Chapter 1

Practices in Constructing High Rockfill Dams on Thick Overburden Layers

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Additional information is available at the end of the chapter

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Abstract

Rockfill dams are very widely constructed all over the world due to their good adaptability to diverse geological and geographical conditions, and their relatively low cost compared to other dam types. However, natural satisfactory sites are increasingly difficult to find in many countries due to past dam development. In some circumstance, building dams over thick overburden layers is unavoidable. In this chapter, Chinese practices in constructing high earth and rockfill dams over thick overburden layers are reviewed. The geological and geotechnical investigation techniques are briefly summarized, and seepage control systems of some selected cases as well as the connection of the impervious systems of both the dams and their foundation layers are described. Commonly used foundation improvement techniques are also presented, followed by simple descriptions of aspects that require further research and development.

Keywords: rockfill dam, overburden layer, seepage control, foundation treatment, in-situ test

1. Introduction

Various types of dams have been increasingly constructed all over the world for irrigation, flood controlling, power generation, environment protection, etc. [1]. Normally, most dams are preferentially built on rock foundations where seepage control is not a very difficult task. However, with the exploitation and exhaust of natural satisfactory dam sites, many new dams have to be constructed on thick overburden layers, as better sites are not available and removal of the existing overburden is technically or economically unfeasible. This adverse situation is often encountered along many hydropower-rich rivers in the southwest and northwest regions of China [2, 3]. When such thick overburden foundation layers can neither be avoided nor removed, a rockfill dam is often a priority due to its excellent adaptability to such geological conditions. In recent years, more than 50 high rockfill dams, including earth core rockfill dams (ECRDs), asphalt core rockfill dams (ACRDs) and concrete faced rockfill dams (CFRDs), have been constructed in China, as selectively listed in **Table 1**. Challenges can be seen from both the height of these dams and the thickness of the overburden layers.

Technical problems requiring special attention in the design and construction of rockfill dams over thick overburden layers include, but are not limited to, the following aspects:

a. *Shear strength and deformability of load-bearing layers*. The shear strength of underlying foundation layers influences the overall stability of the dam, while the deformability of these layers controls not only the deformation of the dam but also the deflection of the cutoff

No.	Name	Type	Year	H_{max} (m)	T_{max} (m)	Cutoff wall		References
						Width(m)	Height (m)	
1	Shiziping	ECRD	2010	136	102	1.2 × 1	101.8	[4]
2	Xiaolangdi	ECRD	2000	160	80	1.2 × 1	82.0	[5]
3	Pubugou	ECRD	2010	186	75	1.2 × 2	78.0	[6]
4	Changheba	ECRD	/	240	79	1.4 × 1 & 1.2 × 1	50.0	[7]
5	Maoergai	ECRD	2011	147	57	1.4 × 1	52.0	[8]
6	Shuiniujia	ECRD	2006	108	30	1.2 × 1	32.0	[9]
7	Luding	ECRD	2011	84	148	1.0 × 1	110.0	[10]
8	Qiaoqi	ECRD	2006	125.5	72	1.2 × 1	70.5	[11]
9	Xiabandi	ACRD	2009	78	148	1.0 × 1	85.0	[12]
10	Badi	ACRD	/	97	120	1.0 × 1	105.0	[13]
11	Huangjinping	ACRD	2015	85.5	134	1.2 × 1	113.8	[14]
12	Yele	ACRD	2005	124.5	420	(1.0–1.2) × 1	154.5	[15]
13	Xieka	CFRD	2014	108.2	100	1.2 × 1	86.0	[16]
14	Nalan	CFRD	2006	109	24	0.8 × 1	18.0	[17]
15	Miaojiaba	CFRD	2011	111	48	1.2 × 1	41.5	[9]
16	Jiudianxia	CFRD	2008	136.5	56	1.2 × 1	30.0	[18]
17	Aertash	CFRD	/	164.8	94	1.2 × 1	90.0	[19]
18	Chahanwusu	CFRD	2008	110	47	1.2 × 1	41.8	[3]
19	Duonuo	CFRD	2012	112.5	40	1.2 × 1	35.0	[20]
20	Laodukou	CFRD	2009	96.8	30	0.8 × 1	29.6	[21]

Note: ECRD = earth core rockfill dam; ACRD = Asphalt core rockfill dam; CFRD = concrete faced rockfill dam; H_{max} = maximum height of dam; T_{max} = maximum thickness of overburden; '/' means the dam is still under construction and has not been finished.

Table 1. Basic information of typical rockfill dams built on overburden layers in China.

wall, if used. The inhomogeneity of foundation materials can result in differential and incompatible deformation within the dam and may ultimately lead to threatening cracks.

b. *Permeability and erosion resistance of the overburden layers*. One of the most important functions that should be achieved in dam engineering is the ability to control the seepage within the foundation. Designing an impervious system for the dam foundation depends, to a large extent, on the permeability and erosion resistance of the involved strata and the available foundation treatment equipment and techniques.

c. *Liquefaction potential of the underlying fine layers*. Earthquake is one of the most disastrous natural events that dams are expected to experience. Cyclic shearing by earthquakes can cause excessive pore water pressure to build up in fully saturated sandy soils, leading to a decrease, or even loss, of their shear strength. As a result, uncontrollable deformation can occur in both the foundation and the dam itself and may result in the worst-case scenario of a dam breach.

d. *Connection techniques for the impervious systems of the dam and its foundation*. An effective impervious system means not only successful control of the seepage through the dam and its foundation but also satisfactory performance of the connection points between different impervious components. These points are usually places where parts with different rigidity levels meet and joint, and are therefore vulnerable to cracks and concentrated leakage.

The main challenge in constructing a rockfill dam on thick overburden layers is the design and successful construction of an impervious system for the foundation, accounting for the distribution of the underlying soil and rock layers as well as their physical and engineering properties. In this chapter, the authors review several high rockfill dams built on thick overburden layers in China in order to provide a reference for similar cases that might be encountered in the future. The chapter starts with general descriptions of some frequently implemented geological and geotechnical investigation techniques. Next, seepage control techniques used in some selected cases are introduced. Attention is also paid to the connection techniques for impervious systems used in different kinds of rockfill dams and to the widely adopted foundation reinforcement measures in engineering practice. Directions that deserve further research and development are presented.

2. Geological and geotechnical investigations

Thick overburden layers generally refer to quaternary materials deposited over river beds, including boulders, cobble, gravel, sand, silt and clay constituents. Mixtures of these complex overburden materials are often much more compressible and permeable than an intact rock foundation. Adequate geological and geotechnical investigations on the distribution, thickness and other relevant properties of the soil strata are necessary for the design of impermeable systems and for the preparation of required foundation treatments during dam planning stages. In particular, weak layers, such as sand lenses, soft clays and collapsible loess, should be revealed in these investigations and then properly treated to eliminate safety risks to both the foundation itself and to the overlying dam.

2.1. Geological investigation

Core drilling is the most useful subsurface exploration method for investigating the location, extent and constituent makeup of soil and rock strata at a potential dam site. Nonetheless, core drilling becomes increasingly difficult through overburden layers thicker than 40–50 m because [22]: (1) the existence of unpredictable super-large rock particles; (2) frequent borehole collapse; and (3) uncontrollable loss of drilling fluid. The mud or water used in ordinary drilling-with-casing operations can also make the analysis of core grading difficult or inaccurate due to washing away of fine particles. Some special core drilling techniques were therefore used to get high quality cores in the geological investigations at Yele, Aertash, Xiabandi and other dam sites. They include: (1) double-tube swivel type diamond drilling with a proper rpm (revolution per minute) and pressure and (2) special vegetable gum and powder drilling fluid circulated under proper flow rates to protect the bit, the borehole and the core. Until now, the deepest overburden core drilling conducted in China is at Yele ACRD, where the overburden thickness reaches 420 m [15].

Geophysical exploration methods, such as electrical and electromagnetic methods, seismic procedures, gravity techniques, magnetic methods, and so on, are now increasingly used in dam engineering. These techniques are mostly used to locate the interface between overburden and bedrock and to detect weak layers. Geophysical techniques generally does not directly measure the parameters desired for designing purpose. The vast majority of objectives is inferred from the known geologic data and measured geophysical contrast [23]. That is to say, an inverse solution is sought usually in geophysical exploration, and in most cases, it is the most likely but not necessarily the unique conclusion. Assumptions used in interpreting geophysical contrasts, such as the distinct subsurface boundaries, the homogeneity of materials and the isotropy of material properties, are also, in many cases, at variance with the reality, which may lead to inaccurate and misleading conclusions. Therefore, geophysical methods are almost always used in combination with irreplaceable core drilling. Thereby, results obtained by different methods can be verified mutually and a most reliable judgment can be made.

2.2. Geotechnical tests and interpretation

While geological explorations give overall information on the overburden layers, geotechnical tests and their interpretation yield more relevant parameters for designing. However, systematic laboratory experiments with overburden materials are usually unrealistic due to difficulties in obtaining high-quality undisturbed samples. Although some techniques do exist for sampling (e.g., in-situ freezing [24]), they are generally expensive and only applicable to shallow layers. Therefore, measurement of engineering properties of overburden materials relies more upon in-situ tests as exemplified as follows:

a. *Heavy and super-heavy dynamic penetration tests*. For layers with high relative densities, heavy or super-heavy dynamic penetration tests are usually used, in which a cone-tipped probe is driven into the ground by a 63.5 or 120 kg weight dropped freely from a height of 76 or 100 cm. The number of blows needed to drive the probe into the tested layer for 10 cm is registered and an average penetration per blow is calculated. The data gathered can then be used to estimate the density states, bearing capacities, and moduli of the tested layers. This method has been used in geotechnical investigation of almost all dam sites [25].

b. *Plate load tests*. Plate load tests are performed by loading a steel plate at a particular depth and recording the settlement corresponding to each load increment. The load is gradually increased until the plate starts to sink at a rapid rate. The total load on the plate at this stage, divided by its area, gives the value of the ultimate bearing capacity of the tested soil. Assuming an isotropic elastic behavior of the tested soil, the elastic modulus can also be evaluated. In the under-construction Aertash CFRD, plate load tests were performed with a plate of 1.5 m in diameter, and with the maximum reaction force of about 1000 tons [26].

c. *In-situ shear tests*. Large-scale in-situ shear tests are widely used in field investigations. A shear box of a specific size is compressed into the overburden and then it is pulled, after applying a designed vertical load upon the enclosed soil, by a jack using a high-strength chain. The applied horizontal force and the displacement of the shear box are recorded, based on which the in-situ shear strength of the tested soil is determined [27]. A special advantage of this method is that it can measure the strength of coarse granular materials under extremely low normal stresses, which is otherwise not so easy in triaxial compression experiments.

d. *Pressuremeter tests*. Pressuremeter tests are performed in-situ by placing a cylindrical probe in the ground and then expanding the cylinder to pressurize the soil horizontally. The radial pressure on the soil and the relative increase in cavity radius are measured, from which the in-situ stress strain curve of the soil is derived. This technique is extremely attractive in testing overburden layers because the loading direction is identical to the hydrostatic pressure upon a cutoff wall if it is to be installed. Abundant information can be obtained from this type of tests, such as the in-situ horizontal stress, the pressuremeter modulus, the limit pressure, etc. This technique has been used in many dam sites [22, 28], and the depth has reached a magnitude of 100 m successfully.

e. *Wave velocity tests*. The most widely used wave velocity tests include the down-hole test, the suspension logging test and the cross-hole test. The first two methods require only one borehole and evaluate the wave velocities vertically along the borehole wall. The third test requires at least two boreholes and measures the wave velocities within a horizontal plane. Boreholes for down-hole and cross-hole tests should be carefully cased and grouted to ensure good seismic coupling between the geophones and the surrounding soils. Suspension logging, on the other hand, preferably uses uncased holes. All three methods have been applied to the investigation of overburden layers for foundations in many important projects [16, 25]. The velocity results obtained, especially the shear wave velocities, can be used to evaluate the density states, the elastic moduli and the liquefaction potential of the tested layers.

f. *Permeability tests*. The permeability coefficients of overburden layers, which are needed to design the underground impervious systems, are determined by various permeability tests. Methods may be selected based on the location of underground water table, the enrichment of underground water, and the hydraulic conductivity of the concerned layer. In principle, pumping tests or water injection tests are conducted to evaluate the permeability of highly permeable overburden layers, while pump-in tests are used for relatively less permeable bedrock layers [25]. Permeability tests are indispensable for almost all dam projects.

Most of in-situ geotechnical investigation techniques listed above require high-quality pre-drilled boreholes. Unfortunately, this becomes increasingly difficult when the thickness of the overburden at the potential site exceeds 50 m. Uncertainty exists in all foundation conditions, and therefore designing and constructing an underground impervious system within a thick overburden is a very challenging task. Adequate geological and geotechnical investigations are undoubtedly the only way to improve design confidence in these systems. It is also important for designing engineers to fully assess the reliability of investigation results, including factors such as the size effect in plate load tests, the field draining condition in pressure meter tests, the influence of underground water on compressive wave velocities, the influence of drilling fluid layer adhering to the borehole wall on measured permeability coefficients, and the possible anisotropy of engineering properties.

3. Seepage control techniques

The two main goals in designing seepage control facilities include controlling the hydraulic gradient within overburden layers to ensure the seepage stability of foundation materials and reducing the seepage loss of reservoir water. For overburden layers where excavation and removal are feasible, deposits right beneath the impervious system of the dam body (e.g., clay or asphalt core and toe plinth) can be removed so that the seepage barrier can sit on a firm rock foundation. In cases where deep excavation is impossible, a horizontal, vertical or combined seepage control measure must be employed to meet the above goals. In the design specification for rolled earth and rockfill dams, a vertical seepage barrier that cuts through the overburden layers is recommended over an upstream horizontal measure. This can be evidenced from the cases listed in **Table 1**, in which all dams use at least one cutoff wall. In this section, seepage control techniques used in some of these example dams are reviewed.

3.1. Earth core rockfill dams

3.1.1. The Xiaolangdi ECRD

The Xiaolangdi ECRD was constructed on the well-known, sediment-laden Yellow River. The thickness of the underlying overburden is approximately 80 m, and it is composed of intricate sand and gravel layers. The dam uses an inclined core wall (low-plasticity loam) as the main anti-seepage barrier, as shown in **Figure 1**. A vertical concrete cutoff wall (1.2 m) was built within the overburden to control the underground seepage. The top of the cutoff wall was embedded into the core wall for 12 m, while its bottom end penetrates the rock surface for at least 1 m. The inclined core was extended using low permeable clayey soils along the top surface of the cofferdam on the upstream side, forming a horizontal blanket that is useful in lengthen the seepage path. The cutoff wall under the cofferdam was elongated into this

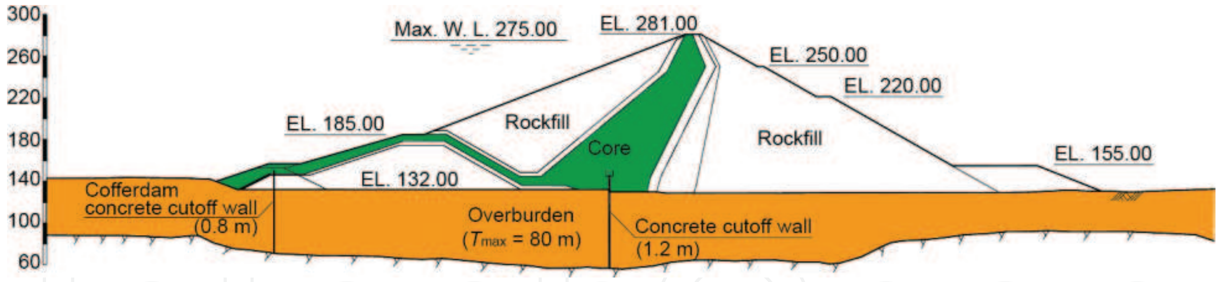

Figure 1. The maximum cross section of the Xiaolangdi ECRD.

blanket. It was assumed that the upstream blanket would connect naturally with sand sediments during long-term operation once the reservoir was impounded.

During the design-phase for the Xiaolangdi ECRD, China had no experience in building such high rockfill dams on 80-m overburden layers, making this project a particularly difficult challenge. A number of alternative design proposals were also considered, including the complete removal of the overburden under the core wall and the use a horizontal impervious blanket without the permanent cutoff wall. Lessons learned from previous cases and, more importantly, technological advances in cutoff wall construction resulted in the final chosen design. The thickness of the cutoff wall was determined based on the allowable hydraulic gradient of concrete materials, the available equipment and stress–strain and seepage analyses results. Conventional concrete with a 28-d strength of 35 MPa was used for the main cutoff wall, while plastic concrete and high-pressure rotary jet grouting were used to construct the temporary cofferdam cutoff wall.

3.1.2. The Changheba ECRD

The Changheba ECRD is currently one of the highest rockfill dams under construction in China (**Table 1**). It sits on a thick, three-layer overburden. All three layers, $fglQ_3$, alQ_4^1 and alQ_4^2 (shown in **Figure 2**) consist mainly of coarse gravel materials and therefore have relatively high deformation moduli and bearing capacity, but also exhibit high permeability. Local liquefiable sand layers are, however, also distributed widely within the alQ_4^1 layer. The dam is located in a high earthquake intensity region, where the peak acceleration for an exceedance probability of 0.02 in 100 years is 3.59 m/s². Sand liquefaction under earthquake condition is therefore a potential problem for this dam. The existence of these sand layers may also cause uneven deformation of the dam. To avoid these adverse risks, sand layers beneath

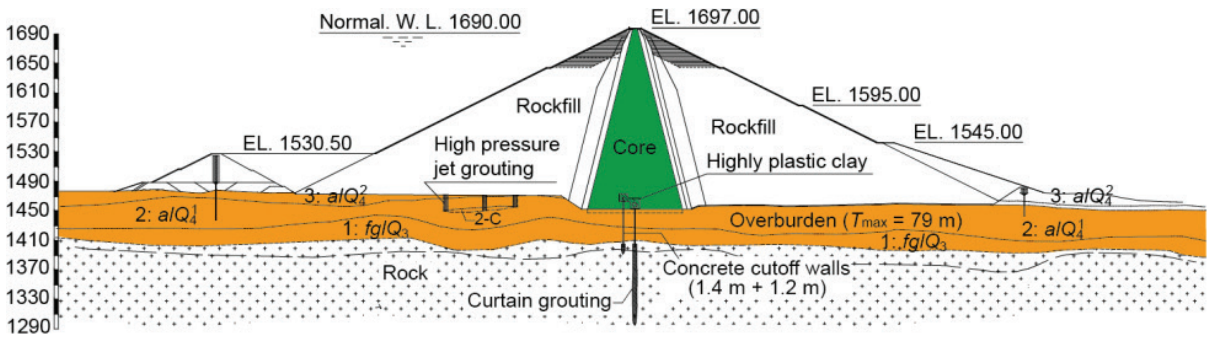

Figure 2. The maximum cross section of the Changheba ECRD.

both the core wall and the filter layer were removed completely. The maximum thickness of the retained overburden under the core wall is about 53 m.

Two concrete cutoff walls were poured with a net distance of 14 m. Both walls penetrate into the bedrock for at least 1.5 m. The main cutoff wall (1.4 m) is located within the dam axis plane and is connected to the core wall by a grouting gallery. The auxiliary cutoff wall (1.2 m) is located upstream of the main wall and embeds into the core wall for 9 m. The core wall is constructed with gravelly soils, where the maximum core material diameter allowed is 150 mm. The percentage of particles finer than 5 mm (P_5) ranges from 30–50%. Another two strict requirements for the core materials are $P_{0.075} \geq 15\%$, and $P_{0.005} \geq 8\%$. Curtain grouting was conducted through the preset pipes within the cutoff walls. In particular, curtain grouting under the main cutoff wall was extended to the level 5 m below the relatively impermeable layer ($q < 3$ Lu.)

3.1.3. The Luding ECRD

The 84-m Luding ECRD was built on an overburden with a maximum thickness of 148 m. It is among the deepest overburden layers used as foundations of a rockfill dam in China. The complex soil and rock strata is shown in **Figure 3**. Four main layers can be observed: the $fglQ_3$ layer, the $prgl + alQ_3$ layer, the $al + plQ_4$ layer, and the alQ_4^2 layer. Basic properties of these layers are listed in **Table 2**. The third sub-layer of the $prgl + alQ_3$ layer consists mainly of fine and silty sands, and therefore has a relatively low deformation modulus and a low bearing capacity. Sand lenses also exist in the second sub-layer of the $prgl + alQ_3$ layer and the first sub-layer of the $al + plQ_4$ layer.

The dam uses a clay core as the anti-seepage barrier stabilized by the rockfill shoulders. The maximum diameter allowed for the core materials is 100 mm. Other restrictions imposed on the core materials are $P_5 \geq 90\%$, $P_{0.075} \geq 60\%$, and $P_{0.005} \geq 15\%$. Repeated compaction near the optimum water content (±2%) produces a barrier with a coefficient of permeability less than 5×10^{-7} cm/s. A vertical concrete wall (1.0 m) was designed to cut off the foundation seepage water. The cutoff wall penetrates into the bedrock at both the left and right abutments of the dam. However, the overburden near the center of the canyon is so thick (148 m) that the current technology limits the capacity for constructing such a high underground wall. Therefore,

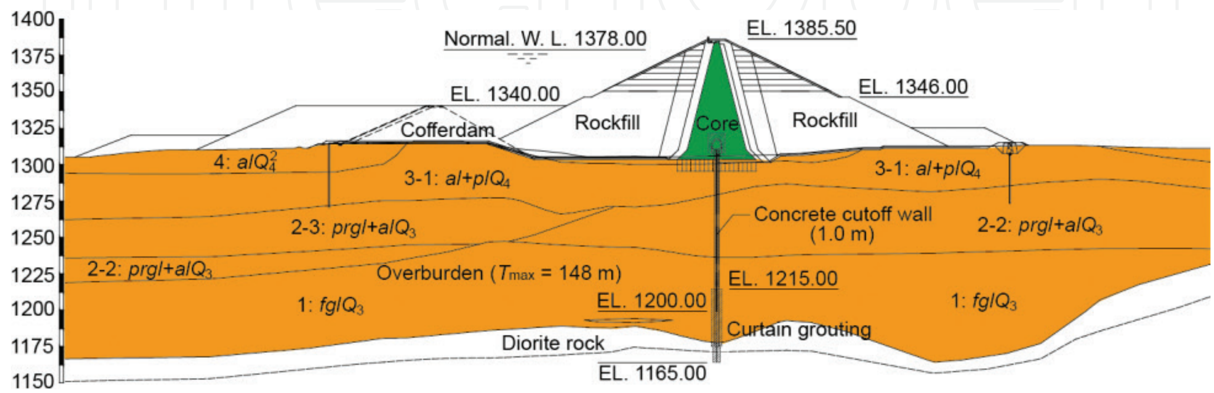

Figure 3. The maximum cross section of the Luding ECRD.

Layer	Density		Modulus and bearing capacity		Shear strength		Permeability	
	ρ (g/cm³)	ρ_d (g/cm³)	E_0 (MPa)	R (MPa)	φ (°)	c (MPa)	k (cm/s)	J_c
1: $fglQ_3$	2.20–2.30	2.05–2.15	55–65	0.55–0.65	30–32	0	2–4 × 10^{-2}	0.12–0.15
2-2: $prgl + alQ_3$	2.05–2.15	2.00–2.05	40–50	0.35–0.45	26–28	0	1–5 × 10^{-3}	0.20–0.25
2-3: $prgl + alQ_3$	1.60–1.70	1.40–1.60	18–22	0.12–0.16	15–18	0	1–10 × 10^{-3}	0.25–0.36
3-1: $al + plQ_4$	2.10–2.20	2.05–2.10	45–55	0.40–0.50	29–31	0	5–10 × 10^{-3}	0.15–0.18
4: alQ_4^2	2.15–2.25	2.00–2.10	50–60	0.50–0.55	28–30	0	1–10 × 10^{-2}	0.10–0.12

Note: ρ = natural density; ρ_d = dry density; E_0 = deformation modulus; R = allowable bearing capacity; φ = friction angle; c = cohesion; k = coefficient of permeability; J_c = allowable hydraulic gradient.

Table 2. Basic properties of the overburden layers in Luding ECRD.

a suspended cutoff wall was designed in the river center with the bottom end located at an elevation of 1200 m within the $fglQ_3$ layer. The cutoff wall was connected to the clay core by a grouting gallery. The maximum height of the wall is 110 m, and the underlying unsealed overburden has a thickness of 40–50 m. Two rows of grouting pipes (φ 114 mm) were preset in the cutoff wall for grouting the bedrock, and two additional rows outside the wall for grouting the unsealed overburden. Both curtains extend into the bedrock, that is, the rock curtain reaches the level where $q < 5$ Lu., and the overburden curtains penetrate the rock for at least 2 m.

3.2. Asphalt core rockfill dams

3.2.1. The Yele ACRD

When high-quality clayey soils are difficult to obtain to construct an ECRD, an ACRD is an appropriate alternative. Asphalt is a highly plastic and impermeable material and has a good

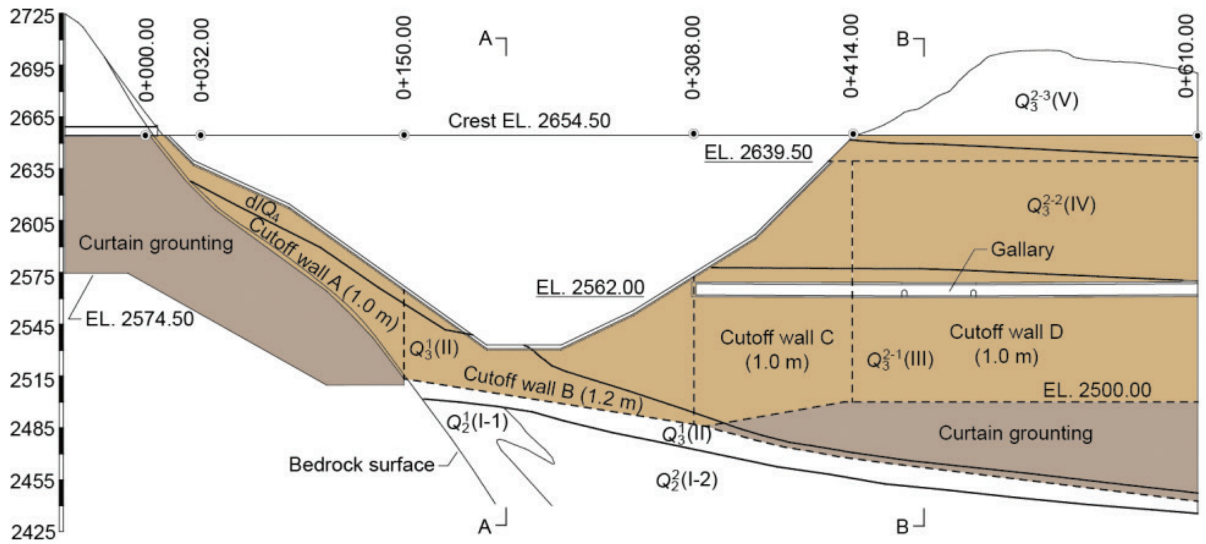

Figure 4. The segments of seepage control barriers in the Yele ACRD.

adaptability to uneven deformation. The Yele ACRD sits on a thick overburden as shown in **Figure 4**, with an extremely thick overburden at the right abutment. There are five main layers under the dam, as divided by the solid curves in **Figure 4**. The first (Q_2^1 & Q_2^2), third (Q_3^{2-1}), fourth layers (Q_3^{2-2}) are mainly composed of weakly cemented gravel materials, while the second layer (Q_3^1) is composed of a mixture of gravel and hard clay. The relatively high fifth layer (Q_3^{2-3}) is mainly composed of silty loam. The second layer (Q_3^1) forms a relatively impermeable barrier in the foundation, the permeability coefficient of which is less than 2.2×10^{-5} cm/s and the allowable hydraulic gradient reaches 10.4. These features were fully used in designing the foundation impervious facility.

The seepage control measures for this dam are divided, from the left bank to the right, into a number of different segments as described below. Curtain grouting was conducted within the gallery in the left river bank (0−150.00–0 + 007.275) to an elevation of 2574.5 m, with a maximum depth of 80 m. From 0 + 007.275 to 0 + 150.00, a concrete cutoff wall (1.0 m) was built into the bedrock for 1.0–2.0 m and curtain grouting was conducted into the weakly weathered rock. The maximum height of the cutoff wall in this segment is 53 m. The third segment starts from 0 + 150.00 until 0 + 308.00, and has a suspended cutoff wall (1.2 m), with its bottom end penetrating the second layer (Q_3^1) for at least 5 m. The height of the cutoff wall in this section ranges from 25 m to 74 m, and curtain grouting was not conducted. From 0 + 308.00 to 0 + 414.00, two layers of concrete cutoff wall were constructed separately. The lower cutoff wall (1.0 m) was cast within the gallery, while the upper wall was constructed from the slope surface. Curtain grouting was conducted into the second layer (Q_3^1) for at least 5 m. The fourth segment (0 + 414.00–0 + 610.00) uses a similar combination of two layers of cutoff wall (1.0 m) and a curtain grouting. The lower cutoff wall was cast to an elevation of 2500 m in the gallery, beneath which a curtain grouting embedding the second layer for at least 5 m was used to cut off the seepage water. The maximum depth of the curtain grouting in this fifth segment is about 120 m. Reinforced concrete was used for the top of the cutoff wall at an elevation of 2639.50–2654.50.0 m.

3.2.2. The Xiabandi ACRD

The Xiabandi ACRD was constructed mainly with gravel materials collected from the riverbed. The thickness of the foundation overburden reaches 148 m, which is almost twice the dam height (78 m). The distribution of the deposited layers is shown in **Figure 5**, where three main influential layers can be seen. The lowest layer ($fglQ_3^1$) mainly contains glacial gravel particles 2–8 cm in diameter. The thickest layer (glQ_3) mainly consists of coarser grains such as boulders and rubble, and it has local bridged structures distributed widely throughout and has a very complicated lithology. Enclosed within the glQ_3 layer is an almond thick sand lens ($fglQ_3^2$) which mainly consists of medium and fine sand, silty loam and silty sand. No high-quality clayey soils are found within 60 km of the dam site, and cement, steel and other necessary construction materials would also have to be imported from places even far away (320 km). The transportation condition to the dam site is rather severe at the time of designing. Traffic interruption is often caused by heavy snows in winter while in summer the flood originated from melting ice and snow often results in debris flow accidents. Because of these natural conditions, using too much steel and cement should be avoided. The dam site, on the other hand, is rich in good aggregate for asphalt concrete. Therefore, an asphalt core is used as the impervious system of the dam.

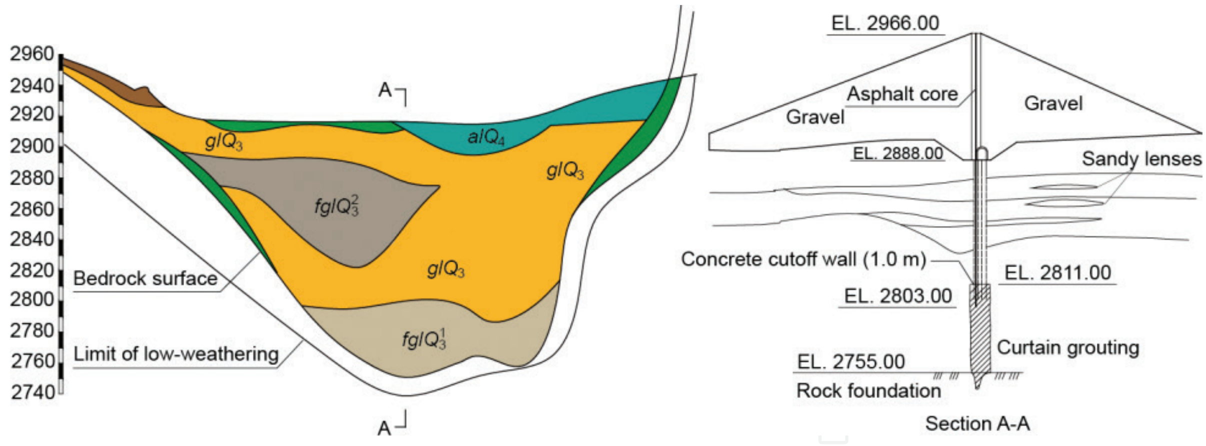

Figure 5. The overburden and seepage control barrier in the Xiabandi ACRD.

A concrete cutoff wall was constructed, with the bottom inserted into the bedrock within shallow bank slopes. At the deepest locations in the center of canyon, concrete was poured from an elevation of 2803 m to an elevation of 2888 m at an ascending speed of 2.0–7.5 m/h, forming an 85-m high-suspended concrete cutoff wall (1.0 m). Four rows of curtain grouting were constructed to extend the impermeable system into the bedrock, including a row of curtain grouting upstream of the cutoff wall and two rows downstream. The middle curtain grouting was performed through the pipes preset in the cutoff wall. The main (inner) curtain grouting penetrates the bedrock for 10 m, and the outer three rows for at least 5 m. The permeability restriction on the curtain grouting is $q < 5$ Lu or $k < 10^{-4}$ cm/s.

3.3. Concrete faced rockfill dams

3.3.1. The Aertash CFRD

The Aertash CFRD, currently under construction, is the highest dam of its type filled upon thick overburden layers. The alluvial foundation materials can be broadly divided into two layers. The upper layer (alQ_4) mainly consists of gravel materials inlayed by boulders, the thickness of which ranges from 4.7 to 17.0 m. The lower layer (alQ_2) is constituted mainly by weakly cemented gravel materials. The total thickness of the overburden layers reaches 94 m, as shown in **Figure 6**. The basic properties of both layers are given in **Table 3**. In general, both gravel layers are in medium dense states and have relatively high strength and deformation moduli. The permeability, however, is also very high and the discontinuous grading makes them vulnerable to seepage failure.

Reinforced concrete face slabs are used to retain the reservoir water and a deep concrete wall (1.2 m) penetrating the rock foundation is designed to cut off the underground seepage. The thickness (t) of the concrete face is $t = 0.4 + 0.0035H$, where H is the depth measured from the top of the face slabs. The concrete face slabs are connected to the concrete cutoff wall by a toe plinth and two horizontal linking slabs. The maximum height of the cutoff wall is 90 m, with the top 10 m reinforced by steel rebar. Curtain grouting is conducted under the cutoff wall into the bedrock to a level where $q < 5$ Lu. The depth of curtain grouting ranges from 17 to 69 m.

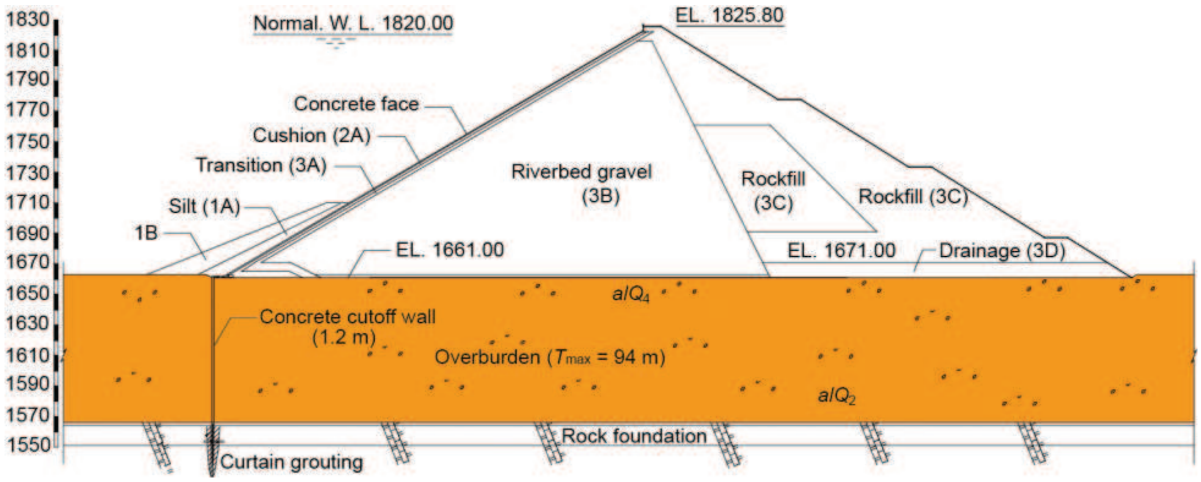

Figure 6. The maximum cross section of the Aertash CFRD.

Layer	Density		Modulus and bearing capacity		Shear strength		Permeability	
	D_r	ρ_d (g/cm³)	E_0 (MPa)	R (MPa)	φ (°)	c (MPa)	k (cm/s)	J_c
alQ_4	0.80–0.85	2.23–2.23	40–50	0.60–0.70	37.0–38.0	0	0.29	0.10–0.15
alQ_2	0.83–0.85	2.18–2.20	45–55	0.65–0.80	37.5–38.5	0	5.00	0.12–0.15

Note: D_r = relative density.

Table 3. Basic properties of the overburden layers in Aertash CFRD.

For concrete faced rockfill dams, it is possible to construct the dam first and then continue with the construction of the cutoff wall, or vice versa. Finite element analyses can be used to optimize the construction sequences. In the current case, the concrete cutoff wall is planned to be built after the dam is filled to a certain elevation. The linking slabs will be cast before reservoir impounding. Connecting the top of the concrete cutoff wall to the linking slabs will also be finalized before impounding.

3.3.2. The Chahanwusu CFRD

The Chahanwusu CFRD is another high dam (110 m) built mainly with gravel materials, as shown in **Figure 7**. The dam sits on sand and gravel overburden layers with a maximum thickness of about 47 m. Three layers can be observed in **Figure 7**: the upper sand and gravel layer with an average thickness of 19.2 m; the medium-coarse sand layer with an average thickness of 5.9 m; and the lower sand and gravel layer with an average thickness of 11.2 m. Both of the sand and gravel layers have similar engineering properties. The average relative density is 0.85 and the average coefficient of permeability is 6.68×10^{-2} cm/s. The middle sand layer has an average relative density of 0.92 and a permeability coefficient of 4.27×10^{-2} cm/s. Therefore, all foundation layers are in relatively dense states. The dam is located within a region of high earthquake intensity, with a design horizontal acceleration of 2.31 m/s². However, liquefaction within the medium-coarse sand layer is considered impossible.

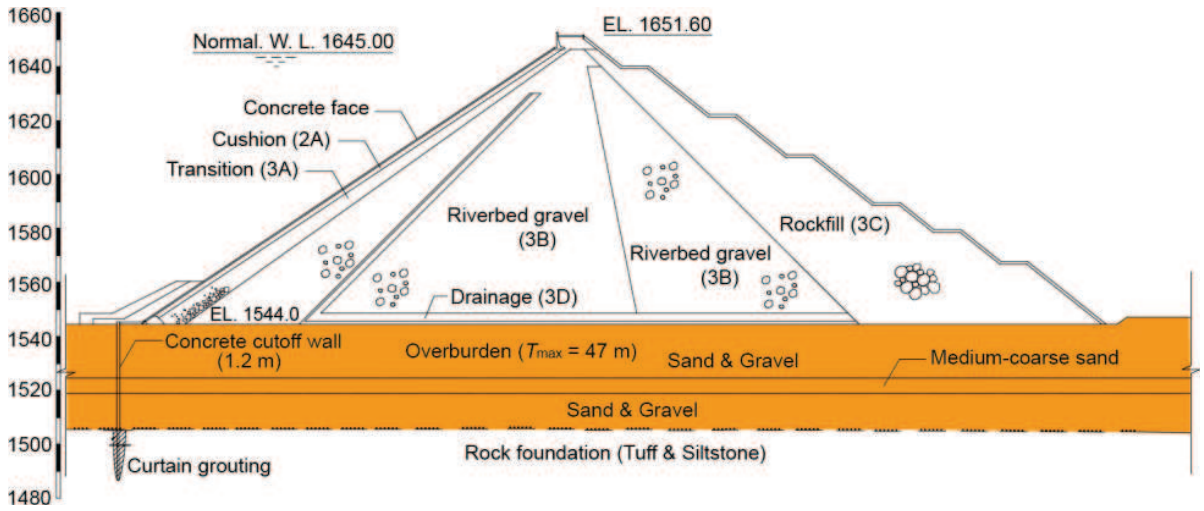

Figure 7. The maximum cross section of the Chahanwusu CFRD.

The dam uses upstream concrete face slabs as the seepage barrier, the thickness of which is determined by $t = 0.3 + 0.003H$. The toe plinths on the left and right bank slopes sit on bedrock, with both consolidation and curtain grouting performed underneath. The toe plinth built on the riverbed is located directly on the gravel layer, removing only the surficial loose deposits (1–2 m). Dynamic compaction was, however, performed to enhance the relative density and modulus of the materials beneath the toe plinth. A concrete wall (1.2 m) inserting the bedrock was constructed to cut off the foundation seepage. The cutoff wall was also connected by two horizontal linking slabs and the toe plinth to the upstream concrete face slabs, forming a closed impervious system. Curtain grouting was also performed under the cutoff wall into the bedrock until the designed level was achieved.

3.4. General remarks

There are other types of rockfill dams and sluices built on overburden layers. Reviewed above are three main kinds of rockfill dams used in water conservancy and hydropower engineering. All the dams in operation reviewed above function well without abnormal performance and major accidents. It could be remarked, in a general sense, that constructing high rockfill dams upon thick overburden layers is technically feasible. Using one or two vertical cutoff wall(s) embedding into the bedrock layer is an effective measure to control the underground seepage. In the case that the underlying overburden layers are extremely thick, a suspended cutoff wall extended by several rows of curtain grouting seems to be a feasible and effective choice.

4. Connection techniques

A reliable connection between the seepage control components within a rockfill dam and its overburden foundation is a prerequisite for a successful impervious system. Connection zones are weak places that require special design considerations. In this section, connection techniques used in different types of rockfill dams are briefly introduced.

4.1. Connection to clay core

For earth core rockfill dams, two basic design schemes can be used to connect the cutoff wall with the clay core. The simplest one is to insert the top of the cutoff wall directly into the clay core for a specified depth. The depth can be determined by the allowable hydraulic gradient along the interface between the cutoff wall and the surrounding soil. Inadequate inserting depth may result in seepage erosion along the contacting path. To avoid shear failure and cracks in the clay core adjacent to the inserting points, a zone of highly plastic clay is used to encapsulate the top of the cutoff wall, as shown in **Figure 8(a)**. Highly plastic clay is more deformable than the clay core, and it can absorb incompatible deformation between the cutoff wall and the core wall without sacrificing its impermeable performance, even under large shear strains.

The second connection method is to use a concrete gallery on the top of the cutoff wall, as shown in **Figure 8(b)** and **(c)**. Using a gallery near the base of the dam has several advantages. Curtain grouting can be performed within the gallery at the same time of dam filling, which may considerably shorten the construction time. Second, the gallery provides a possibility to enforce the foundation impervious system in the case that it does not function well. Without a gallery, repairing the underground seepage control component will be extremely difficult, if not impossible. The gallery can also be used to monitor the performance of the dam and it also provides a path to connect the left and right bank slopes. In the Changheba and Pubugou ECRDs, two concrete cutoff walls are used, one inserting into the clay core and the other connected with a gallery enclosed by highly plastic clay zones.

Careful designing, however, should be exercised when using a gallery. On the one hand, no structural joints are used in general for the riverbed monolith, and the gallery is vulnerable to cracks because of uneven settlement of the overburden layers. The gallery is usually extended into the rock banks, and the connection places of riverbed and rock segments often suffer large shear deformation and damage of water stops. On the other hand, connection of the concrete cutoff wall and the floor of the gallery also require special design considerations. In current practice in China, a rigid connection is mostly wide used, where the top of the concrete wall is reinforced and cast together with the floor of the gallery using an inverted trapezoidal transition cap, as shown in **Figure 8(b)** and **(c)**. A rigid connection may result in

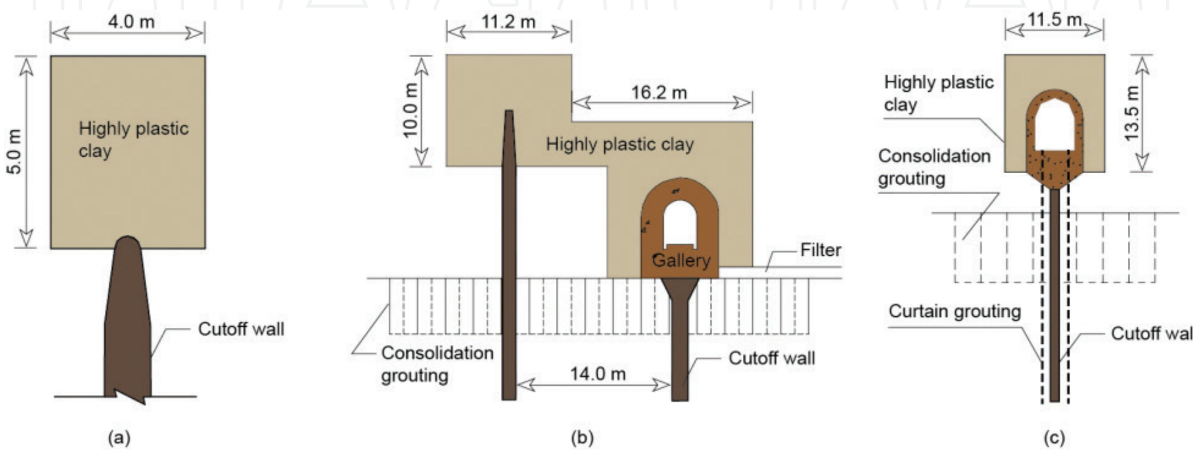

Figure 8. Connection techniques for ECRD. (a) Xiaolangdi ECRD. (b) Changheba ECRD. (c) Luding ECRD.

high compressive stresses within the cutoff wall, but it removes the need for a complicated water stop structure between the cutoff wall and the gallery floor.

4.2. Connection to asphalt core

An asphalt core is a thin plate structure similar to a concrete cutoff wall. A connection between the two structures is often accomplished using a concrete base built on the top of the cutoff wall, as shown in **Figure 9(a)**. The location of the concerned section can be seen in **Figure 4**. An inverted trapezoidal cap is used to accommodate the enlarged foot of the asphalt core wall, with mastic asphalt used to ensure the cementation between the cap and the core wall. The top surface of the base is usually curved slightly downwards to ensure that the asphalt core does not spread. A gallery can also be incorporated into the concrete base for inspection, grouting and communication. Asphalt core has currently not been used in rockfill dams higher than 150 m in China, and the connection with concrete cutoff wall is usually simpler than that in ECRDs as described above.

Another distinct feature of the reviewed Yele ACRD is the use of two layers of concrete cutoff walls that are connected by a construction gallery in the right bank, as shown in **Figure 9(b)**. The upper cutoff wall was constructed on the ground, while the lower one in the gallery (6.0 × 6.5 m). Curtain grouting was also finished in this gallery. To connect the upper cutoff wall with the top of the gallery, joint curtain was constructed using a non-circulation descending stage grouting. Both cement and chemical slurries were pumped into the jointing soils under a maximum pressure of 4.5 MPa. Grouting operation did not cease until the permeability of the curtain reached $q < 5$ Lu.

4.3. Connection to concrete face slabs

If the toe plinth of a concrete face rockfill dam is built on overburden layers and a concrete wall is used to cut off the underground seepage, then a reliable connection between the face slabs

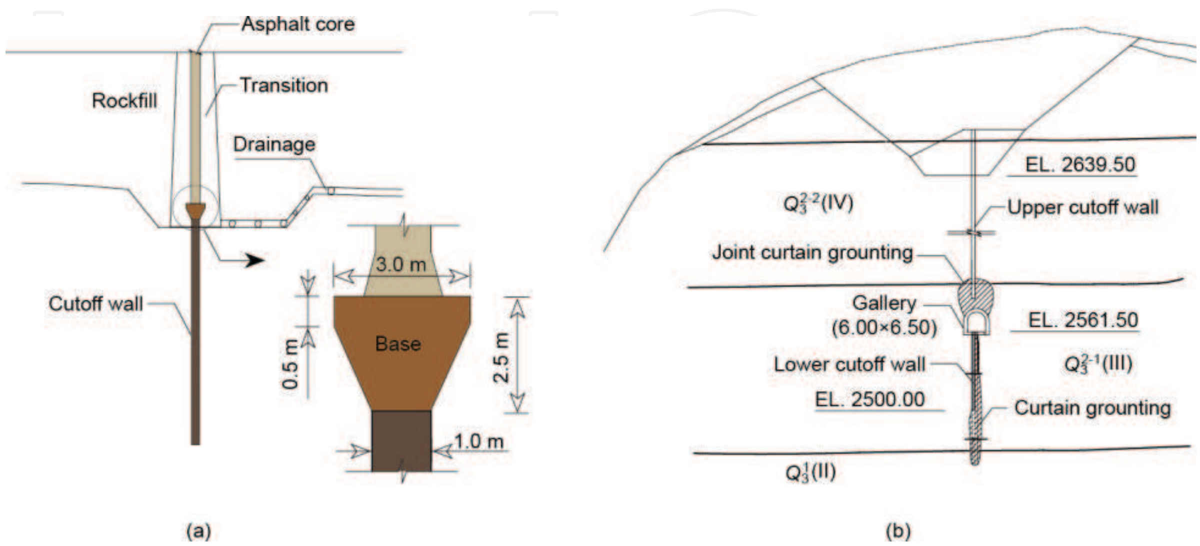

Figure 9. Connection techniques in the Yele ACRD. (a) Section A-A. (b) Section B-B.

and the cutoff wall should be guaranteed. Now, it is a standard way to use one or two linking slabs to connect the cutoff wall with the toe plinth, as exemplified in **Figure 10**. Water stops are installed at the connection points of different components. The width of the toe plinth and the linking slab(s) should be determined based on the allowable hydraulic gradient of the underlying overburden, and on the permitted three-dimensional displacements that are sustainable for the water stop structures. The designing features of connection systems in typical CFRDs are shown in **Table 4**. In these CFRDs, the width of the linking slab(s) usually ranges from 2 to 4 m.

The watertight structure for the perimetric joints used in most CFRDs (e.g., Aertash) consists of three layers. A "W"-shaped copper water stop is used at the bottom, and a wavy watertight stripe is used as the middle sealer. Plastic filling material is enclosed by a "Ω"-shaped rubber

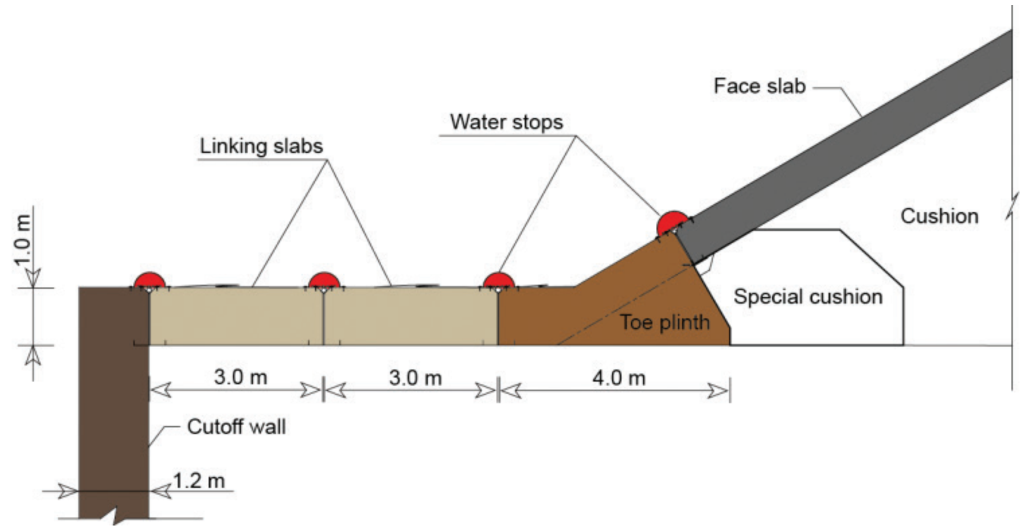

Figure 10. Connection techniques in the Aertash CFRD.

No.	Dam	Dam height (m)	Overburden thickness (m)	Thickness of cutoff wall (m)	Width of toe plinth (m)	Width of linking slab (m)
1	Xieka	108.2	100	1.2	4.0	4.0
2	Nalan	109	24	0.8	8.0	3.0
3	Miaojiaba	111	48	1.2	6.0	3.0
4	Jiudianxia	136.5	56	1.2	6.0	4.0
5	Aertash	164.8	94	1.2	4.0	3.0 + 3.0
6	Chahanwusu	110	47	1.2	4.0	3.0 + 3.0
7	Duonuo	112.5	40	0.8	5.0	3.0
8	Laodukou	96.8	30	0.8	7.0	3.0 + 3.0
9	Jinchuan	112.0	65	1.2	4.0	4.0 + 4.0
10	Gunhabuqile	160.0	50	1.2	4.0	2.0 + 4.0

Table 4. Designing features of the connection systems of typical CFRDs.

plate and fixed at the top of the joints. The gap between the different concrete components is 20 mm, and 12-mm wooden plates are placed in between these components.

5. Foundation improvement techniques

After removing the surficial loose layers, most overburden still requires some additional treatment before using as a dam foundation. Commonly used techniques include compaction, consolidation grouting, vibro-replacement stone columns, high-pressure jet grouting, and so on. These treatment techniques are briefly summarized below.

5.1. Compaction

To provide a firm foundation, vibrating rollers are always used to compact the overburden retained. Dynamic compaction is also commonly used to increase the stiffness and strength of the overburden layers. The authors recommend the Miaojiaba CFRD as an example [9] for dynamic compaction, which was performed before constructing the dam. The average settlement achieved by dynamic compaction was 26.4 cm, and the measured settlement of the overburden during dam operation is about 35–50 cm, indicating that the total settlement may be considerably larger if dynamic compaction has not been performed. Prior to dynamic compaction operations, it is, however, necessary to lower the underground water table.

5.2. Consolidation grouting

Consolidation grouting is always performed to provide a sound foundation for the seepage barrier of dams (e.g., clay core and toe plinth). The depth of grouting generally ranges from 5 to 10 m. The distances between grouting holes and rows range from 2 to 3 m. A concrete plate is usually cast before conducting the consolidation grouting works, serving as a working platform for grouting.

5.3. Vibro-replacement stone column

For weak layers such as sand lenses that are difficult to remove, vibro-replacement stone columns are usually used to densify the soils to form a composite foundation. The stone columns also serve as vertical drainage paths that are beneficial to dissipate the pore pressure within the surrounding soils. In the Huangjinping ACRD [14], vibro-replacement stone columns with a diameter of 1.0 m were set to improve the sand lenses buried more than 25 m below the ground surface.

5.4. High-pressure jet grouting

High-pressure jet grouting is a ground improvement and soil stabilization method, where a stabilizing fluid is injected at a high velocity into the treated soil under a high pressure. The grouted fluid hardens within the soil, forming well-cemented jet grouted columns. High-pressure jet grouting is a very versatile foundation improvement method and has been used not only in building temporary cutoff wall for cofferdams (e.g., **Figure 1**), but also in treating deeply buried sand lenses within the overburden layers (e.g., **Figure 2**).

6. Summary and conclusion

Great advancements in constructing high rockfill dams on thick overburden layers have been achieved in China over the past 20 years. Successful practice can be attributed to progresses in geological and geotechnical investigation techniques, proper designing and connection of the watertight systems, as well as the careful foundation improvement measures. It can be expected that even challenging geological conditions may be encountered in the future, which poses pressing needs in the following aspects:

a. *Reliable assessment of overburden layers and their engineering properties*. Combined use of traditional and newly invented geological and geotechnical investigation methods may considerably improve the reliability of the proposed results. Design engineers should fully understand the geotechnical parameters at hand and the possible limitations involved.

b. *Numerical simulation techniques*. Computational simulations (such as the finite element method) are playing an increasingly important role in designing. Embedding reasonable and simple constitutive models for dam materials and in-situ overburden soils into a fully coupled procedure can yield reliable predictions on the performance of both dams and their seepage barriers in an economical way. Such constitutive models, however, are still scarce.

c. *Effective emergency countermeasures*. It is very difficult to guarantee completely reliable construction quality for seepage control facilities as they are either enclosed inside the dam or buried deep under the dam. In the event that unexpected leakage does occur anywhere in the dam or foundation, effective countermeasures should be in place to eliminate threatens and to prevent amplification of leakage points.

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