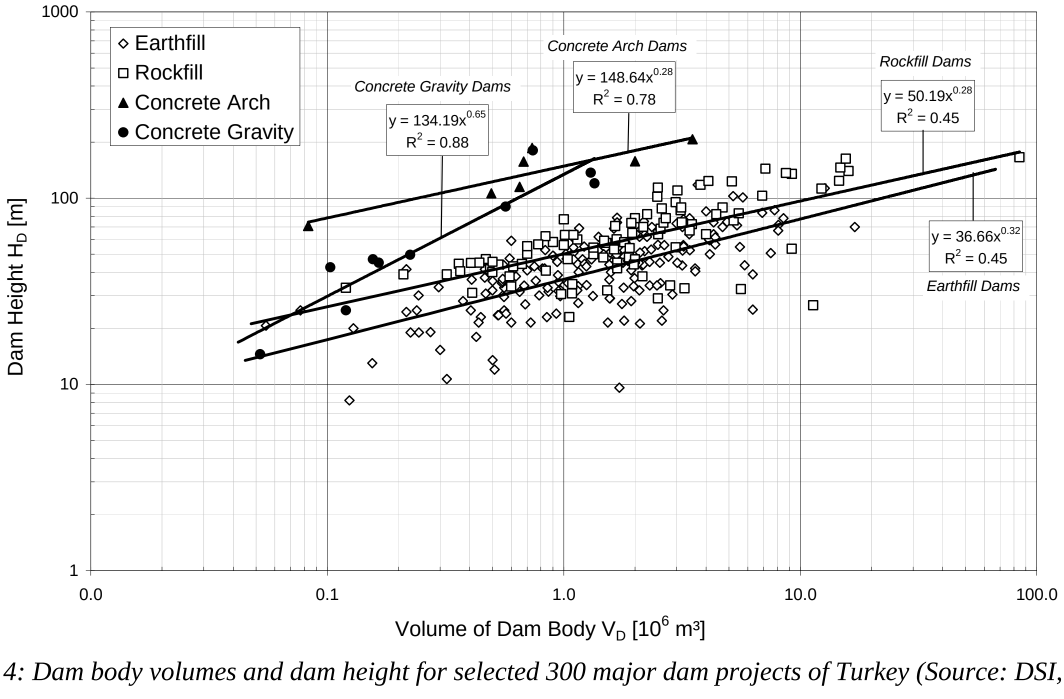

Fig. 4: Dam body volumes and dam height for selected 300 major dam projects of Turkey (Source: DSI, 2002)

If appropriate material (natural clay core) is available, a rockfill dam with clay core is one of the favoured dam types to withstand earthquake impact and to provide an economical design. Nevertheless, the design criteria in the early design stages are chosen in a conservative way and always according to accredited design standards in order to cover uncertainties regarding geology and the properties of the fill material. Furthermore, the application of relatively flat slopes, as indicated in the sketch of a clay core rockfill dam in Fig. 5, confirms that the first steps to an optimal design are done carefully in order to not underestimate earthquake impact and costs. Usually flat slopes of clay core rockfill dams eliminate any discussion about earthquake threat concerning stability and deformation if the subsoil conditions are adequately strong. Therefore, elaborate dynamic earthquake studies determining deformations are not necessarily required but are considered to be covered by pseudo-static considerations and by many benchmark projects worldwide and in Turkey. This is well-known practice for rockfill dams with a height smaller than more or less 50 m.

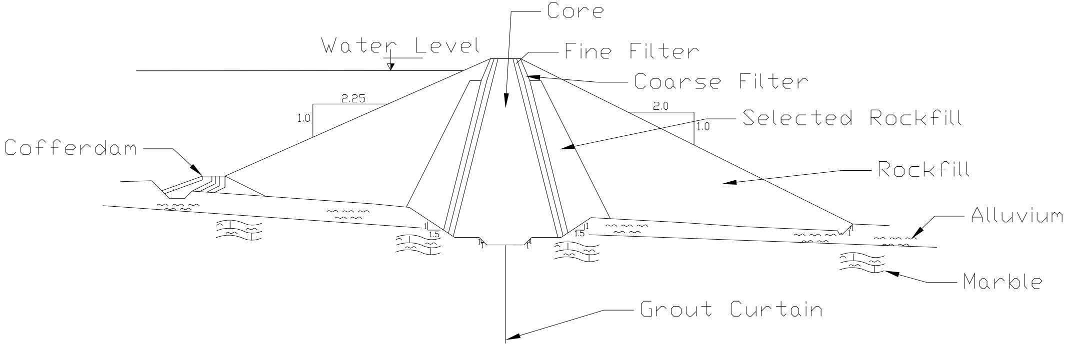

Fig. 5: Typical section of a clay core rockfill dam at feasibility stage (taken from a medium HEPP project in development)

New developments in Turkish dam construction practice tend to use more CFRD dam types based on reliable experience gained in other countries, particularly in respect to the seismic behaviour of these dam types (Tosun et al, 2007). Furthermore, in most Turkish project areas close to mountains (upper river regions) potential rock quarries usually exist close to sites and should contain enough and adequate material for rock as well as for concrete aggregates.

Nevertheless, current trends also head towards the application of RCC dams if lower earthquake impact is expected and adequate foundation rock is close to the ground surface. Some advantages of RCC compared to a CFRD may be found in a faster construction progress, no need for a separated spillway chute and less works for diversion or headrace tunnels. Also, the applicable design flood may be reduced from PMF or similar to lower flood discharges since a concrete gravity dam may sustain limited overtopping as mentioned before. RCC is not only used for main concrete gravity dams but also for upstream coffer dams. As soon as the first diversion stage is completed the upstream RCC cofferdam can be built under dry conditions. Furthermore, the diversion design flood can be decreased from a 25-year to a 5-year flood since overtopping and freeboard requirements are not as strict as they are for a sensitive embankment dam.

Particularly for small hydropower plants, regulator weirs are used to feed the conveyance channels or tunnels. The weirs themselves are usually not controlled by corresponding facilities such as gates. The reservoir level is controlled by the facilities at the energy tunnel or at the head pond structure. If applicable and necessary the weirs are fitted with radial gates. An example for a typical weir section at feasibility stage is shown in Fig. 6. In this example optimization can be carried out with regard to the effectiveness of the energy dissipation in the stilling basin and the design of the base slab for uplift stability.

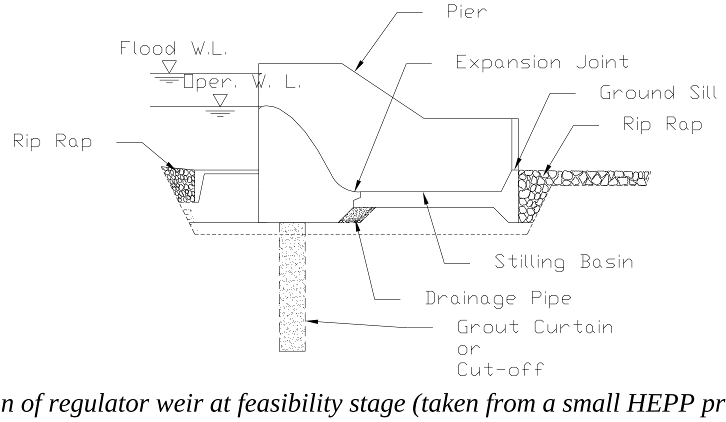

Fig. 6: Typical section of regulator weir at feasibility stage (taken from a small HEPP project in development)

2.7. Electrical and Hydromechanical Equipment

As already stated for the civil structures of a HEPP, the powerhouse and penstock design is carried out according to accredited international design standards and practice. The design at feasibility stage is kept simple and reliable as shown in Fig. 7. The penstocks are preferably placed on the slope surface in order to minimize construction costs.

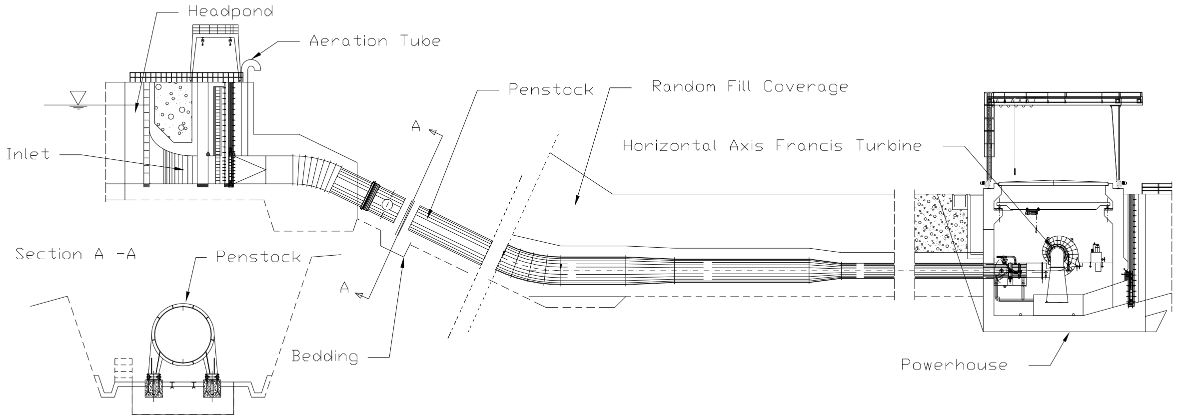

Fig. 7: Typical section of head pond, penstock and powerhouse at feasibility stage (taken from a medium HEPP project in development)

As already discussed in Haselsteiner et al (2009), in the early 1990s Turkish owners started to order E&M equipment for their HEPPs in China. This trend continued as the price of Chinese equipment was a mere half or even third of the cost of equipment from other well-known suppliers. Although in the beginning only the main parts of the equipment such as turbine units were ordered in China the trend continues towards buying wholesale E&M parts, including sometimes even the design work for the powerhouse. Naturally, contractual aspects and quality control are of major interest if Chinese equipment is to be installed. A few years ago only the equipment for small to medium HEPPs were ordered in China. Nowadays China is also supplying equipment for large HEPPs in Turkey. This is evidence of the quality of Chinese equipment and that it must have improved over recent decades.

Still some investors and owners prefer E&M equipment provided by renowned western companies accepting their higher prices. This equipment implies better quality and is reflected in a higher coefficient of efficiency for turbines and generators. The additional costs may be amortized within the considered operation period by a higher annual energy production compared to a plant fabricated in China. Additionally, the renowned E&M suppliers have granted considerable discounts on prices and consequently the price margin between Eastern and Western offers should be not as striking as it was a few years ago. Due to the global market situation it is not easy to receive offers from Western companies for smaller projects. Therefore, Chinese or Indian manufacturers are sometimes the only alternative to obtain all the required components within an acceptable deliverance period. Currently, delivery takes 12 to 24 months depending on the turbine type and special contractual agreements. In individual cases also shorter periods could be guaranteed by the suppliers.

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Mr. Ünal Mesci graduated from Middle East Technical University (METU) in 1970/72 as Civil Engineer (B.Sc. 1970, and M.Sc. 1972). Additionally, he had another M.Sc. Degree in 1980 from Southampton University of England (Engineering Economics). He had worked in State Hydraulic Works (DSİ) in Turkey from 1972 to 1997 at several administrative positions. He was Regional Director in Samsun (from 1986 to 1997) where he had responsibility to develop hydro projects of the basin areas of Yeşilırmak and Kızılırmak rivers. In this period Altınkaya (700 MW), Derbent (56 MW) and Ataköy (90 MW) had been constructed. Besides, many irrigation projects, drinking water supply project and flood control projects were completed. Now, Mr. Mesci is general manager of Nisan Elektromechanical and Electrical A.Ş. in Ankara. The total Energy Project evaluated by Nisan Company is about 142 MW with an annual production of about 510 GWh/a.