2.3 High pressure power plants

[E. Puerer, G. Goekler]

2.3.1 Introduction

2.3.1.1 Purpose and demand

2.3.1.1.1 Energy production

In the field of electricity supply by hydropower, high pressure power plants (HPPP) [91Mos] generally fulfill the function of supplying

- peak load,
- power and frequency control,
- rapidly available power and energy.

The reason for this is that high pressure power plants per definition dispose of a high energy head up to 1000 m (in some cases even more) and of a reservoir for the storage of significant energy production. The reservoirs with their storage capacity allow to react instantly to the power demand in the grid and thus provide the necessary power and frequency control and reserves in case of the break down of another power plant connected to the grid. The annual runoff in the area of a HPPP and therefore also its annual energy production (in [kWh]) are lower than for other types of power plants, but their level of power production is higher due to great gross heads. As the above mentioned types of energy achieve higher market prices than base load, high pressure power plants can be run economically despite their higher investment and energy costs (see Sect. 2.3.6). High pressure power plants are, therefore, particularly suitable for covering short-term peak-demand (see Fig. 2.3.1). With the liberalization of the energy market, power and frequency control – i.e. adapting power production to the constantly fluctuating demand in a grid which exceeds or falls short of forecasts – has become more and more important and is one of the main functions that high pressure power plants and pump storage schemes (see Sect. 2.6) fulfill.

The annual generation hours of high pressure power plants amount to approximately 1000 to 1500 h, while low head power plants at rivers (river power plants) have more than 8500 generation hours a year.

2.3.1.1.2 Multi purpose schemes

In many regions high pressure power plants with their large reservoirs are also used for other purposes [03Gie] which are equally – if not even more – important than energy production, such as

- drinking water supply,
- irrigation,
- regime management of a catchment area,
- flood control or
- recreation.

In terms of power production it is important if the water required for these competing purposes is

- taken out of the reservoir, i.e. before power production, or
- taken out after energy production.

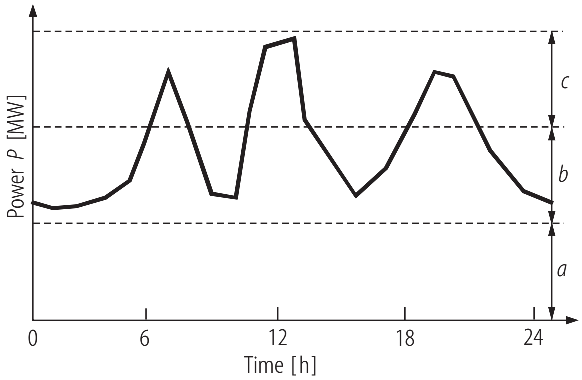


Fig. 2.3.1. Power demand in a grid and coverage by river power plants, thermal power plants, medium head power plants and high pressure power plants. *(a)* Base Load: river power plants, nuclear, thermal; *(b)* Medium Load: hydro power plants, thermal (capable for power control); *(c)* Peak Load: HPPP, gas turbines.

If water from the reservoir is used, the intake of water for power production will be reduced. This, however, does not have a negative impact on the regulation of power production of a high pressure power plant. If water is reused after energy production for the above mentioned purposes, the annual energy production will remain constant while the regulation of power production of a high pressure power plant can be substantially affected. In this case functions such as the supply of drinking water, irrigation or regime management of a catchment area will have priority over energy production and the energy produced by the high pressure power plant will therefore be of inferior quality.

Reservoirs of high pressure power plants can also considerably improve safety against the danger of flooding [02Gos]. Due to their often enormous surface, they show a good water retention ability, are able to cut a peak flood discharge and reduce the danger of flooding. For this reason it is necessary that the reservoir of a high pressure power plant disposes of storage capacity for periods of possible flood discharge (see Sect. 2.3.7.3). Free storage capacity can be a by-product from everyday operations or a result of special management of the reservoir. Keeping these storage capacities free is a matter of public interest which may however have a negative impact on energy production and cost efficiency of the high pressure power plant. It is therefore necessary to make agreements with the government before or after construction of such a HPPP.

2.3.1.2 Layout and design

The design of a high pressure power plant [91Mos] is determined by

- technical criteria such as
  - catchment area inflow and
  - topographic as well as geological conditions to build a dam and a reservoir;
- requirements of
  - the electric grid and
  - the energy market.

2.3.1.2.1 Catchment area inflow, water intake altitude

In catchment areas with heavy precipitation it is necessary to determine the optimum altitude of the water inlets for a diversion of the runoff. First, a prominent level difference in a river or catchment area with big gradients is used as a starting point to identify a topographically suitable location for a powerhouse. Then the levels of energy production are calculated for different water intake altitudes, respectively. The higher the altitude of a water intake, the smaller the annual runoff[1] and the greater the head. The lower the altitude of a water intake, the bigger the annual runoff and the smaller the head. By calculating the potential levels of energy production for different water intake altitudes, the optimum water intake altitude can be determined (see Fig. 2.3.2). On the basis of the theoretically optimum water intake altitude, the technically feasible or best location for inlet structures or reservoir is assessed by taking into consideration

- the topographic features of the area at the optimum water intake altitude or reservoir (dam) and
- the geological features influencing inlet structures, reservoir (dam) and headrace structure.

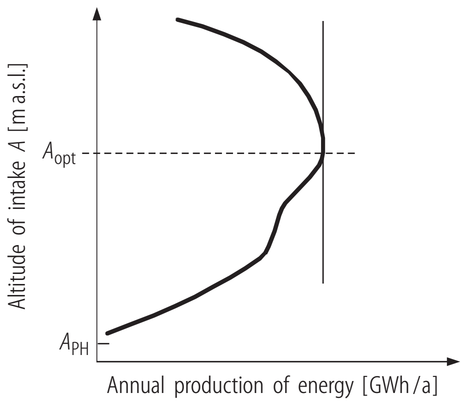


Fig. 2.3.2. Evaluation of the optimum altitude for the intake A_{opt}, with A_{PH} the altitude of the powerhouse.

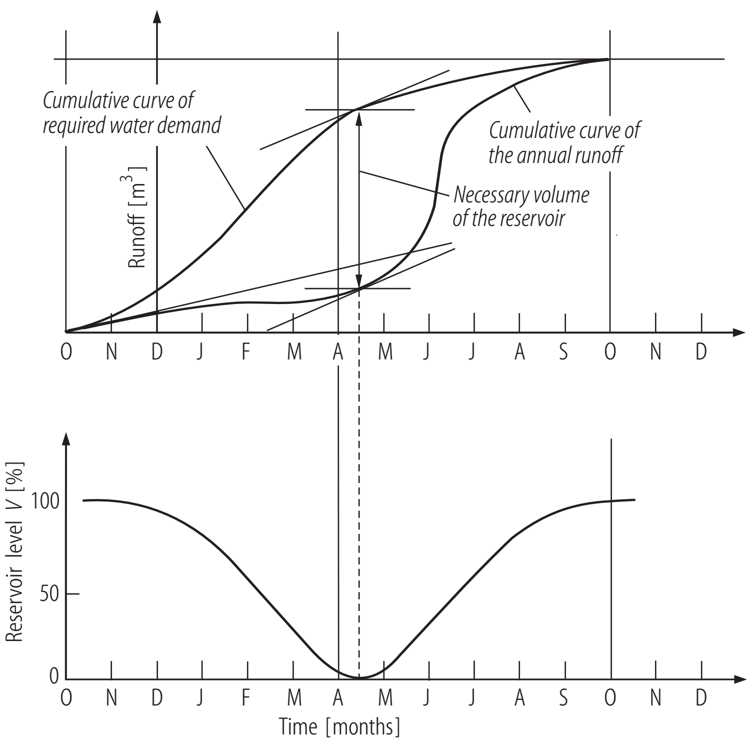


Fig. 2.3.3. Determination of the necessary storage volume of the reservoir.

[1]) Runoff: total annual runoff at a specified point of the river.

2.3.1.2.2 Reservoir volume, storage capacity

The volume of a seasonal storage reservoir required in theory can be calculated on the basis of the line of total utilizable (or line of added-up) runoff and the line of total intended water use as shown in Fig. 2.3.3. As seasonal storage reservoirs are characterized by the fact that the total utilized runoff equals the total inflow to the reservoir, the necessary storage volume is determined by the maximum difference between annual inflow and annual utilized runoff. Apart from that topographic as well as geological conditions will always play an important role when deciding on the size of a reservoir.

In European alpine regions reservoirs are filled at the end of summer and emptied at the end of winter. In regions with dry seasons and wet seasons, reservoirs are filled at the end of the wet season and emptied at the end of the dry season. As electricity is needed in summer or wet seasons as well to supply peak load energy and power and frequency control, the storage volume does generally not equal the annual runoff. As a rule of thumb it can be assumed that the storage volume is approximately 2/3 of the annual runoff.

When determining the storage volume of a multiseasonal storage reservoir, additional factors have to be taken into account. In regions with heavily fluctuating annual runoff, water from wet years can be stored to be used in dry years by building a reservoir of adequate size. The construction of multiseasonal storage reservoirs can also be influenced by economic factors. Dealing with options (energy quotas on the stock exchange), for example, could require that the equivalent amount of energy is stored or remains stored in a reservoir; thereby the storage volume of the reservoir would be influenced by economic considerations.

The size of a weekly storage reservoir is calculated according to the same criteria as used for seasonal storage reservoirs. In general the runoff of one week (Monday to Sunday) is stored in order to be used for producing energy for periods of peak consumption and high rates on workdays (for instance Monday to Friday).

2.3.1.2.3 Determination of capacity of a HPPP

In order to solve the complex issue of determining the power capacity of a high pressure power plant, economic aspects including the demand for electricity as well as the requirements of the grid and the energy market need to be taken into consideration. If the HPPP is a peak load power plant, the annual runoff Q in [m³/a] and the intended annual generating hours (e. g. t_{gen} = 1000 h/a) can be used to calculate the rated discharge $q_A = Q/(t_{gen} \times 3600)$ in [m³/s], and subsequently also the power P in [W],

$$P = \rho \cdot g \cdot \eta \cdot H \cdot q_A \,,$$

where H is the gross head in [m] and η [-] the overall efficiency of the HPPP.

2.3.1.2.4 Efficiency of HPPPs

The total efficiency η of HPPPs [91Mos] is calculated on the basis of the hydraulic losses of the various components as well as the efficiencies of the turbine connected to the generator and the electrical components until the electricity is fed into the grid (efficiency of the transformer 0.995, bus bar etc.). Efficiencies of turbines (see Sect. 2.7) and generators amount to 0.90-0.92 and approximately 0.98, respectively. Hydraulic losses of the components of a high pressure power plant and their impact on the total efficiency are determined by the individual design of these components.

As the hydraulic friction losses of the power conduit have an influence on total efficiency, the design of the power conduit needs to be optimized. This optimization process can be based on approximate values of the velocity of free surface flow tunnels (see Sect. 2.3.4.2) and pressure tunnels (see Sect. 2.3.4.3) with concrete lining, which should range from 1.0 to 2.0 m/s and from 2.0 to 4.0 m/s, respectively, as well as approximate values of the velocity of steel lined pressure tunnels and penstocks, which should vary between 3.0 and 5.0 m/s (maximum 7 m/s) (see Sect. 2.3.4). Construction costs, hydraulic friction losses and present values of losses are calculated for different diameters of the water conduit and then compared.

With increasing diameter of a water conduit, construction costs rise and the present value of losses declines. Figure 2.3.4 illustrates the determination of the optimum diameter of a power conduit.

It can be assumed as a rule of thumb that friction losses of water conduits are between 3 and 5% of the gross head. Considering the above mentioned efficiencies for turbines and generators and the friction losses for intake structure, valves, penstock manifolds, etc., total efficiencies η for high pressure power plants will equal approx. 0.80-0.85 for full load [03Gie].

2.3.2 Types of high pressure power plants

2.3.2.1 General aspects

High pressure power plants are – except from power plants at the toe of a dam (run-of-river mode) – generally operated as diversion power plants. Their major components are

- dam with reservoir or weir,
- head race system and surge chamber,
- powerhouse or underground powerhouse,
- tailrace structure with or without surge chamber and
- tailrace balancing reservoir.

High pressure power plants with reservoir fully benefit from the features of high pressure power plants detailed in Sect. 2.3.1.1.1. If it is not possible to build a reservoir, the high pressure power plant needs to be equipped with a desilting structure and can only be operated like a river power plant (see Sect. 2.2).

2.3.2.2 HPPP with reservoir

The most impressive component of a high pressure power plant is generally the reservoir which is formed by a dam (see Sect. 2.3.3). Connected to the reservoir is the headrace structure which often consists of an elevated pressure tunnel with a small gradient, a surge chamber and a steep pressure shaft or penstock. Occasionally an open channel is built instead of the inclined pressure tunnel. The surge chamber can either be constructed as surge shaft or as surge chambers depending on the intended operating mode of the high pressure power plant. The size of the surge chamber depends upon the characteristics of the turbines and operation modes (see Sect. 2.3.4.4).

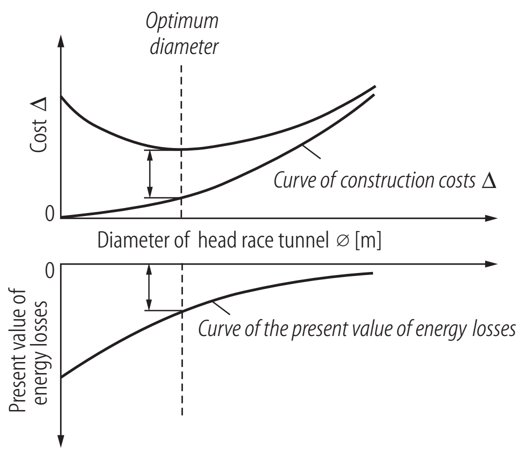


Fig. 2.3.4. Determination of the optimum diameter of a power conduit.

The head race has to be equipped with gated intakes or valves (e.g. two butterfly valves) right after the reservoir – one for operation purposes and one for revision purposes. The installation of a valve at the transition from pressure tunnel to pressure shaft is optional. At the end of the pressure shaft or pressure tunnel a spherical valve needs to be fixed in front of the turbines.

The powerhouse [97Lat] comprising the electromechanical units such as turbine, generator and electrical components can be realized as an underground powerhouse, a shaft powerhouse or an open air powerhouse (see Fig. 2.3.5) as laid out in Sect. 2.3.5.

The tailrace has to be adapted to the requirements of the turbines (free surface flow or under pressure). If the tailrace is very long, it might be necessary to install a tailrace surge chamber.

If geological and topographic conditions are favorable, it might be practical to construct the headrace in form of a pressure tunnel with low elevation and high overburden, and the underground powerhouse deep within the mountain. The advantage with this method can be the favorable rock mechanic prerequisites for a sealed lining of the pressure tunnel of the short headrace system (see dotted line in Fig. 2.3.5a). By locating also the powerhouse deep inside the mountain, the internal pressure of the long tailrace is reduced compared to a pressure tunnel.

The tailrace balancing reservoir allows the water used for power generation to evenly return into the river so that the ecosystem is not damaged. The necessary volume is determined by the amount of water used per day in the course of the year, and the aim of avoiding surges in the river which exceed a ratio of 1:3 to 1:5.

2.3.2.3 HPPP without reservoir

When a high pressure power plant has no reservoir, but a weir and an intake structure, it is necessary to include a desilting structure right after the weir [03Gie]. The desilting structure is generally designed to separate grain with a diameter > 0.5 mm. When designing the intake and desilting structure, it has to be taken into account that the power plant will be operated and controlled via their water level. It is therefore advisable to build a balancing reservoir whenever possible. This reservoir can either be located right after the desilting structure or before the pressure shaft or penstock (see Fig. 2.3.5b) and will considerably improve the operating characteristics of the high pressure power plant.

For the construction of the pressure shaft or penstock and powerhouse the same criteria apply as laid out in Sect. 2.3.2.2. If there is no headrace balancing reservoir, it is not necessary to include a tailrace balancing reservoir.

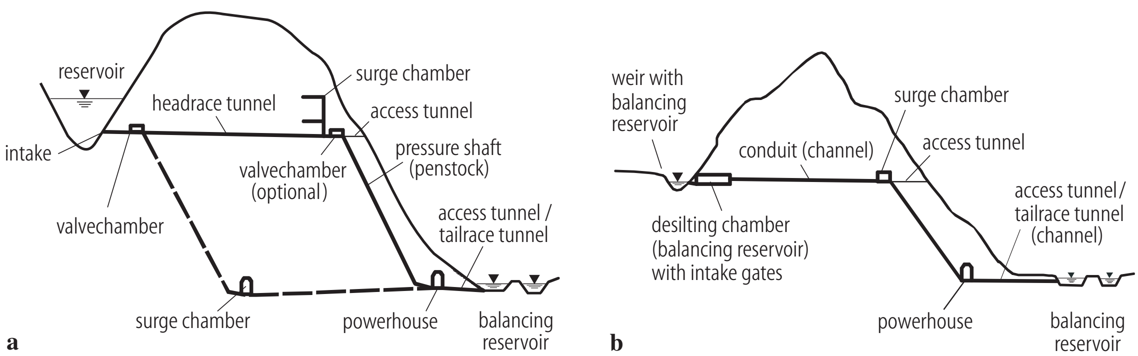


Fig. 2.3.5. **(a)** HPPP with reservoir. **(b)** HPPP with weir and desilting structure.

2.3.2.4 HPPP at the toe of a high dam

One of the most common types of HPPPs are high pressure power plants at the toe of a high dam [94Kac]. This type of power plant is a medium height power plant which uses the gross head created by the dammed water. The following operating modes are possible:

- *Power plant with a changeable reservoir level like a HPPP*
  For this type of power plant a large reservoir is necessary, and fluctuations in the reservoir level must be possible. The available storage volume, which lies between the maximum and the minimum storage level, can be used according to its size as weekly, seasonal or multiseasonal storage reservoir. This type of power plant can then be operated as illustrated in Sect. 2.3.1.1.1 This type of power plant is often used for multipurpose schemes (see Sect. 2.3.1.1.2). The most important components of the power plant are – apart from safety components of the dam and the reservoir – one or several, often short penstocks (occasionally also pressure tunnels) with intake gates and a valve ahead of the turbines, a penstock manifold, a powerhouse comprising the electromechanical units and a tailrace possibly with balancing function. The used turbines are mostly Francis turbines (see Sect. 2.7).
- *Chains of power plants*
  In case of the complete development of a river, the dams will be designed in such a way that the tailrace level of the upstream power plant corresponds during rated discharge to the intended maximum storage level of the downstream power plant. This type of power plant is operated like a chain of river power plants (see Sect. 2.2) and includes the same components mentioned above (see Fig. 2.3.6). Flood management is a task of major importance when operating chains of power plants and should be determined on the basis of precipitation-discharge models.

2.3.3 Intake and storage

The reservoir being an essential part of the HPPP can only fulfill its economical functions according to energy production and water management over the whole year by the adequate size of the storage (see Sect. 2.3.1.2.2). Usually, the storage is built by closing up a suitable valley by means of a dam [94Har]. The type of the dam depends on the shape of the valley, the geological conditions of the damsite and the available materials. In addition to the topographical necessities geological conditions are of basic importance. The imperviousness and the stability of the foundation and the reservoir's slopes are essential requirements for the construction of a barrage (see Fig. 2.3.7). Furthermore, it has to be economically reasonable to seal the foundation and banks of the dam.

Basically two types of dams, concrete dams and filldams, are constructed [91AUS]. Concrete dams are mainly applied when the bedrock is stable and when no filling material is available. Filldams can be based on high overburden. They are economically feasible when suitable filling material is available in the vicinity of the damsite and when seepage losses can be kept under control by reasonable means.

A barrage consists of the dam and appurtenant structures like spillway, bottom outlet and hydraulic steel structures [03Tan]. In respect to public and operational safety large dams have to be equipped with an appropriate safety and measuring system [85SWI]. Whenever feasible the inflow of the catchment area is increased by diverting streams from adjacent valleys.

2.3.3.1 Concrete dams

According to the shape of the valley or the condition of the bedrock, respectively, gravity dams, arch dams, Butress dams, multiple arch dams and – if the topographical conditions are extremely anisotrope or inhomogeneous – combinations of the above dams are being used [87Bli]. The main requirements are a high strength, density, frost-resistance and durability. Further aggregates in the appropriate quality must be available.

Special requirements such as the development of the heat of hydration during the setting time and shrinking and creeping demand special constructive measures and technological properties of the concrete. By an appropriate grain size, distribution of the aggregates and limitation of water and cement content, hydration heat and shrinkage is reduced while strength and workability can be achieved. In addition ice is added to the fresh concrete and cooling systems are implemented [88Gie].

Block sizes are limited to extensions of 15 to 20 m and a maximum lift thickness of approximately 3 m. Thus, it is necessary/inevitable that concrete dams must be built in blocks with vertical construction joints. Whenever a monolithic effect of the dam is required, the vertical joints are grouted after the fading of deformations due to temperature. When choosing the aggregates it is necessary to put great importance on the strength and durability. Furthermore, one has to pay attention that no unfavorable reactions between the aggregates and the binder (AAR, alkali aggregate reaction) occur.

For concrete dams the sealing element is the concrete itself. Where necessary the imperviousness of bed rock is achieved or improved by grouting [98Ris]. The location and depth of the grout curtain is determined by foundation properties, the height of the dam and the draft conception.

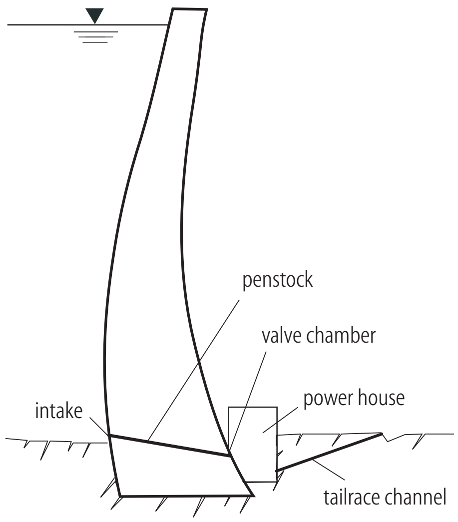


Fig. 2.3.6. Powerhouse at the toe of a dam.

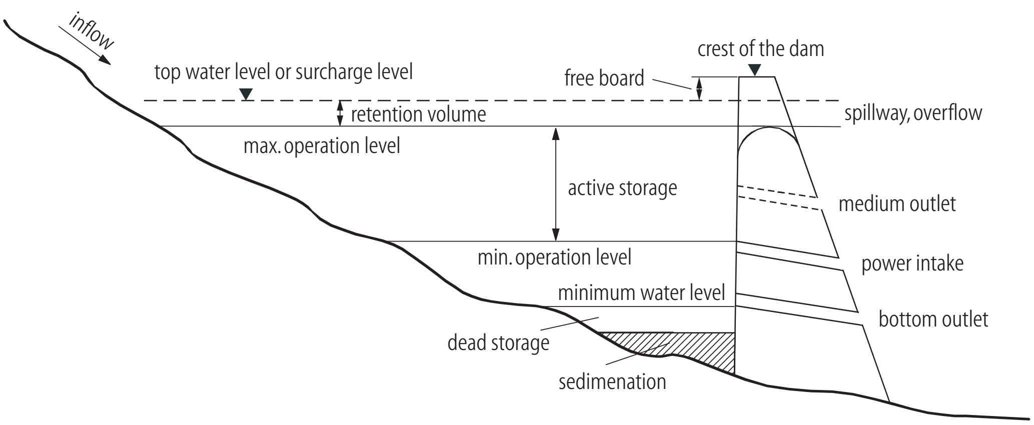


Fig. 2.3.7. Definition of the parameters of the reservoir.

2.3.3.1.1 Gravity dams

Gravity dams are especially suitable for closing up wide U- or trough-shaped valleys. The main range of application is in dam heights up to about 100 m. With an exceptional height of 285 meters the gravity dam Grande Dixence in Switzerland is the highest gravity dam in the world (see Fig. 2.3.8). The definition of gravity dams derives from the principle underlying the design of these dams. It is founded on a simple static system. The weight of each individual concrete block should be big enough to deflect horizontal water pressure downwards so that the shear and friction forces can be transferred to the bedrock foundation to an extent of its safe bearing capacity. Normally the safety should be provided and is calculated without taking into account the bearing capacity in the abutments.

Gravity dams have a triangular form in the cross section, which is directly deducible from the hydraulic load. The layout of the alignment in plan is mostly straight, sometimes slightly curved or bent. The dam's downstream slope inclination is mostly set from 1:0.65 to 1:0.8 [87Bli]; the upstream inclination is horizontal or slightly inclined to the upstream side. The dams are dimensioned with an appropriate security for various loading conditions. For the loads both active external and constrained forces apply:

- Active external forces are dead load, water pressure from headwater and tail water, uplift or pore water pressure, silt and earth pressure, special forces like earthquakes and dynamic forces from overtopping;
- Constrained forces are form-alteration-forces which mainly result from shrinkage, swelling and creeping of the concrete. They only play a subordinate role with the gravity dams.

In the calculation the evaluated and allowable stresses are juxtaposed. However, in the basic loading conditions there should not be tensile stresses at any point of the dam. Furthermore the basic stability requires safety against overturning at any horizontal plane, against sliding in the foundation on any horizontal plane as well as against allowable shear strength. For the stability of gravity dams it is especially important to avoid uplift in both the foundation and the body of the dam itself. This can be achieved by a grout curtain on the upstream head and by a drainage curtain in the bedrock as well as by a drainage gallery in the dam itself.

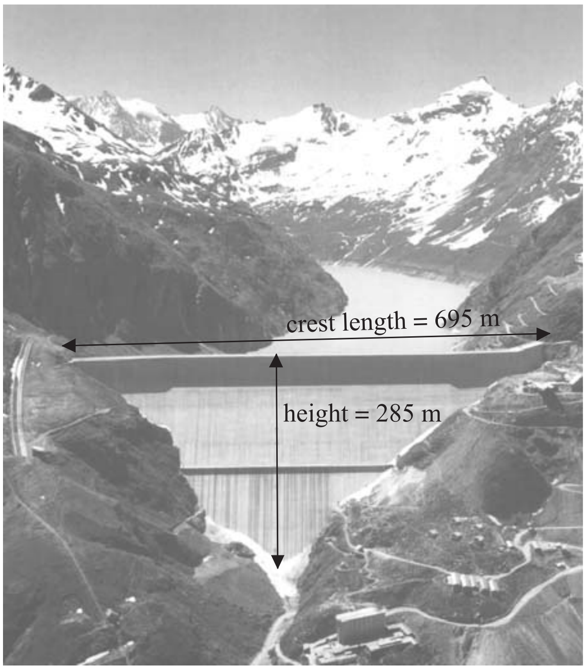


Fig. 2.3.8. Gravity dam Grande Dixence, Switzerland: view from downstream.

2.3.3.1.1.1 Constructive particularities for mass concrete

In order to avoid constrained stresses derived from changes in volume due to temperature and the mass of the concrete, the dam is divided in single blocks, which are connected through vertical and horizontal construction joints. The width of the blocks is 12-20 m; the length should not exceed 30-40 m in one section. The lift in one section is 1.5-3 m. While the core of the concrete is made of a lower cementitious content, the facings of 2-3 m thickness have a higher cementitious content.

Due to the development of hydration heat during the setting period of the mass concrete structure, statically unfavorable cracks can emerge. A reduction of heat development can be reached by the composition of the aggregates themselves, the use of cement with a low hydration heat (e.g. blast furnace cement), the admixture of suitable additional substances (e.g. bottom ash), surface insulation, time interval between lifts of at least three days (optimum 5 days), using low lift heights, additional measures of coolants (e.g. ice adding, cooling the aggregates, installing a cooling coil system) and post cooling [88Gie].

2.3.3.1.1.2 Special methods of construction: RCC and RCD

Recent developments have eased the placing of the concrete through the method of RCC (Roller Compacted Concrete) [94Han]. Concrete of dry non-flowable nature with a relatively firm consistency is placed in layers and compacted by vibratory rollers. Construction joints are not designated. The upstream vertical facing requires a surface with good appearance and durability mostly incorporating a watertight barrier. Therefore slip-formed interlocking conventional concrete elements or precast concrete tieback panels with a flexible waterproof membrane can be used. The same conception is also possible on the downstream face. At a low inclination the RCC can also be placed without conventional form work.

As a special form, the RCD (Roller Compacted Dams) construction method was developed in Japan. There is an inner core with little cementitious content. Only every third layer with a conventional thickness of 25 cm is compacted. The facing layer consists of conventional concrete which is compacted by immersion vibrators (usually without steps).

The advantages of material savings and short construction time led to cost savings, namely 20-30% with RCC and 10-15% with RCD in comparison to conventional construction methods [99Dun]. Additionally, spillways can be directly incorporated into the structure (e.g. stepped spillway, see Fig. 2.3.9). So far there is few long term experience data available with regard to water tightness due to the multiplicity of horizontal joints, crackings due to shrinkage and temperature and the durability, because the first RCC dams were built in the beginning of the 1980s only.

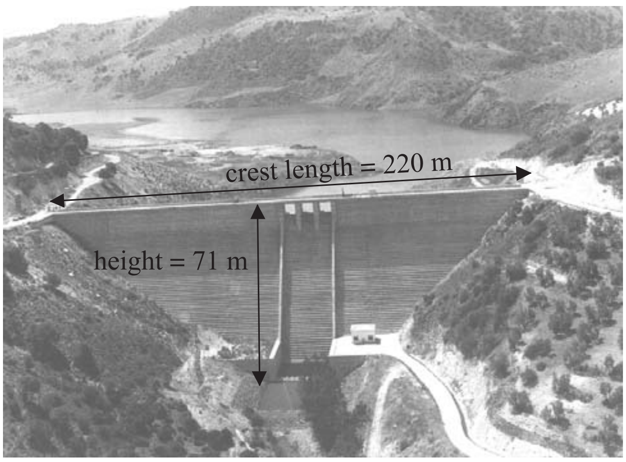


Fig. 2.3.9. RCC dam, Puebla de Cazalla (Spain).

2.3.3.1.2 Arch dams

Arch dams are mainly used in narrow, high valleys. In general, the proportion of crest length and height above foundation should be as small as possible. Especially with tall arch dams and narrow valley forms, the optimized form of the dam can result in remarkable material savings.

Arch dams are shells with a single horizontal or double horizontal and vertical curvature which reach from one valley flank to the other or are occasionally supported by artificial abutments. Due to the form of an arch dam as a shell, the bearing of the loads is mainly done by the abutments. In wide valleys the load bearing system is split up by the arch into the abutments and by the own weight down to the bedrock foundation. They make heavy demands on the abutments' rock quality due to their sensitivity against deformation of the abutments [86Pue].

The first arch dams were circularly shaped, with varying radius and/or angular width according to the form of the valley. At cylindrical arch dams, the center of the radius is on the same place over the whole height. At angle arch dams, the angular width is nearly constant over the whole height. At dome-shaped dams, the radius and the angular width, respectively, are varied. The development of modern calculation methods with the help of powerful computers allows an increasingly efficient adaptation of forms of arch dams according to the given shape of the valley and the often anisotrope foundation. Hence, the stresses in the foundation and maximum bearing capacity could be optimized. As a result of this optimization, arch dams are nowadays curved both horizontally and vertically. The curvatures in plan are elliptic, hyperbolic or parabolic arches. Generally, the radius from the vertex to the abutment increases. In the vertical cross section there are various arch forms used as well; however, overhangs are often obtained.

For the calculation of arch dams, the load cases are the same as for gravity dams [98Ris]. Additionally, the constrained stress induced by temperature changes in the concrete and by the yieldingness of the rock abutment or the displacement of the reservoir's slopes are to be considered. Generally, those stresses increase with the size and width of the valley. The first design is made by simplified models which are often based on broad experiences. Nowadays, the exact design and calculation of stress is usually achieved with the finite elements method (FEM), including the behavior of the foundation [77Zie]. The load-bearing behavior can be made visible very well with photoelastic tests. Hence, especially the behavior of stresses in the abutments and the parts of the biggest and smallest strain are recognizable. The aim is to reach a distribution of stress for a different combination of loads which is as free of tensile stresses as possible. One must be aware of the resulting tensile stresses on the upstream foundation zone which can have negative effects on the sealing of the foundation and the tight connection of the arch dam to the bedrock.

The construction of the arch dam takes place in blocks (see Fig. 2.3.10) similarly to the gravity dams. Besides the effect of a higher strength in the concrete, there are similar concrete-technological demands. The construction joints must be grouted to secure the monolithical load-bearing behavior. Therefore, and for sealing purposes, upstream and downstream waterstops are installed to ensure imperviousness.

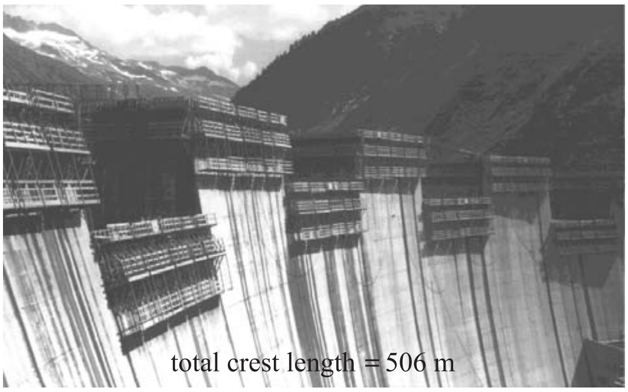


Fig. 2.3.10. Zillergründl arch dam during construction 1984, Austria [91AUS].

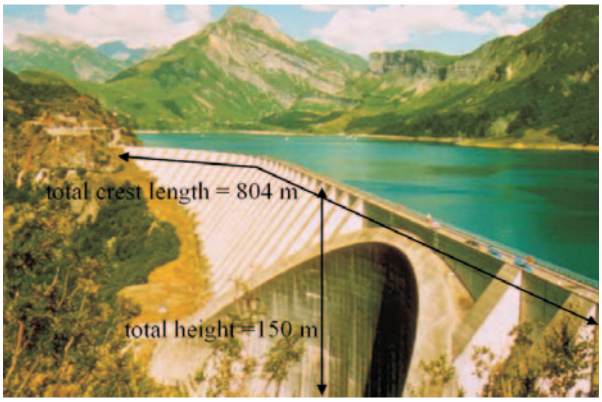


Fig. 2.3.11. Roselend, France, 1962.

2.3.3.1.3 Other types of concrete dams

In a wide shaped valley with usually not too high water head and a decent foundation, butress type dams and multiple arch dams are designed instead of gravity dams. Basically one has to distinguish between gravity dams with additional caverns and butress type dams. At a butress type dam the loads are drawn down into the foundation over flat counterforts. One of the largest butress type dam is part of the bi-national HPP Itaipu on the border of Brazil and Paraguay. 91 blocks form a total length of this dam section of 1438 m; the corresponding maximal height is 81 m.

There are advantages in mass reduction, a better decrease of the heat of hydration and a reduction of uplift forces. The disadvantages are that these dam forms demand a higher form work (partly even with an additional reinforcement). Furthermore, the narrow constructions are exposed more strongly to frost conditions.

In various topographical circumstances both combinations and special forms of butress dam and arch dam can be used (e.g. in Roselend, France; see Fig. 2.3.11). In wide and regular valley forms where single arch dams cannot be applied, multiple arch dams are used. Arches or slabs are spanned between several counterforts which are formed as high beams. Due to the foundation's rise of strain, the height of the dam is limited (to approximately 50 m). When fully using the arch load behavior, the spacing of the counterforts (arch span) can reach several ten meters.

2.3.3.2 Earth- and rockfill dams

Earth- and rockfill dams [97Kut] are predominantly built where the rock overburden is high and whenever the foundation properties are not favorable for concrete dams. They are only feasible when enough filling material of the required quality is available in the vicinity of the damsite. In general, dams constructed by a uniform type of filling material, so called homogeneous dams, are restricted to minor heights.

The filling materials used for construction are predominantly earth, gravel and rock. The material used for the fill zone should have an angle of internal friction which is as high as possible. Furthermore, it should have a high strength and a good permeability with a proportionality constant $k_f = 10^{-1}$ - 10^{-4} m/s. Especially the outlaying zones should be resistant to frost. Natural materials (e.g. clay, moraine) or artificial materials (e.g. asphaltic concrete) fulfill the function of the impervious element. The permeability coefficient of the materials used should have a proportionality constant $k_f < 10^{-7}$ m/s. The materials used and their characteristics demand much larger dam volumes in comparison to concrete dams. The cross-section is trapezoidal. The inclination of the fill dam is usually 1:1.5 to 1:2.5 on the upstream side and depends on the properties of the filling material [91AUS].

The acting forces like the dead load, water head, pore water pressure, earthquakes and special load cases should be taken into account. On the one side, one has to undertake calculations for dam stability and deformation, and calculations of the seepage of the dam under hydrostatic pressure on the other side. The required inclination of the upstream slope derives from the most significant load case of a full reservoir and the quick drawdown by the effect of the pore water pressure. The required inclination of the downstream slope depends on the material properties of the filling material taking into account the influence of the eventual water pressure. In any case, the line of seepage should stay within the cross-section. If there are no suitable permeable materials available, the safety for earthfill dams is achieved by a downstream rockfill bank.

Compared to concrete dams, there is higher seepage, which requires the implementation of adequate materials on the foundation, appropriate sealing conceptions and controlled drainage systems. The advantage of those dams is their insensitivity against larger and different deformations. They can be very economical if there is enough suitable material available in the near surroundings.

As a basic principle of construction, the overflow of a dam has to be strictly avoided in order to obtain safety against outer erosion. Consequently, a large freeboard is necessary (> 3-4 m). In exceptional cases at dams of small height and at reservoirs which are not permanently impounded, overflow can be allowed in designated zones (e.g. flood retention basins).

Statistically, 50% of the causes for failures in dams which are smaller than H = 15 m are due to outer erosion; in dams larger than H = 15 m, more than 50% are caused by inner erosion [03Lis]. So, further construction principles should avoid inner erosion. This can be caused by the erosion of the material (piping) or by flows through gaps. Likewise, especially with higher dams, operation structures should not be led through the body of the dam in order to avoid lacks and therefore dangerous seepage flows. Here, careful monitoring of the amount of seepage flow, the pore water pressure and of deformations is of great importance [85SWI].

The appearance of cracks caused by deformations has to be observed properly in order to avoid the failing of the dam's stability. The main reasons for the latter are instabilities of the fill zone or the foundation and the material's liquefaction originating from earthquakes. The slopes which naturally seal the reservoir represent another question of safety which is similar to all other dams. The stability of the slopes has to be assured for various water levels.

2.3.3.2.1 Types, conception and linings

Homogeneous dams (see Fig. 2.3.12a) are built by one type of material ($k_f < 10^{-6}$ m/s), simultaneously providing stability and sealing. Generally, their heights are limited and their functions are subordinate (e.g. flood protection, embankments).

In zone dams (see Fig. 2.3.12b-d) the individual zones provide stability and sealing. The sealing can be achieved by an impervious facing on the upstream slope or by an impervious core. The latter can be wide or narrow and can be vertical or inclined. The material for the upstream sealing can be asphaltic concrete, concrete and reinforced concrete or plastic membranes for subordinate dams of smaller heights. For economical reasons, lately CFRDs (Concrete Faced Rockfill Dams) have increasingly been constructed worldwide. A vital element of an upstream sealed dam is the proper connection to the subsoil sealing – grout curtain, slurry trench, etc. – respectively. The sealing has to be protected against uplift at low reservoir levels resulting from pore water pressure by an effective drainage. Also, seepage is collected and measured there.

For zone dams, various types of cores are developed. Mineral cores consist of clay and moraine ($k_f < 10^{-7}$ m/s), sometimes improved by adding clay minerals and bentonite. Artificial cores are made of asphaltic concrete, slurry trenches, concrete, etc. and provide a much higher deformability. Erosion being the basic risk for a dam, all efforts are made to counteract this danger. Even when the low permeability of the used material would allow a narrow core, they are designed widely to achieve higher safety against erosion. The use of filling material with a graded grain distribution as well as uniform material with high cohesion provides interior stability against erosion.

Another vital element are filter zones downstream next to the core to stop eroded fines and provide protection against the development of pore water pressure in the downstream fill. The gradient of the slopes of the dam are determined by the sliding resistance given by the friction angle of the filling material. At the upstream slope in a fill of low permeability under draw down conditions, pore water pressure would decrease the sliding resistance. To avoid flatter slopes in this zone, debris containing little fines or rockfill (e.g. blasted rock mass) is implemented. In core dams the upstream face is protected against wind and wave erosion and the development of strand terraces by coarse rockfill as "rip-rap", a layer of blocks (see Fig. 2.3.13).

Another issue in areas with high precipitation and in northern countries is the degree to which the dam construction is affected by rainfall. Filling material with high clay content and plasticity is sensible to the water content. This hazard is considerably reduced in rockfill dams with an artificial core.

Even though a central impervious core is better protected against damages, its maintenance is more difficult. But it is easier to install an inspection gallery (see Fig. 2.3.12d) and to connect the sealing both to the valley flanks and the subsoil. In the course of construction the sealing and the body of the dam do not have to be carried out one after the other but can be built at the same time. This allows the integration of small dams needed for the construction period into the later fill dam and, if of economical interest, a partly operation until completion.

2.3.3.2.2 Foundation, sealing in the subsoil

Before the design of any dam, a complete geologic and soil mechanic investigation of the foundation area is necessary. In most cases the implementation of an effective grout curtain or any type of cut off wall (see Fig. 2.3.12c) in connection with the sealing element of the dam is decisive. The seepage is restricted to a desired limitation, heavily reducing the gradient and capacity of erosion downstream the impervious element.

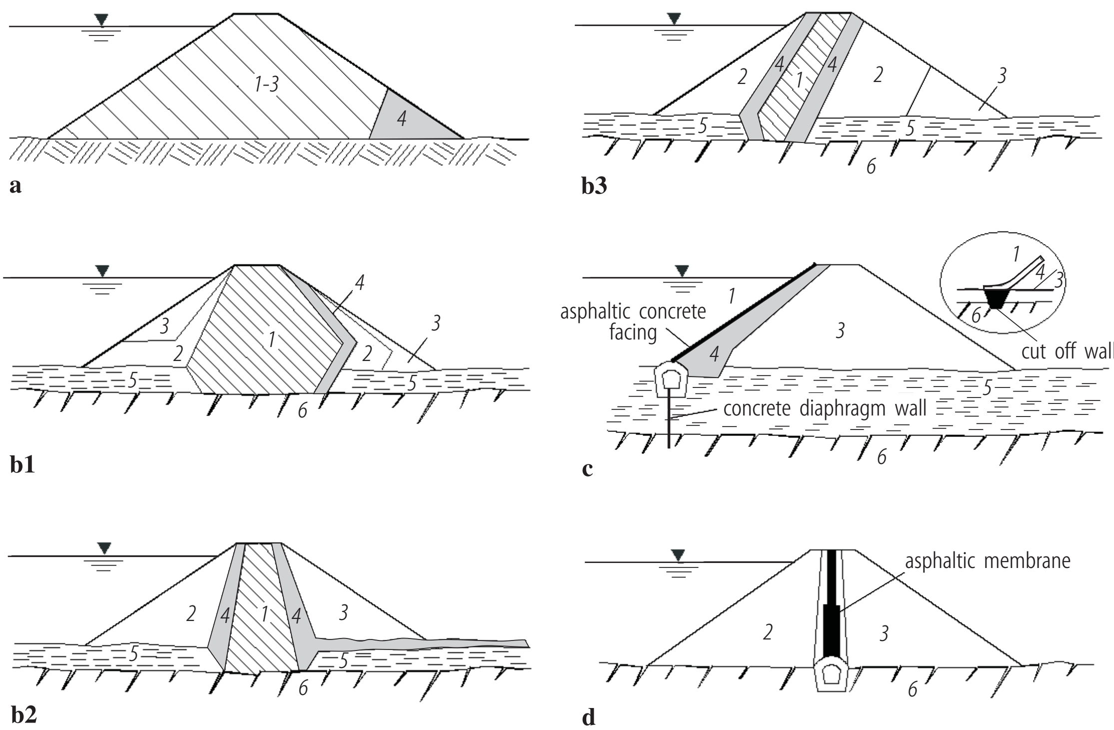


Fig. 2.3.12. Types of dams and their typical cross sections. **(a)** Homogeneous dam. **(b1)** Dam with broad central core. **(b2)** Dam with narrow central core. **(b3)** Dam with upstream inclined narrow central core. **(c)** Dam with upstream sealing membrane. **(d)** Dam with central thin sealing membrane.
Legend: 1 - sealing zone; 2 - transition zone; 3 - fill zone; 4 - filter zones; 5 - subsoil (e.g. alluvial layer); 6 - bedrock.

Fig. 2.3.13. Finstertal Rockfill dam, 1981, Austria. Crest length = 652 m, height = 149 m.

At many dam sites the foundation area needs special attention. The subsoil is natural, while the dam is man made. In addition to seepage and erosion sliding along horizontal soft layers, resistance decreased by pore water pressure as well as shear failure in a vicinity of the downstream toe are the main questions. Stability can be achieved by blankets along the toe. Uplift forces can be reduced and erosion avoided by the installation of relief wells. Compaction grouting can improve liquefiable soil.

A proper connection of the impervious zone in the dam with the subsoil is essential (see Fig. 2.3.12b). In dams with an upstream sealing where the core is based on rock mass, inspection galleries (see Fig. 2.3.12c) are useful but considered unnecessary by many experts. Nevertheless the implementation guarantees the possibility for inspection and for additional grouting when necessary.

2.3.3.3 Safety aspects of dams

The overall safety of a dam [85SWI], [91AUS] is based on

- the structural safety,
- the monitoring system and
- an emergency conception.

Structural safety is achieved by carrying out the state-of-the-art principles of constructions. One very important principle to achieve higher safety is to implement redundancy at various parts of the dam.

The monitoring of the dam is based on the instrumental equipment according to the type of the dam and constant assessment of the resulting measurements as well as the visual inspection of the dam, the surroundings and the reservoir. The basic principle of every monitoring is the early detection of extraordinary events in order to be able to react early and set appropriate measures or maintenance works in time and without time pressure. The visual inspection by the engineers who are most familiar with the dam and its behavior is of a particular significance because experience shows that most changes of a dam are recognized by the employees.

Even though the likelihood of a damage of a dam is minimal if the state-of-the-art principles of constructions were observed and the dam is constantly monitored, an emergency conception based on floodwave calculations has to be established for completing the safety conception. In a floodwave calculation one usually takes the total failure of the dam as a point of reference. The emergency conception based on this calculation has to be worked out with the responsible authorities.

2.3.3.4 Intake structures for diversion

For a hydraulically favorable and economical running of a HPPP with/without a reservoir, the inflow of the reliable yield into the reservoir or the appurtenant structures for operation have to be secure at any time. Accordingly, the intake structure has to be designed and developed very carefully [95CHI]. Special care and thought has to be given to the following factors:

- Exposed location of the buildings which are often hard to access;
- Extreme climatic circumstances in the high mountain locations and side-effects like avalanches, landslides, mud streams or rock fall;
- Location of the inlet and diversion structures in torrents and their strong debris flow;
- Prevention of hindering swimming and floating material from entering the structures for operation.

2.3.3.4.1 Intake structure (weir)

Section 2.2 has already dealt with intake structures at rivers and large streams. In this section hydraulic structures for the same purpose in high mountain locations in the form of so-called “bottom weirs” and the diversion into the reservoir will be explained in more detail [01Nov]. The particular construction of a bottom weir consists of a small weir for the impounding and a following vertical or inclined rack from which the water is pulled down and led through a short canal to a desilting structure [93Rau]. From there, the desilted and cleaned water either flows into the reservoir in the form of an open channel flow or is directly led in a penstock that leads to the powerhouse. The bottom weir is equipped with a rack installed at the bottom of the river which prevents the intake of large sized bed load. In case of a flood, the inflow into the desilter can automatically be closed for operation by a gate and the water is transported over the weir structure into the river bed. It is important that the intake structure is designed as massive and automatic as possible. Furthermore, one should aim at supplying the building autonomously because of the exposed and often unapproachable location.

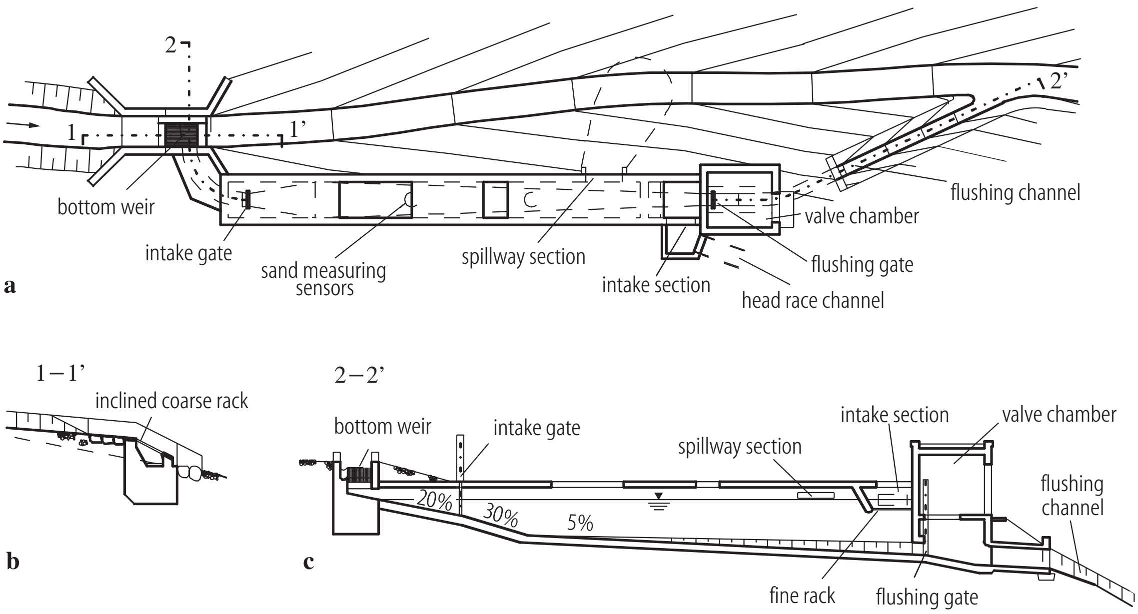


Fig. 2.3.14. (a) Bottom weir with desilting chamber, plan view. **(b)** Cross-section 1–1’. **(c)** Cross-section 2–2’.

The function of the following desilting structure (see Fig. 2.3.14) is to separate off the bed load and suspended load up to a grain size between 0.2 and 0.5 mm in diameter. The principle of the desilting is based on calming the flow and reducing the flow velocity so that the suspended load can deposit on the ground. Desilting structures are based on the most different systems. The most popular form is the longitudinal desander. The material settled down on the ground is returned into the riverbed either intermittently or by a permanent flushing system.

Besides the plant's parts necessary for the sluicing, the equipment of an intake structure consists of an overflow section for the power intake, a safety outlet and an automatic safety gate.

2.3.3.4.2 Power intake

From the reservoir the water is flowing from the appropriate intake structure according to the purpose of its use. It is then transported over channels, galleries, penstocks and pipelines and is finally led to the turbines or possibly other parts of the plant [98Rob]. In reservoirs only used for hydropower production the power intake is mostly on the banks or near the bottom of the reservoir beneath the lowest operation levels. Blockage by sedimentation should be prevented by a convenient location or structures preventing sediment intake. The power intake [95CHI] to the succeeding head race system should be optimized for the flow, mostly shaped in form of a trumpet in order to provide hydraulically favorable conditions and prevent an unfavorable development of eddies. The velocity at the intake in the cross section of the screen should be within the range of 1 m/s. The intake structure is equipped with a rack or fine screen and a gate for operation and revision. In reservoirs which are also used for water-supply, several intakes on different elevations should be planned in order to provide the best possible water quality at any given time. The optimal solution is the construction of an intake-tower, where the gates and intakes are concentrated. Those buildings are accessible over a bridge, an access gallery or by boat.

The design for powerhouses at the toe of a dam is special because the power intake is included in the dam, especially in concrete dams, and a power conduit is led through the body of the dam to the turbines. Because of the short distances, the gates and valves for operation and revision are to be placed economically and practically.

2.3.3.5 Bottom and medium outlets

Dams are generally equipped with safety facilities to lower the reservoir in case of emergency [98Vis]. As one possibility to draw down the water level and to empty the reservoir, a bottom outlet should always be part of the design. Additionally, a medium outlet can be installed to achieve a faster emptying. Moreover, the bottom outlet can be used to keep the operation service structures free of sedimentation by regular flushing. During construction time the bottom outlet can also serve as a bypass due to its deep location.

During normal operation, those outlets are closed. However, they have to be fit for use at any time and guarantee the release of a calculated capacity. Therefore, regular checks with practical tests are useful. There is no standard when dimensioning the outlets. It is recommended to base it on approximations of the time required to empty the reservoir to a head from which on a dam failure can be largely ruled out.

The intake of the bottom outlet should be equipped with a wide trash rack and stoplogs [98Rob]. The bottom outlet is provided with valves (see Fig. 2.3.15) for emergency and revision which are located in a valve chamber or a valve building outside. The location of the valves is set out according to topographical and geological conditions given for the bottom outlet pressure pipe. A location in the cross section of the sealing of the dam up to a more downstream position is favorable. The calculations of the pressure pipe in those cases are carried out either on internal or external pressure. If the conduit after the valves is comparatively long, the cross section for the discharge has to be dimensioned accordingly large in order to lead the jet stream outwards by using a sufficient aeration [98Vis]. A discharge under pressure should be avoided. At the end, a safe energy dissipation [95Vis] has to be guaranteed either by a spillway basin, a flip bucket or by dispersion on hard rock. At concrete dams, medium- or bottom outlets can be included in the dam body. In earth dams they should lay inside the rock bypassing the dam structure.

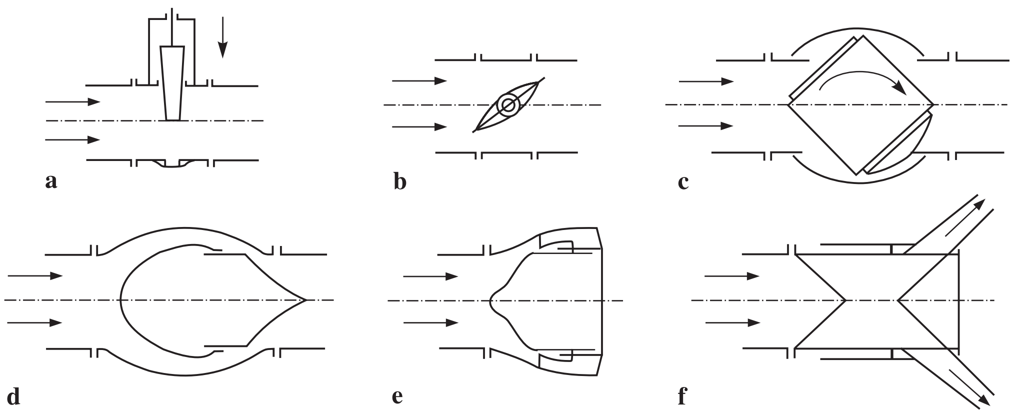


Fig. 2.3.15. Types of valves. **(a)** Wedge type gate valve. **(b)** Butterfly valve. **(c)** Spherical valve. **(d)** Needle valve. **(e)** Hollow jet valve. **(f)** Howell-Bunger valve.

2.3.3.6 Spillways

To ensure a safe operation of a dam during flood, a spillway is necessary. It has to be operational at any time and be able to discharge the maximum design flood without exceeding the highest possible operation level and without endangering the dam at any time [98Vis]. The estimation of the maximum design flood is based on hydrological data and will be explained in more detail in Sect. 2.3.7.3. The various types of dams set the choice of the possible types of spillways. At fill dams the spillway should be situated separately from the dam-body in order to prevent dam erosion and preserving the foundation on the downstream toe of the dam. At concrete dams overflow sections of the crest or outlets in the dam body can be used for discharging the flood.

Generally, one can distinguish spillways with a fixed weir structure where the water level varies but may not exceed a maximum water level, and those with movable structures for discharge where the maximum surface water level is kept constant according to a variable discharge section. In any case, the failure of operation of appurtenant structures should be prohibited, e.g. by redundancy or constructively. Similarly, the necessary discharge section could be kept clear of blockings by debris flow, wood or ice at any time, if necessary by constructive solutions like special rack or rope systems or simply nets.

If a certain maximum operation water level is reached, either the spillway starts operating by itself or the gates have to be operated manually or opened automatically. Before the flood discharge can be given back to the river, a safe energy dissipation has to take place. The return of the water to the original riverbed should at best take place in flow direction without danger of erosion.

2.3.3.6.1 Types of spillways

There are many different systems or special forms for the construction of spillways [95ASC]. The most common types and their functioning are briefly described below. Their dimensioning and calculation can be taken from the technical literature for hydraulics [98Vis] and hydraulic engineering [98Rob].

A fairly economical and efficient way for a concrete dam is the flood discharge directly over the crest if there is sufficiently space for the energy dissipation and the further transport into the tail water. The overflow can take place over a fixed sill or via movable gates, where mainly fish belly shaped shutters and sliding gates are used. The water is passed on downstream over the steep concrete wall. The spillway section can be located either in the middle or at the side(s). In arch dams with an overhanging structure, the flood discharge can take place in a free jet-stream if an according stable riverbed is available where one part of the energy is already consumed by the aeration.

For high dams and relatively large amounts of flood discharge, the spillway structures are built besides the dam. In this case the overflow-crest can be accomplished either fixed or movable, too. Following the intake structure, there is mostly a chute or a tunnel type structure with open channel flow conditions. The same principle applies for the side channel spillway. The conventional side channel spillway consists of an overflow weir followed by a narrow collecting channel in which the direction of the flow is approximately parallel to the weir crest. Further downstream the water is led again over a tunnel or chute.

Morning glories or spillway shafts are another often used form of spillways. They mostly consist of a crest circular in plan, of which the profile is shaped free of hydraulic pressure. A well rounded overflow section takes the water to the vertical shaft. A slightly inclined or nearly horizontal tunnel with open channel flow is situated after a rectangular bend in cross section. It is important for the smooth functioning to install a sufficient air entrainment system from the bend downwards to facilitate an open channel flow. Equally, the overflow should not be submerged, because it is not possible to raise the overflow capacity and unfavorable pulsations occur in the vertical shaft. Morning glories are located on favorable topographical sites in the reservoir or on steep, rocky valley flanks with excavation possibilities on the slope. In very steep slopes it can also be constructed as a directly attached semi circle (see Fig. 2.3.16).

Siphon spillways are a special form of spillways with a comparatively high starting discharge under pressure. A disadvantage is that the discharge capacity at rising water levels does not increase in the same way as with free overflow crests.

2.3.3.6.2 Transition and energy dissipation

The transition especially of high discharges after the spillway mostly takes place as open channel flow in chutes to which a flip bucket is attached or which disembogues into a stilling basin. At spillways with smaller discharge capacity the water can be conveyed as an aerated open channel flow in a conduit system as well. Because of the different discharge values it is rather difficult to convey the water downstream in pressure conduits. Unstable flow characteristics during transition occur and would cause pressure fluctuations and vibrations [94Nau] as well as unacceptable noise. In order to prohibit cavitation [90Fal] and successive damages on a large scale in the concrete structures, chutes should have adequate aeration of the high velocity flow.

The energy dissipation [95Vis] can either take place in a stilling basin as a free jet over a flip bucket or by a direct dispersion of the jet on an opposing stable rock. A very effective form of energy dissipation with a high velocity flow is the flip bucket. During the jet's trajectory to its impact location it spreads and frays by entraining a large volume of air. Nevertheless the major portion of energy dissipation happens at the impact site and can cause bed erosion unless the jet impact area is located in extremely durable rock. Therefore sufficient safety measurements have to be undertaken on the impact area and the river banks. A preformed scour hole or a small artificial impact basin filled with water can also be highly efficient.

Another special form is the stepped spillway, which is especially popular at RCC dams (Fig. 2.3.9). Due to the arrangement in steps energy dissipation by air entrainment and friction over the entire height occurs and already 80% of the total energy is dissipated until the water reaches the stilling basin.

Fig. 2.3.16. Example of a semi circle morning glory.

2.3.4 Water conduits for a HPPP

Water conduits carry the water from the intake of the diversion to the reservoir or from the weir (or the reservoir) to the powerhouse. There are 5 different types of water conduits and headraces:

- Free surface canals;
- Free surface tunnels;
- Pressure tunnels;
- Pressure shafts;
- Penstocks.

2.3.4.1 Free surface canals

Free surface canals have mainly been used in high pressure power plants before the development of modern tunnel technology and are nowadays only implemented in multi purpose schemes with irrigation. They are characterized by their extreme land-intensity. They mostly have a trapezoid or – in the case of very small cross sections – a rectangular cross section and were laid out so as to achieve an equable material balance between embankment sections and sections in the cut off.

The gradient of open canals corresponds to the allowed velocity for the rated discharge. The allowed velocity needs to be adapted to the lining and the cross section of the canal and also depends on economical aspects (see Sect. 2.3.6). When operating a high pressure power plant with a free surface canal in regions with low temperatures in winter, particular emphasis has to be put on avoiding freezing of the water.

2.3.4.2 Free surface tunnels

Free surface tunnels are generally used for diversions and for tail race structures with Pelton turbines. In exceptional cases they are realized as head race systems of power plants without reservoir and minor fluctuations of the water level at the intake (see Fig. 2.3.5b). One advantage of free surface tunnels is that no special requirements concerning imperviousness need to be fulfilled by the lining of the tunnel as they lack any internal pressure. The lining only serves as support and sealing of the tunnel wall and as a means of reducing friction losses if necessary, which is particularly the case if the tunnel was constructed in a drill & blast technique [00Joh]. Free surface tunnels are aligned in order to keep the water conduit as short as possible. In addition to that, the tunnel should go through geologically favorable rock formations and cross instable geological formations only for short distances [97Hud]. High overburden at sections with low strength of the rock should also be avoided (see also Sect. 2.3.4.3).

The suitable cross section of a free surface tunnel depends on the amount of discharge and the used construction technique. The velocity of the water is between 1 and 2 m/s for unlined tunnels and maximum 4 m/s for concrete lined tunnels. The design of the cross section (see Fig. 2.3.17) is determined by the individual construction technique. When using the drill & blast (D&B) method, the cross section preferably takes the form of a horse shoe; when a tunnel boring machine (TBM) is used, the cross section has a circular shape. Furthermore it has to be taken into account that the cross section is within the limits of the technically necessary minimum profile (see Sect. 2.3.4.3).

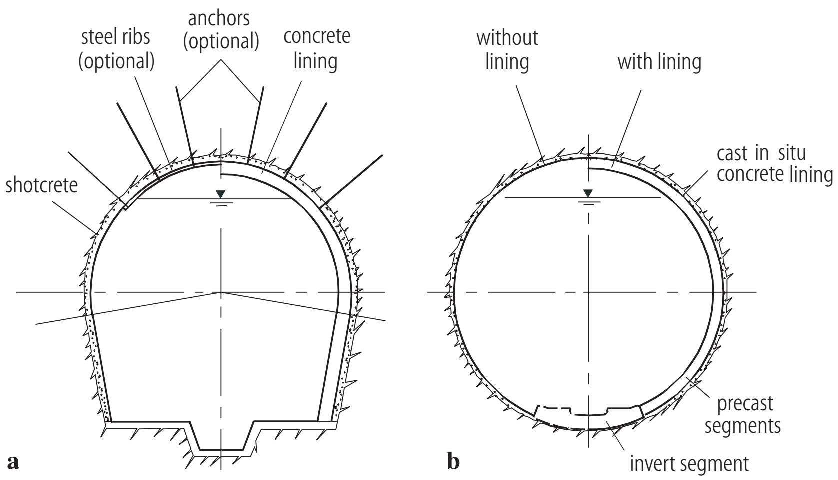


Fig. 2.3.17. (a) Horse shoe cross section without and with cast insitu concrete lining. **(b)** Circular cross section by TBM with and without lining (cast insitu concrete or precast segments).

2.3.4.3 Pressure tunnels

The pressure tunnel, which normally has a small inclination, connects the reservoir to the surge tank and the pressure shaft or the penstock (see Fig. 2.3.5), or in some cases directly connects the reservoir to the powerhouse. Pressure tunnels are used when large fluctuations of the reservoir level occur at the intake or in the reservoir. In accordance with the used construction method, it has proved to be favorable in the case of long water conduits to divide the head race structure of a high pressure power plant into a flat section – the pressure tunnel – and a steep section – the pressure shaft or penstock.

Depending on rock quality, overburden (height of rock mass above the tunnel), crack water pressure and internal pressure, pressure tunnels can be

- not lined,
- lined and not sealed or
- lined and sealed.

It depends on the overburden and the rock properties if a bearing ring for the tunnel is necessary [82Wit]. In the case of a high overburden a bearing ring is always applied. If the internal pressure is higher than the crack water pressure, an impermeable lining has to be attached in order to avoid water losses and the resulting danger of rock slopes.

When designing the alignment of a pressure tunnel the following aspects have to be taken into account: The pressure tunnel should be as short as possible and it should

- be in good rock,
- cross instable weak rock formations at the shortest distance possible,
- follow the alignment route, where the lower principal stress is higher than the internal pressure, as long as possible,
- avoid high pressure due to overburden at sections with low strength of the rock and
- lay out in such a way that the rock water pressure is higher than the internal pressure.

Complying with all the above recommendations – a task which might not always be fulfilled – will result in the construction of the most economical solution.

As it is the case with free surface canals, the cross section of pressure tunnels depends on the amount of discharge as well as the construction method [00Joh] and the type of lining, the latter having an impact on the roughness of the lining and consequently the friction losses. The permissible discharge velocity for unlined tunnels is between 1 and 2 m/s, for concrete lined tunnels a maximum of 4 m/s and for steel lining a maximum of 7 m/s. Another principle which can be applied when designing pressure tunnels is that the friction losses of a water conduit should range between 3 and 5% of the gross head. The final design is the result of an optimization process (see Sect. 2.3.1.2.4 and Fig. 2.3.4) which compares the construction costs to the capitalized value of the friction losses.

2.3.4.3.1 Tunnels without lining and sealing

An important precondition for constructing a tunnel without lining is that the pressure tunnel lies in good stable rock and that the minimum external water pressure is higher than or equal to the internal pressure. Furthermore, it is important that the roughness of the tunnel surface is low which can best be achieved by performing the heading by TBM (see Fig. 2.3.17b).

2.3.4.3.2 Tunnels with lining and without sealing

If a support of the tunnel wall is necessary due to the high overburden, a lining needs to be installed. The dimension of the lining is determined by rock-mechanic considerations. The optimum lining is such that the construction method allows for stress relieving movements at the tunnel wall to a certain extent and the supporting measures and the final lining constitute a compound system which activates the bearing capacity of the surrounding rock.

If the heading is performed by the D&B method or open TBM, a preliminary lining is included which provides support and safety for the tunnel wall. The final cast in situ lining increases the safety of the system formed by the rock/preliminary support/cast in situ lining and provides a low roughness coefficient (see Fig. 2.3.18a). Typically the thickness of the lining ranges between 20 and 30 cm and can occasionally also be higher. If the pressure tunnel is constructed by TBM with precast segment lining, the space between rock surface and segments is filled with backfill gravel and then grouted. This system does not allow major deformations of the rock surface, which results in a higher load to be carried by the lining. Despite these drawbacks this system is the most economical lining in most cases (see Fig. 2.3.18b). Typical dimension of the lining range from 15 to 25 cm, and can occasionally also be higher. If the load of the external water pressure, which has to be considered in the design of the system, cannot be taken by the surrounding rock despite the implementing of constructive measures such as sealing by grouting, relief measures (e.g. check valves) have to be provided for the case of a drained tunnel.

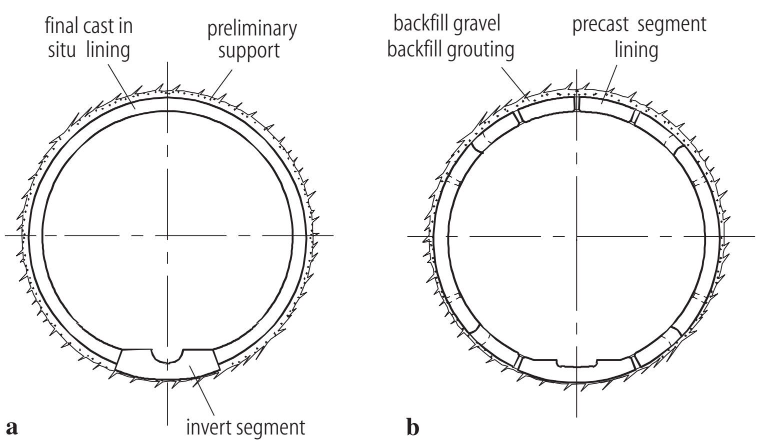


Fig. 2.3.18. Typical cross sections. **(a)** In situ lining. **(b)** Precast segment lining.

2.3.4.3.3 Tunnels with lining and sealing

If the internal water pressure is higher than the external water pressure, the tunnel has to be sealed in order to avoid water loss and consequential damage (e.g. rock and land slides). Which type of sealing is used depends on the initial "in situ stress level" of the rock mass. If this level is higher than the internal pressure, the internal pressure can be taken from the rock and the sealing only serves as sealing. If it is lower than the internal pressure, the sealing additionally has to take up all or parts of the internal pressure. The design of lining and sealing [82Wit] also has to take into account a deformation due to temperature (e.g. cooling of the lining caused by the water) and creep shrink deformation as well as an increased external pressure in case of a drained tunnel. In the following sections, all the sealing systems that have successfully been implemented worldwide are briefly discussed.

2.3.4.3.3.1 Pre-stressed concrete lining

For this type of lining the interface between lining and rock is grouted with high pressure ("Kernring"). Thereby the concrete lining is pre-stressed to an extent which should be higher, after the creeping has faded away, than the tensile stress in the concrete lining caused by the internal pressure. If internal pressures are not too high, this method can economically be applied to precast segment linings with TBM-heading (see Fig. 2.3.18b).

2.3.4.3.3.2 Lining with plastic membranes

A plastic membrane is inserted between concrete lining and rock in the case of high internal pressures when an additional sealing is desired. The membrane itself can serve as sealing or support the pre-stress grouting of the concrete lining (in the case of highly permeable rock). In any case the compatibility of the total system with deformations of the construction (crack width in the rock due to internal water pressure and membrane stresses at the cracks) has to be secured.

2.3.4.3.3.3 Steel lining

In situations where there are very high internal water pressures and comparably low "in situ stress levels", a steel lining is generally used. The design of the steel lining is determined by both the internal and external water pressure, and the lining can either take all the internal pressure or do so in co-operation with the rock. If the overburden is high enough and rock quality is good, it is economical to establish a co-operation between the rock and the steel lining in the design of the steel lining. It is necessary to determine the deformation property of the rock [97Hud] by means of in situ tests and adapt the construction accordingly. Therefore the steel lining is backfilled and grouted after its installation. The grouting pressure (pre-stress grouting) should be high enough to ensure that no gap will occur between steel lining and concrete after fading down of the plastic deformations and cooling of the steel lining caused by the water.

The co-operation of steel lining and rock is shown in Fig. 2.3.19 which also illustrates the levels of internal water pressure taken by the rock and the steel lining, respectively. The plate thickness depends on the diameter and level of internal pressure it has to take, the steel quality etc. It can range from a minimum of approximately 10 mm up to 40 mm or more and is furthermore limited by the welding capability of the steel. If the rock does not carry any pressure, the steel lining has to take the total internal pressure. Nevertheless, the steel lining has to be backfilled with concrete, and it is recommendable to fill the joint caused by shrinkage of the concrete by grouting.

In terms of load it is important to provide for increased external water pressure in the case of a drained tunnel. Additionally, the steel lining has to be designed and constructed as to prevent buckling due to external water pressure. If buckling can not be avoided in an economically sound way by increasing the thickness of the steel lining, the bearing capacity of the steel lining has to be enhanced by means of a ring reinforcement or by welded bolts on the lining. Particularly bigger inside diameters call for such measures as the plate thickness of the lining would have to be substantially higher in order to avoid buckling due to external water pressure than due to internal pressure.

Occasionally, also drainage is used in order to prevent the development of external water pressure. This alternative is not recommended, though, since in the long-term drainage may become ineffective.

2.3.4.3.3.4 Thin-walled steel lining

Thin-walled steel lining is a special type of steel lining where the steel lining is placed between the concrete lining and the rock [82Wit]. The advantage of this type of lining is that the external water pressure is taken by the concrete and that the design of the steel lining, therefore, does not have to allow buckling. In terms of construction, thin-walled steel lining is a rather complicated method, and it is advisable to install the steel pipe together with the preformed concrete lining.

2.3.4.3.3.5 Pressure tunnels with conventional or pre-stressed reinforcement

Conventional or pre-stressed reinforcement is occasionally used for pressure tunnels with sealing in order to avoid cracks in the lining. Normally, however, such reinforcement is not necessary as long as the above mentioned design principles are observed. As this type of lining complicates the construction process, it can not be recommended as constructional element.

2.3.4.4 Surge tanks

Due to power and frequency control, shut down or load trip of machine units (in case of emergency), the body of water being in motion in the head race is accelerated, decelerated or comes to a stand still very unevenly. The faster the changes in the velocity of the water and the longer the conduit, the higher are the fluctuations in pressure in the water conduit caused thereby. The surge tank is used as a storage for the body of water in motion to flow into, whereby the energy of motion is converted into potential energy and the internal pressure load exerted on the conduit is reduced. Thus certain fluctuations in pressure with an oscillation period of several minutes develop in the conduit from the reservoir to the surge chamber. A water hammer with an oscillation period of a few seconds builds up in the conduit from surge chamber to powerhouse (see Fig. 2.3.20).

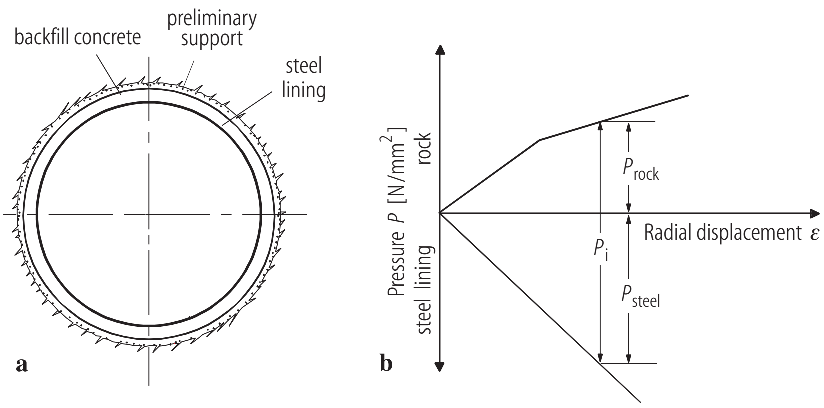


Fig. 2.3.19. **(a)** Typical cross section with steel lining.
(b) Schematic stress-strain diagram for the compound system rock-steel (with P_i the internal pressure).

The design of the surge chamber depends on the length of the water conduit, the velocity of the water in the water conduit, the velocity and mode of the power and frequency control, the characteristics of the turbines, the velocity of the shutting off movement of the turbine valves as well as on the used type of surge chamber. The simplest type of surge chamber is the surge shaft type (Fig. 2.3.21a). Since it needs a large reservoir volume in order to limit the oscillation movement, its construction is rather costly. The surge tank type (Fig. 2.3.21b) comprises either a top chamber only or a top and a bottom chamber and is characterized by an improved damping capacity and lower construction costs. Throttle type surge chambers (Fig. 2.3.21c) show a higher entrance resistance at the connection between conduit and surge chamber, which can have an effect on just one or on both flow directions. The construction of differential surge tanks (Fig. 2.3.21d) is more complicated than that of conventional throttle type surge chambers. However, this type shows the lowest water level changes and therefore also the lowest pressure changes in the tunnel. A special type of surge chamber is the compressed air vessel type (Fig. 2.3.21e) which can only be realized for deep pressure tunnels and when excellent rock conditions are given. It can be located close to the powerhouse, allows fast power and frequency control and reduces the dynamic load of the overall system. There is a variety of special types of surge chambers and combinations of elements of the different types that can all be adapted to optimally meet the requirements of the high pressure power plant.

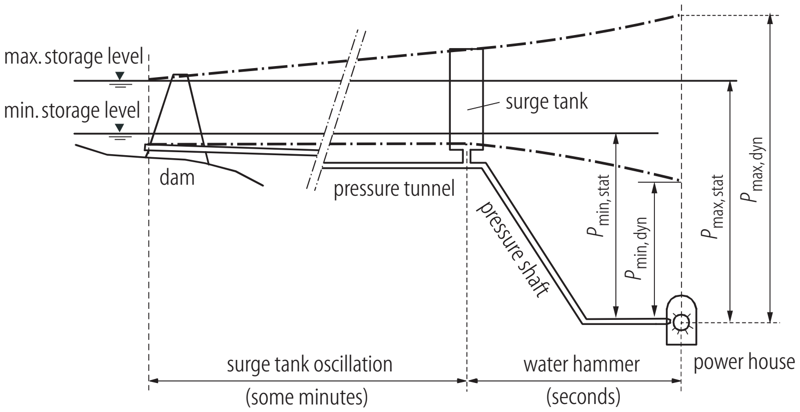


Fig. 2.3.20. Static and dynamic pressure lines.

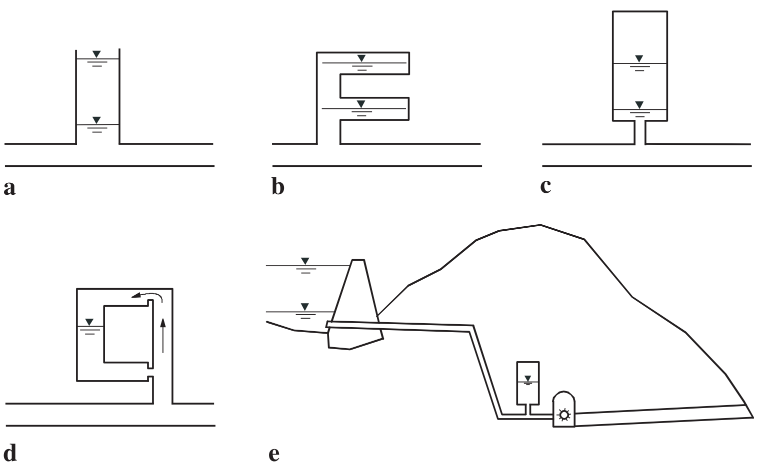


Fig. 2.3.21. Surge chamber types. **(a)** Surge shaft. **(b)** Surge tank. **(c)** Throttle type. **(d)** Differential surge tank. **(e)** Air vessel type.

2.3.4.5 Pressure shaft

Similar to penstocks (see Sect. 2.3.4.6) pressure shafts connect the pressure tunnel and the surge chamber to the powerhouse (see Fig. 2.3.5a). They generally show high internal pressures, whereby areas of moderate internal pressure are equipped with a pre-stressed concrete lining (occasionally also with a plastic membrane) and areas with high internal pressures with steel lining (see Sect. 2.3.4.3.3) [82Wit]. Compared to penstocks, pressure shafts have the advantage of being protected against falling rocks, avalanches and sabotage and do not require external corrosion protection which leads to low maintenance cost. If the pressure shaft is designed deep in the rock and the internal pressure is taken by both the steel lining and the rock, it is at most as expensive as or even cheaper than a penstock.

Apart from structural design for the internal pressure (see Sect. 2.3.4.3.3) the external water pressure has to be particularly taken into account. Constructive measures have to prevent that the total external water pressure, which equals the crack water pressure at the top end of the pressure shaft down to its bottom end, builds up in the possible gap between steel lining and backfill concrete.

The pressure shaft can be constructed as vertical shaft or inclined shaft (see Fig. 2.3.22). A vertical shaft is typically built utilizing the raise boring system; in the case of an inclined shaft up to a gradient of 45°, a TBM is used. Shafts with higher gradients are constructed according to the D&B method. However, gradients of 35° or more render the placing of the steel lining and back filling rather difficult. Typically single steel lining pipes are introduced from the top to the bottom which is why the steel lining is built from the bottom to the top. Even if no compound bearing with the rock is given, it is recommendable to grout the shrinkage gap between concrete and steel lining after its back filling with concrete in order to secure corrosion protection and to avoid the development of excessive external water pressure.

2.3.4.6 Penstocks

Before modern tunneling techniques had been developed, penstocks were the most common connection between the pressure tunnel or surge chamber and the powerhouse (see Fig. 2.3.5). Nowadays they are used when favorable topographic conditions are given and/or a pressure shaft has to be designed without compound bearing with the rock.

Penstocks can be constructed on the surface, in a ditch or enclosed in concrete, and in a pipeline tunnel. Disadvantages connected to penstocks on the surface are the danger of falling rocks, avalanches and sabotage as well as the necessary costly corrosion protection. Furthermore, penstocks on the surface or in a ditch are exposed to the danger of land slides and settlements. Additionally it is important to avoid internal icing when the high pressure power plant is out of operation. Penstocks in a pipeline tunnel are preferably used for water conduits which have to be within a tunnel and have a minimum cross section (approx. $10 m^2$) which is substantially larger than the necessary cross section of the penstock.

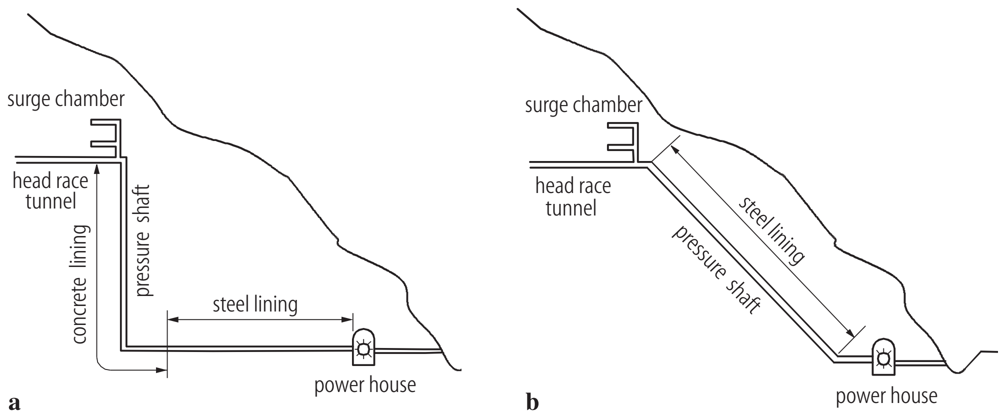


Fig. 2.3.22. Types of pressure shafts. **(a)** Vertical shaft. **(b)** Inclined shaft.

Materials used for penstocks are

- steel suitable for high internal pressure,
- ductile steel casting for small diameters and internal pressures up to 40 bar and
- glass fiber reinforced pipes for diameters up to ca. 2.0 m and internal pressures up to 20 bars.

In the following penstocks constructed on the surface will be discussed in detail. The standard penstock design is characterized by subdividing the penstock according to fix points (thrust blocks) at horizontal and vertical changes in direction of the location line as well as at sections which are too long (≤ 150 to 200 m) in order to avoid temperature strain movements (see Fig. 2.3.23). In between these fix points or thrust blocks, the penstock sections are movably supported by individual bases every 10 to 20 m. At the upper end of each subsection defined by two thrust blocks a dilatation is disposed in order to take up the deformation extensions caused by temperature variations.

For the construction of penstocks, high-strength, brittle fracture safe, notch toughness types of steel are utilized. The structural analysis of penstocks incorporates static loads and fatigue resistance as well as fracture mechanic requirements due to dynamic loads caused by power and frequency control. The individual pipe for example is exposed to static loads comprising internal pressure, dead load, water load of the filled penstock and friction forces occurring in the supports due to temperature extensions as well as dynamic loads including earthquake forces and pressure variations caused by power and frequency control. In accordance with the internal pressure, the plate thickness of the pipes is gradually (stepwise) strengthened from the top to the bottom. However, the plate thickness is limited by the technology used for manufacturing the plates as well as the welding capability of thick plates. The maximum plate thickness is approximately 60 mm, occasionally also more. Pipe diameters are limited by the maximum possible internal pressures and a feasible plate thickness. Additionally, pipes produced in the factory should not exceed a diameter of 3 to 3.5 m in order to be transportable on the road or by rail. Therefore it may be necessary to install several parallel pipelines.

The pipes are movably supported by individual bases and are loaded by the dead load, the water load in the pipe as well as friction forces and earthquake forces. In order to limit bending stresses lengthwise of the pipe, suitable distances between the individual bases range from 10 to 20 m. In the thrust blocks hydraulic forces due to the change of direction and longitudinal forces due to friction in the supports and expansion joints caused by temperature extensions as well as earthquake forces are transmitted to the ground. The hydraulic forces due to the change of direction comprise hydrostatic forces and forces due to impulsion. If stability can not be achieved by the dead weight alone, the installation of pre-stressed anchors is necessary.

2.3.4.6.1 Penstock manifold

The penstock manifold serves as connection of the water conduit (pressure shaft, penstock) to the individual turbine and is made out of steel only. Right ahead of the turbines each feeding pipe is equipped with a valve (see Fig. 2.3.15) which in most cases has a form of a spherical valve due to the high pressures exerted. The penstock manifold is exposed to hydraulic forces, forces from the valve, extensions due to temperature variations as well as internal pressure. It is important to avoid that any deformations from the penstock manifold are transferred to the turbines (see Sect. 2.7).

The penstock manifold of an underground powerhouse or shaft powerhouse is generally constructed in the rock which also takes up the hydraulic forces as well as the forces from the valve and facilitates the control of extensions due to temperature or load. The penstock manifold of an open air powerhouse can either be imbedded in concrete or constructed on the surface. If it is imbedded in concrete, the concrete which may also be the foundation of the powerhouse fulfills the function of the thrust blocks. In the case of penstock manifolds on the surface, a thrust block is needed right in front of the penstock manifold and constructive measures have to secure that the turbines are free of constraining forces. This can be achieved by providing expansion joints and allowing bending stresses in the penstock manifold.

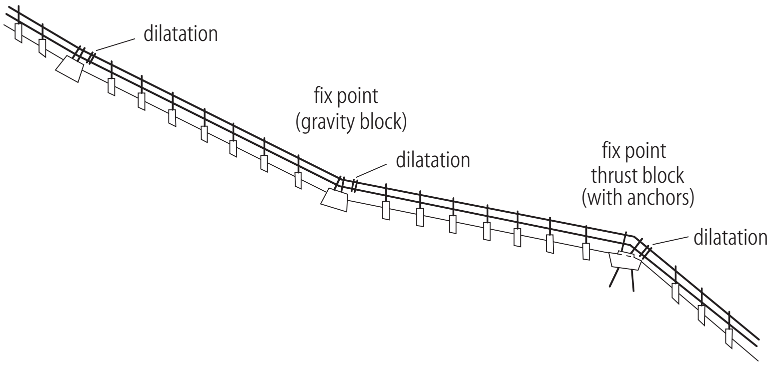


Fig. 2.3.23. Penstock.

2.3.5 Powerhouse of a HPPP

2.3.5.1 General arrangement of the powerhouse

The power house is the part of the hydro power plant which houses the turbines and the generator unit. Here, the actual power generation takes place. The water is led from the penstocks over the adjoining penstock manifold to the turbines. From here the used water is returned to the original riverbed by tail water races of different length. In order to achieve a surge reduction in the tail water channel, a balancing basin has to be installed before final restitution.

The range of water head of a HPPP is between (several) 100 m to almost 2000 m. Therefore, in plants with lower head Francis or Pelton turbines can be used (see Fig. 2.3.24a), whereas in plants with high head only Pelton turbines can be applied (see Sect. 2.7). Pelton turbines require an open channel flow downstream, must not be submerged from the tail water and therefore need a higher elevation of the turbine axis (see Fig. 2.3.24c). For an ideal mode of operation, Francis turbines need a certain tail water head and call for a low position of the turbine axis. Francis turbines in deeper elevation with an adjoining draft tube can be used with largely varying tail water levels. If this requires an especially low position of the axis of the turbine, the construction in a deep shaft is recommended (see Fig. 2.3.24b). This is often used in combination with pumpwater storage plants (see Sect. 2.6). A roll gate is often installed in a tail water shaft for shutting off the downstream flow. Both types of turbines can be installed either horizontally or vertically:

- *Horizontal-shaft turbines*
  Francis turbines with horizontal-shaft are no longer up to date and therefore only recommendable for smaller plants. The major disadvantages are the lacking flood-security, an unfavorable static load bearing behavior and a larger need for ground floor area compared to vertical-shaft turbines. Pelton turbines with a horizontal shaft are often built in combination with one- or two-jet wheels. One advantage lies in a special construction when two machines and one generator lie on one shaft. The construction is low in height and there is generally a good accessibility for maintenance and revision. Hydraulically, there are no disadvantages compared to a vertical position.
- *Vertical-shaft turbines*
  With a vertical axis, the power house is divided up into the substructure and the superstructure. Integrated into the substructure are the draft tube or the tail water pits and the turbines with the manifold and the valves. In the superstructure is the generator and the machinery hall. Pelton turbines with vertical shafts are mostly used in multijet (from two up to six) wheels. In order to achieve a hydraulically favorable solution, the inlet bend at the nozzle should be given a sufficiently long radius. This means that it requires more space in comparison to a Francis turbine with a vertical shaft. However, there are advantages because Pelton turbines have a wider range of adjustment.

Before any construction planning, the geological and rock mechanical conditions of the foundation have to be investigated carefully in order to ensure an optimal position of the plant from all possible operational, static and economical perspectives [92Mah].

Within the power plant, numerous mechanical and electrical equipment has to be placed near the turbine and the manifold: generators, transformers, hydraulic valves and gates for inflow and outflow; spiral cases and (if applicable) a draft tube, manuals, aggregates for regulating and controlling; cranes and workshops for the assembly, installation and maintenance as well as necessary areas for depositing parts during revision and finally all the operational and social auxiliary rooms. For the conception, the choice of the main power connection to the grid plays a key role and requires additional security measures and accessibilities, especially in underground power stations. Besides the actual machinery equipment, a well-to-do and safe operation of all parts both for the primary installation and for the further inspection and maintenance is decisive for the size of the power plant. The required cranes have to be designed according to the largest and heaviest parts and therefore influence the main length, width and especially the height of the plant. Generally, protection of all electrical and electromechanical parts against high water levels deriving from flood situations has to be taken care of, which can be achieved either by a relatively high and therefore safe arrangement or by a safely executed sealing against the ground water pressure. To remove seepage from the plant, a dewatering system with drainages and pumps has to be installed which has to be situated on the lowest point of the power plant.

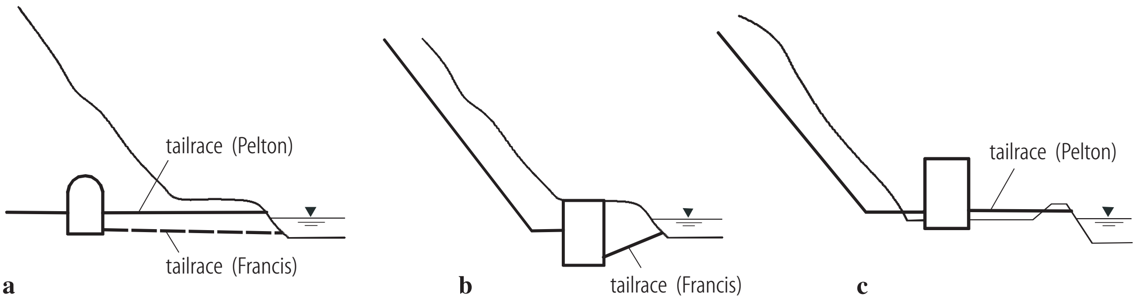


Fig. 2.3.24. Schematic types of powerhouses. **(a)** Underground powerhouse. **(b)** Shaft powerhouse. **(c)** Open air powerhouse.

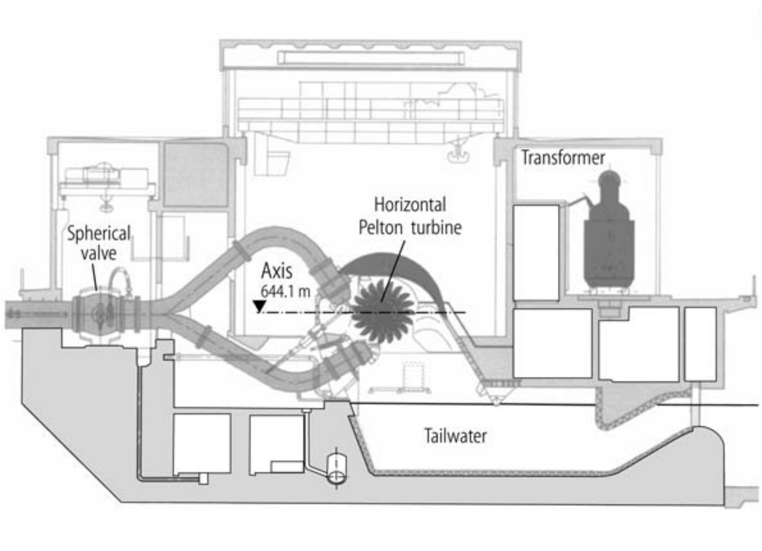


Fig. 2.3.25. Conventional powerhouse of the HPPP Mayrhofen, Austria, with a two-jet Pelton turbine.

2.3.5.2 Open air powerhouse

Those types of plants (see Fig. 2.3.24c) are used when there are sufficient space and appropriate conditions for the foundation. By an attractive architecture, conception and a minimization of the volume of the building (resulting in less needed concrete), an economical powerhouse can be obtained (see Fig. 2.3.25). A construction built into or adjacent to a slope may have additional advantages, if, by this construction, the foundation can be set up on a rock.

2.3.5.3 Underground power stations

Underground hydro power stations (see Fig. 2.3.24a) are always considered if there is a need for a deep turbine axis, if there is not enough space for an open air power station or if natural dangers (avalanches, landslides, rock fall etc.) or other security aspects hint at its favorability. However, also the functional requirements in high pressure plants can encourage an underground power station, especially to bear high static and dynamical forces. Sometimes only turbines, generators and valves are placed underground. Additionally, also the transformer and the switch-yard can be situated in caverns that are detached from the main cavern or in the machinery hall. In very large plants it can become geologically necessary to build a separate cavern for the valves and gates in order to restrict the dimensions of the various caverns [92Mah]. Depending on their arrangement, three types of underground power stations are distinguished:

- *Alpine type*
  Downstream station characterized by a long, almost horizontal pressure tunnel, a steep or vertical pressure shaft and a generally very short tailrace after the powerhouse-cavern, requiring appropriate topography. In this section, mainly this type will be explained in detail.
- *Swedish type*
  Deep upstream station characterized by a short head race with pressure shaft and a long tailrace tunnel with a surge chamber downstream of the powerhouse at the entrance to the tailrace tunnel.
- Intermediate station, characterized by a relatively long pressure tunnel, a short shaft to the powerhouse and a long tailrace tunnel with a surge chamber at the beginning, following the low points in the terrain due to special topographical conditions.

The main reasons for setting up an underground power station are:

- No consumption of land in the valley;
- No restriction of the landscape and environment;
- Favorable load bearing behavior and distribution of forces;
- Protection of the power plant from avalanches, rock fall, landslides, climatic extremes in a high alpine region;
- Operational safety and good protection of the complex against military or terrorist attacks;
- Economical construction due to modern rock mechanic technologies.

Very often, the need for a deep turbine axis, the consumption of land, the environmental protection and the economic viability are the crucial requirements when it is decided to construct an underground power station. However, it is hard to objectively assess the careful treatment of nature on a monetary basis.

The economical issues of an underground power station are as follows:

- Generally, the construction costs per m³ construction volume are higher, especially if parts of the powerhouse could be cheaply constructed open air otherwise.
- Additional costs from the construction of access galleries, safety galleries, air ducts, air-conditioning, connecting galleries, drainage galleries, cable and/or bus galleries and the like which would not apply otherwise.
- The construction of a separate cavern for the transformers is usually more expensive than the open air set up.
- Larger operating costs for light, air-conditioning, ventilation and drainage.

However, these issues have to be seen in relation to the higher safety of an underground plant. A complete listing of contributing factors is generally not possible; however, prior to any planning of a HPPP an extensive study of variations comparing all advantages and disadvantages concerning the construction, operation, economical and social issues should be carried out,.

Due to very high standards in the geological exploration and evaluation of rock mechanical conditions, the expected states of tensile and shear stress as well as advanced methods of excavation and of support of the of caverns, very large dimensions have been reached in the construction of underground power plants (width > 35 m, height > 60 m, length > 200 m) [87Bro]. For the stability of those underground caverns the orientation of the cavern axis with respect to the direction of natural stress vectors, axes of folding and fracture discontinuities is of great importance. In the rock mechanics the aim is to achieve a unity between the excavation and the surrounding rock by taking advantage of the supporting rock. Because rock structure is often inhomogeneous and anisotrope, certain primary residual stresses can often not be fully estimated and the calculated results for the applying support measurements have to be changed accordingly during the excavation. Monitoring of the displacements during the excavation and in long-term observations is an absolute requirement that enables an economical excavation and yields the proof for the rock stability.

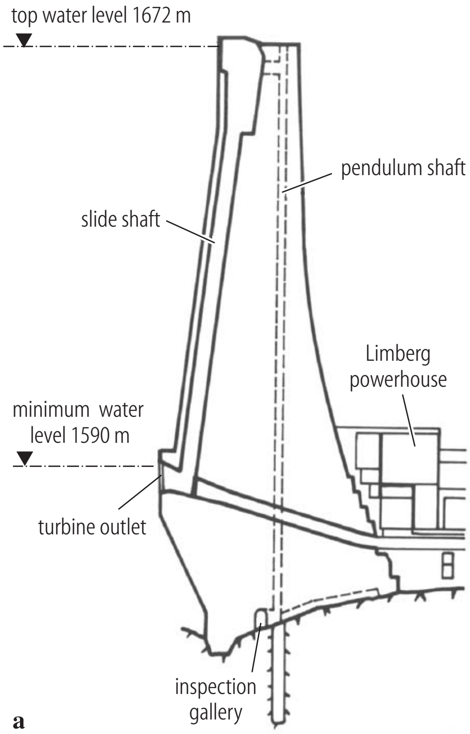
Pre-stressed anchors are a very good and safe support method as the pre-stressed force required during the excavation and afterwards can be adapted and corrected any time. The variation of the state of stress which appears in the excavated cavern is monitored by anchors with load measuring devices which are installed like usual anchors immediately after the excavation. These measuring devices are installed on the head of the anchor. Hence, a constant control during the period of stabilization as well as for the entire service life of the structure is given. For ensuring the pre-stressed anchors' long-term functioning, a perfect protection against corrosion of the steel tendons, a free movement of the tendon and a grouted borehole free of cracks are inevitable.

After the excavation of the cavern is completed, the concrete works are carried out and all further mechanical and electrical parts of the power plant are installed. It is highly recommended to provide a sufficient lag time between excavation and installation of M&E parts to allow the deformations to decline to a marginal extent.

2.3.5.4 Powerhouse at the toe of high dams

The particularity about power stations located at high dams is that the usage of an average water head (50-200 m) with very short head race tunnels, successively causing low hydraulic losses, is made possible by a power house which is situated directly at the toe of a dam. There are basically two different possibilities for the exact location of the powerhouse: either the powerhouse is separated from the dam or a design is chosen where the power house is integrated into the dam body or directly attached to it (see Fig. 2.3.26a). Besides the low water head, the first possibility technically allows all types of power plants that have been described (open air, underground). Therefore, this section will only deal with the types of power plants which are mostly attached to concrete dams. With earth and rockfill dams, the integration is in most cases only possible as a variation with a gravity dam section or in the adjacent downstream banks. Hereby, special attention has to be paid to the sealing at the transition of different construction methods.

In very narrow valleys the head race tunnels and the power house as well as the spillway or the bottom outlet can each be situated sideways (e.g. Hoover dam , see Fig. 2.3.26b) above ground or underground. The power intake of power plants at the toe of a dam (some of which have a very large diameter) can either be led to the turbines with short pressure pipes through the body of the dam or with pressure pipes through the adjacent steep downstream rock slopes. Furthermore, there is the possibility to build separate intake structures such as towers for power intake and to convey the water to the turbines via short power conduits.



a

b

Fig. 2.3.26. Powerhouse located at high dams. **(a)** Limberg arch dam, Austria. **(b)** Hoover dam, USA.

2.3.5.5 Tailrace

According to the type and arrangement of the power plant various tailrace systems are possible:

- In open air power plants the tailrace is mostly designed as an open channel flow. This mostly trapezoidal channel can either be covered, uncovered, or partly covered when designed as a syphon or crossing other infrastructures (roads, rails). Any kind of crossings by bridges is also possible. An open channel flow can only be realized if there is a wide tunnel and a short tail water conduit.
- In underground power plants the design of the tailrace depends on the type of the turbine (see Sect. 2.3.5.1), the size of the rated discharge and – very crucial – on the length of the tailrace. Also, the different height of the water level in the tail water influenced by flood situations and low discharge in the river is an important factor.

In order to avoid damage due to the water hammer in long tailrace tunnels, it can become necessary to design surge tanks downstream the turbines. They can either be carried out as expansions in the gallery or as vertical shafts. Very rarely, a throttle is applied. In any case, a sufficient aeration system is required.

If the flow under pressure is nearly constant, a tailrace tunnel can be fairly long. Large variations of the water level downstream are relatively unproblematic. Special attention is required if the open channel flow and flow under pressure are changing depending on the height of the water level of the river downstream. This, however, should be avoided as it causes a discontinuous flow as well as vibrations and uncontrolled pulsations [94Nau]. The tunnel size should be dimensioned accordingly. As a rough guideline for dimensioning economical cross sections and low water head losses, one can anticipate a flow velocity of about 2 m/s in an open channel flow and 4-5 m/s for discharge under pressure.

In order to guarantee a fairly constant tail water level, the reinforcement of the riverbed downstream is important. This can mostly be achieved by a fixed sill in the structure for restitution. As a design principle for the surge limitation in the river bed when returning the water, a balancing basin or reservoir with a controlled discharge into the recipient can be arranged.

In conclusion, it is to point out that tail water conduits are artificial buildings with changing discharge according to the specific plant operation. Since today's mode of function of high pressure plants has rapidly changed, one can encounter changing conditions at any time. In those cases it is necessary to provide a balancing basin or reservoir or to set up protections and warning signs at the banks of the river in order to guarantee that nobody is surprised or endangered at any time.

2.3.6 Economical aspects

2.3.6.1 Investment costs

In comparison to river power plants or thermal power plants, HPPPs have rather high investment costs. They consist of construction costs of the individual components such as the reservoir with dam, headrace structure (headrace tunnel and penstock), the powerhouse with electro-mechanical units and the tailrace structure with tailrace balancing reservoir, power cables and switch yards, but also of the escalation of prices and the interest accrued during the construction period. Due to the relatively high construction costs of some of the components, both price escalation and interest during construction can be very high. It is therefore necessary to optimize the construction schedule and to use state-of-the-art construction methods in order to minimize the time that is spent on building the HPPP at the critical path and to ensure a well-organized sequence of the individual construction steps.

It is difficult to specify average specific construction costs for HPPPs in €/kWh (investment costs/annual production) or €/kW (investment costs/installed capacity) as they depend upon the most cost-intensive components which differ from plant to plant. The specific construction costs (depending on the installed capacity) of HPPPs built in Central Europe in the last couple of decades range from 600 to 1100 €/kW. The specific construction costs depending on the annual production also vary considerably between 0.70 and 1.50 €/kWh. A value of 0.70 €/kWh is possible when a large reservoir can be built with only a small dam and a large head can be achieved with short water conduits. The mentioned rates only allow a rough comparison of HPPPs to other competitive types of power plants and can not be used for estimating the potential construction costs of new power plant projects.

The specific costs of reservoirs impounded by dams (investment costs/volume of the reservoir) built so far have amounted from 1 to 2 € per m³ of stored water. The more suitable the location of the dam and the better the topographic characteristics of the reservoir, the lower are the costs per m³ of stored water.

The costs of the pressure tunnel depend above all on the diameter, sealing requirements, geological conditions and overburden. One meter of a pressure tunnel with an outside diameter of 6.25 m, a length of approximately 20 km and an average 20 cm of concrete lining costs e.g. 3500 € per running meter; a pressure tunnel with an excavation diameter of 4.2 m, a length of 30 km and precast segment lining 3000 € per running meter.

The costs of a pressure shaft with an inside diameter of 2.70 m, a length of 1350 m and a maximum internal pressure of 800 mWc amount to 8500 € per running meter.

The costs of a powerhouse on the surface, a shaft powerhouse and an underground powerhouse for power plants are in the ratio of approximately 1:1.1:1.2. As a standard of comparison it can be assumed that the specific construction costs (investment costs/installed capacity) of an underground powerhouse amount to approx. 300 €/kW.

All the above mentioned figures are only approximate values for investment costs of high pressure power plants. Investment costs of the individual project can widely differ from these figures.

2.3.6.2 Costs for operation and maintenance

It is a major advantage of high pressure power plants that no costs for fuel accrue during daily operations. Costs for operation and maintenance are therefore relatively low. Depending on the complexity of the individual power plant, annual costs for operation can range from 0.5 to 1.0% of the construction costs. Such operating costs include non-recurring costs which accrue in the course of long time intervals such as general revisions of machine units. Generally it can be assumed that these costs incur every 15 to 20 years and equal 0.25% of the purchase cost of hydraulic and electric machines. Costs for maintenance of constructional components are included in the above mentioned figures and are relatively low. Cost for protective measures against corrosion at the steel components of the power plant and steel linings as well as maintenance measures at the valves are periodically recurring cost and are included in the above mentioned numbers. High cost for constructional maintenance can accrue if measures against sedimentation of the reservoir (see Sect. 2.3.7.1) have to be taken. This is not included in the above mentioned figures.

2.3.6.3 Service life

When talking about the service life of a high pressure power plant, one has to differentiate between the actual service life and depreciation periods. The actual service life is substantially longer than depreciation periods and also depends on maintenance standards. For constructional components (e.g. dams, pressure tunnels) it can be assumed that service life is at least 100 years or more. Service life of electromechanical units can be between 40 and 50 years with adequate maintenance and service. Depreciation periods differ from component to component of the power plant, but also from economy to economy. In general, depreciation periods for constructional components are 50 years, and 30 years for electromechanical units.

2.3.6.4 Energy costs, price

Energy costs comprise fixed costs such as depreciation or interest on equity capital and variable costs for operation and maintenance. While the fixed costs can not or should not vary, costs for operation and maintenance depend to a large extent on the operation mode and the type of service to the grid (base load, power and frequency control, etc.) as well as the requirements on the availability of the power plant. High availability and frequent change of operation mode lead to elevated costs for operation and maintenance. Therefore, energy costs for energy produced by high pressure power plants vary substantially and can range from 2.5 to 5 €-ct/kWh in Central Europe, depending on the energy quality.

Balancing power, which is dispatched from contractually bound power plants, is tendered in form of a control band of ± *xy* MW. At the beginning of 2003 market prices for balancing power were around 50 €/kW. In economies where a liberalization of the energy market has taken place, costs for operation and maintenance depend to a high degree on the prices that can be achieved on the market or at the stock exchange and have in part dramatically declined over the last couple of years.

2.3.7 Further aspects of HPPPs

2.3.7.1 Sedimentation

One key issue for the irreproachable function of a high pressure plant is sediment management as reservoirs of HPPP highly intervene into the natural bed load transport. The sediments entering the reservoir are being deposited and can mostly not be transported on without much effort [83Gra]. Hence, reservoirs are silted up (see Fig. 2.3.27) and thereby reduce the active storage [02Sch]. This should be taken into account when planning the HPPP in order to keep the constraints for the functionality and the appurtenant structures for operation like power intake and bottom outlet as small as possible. With the exception of high alpine or mountainous regions, there are hardly any constructions where – in view of the life time of a structure – no additional measures are necessary. A corresponding sediment management conception should be pursued already in the design stage of a construction.

A very effective, however in many cases not feasible measurement is to prevent sediments from entering the reservoir either by management with sediment shut offs or by sufficiently large settling basins and sediment reservoirs upstream the operation reservoir. In the best case, the size of those basins matches the service life of the plant. Additionally, the dead storage volume of a reservoir provides a similar function as long as the intake structure and the bottom outlet remain operational.

Lately, there are endeavors in coordination with the machinery conception to directly transport parts of the suspended load during operation via the turbines into the tail water. Here, one has to compare the increased wear and tear at the machinery and the shorter intervals between revisions with the costs for other desilting measures. In smaller intake structures which additionally have an inflow by diversions leading to the main reservoir, desilting structures [93Rau] can depose the bed load and regularly lead it directly back to the river by sluicing or flushing. Such regular flushings are also extremely useful for the reservoir itself, even though a complete desilting of the reservoir can hardly be achieved. However, the important parts of the operation structures can be permanently protected from sedimentation.

As the last and very costly solution, one has to mention the dredging method. The dredging takes place with the aid of sonic depth finders and GPS-control. The depth for usage is limited, though. Nevertheless, the problem of the definite deposit of the sediment still occurs [02Pri].

2.3.7.2 Effects of a HPPP on the ecology

Each larger building has got certain effects on its environment, also hydro power plants. The impacts concern the socio-economical but also the natural environment. One has to distinguish temporary impacts during the construction period from permanent impacts. For the designer and the engineer the assignment is to determine those impacts and to keep the effects as small as possible or avoid them by employing counter measures or, if necessary, adjust or even compensate those impacts. One of the key elements to include all potential effects is the composition of a comprehensive matrix which presents the measures and their consequences in the realization of the HPPP. Detailed collection of evidence should be initiated long before the project, especially in the field of water management (groundwater, wells and springs, water consistence and quality).

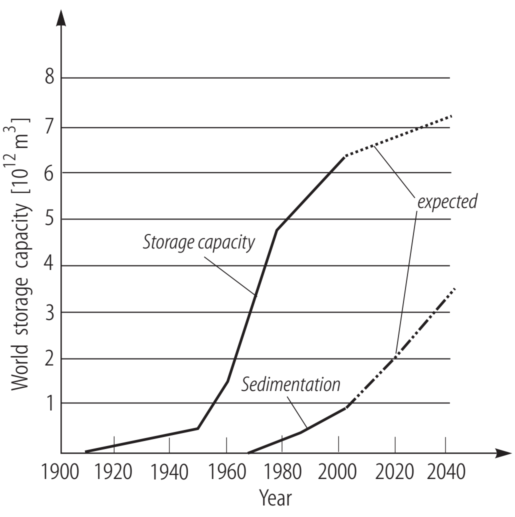


Fig. 2.3.27. Worldwide growth in reservoir storage capacity and sedimentation.

One of the major effects when building a reservoir for the HPPP can be the loss of settlement area and agricultural land. If it is necessary to relocate inhabitants in the course of building a HPPP, compensation or reparation for the parties involved have to be secured and agreed on.

Since, from today's point of view, the effects on the aquatic and terrestrial nature are of great interest for the communities, those major influences will briefly be outlined. The largest impact surely derives from the dam and the reservoir which are, hence, artificially created. The alteration of the regime of the natural flow, the changes of the natural flow throughout the year, the diversion of the inflow into another catchment area and the residual flow below the dam have to be considered. Also, geophysical aspects like the sediments within the reservoir or the downstream lack of bed load, the stability of the adjoining river bank, a possible elicitation of micro-organisms by the water mass of the reservoir or the pressure of the water which infiltrates into gaps have to be kept in mind.

The operating mode of high pressure plants often results in surges when the tail water is returned. Those frequent quasi-flood situations can have lasting effects on the limnology. They can be avoided by balancing basins with control structures for returning the water into the river. The preservation of the continuity of rivers by fish ladders for up- and downstream, the sufficient charging of sections with residual flow and the impact on the ground water regime were already been described in Sect. 2.2. When very large reservoirs with a large water surface are created, one has to regard the effects on the immediate climate in the near surrounding.

Further aspects of a HPPP are the construction of artificial water ways, the construction of large dams with large water reservoirs behind them and the coherent interventions in the water resource management (seasonal fluctuations, surge). The construction of reservoirs, tunnel systems and channels can have an effect on the rock water pressure which must not be neglected. Therefore, sufficient hydrogeological investigations are necessary to design effective measures in order to avoid such impacts.

Finally, a reasonable consideration of interests is necessary. The juxtaposition of all economical and structural benefits including the aspect of minimizing unwanted effects eases the decision. Generally, one can conclude that, apart from special cases, the effects of large water power projects are much smaller in comparison to several smaller projects with an equal total energy production. In order to comprehend all aspects and their complex interrelations, it is beneficial to work interdisciplinary and to compile an assessment of environmental effects.

2.3.7.3 Reflections on flood dimensioning

One of the most important security features for dams of HPPPs are the spillways. They guarantee the secure through-passing of the maximum design flood without endangering the stability of the dam construction [85SWI]. There are most different methods and guidelines for calculating this maximum value. They are mostly based on known flood incidents and the causing rainfalls (Probable Maximum Perception, PMP). The static or deterministic projection of a several millennia incident provides the assessment value for the capacity of the spillway. Worldwide, extreme floods are defined as 5000-, 10000-yearly or as Probable Maximum Flood (PMF).

The consideration of the various different interdependences of the catchment area's size and their characteristic, the climate zone, the quality of the subsoil, the pre-saturation of the soil, the duration and intensity of the rainfall and numerous further aspects shows that the projected maximum values for the design flood always have a relatively large varying field. Therefore, certain security considerations are necessary in which the type of the dam (concrete or fill dam) is very important. An internationally recognized norm suggests that, in the case of a catastrophe, a construction has to sustain, but damages can be accepted.

2.3.8 Examples of HPPPs and PPs at the toe of a dam [98ICO], [02WAT]

Name of plant or dam	Country/ Code	River	Year of comm.	Installed capacity [MW]	Turbine type (Pelton/ Francis)	Gross head [m]	Mean annual prod. [GWh]	Type of dam	Storage capacity [$10^6 \cdot m^3$]	Pur-pose	Max. height [m]	Crest length [m]
Guri	VEN	Caroni	1986	10300	20F	140	31630	PG/ER/TE	111104	H,I	162	7426
Itaipu	Bra/PRY	Parana	1983	12600	18F	126	93400	ER/PG/TE	29000	H,I	196	7297
Grand Coulee	USA	Columbia River	1988	6480	24F	115	20215	PG	11600	H,I	167	1592
Ertan	CHN	Yalon	1999	3492	6F	165	3900	VA	5800	H	240	775
Svartisen	NOR	Storglomvatn	1997	350	1P	543	2170	ER	3470	H	125	820
Yele	CHN	Lianchahe	2006	240	2P	628	1135	ER	298	H	126	411
Reisseck	AUT	Möll, Drau	1961	67	3P	1772	307	PG	17	H	46	433
La Miel	COL	La Miel	2001	405	3F	235	1135	RCC	565	H	188	341
Agua Prieta	MEX	Moctezuma	1989	240	2P	518	n/a	VA	1426	H	200	80
Hintermuhr	AUT	Mur	1991	68	2P	601	67	ER	15	H	45	273
Gepatsch	AUT	Faggenbach	1964	392	5x2P	861	661	ER	138	H	153	600
Oscheniksee 3	AUT	Fragant	1979	108	3P	1186	82	ER	33	H	116	530
Sellrain-Silz	AUT	Ötz, Inn	1981	292+500	2F/2P	394/1250	719	ER, TE	60+3	H	149/45	652/407
Amaluza	ECU	Paute (C)	1986	125	1P	667	n/a	VA/PG	120	H	170	400
Grande Maison	FRA	Eau d`Olle	1985	1800	8P	950	1420	RF/TE	140	H	160	560
Terror Lake	USA	Terror	1984	14	2F	385	n/a	ER	138	H	59	747
Three Gorges	CHN	Changjang	2009	18200	26F	80	84700	PG	39300	H,I,F,N	175	2310
Atatürk	TUR	Euphrat	1993	2400	8F	154	8100	ER	48700	H,I	169	1664
Berke	TUR	Ceyhan	2001	513	3F	186	1700	VA	427	H	201	270
Karakaya	TUR	Euphrat	1987	1800	6F	147	7354	VA,PG	9380	H	173	462
Sima	NOR	Sima	1980	260-350	4P	1158	2700	ER	660	H	57	320
Bieudron	CH	Grande Dixence	1998	1269	3P	1874	2000tot	PG	400	H	284	695
Churchill Falls	CAN	Churchill	1971	5428	11F	313	35000	TE	32640	H	32	5506
El Cajon	HND	Humuya	1985	300	4F	180	n/a	VA	6500	H,I	234	382
Cirata	IDN	Citarum	1988	1000	8F	107	n/a	ER	3165	H	125	453
Pigae	GRC	Aoos	1990	230	2P	677	n/a	ER	260	H	78	300
Grosio	ITA	Grosio	1960	400	4P	588	n/a	CB	1.2	H	51	286
Soutelo	ESP	Cenza	1993	130	1P	606	n/a	RCC	43	H	49	609
Bissorte III	FRA	Bissorte	1981	156	1P	1186	n/a	PG	39	H	63	545
San Carlos	COL	Guapape	1986	1300	8P	1130	n/a	TE	72	H	70	800

Legend: VA: Arch dam; RCC: Roller compacted concrete; CB:Butress dam; ER: Rockfill dam; TE: Earth dam; PG: Gravity dam
I: Irrigation; H: Hydropower; F: Flood Control; W: Water supply; N: Navigation; n/a: not available

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