# of pages

The idea of writing a bulletin on cutoffs for dams came from Prof. T.Strobl. in 1997, and was briefly discussed at the Florence Congress. At the Annual Meeting in Delhi at tentative table of contents was presented by Messrs. Strobl, Marulanda and Larocque and the Executive Meeting approved the bulletin by adding its scope to the terms of reference of the Committee. The new bulletin was actually conceived as a joint publication of the Committee on Dam Foundations and the Committee on Materials for Fill Dams since the latter committee had also in mind to produce a bulletin of a similar content. Authors of the various chapters were appointed and also a suggestion for the approximate number of pages for each chapter was given. The bulletin was then thought to be a rather thin publication with not more than about 70 pages (in one language) plus some case histories. Progress of the bulletin, however, was slow and at the Antalya meeting in 1999 only a draft on jet grouting had been received written by a group of UK engineers lead by L. Attewill.

When the ad hoc Committee on Dam Foundations was dissolved at the Beijing meeting in 2000, Peter Brenner felt obliged to bring the bulletin to the end since some authors had already put some substantial effort into it. It was fortunate that he could join the Committee on Materials for Fill Dams as a delegate from Switzerland since there was not yet any member from this country in this committee. The authors of the bulletin however were not from this committee and some of them were not even former members of the foundations committee. The eight chapters of the main text were ready in draft form at the Foz do Iguaçu meeting in 2002, but there were as yet no case histories. Practically no progress was made between 2002 and 2005. After that, Peter started writing case histories and edited the various chapters into the present form. Additions were made mainly to Chapters 3 and 4 which originally were delivered without any references in the text.
David Kleiner actively commented to the bulletin and to the case histories and also Dr. Wynfrith Riemer read carefully through the main chapters. Finally, the assistance of Ingetec in preparing the bulletin into a form that can be sent to the ICOLD secretariat in Paris is highly appreciated.
Main contributors of this final effort are indicated below:

Chapter	**Main Chapters**	**Authors/Contributors**
1	Introduction	R.P. Brenner (Switzerland)
2	Need for cutoffs	G. Larocque (Canada) first draft, completely reworked by R.P. Brenner
3	Diaphragm walls	F. Kleist & Th. Strobl, edited and extended by R.P. Brenner
4	Vib walls	F. Kleist & Th. Strobl (Germany)
5	Pile walls	R.A. Millet (USA)
6	Superposed concreted galleries	A. Marulanda (Colombia)
7	Jet grouting	L. Attewill, T. Bruggman, R. Essler & T. Miller (UK) first draft, final text by P. Croce (Italy)
8	Deep mixing	R.P.Brenner
Appendices	**Case histories**	**Authors**
A1-1	General	R.P. Brenner
A1-2	Dhauliganga	R.P. Brenner

A1-3	Arminou	R.P. Brenner
A1-4	Eastside	R.P. Brenner
A1-5	Xiaolangdi	R.P. Brenner
A1-6	Colbun	R.P. Brenner
A1-7	Convento Viejo	R.P. Brenner
A1-8	Cleveland	R.P. Brenner
A1-9	Twin Buttes	R.P. Brenner
A1-10	Brombach	R.P. Brenner
A1-11	Tadami	R.P. Brenner, with special information from H. Sugawara (JCOLD)
A1-12	Péribonka	Hydro Quebec & SNC Lavalin
A1-13	A.V. Watkins	B. Demars (USA)
A1-14	Wanapum	J.M. Dyoco (USA)
A1-15	Manasquan	R.P. Brenner
A2-1	Pielweichs weir	Th. Strobl & F. Kleist (Germany)
A3-1	General	R.P. Brenner, with special information from H. Sugawara (JCOLD)
A3-2	Zoccolo	R.P. Brenner
A3-3	Khao Laem	R.P. Brenner
A3-4	Walter F.George	R.P. Brenner
A3-5	Beaver Creek	R.P. Brenner
A3-6	Wolf Creek	D. Kleiner (USA)
A4	Superposed concreted galleries	A. Marulanda
A5-1	SM-3 Cofferdam	R.P. Brenner
A5-2	Ertan	R.P. Brenner
A5-3	Thika	R.P. Brenner
A5-4	El Tambor	A. Marulanda
A6	Deep mixing	R.P. Brenner

Undoubtedly this document will provide an important reference to the ever significant subject of foundation treatment.

Alberto Marulanda
Chairman
Committee on Materials for Fill Dams

1. INTRODUCTION

Dams are constructed to retain or store water. To minimize the flow of water through the dam/foundation system special impervious zones or elements must be designed and constructed. Dams constructed of concrete can be considered practically impervious, except for possible leaky joints. Embankment or fill dams require a zone of low permeability soil, asphalt, or concrete, which can be placed either in the interior of the dam or with the latter two materials also on the upstream face. Flow of water through the foundation below the dam is controlled by the prevailing geological conditions. Seepage through pervious strata (alluvial deposits, residual soils, etc) can be controlled by a barrier or cutoff consisting of a sequence of impervious elements (piles, panels) reaching down to a stratum of much lower permeability, usually rock. Seepage through rock is usually controlled by single or multiple row grout curtains. Rock, however, can also be highly pervious, for example in the case of karst, which may reach to great depth, or when rock is intensely broken or crushed in regions of high tectonic stresses. For such cases cutoffs may be more appropriate than grout curtains. Grout curtains have been constructed in overburden materials using the so-called sleeve pipe (or tube-à-manchette) method. The largest grout curtain of this kind was constructed for the foundation treatment of the High Aswan Dam in Egypt (Shalaby, 1991). The curtain is up to 193 m deep, 40 m wide at the top and 20 m at the base. It consists of 15 rows at the top which are telescopically reduced to 8 rows in the lower part. The curtain has performed satisfactorily and no repair work has been required as of 2009. Installing grout curtains in alluvial deposits, however, can require more time than the construction of a cutoff. Moreover, the residual risk of some imperfections is considerably higher than with a cutoff. Today, diaphragm wall cutoffs with depths of 50 to 60 m can be constructed at rates of up to 250 to 300 m^2 per day (e.g. Naga Hammadi, Egypt).

In certain situations a combination of a cutoff in the overburden and a grout curtain in rock has been selected, as, for example, when the top few meters of the rock are highly fractured. However, the two treatment methods must be connected or overlapped. The grout curtain may be installed prior to the construction of the cutoff or later by drilling the grout holes through or adjacent to the cutoff wall. Drilling of grout holes also enables a fairly precise determination of the bedrock surface.

This Bulletin is limited to foundation treatment methods using cutoff-type barriers. Due to recent experiences, high emphasis is given to alluvial deposits throughout this document; however, different materials, such as pervious residual soil, pervious laterites and saprolites, highly fractured and weathered rock, and karst may require cutoff. The construction of cutoffs has made significant advances during the last two decades, mainly through the development of more powerful machinery for drilling and excavation, but also through the introduction of new concepts and techniques, such as jet grouting and deep soil mixing. The concept of diaphragm wall techniques using a cutter to provide continuous excavation originated in Japan in 1980. Rapid developments in Europe followed and cutter wheels with rock-roller bits were introduced around 1990. Since then, cutoff depths exceeding 100 m with vertical deviation of less than one percent have been accomplished.

In addition, there has been development of the materials used for the sealing elements, such as plastic concrete. These less rigid materials provide better compatibility with the in-situ ground conditions surrounding the cutoff wall elements.

The following types of cutoffs are presented in this Bulletin:

- Diaphragm walls
- Vib walls
- Pile walls
- Superimposed concreted galleries
- Jet grouting
- Deep mixing

These methods are briefly described in Chapter 2 and again more explicitly in Chapters 3 to 8. In addition, the practical application of each method is illustrated by selected case histories. These case histories also demonstrate how certain difficulties specific to a particular dam site have been dealt with. The most widely used type of cutoff is the diaphragm wall and fairly well documented case histories have been published in the technical literature, especially in the Transactions of ICOLD Congresses, whereas case histories for jet grouting and deep soil mixing in dam foundations, are rare.

The factors affecting the selection of a cutoff are:

- Depth of the pervious strata to be treated
- Shape (morphology) of the valley
- Characteristics of the in-situ materials (alluvium, rock) to be treated, including the presence of boulders that may cause problems
- Hydraulic gradient at operating conditions
- Available equipment for constructing the cutoff
- Project history (new or existing dam)
- Personal preference may also influence the choice of the cutoff.

A cutoff cannot completely eliminate the flow of water through the foundation. The cutoff itself also has permeability, although of low magnitude, and flow can take place around the cutoff or through incompletely treated strata below the cutoff. The cutoff wall, if not carefully constructed, may also have defects, which can develop into preferred paths of seepage. Seepage can be defined as the slow, uniform flow of water through a porous medium, whereas leakage is the concentrated, uncontrolled flow of water through a crack or any other defect (Charles, 1997). Powell & Morgenstern (1985) analyzed case histories of various kinds of cutoffs and also tried to find a value for unacceptable seepage. Most of the case histories evaluated were associated with a hydro-electric scheme and for the survey the threshold was based mainly on the opinions of the various authors as reported in the literature. Seepage beneath a dam founded on alluvium was considered unacceptable when it exceeds 1.6×10^{-4}

m^3/s per linear meter of dam. However, the criteria concerning acceptable seepage are dependent on geologic conditions and costs to restrict seepage to small amounts.

The importance of foundation seepage control requires that the construction of the cutoff be entrusted to an experienced specialized sub-contractor. Strict quality control is essential to assure successful performance of the cutoff.

The performance of cutoffs should be monitored so that their efficiency in reducing flow and piezometric head can be evaluated. Piezometers installed in the foundation upstream and downstream of the cutoff are required to satisfy this objective.

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2. NEED OF CUTOFFS IN DAM FOUNDATIONS

2.1 FOUNDATION SEEPAGE CONTROL

Practically all storage dam projects require some form of seepage control. Such measures are particularly relevant with embankment dams on alluvial foundations because most alluvial deposits are stratified and heterogeneous with highly pervious strata. For such conditions the reasons for seepage control are mainly twofold:

- To ensure stability of the downstream portion of the dam by preventing excessive uplift pressures and possible piping phenomena which may lead to failure of the dam,
- To prevent excessive losses of water, which may jeopardize the economic viability of the dam project.

Seepage control is a lifelong task in an embankment dam project. It starts during the design phase and later becomes the most important component in dam surveillance for the purpose of dam safety. In many projects seepage through the foundation was found to be excessive after impounding and remedial measures were necessary, for example, several earth dam sections in the La Grande project in Canada (Paré, 1984).

When the impervious stratum can be encountered at shallow depth, i.e. less than about 5 to 8 m, a *core trench,* is usually excavated. The excavated volume of pervious material is then replaced by a low permeability fill, which is connected with the impervious element in the embankment to ensure continuity of the seepage barrier. The core trench should have relatively flat slopes to avoid stress concentrations and to maintain an acceptable hydraulic gradient. The core trench is the best treatment for an alluvial foundation if the geological conditions are favorable. It allows a visual inspection of the excavation and an assessment of the excavated material. If there is a need for compaction or treatment of the foundation prior to fill placement or the inclusion of filter zones, this can be easily accomplished because the ground conditions are visible.

Other methods must be used to control foundation seepage when thicker alluvium or other foundation materials are encountered that require treatment. These can be distinguished as:

(1) Methods to control the hydraulic gradient; these include a combination of one of the following features:

- upstream blanket
- partial cutoff
- downstream drainage trench
- downstream filter blanket
- downstream relief wells

- downstream filter

(2) Methods to control the quantity of seepage; these involve the construction of positive or semi-positive cutoffs, i.e.:

Positive cutoffs
- Diaphragm walls
- Slurry walls
- Sheet pile walls
- Pile walls
- Vib walls
- Superposed concreted galleries

Semi-positive cutoffs
- Walls produced by jet grouting
- Walls produced by deep soil mixing

In a positive cutoff, the wall consists of engineered material placed into a previously excavated trench or driven hole. In a semi-positive cutoff the barrier is constructed by mixing existing ground with a binder, usually cement.

The type of cutoff and its depth will depend on the geological characteristics of the foundation. A considerable effort in site investigation may be necessary to adequately describe the foundation characteristics. Cutoffs are constructed to reach an impervious stratum. This stratum may be the sound or slightly weathered bedrock or it can be a continuous layer of impervious material (clay) underlain by further pervious layers. In the second case the barrier is a *partial cutoff.*

Grout curtains, both in alluvial soils and in rock are not considered positive cutoffs. The barrier effect in grouting is created by permeating the voids of the foundation material with a solidifying mixture. This method of seepage control is not discussed in this Bulletin.

2.2 METHODS TO CONTROL THE HYDRAULIC GRADIENT

2.2.1 General principles

The fundamental guiding principle in the design and construction of a cutoff is to control the flow of water through the foundation, rather than to reduce or stop water flow. Hydraulic gradients that might cause the migration of fines within an alluvial foundation or the erosion of fines within a fractured and/or weathered rock foundation must be controlled. Design concepts that achieve this are:

(1) Lengthening the seepage path. This is equivalent to extending the impervious element along the dam/foundation interface, to reduce the hydraulic gradient on the downstream

part of the dam, particularly the exit gradient at the toe of the embankment. This method will also reduce seepage losses.

(2) Relieving high pore water pressures in the vicinity of the dam toe in order to reduce uplift pressures and counteract piping processes. Foundation pore pressures can exert significant uplift forces on a confining layer of soil downstream of the dam if this layer is underlain by a more pervious layer. If the pore water pressure on the bottom of the confining layer exceeds the overburden pressure created by the confining layer, piping can then occur in the lower layer. Means to relieve pore water pressures are relief wells, drainage/filter blankets and drainage trenches. These measures will, however, increase the quantity of seepage.

(3) Stabilizing the ground in the area of the dam toe by increasing the confining pressure. This is accomplished by constructing a weighting berm at the toe of the dam.

2.2.2 Upstream blanket

An upstream blanket consists of low permeability material forming an extension of the embankment water barrier. Natural occurrences of such material may already cover part of the area upstream of the dam and can then be incorporated into the overall blanket construction. Upstream blankets are often combined with downstream drainage (relief wells). The effectiveness of blankets may increase with time because of the accumulation of sediments on the reservoir bottom. A comprehensive treatise on the design and effectiveness of upstream blanket in combination with relief well systems was presented by Turnbull and Mansur (1960).

The length of the blanket measured from the upstream side of the impervious element in the dam body is about 10 to 12 times the hydraulic head. Its thickness is given by the allowable hydraulic gradient, which is defined as head difference between upstream and downstream water level divided by the base length of the dam including the upstream blanket. From the review of several case histories, Powell and Morgenstern (1985) found an average value of 0.06.

Although Cambefort (1967) considered impervious blanket solutions as elegant and more natural, they rarely provide the designer with the same confidence and safety as does a positive cutoff. Today, with modern numerical tools, such as groundwater computer models, seepage calculations can be carried out with many different assumptions, which may compensate partly for the uncertainties that exist in assigning values of hydraulic conductivity to the various geologic materials in the foundation.

The quantity of seepage may decrease with time as a result of siltation immediately upstream of the dam. In general, if this decrease does not take place within the first three years after impounding, it will not occur later.

2.2.3 Partial cutoff

The upstream blanket solution may be less attractive when there is a continuous impervious stratum occurring in the lower part of the alluvial foundation profile. The designer may also eliminate the blanket solution if there is a rather thick pervious stratum at the surface of the foundation. Then a decision has to be taken whether seepage control can be accomplished using a complete cutoff or whether a partial cutoff would be appropriate.

Unless the partial cutoff connects to an impervious stratum embedded in the pervious alluvium, the amount of seepage flow will still be quite high as compared to a complete cutoff and downstream drainage facilities will be necessary. In homogeneous materials the efficiency remains low until about 90 to 95 % of the total cross section available for flow has been cut off (Cambefort, 1976). Therefore, in dam projects where water is precious, complete cutoffs are the most appropriate. Partial cutoffs are also combined with an upstream blanket solution, as, for example, with Doroodzan dam in Iran. Seepage control is achieved by an approximately 27 m deep partial cutoff to an impervious stratum, a 400 m long, 3 m thick impervious blanket and a row of relief wells that mainly relieves the artesian pressure from the pervious alluvium below the impervious stratum (Brenner et al., 1994).

The decision to construct a partial cutoff depends on a number of factors:

- Economic comparison between the value of water loss and the additional cost for a complete cutoff,
- Susceptibility to backward erosion and piping of the materials located below the proposed tip of the partial cutoff,
- Ease to monitor leakage and detect early signs of uncontrolled seepage downstream of the dam during operation,
- Amount of fines (silt and clay-sized particles) available for sediment formation on the reservoir bottom, and
- Presence of an intermediate impervious stratum of sufficient extent where the tip of the partial cutoff could be embedded.

2.3 METHODS TO REDUCE THE QUANTITY OF SEEPAGE

This section describes methods used to reduce the quantity of seepage; however, they are also effective controlling hydraulic gradient. Methods used exclusively to control hydraulic gradient are addressed in the previous section

2.3.1 Design and construction considerations

The most important design consideration is the selection of the type of cutoff, its depth and extent. This is a function of the foundation geologic conditions, the sensitivity of the foundation to damage by flowing water, the need to restrict total seepage flow, and the hydraulic gradient imposed on the foundation. The cutoff must fit the overall foundation conditions consistent with the type of dam and its function.

The design of a cutoff wall is a soil-structure interaction problem and the most important issue is to select the proper modulus of deformation of the wall. The wall has to be constructed prior to embankment fill placement although cutoff walls have been installed also through the core of existing dams. When the embankment is constructed and the foundation loaded, the alluvial material will undergo deformations, both in vertical and horizontal directions. Upon filling of the reservoir, the wall has to carry an additional hydrostatic load. The wall should have a stiffness such that it can follow these deformations without cracking. If the wall is too rigid to follow the soil when it settles under the weight of the dam, load will be transferred to the wall. The additional stresses caused by this load transfer may lead to cracking. Cracks in the cutoff wall reduce its sealing efficiency.

If, on the other hand, the cutoff wall is too plastic, its resistance to shear forces and internal erosion by seepage flow is reduced. If the expected settlements and lateral deformations under the dam are large, some designers choose to install the cutoff near the upstream toe of the dam in order to decrease the load on top of the cutoff. However, horizontal deformations in the dam foundation are not eliminated and may be even greater at that location than below the central part of the dam. Finite element modeling is most suitable to treat such questions and to find an optimal location for the cutoff wall.

A delicate point is also the connection between the top of the cutoff and the impervious element of the dam. Precautionary measures must be taken to avoid damage if cracks develop in the top part of the cutoff or in the surrounding ground. The literature describes many examples of such measures, for example: Manicuagan 3 dam in Canada (Benoit et al., 1967; Dreville et al., 1970), Bighorn dam in the USA (Forbes et al, 1973), Zoccolo dam in Italy (Dolcetta & Chiari, 1967), Obra dam in India (Garg & Agarwal, 1967) and Brombach dam in Germany (Strobl, 1991).

2.3.2 Diaphragm walls

A distinction must be made between the structural diaphragm wall and the cutoff barrier wall. The structural diaphragm wall acts as a retaining wall or as a foundation barrier wall in urban excavation work. The cutoff barrier wall is used to provide:

- a hydraulic barrier to water seepage below and/or within dams and dikes, and beneath canals,
- a barrier for water inflow into construction excavations, and
- a barrier to contain hazardous waste and its leachate (Millet et al., 1992).

In this bulletin only the cutoff barrier wall for use in seepage control is treated.

Construction of a diaphragm wall is performed by excavation of a trench without the use of significant support other than that provided by a bentonite-water slurry. Excavation is done in segments, usually called “panels”, and after completion of a segment the stabilizing mud is displaced by concrete (tremie process) to create a load- bearing wall. The wall is therefore composed of segments or panels with joints in between. The joints must ensure continuity and watertightness. Various techniques were developed to form the joints (Dupeuble, 1985;

Magnet & Mussnig, 1970). Temporary vertical guide walls are constructed at ground level to control the alignment of the trench and support the upper portion of the excavation. The design of the concrete mix, usually composed of cement, bentonite, aggregate and water, is crucial and will determine the stiffness and permeability characteristics of the wall.

Millet et al. (1992) distinguished four basic types of walls constructed by the slurry wall technique, namely:

(1) Soil-bentonite walls: The bentonite slurry is displaced by a soil-bentonite mixture similar in its consistency to high slump concrete. This procedure results in a continuous highly plastic low permeability cutoff wall limited to depth of about 25 m. (See also Fig. 3.16)

(2) Cement-bentonite walls, also known as “grout walls”, e.g. Little (1975), : Cement is added to a fully hydrated bentonite-water slurry. The cement-bentonite-water slurry is then used to both stabilize the slurry trench during excavation and upon setting of the cement, for the permanent cutoff wall itself. (See also Fig. 3.1)

(3) Plastic concrete walls: Aggregates are added to a cement-bentonite-water mix. This “plastic” concrete backfill is then placed by tremie pipe displacing the bentonite slurry to form a strong but still plastic wall.

(4) Rigid concrete walls (usually reinforced): The trench is backfilled with structural concrete using a tremie process.

The first two types of wall are commonly known under the collective term of "slurry walls" while the third and fourth types are denoted as "concrete walls". But the term “slurry wall” is also widely used for the concrete walls, especially in North America (Xanthakos, 1979)

2.3.2.1 Slurry walls

Slurry walls (or slurry trench cutoff walls) were pioneered in the United States in the late 1940’s. Since then they have gained increasing acceptance as an economic means of controlling seepage, mainly underneath cofferdams but also under main embankments. Excavation of the trench is by backhoe or dragline through pervious deposits down to suitably impervious materials. The trench width is normally between about 1.5m and 3m. Caving of the trench walls is prevented by bentonite slurry retained in the trench above the groundwater table. The backfill is a well-graded soil, similar in gradation to a glacial till, which is blended with bentonite in the range of 2 % to 4% by weight. The fines content (<#200 sieve) should be on the order of 10 % to 20 %. The maximum grain size should not exceed about 50 mm. The larger grain sizes are used to reduce settlements of the trench. Permeabilities of such mixtures are on the order of 10^{-9} m/s. The density of the backfill should be at least 80 kg/m^3 greater than the density of the bentonite slurry (Millet et al., 1992).

Wilson & Marsal (1979) give examples of slurry trench cutoffs in dam projects, for example Khancoban dam in Australia (Kotowicz, 1967) and Tortolas dam in Mexico where a

23 m deep partially penetrating slurry wall was constructed through pervious sand and gravel deposits. Hydraulic gradient across slurry trench cutoffs vary between about 7 to 12 (Gamboa et al., 1970).

The cement-bentonite (self-hardening) slurry wall may be selected if suitable backfill material cannot be found in the vicinity of the dam site. In this case a cement-bentonite slurry will be used to excavate the cutoff trench (for support of the trench walls). This slurry is then left in place and will harden after a few hours. After 12 hours it has about the consistency of butter. The final set takes about 90 days. Typical permeabilities of cement-bentonite slurry cutoff walls are on the order of 10^{-8} m/s. The most important factor controlling the characteristics of the hardened cement-bentonite mixture is the water-cement ratio.

The cement-bentonite wall is likely to settle considerably and there is a danger of formation of a gap between the top of the cutoff and the base of the impervious element (core). To counteract this trend the top of the trench is often flared to provide a transition zone. It is also usual to have some time between the end of cutoff construction and the placement of fill above it.

2.3.2.2 Concrete diaphragm walls

Emphasis in this bulletin is on the plastic concrete wall which has been widely used in recent years as more powerful equipment for excavation has become available. Cutoff walls have been constructed to depth beyond 130 m. The use of a qualified contractor is essential for the success of a cutoff wall construction.

The critical design criteria for cutoff barrier walls are (Millet et al., 1992):

- permeability
- deformability
- permanence

Permeability is considered the most important characteristic of the cutoff wall since the wall is constructed to minimize the passage of fluid. *Deformability* addresses the capability of the wall to sustain foundation strains (which may amount to several percent), without cracking. However, in the case of Péribonka dams, cracking of the wall because of construction defects or deformation was accepted as a possibility, thus a series of laboratory erosion tests in pre-formed cracks were performed successfully to evaluate erosion resistance under large hydraulic gradient. *Deformability* can be adjusted by using a proper mixture for the backfill. Finally, *permanence* refers to the stability of the wall with respect to decomposition by leaching or aggressive environmental conditions. Specifications for cutoff walls should be based on these design criteria but keeping in mind the goals to achieve with this structure. More details are given in Chapter 3 of this Bulletin.

2.3.3 Sheet pile walls

The use of sheet piles as positive cutoffs to control seepage has all but disappeared in permanent works and recent references to this construction method do not seem to exist. Lane & Wohlt (1961) discussed results of observations made on three earth dams along the Missouri River, constructed on extensive deposits of alluvial sands and gravels. Sheet piles were driven to bedrock to form a cutoff. The head loss across these cutoffs varied from 10 to 20 percent of the total net head, but increased with time due to the migration of fines and corrosion in the interlocks (cf. Wilson & Marsal, 1979).

The weak points in a sheet pile cutoff wall are the interlocks (joints) which are not watertight and the susceptibility to corrosion. However, in recent lock designs considerable improvements have been made and satisfactory sealing can now be obtained with smaller hydraulic loads. Installation of sheet piles can be quick and easy in certain ground conditions (e.g. finer grained soils).

In the past, sheet pile walls were used to isolate specific portions of the foundation under a dam and to prevent particle migration. Many gravity dams of limited height were founded on pervious foundation materials and sited on such enclosed spaces (Evdokimov & Vedeneev, 1967). De Luccia (1958) presented a sheet pile cutoff scheme consisting of an H-pile section with interlocks at each corner formed by welding standard straight sheet pile sections onto each flange. After driving these H-pile sections they formed a wall with cells which after cleaning out the soil material were filled with concrete.

The depth of sheet pile walls is usually limited to about 15 to 20 m.

2.3.4 Pile walls

Pile walls for use in watertight cutoff construction consist of a series of overlapping bored piles and can be considered as a special case of diaphragm walls. The great advantage of this construction method is that the installation of the piles can be accomplished in almost any type of ground, especially in rock, such as karstic limestone. Thus it can be assured that the bottom of the wall is socketed into a firm base without leaving a gap between wall and the pervious ground. Depths of more than 120 m have been reached successfully with such pile cutoff walls.

In cutoff wall configurations the bored pile elements of the wall are usually overlapping or, more seldom, contiguous, i.e. piles are just in contact with each other. A disadvantage is the large number of joints, which have to be watertight. Verticality tolerances are critical since the thickness of the wall at the joints is relatively thin (depending on the amount of overlap) and various techniques and tools to form the joints have been developed (F.P.S., 1985; A.J.K.M., 1985; Anik, 1967; Croce and Dolcetta, 1970; Forbes, 1973; Watakeekul and Cole, 1985). A minimum diameter of 60 cm is normally chosen for the pile cutoff and the joints between the piles have to be constructed accordingly.

With the development of modern excavation tools for diaphragm walls, such as the trench cutter, which is also able to cut through relatively hard rock, the use of pile walls as cutoff in dam projects has diminished, mainly because of economic reasons.

2.3.5 Vib walls

Vib walls, also known as "vibrated membranes" are constructed by vibrating a steel beam into the foundation and backfilling the space left by the beam with grout or mortar while pulling the beam out of the slot it has generated. The ground to be treated must be free of boulders and cobbles. The thickness of the wall varies from a few centimeters to decimeters. Continuity must be guaranteed with such a thin wall and a two-row wall improves watertightness and should be considered.

Vib walls are used most frequently in temporary work where water loads are small, such as cofferdams or dikes. The maximum depth that can be reached with vib wall cutoffs is around 20 m to 25 m.

A special type of vib wall is a flat sheet pile barrier system consisting of high density polyethylene extruded sections. Installation is by means of a steel plate which pushes the sheet pile sections to the required depth using a vibrator. The elements are locked together ensuring watertight joints. This system is suitable only for small depths and requires a relatively soft soil.

2.3.6 Superposed concreted galleries

This type of cutoff consists of galleries progressively excavated in the plane of the watertight barrier and backfilled with concrete. The vertical connection between the concrete filled galleries is usually through wall elements or sections of a grout curtain. This method of cutoff construction is only used in very difficult situations, mainly in foundations that are severely karstified or fractured where a grout curtain alone cannot achieve watertightness. Well-known examples of dam sites where such concreted galleries were constructed are Keban dam in Turkey (Gilmore et al., 1991), Khao Laem dam in Thailand (Watakeekul & Cole, 1985), La Honda dam in Venezuela (Antonopoulos et al., 1991), and Castilletto dam in Switzerland (Zingg, 1964).

2.3.7 Walls produced by jet grouting

Jet grouting is a relatively new technique developed mainly for foundation strengthening, soil stabilization, and excavation support; however, limited applications exist also in groundwater control, i.e. cutoff construction. The method uses hydraulic energy to cut and mix in-situ soil materials or also weak rock. A fluid grout is injected at very high pressure (40 to 60 MPa) and high velocity to create a stabilized mixture of soil and grout.

Jet grouting started in the 1970s by first creating an oriented narrow slot in the ground by means of a high-pressure jet and immediately filling it with grout. The result was similar to what is achieved today with the vib wall technique. The method developed into a ground reinforcement tool creating cylindrical columns or planar panels, which can interlock to form a jet-grouted structure. Compared with other types of cutoffs, jet grouted walls can also be constructed at locations with difficult access because of its greater flexibility.

For use in cutoff construction, continuity of the structure is critical. There are a large number of joints and the uncertainty about the relative position of each jet-grouted element in space limits the depth to which such cutoffs can be constructed. Still Hammamji et al. (1999) report applications up to a depth of 65 m. Furthermore, at the Bujagali Hydropower Project (Uganda), which is currently under construction, a two row jet grouted cutoff is being installed through residual soil, saprolite (with boulder locally present), and weathered rock into slightly weathered rock. Borings with water pressure tests will be used to check the integrity of the completed cutoff, and conventional rock grouting will be performed below the jet-grouted cutoff to reach the required depth.

The material requirements for jet grouting are higher than those for conventional grouting, but the construction time is much less. Also jet grouting can take over construction tasks which have formerly been solved by conventional grouting using chemicals. It is expected that the use of jet grouting in dam construction will still increase as further developments will be made in the jet grouting equipment.

2.3.8 Walls produced by deep mixing

Deep mixing is an in-situ soil mixing technology that mixes existing soils with cementitious materials (cement, lime, etc.) using a special tool performing rotational and lifting movements. This mixing tool may or may not be identical to the drilling tool. The binder (usually cement, or a mixture of cement and bentonite, often with some filler) may be introduced on the way down or on the way up, but normally on the way up. The result is a cylindrical column of soil-cement, which is denser and more impervious than the surrounding soil. Multiple-auger mixing tools can create wall elements or panels consisting of continuous soil-cement columns. The method is known under different names and the exact installation and mixing procedure may vary with different equipment manufacturers.

For cutoff construction by the soil mixing technique, an overlap of the individual panels is needed to ensure continuity of the soil mix wall. The soils to be treated by deep mixing are generally coarse-grained with a high permeability or interbedded strata of fine-grained and coarse-grained materials. Typical maximum wall diameters are around 0.9 m. The depth of wall is limited to about 20 m.

Applications of the deep mixing technique to cutoffs for dams are quite limited so far. Usually, the method is used in rehabilitation work, e.g. for dam heightening, stabilization of foundations with loose soils, or provision of additional seepage control under existing hydraulic structures.

2.4 CUTOFF EFFICIENCY

The efficiency or effectiveness of a cutoff must essentially be evaluated in terms of the quantity of seepage through the foundation and abutments, the value of the water and the stability of the dam as affected by seepage, especially with respect to possible internal erosion of materials (piping) through which the water percolates (Marsal & Reséndiz, 1991). Terzaghi and Peck (1967) define efficiency of a cutoff as the ratio between the loss of head caused by the barrier (ΔH) and the total hydraulic head across the dam, as shown in Fig. 1.1.

Efficiency $$E = \frac{\Delta H}{H} \tag{1.1}$$

To evaluate the efficiency, piezometric measurements on both sides of the cutoff must be available. However, in some cases piezometric measurements were only available downstream, and the full reservoir head was assumed to be applied to the cutoff wall. For ΔH an average value should be taken since the head drop varies because the cutoff is usually not perfectly impervious or homogeneous.

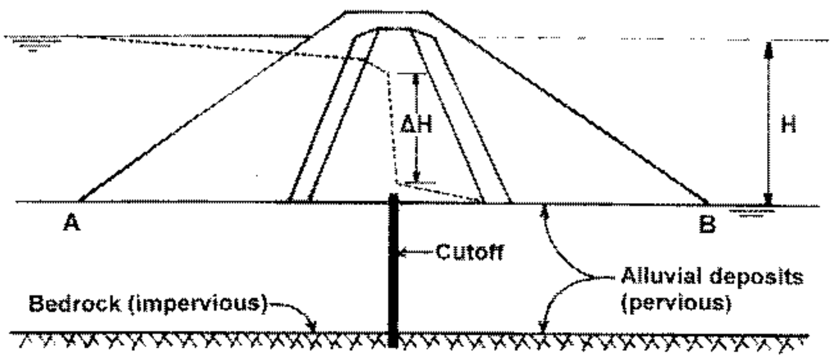

Fig. 1.1 Definition of efficiency

An alternative definition for the efficiency was given by Kratochvil & Hálek (1961) as follows:

Efficiency $$E_Q = 1 - \frac{Q}{Q_0} \tag{1.2}$$

Where Q is the seepage through the vertical section along the cutoff and Q_0 is the estimated flow assuming that the barrier does not exist and the reservoir is full. In general, it is easier to measure piezometric pressures than to gauge seepage flows.

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3. DIAPHRAGM WALLS

3.1 PURPOSE OF DIAPHRAGM WALLS

Diaphragm walls are a major construction element in foundation engineering. They usually have one of the two following functions: (1) sealing of geomaterials (earth or rock) against the flow of fluids and transport of solutes and solids and (2) support of structures as a structural wall element in a trench. The sealing function is not limited to hydraulic structures but also includes environmental applications such as the containment of waste disposal sites and sanitary landfills. The treatment of diaphragm walls in this Bulletin is restricted to their application and sealing function in dam engineering.

The primary use of diaphragm walls in dam engineering is to reduce the seepage through the dam foundation or through the embankment itself. The diaphragm wall is used to seal overburden material and weathered rock. Occasionally, it has been used in heavily karstified foundation rock. Since concrete dams are usually founded on good quality rock which is normally treated by grouting, diaphragm walls are therefore almost exclusively associated with embankment dams. There are very few masonry or concrete barrages which also rely on diaphragm walls. Diaphragm walls have also been used as interior sealing elements in dam bodies, often in connection with dam heightening projects. A special case is Mud Mountain Dam where a diaphragm wall was constructed through the core of an existing dam which had shown progressive increase in leakage (Graybill & Levallois, 1991).

3.2 PRINCIPLE OF DIAPHRAGM WALL CONSTRUCTION

The diaphragm wall is a positive cutoff. The principle steps are the excavation of a slot or trench in the medium to be sealed (foundation or dam body) and the subsequent filling of this trench by an impervious material. In order to prevent the collapse of the trench during or after excavation, it is filled by a supporting fluid. The hydrostatic pressure exerted by this fluid on the trench wall balances the earth pressure and stabilizes the wall against caving, typically in conjunction with a mud cake and the seepage gradient associated with it. Depending on whether the supporting fluid will become part of the wall or is removed, the following distinction is made:

- Single-phase method: The stabilizing fluid remains in the trench and will after completion of the excavation become part of the wall (slurry walls). The cement/bentonite cutoff wall uses the single-phase method.
- Twin-phase method: When the excavation has been completed the stabilizing fluid is displaced by the sealing material, often concrete which then sets in the trench and forms the wall (concrete walls). Other materials are also used to seal the cutoff, such as, soil/bentonite and soil/cement/bentonite mixtures.

The US Army Corps of Engineer's Manual EM 1110-2-1901, Seepage Analysis and Control for Dams, presents descriptions, case histories, and analyses of positive cutoffs using the single phase and twin phase methods. The document is available on the internet.

3.3 HISTORICAL DEVELOPMENT

3.3.1 Development of the cutoff construction method

According to Xanthakos (1979), the first slurry trench cutoff was probably built at Terminal Island, Long Beach, California, in 1948, documented by Sherard (1969). In Europe, the method of constructing diaphragm walls using stabilizing suspensions became popular around 1950 when Lorenz (1950) recognized the advantages offered by thixotropic liquids in foundation engineering. At nearly the same time, Veder (1953) patented a series of possible applications of thixotropic fluids in foundation engineering. Veder and the Italian ground engineering contractor ICOS (Impresa Construzioni Opere Specializzate) were granted a patent in 1957 for the construction of walls of overlapping cast-in-situ piles in the United States. This represented a decisive step towards modern day diaphragm wall technology. The further development of excavation technology, such as cable grabs has considerably increased the efficiency of the diaphragm wall method. In 1976, it became possible to construct the 131 m deep diaphragm wall under the Manicouagan-3 dam in Canada by means of the technique developed by Veder.

3.3.2 Excavation methods

Until about 1960, diaphragm walls were produced by wall elements (called herein as panels) cast either in succession or in alternate order. The latter method is much more common and is also known as the "pilgrim step" method, which even today is still applied frequently (Fig. 3.1). The procedure is as follows: After producing the first diaphragm wall panel, the next element is omitted initially and the next panel in sequence is excavated under the protection of a stabilizing fluid. The intermediate panels (secondary panels) are produced once the first (primary) panels have been concreted. In this way it is possible to achieve an enhanced stability of the panel excavation. Moreover, it is possible to produce a better controlled joint connection between the individual panels with a prescribed overlap. As part of a test on vibration damping, a continuous diaphragm wall stabilized with a fluid was produced for the first time in Berlin in 1960. It demonstrated that under particular conditions, a panel excavation is structurally stable even when produced continuously. Note that in this chapter the term "slot" will be used for the excavated panel filled with stabilizing slurry, while the term "panel" is used for the slot back-filled with concrete or a cement-bentonite mix.

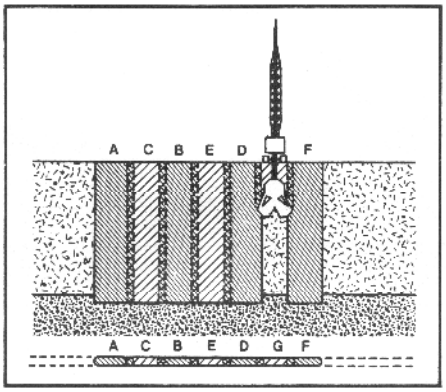

Fig. 3.1 Principle of the pilgrim step method (single-phase) (after ICOLD, 1985)

3.4 CONSTRUCTION METHODS AND EQUIPMENT

3.4.1 Guide walls

Diaphragm walls may be produced in a number of different ways, but normally all of them are excavated under the protection of guide walls. Guide walls are L-shaped reinforced concrete elements that reach to a depth of at least 1.5 m and stabilize the diaphragm wall trench in the upper region directly under the ground surface where the suspension pressure is not high enough to counteract the earth pressure. They usually are 150 to 300 mm thick and are often pre-fabricated. They may possibly be re-used after finishing and hardening of a certain section of the wall.

The distance between the inner vertical faces of the guide walls should be the thickness of the cutoff wall plus 50 mm (Tamaro et al, 1992).

In case of high groundwater pressure, as it may occur with artesian conditions it may be necessary to place the guide walls at a higher level and augment the distance to the groundwater table in order to increase the fluid pressure against the trench walls. The unit weight of the slurry can be increased by adding barite.

The alignment of the guide walls should be surveyed periodically to assure that undetected movements have not occurred during panel excavation.

3.4.2 Equipment for trench excavation

When selecting an optimum method of excavation, the following considerations should be made:

- Possible and required depth of the connection to an impervious stratum
- Equipment dimensions and weight relative to the boundary conditions of the construction site
- Accuracy of excavation guidance control
- Performance capability of the selected method

With the excavation tools available today, different types of excavation procedures can be distinguished, including:

- Excavation by means of a fixed excavator (Kelly grab)
- Excavation by means of the cable grab
- Excavation by means of the hydraulic grab
- Excavation by means of a hydro-mill
- Excavation without the use of a supporting fluid

These excavation procedures are described in further detail in the following subsections. However, other procedures, such as excavation with long-stick backhoe, which can reach up to approximately 20m of depth and is efficient for shallow depths in easily excavated sands and gravels, are also used.

3.4.2.1 Kelly grab and hydraulic grab on rigid arm

Experiences made with the Kelly grab (Fig. 3.2) and the hydraulic grab on a rigid arm are not altogether positive. These grabs, for example, are often in need of repair due to high moment forces in the transition zone between the guide arm and the grab. A further disadvantage of this technique is the fact that the trench depth is limited to about 40 m. However, due to the fixed attachment of the grab to the carrier vehicle, it is possible for less experienced grab operators to work with this machine, which is not the case with other types of excavators.

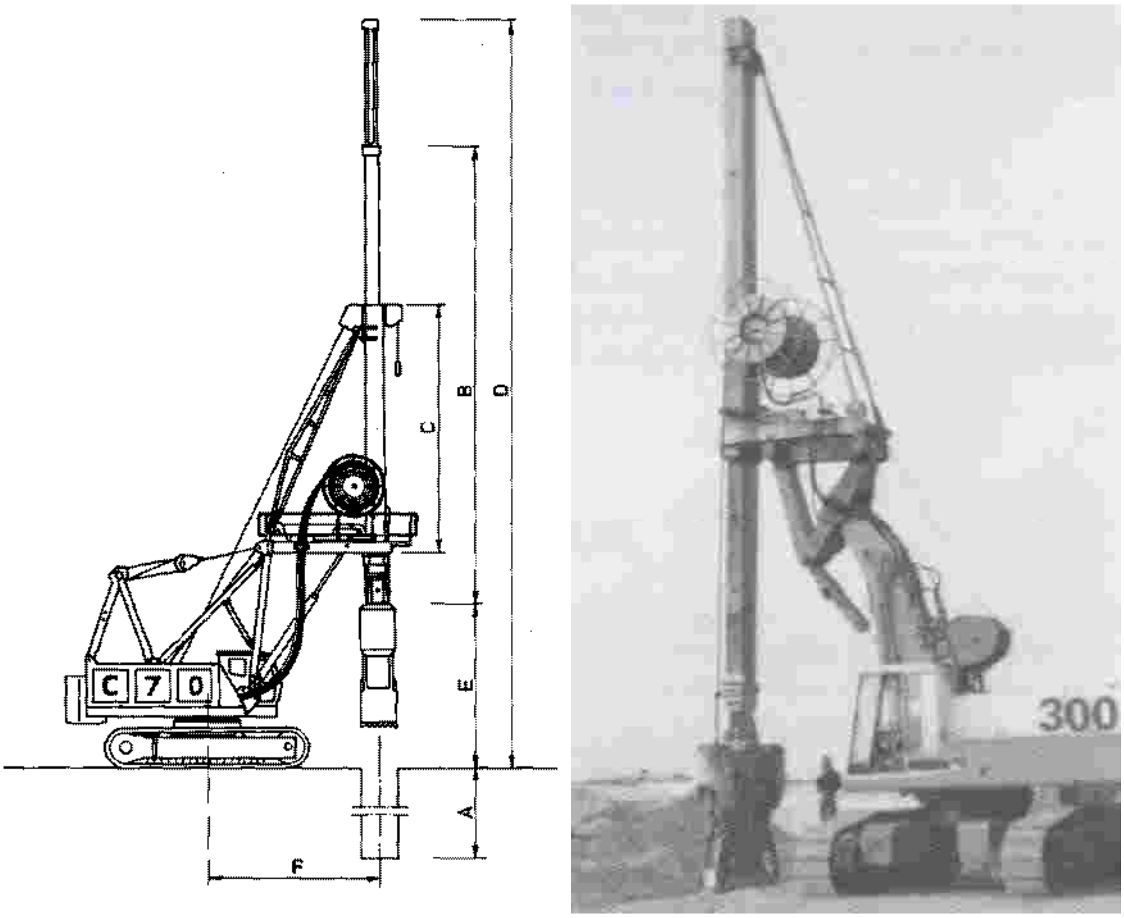

Fig. 3.2 Kelly grab (Casagrande system)

The positioning accuracy of this equipment is rather marginal. Deviation of the trench from the vertical can be up to 3 % of the trench depth. But, with specialized monitoring tools it may be possible to reach accuracies of about 1 %.

The performance capability of this equipment in medium dense or medium stiff soil amounts to about 4 to 5 m of excavation depth per hour. Ancillary equipment required to use with the Kelly grab are listed in Table 3.1.

Table 3.1 Equipment required for trench excavation using the Kelly Grab

KELLY GRAB	
Carrier vehicle (excavator)	• 30 ton to 60 ton overall weight (fully equipped) • 18 m lifting height
Arm drive	• Approx. 70 kW power rating
Grab	• Grab weight incl. telescopic arm approx. 20 tons • Jaw opening width 2 m to 4 m • Grab width 0.5 m to 1.2 m

3.4.2.2 Cable grab

The cable grab (Fig. 3.3) allows the excavation of considerably greater trench depths. The closure of the grab by means of a cable winch generates on the grab a vertically upward acting force, which can be as much as one third of the grab weight. However, the surcharge on the grab bucket is reduced by the amount of the closure force, which in turn reduces the cutting performance. Particularly in the case of soil types which are difficult to loosen, this excavation method is not suitable.

Examples of diaphragm walls produced by this method are: (a) a twin-phase diaphragm wall with a depth of 104 m constructed at Zell am See in Austria, in 1994, and (b) a 131m deep diaphragm wall installed at the Manicougan-3 dam site in Québec, Canada, by means of a cable grab.

The positional accuracy that can be achieved with the cable grab method is 0.5% of the trench depth provided inclinometers are available for online position monitoring together with a large grab frame (H>7m). By recording the inclination data, it is thus possible to exactly represent the position of each panel (quality monitoring). A positional accuracy of 1% to 2% may be expected with an experienced machine operator. Further improvements of the positional accuracy may be obtained with the aid of a controllable pivot between grab and the support frame. With such an arrangement it is possible to achieve directional corrections in the vertical of up to 1°. The performance capability in medium dense gravels is of the order of 120 m^2 in one 8-hour shift. At large depths, the deployment of hydraulically driven winches is indispensable, as this allows the lowering and lifting speed of the grab to be increased considerably.

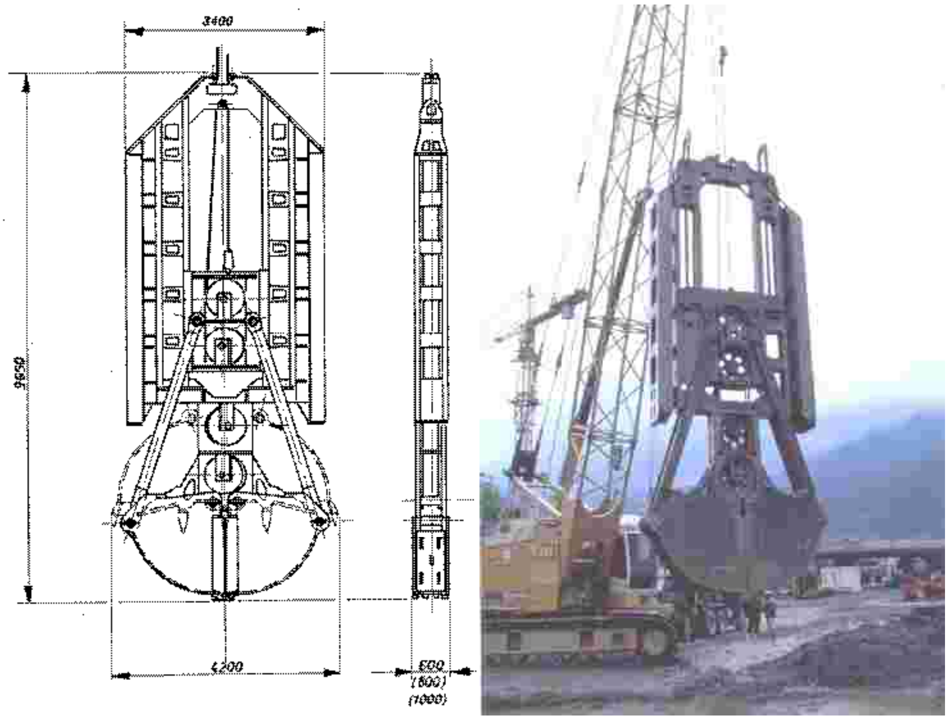

Fig. 3.3 Cable grab (Leffer system): Excavation of a 104m deep trench at Zell am See (Austria)

The required ancillary equipment for use with the cable grab is listed in Table 3.2

Table 3.2 Equipment required for trench excavation using the cable grab

CABLE GRAB	
Carrier vehicle (excavator)	• Up to 21 m boom height • 70 to 100 tons overall weight (fully-equipped)
Winch of grab cable	• up to 200 kW power rating • lowering and lifting speed 50 m/min to 100m/min • grab weight 8 tons to 20 tons • bucket closure force up to 300 kN

3.4.2.3 Hydraulic grab

The hydraulic grab (Fig. 3.4, Fig. 3.5 and Fig. 3.6) represents a considerable improvement in the cutting performance as compared to the cable grab. In dense or stiff soils a trench width of over 1.2 m may be achieved if the grab buckets are hydraulically operated. When closing the grab buckets, the hydraulic grab is able to activate its full dead weight during excavation. In the case of the cable grab, up to one third of the surcharge is lost because of the closure force. At present, trench depths of up to 150 m have been reached with the hydraulic grab technique.

In the case of the hydraulic grab, the hydraulic lines must extend as far as the grab hydraulics. The hydraulic hoses are guided over large rollers which are mounted on the carrier vehicle. For this reason lowering and raising the grab within the trench is slower than with the cable grab. On the other hand, boulders larger than the trench width, which cannot be lifted in one piece, may be easier to crush than with the cable grab because of the higher closing force. For this reason, a greater operational safety is created with the hydraulic grab as compared to the cable grab. Crushing boulders with heavy chisels is of course also possible.

Monitoring the excavation process in the case of the hydraulic grab is about the same as with the cable grab. Because of the lower level of loading (vibrations) during excavation, the hydraulic grab may be operated with a slightly smaller carrier vehicle than the cable grab. Corrections in the position of the grab during excavation can be made with the aid of control plates mounted on the frame. The control plates enable the frame to be slightly tilted relative to the vertical axis of the trench. For the same grab dimensions hydraulic grabs are somewhat heavier than cable grabs

The first hydraulic grabs used for cutoff construction were deployed around 1960 and were operated with hydraulic cylinders mounted on the sides. Today, the closing operation is performed mostly by a centrally-located cylinder.

The performance capability of modern hydraulic grabs for cutoff wall construction in dense gravel is up to 200 m^2 in one 8-hour shift. Diaphragm wall grabs with opening width of up to 4.5 m are used for this purpose.

The ancillary equipment for use with the hydraulic grab is listed in Table 3.3.

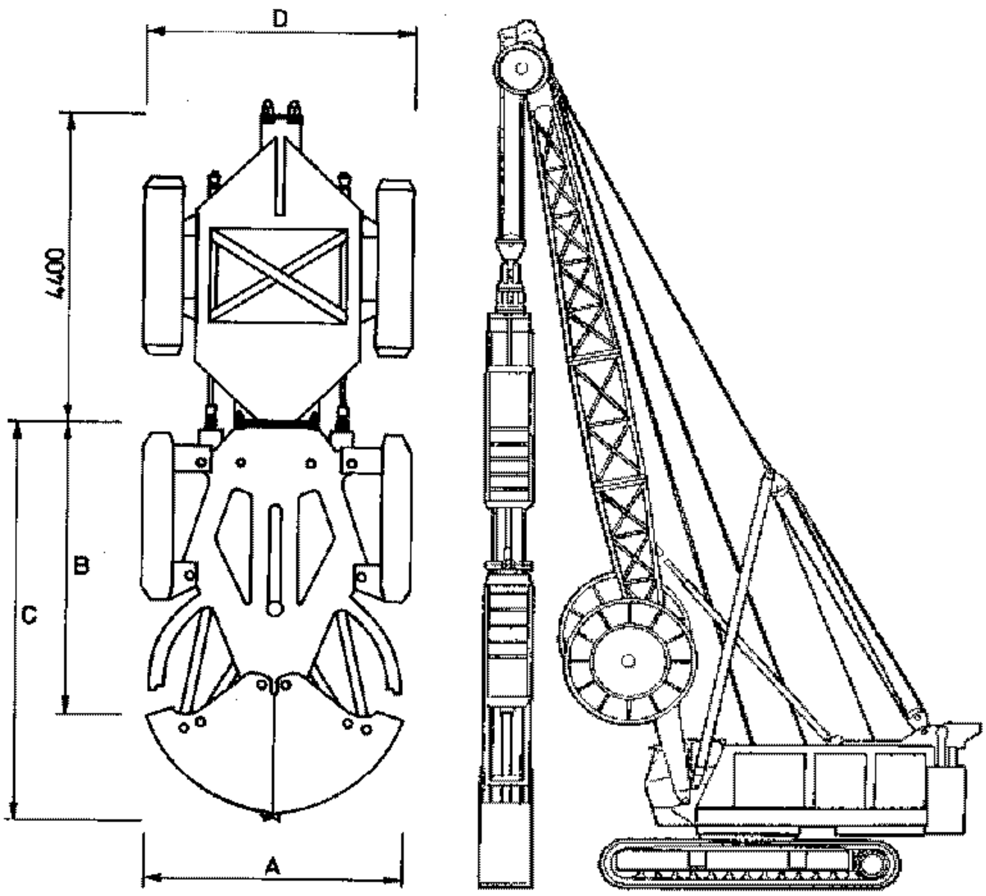

Fig. 3.4 Hydraulic grab (Casagrande system)

Table 3.3 Equipment required for trench excavation using the hydraulic grab

CABLE GRAB	
Carrier vehicle (excavator)	• Up to 18 m lifting height • 70 to 100 tons weight (fully-equipped)
Grab	• 80 kW power rating of grab bucket drive • lowering and lifting speed up to 60 m/min • grab weight 8 to 22 tons • bucket closure force up to 1500 kN

In order to ensure a proper connection between the panels, careful attention must be given to the joints. There are various types of joints; three of them are shown in Fig. 3.7. The circular stop-end joint is widely used with structural, load-bearing diaphragm walls made of rigid concrete. By removing the stop-end tubes from the primary panels, a semi-circular guide is provided for the adjacent secondary panel (Millet et al. 1992). Extraction of the stop-ends must occur before the final setting of the concrete and the pipes should be moved continuously to prevent them from sticking to the setting concrete or cement grout. The proper timing for this extraction is often difficult to determine. Stop-end pipes can be used up to the depth of about 40 m. An improvement is the CWS joint where the end form is pulled out laterally only after the adjacent panel has been excavated (Vanel, 1992; Dupeuble, 1985).

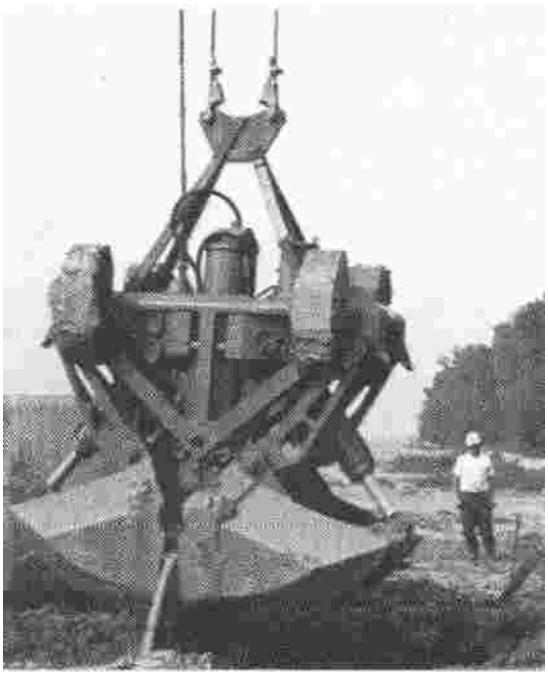

Fig. 3.5 Hydraulic grab (ICOC Veder type, 1965)

Fig. 3.6 Hydraulic grab (Bauer type 1998)

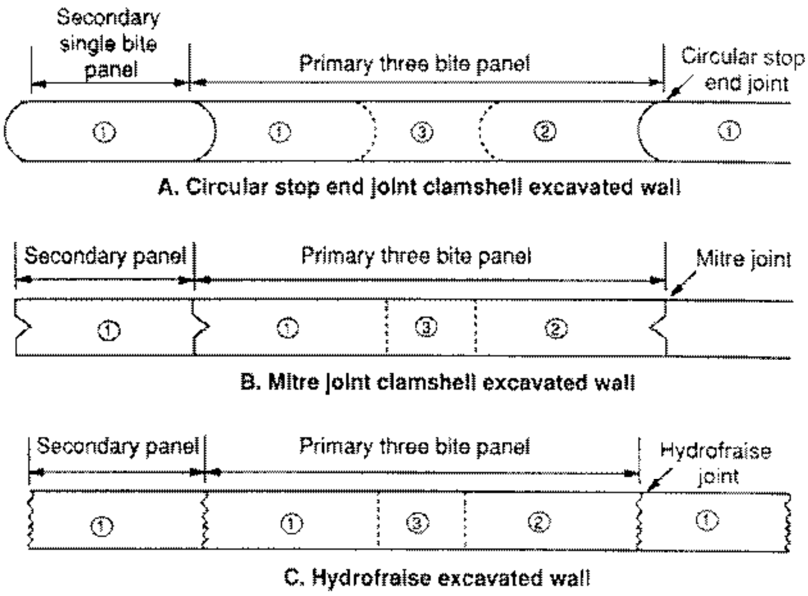

Fig. 3.7 Three different types of diaphragm wall joints; all of them have similar panel sizes (Millet et al., 1992)

3.4.2.4 Hydro-mill or trench cutter

At the end of the 1970s the French company SOLETANCHE developed the so-called hydro-mill, known under the trade name of "*Hydrofraise*". This is a continuously operating trench cutter employing the reverse circulation rotary cutting technique. It loosens and breaks up the soil by two drums fitted with steel bits counter-rotating on horizontal axes (**Fig. 3.8**). The cuttings are transported in a slurry which is pumped to a recovery station. The method has considerable advantages over conventional excavation methods, especially in the case of deep trenches and soil types difficult to loosen. For instance, no time is wasted to lift the cutting tool, a factor which considerably reduces productivity at great depths in the case of grabs. In addition, the hydro-mill permits continuous monitoring of the deviation of the trench walls from the vertical position and control of the cutter tool. This results in a maximum deviation of the trench from the vertical of less than 0.5 % of the depth.

Excavation by hydro-mill can ensure a clear tight connection at the panel joint, especially also for deep walls, by shaving off 0.2 to 0.3 m of the adjacent panels (Fig. 3.7).

Using the toothed hydro-mill, it is possible to cut through rock with an uniaxial compressive strength of up to 100 to 120 MPa. The compressive strength of the rock is, however, not the only factor governing the economic viability of the hydro-mill cutter deployment. Another important factor is the wear of the machine, which largely determines the performance capability of the cutter. As worn teeth must be replaced more frequently, the cutting performance is reduced. The Cerchar Abrasiveness Index (CAI), which is determined as the abrasion of a metal pin scratched over freshly cut rock (Suana & Peters, 1982), may be adopted as a measure of wear to be expected. Fig. 3.9 enables an estimation of the economically-viable application of the hydro-mill cutter. This lies below the gray region. Table 3.4 presents the required ancillary equipment for use with the hydro-mill.

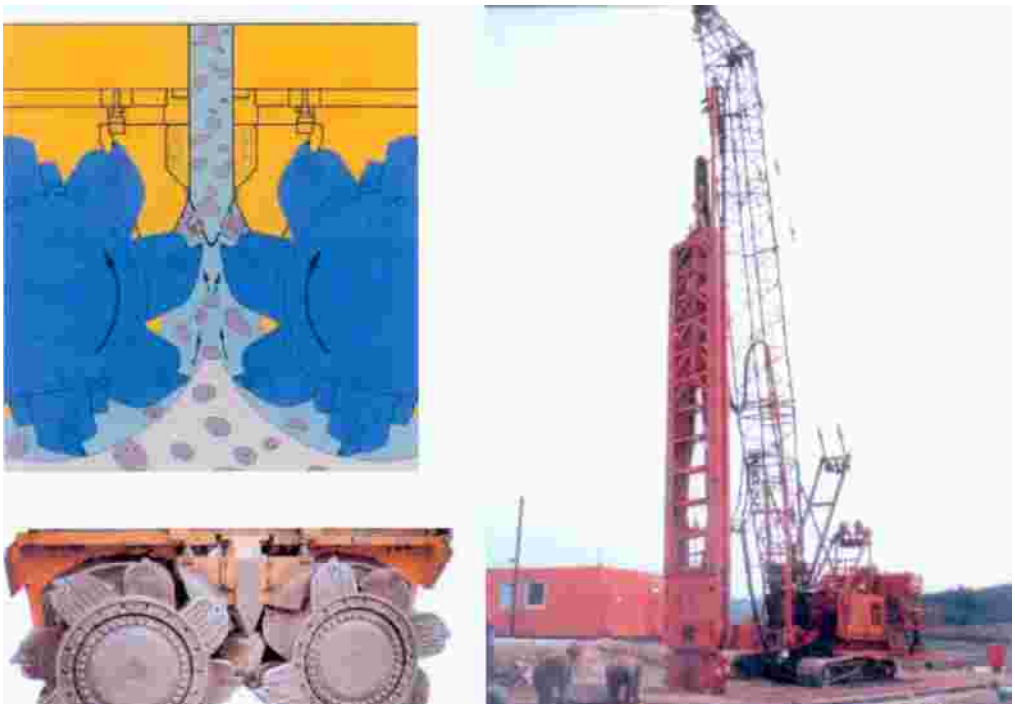

Fig. 3.8 Milling tools and equipment on site: Bauer system (left) and Solétanche-Bachy system (right)

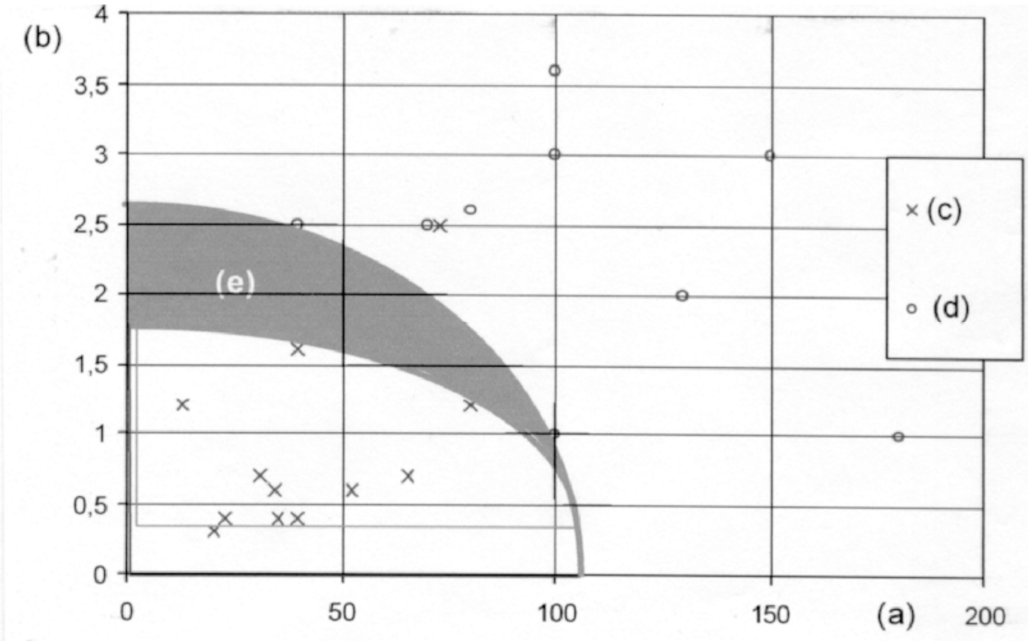

Fig. 3.9 Limits of hydro-mill trench-cutting capability (Strobl 1999)

(a) Uniaxial compressive strength of rock, q_u (MPa); (b) Cherchar CAI value (-);
(c)Cutting performance > 0.05 m^2/min; (d) Cutting performance < 0.05m^2/min
(e)Transition zone

Table 3.4 Equipment required for trench excavation using the hydro-mill

HYDROMILL – TRENCH CUTTER	
Carrier vehicle (excavator)	• 20 m boom height • 70 to 115 tons weight (fully-equipped)
Hydraulic cutter Hydro-mill cable winch Pump for transporting cut material	• 100 kW to 160 kW power rating per cutter wheel • Weight of hydro-mill 25 to 50 tons • lowering and lifting speed about 40 m/min • 250 m^3/h to 700 m^3/h slurry discharge • 100 kW to 300 kW power rating

During excavation of the trench, it is common practice to check the verticality in two directions by inclinometers and the twist of the cutter by a gyroscope (Fig. 3.10). With current technology, trench widths of 650 mm to 3200 mm can be achieved. The usual length of the panels excavated by the trench cutter ranges between 2400 and 3200 mm and their depth can reach as much as about 150 m. The hydro-mill usually works in combination with a hydraulic grab. Typical panel lengths in recent projects were 7.8 m for the primary panels, excavated in three bites by the grab, and 2.8 m for the secondary panels.

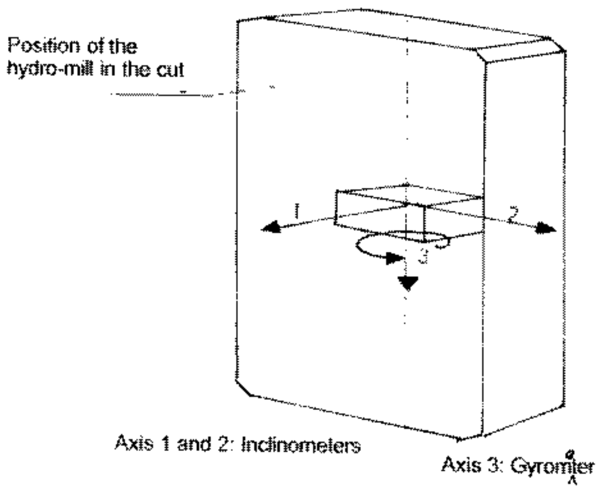

Fig. 3.10 Position control of the hydro-mill (online) (Solétanche-Bachy)

The cutoff wall construction method by means of the trench cutter, originally developed for twin-phase operation, can also be adapted to single-phase cutoffs.

For application in hard rock, the Bauer Company developed a cutter with *roller bits* instead of cutter teeth (Fig. 3.11). This cutter offers considerable advantages in the case of stratified rocks with extremely hard individual layers. With the roller bit hydro-mill, it is possible to attain trench depths of over 200 m.

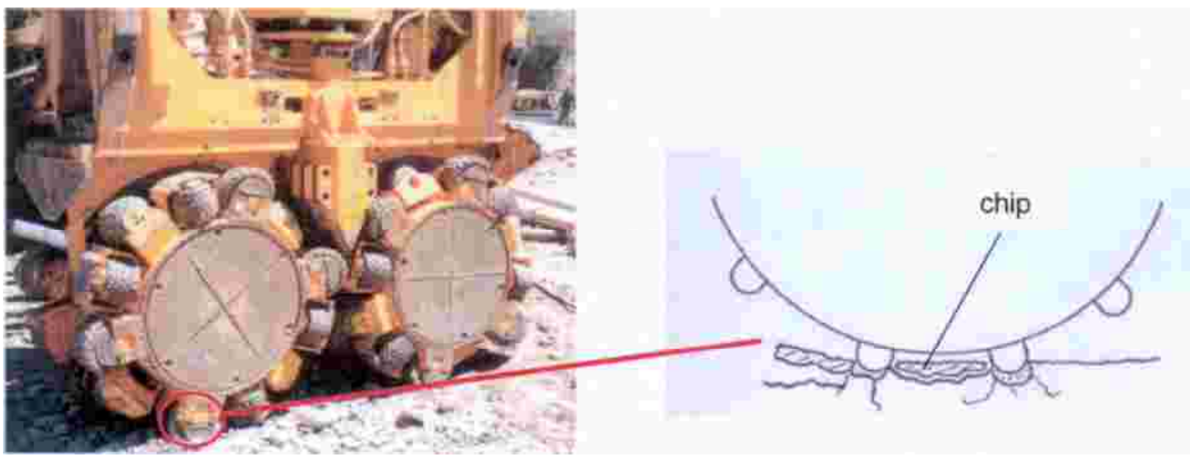

Fig. 3.11 Roller bit cutter wheel and illustration of the loosening process

The cutter frame and the auxiliary machine equipment of the roller bit hydro-mill are similar to those of the toothed cutter. In the case of the roller bit hydro-mill, however, the rock to be loosened is not torn away as with the toothed cutter but crushed. For this reason the loosened material consists of smaller pieces than what is obtained from the toothed cutter. The roller bit hydro-mill is more suitable for excavating hard rock. Capability limits of this cutter, as quoted by Bauer, are shown in Fig. 3.12).

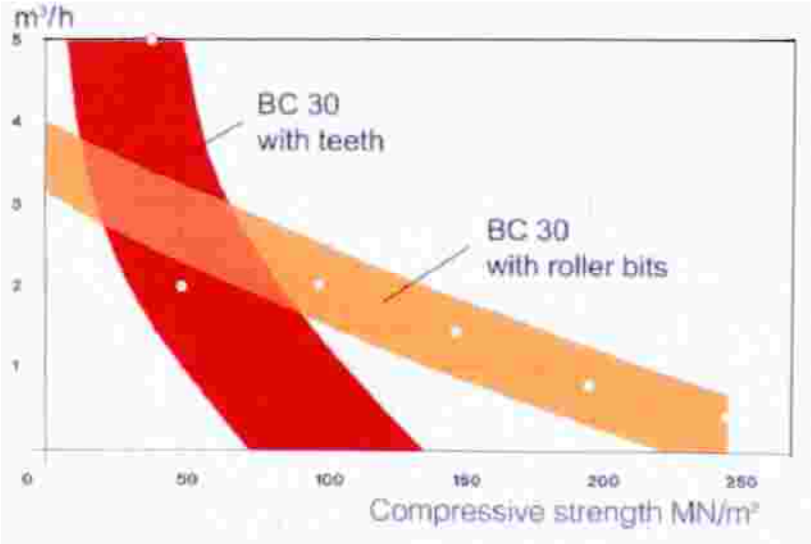

Fig. 3.12 Capability limits of roller bit and toothed cutter (Bauer system)

For both types of cutters, during operation, the cuttings are pumped in a suspension flow from the cutter elevation to the top of the trench. The suspension is then reprocessed and the cuttings are removed from the suspension in a desander or in a so-called cyclonic plant. The latter separates the chips from the fluid through centrifugal action. A detailed description of this process is beyond the scope of this chapter.

3.4.2.5 Excavation without stabilizing fluid

This method can be used for the construction of plastic concrete diaphragm walls of small depth, i.e. less than about 4 m. The principle of the method is illustrated in Fig. 3.13 and consists of the following steps (DVWK, 1990):

1) A stable trench is excavated in the foundation or in the embankment fill, lined with a geotextile and filled with plastic concrete
2) The surface of the plastic concrete in the trench is covered by a geotextile upon which a new fill layer is placed
3) Excavation of a trench in the new fill layer reaching down to the previously cast concrete surface.
4) The protective geotextile is removed and the surface cleaned. Then the trench is again filled with plastic concrete up to the surface of the new fill layer.

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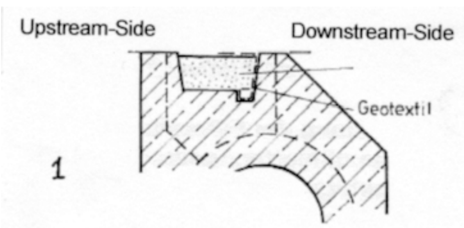
<tr>
<td>
</td>
<td>STEP 1:
• Connection of the trench wall to an inspection gallery or plinth
• Placing geotextile mats into the connection recess of the inspection gallery
• Filling the recess with plastic concrete
• Fill placement up to the level of permissible excavation depth</td>
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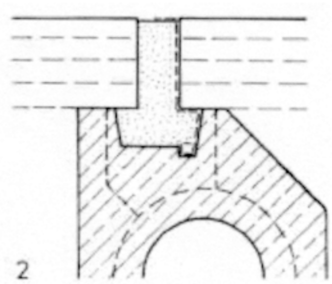
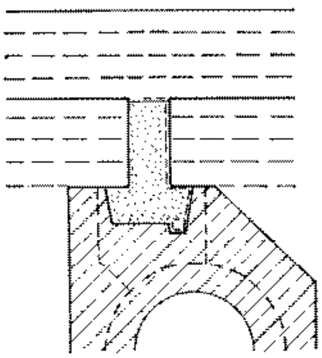
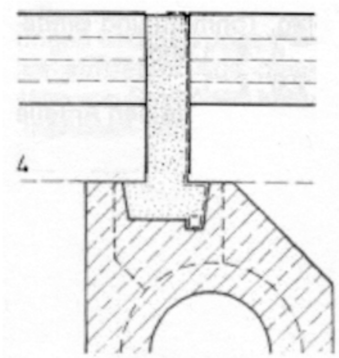
<tr><td>
</td><td>STEP 2:
• Excavation of a slot in the dam fill
• Placing geotextile mats on the exposed side of the trench wall with appropriate overlapping
• Cleaning of slot and filling it with plastic concrete</td></tr>
<tr><td></td><td>STEP 3:
• Placing fill up to the level of the permissible excavation depth</td></tr>
<tr><td>
</td><td>STEP 4:
• Excavation of a trench (slot) in the embankment fill
• Continuing as shown under Step 2</td></tr>
</table>

Fig. 3.13 Construction of an internal dam cutoff (trench wall) (modified from DVWK, 1990)

This method is particularly suitable for small dams in countries with low labor costs. The equipment required for this method is relatively simple and locally available, and it comprises the following:

- Conventional back-acting excavators (back-hoe)
- Compressed air cleaners
- Suspension mixing plant or pre-swell and mixing containers for bentonite
- Mixing plant for plastic concrete

An example of this method is illustrated in **Erreur ! Source du renvoi introuvable.** where a plastic concrete cutoff wall was constructed for Wadi Tanuf in the Sultanate of Oman:

Fig. 3.1 Plastic concrete wall construction in Wadi Tanuf (Sultanate of Oman)

3.1.1 Quality control

The following quality criteria must be addressed when excavating a trench for a diaphragm wall:

- Verticality of the trench in order to avoid deviation of the individual panels in both directions i.e. perpendicular and parallel to the trench line. Control is by inclinometers and/or echo sounder (e.g. made by Koden). The echo sounder utilizes ultrasonic waves traveling from a sensor through the slurry to the wall of the excavated panel slot and, after reflection, back to the sensor. From the travel time verticality of the wall and possible defects, such as cave-ins, can be deduced from the record. The method cannot work if the slurry is too dense.
- Sufficient depth of the wall to achieve sealing objective
- Sufficient overlap between adjacent panels

Minimum standards for quality control are:

- Deviations in excavation perpendicular to the plane of the wall should not exceed 1/3 of the thickness of the diaphragm wall
- Deviations in the plane of the wall should not exceed 1/3 of the overlap between panels (step-length control)
- Deviations during excavation should be limited to 1% of the current excavation depth. Highly experienced contractors may be able to comply with deviations of not more than 0.5%.
- Progress in wall construction and current deviation values should be represented graphically. This applies particularly to single-phase walls where the time sequence of panel slot excavation should be documented precisely. (Compliance with a maximum possible processing time).

- The depth of embedment in an impervious stratum can be inferred from the depth of the panel and the rate of excavation. The design assumptions have to be verified by checking the excavation material. Nevertheless, the bottom of a diaphragm wall can be a potential weakness because cuttings and caved material may have accumulated, which may lead to a wall with inadequate sealing efficiency. For this reason diaphragm walls are often taken an extra depth into the impervious base.

3.1.2 Single-phase versus twin-phase cutoffs

Single-phase diaphragm walls have certain advantages over twin-phase walls, namely: Reduced cost in producing the wall because the stabilizing fluid, which in the single-phase method is produced from water, bentonite, rock flour, cement, and usually additives, will remain in the trench and will start setting after excavation of the trench. The time during which the slot for a panel may be excavated is therefore limited by the time during which the slurry is still workable. During the setting process the slurry should harden without any disturbance. Based on a comprehensive testing program investigating the setting behavior of fresh bentonite-cement slurries carried out by Caron (1973), it was found that the duration of excavation should not exceed eight hours. During this time interval it is possible to achieve a trench depth of 50 to 60 m under normal site conditions using the grab method. Hence, the maximum attainable depth is given by the working time and based on experience and is in fact about 60 m (Fig. 3.15). Alternatively, for easily excavatable sand and gravel deposits up to about a depth of 20 m the use of the long-stick backhoe (Fig. 3.16) may be more efficient. Table 3.5 compares grab characteristics with those of the long-stick backhoe.

1 Excavation
2 Concrete placement
3 Impervious stratum
4 Pervious layer to be sealed

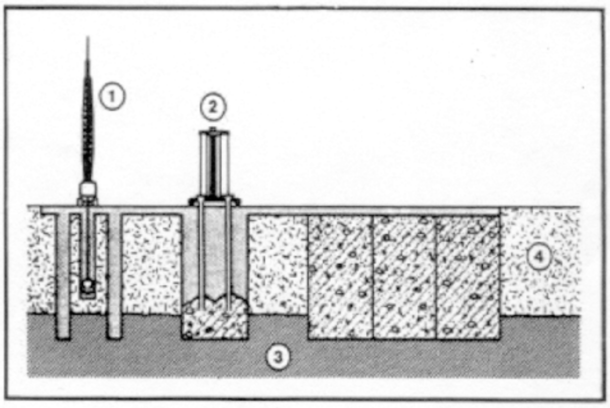


Fig. 3.2 Twin-phase method trench excavated under bentonite slurry and filled with concrete by tremie (ICOLD, 1985)

For excavation with the hydraulic grab, the pilgrim step technique (**Erreur ! Source du renvoi introuvable.**) is mostly used in the single-phase method, in order to avoid disturbing the setting phase of the immediately adjacent panel.

Fig. 3.16 Long-stick backhoe excavator

Table 3.5 Long-stick backhoe and grab excavators specifications (Di Molfetta et al., 2006)

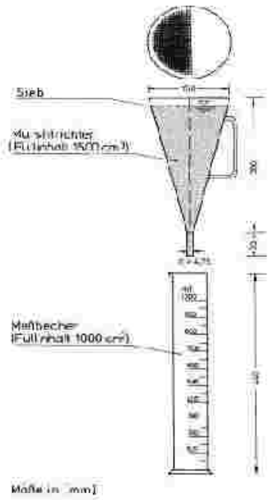
	Backhoe excavators[1]	Grab excavators[2]
Max. power	50-485 kW	240-400 kW
Base machine weight	7,000-110,000 kg	42,000-300,000 kg
Lifting capacity	3,500-40,000 kg	20,000-30,000 kg
Weight of bucket/grab	300-3,000 kg	8,000-24,000 kg
Excavation width	0.4-3.0 m	0.5-1.2 m
Excavation length	-	2 – 4.2 m
Capacity of bucket/grab	0.2-1 m^3	1-1.2 m^3
Bucket/grab digging force (ISO)	50-430 kN	300-400 kN
Stick crowd force	Inversely proportional to stick length	-
Excavation depth	0-30 m	0-70 m
Excavation rate	400 m^2/day	300 m^2/day

3.2 MATERIALS FOR STABILIZING SLURRIES

3.2.1 Required properties of fresh bentonite slurries and quality control

The required properties of the slurries used for supporting excavated trenches are governed by the boundary conditions of the slurry trench construction conditions and by the characteristics of the soils forming the trench walls. The requirements for the fresh slurry and the available methods of quality control are summarized in Table 3.6 and Table 3.7. Further details can be found in ICOLD Bulletin 51 (ICOLD 1985).

Table 3.6 Requirements for slurry properties in the fresh state

PROBLEM	GOALS TO ACHIEVE AND ADVERSE PRCESSES TO OVERCOME	METHODS OF CHECKING, MITIGATION AND TESTING
Slurry stability	Maintaining chemical stability of the slurry, also in aggressive groundwater (e.g. sulphate bearing)	A sample of the slurry can be mixed in the laboratory before its use on site with the mixing water containing at least 30% of aggressive groundwater
	In aggressive groundwater some types of bentonites may shrink and the slot can collapse	The pH-value and electrical conductivity of the groundwater must be checked periodically
Trench stability	Generating sufficient support to stabilize the slot wall	Checking the density of the slurry by the mud balance
	Prevention of squeezing the slurry out of the slot by the horizontal earth pressure	The stabilizing hydrostatic pressure exerted by the slurry is given by its density
Slurry loss into trench wall	Minimizing slurry losses into the base of the excavation	The yield stress is the controlling parameter. It enables the transfer of shear stresses from the slurry to the pore wall.
	Slurry may penetrate into the pores of the soil forming the trench wall. Larger pores promote increased flow.  Marsh funnel	In case of very large pores (e.g. in gravels) addition of sand may be helpful to mobilize the yield stress as well as other geometrical constraints (filters). Especially in very deep cutoffs the yield stress of the slurry becomes increasingly important with regard to the expected suspension losses. In the case of narrow pores, slurry losses occur more slowly and the viscosity of the slurry may also have an influence on the rate losses. The Fann V-G rotational viscosimeter can be used to design the slurry mixture in the laboratory while the Marsh funnel viscosimeter is employed for checking the slurry at the site.
Slurry recycling	Sufficient recycling capacity of used slurry.	Separation speed of excavated material from slurry (desanding) is controlled by the yield stress and the viscosity.
	During loosening and removal of the soil from the cutoff trench the slurry is contaminated with soil. Especially with trench cutters the excavated material is thoroughly mixed and the slurry needs to be cleaned which takes place in a desander.	The magnitude of yield stress and viscosity must be limited to their absolute acceptable minimum because excessive values in these parameters introduce unnecessary operational complications. These parameters must be checked by the Fann V–G rotational viscosimeter and by the Marsh funnel.

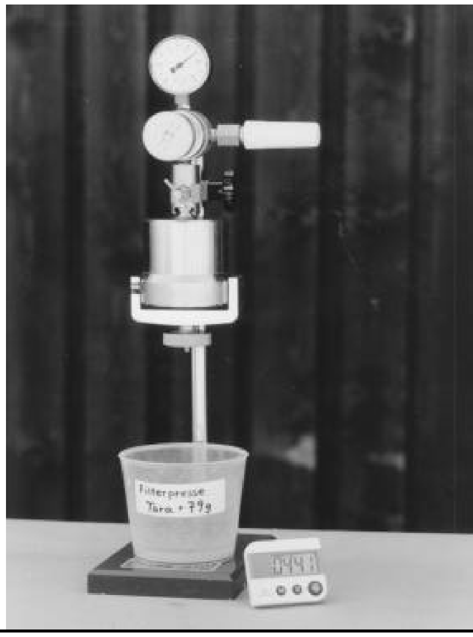
Slurry segregation	Slurry segregation inside the trench may cause stability problems and suspension losses.	Slurry segregation is related to slurry stability
	Separation of the soil constituents from the slurry within the slot must be avoided. In deep trenches there is a high risk of solid materials accumulating at the base of the trench, which can produce insufficient density of the slurry in the upper parts of the trench. The solid material can also drastically change yield stress and viscosity of the slurry	The bleeding of the slurry must be as low as possible in order to achieve uniform slurry properties over the entire depth of the trench and ensure stability in its upper part. The choice of a suitable bentonite will guarantee suspension stability even in contaminated or untreated groundwater.
Filter cake formation	Prevention of excessive filter cake formation along the contact between slurry and natural ground of the trench wall. With fine-grained soil types the slurry is pressed against the trench wall due to the excess hydrostatic pressure. Low permeability prevents the colloidally-mixed suspension particles to penetrate into the pores of the soil forming the trench walls. If the slurry is not sufficiently stable separation between water and the solid constituents of the slurry may occur with the water flowing into the pores while the solid material form a cake on the trench walls. This may cause a reduction of the trench width which may impede the excavation tools from entering the trench if the cake becomes thick enough. Filter press	With the Baroid filter- press test the separation susceptibility can be determined by the application of an excess pressure. The extruded water filtrate serves as a measure of the separation susceptibility. A more precise test is the stagnation – gradient test. 

Table 3.7 Methods of measuring properties of fresh suspensions, recommended limiting values and measurement frequencies

QUANTITY TO MEASURE, MEASURING DEVICE AND PROCEDURE	RECOMMENDED LIMITING VALUES AND MEASUREMENT FREQUENCY
Slurry density, ρ Apparatus: Mud balance, hydrometer Measurements may be performed up to 10 minutes after mixing	Stabilizing suspension: 1.1 Mg/m^3 < ρ < 1.25 Mg/m^3 Single-Phase slurry: 1.2 Mg/m^3 < ρ <1.3 Mg/m^3 Two measurements per layer
Marsh cone viscosity Apparatus: Marsh funnel viscometer Measurements must be performed immediately after mixing Basic fill volume: 1.5 liters Results: runout time, t_M, for 1 liter	The recommended Marsh cone viscosity of the stabilizing slurry is dependent on the filling material used for the twin-phase wall: Concrete: $t_M \approx 43$ s Plastic concrete: 54 s < t_M < 60 s Earth concrete: t_M > 43 s Single-phase suspension: 40 s < t_M < 55 s (extreme case: at the end of excavation 76 s) Two measurements per layer
Yield stress, τ_F *Plastic viscosity, apparent viscosity* Apparatus: Fann V-G rotating viscometer	Can be determined from Marsh cone viscosity $\tau_F = 28.3 \ln t_M - 95.4$ (in Pa) Input value is the Marsh funnel viscosity t_M for 1.0 liter (in seconds)
10-min gel strength Apparatus: Rotational viscometer	
Filtration or fluid loss, F Apparatus: Filter press according to API Measurements should be competed within 10 min after mixing	Stabilizing suspension: F < 30 cm^3 Single-phase suspension: F < 100 (130 cm^3)
Filter cake Apparatus: Fluid loss test, triaxial test	
Bleeding, A Apparatus: Reading to be taken after at least 30 min settling time	Bleeding after 30 minutes: A < 3 % Bleeding after 60 minutes: A < 4 %

3.2.2 Selection of base materials for stabilizing slurries

Bentonites. Bentonites are clay minerals with a high swelling capacity. They are delivered to the construction site in powder form. By colloidal mixing with water in high speed mixers the water molecules penetrate between the silicate sheets of the clay particles causing a multiple increase in the volume of the bentonite (hydration). High speed mixers can reduce the hydration time from about 24 hours (without the use of mixers) to about 10 minutes.

Naturally occurring bentonites can be subdivided into sodium and calcium bentonites. By chemical treatment it is possible to increase the swelling capacity of the otherwise low-swelling Ca-bentonite. These bentonites are then referred to as activated bentonites. They have a swelling capacity nearly equal to that of Na-bentonites.

Na-bentonites or activated Ca-bentonites with swelling capacities of up to 600 % should be avoided in the case of untreated seepage water in the excavation trench and only Ca-bentonites or clay powder should be employed. In extreme cases it is conceivable to deploy polymers whose molecular structure is considerably less prone to shrinkage than that of Na-bentonite.

Filter ash/rock flour. Filter ash or rock flour can be added to the bentonite slurry if an increase in density is needed and when no other special requirements are placed on the flow properties of the slurry. The exclusive use of bentonite or clay powder would then be too costly and uneconomical. This may be necessary in the case of high groundwater pressures or low trench structural stability (e.g. when liquefiable sand layers are present). Before the commencement of construction work, however, it is necessary to ensure that the slurry is not susceptible to detrimental changes in properties due to "incompatibility" between the base materials. For example, some sort of bentonites are not compatible with certain types of cement or some filter ashes.

Sand. If large voids are present in the soil being excavated (e.g. layers with coarse gravel), addition of sand to the slurry may be practicable because it results in a more effective limitation of the penetration depth of the slurry into the soil than can be achieved by increasing the yield stress.

The influence of the base materials on the properties of a mix is shown in Table 3.8. Possible mixing proportions for one cubic meter of stabilizing slurry might consist of the following:

- Bentonite: 40 – 100 kg
- Rock flour: 0 - 300 kg
- Water: 850 – 950 kg

Table 3.8 Influence of base materials on the properties of a stabilizing slurry

Increase in the amount of:	Influence on density	Influence on yield stress	Influence on Marsh cone viscosity
Water	↓↓	↓↓	↓↓
Bentonite	↑	↑↑	↑↑↑
Rock flour	↑↑	↑	↑↑
Sand	↑↑	↑	↑↑

Note: ↑ increase ↓ decrease

3.3 MATERIALS FOR DIAPHRAGM WALLS

3.3.1 General requirements

The basic materials for the construction of diaphragm walls are:

- Bentonite, used both in bentonite-water suspensions (slurries) as support fluid in the excavated trench (see Section 3.5) or as part of a mix proportion for the wall material
- Cement, used as a component of diaphragm wall mix (slurry or concrete)
- Aggregate
- Water

"Diaphragm wall materials" means the construction materials composing the wall after the setting process. In the case of the single-phase wall, they refer to the set suspension; while in the case of twin-phase walls, they refer to the material set after filling the excavated trench in the second phase.

The requirements which must be satisfied by the diaphragm material for each of the wall types are listed in Table . Additional information on diaphragm materials can also be found in ICOLD (1985) and in Xanthakos (1979).

Table 3.9 Requirements and testing methods for diaphragm materials

Material requirements	(1) Controlling physical property (2) Possible testing method
Watertightness (sealing capability)	(1) Permeability (2) Permeability test
Deformability	(1) Modulus of deformation, Young's modulus (2) Uniaxial compression test (3) Triaxial test
Self-healing	(1) Particle size distribution (gradation) Bentonite fraction (2) Pinhole test
Erosion resistance	(1) Particle bond Permeability (2) Permeability test Pinhole test/self-healing Pinhole test + chemical erosion
Immiscibility with the first phase (twin-phase wall) or miscibility with the first phase (slurries)	(1) Difference in density between phases (2) Marsh Cone viscosity of Phase 1 Slump of Phase 2

Mix design is usually based on a reference mixture developed in the laboratory and modified by trial and error in a desired direction by a slight change in the constituents, or on required strength and stiffness properties of the mixture employing a comprehensive strength testing program (see Section 3.6.5). Once the specified properties have been attained in the laboratory, the mix proportions are passed on to the site. An attempt is then made using the on-site mixture to attain the properties of the laboratory mixture by only changing the mixing time. If the properties of the laboratory mixture cannot be achieved with the on-site mixer the mixing proportions are slightly altered again.

Batching of the mix in the laboratory and in the field may, however, differ. On the construction site, bentonite used for the mix, e.g. plastic concrete, is first mixed with water in a mixer until the bentonite is fully hydrated. This bentonite slurry is then added to the cement and aggregate mix. In the laboratory, sample preparation is often performed according to ASTM C192 where bentonite powder and cement are mixed together before aggregate and water are added. Mixing is then with all the constituents together in a mixer for a certain length of time during which the bentonite hydrates. The properties of the mixes produced by these two different procedures may not be entirely identical.

3.3.2 Single-phase mixes (Cement-bentonite walls)

In a single-phase slurry system the slurry properties must be balanced to fulfill both, support in the liquid state, and strength, watertightness, and erosion resistance in the set state. Typical characteristics should be:

- Density of the fresh slurry should be between 1.2 and 1.3 Mg/m^3. On completion of slot excavation the suspension density should be less than 1.44 Mg/m^3.
- Yield stress should be adapted to the surrounding material in order to avoid high fluid losses.
- Permeability should be between 10^{-7} and 10^{-8} m/s. However, if achievable, lower permeability, i.e. 10^{-8} to 10^{-9} m/s, should be specified.
- Strength of set suspension should be between 0.30 and 0.35 MPa after 28 days.
- Modulus of elasticity should be less than twice that of the surrounding material.
- The possible working time should be twice the theoretically necessary working time.
- Blast furnace slag cement is preferable over Portland cement owing to its high erosion stability and chemical resistance.

The classic single-phase cutoff wall consists of a cement-bentonite mix containing no aggregate other than some soil mixed with the slurry during excavation. The method re-uses the bentonite slurry in the excavation and transforms it into a construction material by adding cement. The unconfined compressive strength of the solidified material can reach values between 1 and 3 MPa.

The effects of adding different base materials on the properties of a single-phase slurry are listed in Table 3.10. Comparing this table with Table 3.8, cement has been added as a base material. Construction of these walls is in panels as illustrated in Fig. 3.1 or by a combination of back-hoe and panels.

Table 3.10 Influence of base materials on the properties of a single-phase suspension

Increase in the amount of	Density	Marsh cone viscosity	Yield stress	Final strength	Permeability
Water	↓↓	↓↓	↓↓	↓	↑
Rock flour	↑↑	↑	↑	insignificant	↓↑(*)
Cement	↑	insignificant	↑	↑↑	↓↓
Bentonite	↑	↑↑	↑↑	↑	↓

(*) Plot of rock flour versus permeability yields a curve with a minimum

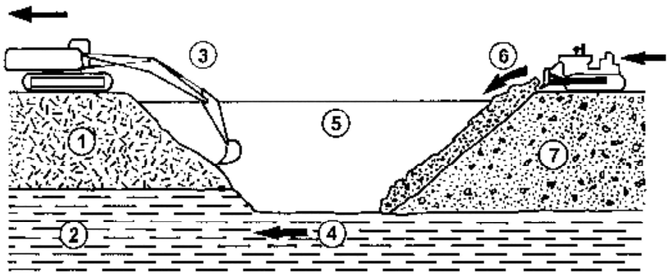
Possible mixing proportions for one cubic meter of a single-phase suspension may look as follows:

- Rock flour or filter ash: 80 – 350 kg
- Blast furnace slag cement: 150 – 350 kg
- Bentonite: 30 – 50 kg
- Water: 750 -800 kg

Correctly produced single-phase diaphragm walls are capable of sustaining a hydraulic gradient of i ≈ 50. In 2002, the cost (year 2002) of producing single-phase walls using the grab excavator was approximately between US$ 65.00 and US$ 100.00 per square meter of sealing surface.

3.3.3 Soil-bentonite walls (slurry trenches)

The slurry trench is produced by mixing soil material (aggregate, i.e. suitably graded sand and gravel) into the existing stabilizing bentonite-water slurry. Depending on the hydraulic gradient which may later act on the slurry wall, some cement may be added. The trench is excavated by backhoe, drag bucket or clamshell (Fig. 3.17). The slurry-impregnated aggregate has a grain-size range from 0.2 mm to 30 mm and is usually extracted from the excavation. Thorough mixing of aggregates and slurry is essential and is usually achieved in a mixing station or with a dozer beside the trench. During mixing dry bentonite may also be added as required. The aggregate should have a smooth grain size distribution curve and the slump of the mixture should be in the range 10 to 20 cm.



Construction of S-B wall with slurry trench method

1 Permeable in situ ground
2 Impermeable layer
3 Trench excavation
4 Direction of advancement
5 Trench filled with bentonite slurry
6 Backfill placement
7 Completed cutoff wall

Fig. 3.17 Soil-bentonite wall excavated by a backhoe under bentonite slurry, then refilled with slurry impregnated aggregates. Method limited to shallow trenches (modified from ICOLD, 1985).

The slurry trench method is only applicable for diaphragm walls of small depth. Slurry trenches may sustain a hydraulic gradient of up to i = 30. Production costs for slurry trench construction range between about US$ 40.00 and US$ 80.00 per square meter of sealing wall surface. A possible mix proportion for one cubic meter of slurry wall material can be as follows:

- Aggregates d_{max} = 25 – 38 mm: 950 – 1350 kg
- Bentonite suspension: 450 - 550 kg
- Bentonite fraction in the suspension: 5- 15% (by weight)depending on swelling capacity
- Cement: 0 – 50 kg

3.3.4 Mix proportions for rigid concrete diaphragm walls

The materials for a twin-phase system may be independently matched to fulfill a specific task. Twin-phase walls are mainly used in dam construction, but rigid concrete walls have all but disappeared. They found application in the case of:

- very high hydraulic gradients (e.g. i > 100)
- very large depth of panels (e.g. > 60 m)
- where it was necessary to carry or resist large static loads

For economical reasons alternative methods are preferred. Although it is possible to produce rigid concrete walls without reinforcement this is seldom practiced because the non-reinforced rigid concrete wall has hardly any advantages over the single-phase wall but incurs far higher costs. Intensive reinforcement has the task to sustain the loads during fill placement and impounding. Possible deformations between the dam body and the sealing wall must be minimized to prevent fracture of the wall panel in the slot.

Homogenization of the suspension in the slot must be accomplished prior to concreting. This is generally achieved by raising and lowering the excavation tools several times.

The concrete which is filled into the slot as the second phase material should satisfy the following requirements:

- The cement content should be in excess of 350 kg/m^3; the water/cement ratio, w/c should be above 0.67.
- The difference in density between the concrete and the stabilizing slurry should be at least 0.24 Mg/m^3
- The consistency of the concrete should correspond to a slump greater than 180 mm in order to avoid trapping of slurry bubbles in the concrete wall.
- To ensure sufficient stability against separation, the gradation of the aggregates of the concrete plays an important role. The largest particle size should be less than 25 mm. Rounded particles are preferred. The aggregate should be comprised of at least 40 % of fine fractions (< 0.075 mm).
- When employing stop-end tubes, the concrete should reach a sufficiently high strength after eight hours such that work can be carried out continuously in two day shifts of eight hours each and the stop-end tubes can be withdrawn
- The bonding strength of the concrete on the stop-end tubes should develop in such a way that their withdrawal is possible over a prolonged period.
- The permeability of the concrete should be between 10^{-9} and 10^{-13} m/s.
- The compressive strength after 28 days should be between 20 MPa and 31.5 MPa.
- Retarding agents, plastifiers or other additives should only be used if a deterioration of properties in the long-term can be excluded, or when the consequences of an additive are know and are acceptable.

Special attention must be paid to placing concrete into a slot already filled with stabilizing slurry. For this case the following basic rules apply:

- Free fall of the concrete from the surface of the trench into the suspension is not permitted; the concrete must be placed by means of a tremie pipe.
- Concrete filling should start out at the ends of the slot from where the concrete is worked towards the middle of the slot
- Horizontal working joints in individual panels must be avoided
- Vertical working joints are usually produced by means of stop-end tubes. Where necessary, the adjacent already existing panel is milled to create an interlocking connection with the neighboring slot elements.

A correctly produced rigid concrete diaphragm wall can sustain a hydraulic gradient of $i \approx 200$. The costs of a rigid concrete twin phase wall are considerably higher than for a single phase wall. Together with the necessary reinforcement and using a grab excavator, the price for one square meter of sealing surface amounts to about 100.00 to 150.00 US $. Possible mix proportions for one cubic meter may be as follows:

- Aggregate (0 mm to 25 mm): 400 – 1300 kg (out of which fraction < 2 mm = 180 - 250 kg with possible addition of rock flour or filter ash)
- Cement: 150 – 450 kg
- Water: 100 – 350 kg

3.3.5 Mix proportions for diaphragm walls of plastic concrete and plastic mortar

3.3.5.1 General

The use of conventional (rigid) concrete for cutoff walls may be problematic because of its inherent brittleness, which may lead to cracks in the wall, for example under cyclic loading, caused by fluctuating reservoir levels or seismic tremors. Cracks can develop into sources of leakage decreasing the efficiency of the cutoff. Rigid concrete cutoffs are much stiffer than the surrounding materials and in compressible foundations the soil may slip along the wall and transfer load through negative skin friction. If the wall has stiffness similar to the adjacent soil, stress-strain characteristics will be compatible and the wall will deform together with the soil without acquiring significant additional stresses.

Diaphragm walls made of plastic concrete are designed as classical twin-phase walls. Plastic mortar differs from plastic concrete with respect to the grain size distribution of the aggregate. The aggregate in case of plastic mortar is comprised of fine to medium-grained sand whereas with plastic concrete it is composed of gravel with maximum grain sizes of up to 30 mm. Plastic mortar is even more deformable and hence more ductile than plastic concrete. The uniaxial strength of plastic concrete is in the range of 0.6 – 5 MPa. The first plastic concrete diaphragm wall, 20 m deep and 1.2 m thick, was constructed in 1959, below the Santa Lucia dam in Italy (Dupeuble & Habib, 1969). Starting out from a conventional concrete mix, clay was added and the cement content reduced to 100 kg/m^3.

The slot for the plastic concrete panel is excavated under support of bentonite slurry and the plastic concrete mix is subsequently introduced into the trench. In the case of small trench depths, the filling process may be executed from the ground surface without auxiliary equipment. With deep excavations filling of plastic concrete must be made by tremie pipe otherwise there is a risk of segregation. In general, the same basic rules apply as given for the rigid concrete wall (Section 3.3.4).

The production costs for plastic concrete or plastic mortar diaphragm walls are in the range of US$ 50.00 to US$ 100.00 per square meter of sealing surface using a deep hydraulic grab excavator for excavation. If no laboratory tests are available yet possible mix proportions for one cubic meter may look as follows:

	Plastic concrete	Plastic mortar
Aggregates	1400 – 1700	500 -1000 kg
Cement (w/c = 1.6-2.0)	100 – 150 kg	75-290 kg
Bentonite (in suspension)	400 – 500 kg	375- 750 kg
Bentonite fraction in suspension	2 – 12 % by weight	

3.3.5.2 Design requirements for plastic concrete

ICOLD Bulletin 51 (ICOLD 1985) lists the following characteristics to be satisfied by a plastic concrete used for a cutoff wall:

- Permeability of the wall should not be higher than 10^{-9} to 10^{-10} m/s
- To achieve sufficient deformability and strain compatibility of the wall with the surrounding material, Young's modulus of the plastic concrete should be 4 to 5 times higher than that of the surrounding soil. In addition, the plastic concrete should have a high failure strain, i.e. $\varepsilon_f > 1\%$. The failure (or ultimate) strain is defined as the strain at the peak deviator stress.
- The compressive strength of the plastic concrete shall be as low as possible, but high enough to support the weight of the diaphragm wall, to sustain the earth pressure at depth and to resist erosion and hydraulic fracturing. It shall also resist chemical attack.
- The plastic concrete shall meet the workability needed for tremie placement. This implies that the plastic concrete has sufficient flowability when placed into the trench to avoid clogging of the tremie pipe and to provide for uniform distribution along the bottom of the trench. To achieve these objectives, a slump of 20 ± 2 cm is usually specified.

3.3.5.3 Design of plastic concrete mixes

Design of plastic concrete mixes in the past has been based mainly on trial and error. A viable design procedure was developed by the US Army Corps of Engineers (USACE) in the early 1990s (Kahl et al. 1991). The basic philosophy was to correlate results of simple tests with well-defined material and testing parameters. A comprehensive testing program was carried out involving unconfined compression tests, isotropically consolidated, undrained (CIU) triaxial tests, splitting tensile (Brazilian) tests, and permeability tests. The triaxial tests were conducted mainly to investigate the influence of consolidation and horizontal confinement on the stress-strain-strength behavior and the permeability of the plastic concrete. Plastic concrete was produced in the laboratory according to ASTM C192 "Standard Practice for Making and Curing Concrete Test Specimens in the laboratory"

Consolidation of the plastic concrete after having been placed into the excavated slot, will occur under the vertical stress imposed by the weight of the overlying concrete (some of this stress may be taken up by arching if the sides of the excavation move laterally). Due to the low permeability of the plastic concrete very little migration of water will take place within the cutoff wall after consolidation.

Garand et al (2006a) carried out a CIU and CID (consolidated drained) triaxial tests on plastic concrete (sample size 100 mm in diameter and 200 mm long with maximum particle size of aggregate of 20 mm) and found that 90 % consolidation was reached after 3 to 19 days depending on the mix tested. The coefficient of consolidation was around 10^{-6} m^2/s. This relatively high value is due to the high value of the bulk modulus of the concrete skeleton (which is quite different from a plastic impervious soil, such as stiff clay).

Plastic concrete consists essentially of cement, bentonite, aggregate and water. In order to quantify these material parameters to develop design data from tests on different mixes, Kahl et al. (1991) defined the following batch parameters:

- Cement factor (CF): This is the total amount by weight (in pounds or kg) of cement and bentonite in a unit volume (cu yard or m^3) of plastic concrete, i.e.

$$CF = (W_{cement} + W_{bentonite})\ (kg/m^3)$$

- Bentonite content (BC): This is the percentage of cement factor which is bentonite, i.e.:

$$BC = [W_{bentonite}\ (kg/m^3)/CF\ (kg/m^3)]\ x100\ (\%)$$

- Water-cement ratio (w/c): This is the ratio of the weight of water to the total weight of cement + bentonite, i.e.

$$w/c = W_{water}/(W_{cement} + W_{bentonite})$$

- Coarse to fine ratio: Ratio of coarse aggregate to fine aggregate (by weight)

According to ASTM C-125, *coarse aggregate* is defined as that portion of an aggregate retained on the 4.75 mm (No. 4) sieve. This corresponds essentially to the gravel fraction. *Fine aggregate* is defined as that portion of an aggregate passing the 4.75 mm sieve and being retained on the 75 μm (no. 200) sieve. This corresponds to the sand fraction.

With the above parameters, the procedure to design a plastic concrete mix for testing in the laboratory proceeds as follows:

(1) For a given cement factor and bentonite content, estimate the water-cement ratio that would give a slump of 20 cm
(2) Calculate the actual weights and volumes of cement, bentonite and water (in kg and m^3) required to produce 1 m^3 of plastic concrete
(3) Calculate the required volume of fine and coarse aggregate as the difference between one cubic meter and the sum of the volumes of cement, bentonite and water.
(4) Back-calculate the weights of fine and coarse aggregate from their volumes

(5) Correct the weights of all constituents for hygroscopic moisture content and scale the quantities to produce the desired batch volume
(6) After completion of batching, correct the cement factor and the water-cement ratio for any additional water added during mixing.

The aggregate ratio (fine aggregate to coarse aggregate) is often taken as 1.0 for tremie concrete.

The wet plastic concrete is tested for:

- Slump
- Unit weight
- Air content
- Water content
- pH-value
- Temperature

Testing follows accepted and widely used international standards.

In the design of a plastic concrete mix for a particular cutoff wall, strength and stiffness of the wall are of primary interest. In the investigation carried out by Kahl et al (1991), values of unconfined compressive strength (obtained from 150 mm diameter and 300 mm high specimens) were correlated with the water-cement ratio and the elastic modulus, with bentonite content and curing time as parameters. These relationships are plotted as graphs. All data are based on a slump of 20 cm.

Erreur ! Source du renvoi introuvable.18 shows the relationship between cement factor and water-cement ratio with the bentonite content as parameter, i.e. BC = 0, 10, 20, 40 and 60 %. It can be seen that for a given BC a decrease in CF requires an increase in w/c to maintain a slump of 20 cm.

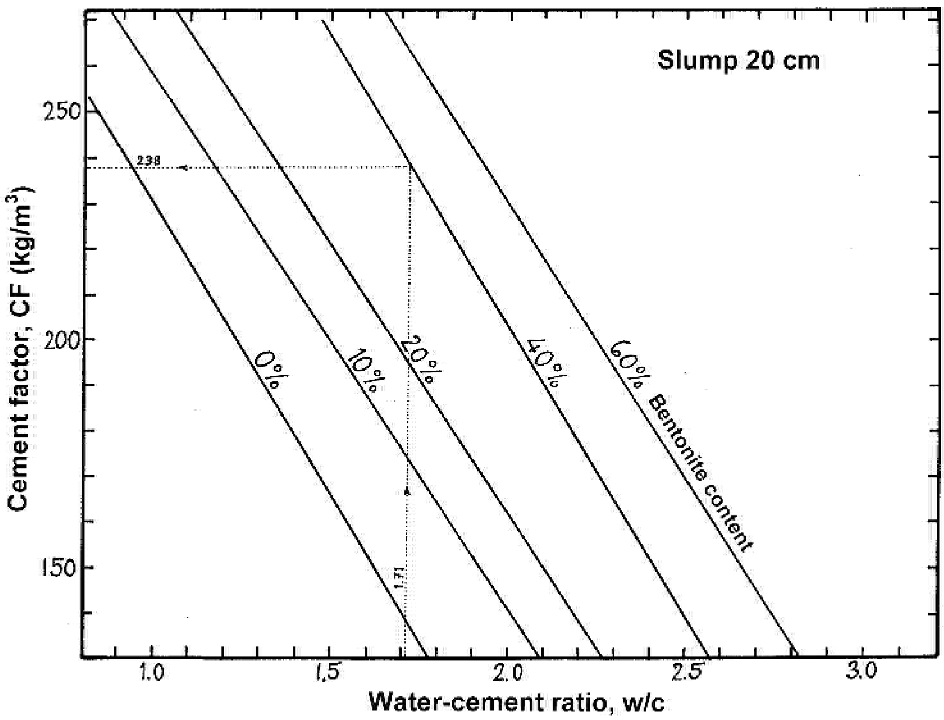


Fig. 3.18 Relationship between cement factor and water-cement ratio for bentonite contents of 0, 10, 20, 40 and 60 % (Kahl et al., 1991)

Fig. 3.19 presents the relationship between unconfined compressive strength and the water-cement ratio for different curing ages and bentonite contents.

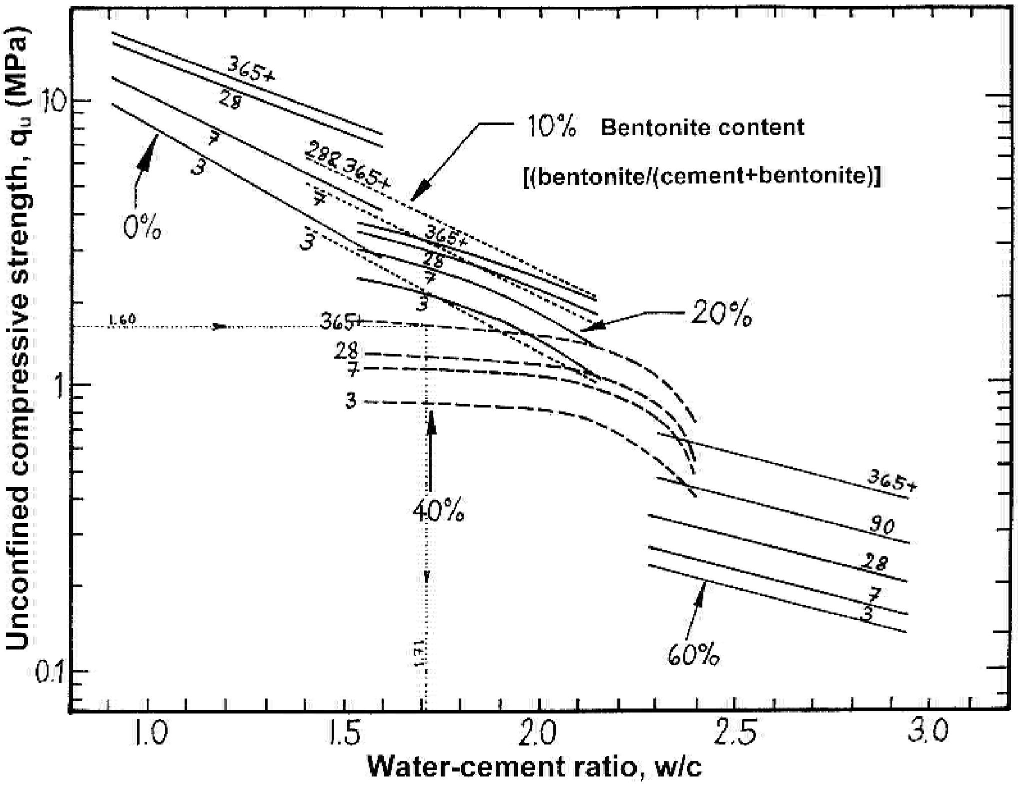


Fig. 3.19 Unconfined compressive strength versus water-cement ratio for all bentonite contents with lines being isochrones of curing age (Kahl et al., 1991).

Finally, Fig. 3.20 gives the relationship between unconfined compressive strength and elastic (or Young's) modulus for all curing ages and bentonite contents. The plot shows two regression lines because the elastic moduli were calculated from two different methods of strain measurements, namely by gross deflection and by compressometer (see ASTM C465). The second method of measurement seems to be more accurate, but scatter of the data is larger than the differences obtained in the strain values. The graph clearly demonstrates that the elastic modulus increases with the unconfined compressive strength.

To make use of these graphs, the following example is given: A designer requires for the cutoff wall a certain compressive strength and elastic modulus, e.g. based on calculations and in situ testing of the foundation soil. Assume E = 1400 MPa and obtain from Fig. 3.20 an unconfined compressive strength, q_u of 1.60 MPa. Then enter Fig. 3.19 where there are several options depending on the time of curing, e.g. 3 days and 10 % bentonite content, or 7 days curing time with 20 % bentonite content, or 365+ days with 40 % bentonite content. Since the criterion is long-term strength, a mixture with a bentonite content of 40 % is chosen. The corresponding water-cement ratio is then 1.71. From Fig. 3.18, a cement factor of 238 kg/m^3 is then obtained for a water-cement ratio of 1.71 and a bentonite content of 40 %. With these data the designer has now all the information needed to proportion the mixture.

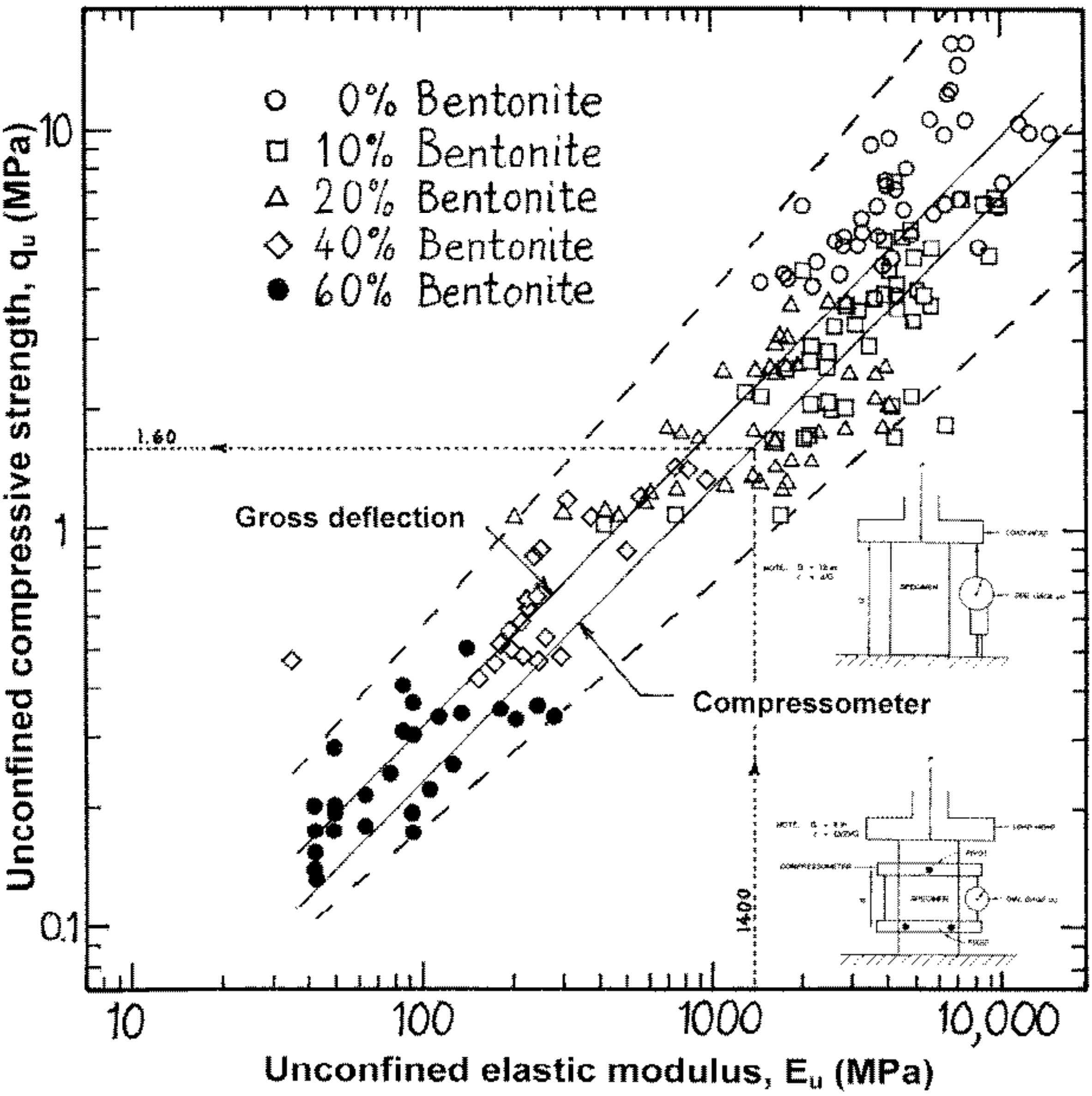


Fig. 3.20 Relationship between unconfined compressive strength and elastic (Young's) modulus for all curing ages and bentonite contents (Kahl et al., 1991)

Analysis of unconfined compression test results obtained with plastic concrete (Kahl et al, 1991; Garand et al, 2006a) revealed the effects of the mix proportions (expressed by cement factor and bentonite content) and the curing age on the unconfined compressive strength and the water-cement ratio. These are summarized qualitatively in Table 3.11.

Table 3.11 Effects of CF, BC and curing age on unconfined compressive strength, failure strain and w/c ratio

Increasing variable	Effect of increase on				
	Unconfined compressive strength, q_u	Elastic modulus, E	Strain at failure, ε_f	Water-cement ratio, w/c	Air content of wet plastic concrete
Cement factor, CF	↑	↑	↓	↓	-
Bentonite content, BC	↓	↓	↑	↑	↓
Curing age	↑	↑	↓	-	-

Note: ↑ increasing ↓ decreasing

3.1.1.1 Influence of confinement and consolidation

In the cutoff trench the plastic concrete, after having been placed by tremie pipe, is laterally confined and subject to overburden stress caused by the overlying concrete and later by the embankment load. Under this stress field the plastic concrete will consolidate. The triaxial test enables a more realistic modeling of the stress path the plastic concrete will undergo in the trench, especially at greater depth. In order to investigate the role of confinement and consolidation on the stress-strain-strength relationships and the permeability of plastic concrete, Kahl et al (1991) performed CIU triaxial tests with bentonite contents of 0, 20 and 40 % and curing ages of 3, 7, and 14 days. Garand et al (2006a) tested various mixes of plastic concrete under both undrained and drained conditions (CIU and CID triaxial tests). The main purpose of their test program was to establish stress-strain relationships for use in finite element modeling of the cutoff wall. The consolidation pressures varied from 100 to 800 kPa and failure strains were typically in the range of about 4 to 9 %. Poisson's ratio obtained from CID tests was 0.12. A linear relationship between elastic modulus and deviator stress could be established of the form:

$$E_e/(\sigma_1-\sigma_3)_e = 190$$

where the modulus, E_e is the secant modulus to the yield point (end of the quasi-elastic range) of the stress-strain curve.

The triaxial test results indicated that shear strength, elastic modulus and strain at failure all increase with consolidation pressure and confinement. The failure strain in CIU tests increased greatly with the addition of bentonite, but the elastic modulus decreased. This means that the use of plastic concrete significantly increases the ductility of a diaphragm cutoff wall, particularly at greater depth.

The effects of mix proportions, curing age and confinement/consolidation on the stress-strain-strength behavior are qualitatively summarized in Table 3.12.

Table 3.12 Summary of stress-strain-strength behavior of plastic concrete

Increasing variable	Effect of increase on			
	Shear strength (Deviator stress at failure) $(\sigma_1-\sigma_3)_f$	Elastic modulus, E	Strain at failure ε_f	Permeability, k
Bentonite content, BC	↓	↓	↑↑	↔
Curing age	↑	↑	↓	↓
Confinement, σ_3	↑	↑	↑	-
Consolidation, σ_c	↑↑	↑	↑	↓

Note: ↑ increasing ↓ decreasing ↔ more or less unchanged

Triaxial testing of plastic concrete is essential if the cutoff is modeled in a numerical analysis to understand its behavior under undrained and drained conditions. Garand et al (2006a) found that the undrained conditions can be modeled by an elastic-plastic Mohr-Coulomb model, while the drained behavior fits well the well-known hyperbolic model of Duncan-Chang (Duncan & Chang, 1970).

3.1.1.2 Permeability

Permeability, expressed by the coefficient of hydraulic conductivity, k, has been measured in triaxial tests. It depends mainly on four factors, namely: the bentonite content, the water-cement ratio, the consolidation stress, and the curing age. Consolidation and curing age decrease the permeability. The k-values measured in the CIU and CID tests were in the range of 10^{-9} to 10^{-10} m/s.

The bentonite content, however, does not appear to have much influence on the permeability. This is because of opposing effects of bentonite content and water-cement ratio. It can therefore be inferred that a plastic concrete cutoff is just as effective against seepage as a cutoff made of conventional (rigid) concrete.

3.1.1.3 Erosion resistance

There has been some reluctance to use plastic concrete instead of rigid concrete for reasons of durability and erosion resistance in case of cracks, holes or other singularities in the cutoff wall. There are, however, relatively few published investigations regarding this problem. Garand et al (2006b) carried out a fairly comprehensive experimental program to study the effect of severe conditions of water flow on openings in plastic concrete, mortar and grout. The program included three kinds of erosion tests and a finite element modeling of water flow trough an open crack. The three erosion test series were:

- High pressure water jet (HPWJ) tests
- Modified pinhole (MPH) tests
- Controlled water velocity (CWV) tests

The *HPWJ tests* were carried out on block-shaped samples (300x300x75 mm) of four different mixes, which represented grout, mortar, and concrete, all using bentonite. The samples were exposed to a jetting pressure of 120 bars in different directions, also with the nozzle and the sample submerged. The purpose of this test series was to find out whether the plastic concrete is more resilient to erosion than plastic mortar or plastic grout of the same mix proportions of water, cement and bentonite. If this were the case, small-scale erosion tests (e.g. pinhole test) on grout and mortar mixes would be representative of the erosion resistance of plastic concrete.

The results showed that the concrete specimens were significantly more resistant to high energy water jets than mortar or grout.

MPH tests were carried out with an apparatus designed on the basis of a similar device developed by the US Department of Agriculture. Samples of plastic mortar were made of cement, bentonite, sand and water. Using eight different mixes and flow gradients of 60, 100, and 300, the samples were exposed to the fixed water head for a duration of 60 minutes. The results showed that the erosion rate was initially high, then stabilized at a lower value. The erosion rate was dependent mainly on the gradient rather than on the mix proportions. The rate of increase was more rapid when the gradient exceeded 100.

The HPWJ and MPH tests must be considered as qualitative tests as they can only express trends. The measured parameters cannot be applied to a full-scale project with plastic concrete.

The *flow through an open crack* was analyzed for a semi-infinite, vertical cutoff in a permeable foundation. The wall had a horizontal empty crack with openings of 15, 30, 60 and 120 mm. The results showed that for a given permeability of the soil surrounding the crack, the velocity of the water flowing through the crack increases as the width of the crack diminishes. This can also be seen from Fig. 3.21, which shows the ratio of gross gradient over water velocity (in m/s) in the crack plotted versus the permeability of the soil surrounding the crack.

The *CWV test* was developed for larger size specimens of plastic concrete, which had particle sizes of up to 20 mm. The test exposes a surface of 0.09 m^2 inside a crack to a controlled velocity water flow. The test set-up is shown in Fig. 3.22. The test specimen, 150 mm in diameter and 300 mm long, consists of a cylindrical body which has been split into two halves. The two halves are then placed inside a cylindrical casing and grouted to fix their position. Aluminum spacers glued on both sides of the mix create a rectangular opening for the water to flow through. The spacing between the two halves is kept at 30 mm such that the largest particle (20 mm), if it has been torn out of the concrete mass, can pass through the crack and be washed away. Guide vanes are copper tubes placed inside a feeding pipe to ensure laminar flow.

Samples tested had mix proportions of BC 20, 30 and 40 % and w/c of 1.8 to 2.2. Initially samples were exposed to a water velocity of 14.5 m/s, but in order to preclude cavitation at the downstream end of the specimen, a back pressure had to be applied which reduced the maximum flow velocity to 12 m/s. The velocity of the water flow was increased in steps with total test duration of 160 hours. Erosion was measured by determining the volume of the cavity created by the flowing water.

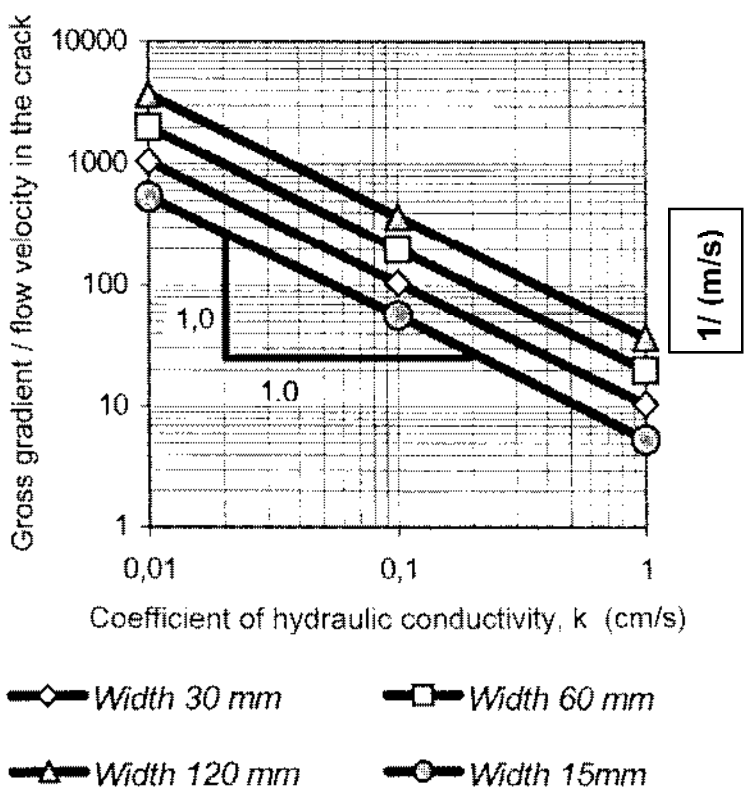
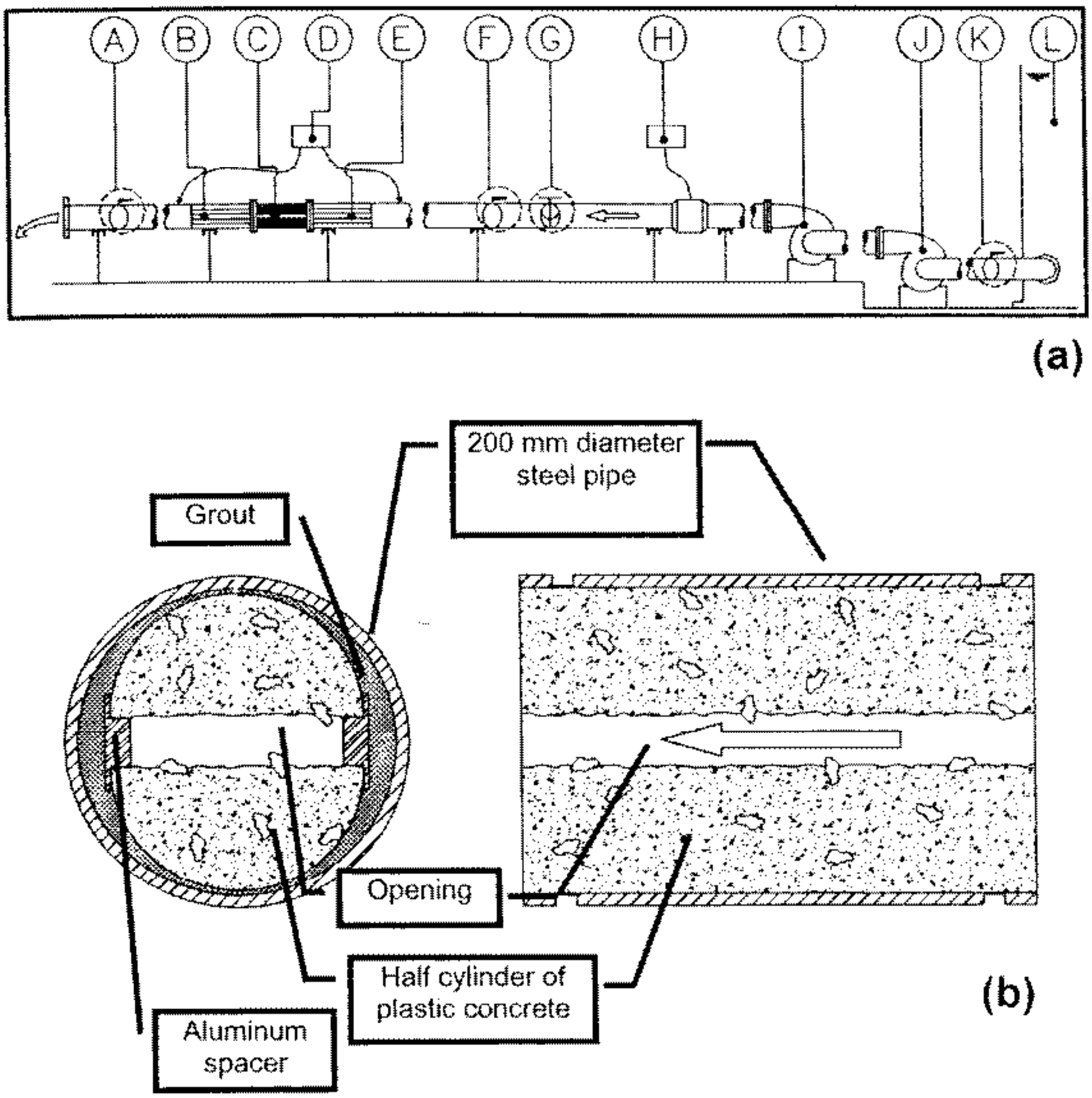


Fig. 3.21 Flow through an empty crack in a semi-infinite cutoff wall: The gross gradient is the difference in water head between the reservoir water level and the tailwater level (Garand et al, 2006b)

Results showed that the plastic concrete can easily withstand high velocity water flow. The erosive process stabilized after about 6 hours in all velocity steps. The material removed seemed to originate mainly from the cement-bentonite binder. Surprisingly, the weight loss decreased with increasing bentonite content, BC. Garand et al (2006b) explained this with the demand for water of the bentonite when it is hydrated in the mixing process along with the cement. There would then be less water left for cement hydration and the resulting binder would then be harder but not necessarily stronger.

The flow velocity through a crack decreases when the crack widens, as demonstrated in Fig. 3.21. In a the very pervious zones of an alluvial foundation soil, the water velocity may be around 10^{-2} m/s. From Fig. 3.20, the water velocity for a gross head of, say 60 m, would be about12 m/s and 1.75 m/s for crack widths of 15 mm and 120 mm, respectively. Hence, the width of the crack subjected to erosive water flow, is expected to stabilize once it has widened to some extent. It can also be assumed that eventually the crack would be plugged by particle migration from the surrounding soil mass.

Two erosion tests were also carried out by USACE by observing the erosion caused by water flow through a 4.76 mm hole at a velocity of 5.2 m/s. The mix proportion of the sample were BC = 0% and 60 %, CF = 178 kg/m^3 and curing time 3 and 8 days. None of the specimens showed susceptibility to erosion or piping.



A	Exit valve	E	Guide vanes	I	Pump #2
B	Guide vanes	F	Shut off vane	J	Pump #1
C	Specimen	G	Control valve	K	Shut off valve
D	Differential manometer	H	Magnetic flow meter	L	Constant head tank

Fig. 3.22 Controlled water velocity (CWV) test: (a) test set up, (b) longitudinal and cross section through sample and container.

A somewhat different approach to erosion stability was taken by Beier & Strobl (1985) in that the critical hydraulic gradient was based on the results of leaching tests. Important variables for erosion tests were:

- Properties of the water flowing through the sample (pH, chemistry)
- Hydraulic gradient
- Duration of loading (or leaching)
- Characteristics of cutoff wall material (mix proportions, permeability, unconfined compressive strength)

According to their experience erosion failure has to be expected when more than 30 % of the calcium has been removed by leaching. They defined a leaching factor, A, as the ratio of eroded calcium to the calcium in the sample prior to leaching.

$$A = c\, i^2$$

where i is the hydraulic gradient and c is the erosion constant. The latter can be obtained from long-term seepage tests of at least 26 weeks duration.

The limit gradient, i_{lim}, can be obtained by setting A = 0.3 and $i_{\lim} = \sqrt{\frac{A}{c}}$

Leaching removes part of the calcium-silica hydrate (CSH) which is crucial for the strength of hardened cement. This process promotes the reduction of the remaining CSH to SiO_2 and $Ca(OH)_2$ to maintain chemical equilibrium. If the amount of CSH exceeds a certain value, the stability of the CSH is lost and the erosion process starts and continues until concrete has been destroyed.

Beier & Strobl (1985) tested three cutoff wall materials used in dams in northern Bavaria, Germany, namely:

- Slurry used for vib wall construction. consisting of bentonite, blast furnace cement, filler (rock flour) and water
- Slurry used for a one-phase diaphragm wall consisting of bentonite, blast furnace cement, filler (electro filter ash) and water
- Plastic concrete for a two-phase diaphragm wall consisting of bentonite, blast furnace cement, filler (clay, rock flour, sand, gravel), and water

However, the proportions of these mixes were quite different from those used by UASCE or Garand et al (2000b). For the plastic concrete the following values apply: BC = 17 % and w/c = 3.3. Based on leaching tests lasting for up to 45 weeks with hydraulic gradients in the range of 40 to 230, the following limiting gradients were established:

for vib wall slurry: $i_{lim} = 200$
for one-phase wall: $i_{lim} = 100$
for two-phase wall: $i_{lim} = 200$

For design of the cutoff wall a safety factor of 2.0 is recommended, i.e.

$$i_{allowable} = 0.5\, i_{lim}$$

The permeability of the finished diaphragm wall is also a significant factor in the erosion resistance. If the k-value of the cutoff wall is clearly greater than 10^{-8} m/s, the risk of erosion increases abruptly. The erosion resistance of the vib wall and the one-phase wall can be enhanced by higher cement content, but at the cost of lower deformability.

Summarizing, it can be stated that plastic concrete is essentially erosion resistant for flow velocities through a crack of up to about 15 m/s and for gradients of at least 150.

3.2 SUITABLE MIXING EQUIPMENT FOR PREPARATION OF SLURRIES

Bentonite is a major constituent in any of the diaphragm material mixes except for rigid walls although its proportion is small. It reduces bleeding, increases the yield stress of the slurry and acts as a lubricant to reduce pump wear. In the set state of the wall it has a self-healing function and can stop minor leaks caused by cracks.

In order to be efficient the bentonite must be fully hydrated and evenly distributed in the slurry. This has to be accomplished by mixers. Simple mixers can only distribute the bentonite, but to accelerate full hydration, sufficiently high shear stresses must be applied to the clay particles. This can be accomplished by means of high speed mixers. The penetration of water molecules between the silica sheets of the clay particles can then take place within a relatively short time. Mixers capable of generating such shear stresses are referred to as colloidal mixers.

The literature quotes frequently rotational speeds of between 1500 and 2000 rpm for mixers able to produce a deflocculated colloidal mixture. Manufacturers of mixing devices often specify the speed of the mixing vanes, which should be about 15 m/s. Besides this, it is also possible using on-site mixers to try to attain the values of Marsh funnel viscosity, which is a measure of the stability obtained in suitability tests in the laboratory, and hence determine the required mixing time to produce a colloidal deflocculated mixture using the available mixing equipment.

Two different types of mixers are available on the market. For a slurry demand of up to 5 m^3/h mixing, the so-called charge mixers is practicable, whereas for demands higher than 5 m^3/h it is more appropriate to employ through-flow mixers which can have capacities of up to 60 m^3/h.

In order to accommodate fluctuations in the demand of suspension, a buffer container is installed between the suspension pump and the mixer. This buffer container is equipped with an agitator, which has to ensure that properties of the slurry remain constant whilst in the buffer container. The residence time of the slurry in the buffer container should be limited. Fig. 3.23 illustrates schematically a mixing plant for cement-bentonite slurry.

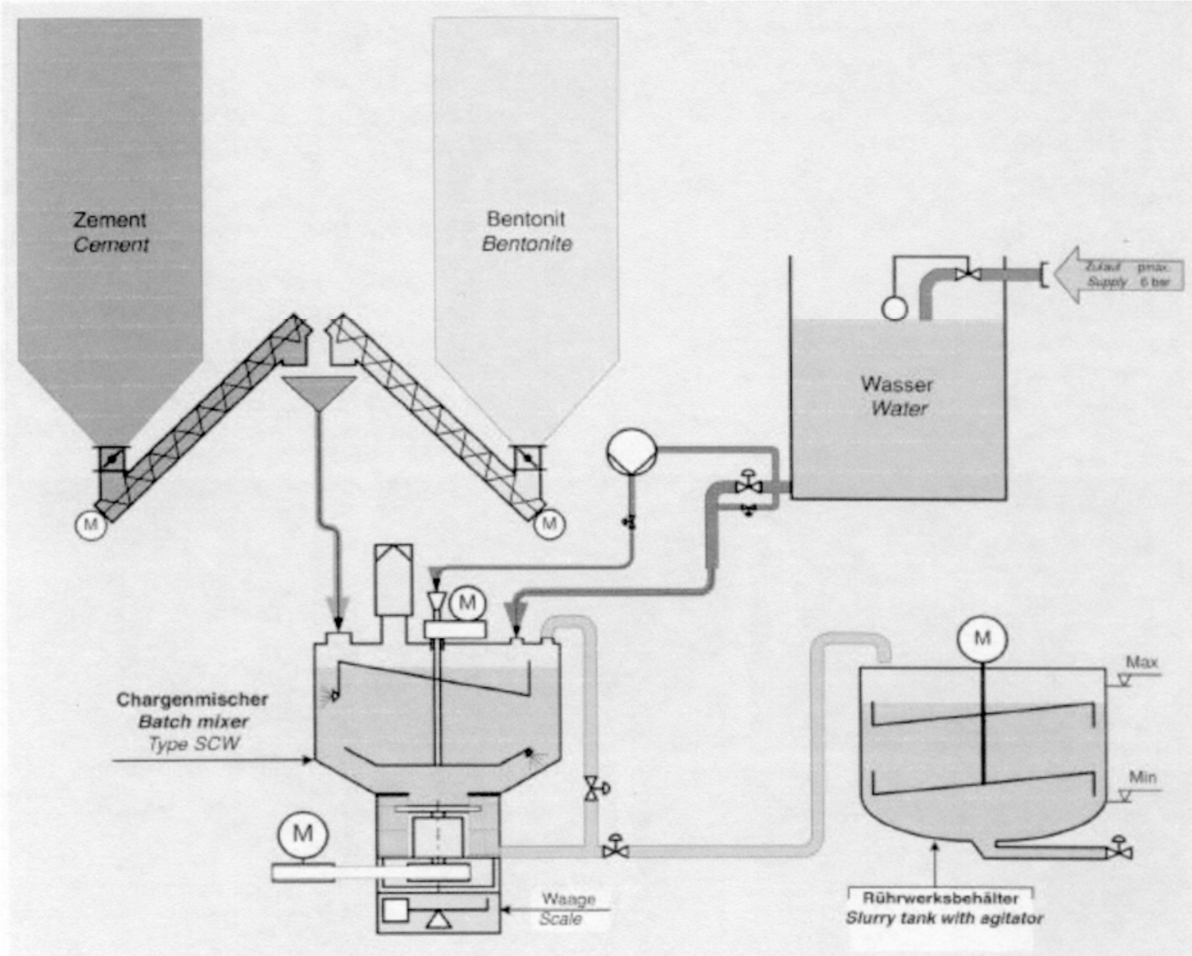


Fig. 3.23 Mixing plant for cement-bentonite slurry showing the flow of materials

3.8 LONG-TERM PERFORMANCE OF DIAPHRAGM WALLS

3.8.1 Factors influencing the long-term performance

The purpose of constructing a diaphragm wall is not only to decrease seepage through a dam foubndation, but also to enhance the safety of the dam with respect to adverse uplift water pressures.. However, it is important to recognize that under certain subsurface conditions the installation of a seepage barrier can create additional potential failure mechanisms. This is because the construction of the wall leads to a build up of hydraulic pressures and increased hydraulic gradients. These changes in the hydraulic pressure regime must be accounted for when designing a cutoff wall, or alternatively assessing existing diaphragm walls or monitoring dams with barriers.

Figur 3.24 illustrates several of these mechanisms. The differential water pressure generated by the installation of the diaphragm wall will produce increased hydraulic gradients at several locations, e.g. around the wall boundaries, at possible defects (e.g.cracks) in the wall, and also through structures in the rock foundation (joints, solution voids). Furthermore, concentrated seepage forces will cause a deflection of the wall.

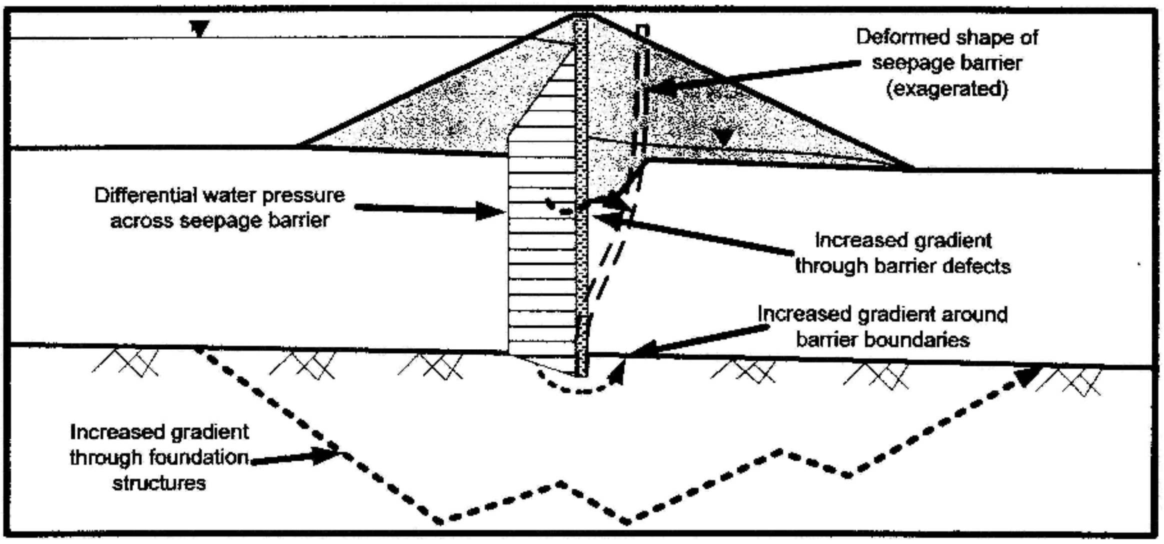


Fig. 3.24 Development of differential water pressures and their effect on diaphragm wall leading to wall deflections and increased hydraulic gradients (Rice, 2009)

Records on the long-term performance of cutoffs are not readily available in the literature usually accessible to dam engineers, with some exceptions, such as the performance of the Manicouagan-3 cutoff in Canada (Dascal et al, 1991). Rice (2007, 2009) and Rice & Duncan (2007) have collected information on the performance of seepage barriers of 30 dams in the Unites States, which had been in place for over 10 years, covering all types of diaphragm walls, i.e. from soil-bentonite walls to rigid concrete walls. The majority of the information was obtained from reports of the US Army Corps of Engineers and the US Bureau of Reclamation. The following text is largely based on the findings from this survey.

3.8.2. Influence of foundation conditions

Three groups of foundation conditions were distinguished:

- Dams on karstic limestone foundations
- Dams on jointed bedrock foundations
- Dams on soil (overburden) foundations

Dams on foundations containing solutioned limestone are most problematic. They exhibited a higher frequency of post-construction problems per dam than any other group of dams. With a cutoff through overburden and partially penetrating karstic limestone, the resistance to seepage is usually much higher in the overburden than in the solutioned limestone which contains voids and interconnected channels. The seepage volume is then controlled by the overburden and the number and characteristics of points where the water can enter and exit the karst. Seepage will flow mainly through the network of solution channels which are not blocked by the wall. This results in increased flow velocities which can cause erosion of the solution voids.

With jointed bedrock foundations, increased flow through the rock below the cutoff tip can develop if (1) the joints contain erodible infills which can be washed out under an increased hydraulic gradient, or the rock itself is eroded from inside the joint, and (2) seepage flow is maintained over long distances with little head loss due to persisting open joints. The hydraulic gradients at the exit points of these flow downstream of the dam can be high and they may have the potential to initiate piping erosion.

With dams on soil foundations (e.g. a pervious stratum overlying a clay layer) problems may arise with imperfections (e.g. cracks) in the wall and internally unstable soils which are not protected by adjacent filters or rock. Unstable soils are susceptible to suffusion.

3.8.3 Post-construction leaks in diaphragm walls

Observations on existing diaphragm walls have shown direct evidence of cracks or leaks in the wall. Usually small leaks are not readily detected unless there is a piezometer in the immediate vicinity of the leak. Crack development was found to occur more frequently with rigid concrete walls. Finite element analysis revealed that deformations caused by concentrated seepage forces resulting from the differential water pressure acting across the wall are often of sufficient magnitude to cause cracking. The most comprehensively documented example of wall cracking is the rigid concrete wall of Navajo dam (USA) where core drilling and sonic testing were performed to map the locations of the cracks in the concrete wall (Dewey, 1988; Davidson, L., 1990).

According to FE calculations by Rice & Duncan (2007) assessing the effects of cracks in a diaphragm wall, it was demonstrated that cracks with small aperture (<1mm) can locally increase the hydraulic gradient of the wall by several orders of magnitude, but as the crack widens due to erosion the hydraulic conductivity decreased and the flow stabilized. However, there may be special situations where a crack may lead to significant erosion. In general, there are three factors which can promote the widening of a crack or a defect, namely: (1) the aperture of the cack or the size of the crack (2) the hydraulic conductivity of the surrounding soil, and (3) the erosion susceptibility of the seepage wall material.

The designer of diaphragm walls must be aware of such possible scenarios to ensure confidence in a satisfactory and safe long-term performance of the wall.

Case histories relating diaphragm wall applications are discussed in Appendix 1.

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4. VIB WALLS

4.1 PURPOSE AND USE OF VIB WALLS

Among the positive cutoffs, vib walls represent the most economical sealing element for hydraulic structures. Vib walls are preferably used as impervious barriers for dykes, low dams, and around landfills. The commonly maximum attainable depth of vib walls is about 25 m, however, recently constructed vib walls have reached depths of up to 30 m. Vib walls are also known under the name of “thin walls”, “vibrated beam slurry walls”, “narrow walls” and “paroi mince” (in French). A new method of vib wall construction is the Vibrosol technique, which offers great advantages especially for the sealing of post-heightened earthfill dams.

4.2 PRINCIPLE OF VIB WALL CONSTRUCTION

The construction process consists of vibrating a wide flange steel I-beam (web height 500 mm to 1000 mm, enhanced web thickness of 60 to 80 mm, and flange width 300 to 500 mm (or a specially designed beam) from the surface into the loose ground by a dredge (200 kW and 40 tons). Vibration frequency is about 30 Hz and the amplitude about 10 mm). During beam penetration (and later extraction), premixed slurry is injected through nozzles affixed to the bottom of the beam (pressure approx. 1 MPa). The beam generally penetrates vertically, but may be inserted at a batter to contain small streams and canals. After the beam has attained its required depth, it is extracted at a controlled rate to fill the void left by the beam extraction, creating an in-ground panel of slurry wall with an approximate wall thickness of 10 cm. This process is repeated along the wall alignment, with each beam insertion overlapping the previously inserted panel. In this way a continuous wall is created without the need for excavation (Fig. Fig. **4.1** and Fig. 4.2.)

The steel beam with the wide flange is provided with an approximately 350 mm wide fin which serves as a guide for the beam to ensure a continuous wall by following the path of least resistance. Attached to the wide flange beam are slurry pipes for the injection of the slurry. The same beam is used continuously. It must be of sufficient length to penetrate the maximum expected depth and must be able to sustain the wear from repeated penetrations.

The Vibrosoil method derives from the vibrated beam technique, but enhances its efficiency considerably and also improves the continuity of the wall under better conditions (Gouvenot & Chazot, 1998). Penetration of the beam is aided by the injection of slurry under high pressure (about 35 to 40 MPa) similar to jet grouting. When the beam is withdrawn, the void is filled with cement slurry under low pressure (approx. 1 MPa). The high pressure jet at the base of the beam is oriented horizontally in the direction of the previously completed panel. In this way, a re-opening of certain zones along the slot, which may have become closed under the effect of the vibrations during the downward movement of the beam, is possible, thus eliminating the formation of windows in the completed panel.

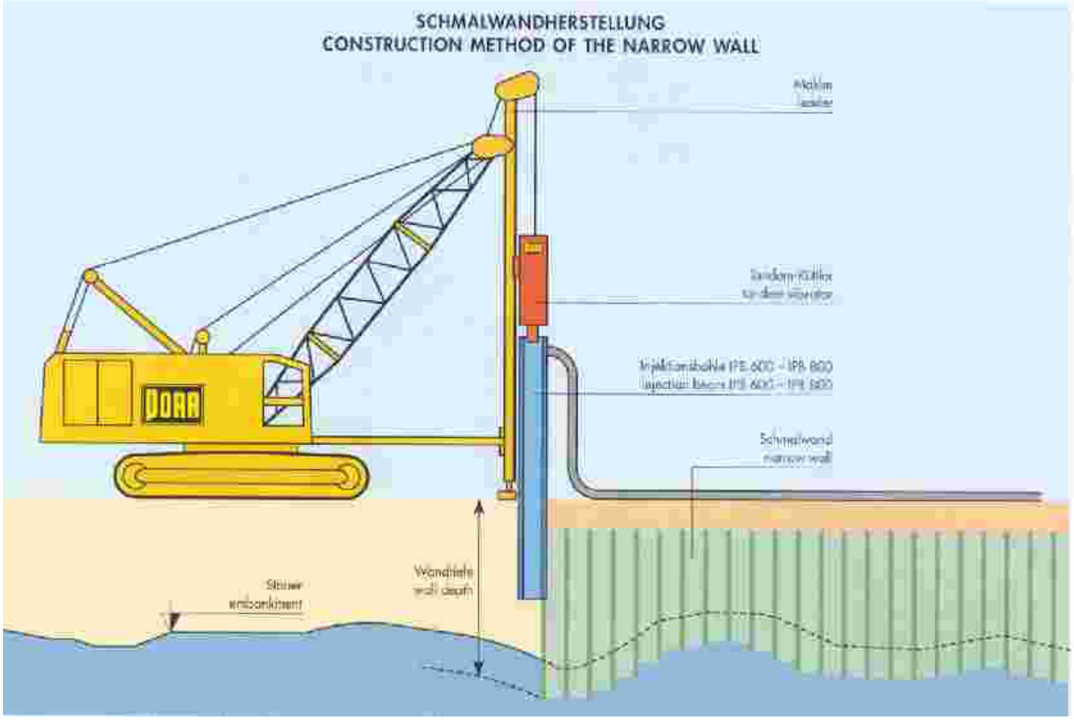

Fig. 4.1 Principle of vib wall construction

Fig. 4.2 Partly excavated Vib-wall at Lake Altmühl (Germany)

The production of vib walls often requires the use of a mobile mixing plant and pump station. The path of the slurry from the storage containers to the mixing plant and the pumping station and from there to the slurry pipes on the vibrated beam is illustrated schematically in Fig. 4.3. If the vibrosol method is used, two mixing plants are required, one for the conventional slurry injected at low pressure and another for the jetting slurry. The pump station should be able to provide a slurry flow of about 400 liters per minute with a pressure of 1 MPa. For vibrosol walls the jet grouting pump discharge should be up to 700 liters per minute with a pressure of about 40 MPa.

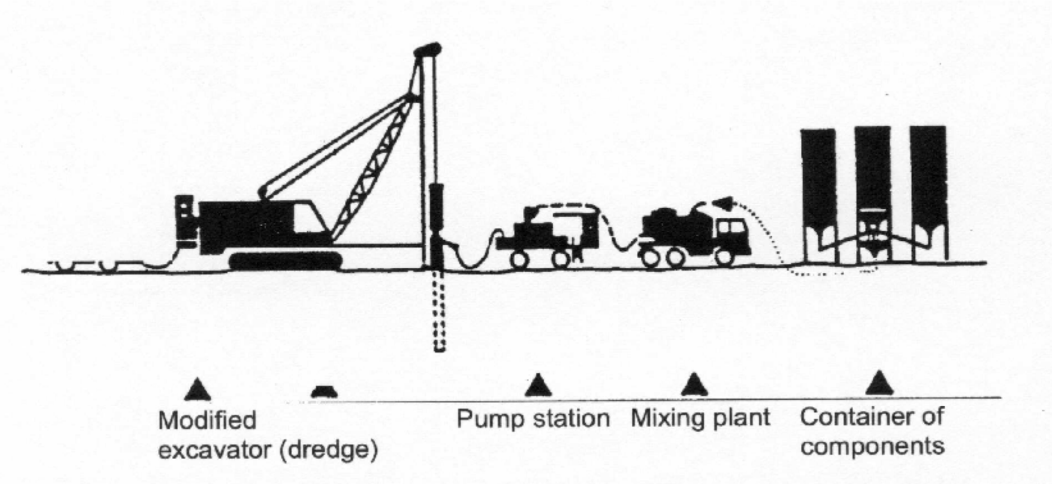

Fig. 4.3 Path of slurry from the storage container to the slurry nozzle

4.3 HISTORICAL DEVELOPMENT

In 1949, the French company "Etudes et Travaux des Fondations (ETF)" invented the forerunner to the vib wall method as it is known today. An improvement to the technique was introduced by Solétanche in 1966. Today, the vibrator is usually positioned at the top of the steel beam. However, during the years between 1970 and 1980 vibrators were also attached to the toe of the beam, a method developed by GKN Keller. But with the toe vibrator it was difficult to stabilize the position of the vib wall elements. In addition, employing the toe vibrator caused greater problems with soil liquefaction. Nowadays, the toe vibrator is no longer used.

Fig. 4.4 Shape of the lower end of a vibrated beam

The steel beam has developed from a conventional I-shape into a widened form which ensures stability against wear at the toe. The additional fin (Fig. 4.4) was developed to stabilize and follow the previously produced panel during production of the actual slot.

The vibrosol method was first applied by Solétanche on a dike construction on the river Danube in 1992. This method can be applied in sandy and silty soils where the conventional vib wall technique does not show optimum results.

4.4 REQUIRED PRE-INVESTIGATIONS

The vib wall technique is not suitable for all ground conditions and it is necessary to check certain conditions before selecting this method. The following information is needed for an assessment of the feasibility:

- Soil type
- Soil density
- Hydraulic gradients prevailing during construction
- Embedment of wall
- Groundwater characteristics

4.4.1 Soil type

The web thickness, d, of the vibrated beam depends on the texture (gradation) of the soil to be penetrated. It becomes wider in granular (pervious) soils and narrower in cohesive (impervious) soils. Fig. 4.5 shows the vib wall thickness as a function of the grading. Fine-grained soils with $D_{90} < 1$ mm are not recommended for conventional vib walls.

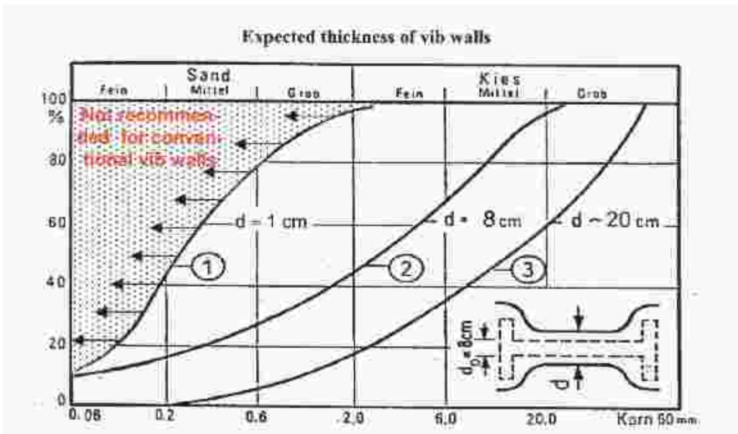

Fig. 4.5 Thickness of vib wall with corresponding grain size regions (modified from DVWK, 1990)

Sandy layers and silty soils with low elastic moduli have the tendency to squeeze the vib wall slurry out of the slot¨. Because of the possibility of liquefaction water-saturated sandy zones are not suitable for either vib wall or vibrosol wall installations.

With the vibrosol technique, however, vib wall construction in sandy soils becomes feasible because of the fin which mixes the sand with the slurry in the foregoing element.

4.4.2 Soil density

The density of a predominantly granular soil strongly influences the drivability of the vibrated beam. Very dense gravels or boulders prevent vibrating of the beam into the ground. The density and drivability should therefore be investigated prior to the choice of the vib wall as the preferred sealing element. The Super Heavy Dynamic Probe (DPSH) was found to be most suitable for this purpose. The fall weight should be 2 kN. The shape of the probe is identical to the Dynamic Heavy Probe (DPH) and shown in Fig. 4.6.

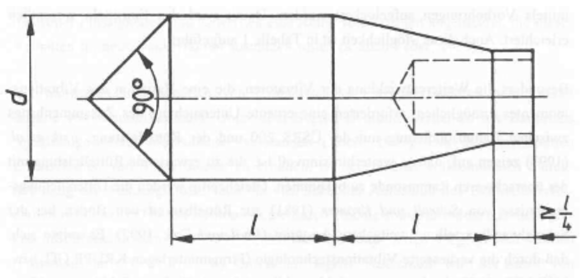

Abb. 2-4: Abmessungen der Spitze der überschweren Rammsonde (wie DPH, DIN 4094, 1990); d = 43,7 mm; ℓ ≥ d

Fig. 4.6 DPSH probe

The relationship between soil density as determined by the DPSH and the drivability are the following:

1 to 5 blows per 10 cm	easy to drive
6 to 10 blows per 10 cm	Normal to drive
11 to 14 blow per 10 cm	Low production rates with standard procedure, normal to drive with vibrosol technique
15 to 18 blows per 10 cm	Pre-drilling to loosen up the ground become necessary
> 18 blows per 10 cm	Limit of both the vib wall and the vibrosol technique

4.4.3 Hydraulic gradient during construction

Flowing water in the ground as a result of a hydraulic gradient may be troublesome as it may wash the slurry out of the slot. For a particular type of slurry the velocity of the groundwater should not exceed a certain limit. For example, with a slurry having a Marsh funnel viscosity $t_M > 50$ s, the critical groundwater velocity in coarse gravel is v = 0.1 cm/s.

When using accelerating agents in the slurry, the critical velocity increases to 2 cm/s.

Attention shall be paid to possible erosion along imperfections that may be present in the wall.

The groundwater flow along the alignment of the vib wall should be checked by exploratory drilling next to the vib wall axis. However, direct measurements of the groundwater flow velocity are rather difficult and time-consuming. Often it is sufficient to make an estimate based on gradient, permeability and effective porosity.

4.4.4 Wall embedment

Whenever possible the vib wall should be embedded into an impervious stratum. The minimum depth of embedment should be 0.5 m. The depth of such a layer should be investigated by means of exploratory drillings along the vib-wall axis. With an embedded wall erosion problems will be minimized.

4.4.5 Groundwater characteristics

The chemistry of the groundwater must be investigated prior to construction of the wall. To protect the wall from aggressive water, special measures may have to be taken, especially with the composition of the slurry used for the wall.

4.5 QUALITY CONTROL

With modern equipment, the production of beam elements can be checked by measuring a number of parameters in real time. Such production records may contain data on the following quantities (see also Fig. 4.7):

- Step distance between two adjacent elements
- Flow of slurry
- Slurry pressure
- Vibration amplitude and frequency
- Hydraulic pressure
- Depth of beam
- Rate of penetration

Solétache-Bachy developed the control unit SANPAM (Système d'Acquisition Numérique pour Parois Minces). It records and stores the following quantities: depth reached by the beam and the pressure of the vibro-hammer. For the crane operator the position of the crane, the slurry pressure and the speed of penetration are given in real time. SANPAM also registers the inclination of the beam profile penetrated and allows control of the verticality of each panel. A synthesis of all the data recorded permits to control the embedment of each panel, the panel overlap, the slurry volume in every region of the wall, and a developed view of the wall.

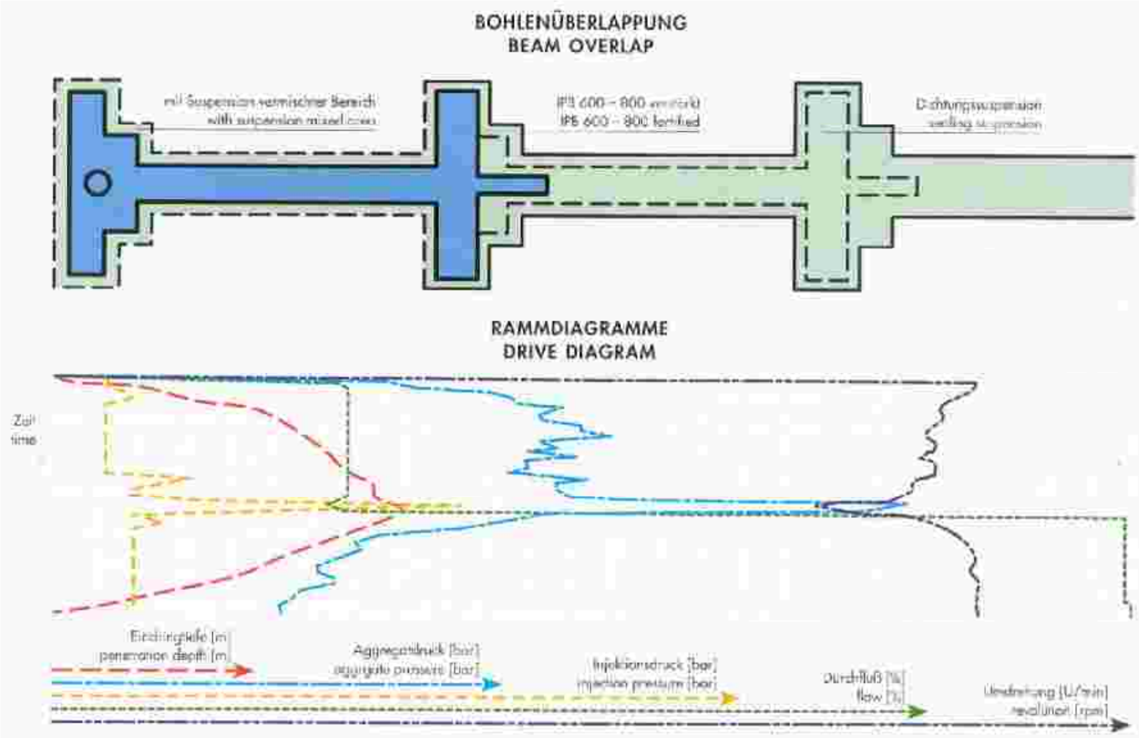

Fig. 4.7 Example of a record of parameters relevant in the production of a vibrated wall element

4.6 CHARACTERISTICS OF SLURRY MATERIALS

Slurries for vib-walls are in their composition similar to those of single phase diaphragm walls. There are, however, additional requirements that vib wall slurries have to fulfill, as explained below.

4.6.1 Base materials

4.6.1.1 Bentonite

The characteristics of bentonite for use in slurries are described in Section 3.5.2 in the chapter on diaphragm walls.

4.6.1.2 Filter ash/rock flour

Filter ash or rock flour added to the slurry increase the density. Slurries with higher densities show enhanced resistance to erosion and also offer greater stability in the vib-wall slot. It is essential that the grading curve of the rock flour or filter ash is nearly identical to that of the bentonite/cement mixture (see also Section 3.5.2).

4.6.1.3 Cement

Blast-furnace cements offer an increased resistance to erosion and chemical attack. These cements are therefore recommended especially for use in vib-wall slurry mixes. Ideally, the grading curve of cement should be nearly identical to the grading curve of the other base materials, but cement with a somewhat increased Blaine, i.e. around 4000, should be adequate.

4.6.2 Required properties of fresh slurries and quality control

The physical and engineering properties required for slurries used in vib wall production are governed by the characteristics of the ground to be sealed and by the construction boundary conditions. The information presented below is largely derived from ICOLD Bulletin 51 (ICOLD, 1985). Table 4.1 gives a brief account of the required properties of fresh slurry. These are almost identical with those specified for single and two-phase systems (see Section 3.5). Table 4.2 lists the methods to measure slurry properties for vib walls and indicates limiting values.

Table 4.1 Requirements for slurry properties in the fresh state

PROBLEM	GOALS TO ACHIEVE AND ADVERSE PROCESSES TO OVERCOME	METHODS OF CHECKING, MITIGATING AND TESTING
Trench stability	Generating sufficient support to stabilize the slot wall	Checking the density of the slurry by the mud balance
	Prevention of squeezing the slurry out of the slot by the horizontal earth pressure	The stabilizing hydrostatic pressure by the suspension can be adjusted by the density of the suspension
Slurry loss into trench wall	Minimizing suspension losses into the base of the excavation	The yield stress is the controlling parameter. It enables the transfer of shear stresses from the suspension to the porous wall. There could also be filtration losses, especially if during construction the groundwater level is low.
	Suspension may penetrate into the pores of the soil forming the trench wall. Larger pores promote increased flow.	In case of very large pores (e.g. in gravels) addition of sand may be helpful to mobilize the yield stress as well as other geometrical constraints (filters). Especially in very deep cutoffs the yield stress of the suspension becomes increasingly important with regard to the expected suspension losses. In the case of narrow pores, suspension losses occur more slowly and the viscosity of the suspension may also have an influence on the rate of suspension losses. The Fann V-G rotational viscosimeter can be used to design the suspension mixture in the laboratory while the Marsh funnel viscosimeter is employed for checking the slurry at the site.

Slurry segregation	Slurry segregation inside the trench may cause stability problems and suspension losses.	Slurry segregation is related to slurry stability
	Separation of the soil constituents from the slurry within the slot must be avoided. In deep trenches there is a high risk of solid materials at the base of the trench, which can produce insufficient density of the slurry in the upper parts of the trench. The solid material can also drastically change yield stress and viscosity of the suspension	The bleeding of the slurry must be as low as possible in order to achieve uniform slurry properties over the entire depth of the trench and ensure stability in its upper part. The choice of a suitable bentonite will guarantee suspension stability even in contaminated or untreated groundwater. For vibrosol the transport capacity should be adequate to prevent settling of coarse components remaining in the slot
Filter cake formation	Prevention of excessive filter cake formation along the contact between slurry and natural ground of the trench wall.	Filter press test
	With fine-grained soil types the slurry is pressed against the trench wall due to the excess hydrostatic pressure. Low permeability prevents the colloidally-mixed suspension particles to penetrate into the pores of the soil forming the trench walls. If the slurry is not sufficiently stable separation between water and the solid constituents of the suspension, may occur with the water flowing into the pores while the solid material forms a cake on the trench walls. This may cause an arching effect by the solid constituents. With very permeable soil layers below such an arch, a slurry deficit may develop.	With the filter press test the separation susceptibility can be determined by the application of an excess pressure. The extruded water filtrate serves as a measure of the separation susceptibility.

Table 4.2 Methods of measuring properties and recommended limiting values for vib-wall slurries (see also Section 3.5.1)

QUANTITY TO MEASURE, MEASURING DEVICE AND PROCEDURE	RECOMMENDED LIMITING VALUES
Slurry density Apparatus: Mud balance, hydrometer Measurements may be performed up to 10 minutes after mixing	Slurry density: $\rho > 15$ kN/m^3 Measurement frequency: Two measurements per working shift
Marsh cone viscosity Apparatus: Marsh funnel viscometer Measurements must be performed immediately after mixing Basic fill volume: 1.5 liters Results: runout time for 1.0 liter = t_M	Recommended Marsh cone viscosity of the vib wall slurry: $38\ s < t_M < 85\ s$ Measurement frequency: Two measurements per working shift
Yield stress Apparatus: Marsh funnel viscosimeter or Fann V-G rotating viscometer Measurements must be performed immediately after mixing	Can be determined from Marsh cone viscosity $\tau_F = 28.03 \ln(t_M) - 95.4$ [Pa] Input value is the Marsh cone viscosity, t_M, for 1 liter (in seconds)
Filtration or fluid loss	Single-phase slurry: $F < 60$ cm^3

Apparatus: Filter press according to API Measurements should be completed within 10 minutes after mixing	
Bleeding Apparatus: Measuring cylinder Readings should be taken after at least 30 min. settling time	Bleeding as a measure of stability: after 30 minutes: A < 3 % after 60 minutes: A < 4 %

Vibrosol jet grouts are mixed with the surrounding soil. It is therefore not necessary to optimize yield stress and viscosity. Based on recommended water-cement ratios for vibrosol jet grouts which result from the properties of the hardened soil/slurry mixture, the following parameters are proposed:

- Density $\rho > 15$ kN/m^3
- Stability (bleeding) A < 4 %
- Marsh cone viscosity $t_M < 70$ s
- Filter stability F < 60 cm^3

4.6.3 Mitigating the formation of imperfection zones, rate of beam withdrawal

During the construction of vib walls system-related imperfection zones may be created, even under optimum conditions. These zones may be generated, for example, as a result of extrusion of the slurry from the vib wall slot. The literature presents various explanations on the apparently unavoidable formation of such imperfections during wall construction. A common feature of these imperfection zones is that extrusion takes place mainly in slightly permeable soil layers, i.e. soft silt, mud and sand.

Below extrusion zones there may be zones with no slurry because the slurry has penetrated into the pores of more pervious strata, as explained in Fig. 4.8, and cannot be replenished from the withdrawing beam. A zone with a slurry deficit has been created.

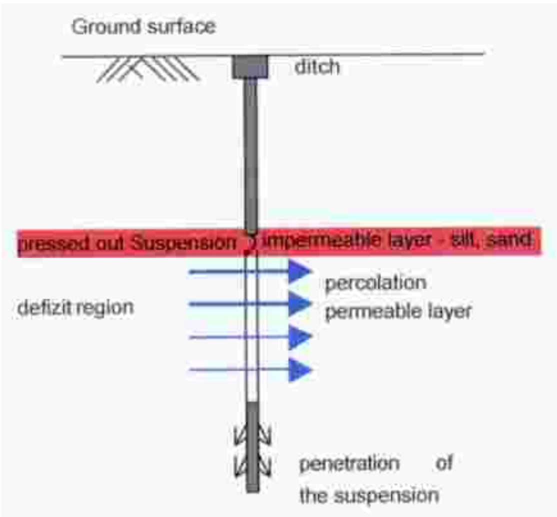

Fig. 4.8 Extrusion zone and development of a zone with imperfections (slurry deficit) (Kleist & Strobl, 1999)

A certain degree of mitigation is possible by controlling the rate of withdrawal of the vibrated beam. The withdrawal rate must be slow enough to ensure that a sufficient volume of slurry is injected to fill the cavity below the beam. From experiments at the Technical University in Munich and from actual measurements at construction sites, a maximum velocity of withdrawal was determined which takes into account rheological parameters of the slurry, soil parameters, and performance characteristics of the construction equipment (Kleist & Strobl, 1999).

$$v_z = \frac{Q_P}{A_{Beam} + f_z \ s} \tag{4.1}$$

where: v_z = recommended maximum velocity of withdrawal (m/min)
Q_P = pump feed rate of slurry (m^3/min)
A_{Beam} = cross sectional area of vibrated beam (m^2)
f_z = factor to account for the penetration depth of the beam into the surrounding ground
s = web height of vibrated beam

Values for the factor f_z are listed in Table 4.3 as a function of the Marsh cone viscosity, the slurry pressure and the grain size D_5.

Table 4.3 Factor f_z for use in Eq. 4.1 to estimate the recommended beam withdrawal rate

Hydrostatic slurry pressure in the soil layer considered (kPa)	Grain size D_5 (5 percent passing) of soil layer under consideration (mm)														
	0.06			0.2			0.6			2			6		
	Marsh cone viscosity (s)			Marsh cone viscosity (s)			Marsh cone viscosity (s)			Marsh cone viscosity (s)			Marsh cone viscosity (s)		
	35	45	55	45	55	65	55	65	75	65	80	35*	85	35*	45*
30	0.022	0.008	0.005	0.027	0.018	0.014	0.055	0.043	0.036	0.143	0.112	0.081	0.318	0.243	0.175
60	0.032	0.012	0.008	0.040	0.027	0.021	0.080	0.063	0.053	0.210	0.165	0.119	0.467	0.358	0.257
100	0.044	0.017	0.011	0.055	0.037	0.029	0.111	0.087	0.073	0.290	0.228	0.165	0.644	0.494	0.354
150	0.058	0.022	0.015	0.073	0.049	0.038	0.146	0.114	0.096	0.381	0.300	0.216	0.847	0.649	0.465
200	0.071	0.027	0.018	0.089	0.059	0.047	0.178	0.140	0.118	0.466	0.367	0.265	1.036	0.794	0.569
250	0.083	0.031	0.021	0.104	0.070	0.055	0.209	0.164	0.138	0.546	0.430	0.311	1.216	0.932	0.668
300	0.095	0.036	0.024	0.119	0.080	0.062	0.239	0.187	0.158	0.624	0.492	0.355	1.389	1.065	0.763
350	0.106	0.040	0.027	0.134	0.089	0.070	0.268	0.210	0.177	0.699	0.551	0.398	1.556	1.193	0.855

Notes: (1) Runout times marked with asterisk (*) represent values obtained with the DoKW (Donau Kraftwerke) funnel with 20 mm long tip and opening diameter 10 mm, as compared to 50 mm and 4.75 mm for the Marsh funnel.
(2) The shaded figures in the table show critical conditions; thin walls are not practicable unless accelerators are used

4.6.4 Mix design

The following ranges for the components of mixtures are recommended for conventional vib-walls and for vibrosol walls:

Conventional vib-walls:

Blast furnace cement	140 to 160 kg/m^3
Rock flour	550 to 800 kg/m^3
Bentonite	20 to 120 kg/m3
Water	600 to 700 kg/m^3

Vibrosol walls:

Blast furnace cement	300 to 400 kg/m^3
Rock flour	400 to 800 kg/m^3
Bentonite	10 to 80 kg/m3
Water	600 to 700 kg/m^3

4.6.5 Properties of hardened slurries

Table 4.4 presents recommended properties of the hardened conventional vib-wall slurry and of the vibrosol slurry. The table also indicates the justification behind the values required.

Table 4.4 Recommended properties of slurries for vib-walls and vibrosol walls

CONVENTIONAL VIB-WALL	
Recommended property	Justification
Density $\rho > 15$ kN/m^3 Uniaxial strength $q_u > 0.3$ MPa	Stability against erosion
Modulus of elasticity $E_{wall} < 4\ E_{soil}$	Deformability, ability to sustain large strains
Coeff. of hydraulic conductivity, $k < 10^{-8}$ m/s	Wall has sealing function
VIBROSOL WALL	
Density $\rho > 15$ kN/m^3 Cement/water ratio > 0.5 Uniaxial strength $q_u > 0.3$ MPa	Stability against erosion
Modulus of elasticity $E_{wall} < 5\ E_{soil}$	Deformability, ability to sustain large strains
Coeff. of hydraulic conductivity, $k < 10^{-9}$ m/s	Wall has sealing function

4.7 PRODUCTION RATE AND COSTS

With the conventional vib-wall method of construction, production rates of 40 m^2 per working hour are possible. The vibrosol technique allows maximum production rates of about 60 m^2 per working hour.

It must be pointed out here that vib-wall construction may not be possible when temperatures fall below 5°C to 10°C.

The costs for conventional vib walls are about US$ 25.00 to 35.00 per m^2. Vibrosol walls have a price of about US$ 45.00 to 65.00 per m^2. (Basis: year 2000).

A case history regarding the application of vib walls for the Isar River Dykes (Germany) is presented in Appendix 2.

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5. PILE WALL

5.1 PRINCIPLES OF CONSTRUCTION

A thorough geotechnical investigation is needed during the design of a project in order to define the depth and extent of the cutoff wall and the materials required to be penetrated. In addition, during construction, exploratory borings have been drilled at relatively close intervals ahead of wall construction to define the depths in variable ground conditions, for example, in weathered rock profiles, where depths of cutoffs can change significantly over short distances.

So-called “pile” or “shaft” walls can be constructed by a variety of methods. “Secant” walls, shown in Fig. 5.1a, are typically constructed by drilling a series of overlapping shafts and backfilling them with concrete to form a seepage barrier. In this method alternate (primary) shafts are drilled and backfilled with concrete and allowed to set before closure (secondary) shafts are drilled and backfilled with concrete. The secondary shafts are drilled when the concrete of the primary shafts has reached an acceptable strength, usually within three to seven days after concrete has been placed in the primary shafts.

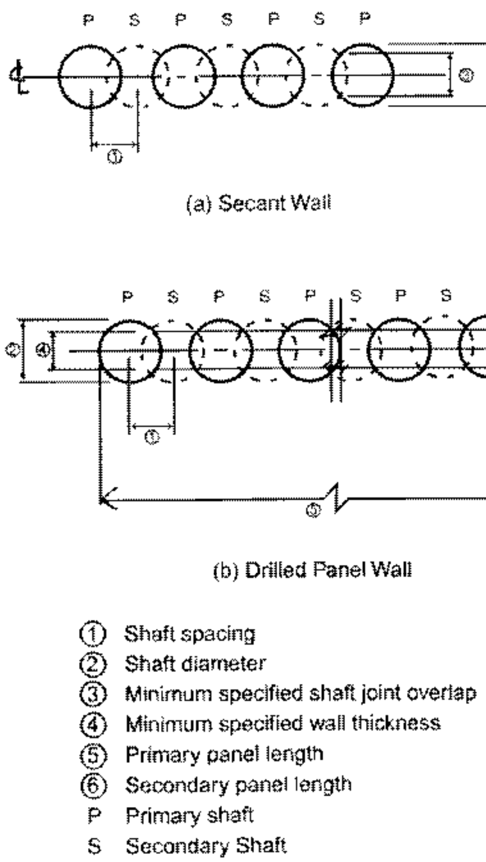

Fig. 5.1 Cutoff types

An alternative method of construction, which was done for a 3000 m long pile wall at the East Dam of the Diamond Valley Lake project in California, involves drilling primary shafts over a specified primary panel length, say 10 m, and then removing the materials between the shafts either by drilling them out (secondary shafts) or by breaking them out with a chisel or clamshell (Fig. 5.1b). After proper concrete strength is reached in the primary panels, closure (secondary) panels of similar length are then drilled out (secondary shafts) and backfilled with concrete. The panel excavation is then backfilled with tremie concrete. The advantage of this alternative is that there are much fewer joints than for the secant wall, and thus fewer opportunities for defective joints to develop in the wall. A disadvantage is that drilling the intermediate secondary shaft holes in the panels can be difficult due to drill displacement. The remaining materials must then be removed by chisel, clamshell or other equipment. Overbreak and slow productivity can result by using this technique.

Pile shaft extension is typically done with large diameter drills, such as downhole hammer drills, rotary drills and cluster drills. Typical shaft diameters range from 0.6 m to 0.9 m. High volume compressors to remove drill cuttings are also required for downhole hammer drills.

If shaft excavations require support, bentonite-water slurry can be utilized. The limiting properties (per the American Petroleum Institute) of the slurry are summarized in Table 5.1.

Table 5.1 Recommended limiting properties of slurries used for excavation support

Property	fresh	in excavation
Viscosity	32 to 40 Marsh seconds	36 to 50 Marsh seconds
Density	1.03 to 1.04 g/cm^3	1.04 to 1.20 g/cm^3
pH	6.5 to 10	6.5 to 11
Filtrate loss	15 to 20 ml	15 to 25 ml
Cake thickness	1 to 3 mm	1 to 3 mm

During shaft/panel excavation and prior to concreting, the bentonite slurry is maintained at a level well above the groundwater table (minimum of 1 m) in the shaft excavations and, as necessary, can be pumped to a de-sanding plant and re-circulated back to the cutoff excavations.

After excavation of a pile shaft, the hole is cleaned out by air-lift, suction, and/or cleanout bucket. Concrete is then tremied into each shaft. The tremie pipe (non-aluminum) is lowered to the bottom of the shaft as far as about 0.5m above the bottom of the shaft. A plug is used for the first concrete placed through the tremie pipe to keep slurry or groundwater from mixing with the concrete. The tremie pipes are typically 250 mm in diameter, and are located in the center of the shafts. As the concrete is being placed, the tremie pipe is withdrawn but kept about 3 m below the concrete surface.

Examples of dams where pile cutoffs have been used are presented in Table 5.2. The table includes cutoffs for both new dams and for seepage control remediation of existing dams. Case histories regarding some of these projects are presented in Appendix 3.

Table 5.2 Pile wall cutoffs for seepage control in dams

Project	Location	Ground materials	Max. depth	Shaft diameter (m)	Shaft spacing (m)	Shaft overlap (m)	Concrete strength (MPa)
Beaver Dam	Arkansas, USA	Limestone with clay-filled cavities	60	0.86	0.61	0.61	21
Long Park Dam	Utah, USA	Interbedded sandstone and shale, with open joints	18	0.51	0.41	0.20 specified	14
East Dam, Diamond Valley Lake	California, USA	Weathered gneiss and schist	34	0.76	0.60	Panel length of 10 m. End shafts drilled out for panel connection	5.5-8.3 (plastic concrete)
Wolf Creek Dam	Kentucky, USA	Embank-ment dam and underlying limestone	85	0.60	-	-	-
R. D. Bailey Dam	West Virginia, USA	Fractured sandstone	18	-	-	0.30	-
Khao Laem Dam	Thailand	Karstic limestone	55	0.76	0.61	0.45	-

5.2 CONSTRUCTION MATERIALS

Shafts in unweathered rocks for water seepage cutoffs (non-structural applications) are typically backfilled with concrete generally ranging in strength from 14 MPa to 21 MPa. To match foundation stiffness, plastic concrete, a mixture of aggregate, cement, bentonite and water, can be used. The aggregates include non-plastic silty, sandy soils. The mix proportions are determined from mix design tests to achieve the required strength and permeability requirements. At the East Dam for the Diamond Valley Lake project in California, plastic concrete in a weathered rock foundation had specified 28-day compressive strengths of 5.5 MPa to 8.3 MPa and a hydraulic conductivity not greater than $5 x 10^{-7}$ cm/s.

Cement should conform to ASTM C150; typically Type I-II cement is used. If resistance to sulfate attack is required, Type V cement should be used. To improve workability, ASTM C618 Class F fly ash is used to replace a portion of the cement in the mix (about 30% of total cementitious materials by weight). Water reducing agents are used to improve durability and workability and to reduce the water-cement ratio.

The aggregates need to be non-reactive to the alkalis in the cement and should meet ASTM C33 durability requirements as stated in Table 5.3.

Table 5.3 Durability requirements for aggregates

Test	Test standard	Acceptability criteria
Specific gravity	ASTM C127	2.6 minimum
Absorption	ASTM C128	2.0% minimum
Abrasion resistance	ASTM C131	10% maximum loss of weight at 100 revolutions; 40% loss of weight at 500 revolutions
Sodium sulfate soundness	ASTM C88	10% maximum weighted average loss of weight after 5 cycles

5.3 DESIGN CRITERIA

To validate that construction procedures can satisfy specified design criteria, the contractor should be required to construct a cutoff wall test section. The test section can be part of the production cutoff wall. If the test section fails to meet specified criteria, it can be repaired as stated below. The test section should include verification that shaft verticality criteria can be met, shaft cleaning procedures are acceptable, concrete placement techniques are effective, and joint overlap meets the specified requirements.

The designer should specify the minimum wall thickness required, or the minimum joint overlap (perpendicular to centerline of wall) between intersecting shaft elements of a secant wall. For deeper walls, greater overlap is prudent to account for drift of the shaft elements. In general, the overlap should not be less than 200 mm for cutoffs less than 20 m deep. The diameter and spacing between drilled shaft elements should be left to the contractor to achieve the minimum joint overlap requirement. The spacing of the shafts should be less than theoretical spacing to allow for slight inclinations of the shafts. If the required overlap is not achieved, the joint can be over-drilled and backfilled with concrete. For this purpose, a larger diameter drill than for the production cutoff shafts can be used.

The allowable inclination of the shafts should be limited to one percent. However, this requirement could conflict with the minimum required joint overlap. If this is the case, then the minimum overlap requirement should govern since overlap controls wall continuity.

The criterion that is often used, where cutoffs are used in dams in seismically active areas, is to have the stiffness of the plastic concrete be approximately the same as that of the foundation materials through which they are constructed. This is done to reduce the potential for cracking of the cutoff wall under seismic loading. Unconfined compressive strengths are used as an index to control deformation moduli.

Hydraulic conductivities for plastic concrete are generally specified to be less than $5x10^{-7}$ cm/s (Millet et al., 1992). Hydraulic conductivities of plastic concrete are measured in accordance with ASTM D5084. For conventional concrete, hydraulic conductivities are less than $1x10^{-7}$ cm/s.

To provide for adequate workability of the concrete or plastic concrete mix, the slump is usually between 180 mm (7 inches) and 230 mm (9 inches).

5.4 LIMITATIONS

Pile cutoff walls have been constructed in a variety of foundation conditions, including soils and weathered and fractured rock (see Table 5.3). The cutoff at Khao Laem in Thailand was constructed to a depth of 55 m through karstic limestones (Watakeekul & Coles, 1985). At Beaver Dam in Arkansas, USA, the cutoff was constructed to a depth of 60 m in limestone with cavities filled with clay (Bruce & Dugnani, 1995). At Wolf Creek Dam in Kentucky, USA, a secant wall was constructed from the crest of an embankment dam into the underlying limestone foundation. The cutoff at R.D Bailey Dam in West Virginia, USA, was constructed to a depth of 18 m through fractured sandstone (Beene & Pritchett, 1985). Maximum achievable cutoff depths depend on materials through which the cutoff has to be constructed and on efficiently maintaining verticality, and thus the desired overlap, of the drill holes.

In open-jointed rock foundations, stabilizing bentonite slurry will leak out through joints and cannot be used for ground support unless other actions (e.g. grouting) are implemented. Caving ground conditions may require casing for support, or stabilization by pre-grouting or down-stage construction, as was done at Beaver Dam. Pre-grouting includes drilling and grouting holes just upstream and downstream of the cutoff alignment to stabilize the ground. Down-stage construction is accomplished by constructing the upper portion of the cutoff and backfilling the shaft holes with concrete. After sufficient elapsed time (more than 24 hrs), the cutoff is deepened by drilling through the upper stage concrete-stabilized zone.

Drill rates depend on ground conditions and drill type diameter. At Beaver Dam (Bruce & Dugnani, 1995) for 0.86 m diameter shafts, drill rates through limestone for primary shafts ranged from 2.4 m to 6.4 m per hour, and averaged 4.4 m per hour. For secondary shafts, the drill rate ranged from 4.1 m to 7.0 m per hour, and averaged 5.5 m per hour. At Long Park Dam in Utah, USA, for 0.51 m diameter shafts, the average drilling rate through interbedded sandstone and shale averaged about 18 m per hour.

5.5 METHODS OF QUALITY CONTROL

Shaft verticality is checked using pendulum measurement, internal inclinometer, Koden sonic imaging profile (in water-filled holes), or borehole deviation probe. These tools are lowered into the hole and measurements of verticality are made incrementally. From the relative vector positions between the shafts at specific elevations, and the shaft diameters, the joint overlap can be calculated.

Borehole cameras are also used to check primary shaft concrete from the open secondary shafts. The camera is lowered down the secondary shafts and the primary shaft concrete can be observed to check that concrete is exposed in the primary shaft walls. Also, the condition of the concrete can be checked for honeycombing and openings. Sufficient illumination will be required for borehole camera operation. Borehole cameras can be used below groundwater provided that water is clear. Use of a borehole camera in bentonite slurry stabilized holes is not possible.

Concrete compressive strengths (ASTM C39) on cylinders are checked at 7 days and 28 days, and if needed at 56 days. Compressive strengths for plastic concrete mixes are measured in accordance with ASTM C143 and are checked at the point of placement.

The following items are also checked as part of quality control:

- minimum overlap thickness
- depth of shaft excavation
- groundwater and slurry levels
- excavation time
- difficulties during drilling and/or backfilling with concrete (e.g., loss of drilling fluid or cross-connection of air or water between shafts),
- shaft verticality and horizontal alignment,
- cleanliness of bottom of hole and joint surfaces, and
- volume of concrete placed in each shaft (which is compared with the theoretical volume of concrete),
- backfill strength and hydraulic conductivity

The advisability of core drilling in the cutoff wall joints for quality control verification of continuity should be carefully considered before this is attempted. The core drill could drift out of the wall that would lead to erroneous conclusions. Instead, core drilling should be reserved for verification of wall continuity in unusual circumstances, such as suspected caving. The core hole could also be used for supplemental methods of verification, including borehole camera observations and borehole caliper testing. Any exploratory core holes must be properly backfilled with grout.

5.6 METHODS OF ANALYSIS

Methods of analysis are not specific to pile wall cutoffs. The depth and length of the cutoff is based on geologic conditions and on the results of seepage analyses. The cutoff depths are often determined by the requirement to reach a relatively impervious stratum. The extent of the cutoff in an abutment to reduce an "end-run" seepage condition is also based on seepage analyses and geologic and topographic conditions.

Mix design testing is required for plastic concrete mix design. The proportion of cement, bentonite, aggregate and water are varied by a trial-and-error process to observe their effect on strength, modulus, and hydraulic conductivity. Hydraulic conductivity testing and unconfined compressive strength testing are performed on the mixes. In addition, triaxial compression tests are performed to evaluate modulus. The modulus is compared to that for the in-situ ground, which can be evaluated by borehole dilatometer or pressuremeter testing.

5.7 DESIGN DETAILS

In order to control shaft alignment and verticality, guide slabs are used at the surface, as shown on Fig. 5.2. These slabs can have any width, but are usually wide enough to allow crane equipment to use them as platforms. Guide slab widths have ranged from 7.6 m to 10.7 m and the minimum specified thicknesses have ranged from 0.3 m to 0.9 m. Besides serving as a working platform, guide slabs also function as an impervious element at the critical contact of the cutoff wall with the core of the dam.

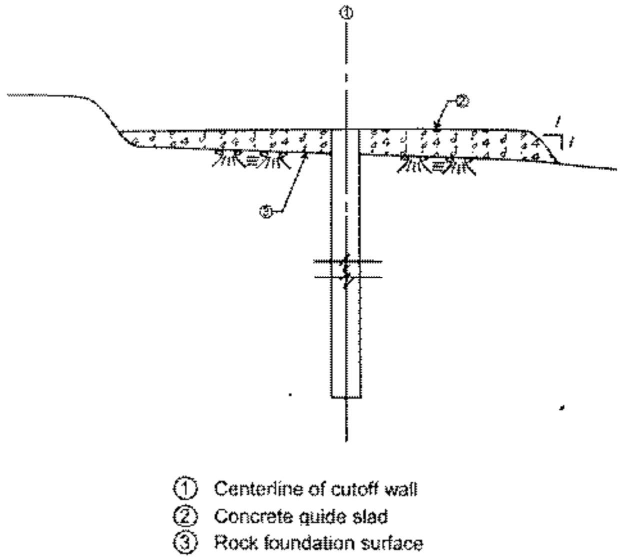

Fig. 5.2 Typical section through cutoff and guide slab

On sloping ground, the slabs can be stepped as shown on Fig. 5.3. To avoid sharp discontinuities under the core at the steps, backfill concrete is placed as fillets to smooth out such steps. At the Diamond Valley Lake Project, these steps were as high as 3 m. Alternatively, a sloping guide slab can be used to construct cutoff walls on abutments. At Long Park Dam in Utah, USA, a 143 m long secant wall was constructed on a 3.25:1V continuous slope. The crane was winched from the top of the guide slab from a pulley system.

To improve shaft alignment, a steel template can be bolted to the guide slab. Drilling then commences through the template.

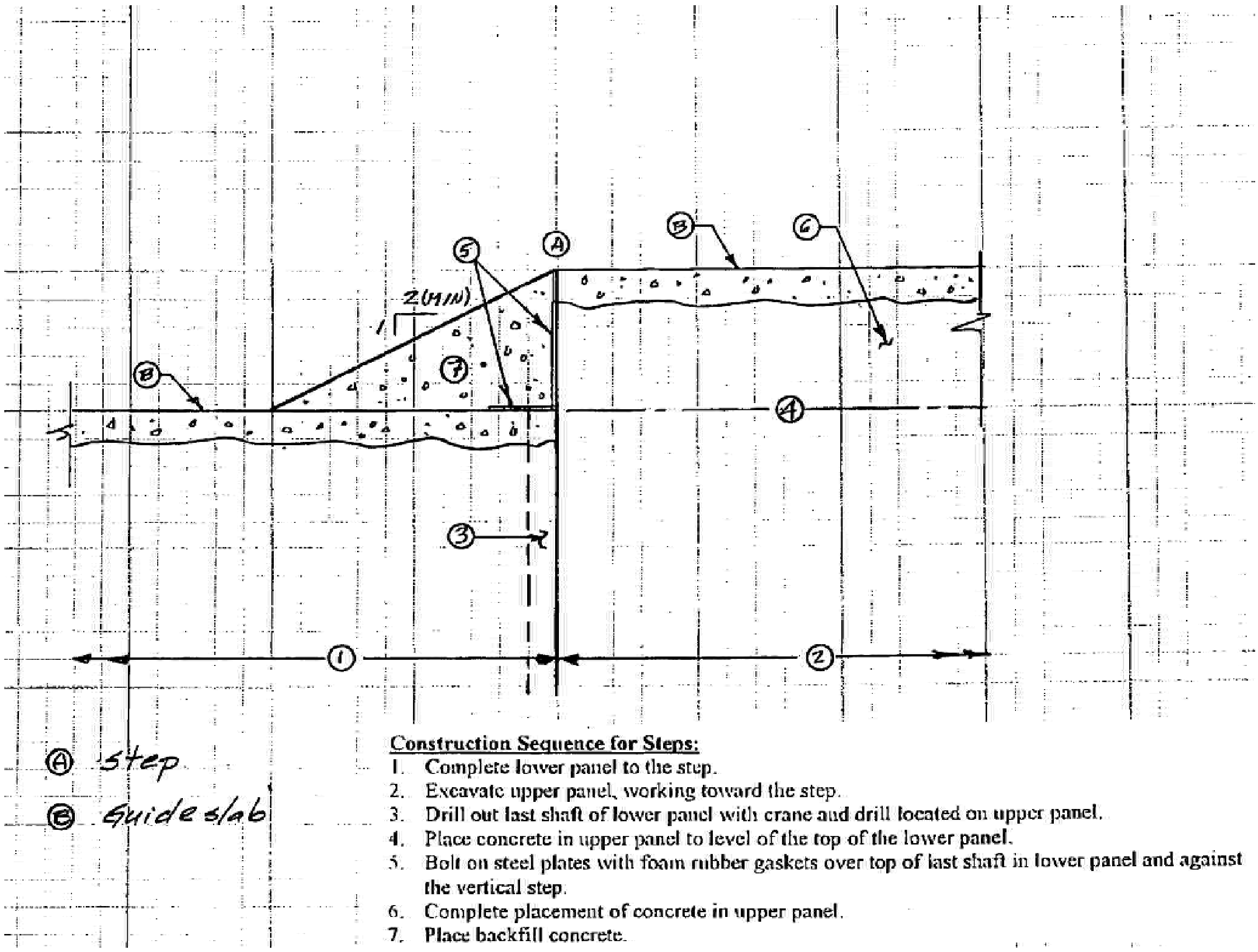

Fig. 5.3 Cutoff construction on sloping ground

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6. SUPERPOSED CONCRETED GALLERIES

6.1 INTRODUCTION

The utilization of superposed galleries, as a construction procedure to develop a positive cutoff is a not a common practice but in some instances, due to the site conditions and nature of the materials found, conventional treatments are not deemed suitable. Materials with certain consistencies hinder the efficacy of conventional grouting. As an alternative, chemical grouting has sometimes been considered, but later was discarded because of elevated costs and uncertain duration of the work. The last inconvenience is particularly important for the overall construction program of the whole dam, because interferences with other critical activities in the construction process can have a negative effect on the overall duration of the project.

The decision to implement a positive cut-off wall for a dam on a rock foundation is directly related to friable characteristics of the rock mass. Friable means here that due to lack of cementation of the matrix of the rock mass, it tends to crumble easily. Under this condition, joints generated by tectonic or thermal processes are often filled with material of the matrix. The low effectiveness of conventional grouting for the treatment of these materials can lead to piping under high pressures and to considerable seepage.

Construction of a cutoff by mining processes has been applied frequently also in karstic rock where the presence of large voids or residual clay filling renders construction of a conventional grout curtain costly and of questionable efficiency and durability. Typical examples are Karakaya (Turkey) and Henne (Germany). Mined cutoffs have also been constructed for mechanical strength, e.g. Karun I, (Iran) for the right bank shear.

Until now, the use of positive cut-offs and modern jet-grouting techniques have been mainly associated to wide valleys. When geological conditions related to the need of a cutoff are also linked to steep valley slopes, those solutions become inapplicable because of construction difficulties. For these circumstances, the concept of superposed galleries has been employed to install an impermeable barrier.

6.2 PRINCIPLE OF SUPERPOSED GALLERIES

Basically, the technique consists of superimposing adjoining galleries sequentially to achieve a continuous wall that could not be constructed otherwise. The high permeability and low resistance to erosion of the materials demanding the use of superposed galleries (or the circumstance that conventional grouting does not give useful results as in karstic rock with much residual clay), implies the use of construction procedures that avoid the cracking of the barrier elements between the galleries, and guarantee a good connection between the bordering galleries.

The approach to deal with this technique has varied from project to project. To illustrate the concept, some selected case histories are presented in Appendix 1 Section 4. Three distinct types of dams in three different stages are described. La Honda Dam (a zoned earthfill dam) has been in operation for more than 15 years; Salve Faccha Dam, a rockfill dam with central core whose construction was completed in 1999; and Sogamoso Dam, a concrete face rockfill dam, is currently under construction.

7. JET GROUTING

7.1 INTRODUCTION

Jet-grouting is a soil treatment technique by injection of cement grout into the subsoil, at very high speed. The result of a single treatment is a cemented soil body of quasi-cylindrical shape, which is usually named jet-column or also soilcrete column. Adjacent jet-columns can be interpenetrated with each other and thus, by appropriately arranging several treatments, it is possible to create water cut-offs for dam foundations.

Jet-grouting differs substantially from conventional grouting mainly in two respects. The first is that the original soil is entirely replaced by a new material (soilcrete) characterized by high strength, low deformability and very low permeability. The second is that both geometry and properties of the soilcrete elements can be reasonably defined at the design stage and experimentally verified during construction.

The use of jet grouting offers ease of construction, because heavy equipment is not needed. In fact, jet grouting may constitute the cheapest or only means of constructing a cut-off, in circumstances where normal methods such as excavation and diaphragm walling are too difficult to be applied.

Jet grouting effectiveness depends on the original characteristics of the foundation soils. Treatment results are particularly satisfactory for coarse-grained soils, while the use of jet grouting becomes questionable for fine grained soils or fissured rocks. A very convenient application is in well-graded river gravels including large boulders, because perforation is still feasible throughout the subsoil and large soil particles are then included in the cut-off.

7.2 JET-GROUTING TECHNOLOGY

7.2.1 Treatment methods

Three main types of jet grouting are distinguished, depending on the number of fluids injected into the subsoil: namely (1) grout (usually water-cement mixture), (2) air + grout, and (3) water + air + grout. The fluids are injected radially from the borehole into the soil, at very high speed, through small diameter nozzles placed on the bottom element (monitor) of the tool string. Therefore, the grout propagates in radial direction, with respect to the treatment axis, inducing a complex mechanism of soil erosion, mixing and/or permeation. The monitor is rotated at constant rate and is slowly raised towards the ground surface. After some time, the injected cement grout solidifies underground and a solid body of quasi cylindrical shape made of soil and hydrated cement (soilcrete) is finally obtained (Fig. 7.1).

Other shapes can occasionally be created by appropriately modifying monitor rotation. In particular, panels of treated soil may be formed by raising the monitor without rotation.

Jet grouting procedures and their effects are sketched in Figs. 7.2 to 7.4 while typical monitor features are reproduced in Fig.7.5, for single, double and triple methods of treatment. In particular, in the single and double fluid methods, the injection hole is drilled with water or mud circulation and left without casing. The rod is then extracted successively in steps of 40 to50 mm, and rotated at constant speed, while grouting. In the single fluid method, the jet is made only by grout, which is the only means producing soil erosion and mixing. In the double fluid system, the grout is surrounded by an annular air shroud, which improves jetting efficiency.

In the triple fluid system, boreholes are normally cased. The injection rod is then inserted into the borehole and slowly extracted while turning and grouting. In this case, soil erosion is obtained by an upper jet of water surrounded by an air shroud and the cavity is then filled by a lower jet of grout.

For each method of treatment, the jet-column diameter can be increased by preliminary soil remoulding, obtained by water or air/water jetting, performed in a descending pass before grouting in the subsequent ascending pass. This method allows a more economical use of grout because the column is created by two passes of which the first one does not use grout.

7.2.2 Grout mix

The grout is usually prepared by mixing Portland cement (c) and water (w), with weight ratio c/w ranging between 0.8 and 1.5. Bentonite may also be added to the grout mix to improve the pumping characteristics of the grout and to enhance the deformation properties of the set material.

Cement which complies with BS 12, "Ordinary and rapid-hardening Portland cement" or ASTM C150, "Specification for Portland Cement" is generally suitable for jet grouting. However, the cement type should be compatible with the groundwater chemistry and it may be necessary to use other types of cement such as sulphate resisting cement.

Where strong groundwater flow exists, the grout could be eroded before hardening. In such case, a quick setting mix could be prepared by adding about 3% of chlorides ($CaCl_2$ or NaCl) by weight with respect to cement.

7.2.3 Mixing plant

The mixing plant consists of cement silos, mixing/batching units and bentonite circulation reservoirs or mud pits. This plant can also be used for the preparation and delivery of grout for conventional injection grouting. In order to ensure satisfactory mixing, it is recommended that high shear colloidal mixers are employed. Bentonite should be circulated for 24 hours, prior to incorporation in the grout, to ensure full hydration of the bentonite before mixing.

Fig. 7.1 Jet grouting rig in operation above ground

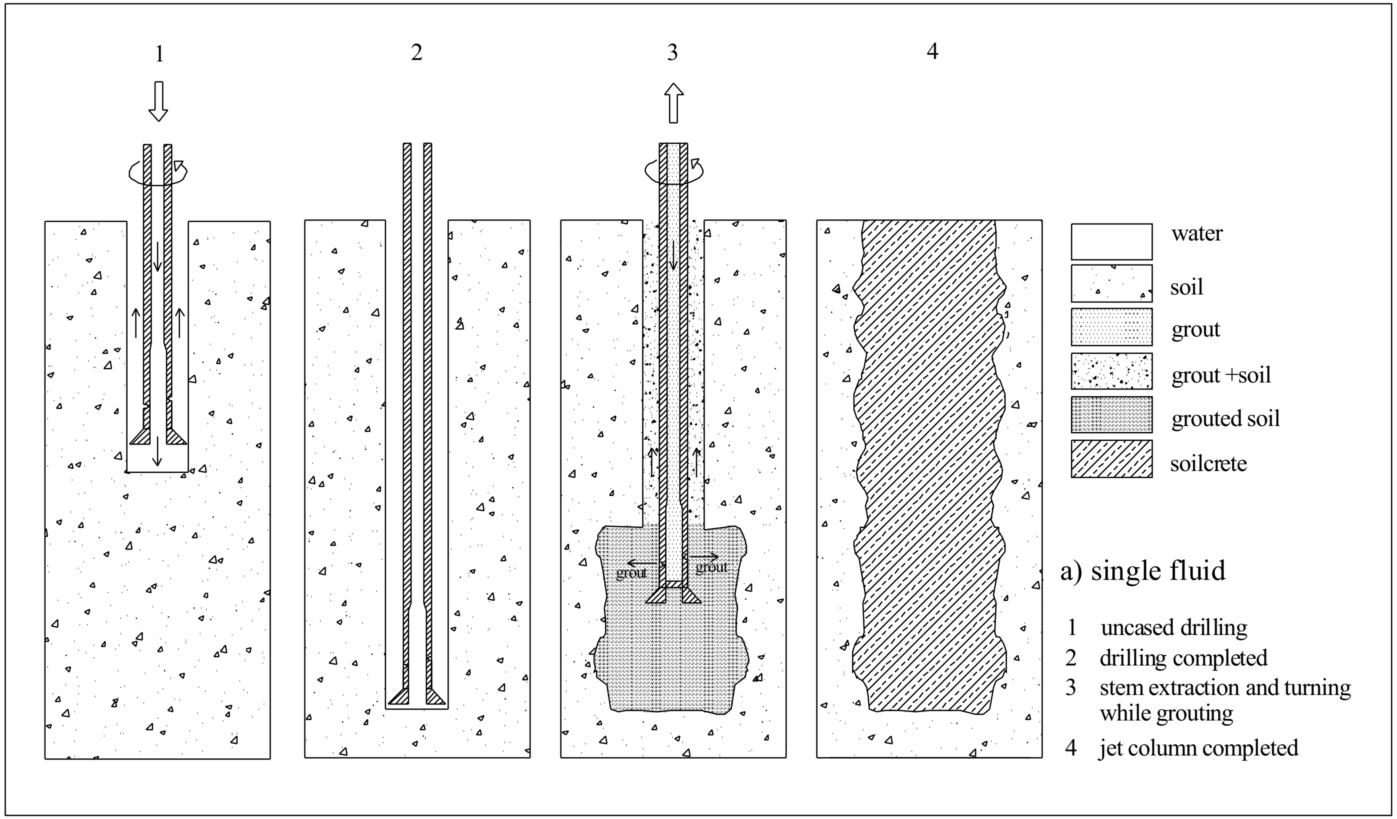

Fig. 7.2 Jet grouting procedure: Single fluid method

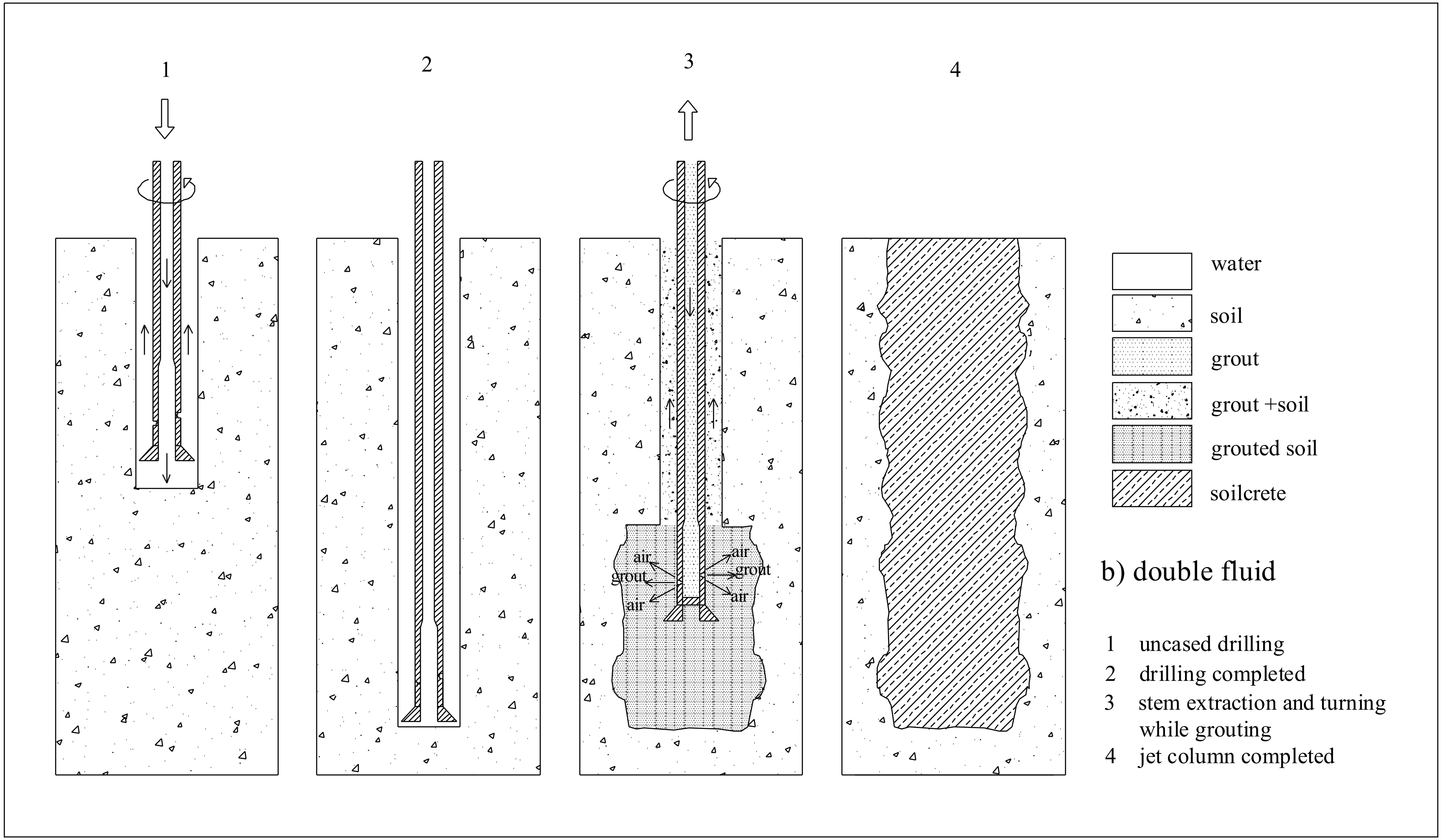

Fig. 7.3 Jet grouting procedure: Double fluid method

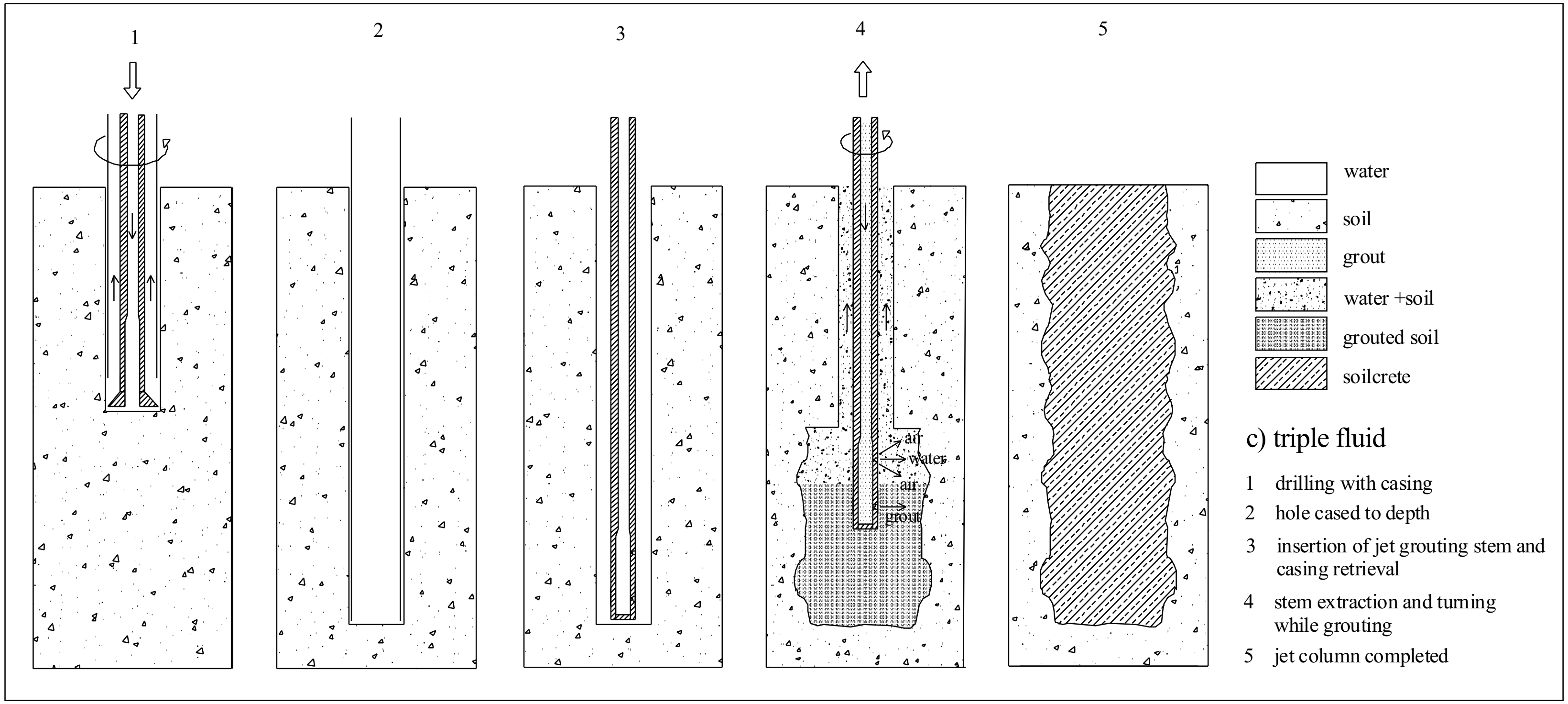

Fig. 7.4 Jet grouting procedure: Triple fluid method

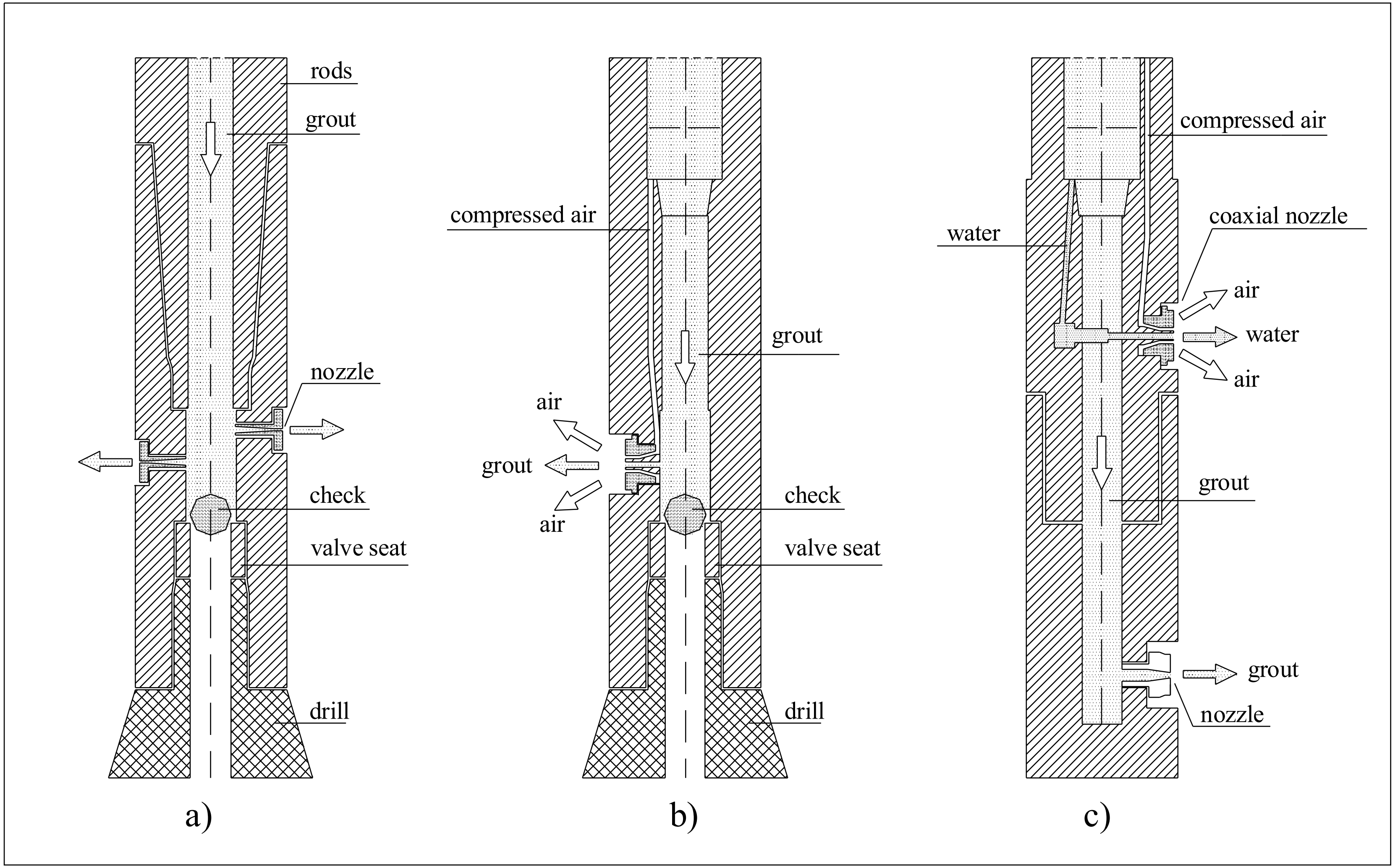

Fig. 7.5 Typical jet grouting monitors: (a) single fluid, (b) double fluid, and (c) triple fluid

7.2.4 Treatment parameters

A jet grouting treatment is defined by grout compositions and the values of the parameters listed in Table 7.1, together with typical ranges adopted. Strictly speaking, these parameters are the only ones which should be specified and controlled for each project. However, other parameters, which may be useful for design purposes, can be calculated from the previous ones. In particular, for the single fluid system, the following parameters have been suggested (Tornaghi, 1989, Croce & Flora, 2000):

Volume of injected grout per unit length of treatment ($V_j=Q_g/v$)
Grout energy per unit length of treatment at the pump ($E=p_gQ_g/v$)
Grout energy per unit length of treatment at the nozzles ($E'=8\rho Q_g^3/\pi^2M^2d^4v$)

where the notation corresponds to that given in Table 7.1 and ρ is the density of the grout in kg/m^3.

Table 7.1 Treatment parameters

TREATMENT PARAMETER	UNIT	SINGLE FLUID	DOUBLE FLUID	TRIPLE FLUID
Treatment time per lifting step, Δt	s	4 – 6	6 – 10	8 – 80
Monitor lifting step, h	mm	40 – 50	40 – 50	40 – 50
Average monitor lifting rate, v ($=h/\Delta t$)	$m/s \cdot 10^{-3}$	4 – 10	1 – 8	0.5 – 5
Rotational speed, ω	rpm	5 – 40	5 – 40	5 – 40
Diameter of grout nozzles, d	mm	1.2 – 4	2 – 4	2 – 4
Number of grout nozzles, M	–	1 – 2	1 – 2	1 – 2
Grout pressure, p_g	MPa	40 – 55	20 – 40	2 – 10
Air pressure, p_a	MPa	–	0.5 – 2.0	0.5 – 2.0
Water pressure, p_w	MPa	–	–	20 – 55
Grout flow rate, Q_g	$m^3/s \cdot 10^{-3}$	1 – 10	1 – 10	1 – 3.5
Air flow rate, Q_a	$m^3/s \cdot 10^{-3}$	–	70 – 150	70 – 150
Water flow rate, Q_w	$m^3/s \cdot 10^{-3}$	–	–	0.5 – 2.5
Cement/water ratio by weight	–	0.8 – 1.5	0.8 – 1.5	1.2 – 1.5

7.3 DESIGN

7.3.1 Design steps

The cut-off below the dam is a very low permeability curtain which has the objective of controlling seepage and piping, as required for the performance of the dam and its foundation. The depth and lateral extent of the cut-off will be defined, in conjunction with drainage provisions, to meet stability and economic criteria. Basic principles for cutoff design can be found in Chapter 2 of this bulletin.

Once the cut-off dimensions have been defined, the use of jet grouting should be evaluated on the basis of technical feasibility and/or economy. One of the advantages in the use of jet grouting is that no excavation is needed prior to cutoff construction, and also that it is not necessary to take the treated zone all the way to the surface.

At the design stage, the feasibility of a jet grouted cut-off should be verified after careful geotechnical investigation, with the aid of some empirical criteria based on previous experiences in similar soils. Treatment technique, however, should be selected before construction, by means of specific investigations including the so called trial fields. For all major projects, trial fields have to be carried out in order to verify design hypotheses.

7.3.2 Investigations

Site investigations must be carried out reaching the maximum depth to be treated. They will include at least borings with SPT. Geophysical explorations are useful as well. They can be easily used also after cut-off completion, as non-destructive controls, and compared with the results obtained before construction to check treatment results.

Soil samples should be retrieved for laboratory analysis in order to determine the grain size distribution at least. In fine grained soils, undisturbed samples can be retrieved for laboratory triaxial testing. These tests will possibly be consolidated undrained (CU) and give the undrained shear strength, s_u.

7.3.3 Single column diameter

The basic choice in cut-off design concerns single column diameter (or panel thickness). In the literature, some empirical correlations exist for the three different techniques (single, double and triple fluid systems), which allow to estimate the jet column diameter as a function of soil grading in case of cohesionless soils or the shear strength in case of cohesive soils. Soil grading is expressed in terms of the SPT N-value and shear strength in terms of s_u, the undrained strength.

Since the volume of data on which these empirical correlations are based is still limited, they can only serve as a preliminary estimate. A field trial is then compulsory to have first-hand information on the effectiveness of the jet-grouting treatment on each specific soil.

The choice of the best jet grouting system to be used for a specific case must be based on local experience, cost effectiveness and feasibility. In general, for a given soil, the diameter increases with the fluid number but, on the other hand, also the cost per treatment increases and the equipment needed for treatment becomes heavier as well.

Once the jet system to be used has been selected and a column diameter estimated on the base of soil characteristics, the jetting parameters needed to achieve the design column diameter and soilcrete strength/permeability have to be specified. If soil properties change along the cut-offs, treatment parameters could be modified accordingly, in order to obtain a diameter as constant as possible.

7.3.4 Layout alternatives

Jet grouting cut-offs are formed by overlapping jet columns in single or multiple rows, but jet panels may also be adopted. Typical column and panel layouts are reproduced in Fig.7.6 (adapted from Bell, 1993). The choice of the proper layout must also consider that the deeper the treatment, the larger are the possible deviations of the boreholes and therefore losses in sealing efficiency.

The spacing and pattern of the columns should be selected such as to achieve the minimum cut-off thickness with the greatest economy and reliability. In selecting the spacing, the designer will take into account the expected variability of the column diameter and the likely deviation of the pilot holes, itself a function of cut-off depth as well as choice of drilling plant. The most efficient and cost effective design is achieved with a column diameter as constant as possible, which can be obtained by adjusting one or more of the treatment parameters, according to the ground conditions.

On the basis of the column trials (see Section 7.4.2) the installation procedure and parameters can be determined. Knowing the column dimensions and the estimated accuracy within each column center, location and deviation from the vertical can be controlled and the final column arrangement is chosen.

Deciding on the proper pattern of columns is fundamental to an effective cut-off. For shallow cut-offs in alluvium a two line staggered pattern has been proved to be most effective. For deep cut-offs, where deviation from the vertical becomes significant, it is essential to check the deviation of the pre-bored drill holes and, from the known primary and secondary column locations at depth, to determine the centres of tertiary or infill columns to maintain a minimum cut-off thickness.

Column patterns should be appropriate for the ground conditions. For instance, where softer lower permeability soils overlie coarser more permeable horizons, it may be possible to rely on a single line of well-overlapping columns in the upper soil horizon, but install a double line of contiguous columns at depth where the average diameter and placement accuracy is less.

Jet grouting cut-offs have been used in new dams as well as a remedial measure in existing dams. Cross section schemes from actual projects are shown in Figs. 7.7 and 7.8.

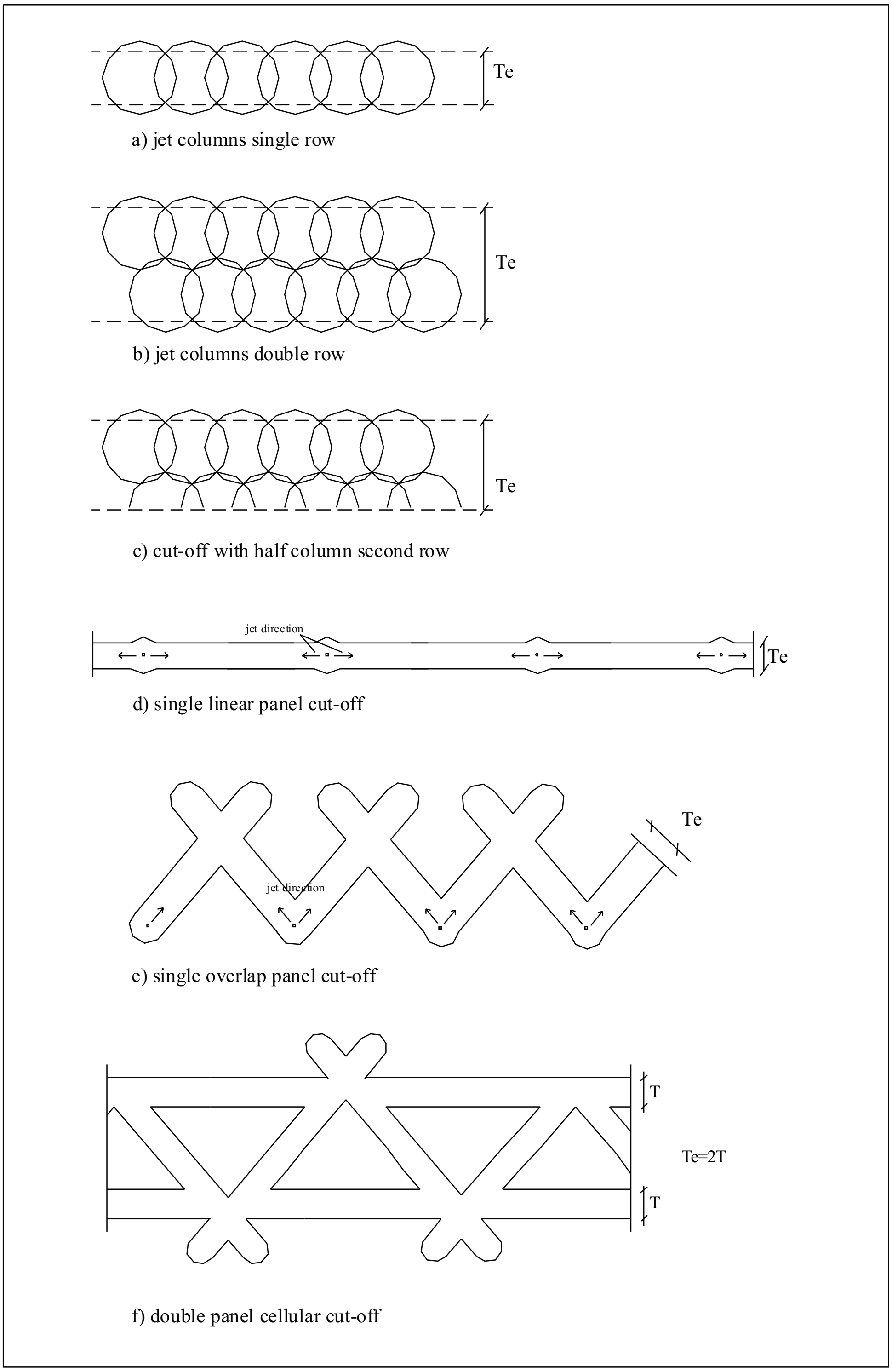

Fig. 7.6 Typical jet (soilcrete) columns and jet panel layout
(Te = effective thinness) (Bell, 1993)

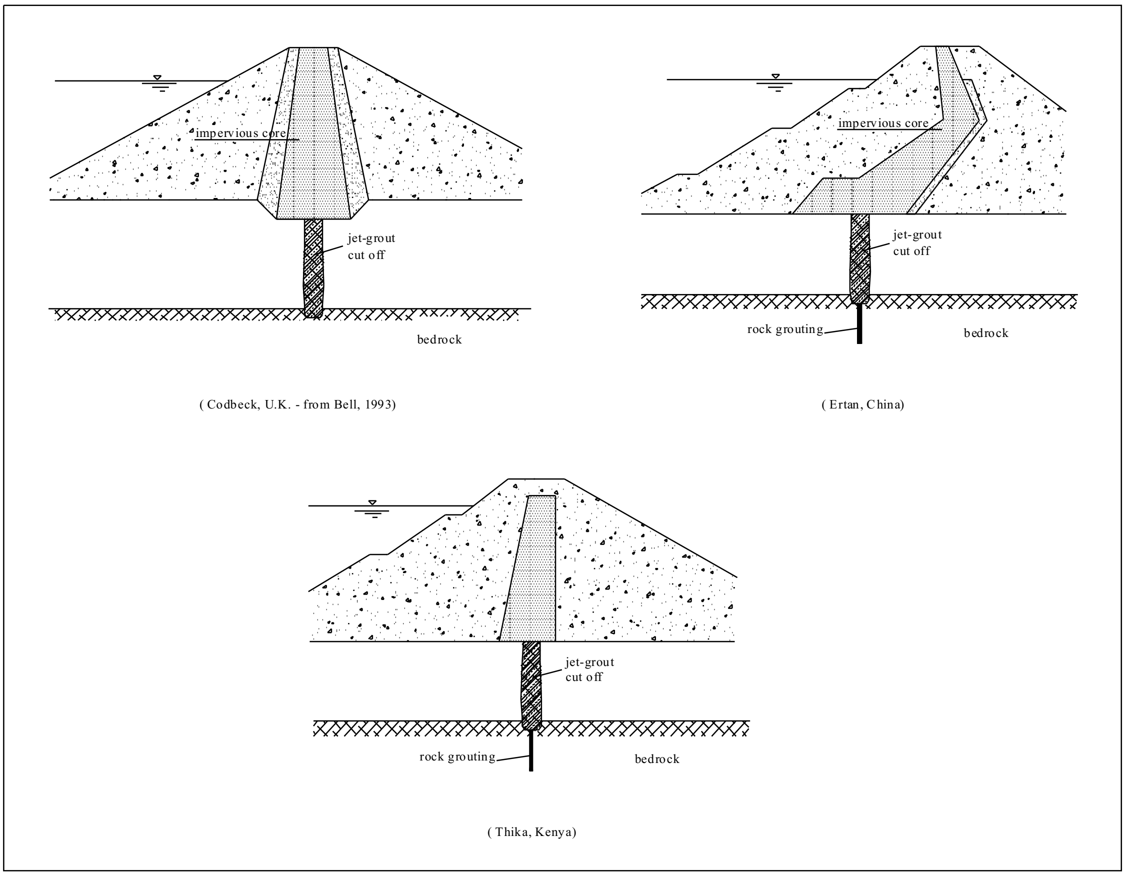

Fig. 7.7 Jet grouted cutoffs installed prior to dam construction

7.3.5 Cut-off permeability

The coefficient of permeability of a soilcrete unit volume is very low, and depends to some extent on the type of soil or other filler material mixed into the cement/bentonite grout, but will normally be lower than 10^{-9} m/s.

Although the overall cut-off permeability may achieve this value for shallow depths and in uniform soil conditions, values of 10^{-8} m/s would be a more realistic target. A further decrease to 10^{-7} m/s may occur for deep treatment and/or difficult conditions, depending on the extent to which the designer needs to reduce permeability by infilling untreated windows, reducing column spacing, or installing additional lines of columns.

The effective permeability of the completed cut-off will depend on its effective thickness and the degree of contiguity of the columns. Good contiguity is most important especially in open work gravels under high head where soilcrete erosion can occur along ungrouted zones. In soils and weathered rocks containing boulders too large to be displaced during jetting, there will be zones which are shielded from erosion by the jet and two or three staggered lines of columns will be necessary to minimise the occurrence of ungrouted «windows» in the finished cut-off. Alternatively, boulders can be encased within the treated zone, especially if two or more rows are used. Verticality control and the minimisation of untreated windows, especially at depth, are fundamental in achieving the lowest effective cutoff permeability. It is important to bear in mind that it will not usually be possible to achieve total continuity. For design purposes a conservative value of the cutoff coefficient of permeability is 1 to 5 x 10^{-7} m/s.

The overall design of the dam and its foundations will dictate the optimum value of the cut-off permeability, generally several orders of magnitude lower than the ambient foundation permeability. For injection grouted foundations a target permeability of 1 – 5 Lugeon is frequently adopted. The target permeability of a cutoff should generally be lower than this to account for the reduced average thickness of a cutoff compared to that of a grout curtain.

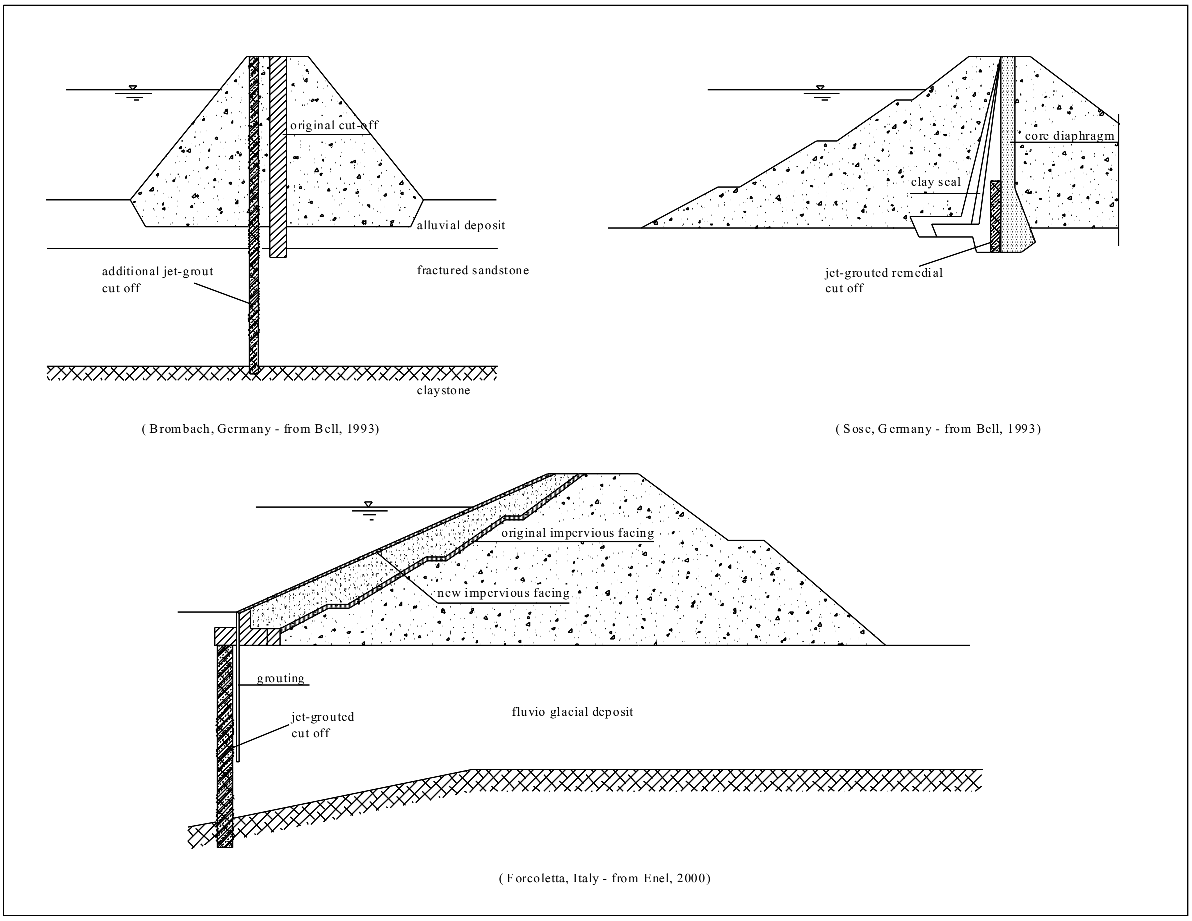

Fig. 7.8 Jet grouted cutoffs installed after dam construction (remedial work)

7.3.6 Cut-off strength

The treated soil (soilcrete) has mechanical properties which differ from those of the original soil. Typical ranges of uniaxial compressive strength are given in Table 7.2.

Table 7.2 Expected uniaxial compression strength of soilcrete

Soil type	Bell (1993) q_u (MPa)	Miki (1985) q_u (MPa)	Shibazaki (1991) q_u (MPa)
Clays	0.5 to 8	< 5	10
Silts	4 to 18		

Sands	5 to >25	5 - 10	30
Gravels	5 to >30		

7.3.7 Cut-off stiffness

Ideally, the stiffness of a cut-off beneath a dam structure should not be significantly different from the host foundation material as this will ensure an even stress distribution around the cut-off and at the foundation interface and uniform deformations during construction. Stiffness compatibility is less important in situations where no significant changes of loading will occur. To some extent, the stiffness of soilcrete will reflect the properties of the soil filler material in the mix and can be adjusted, within some limits, by varying the proportion of bentonite in the injected grout.

Triaxial shear tests on samples of the proposed soilcrete and the host foundation material will be necessary to confirm compatibility. Pre-construction trials and testing are most important in achieving best soilcrete design.

7.3.8 Limitations

Jet grouting for cut-offs rely on being able to place contiguous low permeability columns to build a wall of a regular minimum dimension. This method is therefore not appropriate in zones where fast interstitial water flow would cause open water passages to form during construction, and grout in its fluid phase would be carried away. The method is also not appropriate at depths beyond which it is not possible to control column location with sufficient accuracy to guarantee column contiguity. The maximum depth that can be jet grouted effectively depends on the accuracy of the drilling system used. Heavy drill rigs using heavy drill collars, combined with accurate inclinometer location, will enable most required depths to be achieved. Depths of up to 60 m have been reported.

7.4 QUALITY CONTROL

7.4.1 General

The control process should run in four phases; the first control must be carried out prior to construction (preliminary control), and should verify jet grouting feasibility. The second is the «in process control», which is aimed at ensuring that the process is correctly applied. The third control is the checking to ensure that the process has produced the desired results (post-construction checking) and finally there is the long term monitoring.

7.4.2 Preliminary control

Before starting work in new ground conditions, it is essential to carry out tests and trials so as to determine how to achieve the appropriate soilcrete mix, determine the size of the columns in each soil layer and weathering zone and the arrangement of columns to obtain an effective cutoff. It is usual to proceed in stages.

Stage 1 – Laboratory Testing

A range of manufactured soilcrete mixes can be made, cured and tested for modulus and permeability under laboratory conditions to give some guidance regarding the characteristics of the soilcrete to be achieved in the field. It is important to note that the soilcrete mix will vary with varying geological conditions and that one is seeking a satisfactory compromise.

Stage 2 – Trial Columns

The next stage is to make trial columns on site varying water cement/bentonite ratios, jetting pressures, flow rates and rates of withdrawal to achieve the desired product. At least 3-4 columns should be injected, with varied installation and mix parameters in an area where it will subsequently be possible to excavate around them to determine their dimensions under different ground or treatment conditions.

Samples of grout before injection are taken for viscosity testing (ASTM flow cone), bleed sedimentation and cylinder strength. Ejected slurry samples are taken to determine bleed sedimentation, and cylinder strength. These tests must be considered as indicator tests for monitoring, and not as check tests on actual soilcrete properties.

After a period for curing the columns, they need to be drilled to obtain cores for laboratory testing. The drillhole itself can be used for permeability testing if required.

At the Bujagali Hydropower Project, in Uganda, two rows of jet grout columns will be used in saprolite and weathered rock to achieve a cutoff below an embankment dam. The soil surrounding a group of three test columns was excavated to a depth of about 4m to observe in-situ continuity and variation in diameter of the columns."

Stage 3 - Test Cut-off Section Trial

On the basis of the test column results, the cut-off design can be carried out (see Section 7.3). It is economic at this stage to make the trial cut-off section form part of the working cut-off. Probably the most effective method of carrying out this trial is to install a short straight working section and a semicircular loop section which together enclose a D-shaped untreated zone. Once completed, column testing can be carried out as for the column trial in stage 2. In addition, a well should be installed in the untreated central zone with piezometers installed outside the D-shaped zone. In situ permeability tests can be carried out to check the overall efficiency of the cutoff. Similar D-ring tests can also be carried out as control tests during the main construction programme.

7.4.3 In-process control

7.4.3.1 Measurements

This phase involves the daily routine measurements of the grout properties to ensure that components are batched and mixed correctly. The verticality measurements (logs) are included in this phase. The results, particularly the verticality measurements, are reviewed at the end of each shift. On the basis of these reviews, the batching and mixing procedures may be revised as necessary. The verticality measurements are reviewed to determine the need for additional columns to fill-in «windows» inferred from the deviations and directions from vertical. The elements to be checked are listed under «Records» below.

In addition, the jetting parameters can be recorded automatically. These may include jetting pressure, rate of extraction or penetration, rotation, air pressure, grout pressure and flow. While not essential, the recording of the jetting parameters can highlight potential problems in column construction. If the above parameters are not automatically recorded then adequate manual records should be kept.

The grout pressure is measured close to the injection pump. Since jet grouting effectiveness depends on the energy at the nozzles, head loss along the injection line should be minimised.

7.4.3.2 Records.

The detailed records of jet grout column installation should include:

- Conventional geological drillhole logs
- Drillhole location / diameter / depth / rate of penetration / inclination / details of bits and casing used, etc.
- Fissure grouting records (if carried out)
- Grout materials records
- Grout mix records - mix ratios/viscosity/bleed/strength
- Ejected slurry records – bleed/strength
- Jet grout monitor records/types of column/pressures/flows/rates of rotation and withdrawal
- Column core test records – drill logs/strength/modulus/permeability
- Raked cored check hole records - drillhole logs/packer test results/strength
- D-ring permeability test records – rate of water absorption/ground water levels/calculated permeability.

7.4.4 Post-construction check tests

In this phase, quality control is aimed at checking that the process has achieved the desired effect. This includes checking for cut-off continuity and rotary sampling of soilcrete to evaluate strength, stiffness and permeability. In addition to checking the permeability of soilcrete unit volumes, it is also possible to check the «bulk» permeability of the cut-off by the D-ring tests. Furthermore, with a two row cutoff, in-situ water pressure tests, such as Lugeon or other, can be performed by drilling and testing boreholes located between the rows. On the basis of the results of these tests the as-built cutoff can be compared with the design expectations and further work might be requested. The influence of imperfections on the performance of the cutoff can be assessed on the basis of the results of this testing.

With regard to non destructive testing techniques (NDT), sonic logging, cross-hole and tomography have proved to be very effective in detecting badly treated zones (Croce et al., 1994).

7.4.5 Long-term monitoring

Geotechnical instrumentation and seepage measurement techniques are generally employed in this phase. These would include foundation piezometers to monitor the head drop across the cut-off and seepage collection points incorporating measuring weirs to monitor seepage.

The instrumentation would normally be installed during construction. Readings must start at an early stage, in order to clearly understand the pre-construction ground water regime and quantify the changes induced by the jet grouting cut-off.

Case studies where jet grouted cutoffs for dams have been constructed are the following: Ertan dam, China (Sembenelli & Sembenelli, 1999), Forcoletta dam, Italy (ENEL, 2000), John Hart dam, Canada (Garner et al., 1989), Paduli dam, Italy (ENEL, 2000), Pergau dam, Malaysia (Anon., 1993), Sainte Marguerite-3 cofferdam, Canada (Hammamji et al., 1999), Suio roof weir, Italy (ENEL, 2000) and Thika dam, Kenya (Harris& Morey, 1994)

Some of these case histories are discussed in Appendix 5.

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8. DEEP MIXING

8.1 PRINCIPLES OF DEEP MIXING

The soil-cement mixed wall technique consists of mixing in-place soils with cement grout or other binders by means of multiple axis augers and mixing paddles to construct sets (or panels) of overlapped soil-cement columns. The grout is mixed at a central plant, pumped down the hollow shaft of the augers and injected through their tips. The auger flights break the soil loose, which the paddles then blend with the grout. As the augers advance to greater depth the mixing paddles continue to mix the soil. When the design depth has been reached, the mixing shaft rotation is reversed and mixing continues as the shafts are brought to the ground surface. Some machines are capable to change the sense of rotation individually for each of the augers, both in the upward and downward directions. By overlapping the column sets, extensions to form a continuous soil-cement wall for use as a cutoff for seepage control, or as a support wall for an excavation can be constructed. Wall elements can be arranged in different configurations to suit various applications, for example, liquefaction stabilization of loose foundation soils. Soils have been mixed in-situ to more than 60 m in depth by this technique.

In concept, the deep mixing process attempts to achieve by mechanical mixing what jet grouting accomplishes with hydraulic power through a high-pressure jet.

The soil-cement mixed wall technique is competitive with the vibrosol method (see Chapter 4). For given subsurface conditions suitable for both techniques, mobilization costs for the mixed wall technique are lower than for the vib wall. Another advantage over the vib wall is the nearly vibration-free installation, a positive asset when considering environmental impact or possible crack formation in existing structures.

8.2 HISTORY OF DEVELOPMENT

The process of deep soil mixing has its roots in the early 1950's in the USA and considerable advances and refinements were made since then. In Sweden the technique of in-place lime stabilization of soft clays was developed in the late 1960's and early 1970's (e.g. Broms & Boman, 1977). Significant advances were also made in Japan starting from the 1960's to stabilize soft marine soils with lime. This "Deep Liming Method" (DLM) was later modified to the CDM method (Cement Deep Mixing). By using multiple shaft augers the productivity was significantly improved. The main application of the CDM method is large-scale submarine ground treatment (CDM, 1994).

Parallel with the CDM method, a technique to treat soil on land was introduced which is known under the name SMW method (Soil Mix Wall). Other names sometimes used are the DSM (Deep Soil Mixing) method or DMM (Deep Mixing Method). SMW is now a trade name registered by SEIKO Inc., Redwood City, California, USA. In Europe, the method is also known as the "Mixed-in-Place" (MIP procedure). A somewhat different technique is the DJM (Dry Jet Mixing) method in which a powdery or granular agent is pneumatically fed into soft soil and mixed (Subcommittee, 1997).

The deep mixing technique has been widely applied particularly in Japan, but also in the USA. For certain ground conditions and quality control requirements it can be superior to all other treatment methods.

8.3 PRODUCTION OF DEEP MIXING CUTOFF WALLS

The technology of mixing in-situ soil with cement slurry or soil-bentonite cement slurry to form a soil-cement wall comprises three elements whose balanced use determines the success of the method, i.e.:

- Soil-cement mixed wall equipment
- Mix design
- Installation operations

There exist many variations of the deep mixing technique. The mixing tool may or may not be the drilling tool. The binder may be in powder or in liquid form and introduced on the upstroke or downstroke, or both. In the following subsections the equipment developed by SEIKO, Inc. is described without endorsing this particular system. It is widely used in North America and in Japan (Jasperse & Ryan, 1987; Babasaki et al., 1991; Taki & Yang, 1991;Yang, 1994)

8.3.1 Equipment

The equipment includes a soil-mixing machine, a mixing plant and a flow control unit, described in the following:

The *soil-mixing machine* consists of a multiple axis auger guided by vertical steel lead on a track-mounted base machine as shown in Fig. 8.1. Two to five axis auger shafts with diameters between about 550 and 1000 mm have been used by SEIKO Inc. (Fig. 8.2). The base machine, together with the lead, is supported at three points during operation for maintaining accurate vertical alignment, which is critical to eliminate unmixed zones between column sets and to ensure the continuity of the soil-cement wall. The typical soil-mixing auger consists of an electric or hydraulic drive auger machine, a multiple axis gearbox, two shaft joint bands, three auger shafts and auger heads. The multiple axis gear box distributes the torque from the auger machine to each auger shaft for drilling and mixing. It is capable of concentrating all drilling power to one single shaft for hard drilling. Each auger shaft is mounted with discontinuous auger flights and mixing paddles. The discontinuous auger flight is designed to provide some vertical displacement for soil mixing and to prevent the transporting of soil to the surface. The design of the auger flights and mixing paddles on each auger shaft varies with soil types. Three basic auger shafts are designed for mixing cohesive soils, sand, and gravel, respectively. Frequently, the auger design is tailored to meet specific project conditions. The auger flights and mixing paddles on adjoining shafts overlap at different elevations to produce overlapped soil-cement columns after mixing. The shaft joint bands maintain the space between auger shafts and adjoining shafts as a rigid body to accurately produce one overlapped column panel with a single stroke (Taki & Yang, 1991).

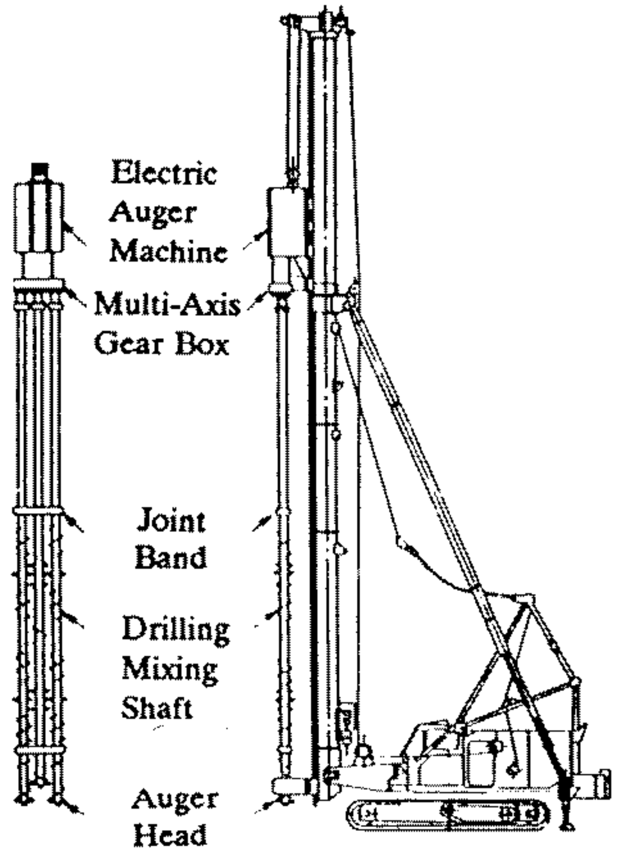

Figure 8.1:
Soil-mixing machine with multiple-axis auger (SEIKO-type) (Taki & Yang, 1991)

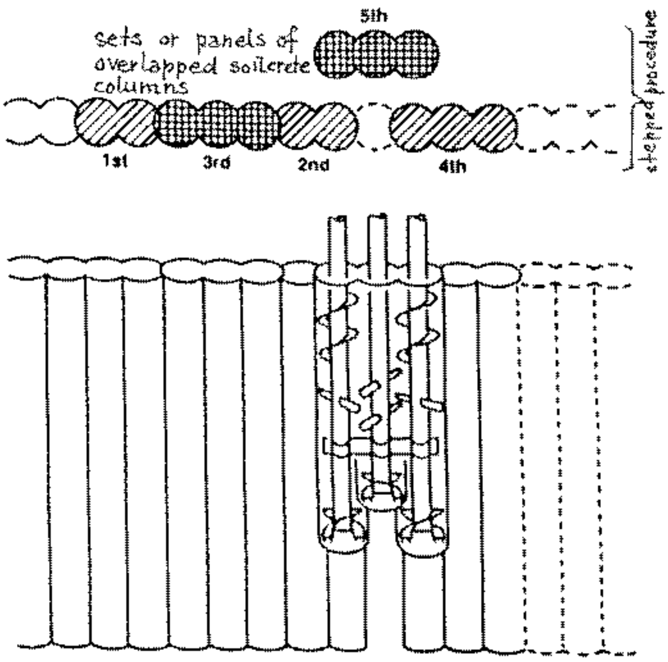

Figure 8.2:
Principle of wall construction by overlapped panels of soil-cement columns (three-axis auger) (Yang & Takeshima, 1994)

The *mixing plant* consists of a slurry mixer, slurry agitator, automatic batching scales, slurry pumps and a computer for mixing and slurry flow control. The automated batching system measures the water, cement and other additives by weight to produce more reliable slurry than that by volumetric batch system. The desired weight of each slurry component can be entered at a control panel and the mix design can be made by simply adjusting the component weights at the control panel. A separate positive displacement pump supplies the slurry to each of the injecting augers for accurate control of slurry flow.

8.3.2 Mix design

Mix design includes the design of slurry and the design of a soil-slurry mix ratio. Basically, the slurry consists of cement, bentonite, water, and usually some filler. Portland cement is the most commonly used hardening reagent for soil-cement mixed wall work. It is more advantageous than quick lime or slaked lime (Saitoh et al., 1985). Blast furnace cement may be a viable alternative because of its improved setting properties (Blast furnace cement is swelling while setting). A small amount of bentonite is usually added to increase the workability of the soil mixing work. Cement-based reagents or additives such as silicate, blast-furnace slag, and gypsum are used for gaining strength in saline or organic soils, or for stabilizing soils with pollutants. In non-aggressive surroundings fly ash or rock flour may be used as filler for economic reasons.

Mix design is governed by the required engineering properties of the soil-cement wall such as strength and permeability. The engineering properties obtainable are, in turn, affected by the type of soil to be treated. For design purposes, the in-situ soil can be mixed with various amounts of cement (or other reagents) with different water/cement (reagent) ratios to develop a spectrum of mix design that can provide the required engineering properties.

The German Geotechnical Society (DGGT) requires the determination of the following characteristics when designing the soil-slurry mix (Wilder et al., 1999):

- Composition and properties of the components of the mix
- Properties of the fresh slurry
- Workability and hardening properties of the mix
- Strength and stress-strain behavior of the hardened mix
- Permeability of the hardened mix
- Density and water content of the hardened mix

The final selection of a mix will also depend on the selection of equipment and installation operations for optimum overall quality, efficiency, and economy (Taki & Yang, 1991).

8.3.3 Installation operation

The installation operation includes control over the following quantities and activities:

- Installation location
- Verticality alignment
- Drilling speed
- Slurry flow

- Quality control through sampling and testing

The *installation location* influences several factors. The continuity of the soil element wall is dependent upon location control, verticality control and stepped installation procedures. Accurate installation location is maintained through the use of a template and survey.

The *vertical alignment* is constantly monitored by an electronic sensor in the lead with readout in the console of the base machine. The lead, together with its auger, is maintained at 1:500 (horizontal to vertical) during operation. The installed soil cement columns should not deviate more than two thirds of one percent from the vertical. This will virtually eliminate unmixed zones between column sets. The stepped procedure drills two primary column sets and then drills a second column set using two boundary columns of the primary column sets as guide holes to construct a continuous wall as shown in Fig. 8.2. The re-drilling in this procedure increases the uniformity of soil-cement. The re-drilling ratio can be adjusted according to the required level of soil mixing. A slightly different procedure was used in the construction of the cutoff wall in the Sylvenstein dam in Germany, as illustrated in Fig. 10 of Appendix 6 and described in the respective case history (Section A6.3).

The *drilling speed* is determined by the properties of soil to be treated and the soil mixing effort to obtain the design parameters of soil-cement.

The *slurry flow rate* has to be adjusted constantly to accommodate varying drill speeds in different soil strata to maintain the design volume of slurry per unit volume of in-situ soil. The slurry is injected both during penetration and withdrawal of the augers. To ensure a uniform distribution of the slurry mixed with the soil over the entire length of a column, the machine operator′s record should indicate the volume of slurry injected for each 0.5 m section of a column.

Quality control comprises the slurry, the soil-slurry mix in the ground and the continuity of the wall. For control of the slurry, the following quantities are checked regularly to ensure that no errors were committed in the mixing process: marsh funnel viscosity and density. The density influences the erosion resistance of the hardened mix. Occasionally, also the volume of filtrate and the sedimentation should be measured.

In colder climates with possible freezing conditions, the temperature of the slurry is also of importance. Hydration of the slurry at too low temperatures is retarded and must be avoided.

For quality control of the soil-cement columns, field wet samples must be obtained during construction for visual observation and laboratory testing. A deep sampler is used to retrieve a soil-cement bulk sample at designated depth inside the soil-cement column immediately after column installation. The test cylinders, usually 75 mm in diameter and 150 mm in height, are prepared using these bulk samples. Unconfined compression strength tests and permeability tests are performed for quality control and quality assurance purposes.

Quality control of continuity, which is most critical in cutoff wall applications, is ensured by the design of the equipment, namely:

- Columns of known diameter are fabricated by the rotating mixing shafts
- The shafts are physically linked together. This means that tangential or overlapping relationships between columns are maintained
- Continuity between adjacent sets of columns (or panels) is guaranteed by re-drilling the last adjacent column with the first column of the new panel

The specially designed soil-mixing machine enables therefore to produce a wall of high geometric precision.

8.4 ENGINEERING PROPERTIES OF SOIL-CEMENT

The following factors influence the engineering properties of soil-cement (the hardened soil-slurry mix):

- Soil type
- Amount of cement or other hardening reagents used
- Amount and type of bentonite added
- Water/cement ratio of the slurry
- Degree of soil-cement mixing
- Curing environment
- Age

Considering the application of the soil-cement wall, the most important engineering properties are: strength, permeability, and modules of elasticity.

8.4.1 Strength

The strength of soil-cement is usually obtained from laboratory unconfined compression tests, but also from triaxial compression and direct shear tests. Test samples include laboratory-produced specimens, field wet samples, core samples prepared before, during and after construction, respectively. For construction quality control the unconfined compression test is the most frequently used test.

The strength of the soil-cement is an indication of its erosion stability. Sufficient stability is usually achieved if the 28-day uniaxial strength has a value of at least 0.3 MPa (Wildner et al., 1999). On the other hand, a very high strength is not desirable because the hardened soil mix wall should have a modulus of deformation of similar magnitude as its surrounding soil, otherwise there is a danger of wall cracking when the soil deforms. (see the case history on Sylvenstein dam, section A6.3 of Appendix 6).

The most dominant factor influencing strength of soil-cement is the type of soil. This is attributed to the adsorption and pozzolanic reaction in the various soils as well as the reaction of the hardening reagent itself. Greater strength is achieved with soils that have a high pozzolanic activity (Saitoh et al., 1985). A pozzolanic activity is a chemical reaction of $Ca(OH)_2$, generated in the hydration of the cement, with any siliceous or aluminous substances in the presence of water to form compounds possessing cementing properties. Figure 8.3a shows the unconfined compressive strength of soil-cement walls installed in clayey, sandy, and gravelly soils obtained from dozens of soil-cement wall projects. It can be seen that increasing the cement dosage increases the strength of a particular soil type but this increase is much more pronounced with sandy and gravelly materials.

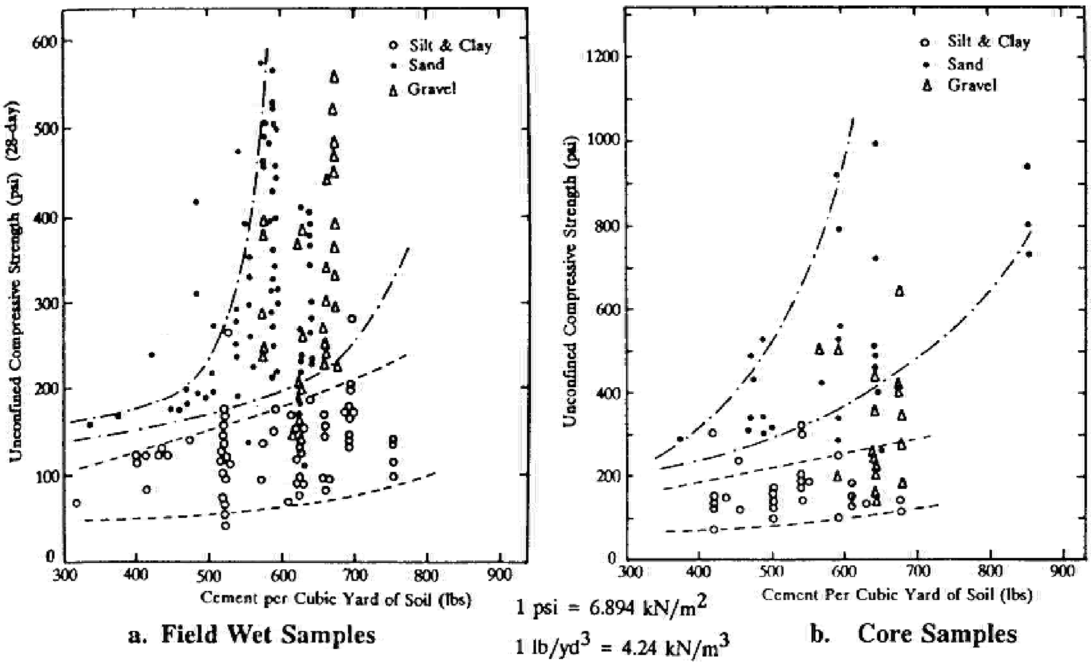

Figure 8.3: Strength of soil-cement as a function of cement dosage and type of soil: (a) field wet samples; (b) core samples from hardened soil-cement (Taki & Yang, 1991)

8.4.2 Hydraulic conductivity (permeability coefficient), k

Cement and bentonite dosage is used to control the permeability of the soil-cement walls. The k-value ranges from 10^{-7} to 10^{-9} m/s based on laboratory testing of field wet samples obtained during construction (Yang et al, 1993). For seepage control in a dam foundation cutoff, soil-cement walls with a coefficient of permeability in the order of 10^{-8} m/s are considered satisfactory. Addition of bentonite, or clay-bentonite slurry, is needed for mixing with the in-situ soil.

8.4.3 Modulus of elasticity

The ratio of the modulus of elasticity, E_{50}, and the unconfined compressive strength, E_{50}/q_u, varies between about 350 and 1000 (Saito et al., 1985) and is affected by the same factors that control the soil-cement strength. For cohesive soils with a sand content less than 10 to 15 %: $E_{50}/q_u \approx 400$ to 600. E_{50} is the secant modulus at ½ q_u. Tests at the University of Technology in Munich indicated that bentonite has the ability to decrease the deformation modulus. Values of E/q_u (at 28 days) < 100 could be achieved. Hence, the use of bentonite in controlling the stiffness of a wall should be studied in particular applications.

8.5 APPLICATION OF SOIL-CEMENT WALLS

8.5.1 Seepage barriers

The SMW method enables the construction of a variety of treatment patterns (Fig. 8.4), and therefore has found many applications, such as:

- Cutoff walls (in dam foundations and inside embankments, Yang et al., 1993)
- Extension of impervious core in dam heightening (Wildner et al., 1999)
- Excavation support walls
- Ground stabilization (soil strengthening in areas of soft soils)
- Liquefaction mitigation (Pujol-Rius et al., 1989)
- Environmental remediation (reduction of leachability of contaminated soil by mixing it with a suitable reagens).

In dam engineering only the application to cutoff structures (including extensions of impervious cores) and liquefaction mitigation are relevant and the description of the method here is limited to these applications.

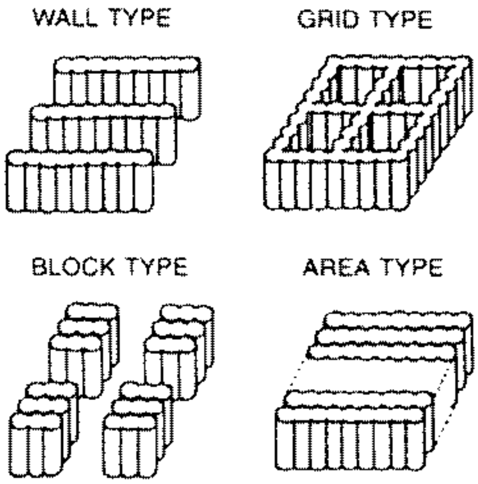

Figure 8.4:
Possible treatment patterns

For use as seepage barriers in dam foundations or in levees and embankments, cutoff walls constructed by the deep mixing technique are made of soil-cement. Such walls have an in-situ strength ranging from 100 to 2000 kPa and permeability from 10^{-8} to 10^{-9} m/s (Yang et al., 1993). In Japan, soil-cement cutoff walls were installed in porous strata or limestone to form a subsurface dam which can be used to contain groundwater as a subsurface reservoir for irrigation purposes (Subcommittee, 1997).

Examples of cutoffs for seepage control constructed by the deep mixing technique and reported in the literature are:

- Jackson Lake Dam, USA (Pujol-Rius et al., 1989; Ryan & Jasperse, 1989, Taki & Yang, 1991),

- Cushman spillway modification, USA (Sehgal, et al., 1992; Yang & Takeshima, 1994),
- Lockington Dam, USA (Walker, 1994).
- Sylvenstein dam, Germany (Wildner et al., 2002)

Details on the cutoffs at Jackson Lake dam, the spillway of the Cushman dam and the Sylvenstein dam are given in Appendix 6.

Soil-cement is considered a porous material from a microscopic point of view. Laboratory studies have revealed that both the coefficient of permeability and the percent of fine pores in the soil-cement decrease with diminishing sand content and water/cement ratio, and as well increase in curing age. The design and construction parameters that produce a higher percentage of fine pores tend to reduce the coefficient of permeability of soil-cement.

Basically, deep mixing is applicable to most soils that allow mixing with the equipment described above, such as:

- Soft or loose alluvial and marine soils
- Organic soils
- Reclamation fill

Typically, clay soils with SPT N-values ≤ 4 and granular soils with N-values of about 10 or less can be treated. Using heavier equipment with higher torque to rotate the shafts, soils with N = 10 to 30 can be treated (Toth, 1993).

The SMW method also presents advantages over the conventional diaphragm wall method in cutoff construction. Cutoff walls constructed by the deep mixing technique do not require any excavation, which may be an asset in environmentally sensitive areas. Since there is no open trench, sudden loss of slurry or collapse of the excavated trench is not possible. Also, under artesian conditions constructing a cutoff by the deep mixing presents no particular difficulties.

8.5.2 Liquefaction stabilization

The purpose of this application is not to produce a cutoff for seepage control but to stabilize the dam foundation by soil-cement walls such that it can resist shear deformations during strong earthquakes when part of the ground may liquefy. The same technique is employed as with the construction of a cutoff, however, the arrangement of the wall elements is different. Since cutoff construction and liquefaction stabilization may be employed in the same project it makes sense to briefly describe this application.

Soil-cement columns can be combined with wall, block or grid patterns (see Fig. 8.4). The soil below the dam foundation, reinforced by such elements, must be able to withstand the shear stresses from the embankment dam when the loose portions of the foundation soil liquefy. Shaking table tests have shown that the grid type pattern is most effective in reducing shear stress and excess pore pressures generated during ground shaking. Hence, liquefaction stabilization is usually achieved by constructing soil-cement grids in the dam foundation. For an existing dam this would mean that the dam over the section that needs stabilization has to be removed and later rebuilt.

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Appendix 1. CASE HISTORIES ON DIAPHRAGM WALLS

A1.1 General

The case histories presented in the following sections were selected to present different aspects of diaphragm wall design and construction. It does not mean, however, that the solution adopted for a particular foundation would have been the only way to treat it. But it can be argued that the designers and contractors have made every effort to approach the optimal solution as closely as possible given the available equipment and the financial means.

The case histories were written based on information available in the published technical literature. Unfortunately, the information content that can be obtained from published articles varies considerably and there are rarely case histories which report all the pertinent data of interest to the readers. Moreover, case histories often describe a cutoff wall project up to the end of construction and information on the performance of the cutoff (e.g. efficiency, deformation response to fill placement and reservoir impounding) is very rare. According to Powell & Morgenstern (1985) unsatisfactory behaviour of a cutoff reveals itself within about the first three years after impounding.

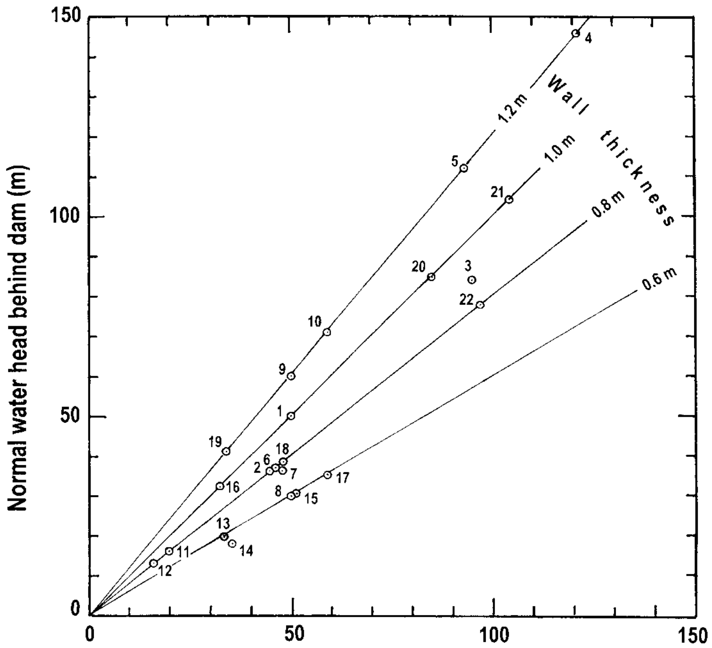
The eleven case histories presented in this appendix provide information on the following issues:

- Diaphragm wall construction with primary and secondary panels; excavation by clamshell and trench cutter (Dhauliganga, Brombach)
- Treatment of overhangs (Xiaolangdi, Péribonka)
- Diaphragm wall construction using hybrid method with primary and transverse panels (Xiaolangdi, Péribonka)
- Bentonite in panel joints (Arminou)
- Keying diaphragm wall into bedrock, milling performance (Brombach)
- Handling of large boulders in deep alluvium (Dhauliganga)
- Connection of diaphragm wall with core (Eastside reservoir)
- Connection of diaphragm wall to plinth of CFRD (Dhauliganga)
- Grout curtain below cutoff base (Arminou, Eastside reservoir, Péribonka)
- Field test with trial panels (Arminou, Brombach)
- Use of an inspection/drainage gallery (Brombach)
- Slurry trench cutoff installation in existing dam (Twin Buttes)
- Diaphragm wall used to seal abutment of reservoir (Cleveland)
- Mix design for plastic concrete (Eastside reservoir, Tadami)
- Stress-strain and strength properties of plastic concrete (Colbun, Convento Viejo, Tadami)
- Quality control
- Diaphragm wall analysis (Dhauliganga Eastside reservoir, Brombach)

Since no two projects involving diaphragm walls are identical, the dimensions of the wall, the hydraulic gradients across the wall and the properties of the materials composing the wall vary to best suit the particular situation of the dam site. However, it can be helpful to compare the design of a new project and the mix proportions established with those of completed projects. For this purpose Table 1, Table 2 and Table 3 have been prepared. The cells of these tables are not all filled since the available information found in the literature is incomplete. But the table should also be a help for presenting information of new projects.

Table 1 lists the dimensions of various diaphragm walls, penetration of the wall into the core of the dam, differential hydraulic heads, and maximum gradients across the wall. Table 2 and Table 3 present data on mix proportions of plastic concrete and stress-strain properties of plastic concrete tested under different conditions.

In Fig. 1 the seepage gradient through the cutoff wall is plotted versus the differential water head under normal operating conditions, taken as the height of water behind the dam, under normal operating conditions (Full supply level). The data include 22 dam projects with diaphragm wall thicknesses ranging from 0.5 (Razaza) to 1.2 m. Gradients vary from about 20 for older dams to about 100 for more recent projects. Xiaolangdi dam has the highest gradient because the panels are made of rigid concrete and only the transverse panels at the joints are cast of plastic concrete.



1	Dhauliganga (India)	12	Jebba (Nigeria)
2	Arminou (Cyprus)	13	Eberlast (Austria)
3	West Dam, East Side Reservoir (USA)	14	Razaza (Iraq)
4	Xiaolangdi (China)	15	Evretou (Cyprus)
5	Colbun (Chile)	16	Marib (Yemen)
6	Convento Viejo (Chile)	17	Lluest Wen & Withens Clough (UK)
7	Twin Buttes (USA)	18	Peñitas Mexico)
8	Brombach (Germany)	19	Verney (France)
9	Merowe (Sudan)	20	Pehuence (Chile)
10	Péribonky(Canada)	21	Potrerillos (Argentina)
11	Tadami (Japan)	22	Francisco Abellan (Spain)

Fig. 1 Relationship between seepage gradient across diaphragm wall and differential water head for various projects

Table 1 Diaphragm wall dimensions and hydraulic loads

Name of dam	Dam size		Cutoff wall dimensions				Penetration		Differential hydraulic head (m)	Max. gradient across wall	Remarks
	Height (m)	Crest length (m)	Max. depth (m)	Length (m)	Area (m^2)	Thickness (m)	into rock (m)	into core (m)			
Dhauliganga (India)	56	265	70	~200	7500	1.0			50	50	Concrete facing
Arminou (Cyprus)	42	200	13	38	400	0.8	>1.0	1.5	~36	45	
East Side (USA)	114	2530	40	735	14,875	0.9	0.6-3.0	4.6	~84	95	
Xiaolangdi (China)	154	1700	~70	~450		1.2		14	146	121	Hybrid wall, only transverse panels are of plastic concrete
Colbun (Chile)	116	1935	68	335	12,800	1.20			~112	93	
Convento Viejo (Chile)	38	730	55	540	16,412	0.8			~37	~46	
Cleveland (Canada)	91		23	306		0.8					
Twin Buttes (USA)	40.8	13,200	~30	6440	130,000	0.76	>0.75		36.6	48	
Brombach (Germany)	37.3	1700	35	~1700	40,000	0.6		Inspection gallery	~30	50	
Tadami (Japan)	18	582.5	23	220		0.8	1.5	1.5	16	20	
Merowe (Sudan)	63	885	63	397	15,310	1.2	~4.0	42	~60	50	
Péribonka Main Dam (Canada)	80	775	116	310	12,000	1.2/1.5	>0.5	6.0	71	59	
Verney (France)	41.5		50		13,000	1.2	1.0	-	41	34	Asphalt facing

Table 2 Plastic concrete for diaphragm walls – I: Mix proportions and strength properties

Note: CF = Cement factor = (W_{cement} +$W_{bentonite}$); BC = bentonite factor = ($W_{bentonite}$/CF)x100 %; w/c = W_{water}/CF; W = Weight, (s) = specified

Name of dam	Mix proportions									Max. particle size (mm)	UC strength 28-days (MPa)	Strain at failure (UC) (%)	Strain at failure (triaxial) (%)
	Cement (kg/m^3)	Bentonite or clay (kg/m^3)	Fine aggregate (kg/m^3)	Coarse aggregate (kg/m^3)	Water (kg/m^3)	CF	BC (%)	w/c	Aggregate ratio				
Dhauliganga (India)	150	20	817	813	318	170	12	1.87	1.0		1.5± 0.5(s)		5-7
Arminou (Cyprus)	162	23	854	741	348	185	12	1.88	1.15	32	1.195	0.73	
East Side (USA)	178	11.9	605	931	344	190	6	1.81	0.65	12.7	1.7-3.0		
Xiaolangdi (China)	150	100	1567		305	250	40	1.22	-		>2 (s)		
Colbun (Chile)	75	140 (incl.121 clay)	1483		423	215	65	1.97	1.0		1.982 (90 days)	0.84	4.15
Convento Viejo (Chile)	80-84	100 (incl. 92 clay)	1670-1740		300-320	184	54	1.74	1.0		0.43	1.18	1.5-5.0
Cleveland (Canada)	143.6	36.6	707	707	416	180	20	2.31	1.0		1.5(s) 1.05		5(s) 10
Twin Buttes (USA)	Soil-Cement-Bentonite mix									38	0.69(s)		
Brombach (Germany)	200	100 (clay)	950	450	360	300	33	1.2	2.1				
Tadami (Japan)	125	25	792	928	279	150	1.7	1.86	0.85		2.5		
Merowe (Sudan)	140	35	872	580	391	175	20	2.23	1.5	19	>0.7(s)	-	>3.5(s)
Francisco Abellan (Spain)	150	40	1170	700	372						1.26±0.38		
Verney (France)	47	117 (clay)	995	663	313	164	71	1.9	1.9		0.6-0.7 (90 days)		
Balderhead (UK)	195	44	1400		400	237	18	1.7	1.0				7.9
Lluest Wen (UK)	227	24	1300		405	251	9.6	1.6	-				1.6
Withens Clough (UK)	61+300 Flyash	25	1050		409	386	6.5	1.1	-				9.1
New Wadell (USA)	183+79 Flyash	10.5	823	908	210	273	3.8	0.8	0.9				

Table 3 Plastic concrete for diaphragm walls – II: Deformation, permeability and workability properties

Name of dam	Triaxial strength		Modulus				Permeability		Slump	Remarks
	Cohesion, c (kPa)	Friction angle, φ (degrees)	Initial tangent (UC) (MPa)	Initial tangent (triax) (MPa)	Secant (UC) (MPa)	Secant (triax) (MPa)	from laboratory k (m/s)	From in situ wall test k (m/s)	(cm)	
Dhauliganga (India)			200-400(s)						20(s)	
Arminou (Cyprus)	17 (total stress)	43 (total stress)	250(s) 280 (mean)		164 (at failure)		$<10^{-8}$(s) 2.4×10^{-9} ($i=50$, $\sigma_3=100$ kPa)	10^{-7} (mean) 4.4×10^{-7} (max)		
East Side (USA)								$<5 \times 10^{-9}$(s) 8.2×10^{-10}-4.9×10^{-9}	19.1	
Xiaolangdi (China)										
Colbun (Chile)			294	630	281-383 75%$(\sigma_1-\sigma_3)_f$	275 75%$(\sigma_1-\sigma_3)_f$			17.8-22.9	
Convento Viejo (Chile)	490 (effec-tive stress)	36 (effect-tive stress)	77	196-353			10^{-8}(s) 3.1×10^{-6}	0.4×10^{-8}-10^{-7}	19	Drained triaxial tests
Cleveland (Canada)							10^{-9}(s) (in confined test 1.7×10^{-9})	1-3×10^{-8}		
Twin Buttes (USA)								$<10^{-8}$(s)	18-25(s)	
Brombach (Germany)										
Tadami (Japan)	387	35			1500	360				
Balderhead (UK)					54 (at 1% strain)		$(0.6$-$2.0) \times 10^{-9}$			Wall through core
Lluest Wen (UK)							10^{-5} – 10^{-6}			Wall through core
Withens Clough (UK)					25 (at 1% strain)					Wall through core

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A1.2 Dhauliganga Dam (India)

A1.2.1 Project description

The 280 MW Dhauliganga run-off river hydro-electric scheme is located in Pithoragarh District in the NE corner of Uttranchal Province in India. The Dhauliganga (river) is a tributary of the Kali river which forms the border between India and Nepal. Storage is provided by a 56 m high concrete face rockfill dam (CFRD) with a crest length of 265 m and a volume of 980,000 m^3. The project was commissioned early 2006. A cross section of the embankment is shown in Fig. 1. A detailed description of the various components of the CFRD can be found in Sen et al. (2005).

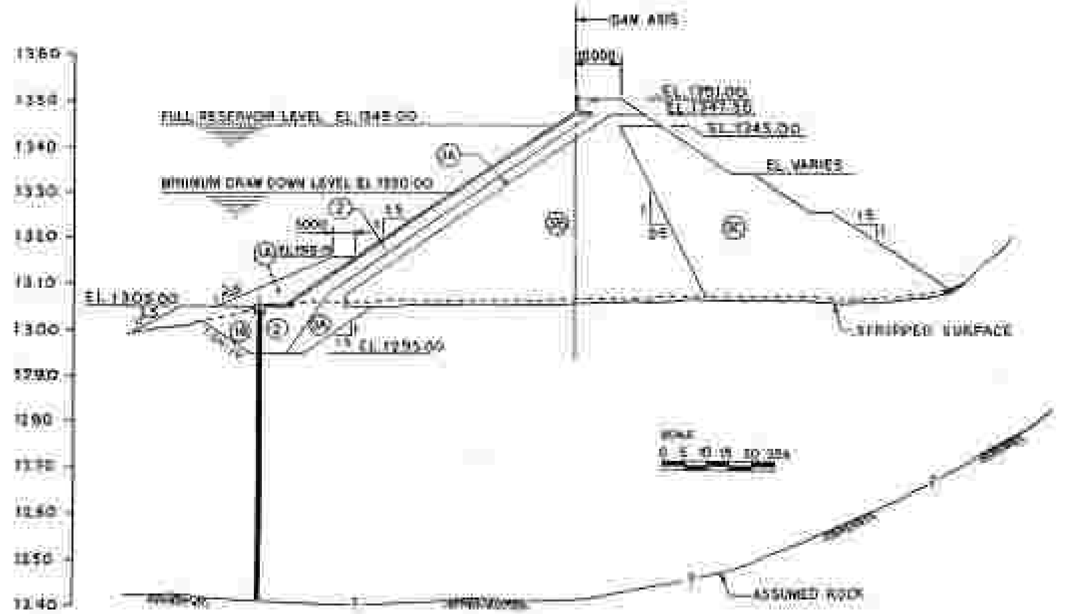

Fig. 1 Dhauliganga CFRD: Typical cross section

A1.2.2 Foundation conditions

The dam site is characterized by highly jointed biotite and augen gneisses. The rockline forms an approximately 70 m deep though filled with river alluvium (see **Fig. 2**). Rock is exposed on both flanks of the approximately 250 m wide valley. The alluvium is stratified with an upper layer of pebbles and boulders in a sandy, gravelly matrix, followed by an up to 20 m thick sand lens, which is underlain again by sand/gravel with cobbles and boulders. The maximum diameter of the boulders in the alluvium was observed to reach as much as 2 to 3 m.

Standard penetration tests in the alluvium yielded N-values which were in general above 25. The permeability of the alluvium was assessed to range between 10^{-4} and 10^{-5} m/s. Waterpressure tests in the bedrock resulted in Lugeon values of >20 in the upper 10 to 15 m, but for greater depth there was a rapid decrease to <1 Lugeon.

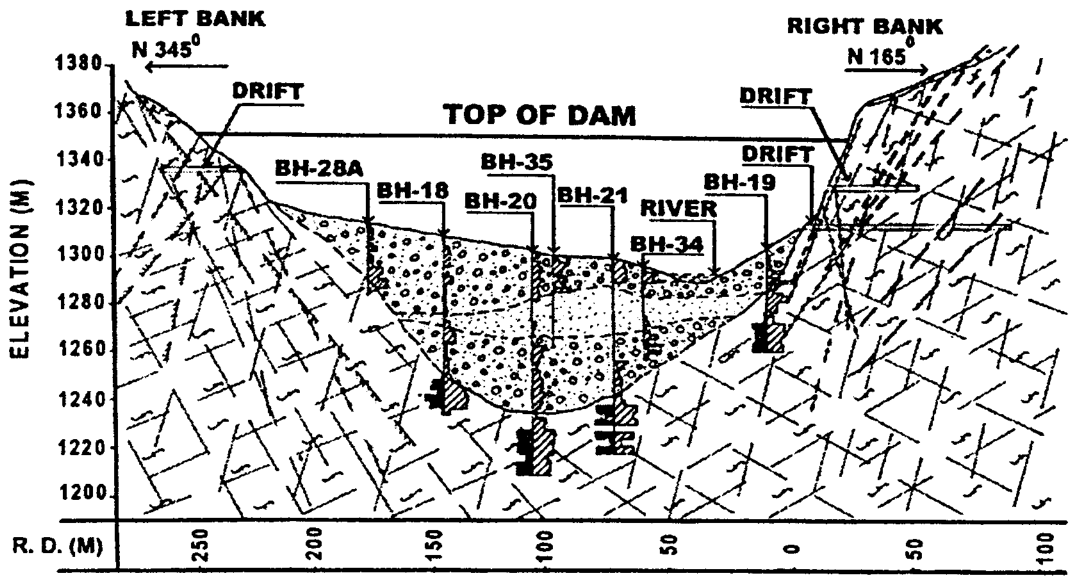

Fig. 2 Dhauliganga HEP: Valley section at dam site

A1.2.3 Foundation treatment

Seepage control has been accomplished by a plastic concrete cutoff wall, 1.0 m thick, about 200 m long, and of 7500 m^2 in total area, keyed into the bedrock. The maximum depth of this wall is in excess of 70 m. On the abutments, beyond and above the cutoff wall, the bedrock is treated by a grout curtain. Since the wall is keyed into the bedrock, it was judged that there was no need for a grout curtain below the wall, although grouting had originally been specified in the tender.

Pore pressure cells were installed in the alluvium on both the upstream and the downstream side of the wall in order to check the efficiency of the wall after impounding.

A1.2.4 Cutoff wall construction

The wall was constructed as a series of primary and secondary panels. Excavation was to be carried out mainly by cable-hung hydraulic grab, supported by various types of chisels and by a trench cutter. However, it turned out that grab and chisel were found ineffective in penetrating the ground for most of the time and the trench cutter became the primary excavation equipment. The use of the cutter was particularly essential for keying the wall into the bedrock.

The trench cutter was a Bauer BC40 mounted on a BS 6100 carrier. The chisels used were: a 14 ton cross chisel, a 13 ton box chisel and an 8 ton cut chisel. The hydraulic grabs were of the type DHG 1000x2800 and DHG 1000x2100. The cutter removed the soil or the rock chips from the bottom of the trench and mixes these materials with the bentonite slurry in the trench. The slurry charged with soil is then pumped through a ring main of hose pipes to a de-sanding plant (BE 500 and BE 110), where the slurry is cleaned, regenerated and returned to the trench. More details on operational procedures are given in Brunner (2004).

Excavation proceeded with two single bites of 2.80 m in length, which were then backfilled with the plastic concrete. The 2.20 m long soil column left between two primary bites of 2.80 m was then excavated by cutting away 0.30 m of plastic concrete on each of the adjacent bites, reducing their length to 2.50 m. The total length of the primary panel was then 7.80 m, as illustrated in Fig. 3. The next primary panel was then excavated and cast at a distance of 2.20 m from the end of the primary panel. After construction of some of the primary panels, the secondary panels were excavated, each in one 2.80 m long bite, shaving off 0.30 m from the adjacent primary panels.

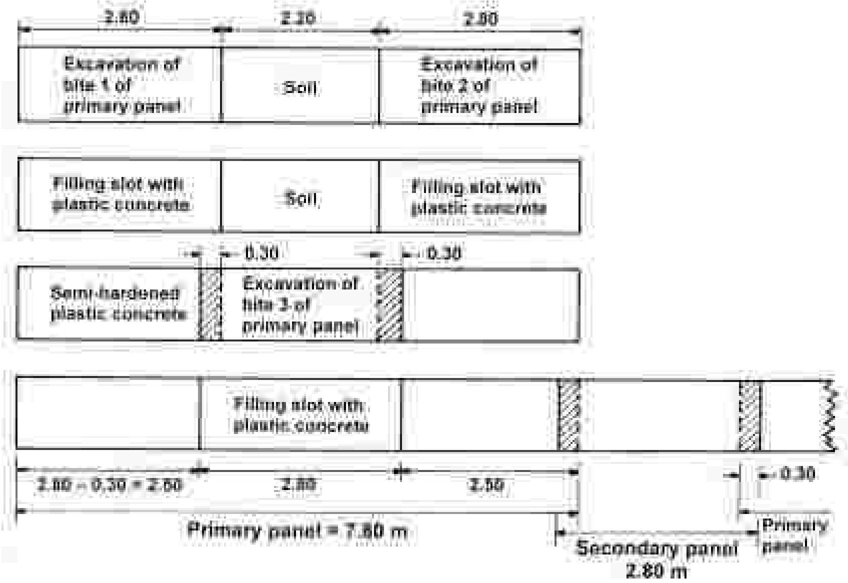

Fig. 3 Method of wall construction with primary and secondary panels

Trench stability was provided by bentonite slurry which had been mixed in a separate mixing plant and then was kept in storage tanks. Sudden slurry loss in the trench was experienced in some of the panels. These events were controlled by back-filling the partly excavated panel with sand and lean concrete and re-excavating it.

The plastic concrete mix was specified to have the following proportions:

- Unconfined compressive strength 1.5 ± 0.5 MPs
- Confined (triaxial) compressive strength 2.5 ± 0.5 MPa
- Elastic modulus, E 200 – 400 MPa
- Strain at failure (confined test) 5 to 7 %

The actual testing program revealed that it was difficult to satisfy all these requirements, especially to obtain a failure strain of ≥ 5 %. The following mix proportions were finally selected:

Cement	150 kg/m^3
Bentonite	20 kg/m^3
Water	318 kg/m^3
Sand	817 kg/m^3
Coarse aggregate	813 kg/m^3

The slump was specified as 200 mm and the maximum aggregate size was limited to 20 mm. The bentonite, Bombay Mineral, was indigenously produced.

A1.2.5 Connection of wall with impervious element of embankment

The impervious elements in a CFRD are the face slabs and the plinth. The reinforced concrete guide walls were not dismantled and a wall cap was placed over the guide walls. This reinforced concrete wall head is connected to the plinth, as shown in Fig. 4. The plinth consists of three independent slabs hinged among each other through impervious elements (water stops, etc.), conceived in such a way that considerable relative movements are possible without rupturing the seal.

In order to minimize the relative movement between the wall and the plinth, a trench was excavated down to groundwater level which was then back-filled by semi-pervious soil compacted to the maximum possible density.

A1.2.6 Analyses performed

Deformation analyses:

A static deformation analysis was carried out prior to the construction of the wall to estimate the following quantities (Visvanathan, 1999; Malla & Wieland, 2005):

- Deformation of the rockfill dam and the alluvial foundation below the plinth
- Deformation of the cutoff wall
- Slip displacement along the upstream and downstream interface between cutoff wall and alluvium
- Axial force, bending moment and vertical stress in the cutoff wall

The finite element numerical model employed assumed the water table to be at the ground surface on the downstream side of the wall. The loads applied were gravity and water load (hydrostatic pressure). The latter was assumed to vary linearly from the top of the wall to zero at its tip. The computer code ADINA was used for analyzing an elastic cutoff wall to which three different elastic moduli were assigned, namely $E = 0.3$, 2.0 and 20 GPa, simulating plastic, intermediate and conventional (rigid) concrete. The moduli estimated for the upper alluvium, the sand lens and the alluvium below the lens were 50, 25, and 75 MPa, respectively. From back-analyses carried out at the end of construction, the average modulus of the alluvium was estimated as about 100 MPa (Sen et al., 2005).

The analysis considered two different modes of deformation, namely:

- without slip between cutoff wall and alluvium (i.e. wall is assumed to be rigidly attached to the surrounding alluvium)
- with slip between cutoff wall and alluvium

For the analysis with slip, a contact algorithm, available in ADINA (1996) was used, which assumes that on the contact surface between wall and alluvium only compressive forces can be transferred. If tensile forces occur, separation takes place. This algorithm, however, makes the analysis non-linear. The coefficient of friction between wall and alluvium was assumed as 0.7.

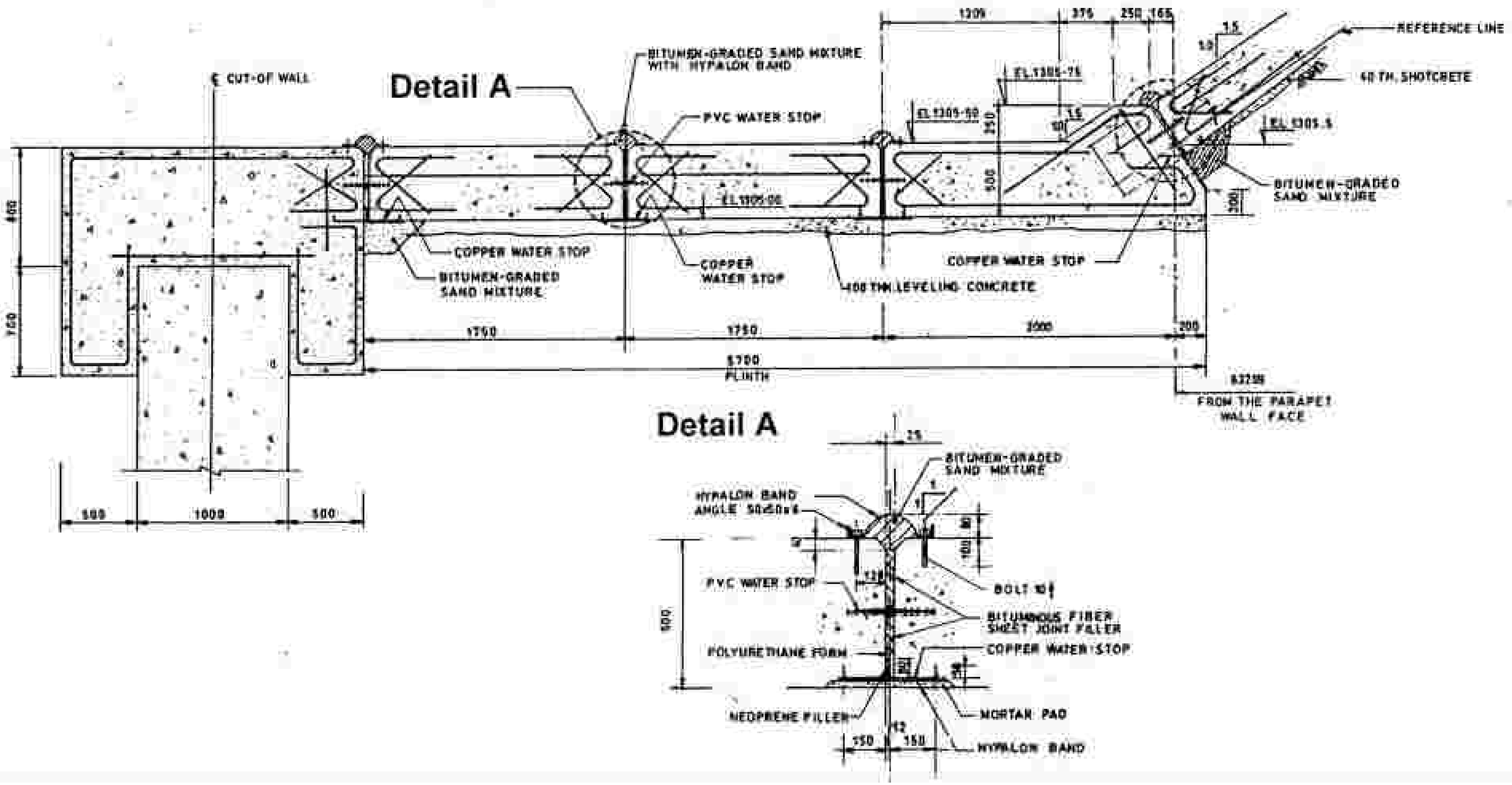

Fig. 4 Connection of cutoff wall to plinth of concrete face rockfill dam

The purpose of introducing slip into the deformation analysis was mainly to estimate the slip on the downstream side of the wall. This slip is relevant for the relative movement of the joint between wall and the plinth slab. If this movement is excessive the joint will rupture. For cutoff walls connected to a central impervious core there are no joints with a sealing function and slip is not considered a relevant issue.

The analysis yielded the following results relevant for the performance of the cutoff wall:

- The stiffness of the wall material has a great influence on the deformation and load distribution behavior of the wall structure.
- With a wall made of conventional (or normal) concrete, the slip is about 32cm on the downstream side, as shown in Fig. 5. Such a wall is very stiff compared to the alluvium and its deformation cannot follow that of the alluvium on the downstream side. On the other hand, with plastic concrete, the slip occurs mainly on the upstream side, while on the downstream side it is on the order of about 1 cm, i.e. the wall practically adheres to the alluvium. It becomes also evident that a small movement of the alluvium relative to the wall will not compromise the integrity of the joint between wall and plinth.

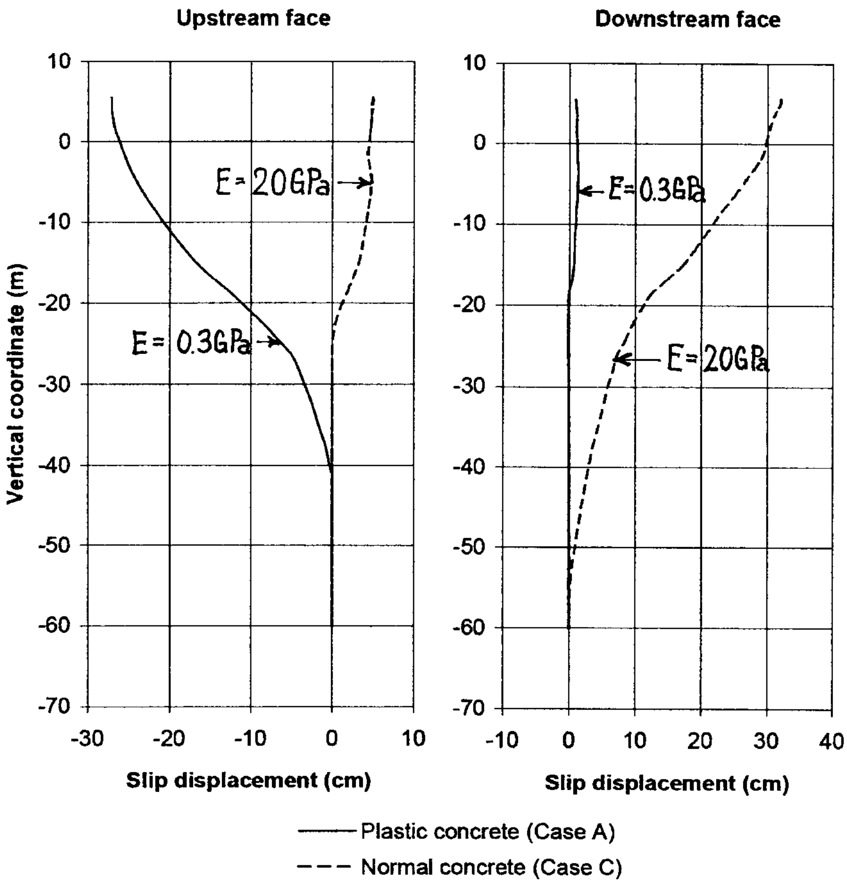

Fig. 5 Slip displacements along upstream and downstream faces for wall of different stiffnesses

Fig. 6 compares axial force and bending moment for the plastic and normal concrete. Much higher values of axial force and bending moment occur in the stiff wall of normal concrete. Slip decreases the axial force in the rigid wall but increases it in the plastic concrete wall. This is because less load can be transferred to the alluvium on its upstream side when slipping takes place. Slip has, however little influence on the bending moment. For the rigid wall, slip reduces load transfer to the wall on the downstream side.

- The linear elastic analysis produced a maximum compressive strain of 0.8 % for the plastic concrete. Under such conditions, the plastic concrete is expected to yield. Therefore, an additional analysis was performed where the plastic concrte was modeled as a perfectly plastic solid with E = 0.3 GPa and with unconfined compressive strengths of 1 and 2 MPa. With a 2 MPa compressive strength the maximum axial compressive stress and strain became 2.4 MPa and 0.89 % respectively.

Seepage analysis:

A two-dimensional seepage analysis using the SEEP/W finite element code (developed by GEOSLOPE International Ltd) was performed to investigate the influence of the depth of the wall on seepage flow. The results showed that a fully penetrating cutoff wall is desirable, but that a partial cutoff is still a viable option, because:

- Seepage losses were found to be small and within acceptable limits
- The prevailing gradients, although high are not expected to cause internal erosion at the toe of the wall due to the fact that the alluvial soils immediately above and below the sand lens are well graded and that at their boundaries with the lens the filter criteria are satisfied.
- The rather rapid deposition of sediments in this river regime will form a natural sealing blanket over the area of the plinth and cutoff wall and reduce the hydraulic gradient.

A1.2.7 Performance of the cutoff wall

No data are available regarding the performance of the wall after completion of impounding

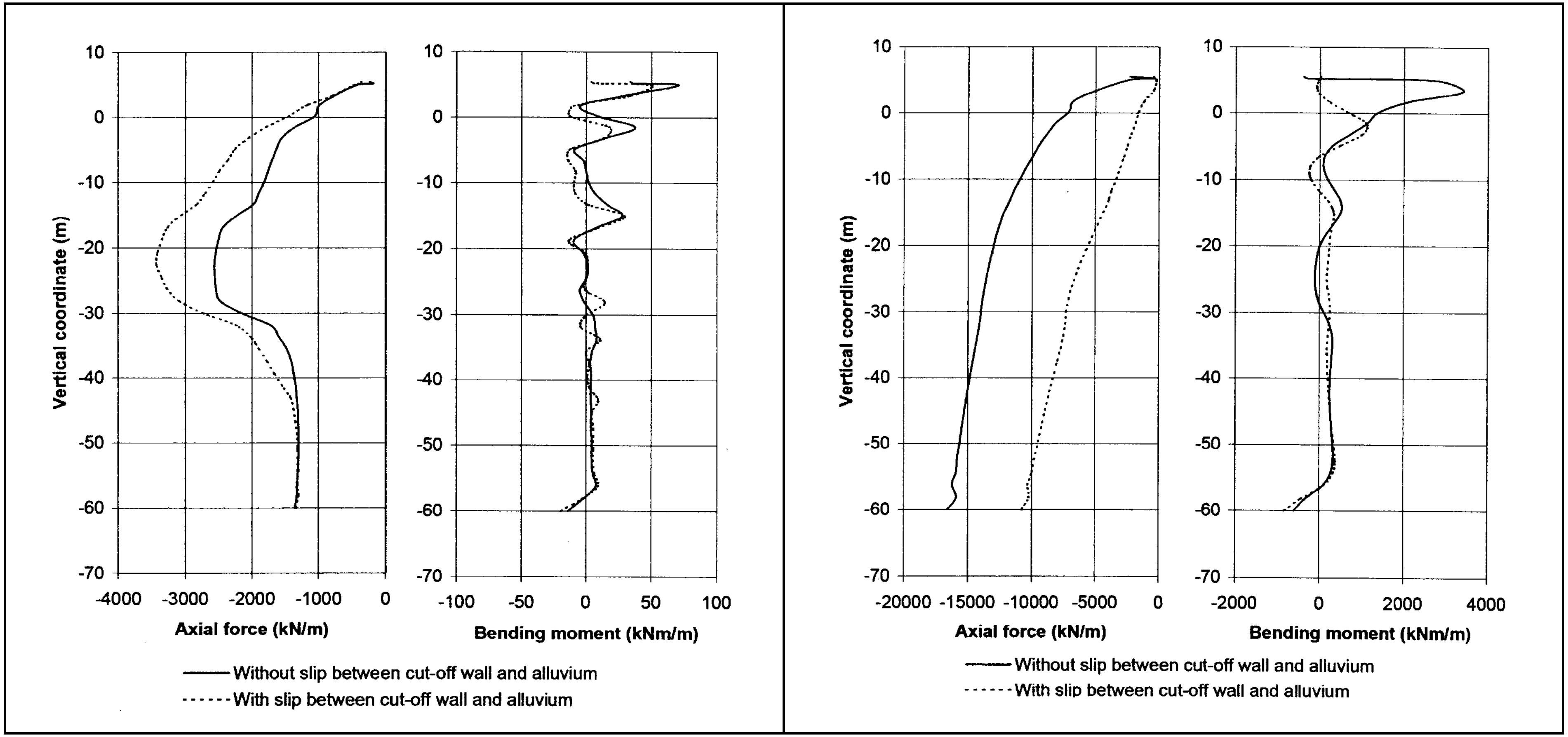

Fig. 6 Comparison of axial force and bending moment for plastic and rigid concrete cutoff wall with and without considering slip

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A1.3 Arminou Dam (Cyprus)

A1.3.1 Project description

Arminou dam is a 42 m high zoned embankment with a central clay core and a crest length of 200 m. It is located in the west of Cyprus on the Dhiarizos River and its main use is for irrigation and flood storage. The dam was constructed during the years 1995 to 1998 with first impounding from December 1998 to March 1999. A cross section of the embankment is shown in Fig. 1 and a longitudinal section in Fig. 2. Descriptions of the embankment, foundation conditions and seepage control are described by Antoniou et al (2000) and by Brown & Bruggemann (2002).

A1.3.2 Foundation conditions

At the dam site, the valley is covered with talus on the slopes and alluvial deposits in the river channel, underlain by pillow lava of the Upper Trodos series. On the left side of the valley floor the rock line forms a 15 m deep and about 40 m wide buried channel with steep side walls (see Fig. 2). The alluvial deposits consist essentially of silty sandy gravel with many cobbles and boulders varying significantly both laterally and with depth.

The groundwater level in the alluvium prior to construction of the dam varied seasonally being above the river bed level in the wet season and about 3 m below during the dry period in summer.

A1.3.3 Foundation treatment

Two options were considered in the tender to cut off flow through the buried channel filled with alluvium, namely: (i) a diaphragm wall of maximum 13 m depth and (2) excavation of the alluvium and founding the core of the dam on bedrock. The cost of the first alternative turned out to be about 60 % of the second.

The diaphragm wall conceived was a 0.8 m thick plastic concrete structure, maximum depth 13 m, length 38 m with a total area of 400 m^2. In addition, there was contact and curtain grouting of the rock below the wall and also a two-row grout curtain in the alluvium downstream of the cutoff wall.

A1.3.4 Wall construction and other sealing measures

Trial panels:

Prior to the construction of the actual wall, three trial panels, each 2.5 m long, were formed across the side of the buried channel, about 20 m upstream of the permanent cutoff wall. Subsequently, this wall section was excavated on both sides to 3.5 m depth and the top 1 m of the panels was removed. Visual inspection revealed the presence of a layer of bentonite filled into the joints of the panels across the full width of the wall section, with a thickness varying from 10 mm at the top of the wall to about 5 mm at one meter depth. The bentonite layer (or cake) adhering to the outer sides of the panels was about 20 mm thick and not varying with depth. However, the overall face roughness was greater than the thickness of the cake, which means that the interface remained sufficiently rough to ensure effective stress transfer from the alluvium to the wall.

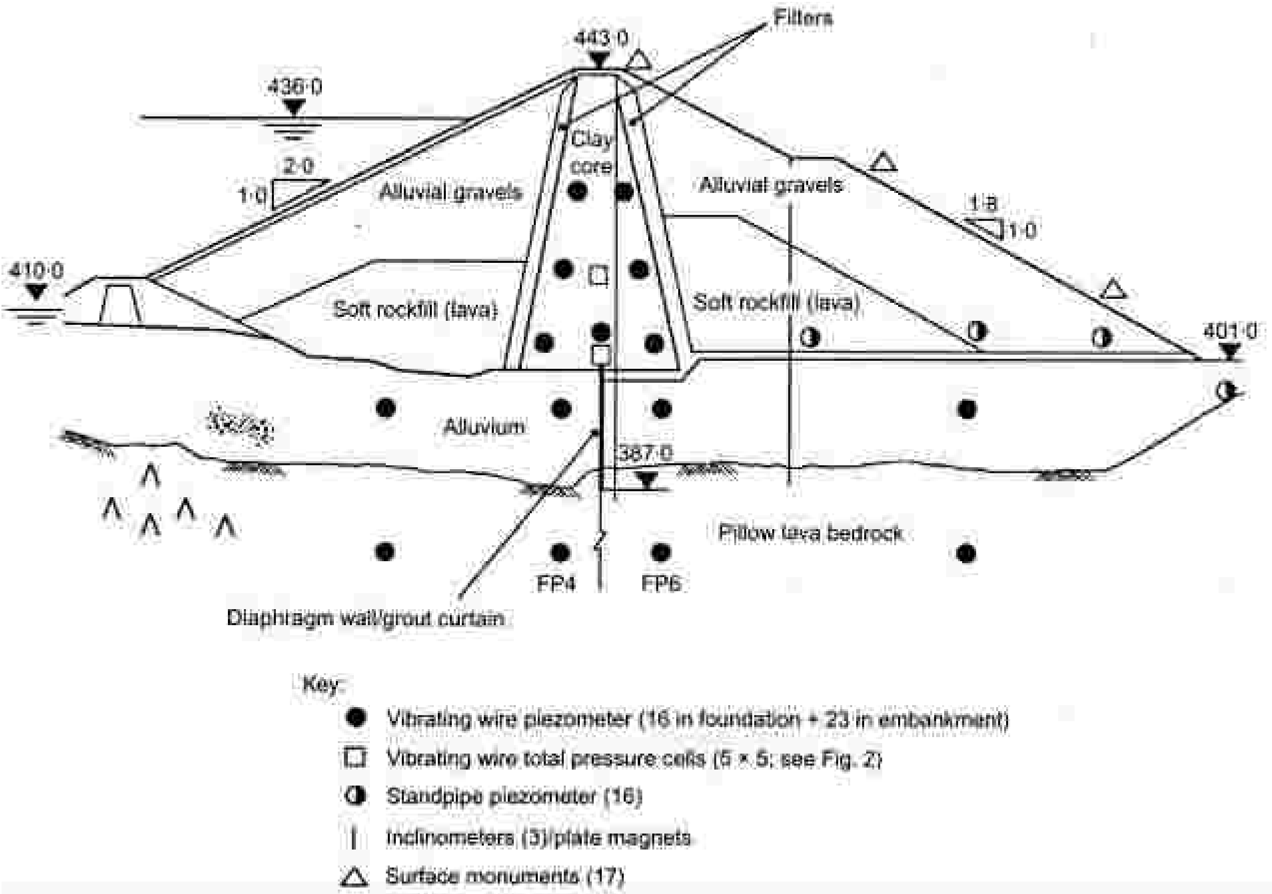

Fig. 1 Arminou dam: Embankment cross section

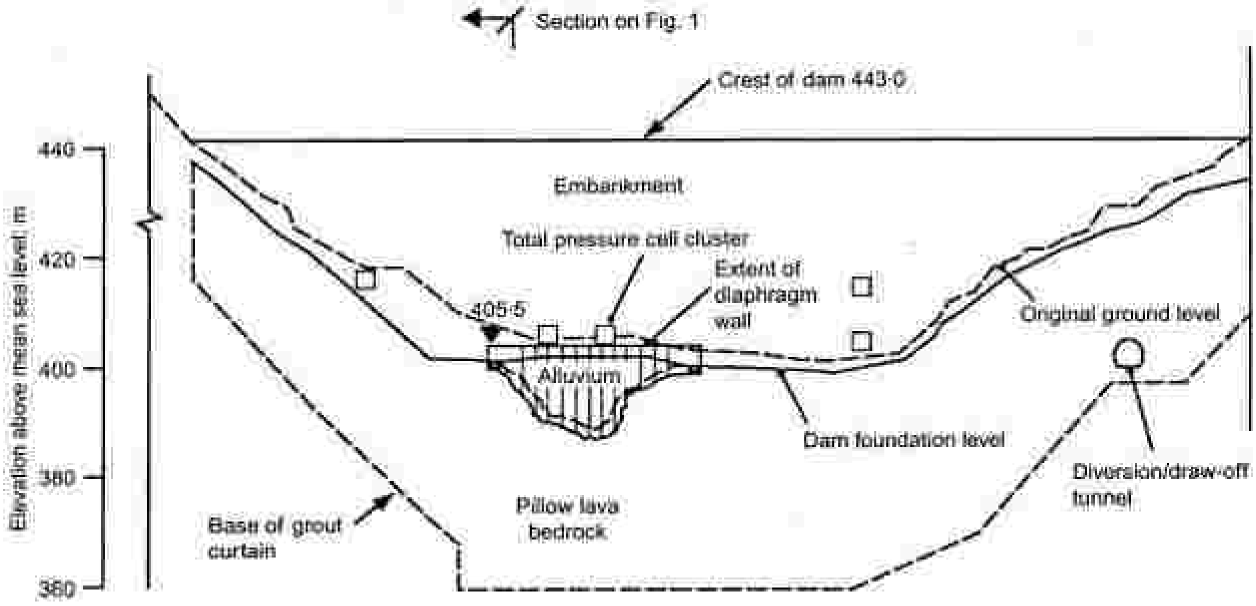

Fig. 2 Arminou dam: Longitudinal section along dam axis looking downstream

Construction of permanent wall:

The wall was constructed with the panel method. Excavation was by crane-operated grab and chisel. Primary and secondary panels were both 2.5 m long. Stop-end pipes with a central triangular section were used to form the ends of the primary panels. They were removed two days after panel construction. Prior to casting the secondary panel, the joints were cleaned by means of a triangular tool. Because of the steep sides of the buried channel, panel construction started from the middle of the channel. The plastic concrete was placed using a single tremie pipe positioned in the middle of the panel. The bottom of the tremie pipe was maintained at 3 to 4 m below the surface of the plastic concrete.

The wall was constructed from a platform 2.0 m above the base of the core and keyed into the bedrock by at least 1 m along the flat part and by 0.5 m (measured vertically to the rock surface) into the steep sides of the of the channel.

The supporting fluid in the excavation was produced from local engineering grade bentonite at 5 % by weight to water. This bentonite had a liquid limit of 410 %. The fluid, after cleaning and immediately before concrete placement had a mean Marsh viscosity of 41 s, a bulk density of 10.3 kN/m^3, and a sand content of 0.5 %.

Material properties:

Specifications for the plastic concrete mix were given for samples set in a mold as follows:

- Permeability (hydraulic conductivity, k) at 28 days under a hydraulic gradient of 50 and a confining pressure of 100 kPa: $k \leq 1x10^{-8}$ m/s
- Initial tangent modulus in unconfined compression test: $E_i \geq 250$ MPa

The mix proportions used in the permanent diaphragm wall were:

- 162 kg/m^3 of Ordinary Portland cement
- 23 kg/m^3 of bentonite
- 854 kg/m^3 of 10 mm crushed aggregate
- 741 kg/m^3 of 0-5 mm sand
- 348 liter/m^3 of water

From the plastic concrete mix placed into the wall samples were also cast in molds for strength and permeability tests. The results obtained are the following:

Mean values from a set of 12 samples:

- 7-day unconfined compressive strength: 522 kPa
- Strain at failure 0.74 %
- Secant modulus at failure 71 MPa

Mean values from a set of 24 samples:

- 28-day unconfined compressive strength 1195 kPa
- Strain at failure 0.73 %
- Secant modulus at failure 164 MPa
- Mean initial tangent modulus 280±310 MPa

The bulk density varied between 20.4 and 21.8 kN/m^3 with a mean of 21.5 kN/m^3. The mean value of permeability measured in a 100 mm triaxial cell under a hydraulic gradient of 50 and a cell pressure of 100 kPa was $2.4x10^{-9}$ m/s.

Two consolidated undrained triaxial tests, without pore water pressure measurement, were also carried out on 100 mm diameter specimens of plastic concrete with a cement content of 135 kg/m^3. The cell pressures were 100 and 200 kPa. The results obtained were: Deviator stress at failure 1240 and 1680 kPa, which indicates strength parameters in terms of total stresses of $c_u = 170$ kPa and $\varphi_u = 43°$.

Ten in-situ permeability tests were carried out in boreholes drilled into the trial panels. The measurement section included the full length of the uncased part of the borehole. The mean value obtained was $k = 1x10^{-7}$ m/s with a maximum of $4.4x10^{-7}$ m/s.

The core recovery from the boreholes drilled into the trial panels was poor although the age of the plastic concrete was between 28 and 45 days. The reason for the core loss is believed to be erosion by the flushing water. This erosion effect produced boreholes with a variable diameter.

Contact grouting of rock socket

The rock below the cutoff wall was grouted through the cutoff wall by means of tubes cast with the wall at 1.5 m centers. These tubes were also used for grouting the contact between the base o f the diaphragm wall and the rock. The packer was set at 0.3 m above the base of the wall and the top 1.5 m of the rock and the contact zone grouted with a pressure limited to 1.5 bars in order to avoid damage to the wall. Higher grout takes were observed where prolonged chiseling for embedment of the wall had been necessary.

Alluvial grouting

The bentonite infill observed in the joints of the trial panels raised the possibility of internal erosion of the bentonite under operation conditions. If the bentonite would be eroded the seepage through the joints would increase. End of casing constant head permeability tests were carried out in four boreholes in the alluvium downstream of the diaphragm wall. The mean and median permeabilities from 19 tests were $6x10^{-4}$ and $6x10^{-5}$ m/s respectively with a maximum value of $k = 5x10^{-3}$ m/s. Subsequently, sleeve pipe grouting was performed in two of these boreholes. The sleeves were spaced 0.5 m center and injected with water at a pressure of 0.2 bar/m. An ordinary PC grout with a water/cement ratio of 2:1 by weight and with 4 % bentonite (by weight of cement) was then used for trial grouting. The absorption rate was up to 109 liters in 10 min. The permeability, k, decreased substantially.

Based on these test results it was decided to install two lines of alluvial grouting downstream of the diaphragm wall, mainly to infill the coarser zones of the alluvium and prevent the erosion of bentonite from the panel joints into such zones. For the production grouting, the bentonite content was increased to 7 % and grout holes were spaced at 1.4 m centers on an equilateral triangular pattern and taken to a depth of 0.5 m below the base of the alluvium. The overall grout take was 5.8 % of the volume of ground treated. The median k-value after grouting was $3x10^{-7}$ m/s from ten constant head tests with a maximum value of $6x10^{-7}$ m/s.

A1.3.5 Connection of wall to impervious core

The wall was constructed from a 0.5 m high fill platform founded 1.5 m above the base of the core. This platform and the upper 0.5 m were later trimmed to leave the wall penetrating 1.5 m into the core.

A1.3.6 Analyses performed

Seepage

The quantity of seepage was inferred from effective mass permeability of the alluvium, which was obtained from pumping tests as 1.2×10^{-3} m/s. The gradient of the river bed was taken as 1 % and the cross-sectional area downstream of the dam to river bed level as 500 m^2. The inferred total flow to saturate the river bed was then estimated to be 5 to 10 liters/s.

Fig. 3 shows the response of foundation piezometers (all of the vibrating wire type) in rock and in the alluvium. The effect of the wall in decreasing the hydraulic head in the rock on the downstream side, according to the flow net concept, is evident. The piezometric readings in the alluvium are average values because of some imprecision in the tip elevation which occurred when the tips moved during withdrawal of the borehole casing. The total embankment and foundation seepage flow was estimated from mass permeability, gradients and cross sections to be 5 liters/s.

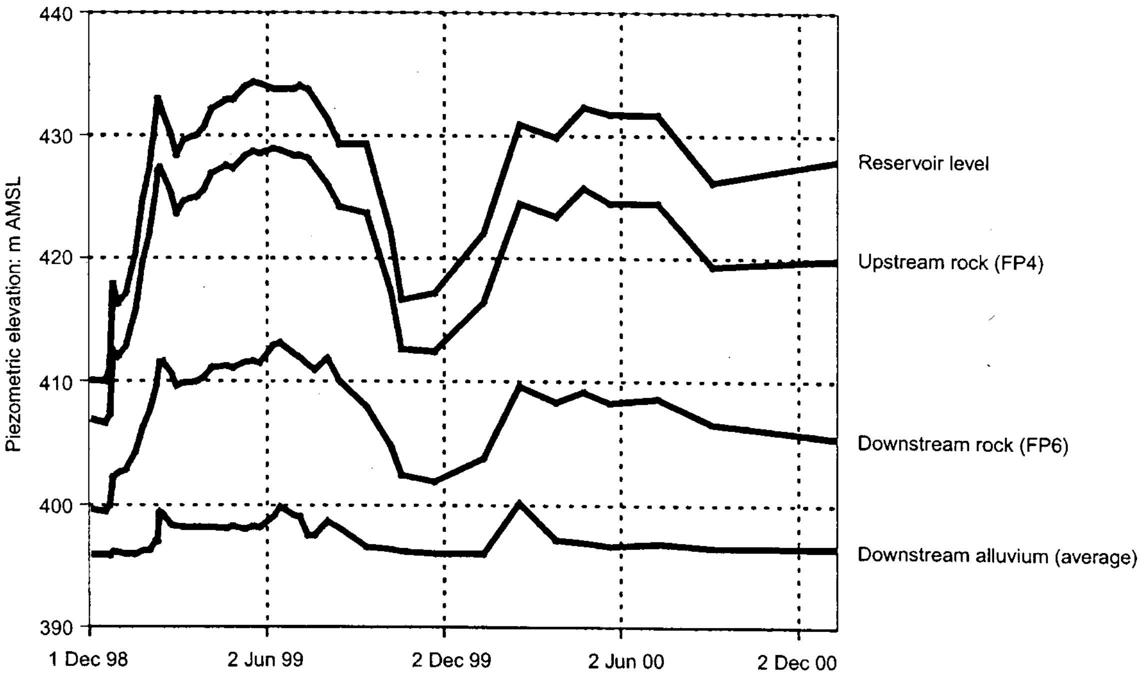

Fig. 3 Variation of piezometric levels in foundation with time

The efficiency of the diaphragm wall can be calculated from:

$$E_c = (q - q_c)/q$$

where q is the seepage flow with no cutoff wall and q_c is the seepage flow with cutoff.

The total seepage with no wall was estimated as about 300 liters/s. Assuming now that the majority of the inferred foundation seepage is through the diaphragm wall, the efficiency would be about 98%.

Deformations:

There were no instruments in the cutoff wall, but the settlement of the alluvium due to the construction of the embankment was measured by plate magnets along two inclinometer tubes (see Fig. 1 for the location of these tubes).The top of the alluvium under the core settled 20 mm. If it is assumed that the rock is incompressible, the vertical stiffness of the alluvium would be around 250 to 300 MPa, i.e. similar to that of the plastic concrete used for the wall construction.

Stresses:

The development of total stress was monitored by five clusters of total pressure cells (see Fig. 1 and Fig. 2). Two of these clusters were installed on the centerline of the core at El. 406 m asl, i.e. 1.5 m above the top of the diaphragm wall. It is assumed that about the same stresses are acting on the top of the cutoff wall. Each cluster consisted of five cells arranged in such a way that one cell measured the horizontal cross-valley stress, σ_{cv}, while the other four cells were arranged in pairs so that in each pair the cells were othogonal to each other. One pair measured the vertical stress, σ_v, and the horizontal stress in upstream/downstream direction, σ_{ud}, while the other pair measured stresses at ±45° to the vertical. These total stresses are plotted in Fig. 4, together with the overburden pressure, σ_o. Also plotted in Fig. 4 are the measured pore water pressures and the hydrostatic pressure.

The cross-valley stress in the cutoff wall plotted in Fig. 4 represents the best estimate resulting from a sensitivity study using three different approaches with ranges of Poisson's ratio, ν, and coefficient of earth pressure at rest, K_o, between 0.25-0.35 and 0.29 -0.55, respectively.

Fig. 4 suggests that with the measured vertical stress, σ_v, in the allucium smaller than the overburden stress, some arching of the core on the shoulders of the dam taking place. This will reduce the vertical load applied to the top of the wall. However, if the stiffness of the wall is significantly higher than that of the alluvium, additional vertical stress will be transferred to the wall through negative skin friction. Also, with the cross-valley stress, σ_{cv}, smaller than the hydrostatic pressure, there is a probability of hydraulic fracturing. The cross-valley stress is considered to be the smallest total stress. Hydraulic fracture in the plastic concrete wall is, however, not likely, because the material has some tensile strength. But the bentonite infill in the panel joints is believed to have nearly zero tensile strength and therefore could develop cracks in the direction orthogonal to the direction of σ_{cv}.The water pressure acting on the bentonite could to some extent reduce the risk of hydraulic fracturing.

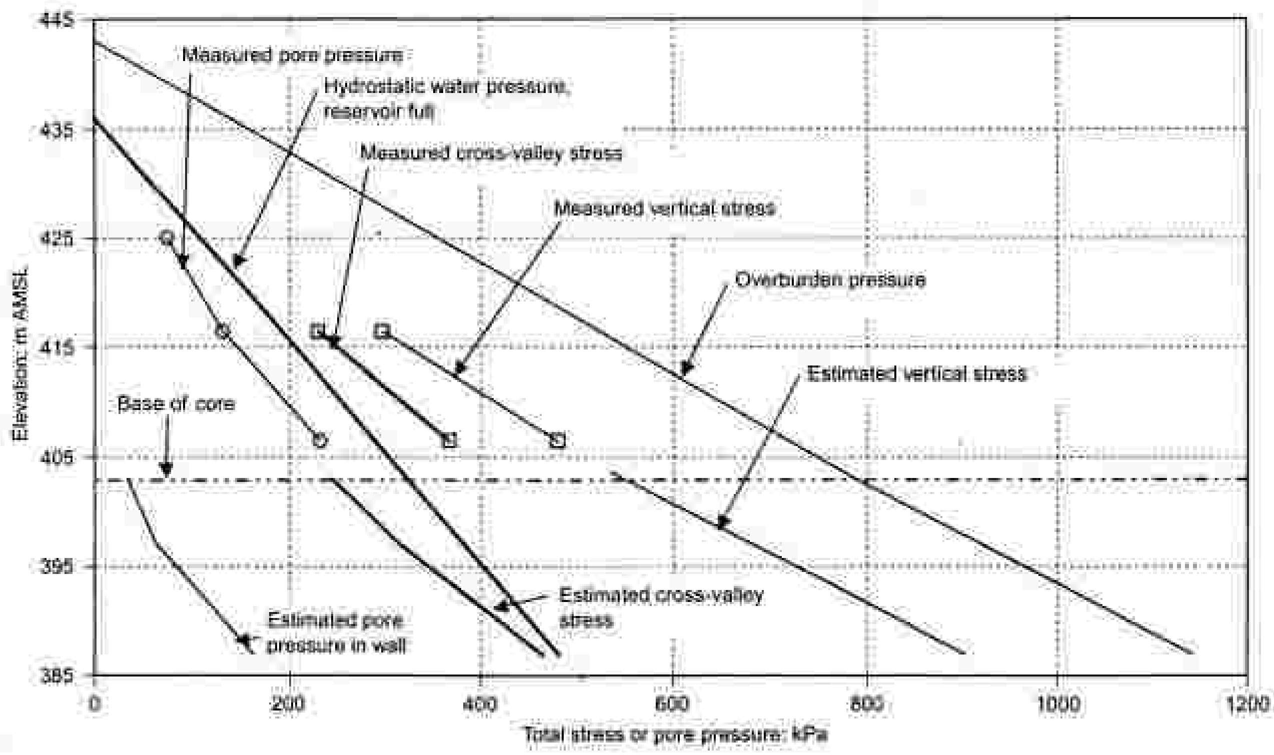

Fig. 4 Measured total stresses and pore water pressures in core and alluvium and inferred stresses in diaphragm wall (Brown & Bruggemann, 2002)

The designer considered it necessary to take steps to mitigate the consequences of possible erosion of the bentonite from the construction joints because the downstream alluvium has a high permeability. The simplest way was to construct grout curtain in the alluvium immediately downstream of the diaphragm wall, such that the bentonite infill would not be eroded.

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A1.4 West Dam of Eastside Reservoir Project (USA)

A1.4.1 Project description

The Eastside Reservoir project, located in Riverside County in southern California, has the purpose to help the Metropolitan Water District of Southern California (MWD) to meet seasonal, draught, and emergency needs. It comprises three earth-core rockfill dams, which were completed in December 1999 (see Fig. 1). The West dam blocks the west end of the Domenigini Valley, the East dam the east end of the Diamond Valleys, and s saddle dam closes a gap at the lowest point on the ridge line of the northern rim of the reservoir. West dam is the largest and highest one of these three dams with a height of 87 m, a crest length of 2530 m and a volume of $50x10^6$ m^3.A cross section of the embankment is shown in Fig. 2.

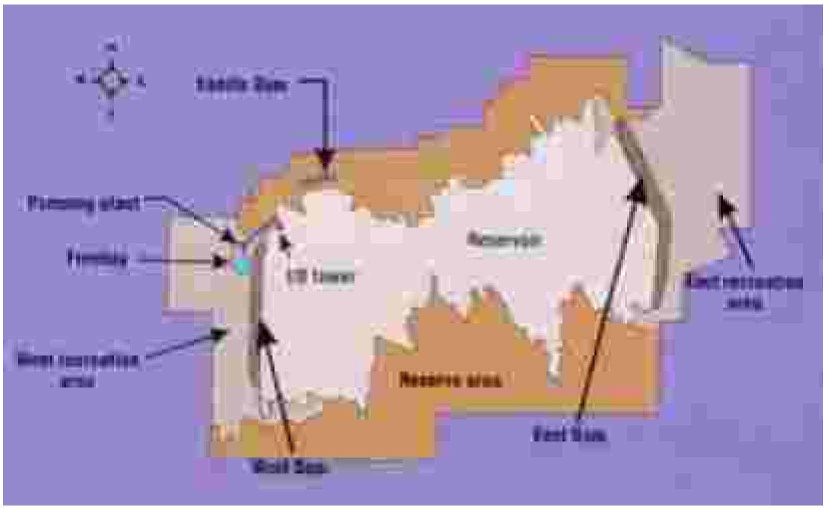
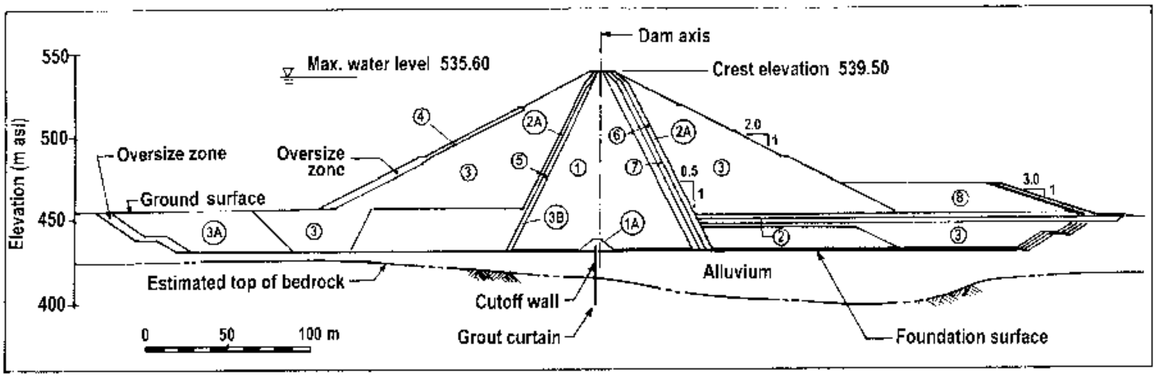


Fig. 1 The three dams of the Eastside project



1	Core	3A	Stripping rock shell	6	Downstream filter
1A	Cushion around top of cutoff wall	3A'	Shell	7	Drain
2	Blanket drain	3B	Fine filter	8A	Random fill
2A	Coarse filter	4	Upstream slope protection		
3	Shell	5	Upstream filter		

Fig. 2 Eastside reservoir, West Dam: Cross section

A1.4.2 Foundation conditions

The flat alluvial valley floor spanning between the abutment ridges nearly 2400 m wide is underlain by an irregular bedrock surface with depth exceeding 55 m. There are three deep buried alluvial channels with a maximum depth of about 40 m below foundation level which traverse the axis of West Dam in a roughly east-west direction. These three channels comprise an area which is about one third of the total footprint of the dam. The other two thirds of the dams foundation area are placed on quartzite and phyllite rock. The dense alluvial material filling t buried channels consists of silty and clayey sands, gravels, cobbles and boulders of up to about 1.20 m in diameter. Pressuremeter tests in the alluvial foundation soil resulted in modulus values ranging between 100 and 630 MPa. However, foundation excavation advanced to 27 m below the original ground surface to remove potentially liquefiable soils.

The pervious zones in the alluvium were identified from field permeability tests in boreholes and from pumping tests. Measured values of the hydraulic conductivity, k, vary widely from about $1x10^{-7}$ to $1x10^{-4}$ m/s. Based on these values, the construction of a cutoff through the alluvium and keyed into the bedrock was considered necessary. The bedrock consists primarily of quartzite with lesser amounts of phyllite.

The project lies in an area of high seismicity and a significant design consideration was the effects from seismic loading. The San Jacinto Fault with a seismic capability of M=7.5 is about 10 km away and the San Andreas Fault capable of generating an earthquake of M=8.0 led to an estimation of peak ground acceleration at the site in the range from 0.4 to 0.7 g.

A1.4.3 Foundation treatment

Design considerations and properties specified:

For the type of cutoff wall, the following alternatives were considered: (i) a soil-bentonite slurry cutoff, (ii) a cement-bentonite slurry cutoff, (iii) a core trench down to the bedrock backfilled with compacted core material, and (iv) a plastic concrete cutoff. Alternatives (i) and (ii) were ruled out because the wall material would have been softer, i.e. more compressible than the surrounding alluvium which could have led to arching of the core onto the shoulders and possibly hydraulic fracturing. With alternative (iii) the backfill would also be more compressible than the alluvium with consequences of differential settlement and subsequent cracking. Hence, alternative (iv) was selected, which enables the design of a plastic concrete with properties matching the surrounding materials.

There are three separate sections of diaphragm cutoff wall corresponding to the three deep channels crossing the foundation with a total length of 735 m. The centerline of these cutoff walls is located 3 m upstream of the dam axis and is contiguous with the upstream grout curtain in the bedrock to provide a continuous grout barrier in rock beneath the cutoff. The thickness of the diaphragm walls is 910 mm (3ft) and the maximum depth is 39.6 m with at least 0.6 m into bedrock. The total area of the walls is 14,864 m^2.

The necessary hydraulic conductivity of the cutoff was determined on the basis of literature studies and seepage analyses as $k = 5x10^{-9}$ m/s. The modulus of the plastic concrete was specified to be similar to that of the surrounding alluvial soils. Compressive strength was used as a guide to the appropriate modulus values. A range of 1.4 to 2.8 MPa

(200 to 400 psi) was considered to be suitable, but the moduli were also verified by triaxial compression testing. Finally, the slump of the plastic concrete was specified to be in the range of 17.8 and 22.9 cm to ensure a workable mix for tremie placement.

To reduce seepage in the rock foundation additional treatment was provided by grouting of the rock below the cutoff walls and in the abutments. Originally, it was planned to install grout pipes within the cutoff wall after the excavated panel had been backfilled. However, it turned out that holding these pipes securely in place would be difficult during placement of the plastic concrete and the idea was abandoned. Instead, a line of boreholes was drilled and grouted 1 m upstream of the cutoff centerline. These holes were cased in the alluvium prior to drilling and grouting in the underlying rock.

Consolidation grouting was performed over the entire width of the core and a two-row grout curtain was installed to a depth of approximately 38 m The grout rows were angled at 30 degrees opposing directions to maximize the intersection of rock discontinuities. Consolidation grouting extended to 15 m depth on each side of the grout curtain rows and reached 9 m for the remainder of the core foundation. Consolidation grout holes were spaced at 6 m centers except on the abutments, where spacing was reduced to 3 m in the inner half of the core foundation.

Finally, a reinforced concrete gallery was constructed beneath the core zone using cut-and-cover techniques. Two levels of 3m wide and 3.7 m high adits were driven into the rock to control leakage through the West dam’s north abutment. The upper adit, located at the crest of the dam, was used for grouting, while the lower adit, 46 m below the crest was used for both grouting and drainage.

Mix design:

In order to obtain a plastic concrete with the desired properties, five mixes were prepared and the components varied by trial and error to observe their effect on strength and permeability. Unconfined compressive strength and permeability tests were performed on all the mixes. On selected mixes, some isotropically consolidated drained triaxial tests were also carried for the purpose to evaluate the initial tangent modulus.

The proportions used in the five trial mixes are listed in Table 1, together with the strength and permeability characteristics.

The materials used for these mixes can be described as follows:

Cement:	Type I-II, according to ASTM C150
Bentonite:	Sodium bentonite meeting the requirements of API (American Petroleum Institute), Specification 13A: "Specification for Drilling Fluid Materials"
Coarse aggregate:	Quartzite, maximum size (12.7 mm)
Fine aggregate:	silty sand from an on-site alluvial borrow area with 46% non-plastic to plastic fines (<74µm), and plasticity index PI < 5
Water:	from a surface water source because groundwater was considered too heavily mineralized

Table 1 Proportions of materials used in trial mixes during design phase

Mix Number	C/A (%)	B/W (%)	B/C (%)	C/W	Coarse aggregate (kg/m^3)	Fine aggregate (kg/m^3)	Cement (kg/m^3)	Bentonite (kg/m^3)	Water (kg/m^3)	Slump (cm)	Bentonite in slurry (%)	Avg. compressive strength @ 28 days (MPa)	Avg. Hydraulic conductivity (m/s)
I	8.5	5.2	22.0	0.24	835	537	118	25.9	500	22.9	6.0	0.3	-
II	8.5	3.2	7.5	0.42	1031	689	146	10.9	345	20.3	5.0	2.0	$4.4x10^{-9}$
III	16.0	4.6	10.2	0.45	768	513	205	20.9	450	24.1	5.0	1.5	$2.9x10^{-9}$
IVA	8.2	2.3	8.0	0.29	960	551	123	9.9	426	21.6	5.5	-	$3.3x10^{-8}$
IVB	10.0	2.6	7.5	0.35	876	584	146	11.0	419	20.3	5.5	1.0	$5.8x10^{-9}$

Table 2 Proportions of materials used in mixes during construction phase

Mix number	C/A (%)	B/W (%)	B/C (%)	C/W	Coarse aggregate (kg/m3)	Fine aggregate (kg/m^3)	Cement (kg/m^3)	Bentonite (kg/m^3)	Water (kg/m^3)	Slump (cm)	Bentonite in slurry (%)	Range of compressive strength @ 28 days (MPa)	Range of hydraulic conductivity (m/s)
C-1	8.5	3.2	7.5	0.42	975	651	138	10.3	326	17.8-19.1	5.0	1.28-1.65 @ 56days	$8.2x10^{-10}$ to $4.9x10^{-9}$
C-2	11.6	3.5	6.7	0.52	931	605	178	11.9	344	19.1	5.0	1.72-3.03 @ 28 days	

Note: C = Weight of cement
B = Weight of Bentonite
W = Weight of water
A = Weight of aggregate (coarse + fine fraction)

A1.4.4 Wall construction

For construction of the wall, a platform made of core material (silty and clayey sandy soil) was constructed (see Fig. 3). It raises 5.5 m above the alluvial foundation surface. The Zone 1A material surrounding the cutoff wall consist of low plastic material with a maximum grain size of 75 mm, 20 to 80 % passing #200 sieve and a minimum plasticity index, PI, of 5. Concrete guide walls, 0.30 m wide and 0.91 m high were then constructed in a slot excavated in the platform. The distance between the inner walls of these guide walls was 0.98 m. The guide walls were removed after completion of cutoff construction.

Prior to platform construction, however, it was considered necessary to core into bedrock by at least 3m to establish the true rock line and to avoid taking a larger boulder for bedrock when excavating for the cutoff. Drillholes were spaced 12 m.

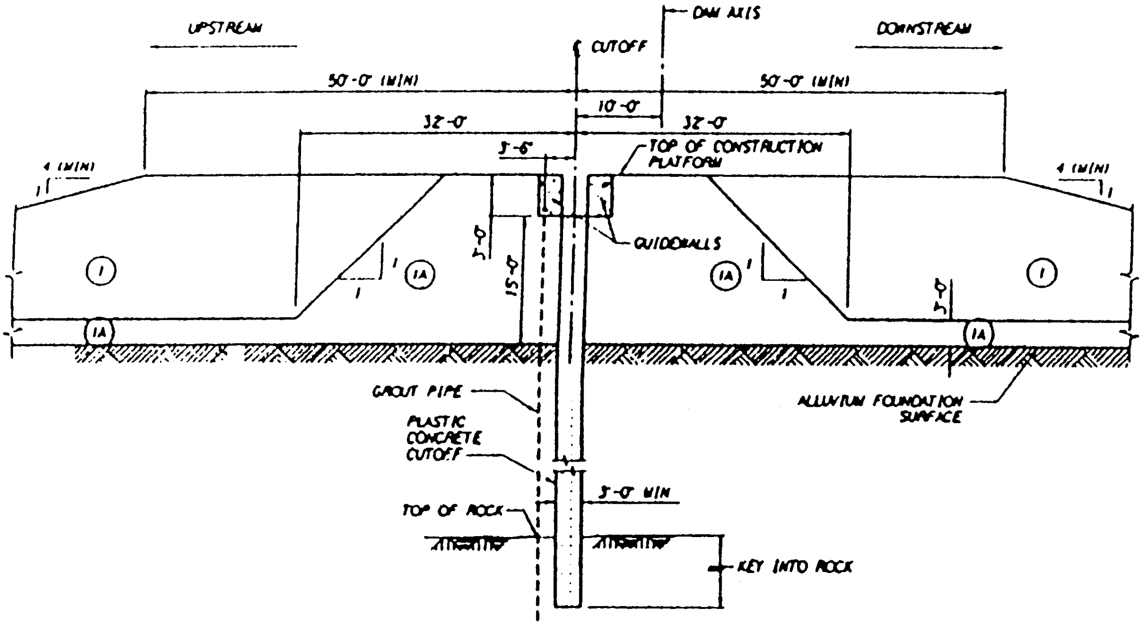


Fig. 3 Details of cutoff wall, and connection with core of dam

Excavation of the trench for construction of the diaphragm wall was performed using in a complementary way a hydraulic clamshell + chisel and a trench cutter (type Bauer). The cutter was able to dig to a maximum depth of 41 m. Clamshell and chisel were used mainly in situations where large boulders made excavation with the cutter difficult. The hydraulic jaws of the clamshell were capable of closing with a force of 1 MN.

The excavation proceeded following the method of primary and secondary panels with a total panel length of 7.0 m. The primary panel consisted of two 2.80 long bites excavated first, and a middle wedge, also 2.80 m long but overlapping the first two bites by 0.70 m. The secondary panels, also 2.80 m long were excavated after all the primary panels had been completed. The overlap was 0.15 m. Excavation of a panel started with the trench cutter until large boulders were encountered. Then the clamshell with chisel came into action. Finally, the cutter was used again to key the wall into the rock. The key ranged from 0.6 to 3m into the rock.

Coarse and fine aggregate were batched in dry condition in an on-site batching plant. To produce the plastic concrete mixture, first the hydrated bentonite slurry (5% solution by weight of water) was introduced into the concrete mixer truck. Then the mixture of aggregate and cement was filled into the truck and all the components mixed together.

Before placement of the plastic concrete into an excavated panel slot, the bottom of the excavation was cleaned with the trench cutter. Placement of the concrete was by the tremie method, using 250 mm diameter steel tremie pipes spaced no more than 3 m apart and embedded at least 3 m below the level of the concrete backfill during placement.

According to the specifications, the Contractor had to provide the mix for the wall. So, initially, the concrete mix used had the composition of Mix II developed during the design phase and listed in **Table 1**. The results were satisfactory with the 28-day strength exceeding the minimum of 1380 MPa (200 psi) and the k-value was 3.1×10^{-10} m/s. which is less than the specified limit. However, after about three weeks of concrete placement the strength of the concrete decreased to below the specified value and it was decided to augment the cement content of the mix; the C/W value was increased from 0.42 to 0.52 (see **Table 2**). All but three panels reached the 28-day compressive strength of 1.38 MPa. One of these panels was removed with the clamshell and new concrete backfill was placed.

A1.4.5 Connection of the wall to the impervious element

The cutoff wall penetrates into Zone 1A of the core (see Fig. 2 and Fig. 3), which consists of clayey sand and sandy silt. The amount of penetration was based on balancing the seepage that could occur over the top of the cutoff with that which could occur through it. Using Darcy's law the penetration was computed by the following equation:

$$D = t(k_{1A} - k_{core})/(2\, k_{core})$$

where:
- D = penetration distance of cutoff into Zone 1A
- t = thickness of cutoff wall
- k_{1A} = hydraulic conductivity in vertical direction of Zone 1A core material
- k_{co} = Hydraulic conductivity of diaphragm wall (assumed to be isotropic)

For the West dam cutoff the following values apply: $t = 0.91$ m, $k_{1A} = 2 \times 10^{-8}$ m/s and $k_{co} = 5 \times 10^{-9}$ m/s, which yields a penetration depth of 1.37 m. However, in view of the variability of k_{1A} and k_{co}, the penetration was increased to 4.60 m.

A1.4.6 Quality control

For purpose of quality control, the following activities or material properties were monitored during construction:

- Panel excavation
- Plastic concrete backfill

Quality control for panel excavation:

The following variables of the excavation and backfilling process were monitored:

- Composition and characteristics of bentonite slurry used as supporting fluid in the excavation
- Depth of the slot at completion of each panel excavation
- Cleanliness of the bottom of each panel using a heavy weighted tape
- Verticality of the excavation and the horizontal alignment
- Continuity of panel joints
- Slurry loss during excavation

The verticality of the panels was kept within the tolerance of 1 percent. The maximum deviation of any panel excavation was less than 0.3 %. Selected panel slots were checked using a 3 mm cable plumb line with a 5.44 kg (53.4 N) weight.

Quality control testing for the bentonite slurry applied the following ranges of acceptable values (Table 3):

Table 3 Accepted ranges for bentonite slurry properties (*)

Property	Accepted range of values	
	Fresh slurry	Slurry inside trench
Marsh funnel viscosity (s)	32 – 40	36 - 50
Mud balance density (g/cm^3)	1.03 – 1.04	1.04- 1.20
Sand content (% by weight of total slurry)	5	5
Filtrate loss (ml)	15 – 20	15 – 25
pH	6.5 – 10	6.5 – 11
Filter cake thickness (mm)	1	1 - 3

(*) in accordance with API recommended practice 13B-1

Quality control for plastic concrete:

The following variables were monitored through control tests:

- Compressive strength, for every 76 m^3 (100 cubic yard) of plastic concrete placed (according to ASTM D4832)
- Hydraulic conductivity, for every 382 m^3 (500 cubic yard) of plastic concrete placed (according to ASTM D5084)
- Slump test, at the trench (for every 38 m^3 of plastic concrete placed (according to ASTM C143).

In addition, the volume of plastic concrete placed versus the theoretical trench volume was monitored for each panel. Deviations in the plastic concrete backfill volume curve would indicate trench collapse. Actually, the volume placed exceeded the theoretical value by 6.5 %.

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A1.5 Xiaolangdi Dam (China)

A1.5.1 Project description

The Xiaolangdi Multipurpose Dam project is located at the exit of the last gorge in the middle reach of the Yellow River, bordering Luoyang and Jiyun cities in Henan Province, China. The zoned earth and rockfill dam is a key element of the Yellow River cascade providing flood protection, sediment control, irrigation, drinking water supply and hydropower. The dam has a height of 154m, a crest length of about 1700 m and a volume of about 50 million m^3. Sealing of the dam body is by an inclined core built of loess. A cross section of the dam is shown in Fig. 1. The upstream cofferdam has been integrated into the body of the main dam. Sealing between the downstream face of the cofferdam and the upstream face of the main dam has been accomplished by placing a zone of blended impervious material. Detailed descriptions on the foundation treatment at the Xiaolangdi dam site can be found in Godde (1998, 1999), Richard & Mazzierri (1999, 2000) and Cao (2000).

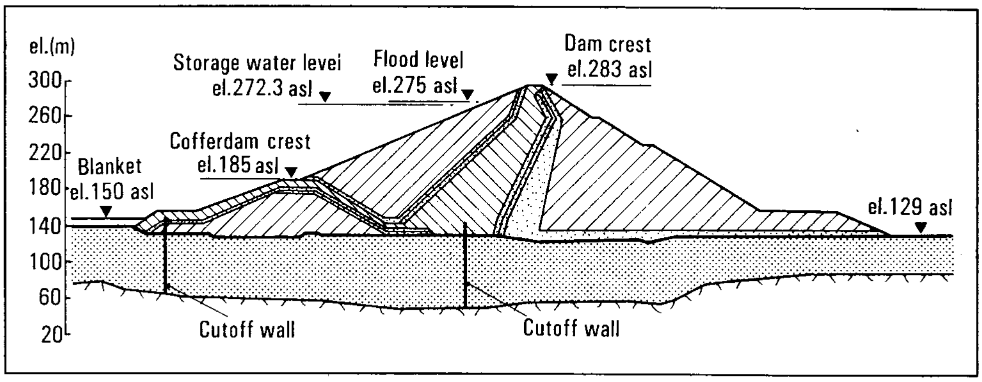

Fig. 1 Cross-section of Xiaolangdi Dam (Godde, 1998)

A1.5.2 Foundation conditions

The subsurface conditions at the dam site consist of Permian and Triassic formations overlain by quaternary deposits. The Permian formation is composed of brownish red silty claystone and argillaceous siltstone interbedded with calcareous fine sandstone and siliceous medium coarse sandstone (see Fig. 2). A layer of surficial quaternary deposits covers much of the dam site, except on the steep slopes to the right and the left of the river. The valley deposits consist of silty soil while in the river channel up to 70 m of sand and gravel layers are present. The hydraulic conductivity of the alluvium in the river channel was determined from pumping tests and lies in the range of 5.8×10^{-5} m/s (sand lenses and bottom sand and gravel) to 9.3×10^{-4} m/s (surficial deposits).

A major regional east-west trending fault F_1, extends over some 45 km and was identified to pass beneath the dam site.

A1.5.3 Foundation treatment

General sealing concept

The sealing of the subsurface has been accomplished by a variety of methods. Below the core of the Main Dam there is a diaphragm wall of conventional concrete reaching down to a depth of about 70 m. Underground sealing of the upstream cofferdam consists of a 0.8 m thick reinforced concrete cutoff wall on the right bank and a cutoff with intersecting jet-grouted columns spaced at 1 m and with a minimum diameter of 1.20 m on the left hand side. In addition, there is a clay blanket, upstream of the cofferdam on the right bank. This blanket replaces the jet-grouted curtain in the river bed and serves to reduce potential seepage along fault F_1 (see Fig. 2). It extends over a distance of some 300 m.

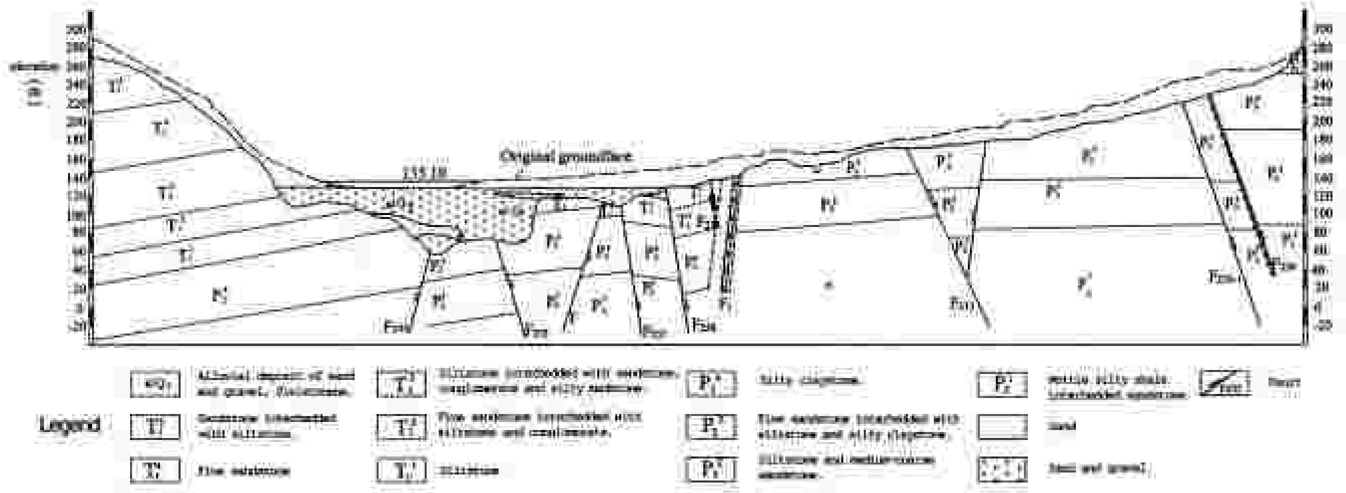

Fig. 2 Geological section along dam axis (Liu et al., 2006)

Outside of the footprint of the dam sealing consists of curtain grouting of the bedrock, which, depending on the conditions encountered was carried out in up to five rows. Prior to the start of curtain grouting, 5 m deep consolidation grouting was carried out covering a grid of 3x3 m over the entire area of the dam's footprint. In total over 100 km of consolidation and curtain grouting were performed. Curtain grouting was also carried out below the flanks of the diaphragm wall, as shown in Fig. 3.

Cutoff wall below Main Dam

This wall was conceived as a 1.20 m thick structure built of conventional (normal) concrete and with a depth of nearly 70 m. It consisted of two parts, namely a lower part to be constructed by the slurry trench method while an upper part (which is above ground level) would be constructed by formwork. Both parts would consist of normal concrete. For the lower part the parameters listed in Table 1 were specified.

Table 1 Parameters specified for conventional concrete mix in slurry trench

PARAMETER	VALUE
Minimum 28 day compressive strength (MPa)	35
Maximum elastic modulus (GPa)	30
Slump (cm)	20±2
Maximum size of aggregate (mm)	40
Coefficient of hydraulic conductivity, k (m/s)	$<10^{-9}$
Maximum deviation from the vertical (%)	0.25

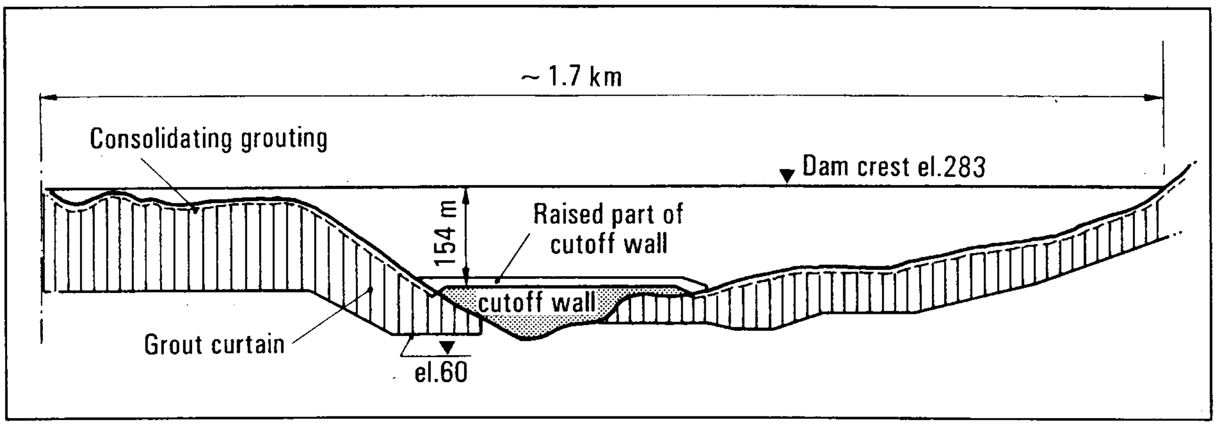

Fig. 3 Longitudinal section along dam axis showing grout curtain and diaphragm wall (Godde, 1998)

The reason for the choice of conventional concrete for the wall was that this material would have a higher strength than plastic concrete and would be better able to cope with the loads imposed by the high dam.

On the right bank, construction of the lower part of the wall had already been achieved in a previous contract, but only up to ground level, as shown in Fig. 5.

The upper part over the entire length of the wall raised by formwork protrudes 14 m into the core material. The top is chamfered and together with a zone of plastic impervious loess material placed above the wall, it is aimed to enable the safe settlement of the adjacent core material.

In the tender, it was proposed to excavate the lower part, i.e. the slurry trench wall, in primary and secondary panels using both grab and chisel and the trench cutter. The excavation would start with primary panels of 6.80 m in length using a standard bite of both the grab and the cutter of 2.80 m (see Fig. 4a). The primary panel could then be excavated in three bites, staring first with the outer bites. Although preference was given to the trench cutter as the excavation tool, it turned out that because of a significant quantity of cobbles and boulders most of the excavation work had to be done with clamshell grabs and chisels. The trench cutter was employed mainly for keying a trench into the bedrock by about 1 m and for verticality control.

Following the excavation of the primary panels, it was planned to remove the remaining soil column, designed to be 400 mm by a single bite of 2.80 m. This would create an overlap of 1.20 m on both sides of the secondary panel and the length of the primary panels would be reduced to 4.40 m. For excavation of the secondary panel, it was foreseen to use the trench cutter, because cutting back the primary panels by 1.20 m could not be accomplished properly by the grab. This operation would create two joints at each secondary panel.

After commencement of the excavation work, further investigations of the subsurface conditions and the position of the rock line revealed three areas with overhanging rock (see Fig. 5), which required special treatment. To each one of these three areas a different method of sealing was applied, namely:

- For Area 1 (see Fig. 5), pre-drilling of the key trench into rock was carried out with pilot borings of 165 mm diameter in a 300 mm center grid. Subsequently, these holes were reamed to a diameter of 381 mm which made them overlapping. The remaining rock columns were then removed by chiseling.
- For Area 2, three lines of intersecting jet-grouted columns were used to treat the area under the overhanging rock. These columns were installed after completion of the cutoff wall in that area.
- For Area 3, the wall was keyed into the rock using the trench cutter.

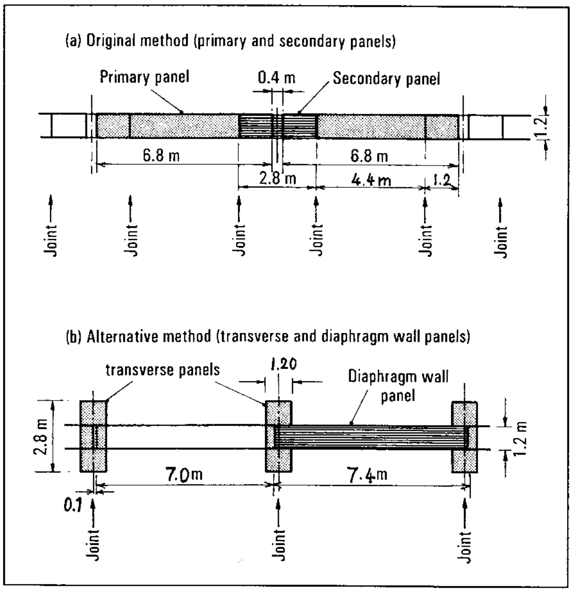

Fig. 4 Slurry trench diaphragm wall: (a) original concept with primary and secondary panels; (b) actually applied method with transverse and diaphragm wall panels (Godde, 1998)

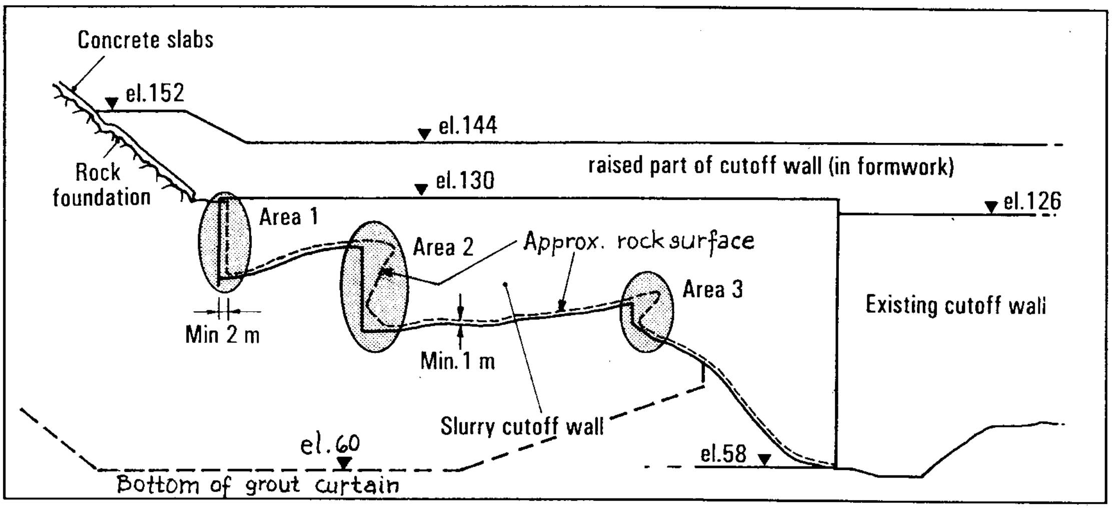

Fig. 5 Problematic areas along cutoff wall with overhangs in the rock surface (Godde, 1998)

The reason for choosing these treatment methods was mainly to reduce the use of the trench cutter because construction of the wall was on the critical path and the trench cutter was slow in cutting the hard rock. In addition, to speed up construction, a new approach was conceived which for the first time used both conventional and plastic concrete in the same wall, a kind of hybrid wall.

In this new technique the diaphragm wall no longer consists of primary and secondary panels but contains now joint panels of plastic concrete, arranged perpendicular to the wall axis, i.e. so-called transverse panels and wall panels of conventional concrete in between. This is illustrated in Fig. 4b where it can be seen that the 6.8 m long wall panels cut back into the transverse panels by 10 cm. The size of the joint panels is given by the dimensions of the grab and the trench cutter tool, which produce a bite of 1.2x2.8 m. Table 2 lists the mix proportions of the conventional concrete originally specified for the primary and secondary panels of the original wall concept and actually used for the wall panels in the hybrid wall, and as well the mix of the plastic concrete poured into the joint panels.

Table 2 Mix proportions used in the diaphragm wall

Component	Conventional concrete (used for wall panels)	Plastic concrete (used for transverse panels)
Cement (kg/m3)	400	150
Water (liters/m3)	180	305
Bentonite (kg/m3)	0	100
Aggregate (kg/m3)	1810	1567
Min. 28-day compressive strength (MPa)	35	2

The hybrid method with transverse panels proved to be very efficient and time-saving. While the progress of cutting back into the primary panels of conventional concrete with the trench cutter was about 0.04 m^2/hr, the rate of cutting into the softer plastic concrete transverse panels was as much as 4.0 m^2/hr, which was similar to the production rate in the alluvial material. This cutting could be accomplished with the grab, and the trench cutter could be employed for specific tasks such as cutting the key trench and making verticality checks. As a final result the calculated number of working days to execute the slurry wall was reduced from 135 to only 80 days.

There were other advantages of the new method, namely the reduction on the number of joints. These were reduced by 50 %. In addition, the joints were protected by the transverse parts of the panel acting as a kind of plug in front of the joint in the wall. For this reason, also adverse effects caused by possible deviations from the vertical become negligible.

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A1.6 Colbún Dam (Chile)

A1.6.1 Project description

The Colbún hydro-powerplant with 400 MW installed capacity is located about 250 km south of Santiago de Chile on the Maule river and is part of the Colbún-Machicura multi-purpose scheme. Colbún dam is a zoned earth-gravel fill dam with a slightly upstream sloping core. It has a maximum height of 116 m, a volume of 13.9×10^6 m^3, and its construction was completed in April 1984. Detailed descriptions of the project can be found in Noguera (1985) and de Pablo & Cruz (1985). A layout and cross section of the dam is shown in Fig. 1.

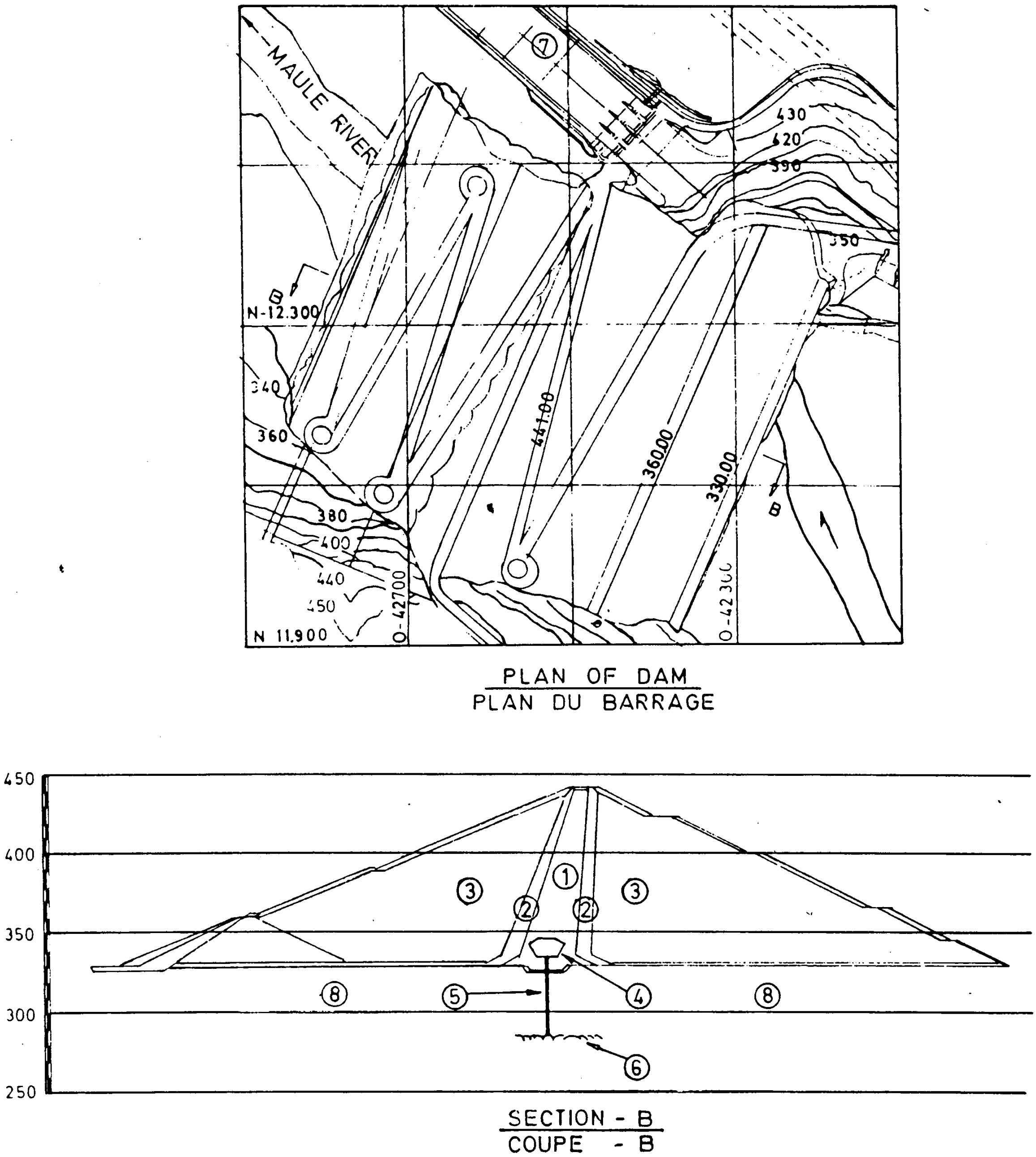


Fig. 1 Layout and cross section of Colbún dam

A1.6.2 Foundation conditions

On its abutments the dam is founded directly on rock and in the central part on quaternary fluvial deposits with a maximum depth of 66 m. The alluvium consists of clean, well-graded gravels with abundant boulders, generally stratified with open gravel zones alternating with sandy gravels. The permeability of the usually dense alluvium varies between 10^{-2} and 10^{-6} m/s with an average of about $2x10^{-3}$ m/s.

The mechanical properties of the alluvium were measured in laboratory tests with the following results:

From triaxial tests on 100 mm diameter specimens:
Strain at failure: $\varepsilon_f = 4.17$ %
Modulus at failure: $E_f = 50$ MPa
Modulus at 75% failure load: $E_{75} = 92$ MPa
Initial tangent modulus: $E_i = 350$ MPa

A1.6.3 Foundation treatment

The rock on the abutments and below the alluvium was treated by grouting. The river alluvium was sealed by means of a plastic concrete cutoff wall. This cutoff wall has a thickness of 1.20 m, a maximum depth of 68 m, a length of 335 m and a total vertical area of 12,800 m^2 and consists of 45 panels of plastic concrete. At its base the wall cuts into the andesite-breccia bedrock.

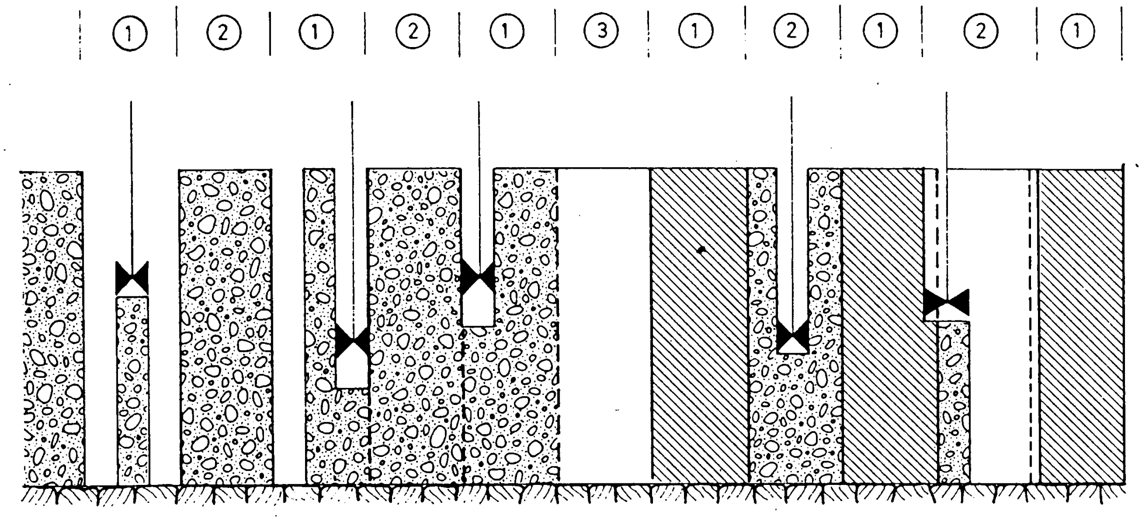
A1.6.4 Cutoff wall construction

Construction was performed by excavating alternate panels, as illustrated in Fig. 2. Each panel had a length of 7.50 m and was excavated in three 'bites' by means of a cable operated clam shell with an opening of 2.70 m and weighing 11 tons. To loosen and crush larger boulders an 8 ton chisel was employed. There was an approximately 0.30 m overlap between adjacent panels. The position of the bedrock surface was verified by exploratory borings

Stabilization of the excavated panel slots was by a bentonite suspension with the properties shown below and in Table 1. The plastic concrete was placed by means of tremie pipes.
Bentonite of type Argentine (ELCHA): Liquid limit = 202 %
Plastic limit = 167 %
Specific weight = 26.00 kN/m3

Table 1 Properties of bentonite slurry

Property	Average values	Standard deviation
Free water (cm^3)	40.5	25.3
Cake thickness (mm)	4.7	3.15
Viscosity (s)	42.8	18.7
Sand content (%)	5.09	4.65
Density (g/cm^3)	1.08	0.07
pH-value	8.2	1.6



1 Primary panel
2 Secondary panel
3 Mixed panel
Fig. 2 Construction of cutoff wall by the panel method

The production rate of the excavation work was monitored for the different materials. The average depth of the wall was 38 m. Table 2 lists the production rates for different zones and materials of the cutoff trench.

Table 2 Excavation rates for cutoff wall

<table>
<tr><td colspan="2">Zone of excavation</td><td>Rate of excavation (m^3/hr)</td></tr>
<tr><td colspan="2">Average instantaneous production rate</td><td>2.21</td></tr>
<tr><td colspan="2">Between surface and 38 m depth</td><td>2.66</td></tr>
<tr><td colspan="2">Between 38 m and 68 m depth</td><td>1.79</td></tr>
<tr><td rowspan="4">According to type of materials</td><td>Upper 10 m of wall with compacted clayey sand (corresponding to the impervious core of the dam)</td><td>6.75</td></tr>
<tr><td>In sand and gravels</td><td>3.60</td></tr>
<tr><td>In coarse gravels</td><td>1.80</td></tr>
<tr><td>In coarse gravels and boulders</td><td>0.90</td></tr>
</table>

The effective operating time of the grab in relation to other activities during the actual excavation work (but excluding idle time) was measured with the following results:

Excavating	60.3 %
Chiseling	22.7 %
Repairing and changing equipment	17.0 %

Excavation work lasted about 10 months, i.e. from November 1981 to August 1982.

Slurry consumption was another quantity monitored during construction. Three different categories responsible for slurry loss were identified, namely:

(a) Consumption due to excavation of the panel slots plus the normal loss due to equipment operation
(b) Exceptional losses due to highly pervious strata of open work gravel (can be reduced by lowering the hydrostatic head of the slurry with respect to the groundwater table)
(c) Addition of slurry between the end of excavation and the start of placing concrete

Table 3 shows the observed slurry consumption as a function of excavation depth and loss category.

Table 3 Slurry consumption

Depth of panel slot (m)	Number of panels	Slurry consumption m^3/m^2			
		Loss category (a)	Loss category (b)	Loss category (c)	Total
0-15	7	0.97	0.14	0.03	1.14
0-30	9	1.06	0.28	0.03	1.37
0-50	16	1.47	1.06	0.12	2.65
0-68	13	1.67	1.33	0.17	3.17

A1.6.5 Properties of plastic concrete

One of the reasons to employ plastic concrete for the cutoff wall of this dam was the high seismicity of the region. Proportioning of the mix aimed at achieving stress-strain properties similar to those of the alluvium surrounding the wall.

The proportions of the mixture were as follows:

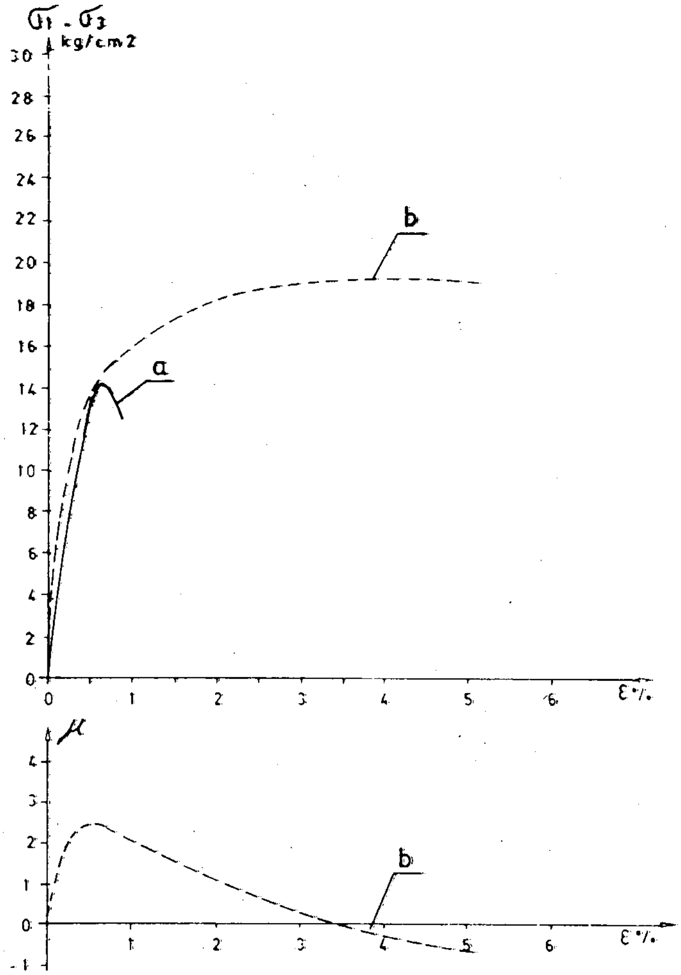
- Manufactured sand and gravel with max. particle size 44 mm (1 ¾ in.) 1483 kg/m^3
- Cauquenes clay 121 kg/m^3
- Bentonite (ELCHA) 19 kg/m^3
- Water 423 kg/m^3

The slump was between 17.8 and 22.9 cm

Cauquenes clay contains 4 % sand and has a liquid limit of 55 % and a plasticity index of 24 %.

The stress-strain and strength properties of the plastic concrete were tested in the laboratory and selected samples were taken from the site. Results of triaxial tests are shown in Fig. 3. Sample 'a' had an age of 28 days and was sheared unconfined, while sample 'b', aged 30 days, was sheared with a confining pressure of 500 kPa. Confinement enlarges the failure strain. The following moduli were measured:

Sample 'a':	initial tangent modulus,	E_i = 294 MPa
	Modulus at 75 % failure load,	E_{75} = 281 MPa
Sample 'b':	initial tangent modulus,	E_i = 630 MPa
	Modulus at 75 % failure load,	E_{75} = 275 MPa

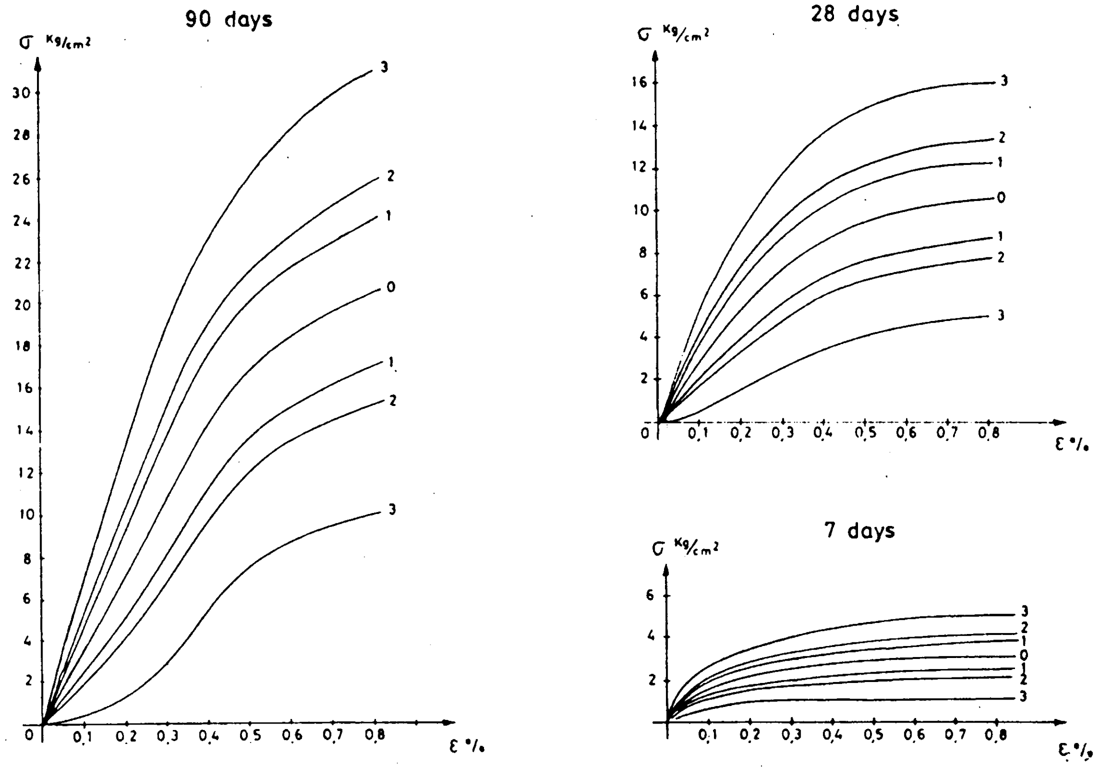


σ1-σ3	Deviator stress
ε	Axial strain
μ	Pore water pressure

Fig. 3 Stress-strain relationship from triaxial tests on plastic concrete

The effect of curing (age) can be seen from Fig. 4. The stress-strain curves are from unconfined compression tests. The stress-strain parameters measured after 90 days are:

Compressive stress at failure,	σ_f = 198 kPa
Failure strain,	ε_f = 0.84 %
Modulus at failure load,	E_f = 235 MPa
Modulus at 75% failure load,	E_{75} = 383 MPa



Unconfined compression tests

Fig. 4 Stress-strain relationships from unconfined compression tests on specimens of different age

A1.6.6 Connection between cutoff wall and impervious core

The cutoff wall penetrates about 8 m into the core (Fig. 1) A compressible clay cushion was built on top of the wall to protect the upper part of the wall and to distribute compression stresses caused by the embankment load. The material used for this cushion is a clay of medium plasticity compacted to 90 % of Standard Proctor. This material had a maximum grain size of 38 mm, 97 to 99 percent passing #4 sieve and 86 to 90 percent passing #200 sieve.

The measured vertical strain of this compressible cushion during construction was about 11%, i.e. more than five times the strain of the core. The modulus derived from the instrumentation results was in the range 10 to 50 MPa, whereas the modulus of the core material was in the range of 45 to 150 MPa.

A1.6.7 Performance of the cutoff wall

The performance and efficiency of the cutoff wall was monitored by inclinometers and pneumatic piezometers respectively. The piezometers were installed upstream and downstream of the wall at a depth of about 20 and 34 m below the base of the dam. Data are available from the construction period until completion of the dam but not from the time of impounding. The cutoff wall experienced a maximum vertical strain of 0.7 % and the alluvium of 0.9 % at a fill height of 100 m. The moduli derived from the instrumentation gave a value for the cutoff wall of around 314 MPa while those of the alluvium in the foundation were around 220 MPa, which means that both materials had similar deformation characteristics.

References

Noguera, G., 1985. Diaphragm cut off wall for Colbun main dam. *Transactions, 15th ICOLD*, Lausanne, 3:1049-1060.

Pablo, L.M. de & Cruz, A., 1985. Performance of Colbun main dam during construction. *Transactions, 15th ICOLD*, Lausanne, 1:1135-1151.

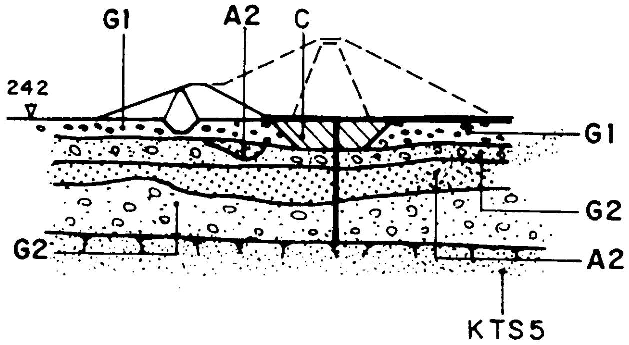
A1.7 Convento Viejo dam (Chile)

A1.7.1 Project description

The Convento Viejo irrigation project is located in the Chilenian Central Valley, about 160 km south of Santiago de Chile. The reservoir is retained by two embankment dams of 38 m and 20 m height. The higher, or main dam, is on the Chimbarongo river and has a central impervious clay core and a crest length of 730 m. Descriptions of the project are given by Alvarez et al. (1982a) and Alvarez et al. (1982b).

A1.7.2 Foundation conditions

The valley at the dam site is about 540 m wide. The bedrock consists of interstratified volcanic and sedimentary rocks of Cretaceous and Tertiary age. In the valley, the rock is covered by quaternary deposits of predominantly fluvio-glacial origin, but there are also lenses of pumiceous volcanic ashes and some talus. The maximum thickness of the sediments is 57 m. A section across the dam axis showing the different strata of the alluvium is shown in Fig. 1.



A2	Very dense silty sand	G2	Very dense sandy gravel
C	High plasticity stiff clay	KTS5	Undifferentiated sedimentary rock
G1	Very dense clean gravel and cobbles		

Fig. 1 Section through the central part of the valley perpendicular to dam axis

A1.7.3 Foundation treatment

In the original design, sealing of the alluvium below the dam was to be achieved by a 14 m deep cutoff or core trench backfilled with impervious material. It was believed that this trench would reach down to less pervious strata, however, when the excavation was nearly completed, it showed that there was no less permeable material and even some zones with serious piping problems were observed. It was then decided to backfill the excavation partially with gravel and then with compacted clay. A diaphragm wall of plastic concrete was then conceived with the assistance of Sif-Bachy. The design and construction of this wall involved a comprehensive testing program for the plastic concrete. Some results reported by Alvarez et al (1982b) are presented below.

The diaphragm wall has a thickness of 0.80 m, a total length of 540 m, and a max. depth of 55 m. It was constructed by the panel method. The length of the primary panels is 7.70 m and that of the secondary panels 6.70 m. The overlap between the panels is .50 m. The time period between concreting the primary and secondary panels was on average 58 days. There are 72 panels giving a total surface of 16,412 m^2. The total volume of plastic concrete is 14,749 m^3. Construction of the wall started in January 1976 and was completed in June 1977. The unit costs per square meter of wall, including claims, were 340 US dollars (Noguera & Gomez, 1991).

A1.7.4 Properties of the plastic concrete

Mix design

The mix proportions selected for the plastic concrete were as follows:

Aggregate (sand and gravel)	1670 – 1740	kg/m^3
Cement	80 – 84	kg/m^3
Clay (liquid limit = 59)	92	kg/m^3
Bentonite	8	kg/m^3
Water	300 – 320	$liters/m^3$

The dry density of the mix achieved was about 2000 kg/m^3 and the slump was measured as 19 cm.

The bentonite slurry used to support the excavation had the following properties:

Marsh viscosity	40 s
Density	1030 kg/m^3
Cake thickness	1.5 mm
Free water	20 cm^3

Placing of the concrete was accomplished by means of a tremie pipe inserted in the center of the panel.

Stress-strain and strength properties

The following tests were carried out:

(a) Unconfined compression tests on cylindrical samples (15cm x 30 cm) and cubes (30x30x30 cm) aged at 7, 14, 28, 90, 290, 320, and 367 days. One sample was tested at the age of 950 days

(b) Triaxial compression tests on cylindrical samples (15 cm x 30 cm) carved from blocks obtained from the upper part of the wall. Some of these samples were tested at the age of three years

(c) Plate loading tests of the diaphragm wall; load applied horizontally through 30 cm square plates.

The results of the unconfined compression tests are listed in Table 1.

Table 1 Unconfined compression tests on plastic concrete of different age

Age of sample (days)	Unit weight (kN/m^3)	Unconfined compressive strength q_u (MPa)	Strain at failure ε_f (%)	Initial tangent modulus E_i (MPa)	E_i/q_u
7	19.3	0.25	1.12	44.6	180
28	20.1	0.43	1.18	77.3	177
90	20.7	1.06	-	-	-
290	19.8	1.27	-	-	-
320	20.5	1.37	-	-	-
367	20.5	1.47	-	-	-
950	20.2	2.40	0.54	785	333

The strength increase takes place mainly during about the first 90 days, after that the development is nearly linear with time. Older specimens have a lower failure strain, i.e. they become more brittle.

The results of drained triaxial compression tests with consolidation pressures from 100 to 700 kPa on three year old samples are shown in Fig. 2 and Fig. 3 and listed in Table 2. Fig. 2 presents the strength parameters in terms of effective stress in a Mohr circle diagram yielding the values

$c' = 490$ kPa and $\varphi' = 36°$

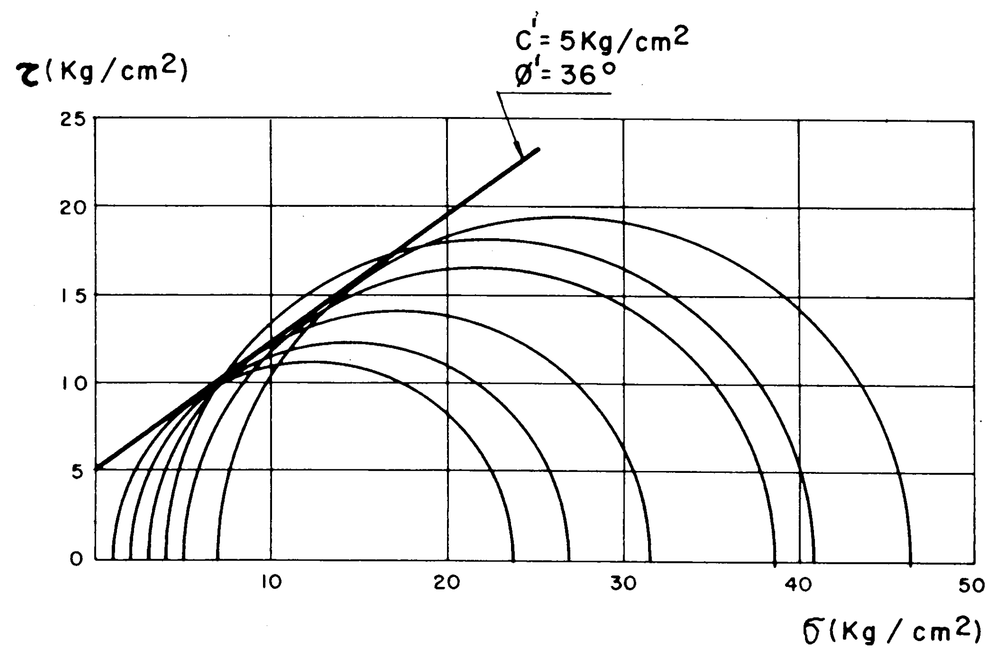
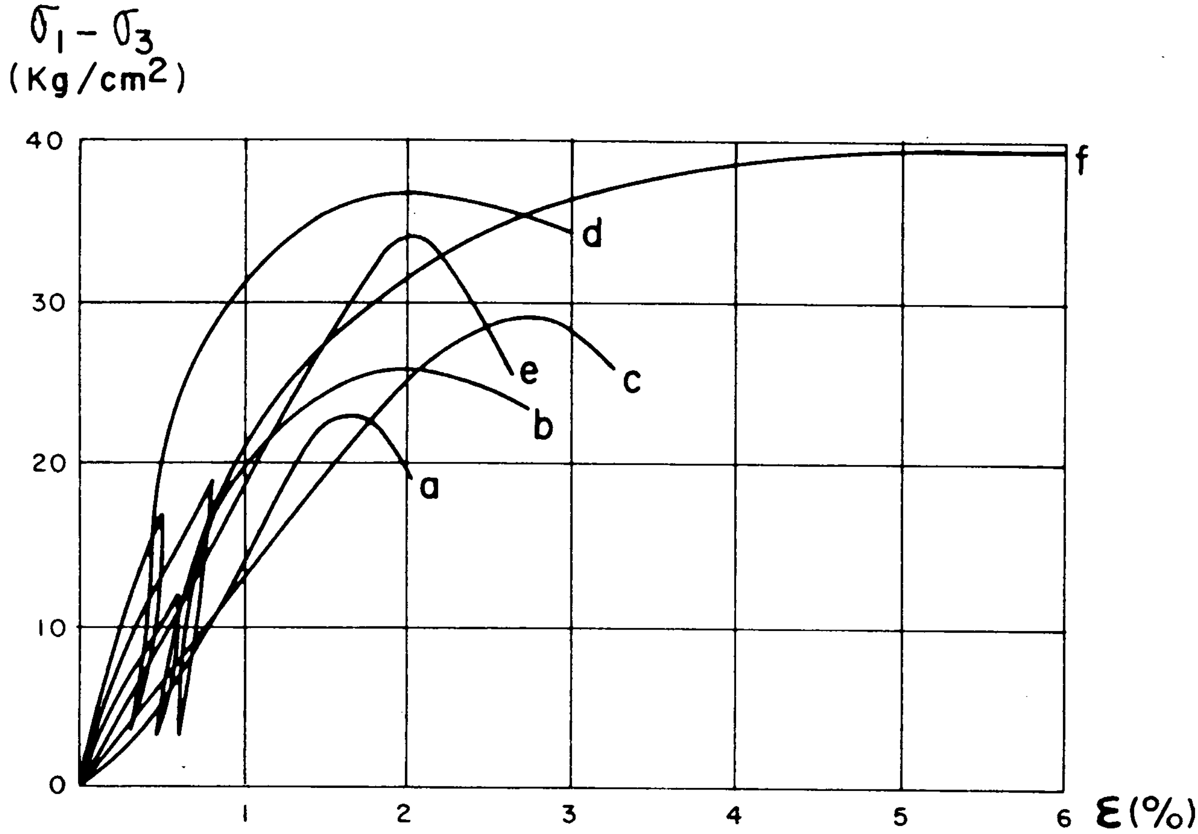


Fig. 2 Mohr circle diagram for 3-year old plastic concrete specimens

The corresponding stress-strain curves are given in Fig. 3. They indicate that the strain at failure is around 2 % and clearly higher than from the unconfined tests.



Curve a: σ_3=100 kPa, Curve b: σ_3=200 kPa, Curve c: σ_3=300 kPa
Curve d: σ_3=500 kPa, Curve e: σ_3=700 kPa
Fig. 3 Stress-strain curves of consolidated drained triaxial tests on 3-year old samples

Table 2 Young's modulus from triaxial tests

Confining stress σ_3 (MPa)	Deviator stress at failure $(\sigma_1'-\sigma_3')_f$ (MPa)	Initial tangent (Young's) modulus E_i (MPa)	Unloading/ reloading modulus E_r (MPa)	Strain at failure ε_f (%)	$\frac{E_i}{(\sigma_1'-\sigma_3')_f}$
10	2.22	196	-	1.5	88
20	2.43	208	404	2.0	84
30	2.79	221	-	2.5	79
40	3.61	353	833	2.0	98
50	3.29	196	-	2.0	60
70	3.87	300	430	5.0	77

The set up used for four plate loading tests on the diaphragm wall is shown in Fig. 4. The loading history comprised loading unloading cycles as illustrated in Fig. 5. The maximum applied stress was about 3 MPa. Each loading step of the first cycle was maintained during 2 minutes, whilst steps in the second and third cycle were kept during 20 minutes. The maximum load in the third step was kept for 200 minutes.

The initial tangent modulus obtained for the first cycle was E_i = 110 MPa and the unloading/reloading modulus E_r = 860 MPa.

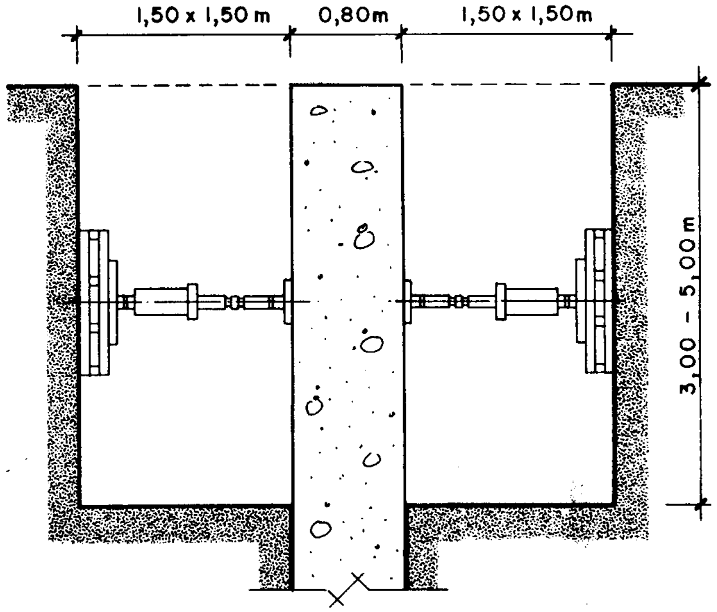


Fig. 4 Set-up for plate loading test on diaphragm wall

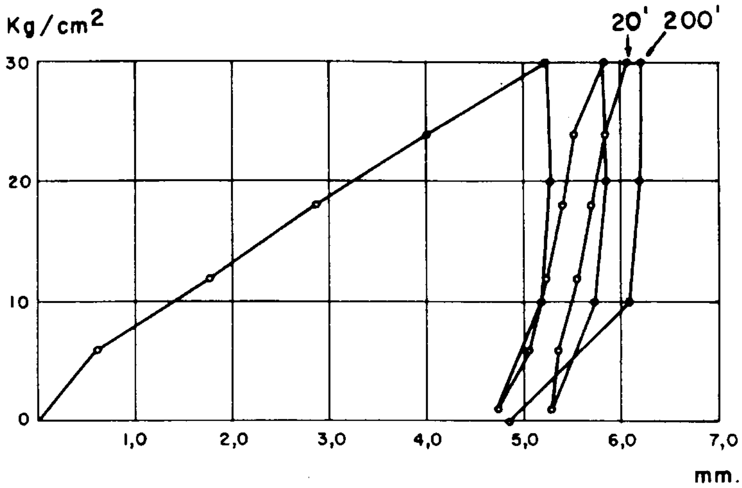


Fig. 5 Load-settlement curves obtained from plate loading test at 3 m depth

Hydraulic properties

After completion of the wall its watertightness was tested by a pumping test program consisting of five pumping wells, 3 to 5 m downstream of the cutoff, with a depth range from 21 to 56 m, twelve borings instrumented with multiple Casagrande piezometers, upstream and downstream of the wall, and twenty-four test pits, some 5 m deep, to control piezometric levels. Water levels in the pumping wells and the piezometers were monitored throughout the testing phase together with the discharges from the wells. The latter were usually kept constant. After completion of the tests, monitoring of the water level continued over a period of not less than 7 hours but not longer than 24 hours.

The analysis of the pumping tests showed that the alluvium valley fill contains several aquifers. The hydraulic conductivity, k, in these pervious zones of the alluvium ranged between $2x10^{-5}$ and $5x10^{-4}$ m/s. Except for a leaky panel, the seepage flow through the cutoff under a hydraulic head of 9 m was 0.014 liter/s per meter wall, a very small value. The total seepage through the cutoff wall after the dam has been built and with full reservoir, was estimated as 29.1 liter/s. Without the wall the probable seepage would have been about 445 liters/s.

To test the hydraulic conductivity of the wall itself, seven holes of diameter 102 mm were drilled in the mid-plane of the wall to a variable depth between 10 and 20 m. Water pumping and level recuperation tests were carried out and k-values of between $0.4x10^{-8}$ and 10^{-7} m/s were obtained.

References

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Alvarez, L., Larenas, J., Bernal, A. & Marin, J.A,, 1982b. Characteristics of the plastic concrete of the diaphragm wall of Convento Viejo dam. *Transaction, 14th ICOLD*, Rio de Janeiro, Q.55, R. 22, 4:371-389.

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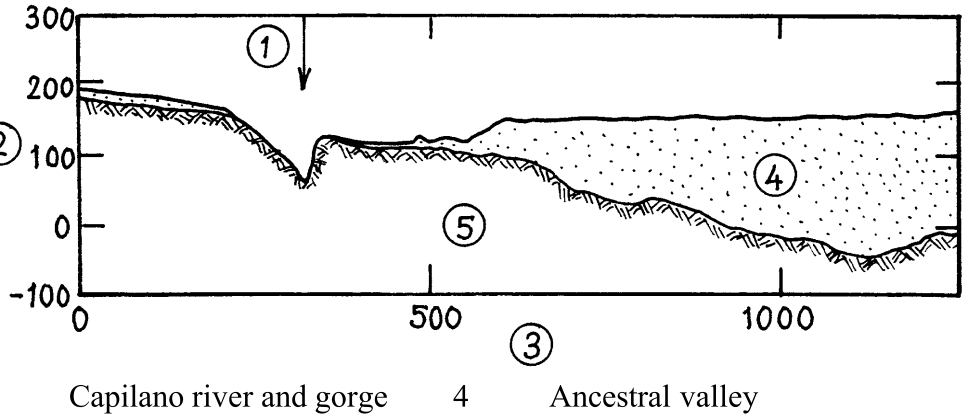
A1.8 Cleveland dam abutment sealing (Canada)

A1.8.1 Project description

Cleveland dam is located on the Capilano River in North Vancouver, approximately 6 km upstream of the Burrard Inlet. The dam is a concrete gravity structure, 91 m high, and its construction was completed in 1954. It provides quality drinking water to the Greater Vancouver region and is a key element in a network of three watersheds operated by the Greater Vancouver Water District.

The dam is founded in a bedrock canyon, which does not pose any sealing problem, however, the presence of a deep valley adjacent to the eastern (or left) abutment causes seepage to flow from the reservoir to the downstream area (see Fig. 1). This buried ancestral valley of the Capilano river is filled with glacial and interglacial sediments, including two aquifers, of which the upper one daylights on the eastern valley slope. Both aquifers are possible seepage paths from the reservoir. The upper aquifer is continuous for hundreds of meters and in fact, 90 percent of the seepage passing through the left abutment is transmitted through this aquifer. The seepage problem was recognized at the time of design and construction of the dam and a 1.5 m thick blanket of compacted till was placed over the upstream reservoir slope on a length of 200 m, together with a single line grout curtain through the overburden to bedrock. The performance of these sealing measures was, however, found to be inadequate and further seepage control measures in the form of drainage adits, emergency pumping wells, and a plastic membrane were installed during subsequent years. However, it was observed that considerable amounts of silt and sand, believed to originate from a 6 m thick sand stratum at the base of the upper aquifer, were eroded into the drainage adits. Hence, there were concerns over the possible development of piping erosion through the upper aquifer and the generation of high pore water pressures compromising the stability of the slope.



1	Capilano river and gorge	4	Ancestral valley
2	Elevation (m asl)	5	Bedrock
3	Distance (m)		

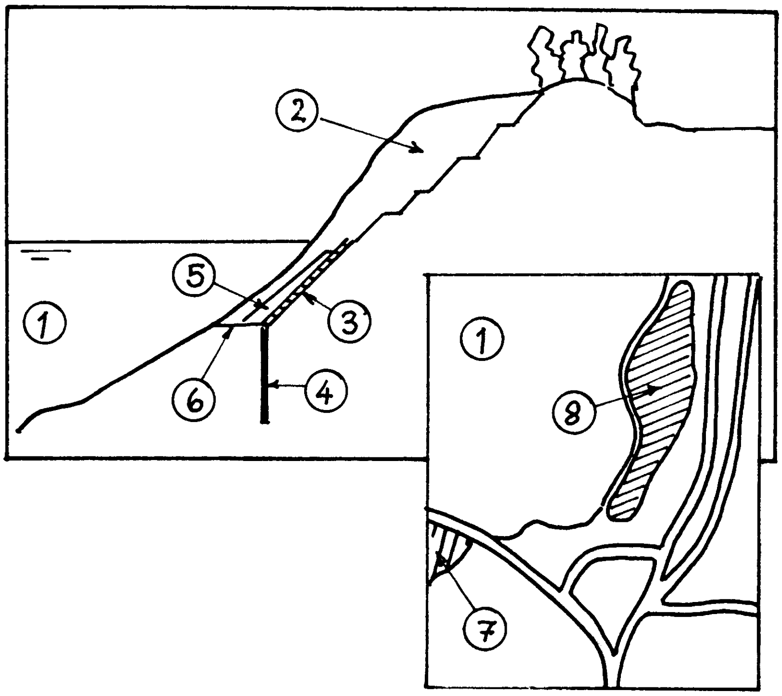
Fig. 1 Section through Capilano canyon and ancestral valley (looking upstream)

Additional remedial measures were executed during 2001/2002 with the aim to reduce the seepage gradients and piezometric heads in the abutment. The new concept was to construct a positive seepage barrier across the upper aquifer consisting of an extension of the existing blanket and a cutoff. Design criteria required an average gradient of 4% with a maximum value of 5.5 % based on hydro-geological modeling. Blanket extension design was based on a conductance (hydraulic conductivity x cross-sectional area/length of flow path) of about $1x10^{-7}$ m/s/m. The new blanket needed to be non-erodible and constructible during rainy weather and be stable under full reservoir and drawdown conditions. Roller compacted concrete (RCC) was chosen as the impervious material for blanket construction.

It was not possible to cover the abutment entirely with the blanket since the lowest drawdown level was 13 m above the lowest elevation of the upper aquifer within the blanket extension. Therefore, for the lower portion of the aquifer a plastic concrete diaphragm wall was selected after evaluating various other alternatives (see

1	Reservoir	5	Fill over blanket
2	Abutment excavation	6	Platform (Elev. 164)
3	RCC blanket	7	Cleveland dam
4	Plastic concrete cutoff wall	8	Treated area

Fig. 2).



1	Reservoir	5	Fill over blanket
2	Abutment excavation	6	Platform (Elev. 164)
3	RCC blanket	7	Cleveland dam
4	Plastic concrete cutoff wall	8	Treated area

Fig. 2 Section through right abutment and layout

Hence, the two key components of the seepage control remedial measures are a 306 m long and 0.80 m thick, vertical plastic concrete cutoff wall with a maximum depth of 23 m topped by an inclined RCC blanket extending over the upper aquifer on the reservoir slopes to about 8 m below full supply level (FSL). There is a non-reinforced concrete cap above the cutoff wall providing the transition from the wall to the RCC blanket extension. Water stops are placed at the top of the plastic concrete wall and at the top of the guide walls, which were left in place. A short section of compacted glacial till ties the existing till blanket to the new RCC blanket extension, and a geosynthetic clay liner provides a low permeability barrier at the contact between the top of the RCC and the natural soils.

To stabilize the RCC blanket during reservoir drawdown, rockfill and granular fill were placed over the blanket to provide ballast support.

The entire seepage control project was constructed within 18 months, at a total cost of US$ 25 million. A photograph of the completed rehabilitation works is shown in Fig. 3. A detailed description of the construction procedures can be found in Sing et al. (2006)

Fig. 3 Aerial view of the seepage control work on left abutment. Cleveland dam is at the lower right hand corner.

A1.8.2 Construction of the plastic concrete cutoff wall

Preparatory works

The cutoff wall constructed for this project differs from the usual walls below dams across river channels in that it had to be constructed adjacent to a steep slope. Stability analyses indicated that 37° excavation slopes with 4 m wide berm spaced at 10 m vertical intervals in the glacially consolidated soils and 35° slopes with 4 m wide benches at 10 m vertical interval in the post-glacial soils would provide acceptable safety factors based on limit equilibrium for all loading conditions, i.e. operating, draw-downs, and earthquake with 0.5g. The final overall slope angle, including benches, became then about 29°. In order to provide a working platform for the construction of the cutoff wall, a minimum bench width of 13.5 m was required. Slope excavation and preparation of the construction bench involved the excavation of about 300,000 m^3 of material.

After cutting the approximately 70 m high slope, it was shotcreted and drains were installed at two elevations.

Excavation

Construction of the entire wall was completed in a four-month period working 24 hours a day and six to seven days per week. The method of wall construction employed the successive panel method using stop-end tubes.. First, so-called 'opening' panels were excavated. These were 6 m long plus 0.8 m on each end for stop-end tubes. So-called 'running' panels were then added on both sides, having a length of also 6 m plus 0.8 m for a stop-end tube on the extension side. At the end of the wall, a so-called 'closing' panel of 6 m length completes the wall. In total, the wall consists of 50 panels, ranging in length from 3 m to 8.8 m.

The guide walls had a width and depth of 1.20 m. Below the inside of the uphill wall a 100 mm diameter drain pipe was installed in gravel drain rock wrapped in non-woven geotextile connected to 60 mm diameter vertical sumps spaced at intervals of about 30m. The aim was to keep the groundwater level at least 600 mm below the top of the guide walls during excavation and backfilling of the panel slots.

The required depth of excavation was explored by drill holes. These holes extended below the final planned excavation depth and they were grouted after completion of drilling. Excavation was by cable clamshell buckets with a nominal width of 760 mm and a bite length of 2.5 m. Where necessary, a chisel was used to trim and crush boulders or also plastic concrete from adjacent panels at depth. Verticality was checked by lowering a plumb bob or a stop-end pipe along a joint and recording the offsets. A cleanup bucket of a width of 810 mm was used for the final cleanout of each slot before concrete was backfilled. At its base the cutoff wall penetrated a minimum of 4 m into the silt layer of an aquitard located at the base of the upper aquifer. Standard penetration test sounding was employed to ensure that the bottom of the panel was in the silt layer. The depth of the wall varied from 6 to 23 m below the elevation of the platform. On its downstream end the wall connects to the existing till blanket.

Support of the excavation was by a bentonite slurry. The slurry level was maintained as high as possible to increase the stability of the trench walls, but always a minimum of 600 mm above the local piezometric level. The bentonite used was a polymer treated product, Bara-kade 90. The density of fresh slurry generally ranged from 1042 to 1060 kg/m^3. In the excavation the density increased to about 1060 kg/m^3 and even to as much as 1200 kg/m^3. The slurry was sent through a de-sander which allowed it to be re-used up to three times.

Concrete was placed by the tremie method with two steel pipe tremie lines per panel. The 760 mm diameter stop-end tubes, made of steel, were placed prior to concrete placement and created a semi-circular joint to connect to the subsequent panel. The stop-end tubes were 'cracked' within six hours after completion of concreting, then lifted an additional 0.5 to 1 m per hour for the next six hours. They were generally removed from the excavation 12 to 18 hours following completion of the pour.

Mix design

The design of the plastic concrete mix was based on requirements for strength and hydraulic conductivity as follows:

Strength: Unconfined compressive strength: 0.75 MPa (7 days)
1.5 MPa (28 days)
Minimum axial strain at failure in triaxial compression: 5 % (to accommodate small strains during seismic loading)

Hydraulic conductivity: $1\text{x}10^{-9}$ m/s in axial direction in a confined (triaxial) test

During trials with various mixes it was found that there was similar behavior using either crushed or rounded aggregate.

The finally accepted plastic concrete mix had the following composition:

- Cement (Portland Type 10) 143.6 kg/m^3
- Bentonite (un-treated Wyoming) 36.6 kg/m^3
- Coarse aggregate (CAN/CSA-A23.1-M max. 20 mm) 707.1 kg/m^3
- Fine aggregate (CAN/CSA-A23.1-M) 707.1 kg/m^3

During construction, the bentonite content was reduced to 24 kg/m^3 to achieve the design strength.

Quality control

Contractor and Engineer maintained checklists for each panel including dimensions, excavation details, stratigraphy of the trench walls, slurry quality, plastic concrete batch slips, and placement summaries. Also a comparison was made between the theoretical volume excavated and the volume of concrete placed.

Control tests on the bentonite slurry used for the support of the trench included twice daily checks on: density, pH, sand content and viscosity. Samples were taken from the top, the middle and the bottom of the panel. Slurry with a density greater than 1106 kg/m^3, a sand content larger than 5 %, or a marsh funnel viscosity longer than 45 seconds was replaced with fresh slurry prior to approval for concrete placement.

Quality control tests on the plastic concrete during construction comprised unconfined compression tests, triaxial tests, and hydraulic conductivity measurements. The following average properties were measured:

Unconfined compressive strength: at 7 days: 0.78 MPa
at 28 days: 1.05 MPa

Axial strain at failure in triaxial compression: 10 %

Laboratory hydraulic conductivity: 1.7×10^{-9} m/s

In-situ hydraulic conductivity (measured in the wall in pre-formed holes): 1 to 3×10^{-8} m/s

The measurement of the hydraulic conductivity in the cutoff wall was initially attempted by coring holes into the completed panel. However, core recovery was poor and the samples highly disturbed. The method was then changed to inserting steel pipes into the fresh plastic concrete prior to its setting. Hydraulic packer conductivity tests were then conducted in the formed and cored holes.

Tests were also conducted by using natural gamma downhole logging with the purpose to check the panel width and possible soil inclusions caused by caving during concrete placement. However, it was found that there was insufficient contrast between the plastic concrete and the in situ soil and further geophysical testing was abandoned.

A1.8.3 Analyses performed

Since the cutoff wall was next to a steep slope, three-dimensional stability analysis of the open cutoff wall trench was carried out to ensure the feasibility of its construction. The analysis showed that safe construction was possible if the following conditions are met:

- Maintaining a specified slurry density
- Maintaining the slurry level at least 600 mm above the adjacent groundwater level outside the cutoff excavation
- Limiting the length of each panel to 6m adjacent to the slope
- Maintaining a minimum separation of 30 m between excavated or recently poured panels.

References

Singh, N., Douglas, R., Ahlfield, S.R & Gant, M., 2006. The case of Cleveland dam. *Water Power & Dam Construction*, January, pp. 16-24.

A1.9 Twin Buttes Dam (U.S.A.)

A1.9.1 Project description

Twin Buttes dam is an earth-fill embankment located about 10 km southwest of San Angelo, Texas, and was constructed in 1960 to 1963. The dam is 13.2 km in length and extends across three streams. Its purpose is to provide municipal water for the city of San Angelo, water for irrigation, flood control storage, and fish and wildlife benefits. The maximum height is 40.8 m and the crest elevation is at 606.8 m asl. The reservoir has a volume of $1.2767x10^9$ m^3 and consists of two pools which are connected through a high ridge by an equalizing channel, as shown in Fig. 1.

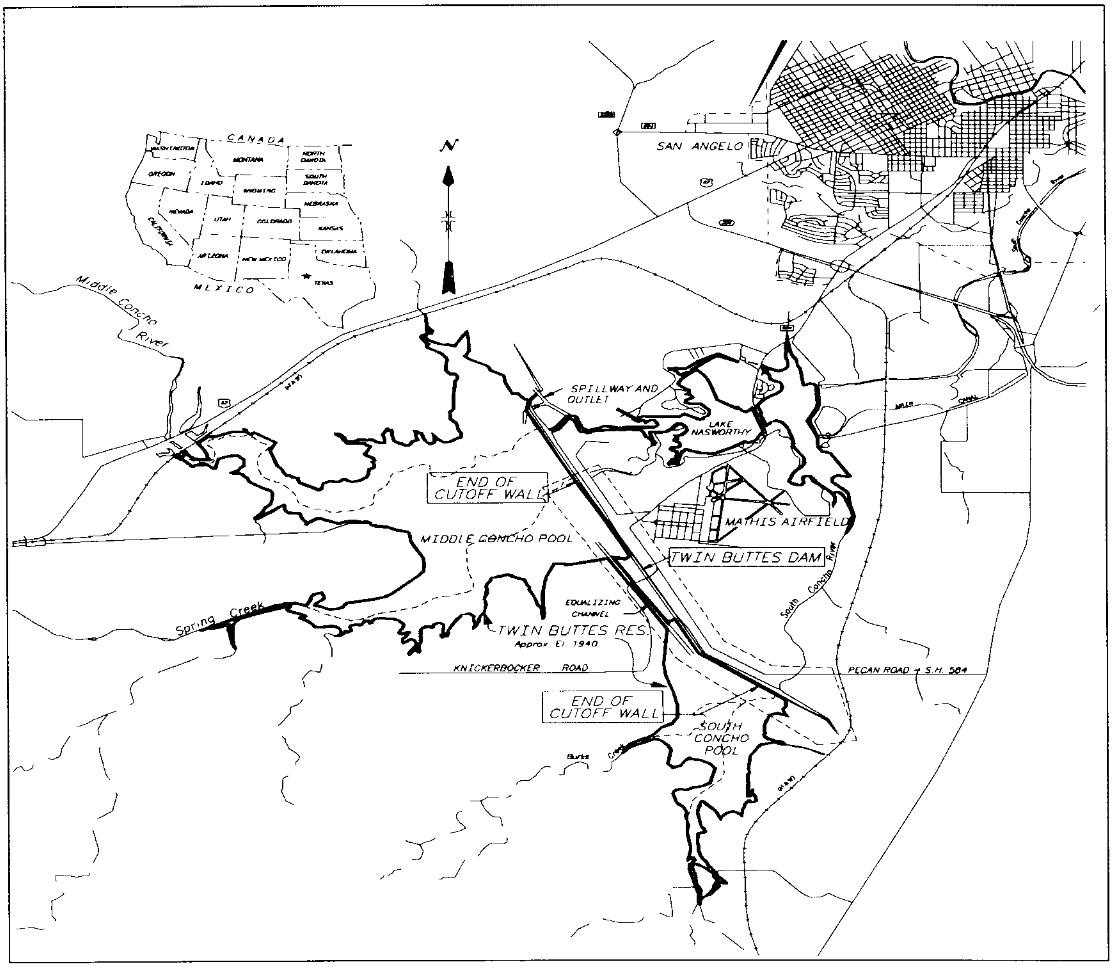


Fig. 1 Twin Buttes dam and reservoir: Plan view

The dam was originally constructed without a cutoff trench to bedrock in its central part where there is a clay deposit of variable thickness overlying an alluvial gravel deposit. The depth to bedrock amounts to about 18 to 30 m. It was believed that the clay would be sufficiently impervious and would cap the more pervious gravel. However, the alluvium outcrops in the reservoir area. This caused significant seepage underneath the dam, estimated to be at least 1.7 m^3/s. Seepage was first observed when during first impounding the reservoir level was about 18 m below the full supply level.

Apart from the economic loss, seepage was considered a hazard to the safety of the dam. Possible failure modes included: (1) high uplift pressures below the downstream shoulder and (2) blowout at the downstream toe with subsequent piping through the foundation. The population at risk in case of a dam failure would have involved around 23,000 people and the loss of life was estimated to be around 2900. The reservoir level was lowered by 16.8 m until remedial measures had been implemented. A detailed description of the project and the remedial measures taken has been presented by Dinneen & Sheskier (1998).

A1.9.2 Foundation conditions

Within the central section of the dam affected by leakage, which extended over a length of about 6.4 km the foundation consists of (from top to bottom):

- a fine-grained lean clay deposit rich in calcite, referred to as "caliche". This layer varies in thickness from about 3 to 18 m and is derived from windblown and fluvial processes. The caliche is locally cemented to various degrees. It has about 85 % passing #200 sieve and contains about 15 % sand.
- Pleistocene alluvial, coarse-grained soils, consisting mainly of clayey gravel, with a thickness varying from 0 to about 20 m. These soils are highly variable in gradation and cementation. Gravel, cobble and boulder fragments are predominantly limestone with some chert and quartzite. The permeability of the alluvium was measured to be on the order of $5x10^{-3}$ m/s or higher.
- Bedrock, consisting of flat-lying shale and sandstone of the Permian San Angelo formation, relatively impervious, but the top 0.3 to 1 m are weathered.

More details on the geology of the foundation and on results of tests on material properties were given by Bureau of Reclamation (1995).

A1.9.3 Foundation treatment

Initial remedial measures

Initial attempts to stop the seepage flow included grouting, installation of relief wells and drainage in the downstream area. Grouting turned out to be not successful. The alluvium in the central part was grouted where there was no cutoff trench but apparently there was difficulty in drilling the grout holes in the loose uncemented sand and gravels due to frequent caving of the holes. Grouting did not effectively reduce the downstream piezometric pressures.

Sixty-one relief wells were then installed at the downstream toe of the dam with the aim to reduce the uplift pressures on the embankment and increase the stability of the downstream slope. In addition, a drainage system was installed to protect some areas from becoming saturated and inaccessible. Analysis of the relief well system, however, concluded that the well system was inadequate to control uplift pressures and that additional remedial measures are needed.

Evaluation of non-structural and structural remedial measures

Non-structural measure, such as reservoir restrictions, early warning systems were considered unacceptable and among the structural alternatives three concepts were evaluated, namely: (1) cutting off the seepage by either constructing a cutoff wall, placing an upstream blanket, or by grouting, (2) control of seepage by drainage downstream of the dam by features such as seepage berm, deep drainage trench, wick drains and relief wells, and (3) elimination of the reservoir storage by breaching the dam.

Of all the remedial measures considered, the cutoff wall and breaching the dam were identified as the only viable alternatives. The cutoff wall was then selected as the most economic solution of these two alternatives.

Cutoff wall design criteria

Design considerations for the cutoff wall included: low permeability, resistance to the prevailing hydraulic gradients, cost, and constructability i.e. compatibility with the standard excavation techniques and common placement procedures. Quantitatively expressed, this means:

- Hydraulic conductivity of wall backfill, $k < 10^{-8}$ m/s
- Wall must be keyed into rock
- Wall must withstand the highest hydraulic gradient. Assuming that the reservoir level is at crest elevation during the peak flood (Elev. 607.2) and on the downstream side the ground water table is at the original position prior to dam construction (Elev. 570.6), a maximum differential head of about 36.6 m across the wall would result

Backfill alternatives and mix design

To satisfy the design criteria several backfill alternatives were evaluated, namely:

- ***Plastic concrete (P-C):*** This backfill was considered technically feasible with respect to constructability and would also have provided sufficient resistance to high gradients. However, this alternative was judged as economically unattractive because of the high cost for aggregate, cementitious material and expensive batching and handling.
- ***Cement-Bentonite (C-B):*** This alternative was not considered technically feasible with regard to constructability. It was believed that trench excavation would be very slow when cemented alluvium was encountered and the C-B slurry would then set up before the excavation was completed. This would result in delays and increased cost if the C-B backfill had to be re-excavated. The use of C-B would also preclude excavation by trench cutter because of the common usage and requirement for de-sanding of the slurry. Also, employing C-B backfill only after first excavating with bentonite slurry was considered not practical because of similarities in the specific weight of the two materials.
- ***Soil-Bentonite (S-B):*** This alternative was basically attractive because of the low material cost, however, there were technical concerns with S-B backfill in narrow trenches, such as: hydraulic fracturing and blowout of the backfill. Settlement of the S-B backfill could cause arching across the trench leading to horizontal cracks and piping of the backfill material into the surrounding pervious alluvial materials.

  Typical trench width that common excavation equipment (backhoes, clam shells, and trench cutters) are able to excavate in a single bite range from 0.6 to 1.5 m. The minimum width of the trench filled with S-B backfill to avoid a blowout failure is obtained as follows:

$$Width - \frac{Differential\ head}{Blowout\ Gradient}\ x\ \ Safety\ \ factor$$

For Twin buttes dam the following values can be assumed:

- Recommended hydraulic gradient (against blowout): 30
- Factor of safety: 3

This results in a minimum trench width of (36.5/30) x 3 = 3.65 m, which is considered not practical for a S-B trench.

Another approach to reduce the blowout potential is applying dam filter criteria to the backfill material (Bureau of Reclamation, 1991). Several S-B backfill mixes were designed to meet filter criteria against actual gradation of open work gravel. Blowout gradient tests according to Xanthakos (1981) were then performed on S-B mixes to confirm the applicability of the engineered backfills to the loading and foundation conditions of the dam site. The critical gradient was defined as the gradient at which the backfill failed, i.e. when hydraulic fracturing or piping into the foundation material occurred. This gradient was found to be between 4 and 32 resulting in safety factors of 0.2 to 1.3 for a 1.5 m thick wall-

- ***Soil-Bentonite with internal, vertical geomembrane:*** The installation of a vertical geomembrane in the cutoff trench provides an additional line of defense against hydraulic fracturing and/or blowout into the foundation open work gravel. Such an approach had been successfully used in Reach 11 Dikes, near Phoenix, Arizona where an approximately 24 km long trench, backfilled with drainage sand, was installed. However, in the Twin Buttes project certain negative aspects of geomembrane use, such as difficult installation, puncturing/rupturing of the membrane, and adequate keying into bedrock could not be resolved satisfactorily and the method was eliminated.
- ***Soil-Cement-Bentonite:*** Adding a small percentage of cement to the S-B backfill increases the strength and provides sufficient resistance to hydraulic fracturing and/or blowout of the backfill into the foundation open gravel. This alternative was selected and the wall was required to have a minimum compressive strength of 0.69 MPa (100 psi) at 28 days, i.e. about twice the maximum pressure head caused by the head difference of 36.6 m.

  Based on the evaluation of the available equipment, excavation rates, cost, and resolution of the blowout issue, the optimum width of the wall was determined as 0.76 m (2½feet).

  Proportioning of the S-C-B mix was the responsibility of the Contractor. Trial mixes showed that proportions would start with: 6 percent (±2 percent) by dry weight, of cement or cement + pozzolan, and 1 percent (±0.5 percent), by dry weight, of bentonite (resulting from use of bentonite slurry). The Contractor was allowed to replace a portion of the cementitious material with pozzolan.

A1.9.4 Requirements for wall construction

The following requirements for the construction of the wall were then specified:

- The wall shall consist of interconnected vertical panels. Each panel must be completely backfilled before the S-C-B material sets up (to eliminate horizontal joints).
- The soil shall have a specific gradation with a maximum grain size of 38 mm.
- Backfill shall be hauled to the site using truck equipped with agitators
- The slump of the S-C-B mix shall be between 18 and 25 cm
- Placement of the backfill shall be from bottom to the top by tremie pipe with a minimum diameter of 25 cm, such that backfill does not flow more than 2.3 m horizontally from a tremie (to prevent segregation)
- The bentonite slurry in a panel prior to adding of the backfill shall have a density of not less than 1.20 g/cm^3, a sand content of less than 5 %, and a marsh funnel viscosity of less than 45 s.
- Rigorous quality assurance and quality control (QA/QC) program consisting of record keeping of batching operations, casting test specimens (150x300 mm) for compressive strength and permeability testing, coring of selected wall panels and water testing of cored holes, and piezometers installed immediately upstream and downstream of the cutoff wall.

A1.9.5 Wall construction

The cutoff was constructed from the top of a working platform (see Fig. 2) whose top elevation is 1.5 m higher than the reservoir level to ensure that the slurry level in the trench would always be 0.9 to 1.5 m above the reservoir level. To facilitate construction, the reservoir was drawn down by 20 m. The working platforms later became part of the permanent construction. They are built of impervious material and are an integral part of the seepage cutoff.

The cutoff wall was to be keyed into the low permeability bedrock by a minimum of 0.75 m. The required depth was determined during construction by core drilling holes, spaced at 30 m centers, to a minimum of 3 m into bedrock and water pressure testing on the bottom of each hole.

Excavation of the cutoff wall began with a 360 m long test section. Primary panels were 15 m (50’) long and secondary panels 2.4 m. Equipment for excavation included a hydraulic Kelly clamshell and cable-operated clamshells supplemented with chisels to break up the cemented alluvial material. Later, the contractor started using a Casagrande trench cutter. Backfill of the 15 m long panels was accomplished by means of four tremie pipes fed by four transit mixers. The wall was completed in early 1999, i.e. 130,000 m^2 in about seven months.

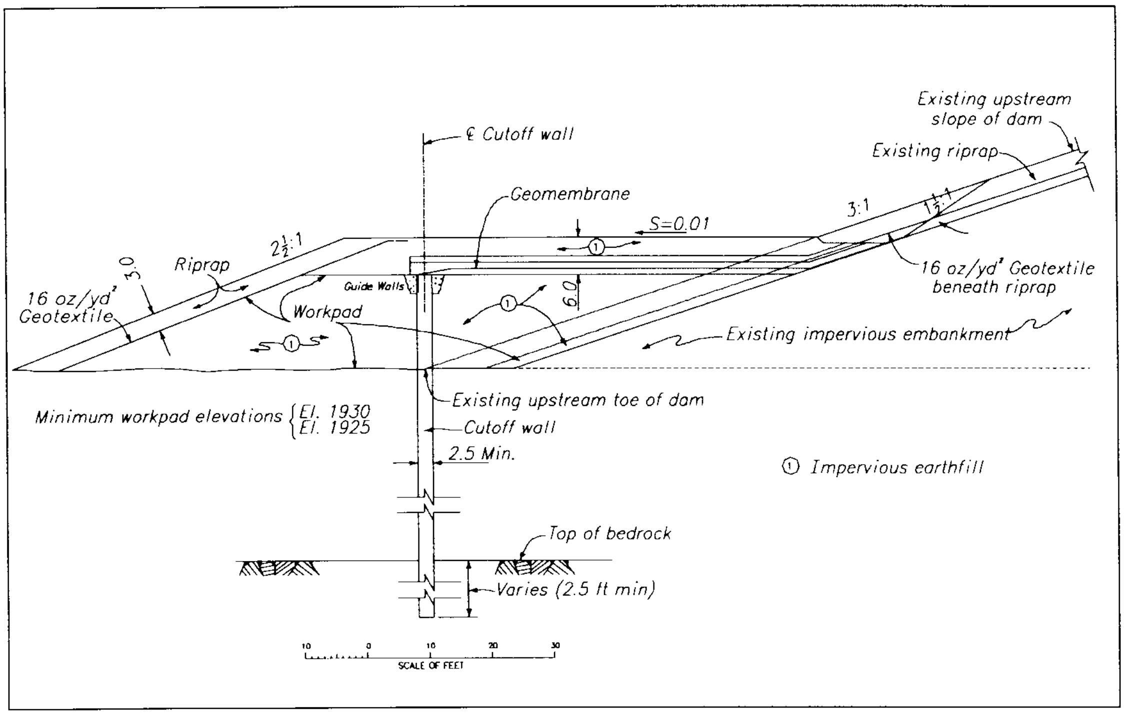


Fig. 2 Typical section of working platform and upstream blanket

After construction of the wall, a 1.80 m high blanket of earthfill was placed over the wall and tied into the upstream face of the dam to ensure complete seepage cutoff. A typical section is shown in Fig. 2. A geomembrane and geotextile barrier were then incorporated into the blanket to assure complete cutoff in the event that desiccation of the blanket occurs.

A1.9.6 Analyses performed

Limit equilibrium stability analyses were carried out on critical sections, e.g. maximum section, section with thickest caliche and alluvium deposits, section with deepest excavation or with high ground water, for the purpose to optimize the alignment of the wall. The model used was an infinitely long trench in two dimensions.

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A1.10 Brombach dam (Germany)

A1.10.1 Project description

Brombach dam is one of the storage facilities of a major water transmission project in the District of Weissenburg-Gunzenhausen, Bavaria, Germany, bringing water from the Danube Basin through the European principal divide to the Main Basin which drains into the river Rhine. Preciptation and runoff are much less in northern Bavaria than in the southern part. Fig. 1 shows that water stored first in the Altmuehl ring dam is brought through a 3 km long tunnel to the Brombach-Igelsbach reservoir and from there through existing and partly transformed rivers to the Regnitz-Main region with focus on the city of Nuremberg. Another transport route for water is to pump it along the Main-Danube navigation canal to the Roth reservoir from where it is distributed to the north as the need arises.

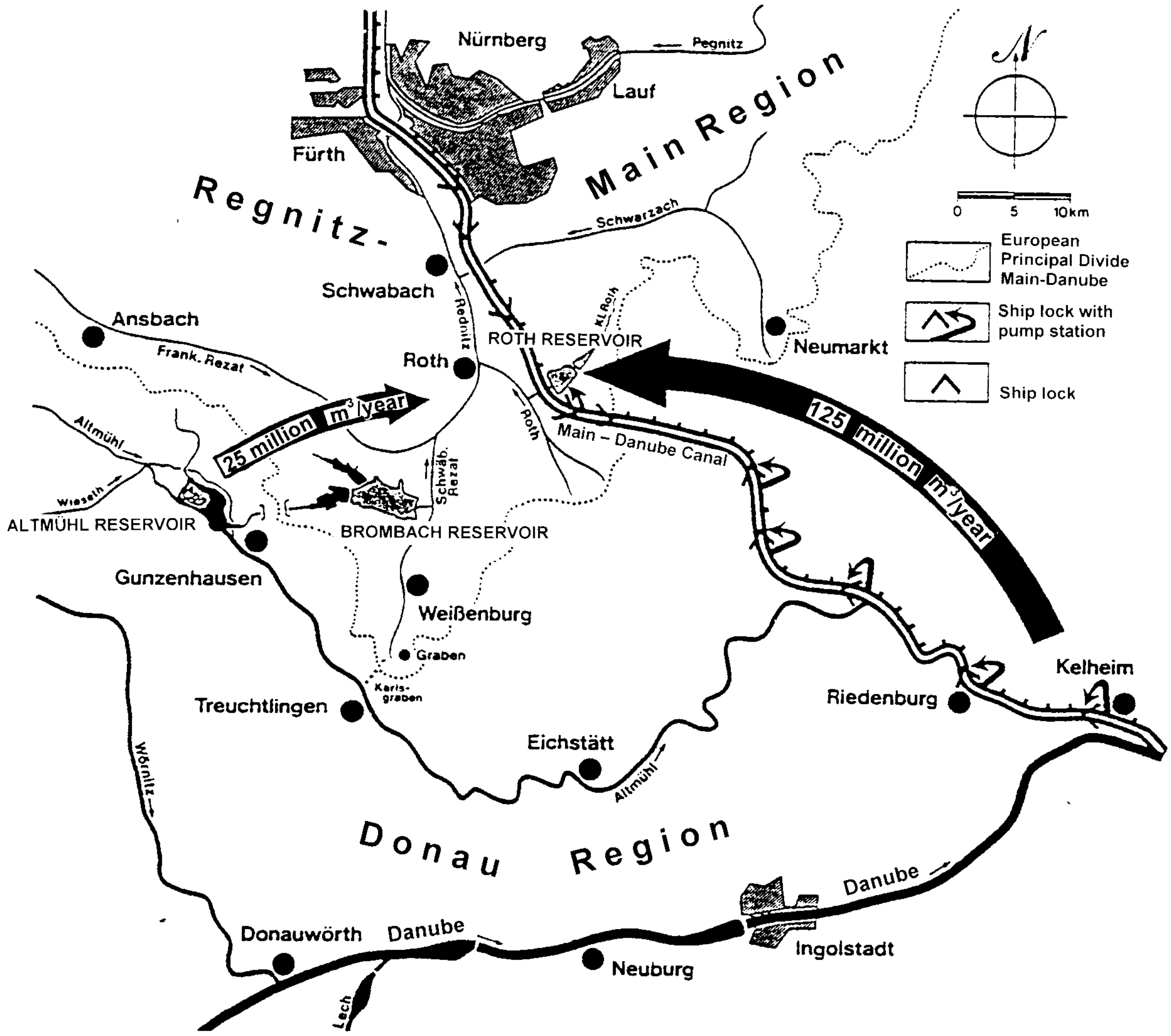
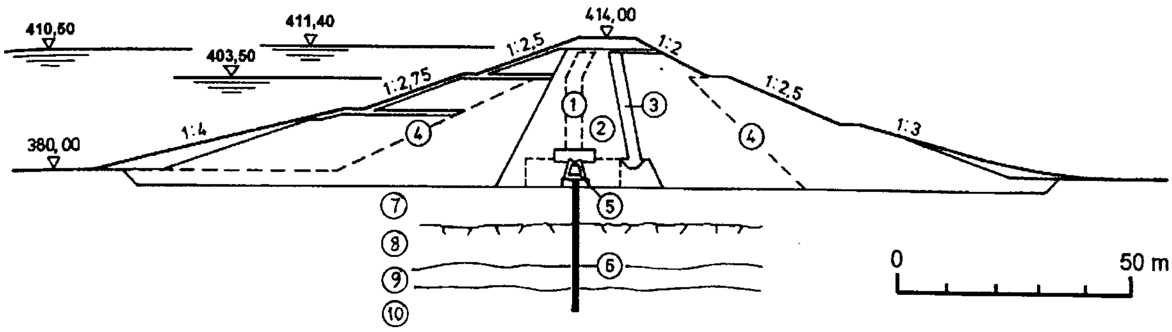


Fig. 1 Layout of Danube-Main water transmission project and location of Brombach dam

This project was completed in the mid 90s and comprises several earthfill dams with heights between 15 and 40 m. The Brombach reservoir contains three dams, namely the Brombach Main dam and secondary or pre-dams of lower height in the two upstream valleys, i.e. the Brombach and Igelsbach valleys. The water level in these secondary reservoirs is kept constant while that of the Brombach Main dam varies. Descriptions of the project were given by Beier & List (1982); Fenoux (1984), Strobl et al. (1985), and Strobl (1989, 1991a & 1991b).

The Brombach Main dam is an earthfill dam with a central impervious core of sandy silt. The embankment has a height of 37.3 m, a crest length of 1700 m and a volume of $3.5x10^6$ m^3. Construction lasted from 1983 to 1992 and operation started in 1993. Fig. 2 shows a section through the central part of the dam



1	Impervious core (improved part)	6	Diaphragm wall
2	Impervious core (normal part	7	River alluvium (sand and silt)
3	Downstream filter	8	Intermediate Burgsandstone
4	Shoulders	9	Mudstone
5	Inspection gallery	10	Lower Burgsandstone

Fig. 2 Brombach Main dam: Cross section with subsurface conditions

A1.10.2 Foundation conditions

Brombach Main dam and also the pre-dams are founded on sedimentary rocks of the Keuper formation, consisting of sandstone, locally known as "Burgsandstone", alternating with mudstone layers with a thickness of several meters. The sandstone is heavily jointed and fractured. The mudstone is impervious and acts as a natural groundwater barrier. In the river channel, the sandstone has been eroded up to 25 m deep and the valley bottom is now filled with alluvial deposits, mainly sand and silt. The permeability of these sediments is high with a coefficient of hydraulic permeability, k, of 10^{-4} m/s.

The stratigraphic units of the Keuper formation, relevant to the foundation of the dam are as follows (from top downward):

- *"Intermediate Burgsandstone"* from 10 to 14 m below excavation level

  This is a weakly cemented sandstone, very pervious, with a k-value of 10^{-5} m/s, mainly caused by secondary permeability (through the pores between the grains)

- *Mudstone* from 10 to 14 m below excavation level with hydraulic conductivities of $k \leq 10^{-7}$ m/s. This layer acts as a groundwater barrier.

- *"Lower Burgsandstone"* from between 14 and 34 m below excavation level. In this facies, the sandstone is better cemented and stronger. The permeability is governed by the discontinuities, mainly joints.

- *Mudstone* from 34 to 39 m below the bottom of the excavation. For practical purpose, this stratum is considered as the lower boundary of the dam foundation model.

The groundwater table was found to be 8.5 m higher in the Intermediate Burgsandstone than in the Lower Burgsandstone.

The material characteristics of the undisturbed Keuper sandstone are:

Unit weight:	21 kN/m^3	
Grain size:	≤ 0.06 mm:	6 to 50 % by weight
	≤ 2 mm	95% by weight
Uniaxial compressive strength:	range: 0 – 8 MPa, average: 2 MPa	
Elastic modulus:	range: 60 – 2000MPa, average: 600 MPa	

A1.10.3 Foundation treatment

Field test:

The sandstone below the Brombach and Igelsbach pre-dams was treated with cement grouting. There is a three row grout curtain in the sandstone below the Igelsbach dam and a one row curtain under the Brombach pre-dam. The latter was later reinforced by jet grouting. Cement grouting was only partially successful (Strobl et al, 1985). Water pressure tests in the untreated sandstone resulted in Lugeon values between 10 and more than 100. The sealing target was set to 10 Lugeon, but even with a three-row curtain with a grout hole spacing of 1.50 m in the center row, only about one half of all the water pressure tests carried out after completion of the grouting program satisfied the target value.

For the Brombach Main dam a field test employing three different sealing methods was carried out to find the most suitable sealing method. These methods included:

- a three-sided wall of overlapping piles
- a three-row cement grout curtain closing the test field formed by the three-sided pile wall on the fourth side
- a soil-cement diaphragm wall using a Solétanche hydromill

The 35 m deep pile wall was specified to have a minimum thickness of 400 mm and a maximum deviation of a pile from the vertical of 1 percent. Construction of the wall revealed that up to 20 m depth the deviation could be kept below one percent, while at greater depth some piles started to deviate and gaps developed. The pile diameter was about 0.9 m.

Grouting was first carried out with a limiting pressure of 6 bars up to a depth of 15 m Thereafter, the grouting pressure was increased to 10 bars. The spacing of the grout holes was 1.5 m.

The efficiencies of these two sealing approaches were measured by means of pumping tests in a well located inside the test field and piezometers on both sides of the pile and grout curtain barriers. By comparing the drawn-down curves before and after installation of the barriers, it could be seen that the reduction on drawdown on the side of the pile wall was much larger than on the side of the grout curtain, thus demonstrating the superior efficiency of the pile wall. A more efficient grout curtain would have required additional sealing with silica gel, which, however, would have increased the costs significantly.

Sealing by means of a diaphragm wall was tested in a 100 m long and 35 m deep trench in the vicinity of the pile wall/grout curtain test field. The diaphragm wall was 600 mm thick and excavated by a Solétanche hydrofraise. The deviation from the theoretical position amounted to a maximum of 3 per mil of the excavation depth. The trench cutter enables correction of its frame position during milling by means of hydraulic presses.

The mix proportions used for the soil-cement backfill were the same as those employed for the pile wall, namely:

- Cement (blast furnace) 200 kg/m^3
- Clay powder 100 kg/m^3
- Sand (0-2 mm) 950 kg/m^3
- Gravel (2-8 mm) 450 kg/m^3
- Water 350-400 kg/m^3

This concrete mixture had a unit weight of 20.6 kN/m^3.

Among the three sealing systems only the diaphragm wall met all the requirements with respect to water tightness and dimensional accuracy for a cutoff below the Brombach Main dam. A comparison of the relevant data of the three sealing methods resulting from the field test is given in Table 1. Note that with a second stage grouting employing silica gel, the unit cost for the grout curtain would have increased to about US $200.

Table 1 Comparison of three different sealing methods (based on a field test carried out in 1984)

Parameter	Cutoff wall excavated by trench cutter	Overlapping pile wall	Three-row grout curtain
Total area of sealing structure (m^2)	2700	1020	600
Work output (m^2/day)	115	26	8
Cost of site facilities (US $)	110,000	20,000	20,000
Actual unit costs (US $/m^2)	150	170	120
Deviation from theoretical position (% of depth)	0.1-0.3	0.2-2.1	0.6-7.7 (25-degree inclined boring)

Cutoff wall for Brombach Main dam

Water pressure tests in the Keuper sandstone underlying Brombach Main dam gave Lugeon values between 1 and 230. A diaphragm wall was found to be the preferred solution after other sealing measures had been evaluated. The wall has a thickness of 0.60 m, a maximum depth of 35 m, and a total area of 40,000 m^2 was excavated by a Bauer hydromill. The extent of the sealing area is shown in Fig. 1. Taking into account the position of the different mudstone layers, it was tried to minimize the extent of the wall. Milling performance was dependent on the strength of the sandstone. Sandstone well cemented by quartz or lime, with an unconfined compressive strength of about 20 MPa, reduced the milling performance significantly. Still, by adjusting the number and position of the cutting wheels it was possible to achieve a mean daily output of about 200 m^2 at an average price of about 200 US$/$m^2$.

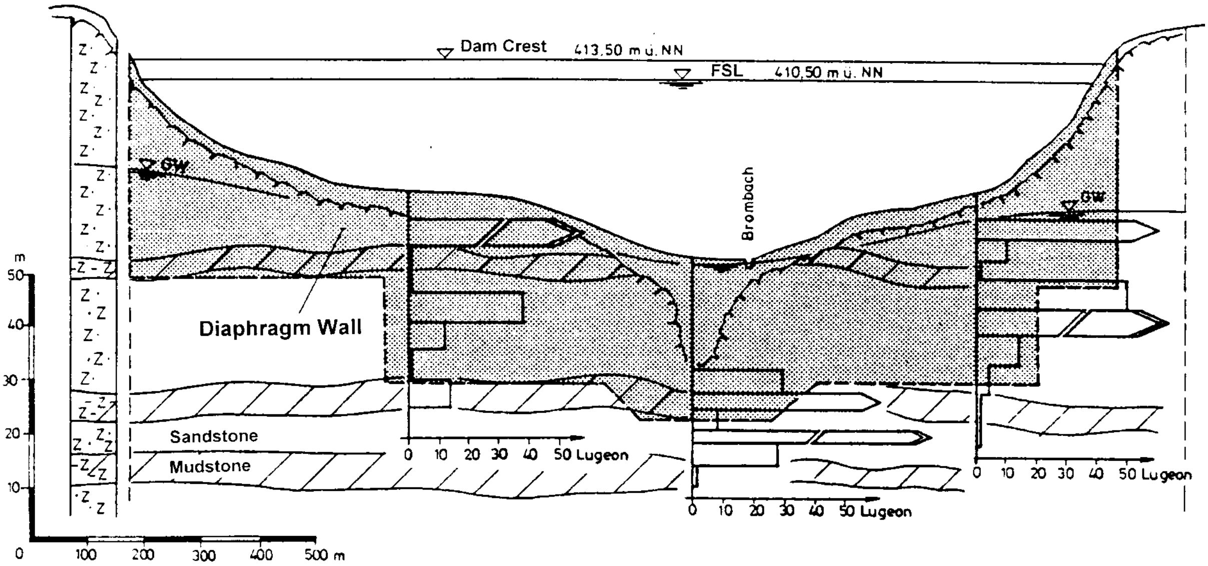


Fig. 3 Longitudinal section along dam axis showing the extent of the diaphragm wall. Also shown are results of water pressure tests. The interrupted pointed bars indicate very high Lugeon values

The proportioning of the concrete mix was the following:

- Cement (blast furnace) 200 kg/m3
- Finely ground fire clay 100 kg/m3
- Very fine sand 100 kg/m3
- Aggregates (0/8 mm) 1300 kg/m3
- Water 360 kg/m3

With this mix the hydraulic conductivity of the wall was measured as 10^{-9} m/s and the unit weight was 19.6 kN/m^3. Excavation was by the panel method. Primary panels had a length of about 7.0 m and secondary panels 2.7 m. The overlap was at least 0.20 m. Before backfilling the concrete, each panel was tested by echo-sounding for possible deviation from the theoretical position. Deviations were in general less than 3 ‰ of the excavated depth. In some of the panels where large fissures were present in the sandstone, the supporting fluid was suddenly lost. In such a case the fissure was closed by filling sand into the excavation. Minor fluid losses in the trench were corrected by adding cement.

To test the cutoff wall, several holes were drilled into the wall at about 100 m intervals and water pressure tests conducted. At different elevations above the groundwater table, horizontal cracks could be observed. It is believed that these were caused by the rapidly changing permeabilities of the sandstone and the high water/cement ratio of 1.8:1 which led to different setting times of the concrete producing an arching effect. A lower w/c value is recommended and as well a high bentonite content in the supporting fluid, i.e. at least 50kg/m^3.

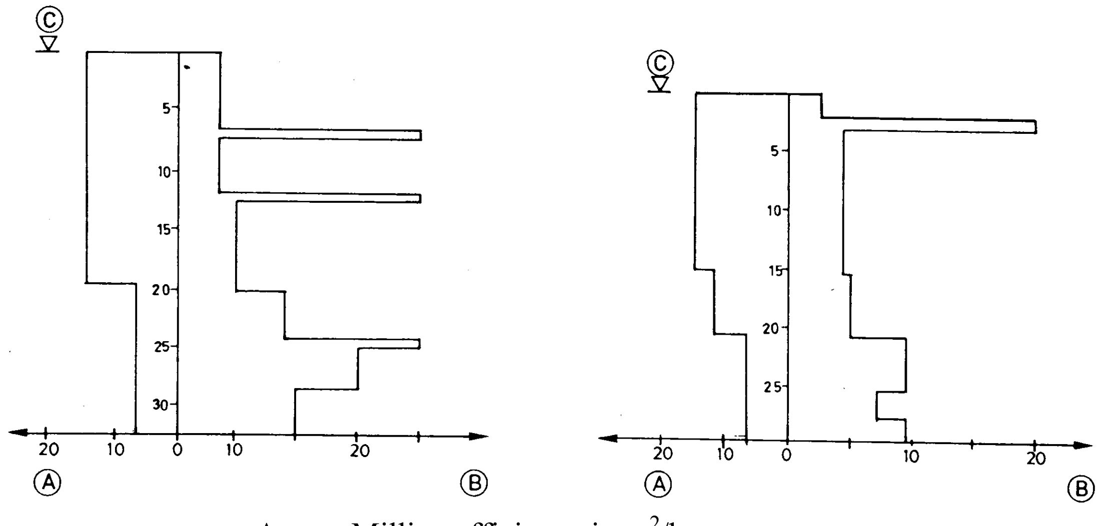
Determination of milling performance

The milling rate of the trench cutter depends on the uniaxial compressive strength of the rock, the degree and orientation of jointing, and the abrasiveness of the rock. For large projects in rock, a field test is therefore always recommended to enable an assessment of the response of the rock to milling. At the Brombach site, the construction of the test wall section was accompanied by a drilling test monitored with ENPASOl, developed by Solétanche. The millability of the rock strata having different hardness could then be predicted from the ENPASOL data, at least qualitatively. ENPASOL gives the following parameters: torque, surcharge, rotational speed and the time for penetration of 5 mm. With these quantities the so-called CWT-value can be calculated:

CWT = Torque x rpm x penetration rate

where rpm is the rotational speed. The CWT value is a decisive parameter for judging the millability of the rock. A Milling efficiency in m^2/hour
B CWT-value x 10^3
C Ground surface

Fig. 4 shows the variation of the CWT-value and the milling efficiency with depth for two boreholes. It can be seen that the milling efficiency below about 20 m depth is reduced significantly. The production rate there may drop to values as low as 0.4 m^2/hour.

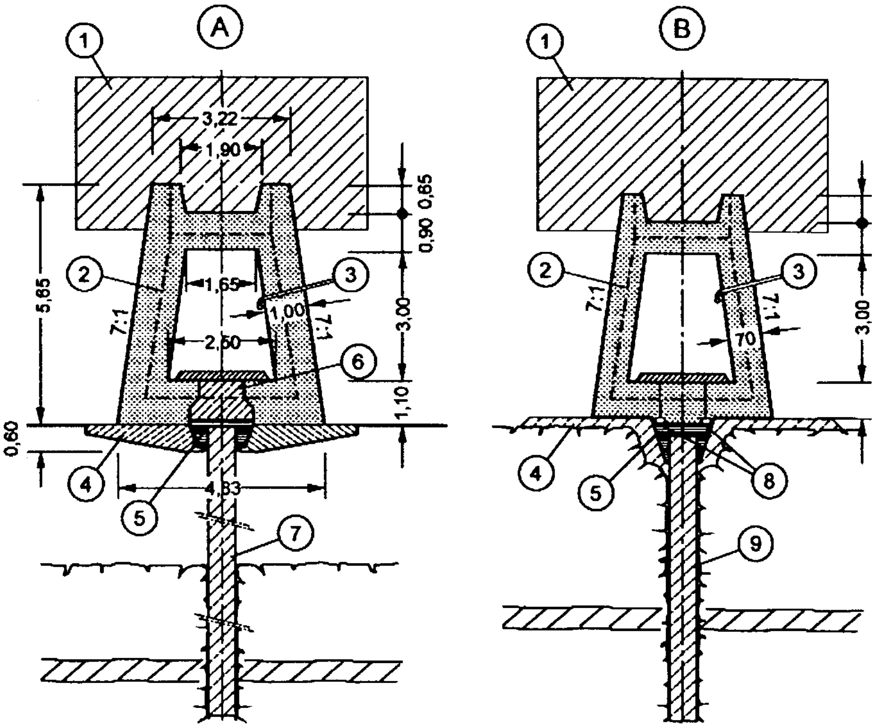


A Milling efficiency in m^2/hour
B CWT-value x 10^3
C Ground surface

Fig. 4 Variation of milling efficiency with depth as a function of the CWT-value

A1.10.4 Connection of wall with embankment

The embankment is provided with an inspection gallery at the base of the core (Strobl & Leininger, 1987) and the diaphragm wall is connected to the base of this gallery, as shown in Fig. 5. The gallery is founded on river alluvium (Section A) and will settle under the load of the embankment. The final connection between the cutoff wall and the gallery is made by an asphalt mastic cap covering the top of the wall. This cap is poured after completion of dam construction when most of the settlements have taken place. The reinforced concrete beam is then cast to close the gallery. The asphalt mastic is compressible and also allows sliding along the wall. The design provides another 0.5 m of settlement during the post-construction phase.



A	Cross section in central part of dam	5	Asphalt mastic (poured after end of construction
B	Cross section on abutment	6	Concrete (reinforced)
1	Compressible zone of impervious core	7	Diaphragm wall (reinforced on top)
2	Sealing band for joint	8	Steel cast
3	Drainage tube	9	Diaphragm wall
4	Bedding concrete		

Fig. 5 Connection of diaphragm wall to inspection gallery

On the abutment (Section B), the gallery is founded on rock and settlements will be small. On top of the gallery there is a zone of the impervious core which is more compressible than the other parts of the core. This compressible portion has a cushioning effect and will distribute the load to the sides of the gallery. Without the cushion the rigid gallery would tend to attract load.

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A1.11 Tadami dam (Japan)

A1.11.1 Project description

The Tadami hydro-power plant with an installed capacity of 65 MW is located on the Tadamin river in Fukushima Prefecture. It was completed in 1988. The dam consists of a main dam, about 18 m high and 354 m long, and a 24m high and 154.5 m long side dam on the right hand part with a 75 m long spillway structure in between. Both dams are embankments with a central impervious core and rockfill shoulders (Fig. 1). The project has been described by Nishigori & Takimoto (1991). A cross-section of the embankment and a longitudinal section of dams and spillways are given in Fig. 2 and Fig. 3 respectively.

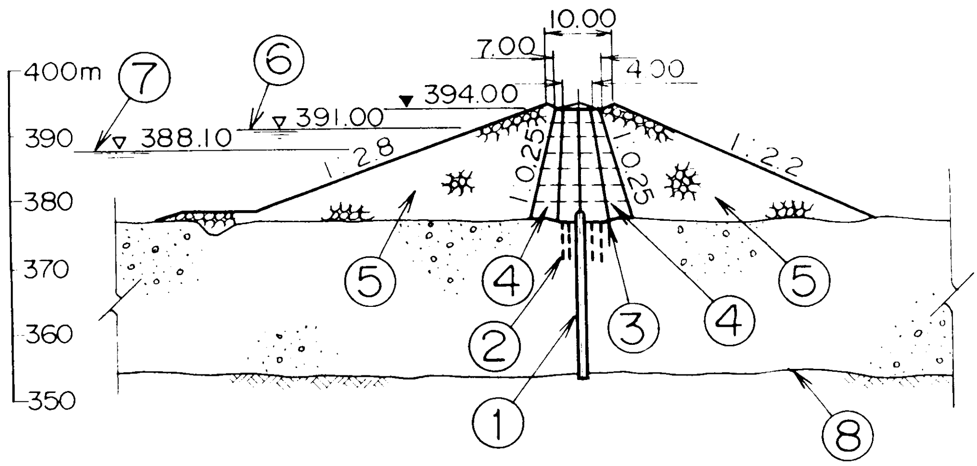
Fig. 1 Tadami dam and spillway as seen from downstream right bank

A1.11.2 Foundation conditions

The main dam is founded on approximately 20 m thick alluvium of variable granulometry, while the side dam is placed directly on bedrock with the thin alluvial deposit excavated. The sandy-gravelly alluvium has a void ratio of about 0.25 and an average deformation modulus, determined by plate load tests, of around 170 MPa. Other properties include: max. particle size ~ 600 mm (only few boulders were encountered), coefficient of hydraulic conductivity, $k=10^{-5}$ to 10^{-4} m/s.

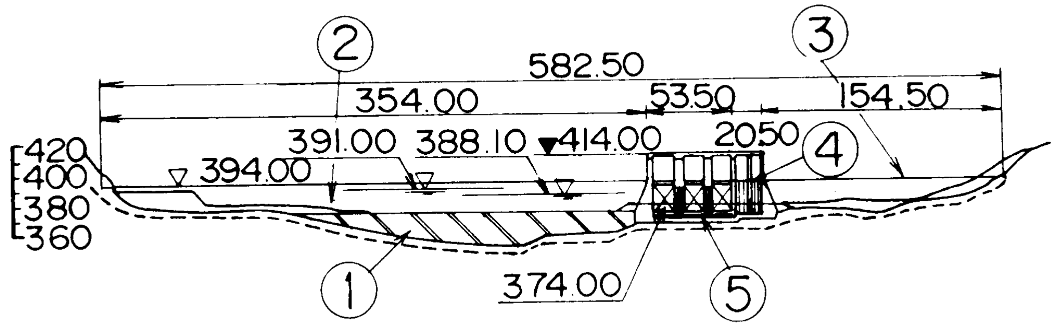
A1.11.3 Foundation treatment

Grouting the alluvium along its full depth was not considered a feasible option because at some locations with k-values of 10^{-5} m/s difficulties were expected. Hence, the decision was taken to construct a diaphragm wall of plastic concrete. This was a pioneering project and one of the first applications of plastic concrete for cutoffs in Japan where I.C.O.S. type walls (see Chapter 5) with rigid concrete elements had been used widely.



1	Plastic concrete diaphragm wall	5	Rockfill zone
2	WMC grout blanket	6	High water level
3	Impervious core zone	7	Low water level
4	Filter zone	8	Rock surface

Fig. 2 Typical cross section of Tadami main dam

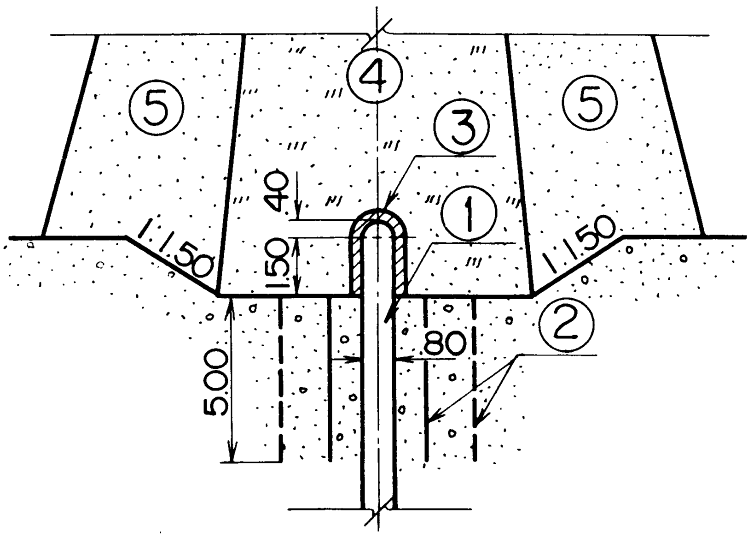


1	Plastic concrete diaphragm wall	4	Intake
2	Main dam	5	Spillway
3	Side dam (right bank)		

Fig. 3 Longitudinal section of Tadami dam

The wall is 0.80 m thick and has a maximum depth of 23 m. The wall area is about 400 m^2. At its base it penetrates about 1.5 m into bedrock and at its top about 1.50 m into the impervious core. Details are shown in Fig. 4.

Along the cutoff wall the top of the alluvial foundation was reinforced by blanket grouting to 5 m depth using WMC (wet micromilling cement) grout.



1 Plastic concrete diaphragm wall
2 WMC grout blanket
3 Highly plastic material
4 Impervious core
5 Filter zone

Fig. 4 Details of the top part of the diaphragm wall

A1.11.4 Cutoff wall construction

Construction of the diaphragm wall was accomplished by the method of primary and secondary panels of length 5.475 m and 6.150 m, respectively. Excavation was by clamshell, preceded by rock auger to improve efficiency.

Placement of the concrete was by means of tremie pipes. Initially the tip of the pipes had to be 0.25 m above the bottom of the trench and later kept immersed at least 2 m in the concrete at all times.

Of interest are the results of a field test where the responses of a rigid and of a plastic concrete wall to fill placement were compared. A 10.4 m by 4.1 m rectangular enclosure of 10 m deep wall elements was constructed (see Fig. 5 a and b). An eight meter long section was made of conventional concrete, the rest was plastic concrete with a previously selected mix design. The tip of the walls penetrated into bed rock and the top ends were embedded in impervious core material. An embankment fill was then placed over the wall enclosure and the settlements monitored.

Fig. 5c shows the results of settlement monitoring at four points at the base of the core material and at the top of the walls. Settlements are given when the embankments had reached a height of 2.1m, 4.1m, 6.6m and 10.0 m. It can be seen that the rigid wall does practically not settle and attracts most of the load. The permeability was tested for different conditions and the order of magnitude was around 10^{-8} m/s.

A1.11.5 Quality control

Detailed measurements were made for specific gravity, viscosity of the bentonite slurry, and of the surface moisture of the fine aggregate. For the plastic concrete the main properties checked were: slump, compressive strength and permeability. Results of quality control tests are given in Table 1.

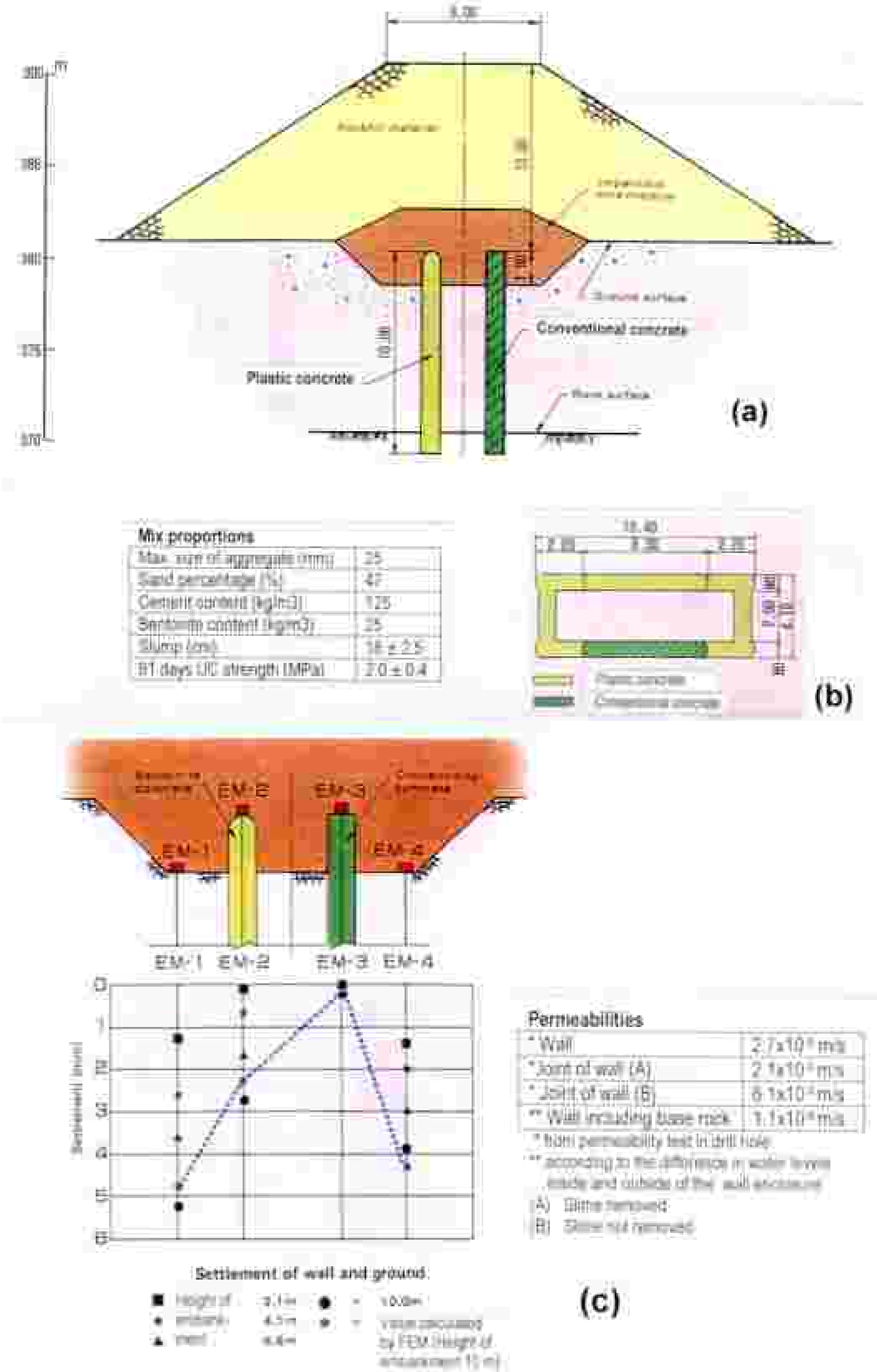


Fig. 5 Field test for feasibility of plastic concrete diaphragm wall: (a) Section through embankment fill with walls of two different materials, (b) cutoff walls in the shape of rectangular enclosure and mix proportions for plastic concrete, (c) test results, settlements and permeabilities.

Table 1 Results of quality control tests of plastic concrete

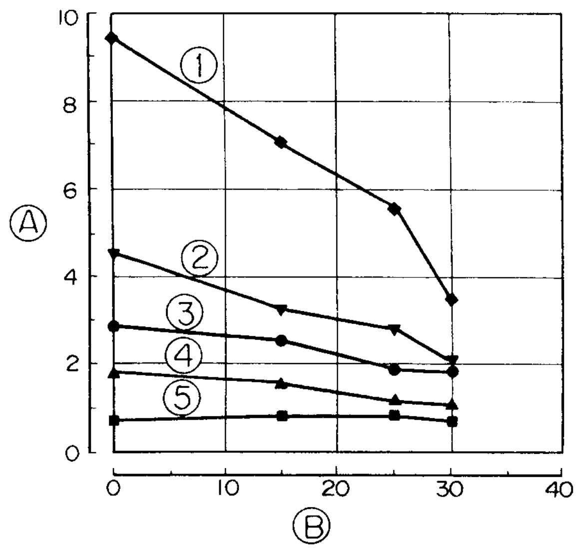
Type of test	Frequency of test	Reference values	Test results		
			Min.	Max.	Average
Unit weight (kN/m^3)	3 samples per element	-	21.4	21.9	21.6
Slump (cm)	Once per 60 m^3 or at least one test per element	18 ± 2.5	16.0	18.9	18.1
Strength (MPa)	3 samples per 60 m^3 or per element	2 ± 0.4	1.73	2.30	2.06
Coeff. of hydraulic conductivity, k ($x10^{-8}$ m/s)	Once per element	1.0	0.12	0.77	0.44

In addition, the permeability of the plastic concrete was also verified from water pressure (packer) tests, carried out in 66 mm diameter test borings drilled into the completed wall. An average k-value of $4.4x10^{-9}$ m/s was found.

A1.11.6

A1.11.7 Mix design of plastic concrete

A deformability of the wall similar to that of the alluvial material surrounding the wall was envisaged. Bentonite was employed to reduce the stiffness of the concrete mix. For mix design trials the following ranges of cement and bentonite proportions were considered: cement: 75 to 200 kg/m^3 and bentonite: 0 to 30 kg/m^3. The water-cement ratio, w/c, varied between 1.2 and 3.5. Fig. 6 shows test results illustrating the influence of cement and bentonite proportions on the uniaxial (unconfined) compressive strength of the plastic concrete.



A Unconfined compressive strength (MPa)
B Bentonite content (kg/m^3)
1 Cement content 200 kg/m^3
2 Cement content 150 kg/m^3
3 Cement content 125 kg/m^3
4 Cement content 100 kg/m^3
5 Cement content 75 kg/m^3

Fig. 6 Influence of concrete and bentonite proportions in plastic concrete on the uniaxial compressive strength

Table 2 summarizes numerical results obtained for uniaxial and triaxial compressive strength tests. The deformation moduli were derived from triaxial stress-strain curves and the E_{50} elastic moduli from the uniaxial tests (secant modulus at 50% failure strain). The values of Table 2 are displayed graphically in Fig. 7. It can be seen that deformation and elastic modulus decrease with increasing bentonite content, i.e. the concrete becomes softer. Friction angle and cohesion increase with decreasing bentonite content for a given cement content.

Table 2 Mix design: Strength and moduli for various proportions of cement and bentonite

Mix proportion	Uniaxial compressive strength (MPa)	Elastic modulus, E_{50} (from UC test) (MPa)	Cohesion intercept (from triaxial test) (kPa)	tan φ (from triaxial test)	Deformation modulus (from triaxial test) (MPa)
C125-B0	4.6	4200	1000	0.748	790
C75-B15	1.5	710	285	0.654	460
C100-B15	2.3	1700	476	0.845	500
C125-B15	2.7	2500	697	0.62	480
C150-B15	3.0	2600	801	0.649	560
C50-B25	0.7	640	125	0.748	150
C75-B25	1.3	740	300	0.748	250
C100-B25	1.9	940	375	0.748	200
C125-B25	2.5	1500	387	0.708	360
C150-B25	2.9	2100	784	0.527	400
C200-B25	3.2	2500	980	0.473	470
C100-B35	1.4	1000	346	0.577	290
C125-B35	2.0	1500	426	0.601	250
C150-B35	2.4	2100	669	0.615	350

Note: C = cement content (kg/m3), B = bentonite content (kg/m3) All test results are from samples cured for 91 days

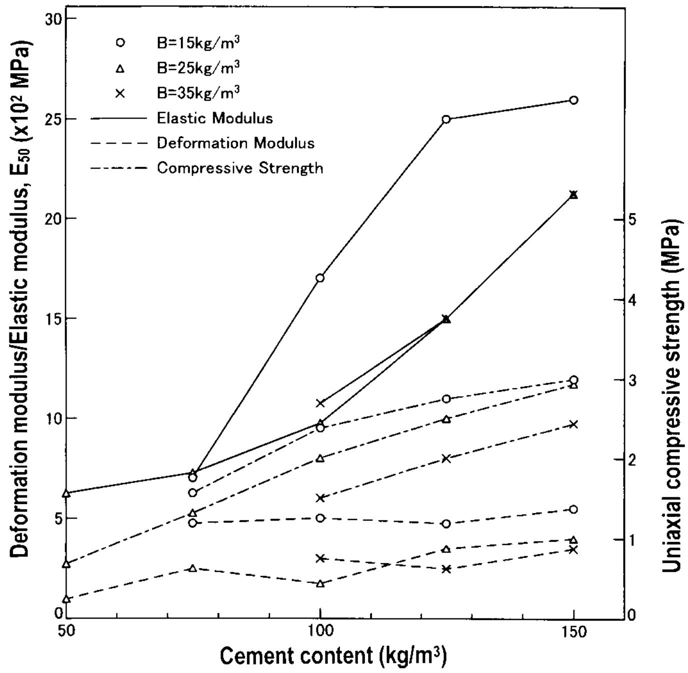
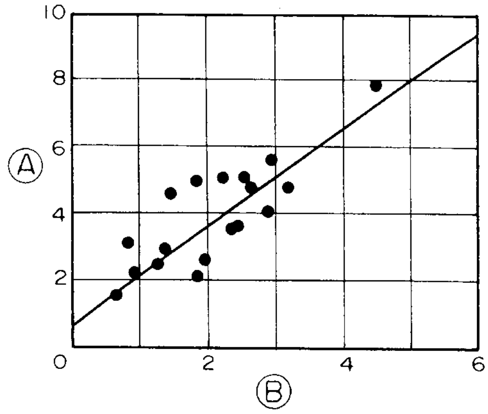


Fig. 7 Properties of plastic concrete for different proportions of cement and bentonite

The relationship between uniaxial compressive strength and deformation modulus at a curing age of 91 days is shows in

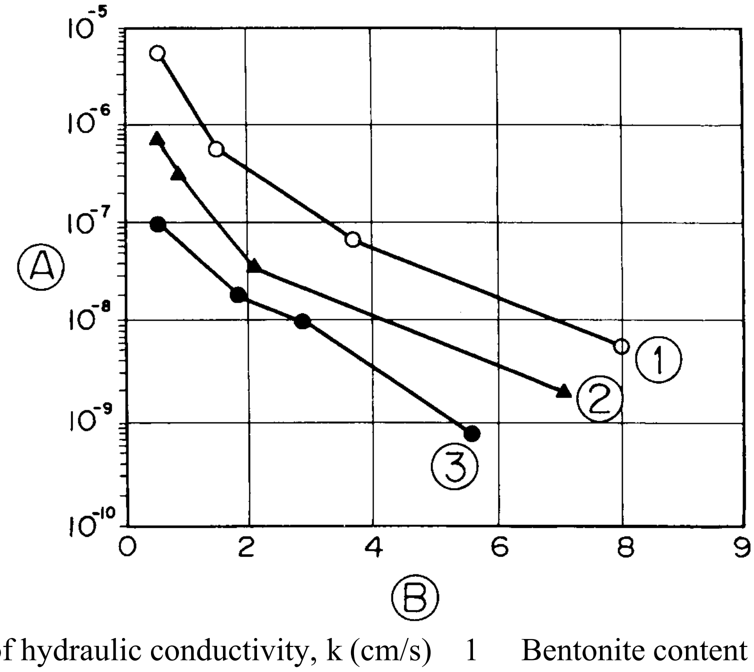
Fig. 8. The deformation modulus was determined from triaxial compression tests at a confining pressure of 0.4 MPa.



A Deformation modulus, E_{50} (x10^2 MPa)
B Uniaxial compressive strength (MPa)

Fig. 8 Relationship between uniaxial strength and deformation modulus

The permeability of the plastic concrete decreases with increasing compressive strength as illustrated in Fig. 9. Adding bentonite shifts the relationship downwards, i.e. to lower k-values. Hence, bentonite makes the concrete more porous.



A Coefficient of hydraulic conductivity, k (cm/s)
B Uniaxial compressive strength (MPa)

1 Bentonite content 0 kg/m^3
2 Bentonite content 15 kg/m^3
3 Bentonite content 25 kg/m^3

Fig. 9 Relationship between uniaxial strength and coefficient of hydraulic conductivity

A standard mix design for the plastic concrete used for the Tadami cutoff wall is listed in Table 3. Variations of this mix were made to adjust to the variable stiffness of the alluvium encountered along the cutoff wall.

Table 3 Standard mix used for Tadami dam

Water-cement ratio, w/c			2.23
Sand percentage S/(S+G) (%)			46
Cement	(kg/m^3)	C	125
Bentonite	(kg/m^3)	B	25
Water	(kg/m^3)	W	279
Fine aggregate	(kg/m^3)	S	792
Coarse aggregate	(kg/m^3)	G	928

A1.11.8 Performance of cutoff wall

The deformation of the diaphragm wall below the Tadami main dam was instrumented with strain gauges, earth pressure cells and vertical and horizontal displacement gauges, as shown in Fig. 10.

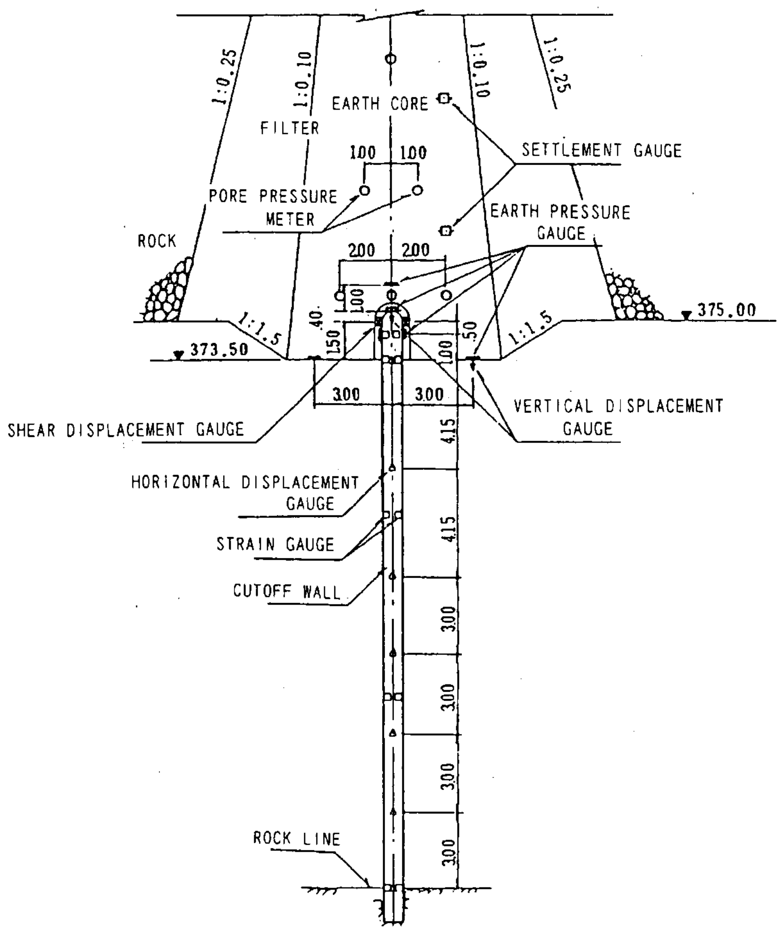


Fig. 10 Typical instrumentation of cutoff wall

At the deepest point the wall had settled as follows:

after completion of embankment fill placement	13.7 mm
after completion of first reservoir filling	15.1 mm
at high water level	14.9 mm

The ground around the wall settled around 4 mm less than the wall. Back analysis of the observed settlements resulted in a deformation modulus in the alluvial deposits in the range of

220 to 400 MPa. The maximum horizontal displacement was about 17 mm at 2 m depth and with full reservoir.

References

Nishigori, T. & Takimoto, J., 1991. Foundation treatment for alluvial deposits at Tadami dam. *Trans. 17th ICOLD*, Vienna, Q.66, R.24, 3:417-436.

A1.12 Péribonka dams (Canada)

A1.12.1 Project description

The Péribonka Hydroelectric Project is located in the province of Québec to the north of Lac Saint-Jean (see Fig. 1). The Péribonka Reservoir is created by the Main dam located around kilometre 152 on the Péribonka River, and by two dikes located less than 2 km north of the Main dam. The dam site is located immediately upstream of the confluence of the Manouane River with the Péribonka River.

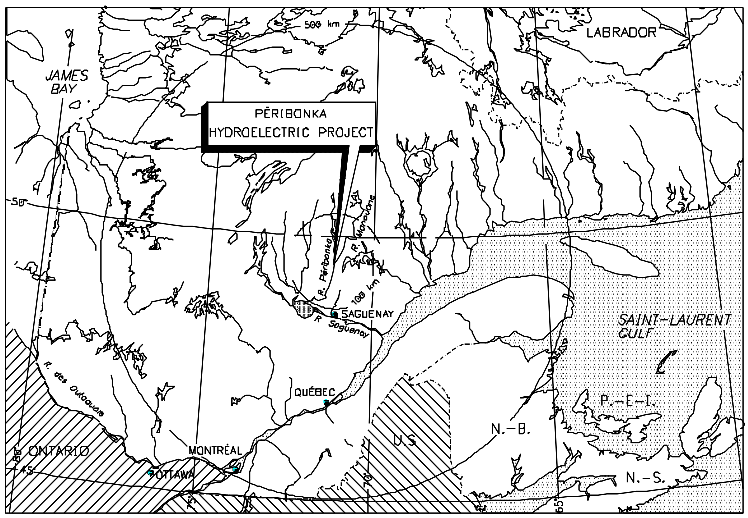


Fig. 1 Location of Péribonka Hydroelectric Project

The Péribonka Hydroelectric Project includes components as shown in Fig. 2:

- a rockfill dam with a till core 80 m in height, a crest length of 775 m and a total embankment volume of 3 900 000 m³;
- a rockfill dike A with a till core 26 m in height, a crest length of 560 m and a total embankment volume of 510 000 m³;
- a sand and gravel dike B with a till core 13 m in height, a crest length of 205 m and a total embankment volume of 59 000 m³;
- a reservoir 35 km in length, an average width just under 1 km and an area of 32 km² at maximum operating level of 244,2 m;
- a temporary diversion tunnel with a capacity of 2 005 m³/s;
- a three-gated spillway with a maximum capacity of 5 300 m³/s;
- an underground powerhouse equipped with 3 Francis turbines for a total capacity of 385 MW and an average annual production of 2,2 TWh.

The main feature of the Péribonka project with respect to foundation treatment is the construction of three plastic concrete cut-off walls. Two of these cut-off walls were constructed under the main dam and the other under dike A. For the main dam in the river valley, foundation treatment also included vibrodensification and sleeve pipe (Tube à

Manchette) alluvium grouting. Detailed accounts of the various foundation treatment methods can be found in Bigras et al. (2005), Lauzon et al. (2006), Maeyens et al. (2006) and Ouellette et al (2007). Balian (2007) and Heinrich (2007) describe the construction method of the wall from the Contractor's point of view.

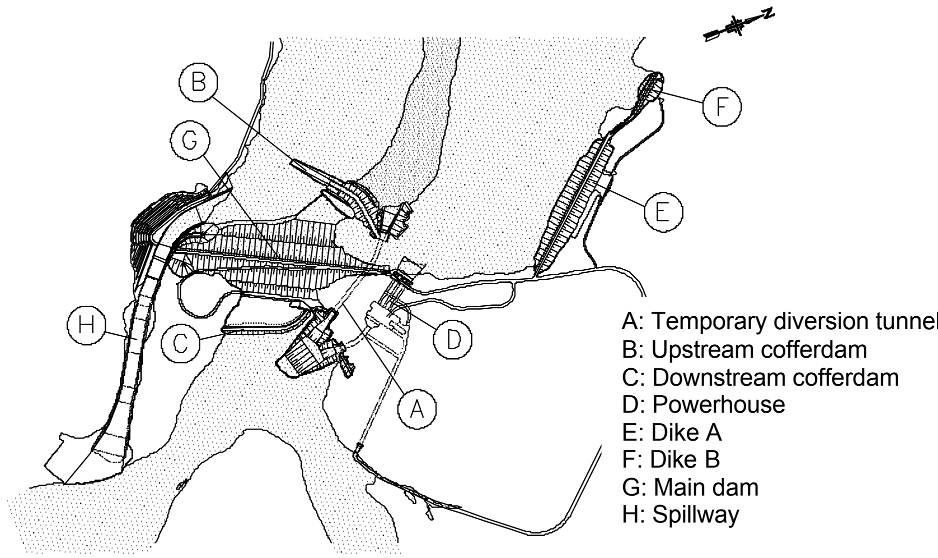


Fig. 2 General layout

Table 1 shows the real and the estimated surfaces of cut-off walls.

Table 1 Estimated and actual surfaces of cut-off walls

	Estimated	Actual
Surface of cut-off walls in bedrock	1 080 m^2	1 695 m^2
Surface of cut-off walls in alluvium	19 850 m^2	26 635 m^2
Maximum depth	126 m	116 m

A1.12.2 Main dam: Foundation conditions

The Peribonka River flows in a valley bordered by sand deposits. The geological conditions of the foundations at the dam site are very complex. Geological and geotechnical investigations at the feasibility stage revealed a deep buried glacial gully some 110 m deep below river level. The materials consist of a very heterogeneous mix from fine and silty sands to a variety of gravels, cobbles and boulders. The permeability was estimated to vary from 10^{-2} to 10^{0} cm / sec. The need for a cut-off wall to sound rock was obvious.

The right bank of the river valley contains a buried gully some 42 m deep filled with alluvium. Rock outcrops at the two abutments as well as on a hill that separates the secondary valley from the main river course (Fig. 1).

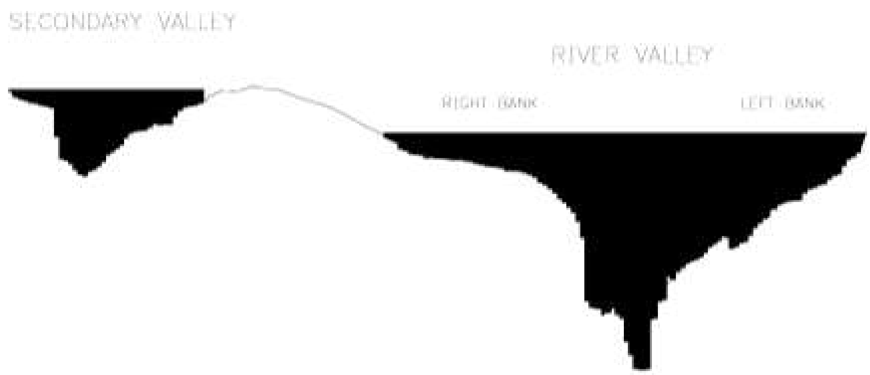


Fig. 3 Location of the river and secondary valleys on the dam axis as viewed from downstream.

A1.12.3 Main dam: Foundation treatment

Scope of work

The safe construction of the cut-off wall in this heterogeneous, pervious, unconsolidated and extremely variable alluvial ground was an interesting challenge. The construction of this dam required that the foundation soils in the river valley be densified to improve their behaviour under the stresses imposed by the dam embankment.

Vibro-compaction treatment in the river valley

Backfilling of the riverbed to elevation 180,0 m was done with sand and gravel hauled from nearby natural sand and gravel pits. About 800 000 cubic meters of material were required to build the platform to its working level. Backfilling was performed in 2 lifts. The first one brought the platform about 1 meter above the average water level of 174 m. The second lift brought the platform to its final grade of 180 m.

The area to be compacted was subdivided into zones according to the depths of treatment requirements. The central area is a band of 30 meters on each side of the dam centreline where the treatment depth is 35 m below the river bed, which corresponds to a maximum of roughly 48 m below the working platform. Elsewhere, the requirements were for treatment depth of 10 m below the riverbed corresponding to a maximum depth of 25 m below the working platform. There were 2 10-meter wide transition zones between the central zone and the side zones where the treatment depth is 23 m below the riverbed, corresponding to a maximum depth of 36 m below the working platform. Fig. 4 shows a cross section of the platform and of the riverbed with the treatment depth levels. Considering the grain size distribution of the coarser layers and the depths to be reached, the use of vibro-compaction in such natural soils was seen as quite a challenge. The possibility that the vibro-probes would not penetrate to the desired depth was considered and minimum treatment depths were predefined in order to be able to face any eventuality.

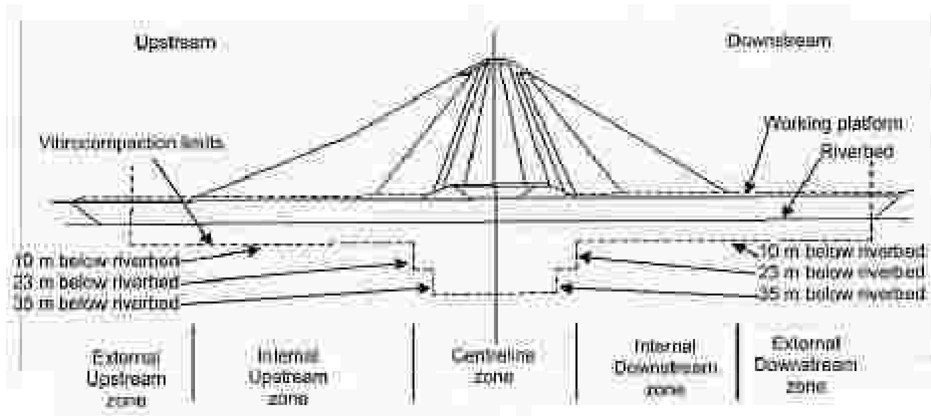


Fig. 4 Cross section of the dam showing the foundation treatment limits

Vibro-compaction was performed following a triangular pattern. The grid spacing used was 3.5 m for the hydraulic probe and 4.2 m for the electric probe. A smaller grid was developed for the central zone where the electric probe was used. The second phase consisted of putting down the vibroflot on a split spacing pattern relative to the original grid (phase 1). In the deepest treatment areas, the first phase points created large settlements and loosened the surrounding compacted soils. By operating in 2 phases, it was possible to generate a first densification that would be enough to ensure that this de-compaction phenomenon wouldn't take place during the second phase.

Dynamic compaction

In order to accelerate the work, a part of the platform was densified by dynamic compaction. Areas with less than 10 meters to be improved, which were mostly located along the river banks, were delimited. Dynamic compaction was performed with a 17 000 kg square tamper, dropped from a height of 20 m. A final levelling pass, with a 12 000 kg square tamper dropping from 12 m completed the compaction.

Densification control and results

Compaction was controlled with cone penetration testing (CPT). To test the compaction below the level where the 20 ton capacity cone met refusal, a special testing technique had to be used. It consisted of drilling a borehole below the refusal level and testing with an iCPT. The iCPT is an "interval CPT", which is basically a CPT equipped to memorize the data during the test. This set up permitted to achieve the testing in the deeper treatment areas in the center of the dam.

To have an overall view of the densification achieved, a geophysical survey was performed over the central area using the Modal Analysis of Surface Waves (MASW). The measurements before and after the treatment showed a clear increase in the seismic velocities over the treated depths. Seismic velocities in the range of 150 to 250 m/s before vibro-compaction, increased to 250 to 350 m/s after the treatment.

A summary of the depths attained in each area is listed in Table 2.

Table 2 Treatment depths achieved

Zone	Average target depth (m)	Average minimum depth (m)	Average treated depth (m)	Treated depth (m) 10th percentile	Treated depth (m) 90th percentile
External upstream	19.8	14.5	16.2	11.3	21.0
Internal upstream	19.4	14.4	19.3	10.6	24.8
Center line	33.2	26.5	30.4	15.4	44.6
Internal downstream	18.1	16.7	17.2	11.1	21.3
External downstream	16.6	15.0	16.9	11.8	20.9

Considering the expected penetration difficulties, especially in the upstream and central zones, the resulting depths of penetration were considered very satisfactory. The minimum treatment depths were attained 95% of the time, and the target treatment depths were achieved 70% of the time. A depth of 52 m was reached with vibro-compaction through in-situ soils: the challenge was not only to reach that depth but to go through layers of gravel and cobble. The objectives of improvement in the densification of the foundation soils have been met, thus improving the long-term behaviour of the dam.

Alluvium grouting with sleeve pipes (Tubes à Manchettes - TAM) in the river valley

The objective was to ensure that no collapse would occur and that no excessive losses of bentonite slurry would occur through the very pervious layers while excavating the trench for the cut-off walls with a hydro-mill / cutter. The other major challenge was to determine the exact rock profile in order to ensure that the cut-off wall would be properly anchored in sound rock and to help with the planning of the cut-off panels' construction sequence.

It was decided to pre-treat the ground by grouting. The Tube à Manchettes (TAM) grouting technique was favored over gravity grouting, particularly as it is possible to re-grout the tubes as many times as needed, without having to re-drill holes. The other advantage of grouting was that the profile of the rock could be determined at the same time. Rock grouting too was done through the TAM holes. The grouting operation was not designed to make the ground impervious, but rather to consolidate it, to avoid collapses of the trench side walls and to ensure no excessive slurry losses were experienced during the wall excavations.

Drilling was done from the platform at elevation 180 m. Fig. 5 shows the typical layout of the TAM holes. It was decided to install tubes à manchettes upstream and downstream of the future cut-off wall at about 3.35 m from the centerline. Holes were spaced 2 m center to center and divided into primary and secondary holes for grouting purposes. The upstream holes were also used to complete the rock grouting below the alluvial materials. The length of the dam foundation treated with TAM was 106 m for a 225 m long cut-off wall and covered the deepest section of the dam foundation. Fig. 6 shows the treated zone and the rock profile.

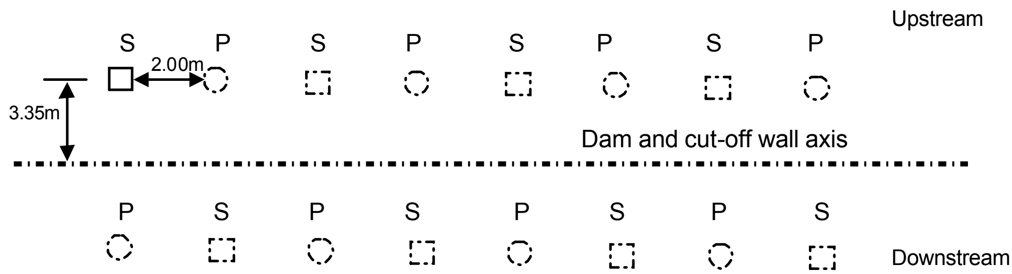


Fig. 5 Tubes à manchettes layout

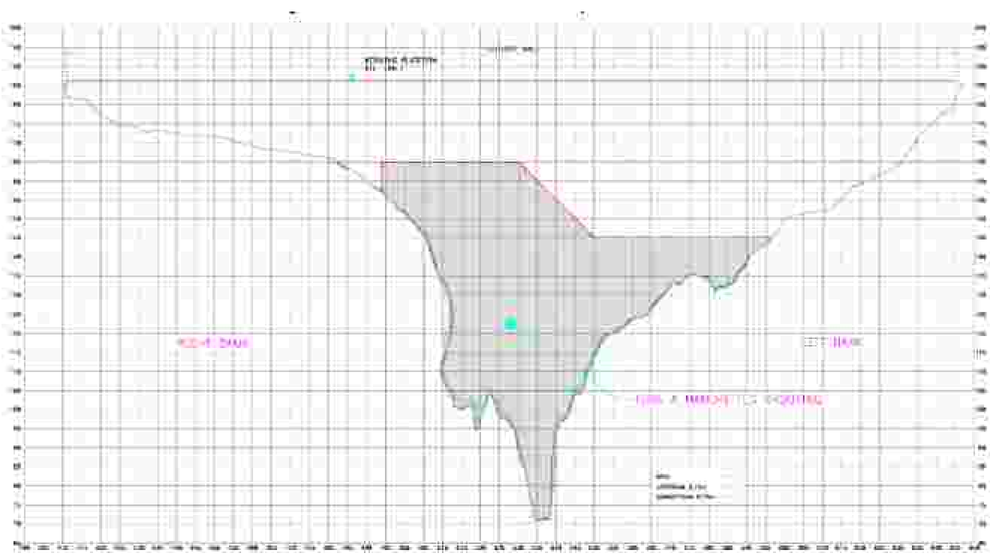


Fig. 6 Rock profile and zone treated by Tube à Manchettes

Originally all holes were drilled with down the hole air driven hammers. The rigs were equipped with an electronic system which allowed recording of advance, pressure on hammer, air pressure, torque and retention pressure on the tool. By combining these parameters, a fairly accurate identification of boulders and bedrock levels was possible.

However, during construction a certain number of problems were encountered with the rigs:

1. Uncertainty of the effects of air pressure on the soil. Two major collapses were recorded requiring several m^3 of backfilling with sand and re-vibration
2. Following this it was decided to use rotary drilling only. This, however, resulted in more and larger deviations beyond the requirements of the technical specifications, compared with the down the hole air driven hammer.
3. The rigs in general were unable to drill at depths greater than 50 m. Boulders, when encountered, were difficult to penetrate. The casing could not follow through and holes had to be abandoned.

4. Drilling progress was relatively slow, as the casing had to be welded which was time consuming in the production process and considering the tight schedule.

A review of the situation was made and different options considered to improve drilling. Sonic drilling was chosen to continue the drilling work. That type of drilling provides representative but remoulded, continuous core samples of any overburden formation in even the most-difficult-to-drill terrain with boulders generally without the use of water, air or mud. The drill stem and sampler barrel are vibrated vertically at sonic frequencies between 50 and 180 Hz such that the sampler barrel normally advances by slicing through the soil. Almost all holes deeper than 50 m were drilled with the Sonic drilling rigs. However, one of the disadvantages of the Sonic drilling rigs was that no recording system could be installed to monitor advances.

The tube a manchettes consists basically of a steel or plastic tube which is perforated at the specified 0,5 m intervals . Perforations are covered with a rubber seal in order to avoid inflow of materials or of sleeve grout during installation. The tube a machettes is inserted in the drilled borehole and sleeve grouting is poured between the borehole casing and the tube à manchette. The casing is extracted gradually and continuous sleeve grout is added to compensate for losses in the ground.

The initial and additional sleeve grout takes are recorded. This has two purposes:

a) to have an idea on the thickness of the sleeve grout, which later will have to be hydro-jacked
b) to have an estimate of the grout absorption in each layer.

A cement-bentonite-water mix with a March Cone viscosity of 80 seconds was used in order to avoid excessive losses of sleeve grout. Excessive build-up of sleeve grout (through losses) requires higher pressures to shear the sleeve when grouting the soil and may cause additional problems.

A strict quality control was implemented in order to ensure proper installation and understanding of the behavior during production grouting. This is illustrated in the flow diagram shown in Fig. 7.

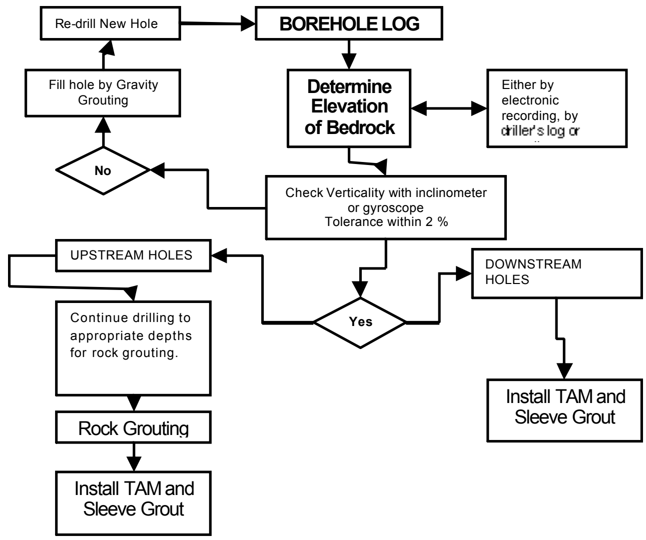


Fig. 7 Quality control applied for the installation of Tube à Manchettes

After the sleeve grout had set, TAM grouting could proceed. The first step is to establish a grouting envelope for each manchette, mainly based on its depth. Grout pressures at the pump are established based on actual depth, depth of groundwater table and pressure losses through the grout system and in the sleeve grout (see Table 3).

Table 3 Tube à manchettes pressure

1	PRESSURE LOSS IN CONDUITS, CONNECTIONS, etc	FLOW 5 L/MIN FLOW 7.5 L/MIN FLOW 10 L/MIN	4 bar 5.5 bar 7 bar
2	PRESSURE LOSS THOUGH CRACK IN SLEEVE GROUT	0.5 TO 10 BAR	This parameter is largely unknown and is estimated in the field from the monitoring system.
3 4	HYDROSTATIC PRESSURES	-L grout Column* 1.13 bar + L water column* 1.0 bar	Where L is the length of the grout or water column
5	SPECIFIED MAXIMUM PRESSURE IN SOIL ACCORDING TO THE TECHNICAL SPECIFICATIONS	Depth of OVERBURDEN *0.2BAR	

The pressure required at the pump is therefore established as 1+2+3+4+5, where parameter 3 is negative. Parameters 1, 3, 4 and 5 are reasonably well established by actual data and measurements. Parameter 2 (Pressure loss through the induced crack in the sleeve) is unknown, as it depends on how many cracks are induced, their widths as well as the thickness of the sleeve grout itself.

The only way to ensure that all criteria were met was to have a close monitoring in real time of the behavior of grout take and pressure at the pump. In order to ensure this, a fully automated electronic grouting and recording system was put into place, allowing visual observation of the grouting behavior (Fig. 8).

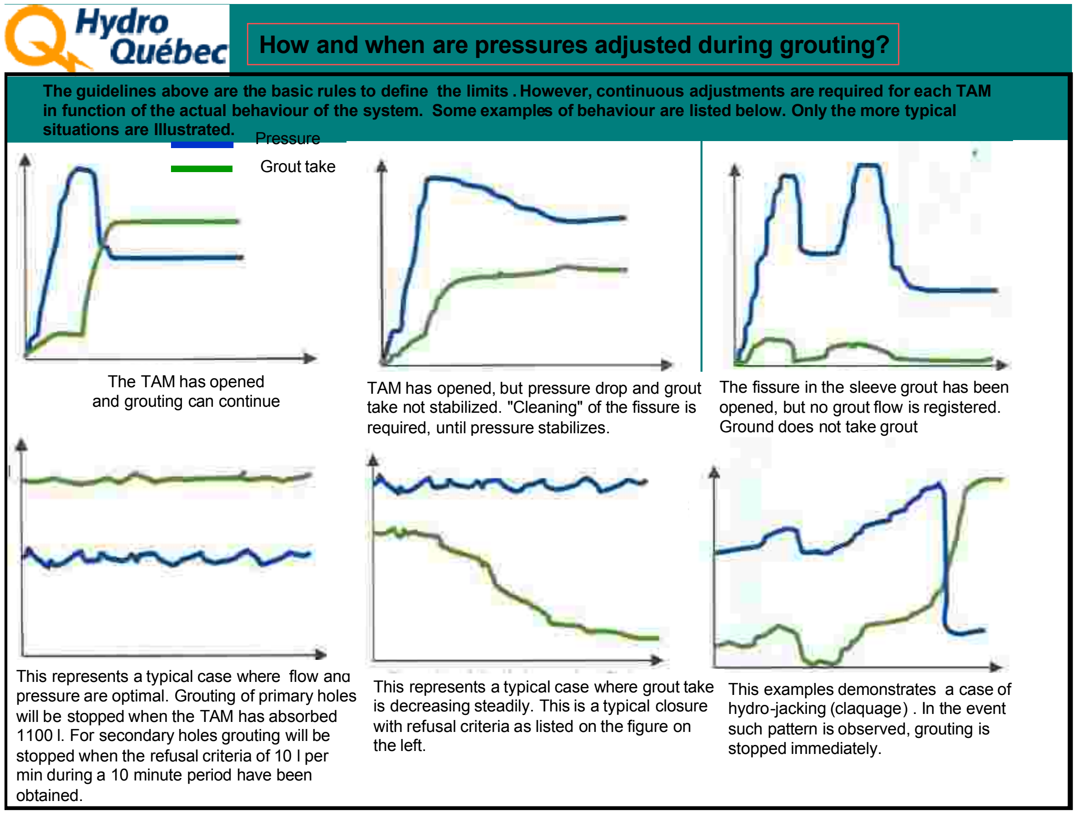


Fig. 8 Graphs of grouting behaviour

It should be noted that all these operations and checks were done under extreme conditions with little or no time on hand, considering the tight schedule, and that therefore all decisions had to be made very quick. Further 5 to 6 holes were grouted at the same time. The set-up, as explained, not only allowed such a fast operation, but also allowed to return back later if any doubts still existed or corrections had to be made.

Refusal criteria were set at 750 L and 1100 L of grout take per manchette (every 0.5 m) for the primary holes, depending on its location in relation to the axis of the cut-off wall. The refusal criteria for secondary holes were set at 10 L per minutes for a 10 minutes period.

Considering the large amount of data generated it was necessary to obtain a clear picture of the results. The only way to observe the grout behavior was to visualize the results using a 3D picture as illustrated in Fig. 9.

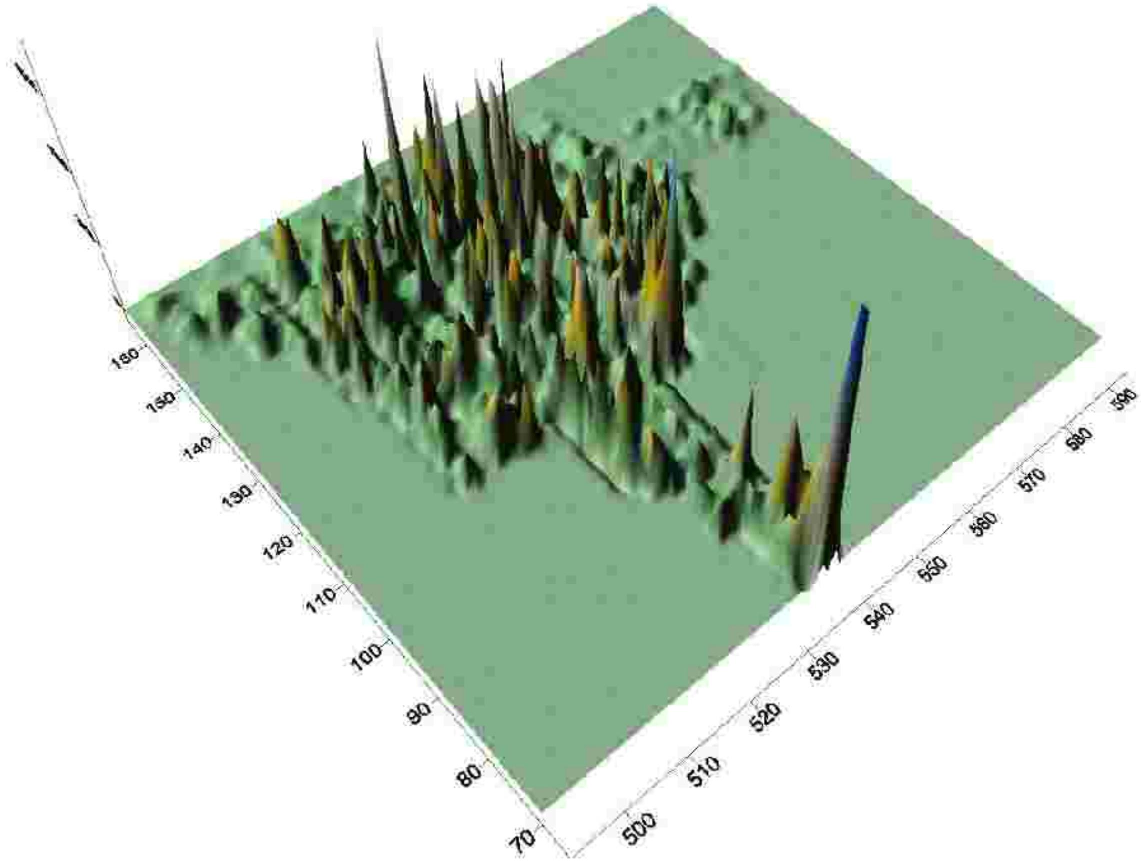


Fig. 9 3D analysis of the grout take – upstream

Cut-off walls construction at main dam

Trench excavation was done in a bentonite slurry. The main design elements and construction specifications for the cut-off walls are as follows:

- Minimum 0.5 m key in sound bedrock;
- In the river valley, the width of the wall is 1.2 m on the banks and 1.5 m in the deepest part of the glacial gully. The width of the wall in the secondary valley and at Dike A is 0.8 m;
- Upstream & downstream minimum overlap between panels is 0.7 m for the river valley cut-off wall and 0.5 m for secondary valley and dike A cut-off walls;
- Minimum overlap between panels along the axis of the cut-off walls is 0.2 m;
- Maximum deviation of panels is 0.5% of depth;
- Use of a hydro-mill was required for all panels of more than 50 m depth, for embedment in rock and for joint excavation between panels;
- Use of clamshell bucket and chisel was permitted for shallower panels and for removal of boulders or other obstacles;

- Primary panels could be done with single bite or with multiple bites (up to three) for a maximum length of 8.4 m;
- Secondary panels had to be done with a single bite for a minimum length of 2.2 m;
- Bentonite slurry for trench stabilisation had to be prepared with a sodium bentonite The use of polymer for trench slurry was permitted;
- Embedment of the cut-off walls in the till core of the embankments is 6.0 m for the main dam and 5.0 m for dike A;
- Till surrounding the top of the cut-off walls is mixed with 5% bentonite to insure impermeable contact between core and plastic concrete and to act as a cushion to minimise risk of the cut-off wall punching into the core due to settlement of the embankment;
- Cut-off walls are constructed with plastic concrete to reduce differential settlement between foundation's soils and cut-off walls under loading of the embankment.

In order to respect the overlap criteria between panels and avoid windows in the cut-off wall, the verticality and the orientation of the panels were controlled during and after the excavation.

One of the most important challenges at Péribonka Dam Site was the building of the cut-off wall with the particular topography of the bedrock in the river valley. A rock topographic model was generated with the data provided by the Tube à Manchette boreholes. As the alluvium grouting was executed on both sides of the cut-off wall, data was available from the upstream and the downstream side. The deviation of each hole was measured and taken into account for the 3D model which was built by triangulation. This model, which has proven to be quite accurate, was used to define the maximum depth of the gully, the steepness of the slopes, the presence of an overhang on the right abutment and some important differences in bedrock elevations from the upstream to the downstream side of the cut-off wall (over 15 m vertically for less than 5.5 m horizontally between the boreholes).

The bedrock topography dictated the layout and the sequence of construction of the panels adopted by the contractor (see Fig. 10). Due to the steepness of the bedrock profile, every panel, except two, was excavated individually in one length of 2.8 m. The two others were excavated in three lengths totaling 7 m, as was done for dike A. Five panels were excavated transversally to the cut-off wall axis. Guide walls were built accordingly. Four of those panels were located where an important upstream-downstream slope had been identified and the fifth one was in the deepest part of the gully adjoining the steepest rock face. The purpose of these panels was to allow a greater transversal deviation of the adjacent panels while maintaining continuity in the cut-off wall with a minimum overlap.

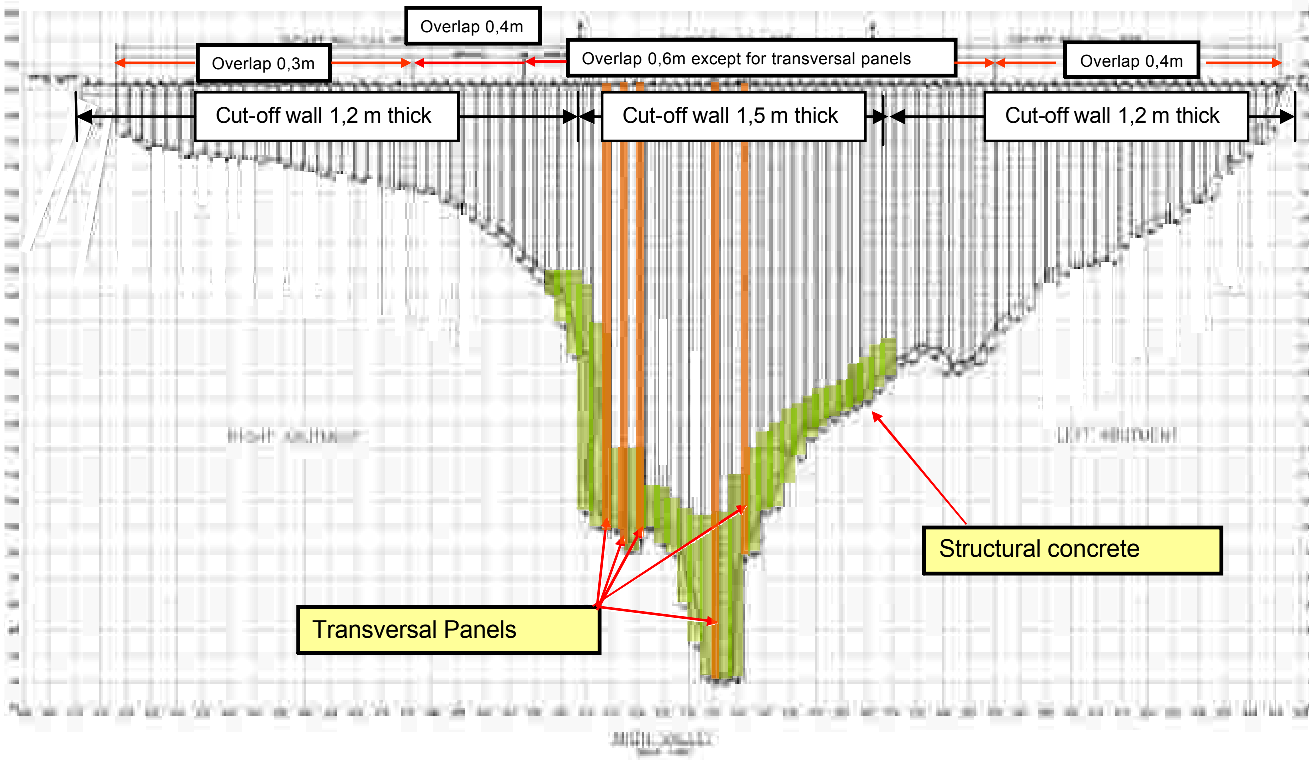


Fig. 10 Panel layout at the river valley

A few primary panels were completed first, then adjacent panels were built supported on one side by an existing primary panel. Finally secondary panels were done to close the cut-off wall between two existing panels.

Panels were mainly excavated with a hydraulic cutter. The clamshell was usually used to open the trench for the first 4-5 meters. In a few cases, the clamshell was used to excavate down to about 25 meters and in one occasion, down to 50 meters.

Sixty-eight (68) panels of 1.2 meter width and up to 60 m deep were excavated with a HDS BC40 cutter. Twenty-nine (29) panels deeper than 60 m were excavated with a 1.5 m wide CBS CBC 135 BC50 cutter, the first cutter of this size ever built by the manufacturer.

Panels were to be excavated beginning at the lowest points of the deep gully and by successively climbing the gully side where the cutter gained lateral support from the adjacent panel. This permitted greater thrust of the cutter wheels on the rock side walls and minimized longitudinal deviations. For this purpose, the contractor was authorized to use structural concrete in the lower part of the panels in order provide greater resistance to thrust. With the preliminary bedrock profile used for the design before the 3D modeling was available, finite element analyses showed that an arching effect could develop creating zones of stress concentration. Maximum elevations for structural concrete were calculated to ensure that plastic concrete was used in these stress concentration zones. Thus, 31 panels have structural concrete in their lower part. However, the contractor didn't always follow this strategy of starting with the deeper panels as it was observed that the cutter could go down straight even without an adjacent panel to lean on.

In order to preserve the minimum overlap requirement of 0.2 m longitudinally, the contractor proposed a panel layout providing theoretical overlaps between panels of 0.3 m to 0.6 m. The extent of the overlap was chosen based on the steepness of the bedrock profile combined with the estimated depth of the panels.

For the Péribonka Dam river valley, panels were completely keyed at least 0.5 m into sound bedrock. This resulted in very deep embedment in some cases, especially where steep rock faces were encountered. A length of over 27.7 m of rock had to be excavated on one side of a panel on the left side of the deep gully. Unconfined compressive strength (UCS) of the bedrock in the river valley site is about 100-150 MPa. The mean rate of excavation for the embedment in sound rock was about 0.35 lin. meter/hour with the 1.5 m wide cutter and 0,56 lin. meter/hour with the 1.2 m wide cutter.

While doing the 3D modeling of the rock profile, an overhang on the right side of the gully was detected. A few exploratory rotary drilling boreholes were done to better define the dimension of the overhang. Drilling confirmed the presence of a 22 m long overhang at least 3.5 m wide a few meters above the bottom of the gully (see Fig. 10). In spite of all the efforts to define the scope of the overhang as accurately as possible, uncertainties remained, especially regarding the depth of the arch under the overhang.

Two panels were necessary to cross the overhang (see Fig. 10). For the first panel, the cutter had just one wheel in contact with the rock face while the other wheel remained in the alluvium. For the second panel, the cutter had both wheels grinding the rock mass of the overhang. To ensure that the cut-off wall was properly embedded in the rock face under the overhang, a steel wire brush was used to scrape the rock face and remove any remaining alluvial material.

Collapses forming sinkholes at the surface of the work area of the cutter occurred in the vicinity of some open trenches. These trenches usually had to stay open for longer periods. The cavities appeared right by the trenches, creating openings 1 to 2 m wide in the working platform. Even though no firm explanation was found for the occurrence of these cavities, material from the till core, in which the cut-off wall is embedded, seemed to have been destabilised thus inducing the collapse. Fig. 11 shows an example of these sinkholes.

The common elements to all the collapses were the depth of the excavations and the duration that they were left open before concreting. However, not all trenches that stayed open for a long period collapsed. Another trigger for the collapse could have been a change in the properties of the bentonite slurry due to sedimentation or chemical reaction with cement in the concrete of adjacent panels.

Fig. 11 Sinkhole near the open trench

An analysis of the concrete consumption curves of all the panels in the river valley clearly showed a tendency for important over consumption of plastic concrete in the upper 6 meters of the cut-off wall. Thus, it was decided to investigate the state of the core fill and of the top of the plastic concrete in the top part of the cut-off wall after the removal of the guide walls.

Concrete protuberances accompanied by bentonite slurry pockets and soft fill material were systematically found in the first few meters on either side of the cut-off wall. It was then decided to excavate and replace the till material around the top of the cut-off wall over a minimum width of 1.5 m and a minimum depth of 2.5 m. The plastic concrete protuberances were trimmed carefully to the minimum width of the cut-off wall using a hydraulic excavator equipped with a special bucket.

In the secondary valley, the panels were 0.8 m in width because of the shallower depth of the panels (Fig. 12). No vibro-compaction and no alluvium grouting were required and rock grouting was executed in the cut-off wall axis. Every panel was excavated individually in lengths of 2.8 m. As was done for the main river valley, the excavation sequence was primary, adjacent, and secondary panels. While drilling boreholes for rock grouting, an overhang was detected on the right side of the valley. Structural concrete was again used in the vicinity of the overhang to provide support when excavating the adjacent vertical rock faces. Overlaps between panels were from 0.3 to 0.6 m.

The main problem in the secondary valley was the poor quality of the bedrock from the middle of the valley up to the right end of the cut-off wall. Panel embedment length was thus more difficult to determine, as sound rock was not easily identified. Progress was fast in the poor quality bedrock, almost as fast as it was in overburden.

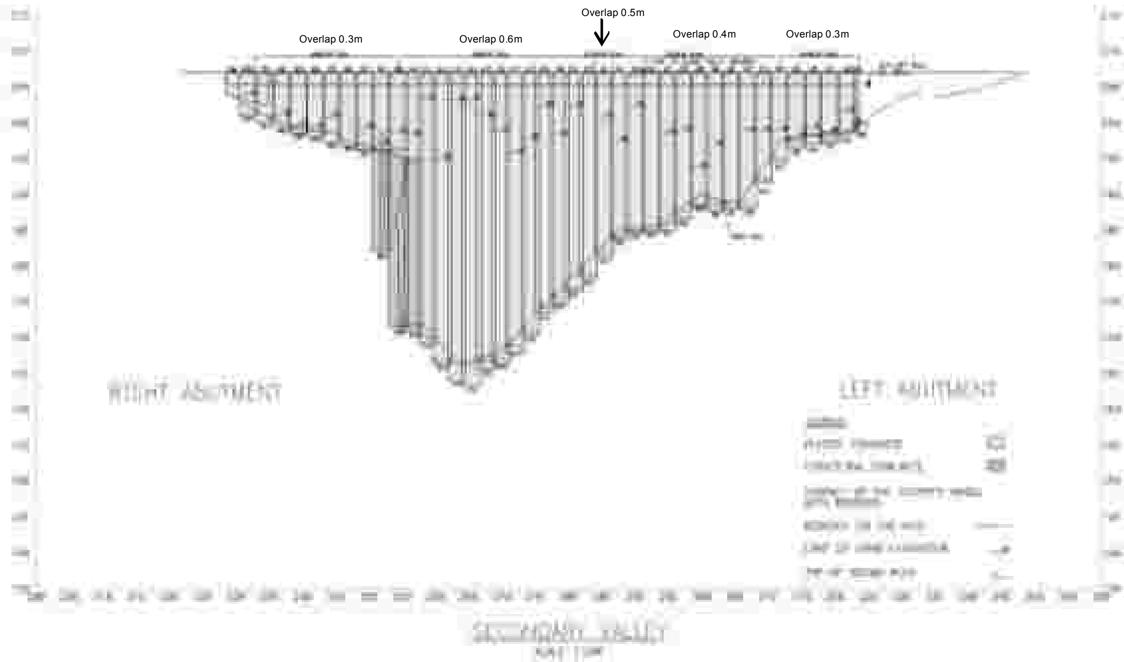


Fig. 12 Panel layout in the secondary valley

A1.12.4 Dike A: Foundation treatment

Before constructing the cut-off wall, dynamic compaction to a depth of 10 m or less was done at dike A to densify the soils. The cut-off wall was built from a working platform at elevation 227 m. Fig. 13 shows the panel layout. The cut-off wall is 0.8 m wide. The primary panels are 7 m long and are excavated in three stages with the trench cutter. The first and second stages are done at the extremity of the panel over a 2.8 m length each and subsequently the remaining material (1.4 m) is removed in the third stage (2.8 m – 1.4 m – 2.8 m). All the secondary panels are 2.8 m in length.

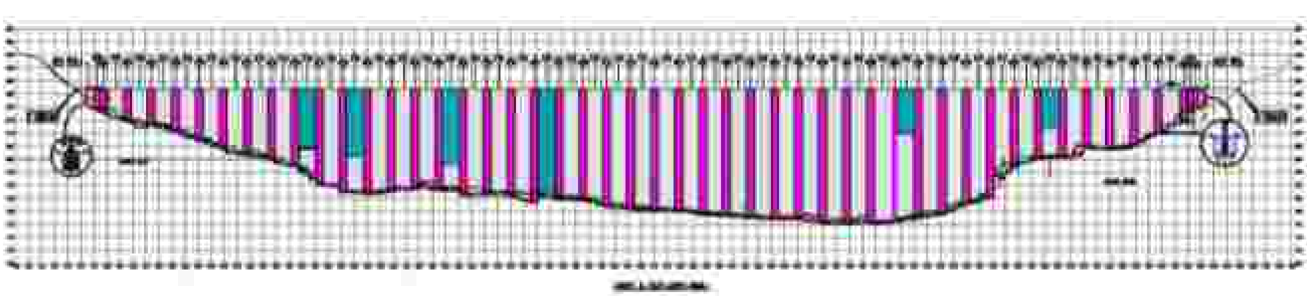


Fig. 13 Panels layout at Dike A

The rock profile was determined by rock grouting drilling. The curtain grouting was done along the cut-off wall axis with a spacing of 6 m center to center. Overlap between panels was made at 0,3 m to ensure a minimum overcut of 0,2 m on the two primary panels. Each end of the cut-off wall is finished with a T shape panel, 7 m long, to lengthen the seepage path around the ends of the cut-off wall.

The bedrock surface is generally weathered and of poor quality. For this reason, it was decided to continue panel excavation for the embedment in sound rock to a minimum depth equal to the top of the grout curtain. The curtain was grouted to 0.5 m below the contact bedrock / overburden because of difficulties with packer's sealing at the top of the boreholes. This longer embedment was made in order to ensure imperviousness continuity between the cut-off wall and bedrock.

Six primary panels were re-excavated and concreted because of doubts revealed by a detailed analysis of the concrete consumption curves: they showed significant under-consumption of concrete. It was decided to re-do these panels to eliminate any chances of contamination of the plastic concrete by material from the walls of the trench. The experience gained with the primary panels of the cut-off wall at dike A led to a revised and more comprehensive follow-up procedure for excavation and concreting of the main dam cut-off walls.

A1.12.5 Conclusions

A major construction challenge was successfully met for the construction of the cut-off walls at Péribonka, especially in the river valley. The topography showed a deep narrow gully filled with alluvial material, a mixture of sand, gravel, cobbles and boulders. The main cut-off wall reached a depth of over 115 m and achieving complete embedment into bedrock was not a trivial task.

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A1.13 A.V. Watkins Dam (USA)

A1.13.1 Project description

A.V. Watkins Dam, located 10 miles north of Ogden, Utah, is a U-shaped, zoned earth fill structure constructed on Willard Bay of the Great Salt Lake. The dam is more than 14 miles long, and is approximately 36 feet high at its maximum section. The dam impounds an off-stream reservoir (fed by the Willard Canal) known as Willard Bay, which has a capacity of over 215,000 acre feet and a surface area of nearly 10,000 acres.

Prior to construction, a drainage canal was excavated downstream and parallel to the alignment of the southern portion of the dam in order to lower the local groundwater table and facilitate construction of the dam's embankment. Now referred to as the South Drain, this canal continues to collect local groundwater, surface runoff (from both precipitation and irrigation), and dam seepage flows and transports them to the Great Salt Lake.

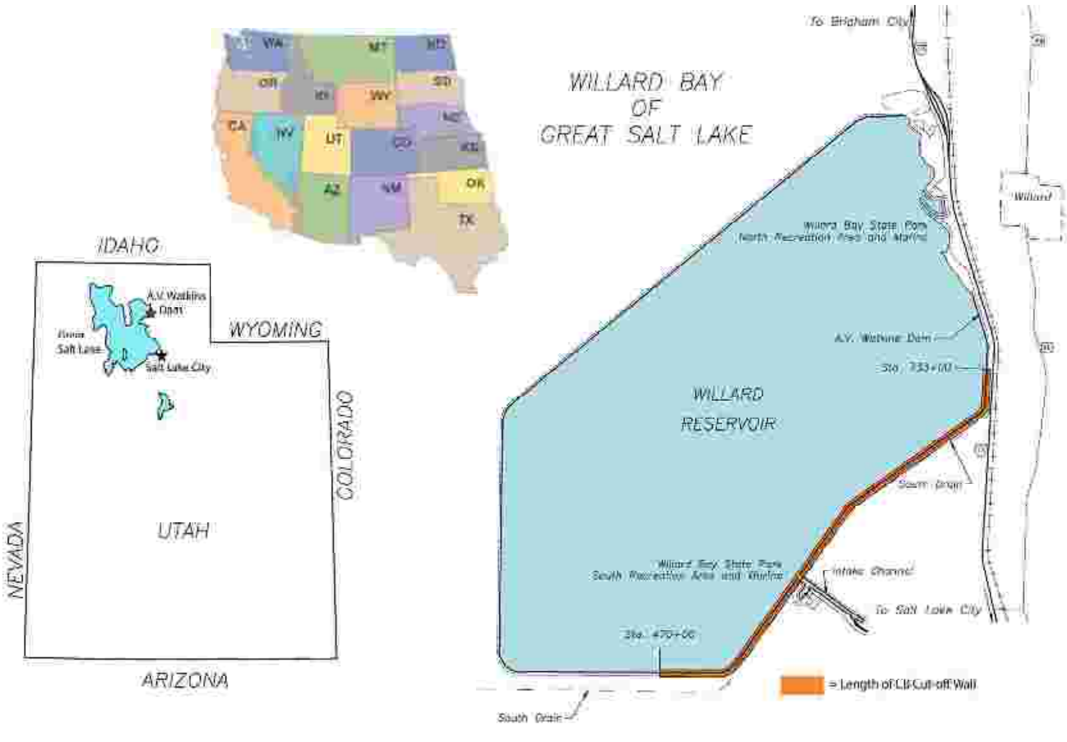


Figure 1: Location of A.V. Watkins dam and reservoir

A1.13.2 Foundation conditions

A.V. Watkins Dam is located immediately west of the Wasatch Range, within 1 mile of the Wasatch Fault, in the Middle Rocky Mountains province. The dam is built across, and atop, a former bay of the Great Salt Lake known as Willard Bay. Geologic mapping by the USGS characterizes the deposits in the southeastern portion of the dam as

"Lacustrine Deposits consisting of gravel, sand, and clay deposited in the fluctuating waters of Lake Bonneville and the Great Salt Lake." (Personius, 1990) Because of the compressibility and low strength of these soils, construction was staged in phases (between 1957 and 1964), to allow for foundation pore pressure dissipation and consolidation.

In November of 2006, A.V. Watkins Dam nearly failed as a result of piping and internal erosion of its foundation soils. Fine grained, sandy soils from the dam's foundation had piped to the downstream toe of the dam and also into the South Drain near Station 639+00, also referred to as the "incident area." Up to 200 gpm of seepage water was exiting from sand boils at the toe of the dam and subsequently flowing into sinkholes as it made its way towards the South Drain. The seepage would then re-emerge at the bank of the South Drain, where it subsequently deposited large amounts of sand into the channel. Figure 2 describes the geology under the dam and the mechanics of this near failure.

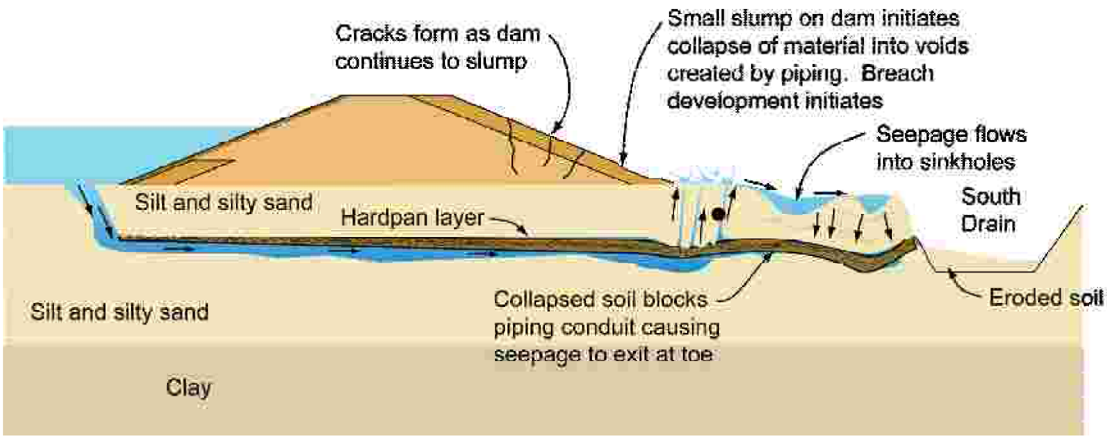


Figure 2: Dam section at incident area describing failure mode

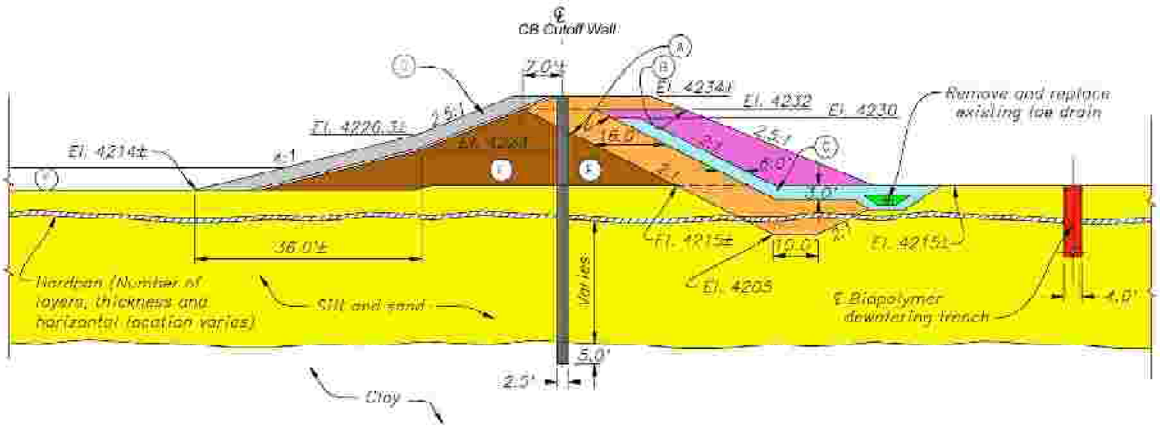
In a heroic effort to save the dam, hundreds of cubic yards of drain rock and filter sand were hauled to the site and placed over the sand boils and sinkholes. In addition, thousands of cubic yards of general earth fill material was also dumped on the upstream side of the dam in order to stem the flow of water into the inlets of piping channels in the dam's foundation. Although the dam was saved, it could no longer store water until a permanent repair could be made to its foundation.

The following year an extensive geotechnical exploration program revealed that over 5 miles of the dam, between Stations 468+00 and 733+00, were considered to be founded directly upon the highly erodible sediments shown in Figure 2.

A1.13.3 Design considerations for foundation treatment

Using seepage models and an extensive collection of geotechnical data, Reclamation's Technical Service Center (TSC) calculated that it was necessary to limit the hydraulic gradient in the foundation soils to a maximum of 0.04 in order to prevent the piping and internal erosion activity observed in the 2006 event.

A number of alternatives were considered, but only the cutoff wall alternative sufficiently lowered the hydraulic gradient and addressed all of the failure modes identified in the risk analysis. For example, a new toe drain at the downstream toe of the dam, constructed with modern filter criteria, did not fully address seepage into the south drain. A filter zone at the south drain would not prevent seepage from occurring at other locations downstream of the dam (which were observed in a number of places during the 2006 incident). Also, any kind of interceptor trench associated with a new toe drain would have to extend below the hardpan layer, making construction both difficult and expensive. Finally, combining an extensive filter zone and a new deep toe drain (in order to address all failure modes) would be extremely cost prohibitive given a repair length of



over 5 miles.

Figure 3: Section of dam showing CB cutoff wall as well as the 300-foot long reconstructed embankment in the incident area

The cutoff wall alternative, while addressing all known failure modes, allowed for several different methods of construction, all of which called for the slurry to support the sidewalls of the trench. A soil-cement-bentonite (SCB) wall was rejected because it would not only be too expensive, but it would be too strong – the wall needed to be ductile, capable of withstanding the movement associated with a floating earthen dam. Also, SCB slurry construction requires a larger staging area adjacent to the wall than does its CB counterpart.

A soil-bentonite (SB) wall was also considered, but later rejected because 1) the narrow crest of the dam did not allow for an appropriately sized mixing area, and 2) soil bentonite's inherent lack of resistance to internal erosion. A cement-bentonite (CB) wall, on the other hand, appeared to capture the advantages of both methods. The cured CB material would provide some strength, resisting internal erosion, yet also be ductile enough to flex with the embankment. The CB construction process would require no working platform on the dam crest, and the slurry itself would have the ability to fill voids and existing defects (encountered in the walls of the trench) in the embankment and foundation. Once inside the voids, the CB slurry would cure and become immovable. According to data available throughout the industry, the cured CB (using normal Portland Cement) would have permeabilities of between 1 x 10^{-6} and 1 x 10^{-5} cm/sec.

Finally, given all advantages and disadvantages, CB appeared to provide the most impermeable, yet ductile, cutoff for the least amount of money. In January 2008, Reclamation entered the project's final design phase using the CB cutoff wall similar to that shown in Figure 3. Excavated from the crest of the dam, the 30-inch wide wall would extend up to 70 feet deep – and be keyed at least 5 feet into the bottom-most clay layer.

Figure 4: Aerial photo showing alignment of CB cutoff wall through the south eastern portion of the dam, including the section that nearly failed (incident area)

A1.13.4 Slurry production and wall construction

Excavation of the CB slurry cutoff wall began with a 2,000 linear foot "test section." It was Reclamation's intention that any inconsistencies in the slurry mixing process, as well as problems inherent with trench excavation, would be ironed out prior to moving west towards the more critical areas of the project. In addition, samples of the CB slurry were to be taken daily through the test section and subsequently analyzed at both 14 and 28 days, thereby providing assurance to Reclamation that the cured CB achieved both the strength and permeability requirements called for in the specification.

The CB slurry, as defined in the specifications, called for 1685 lbs (1 cubic yard) of water, 303 lbs of Portland Cement (Type V), and 101 lbs of bentonite (API 13A Section 9). Typical for the industry (PCA, 1984), this represented a mix design of approximately 6% bentonite and 18% cement by weight of water. Although not specifically identified, Reclamation also allowed for the use of viscosity modifying additives, such as lignosulfonate (LS), in manufacturer recommended proportions.

Wall excavation, as well as slurry production, began in mid-July of 2008 with target units weights of 64.5 pcf for the bentonite slurry and 71.8 pcf for the CB slurry. With the bentonite pond being continuously re-circulated, the bentonite slurry produced filtrate volumes of between 16 and 20 mL. After being mixed with cement and LS, the slurry was pumped (via a positive displacement pump) through a 6-inch HDPE pipeline from the plant to the trench heading up to 2,700 feet away at a continuous rate of up to 400 gpm. Figure 5 shows a typical CB plant arrangement. The excavator, a Komatsu PC1250 with an oversized boom and stick, excavated a 40 to 60-foot long trench, 55 feet deep, 30 inches wide in less than 2 hours. In this fashion, excavation proceeded around the dam 24 hours per day, 6 days per week.

Early quality control testing from 3-inch by 6-inch cylinders, cast on a daily basis, indicated that although the cured CB lacked compressive strength, it did achieve the project's permeability goals. Fourteen-day cylinders cast within the first 5,000 feet of CB wall broke (on average) at 9 psi and achieved an average permeability of 6.1 x 10^{-6} cm/sec.

Figure 5: Aerial photo showing CB wall excavation approaching the slurry plant

As wall construction proceeded in a westerly direction, the contractor crested the learning curve and productivity increased. As the trench was excavated by the Komatsu PC1250, spoils were pulled out through the CB slurry and deposited on the downstream slope of the dam. Poorly graded silty and sandy soils immediately flowed to the downstream toe of the dam, while spoils from the dam's embankment, as well as the target clay layer, remained higher up on the slope. After excavating for approximately 2 hours, the contractor, in the presence of a professional geologist, would sound the depth of the trench and verify that the excavation had penetrated at least 5 feet into the clay. The final depth was then marked on wooden lathe located every 20 feet along the upstream crest of the dam. The photograph in Figure 6 shows the typical CB wall excavation operation.

When the CB plant was relocated to a new location, the contractor moved his excavator several thousand feet away and then excavated in a direction back towards the plant. The contractor found that it was easier to shorten the HDPE supply line by cutting it, rather than lengthen it by fusing on more pipe, as excavation progressed over the 5.5 mile long section of dam.

Figure 6: Photograph of typical CB cutoff wall excavation through the dam crest

A1.13.5 Conclusion

During peak productivity, the contractor was able to excavate over 450 linear feet of wall and 24,000 square feet of wall per day (2 shifts). On a daily basis, CB slurry production consumed over 350 tons of cement, 115 tons of bentonite, and 340,000 gallons of water. Unit weights and viscosities were monitored continuously by the contractor and verified every 2 hours by Reclamation. The end result of these efforts was the successful production of over 175,000 cubic yards of CB slurry, which, upon curing, provided the cutoff wall with an average permeability of 5.8 x 10^{-6} cm/sec (28 day). Both 180-day and 365-day tests indicate that the wall continues to gain compressive strength and decrease in permeability.

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A1.14 Wanapum Dam Left Embankment Soil-Bentonite Slurry Trench Cutoff Wall

A1.14.1 Project description

A1.14.1.1 General

Wanapum Dam was constructed in the early 1960's across the Columbia River in the State of Washington, USA. The cross-section of the dam is shown in Figure I (note that dimensions are in feet, 3.28 ft = 1.0 m). The tallest embankment section is about 45 meters high and is located in the left embankment over the original river channel.

The dam is located in a broad, relatively flat valley. The rock bluffs against which the embankments terminate are about 2 km apart. The main river channel was located near the center of the left embankment. Prior to the construction of the dam, the river channel was about 200 m wide at the dam centerline with a maximum water depth of about 12 meters. The bedrock was generally level along most of the alignment, with the exception of two distinct depressions.

Bedrock at the site consists of a thick succession of lava flows known as the Columbia River Basalt formation. Bedrock structure consists of a broad anticline that trends northwest or roughly parallel to the river. The rock has been described as intensely jointed. The bedrock is overlain by alluvial material consisting primarily of river worn sand and gravel, some of which are highly permeable gravel. Lenses of varying thickness ranging from loose gravel to fine sand were reported throughout, and cross bedding of the various deposits was evident. Zones of silty sand and silt are also present in some areas. The alluvium is typically about 18 meters thick with localized depressions extending to approximately 45 meters. The alluvium is about 9 meters thick in the river channel.

A1.14.1.2 Left embankment

The left embankment of Wanapum Dam is a zoned earthfill structure founded on alluvium. The left embankment extends from the powerhouse, east to the left abutment. Construction of the left embankment started at the powerhouse and continued east to the river crossing section. Construction at the east end of the left embankment was also started and extended west to the river crossing section. The river crossing section was constructed last since it was constructed in the wet and was the final closure section for the Columbia River. The river closure section is about 430 meters long.

For the river section, rock materials, generally smaller than 150 mm in size, were dumped and dozed into the river to construct two parallel dikes across the Columbia River at the locations of the upstream and downstream toes of the embankment. The area between the dikes was filled with sand and gravel dumped into the river in a single lift and then compacted by vibroflotation. Figure 2 presents the sequence of construction.

A slurry cutoff trench wall was constructed across the river channel, through the dumped and densified fill to the top of bedrock, to reduce seepage losses through the foundation. The as-built width of the slurry cutoff trench wall was generally 3.7 meters wide beneath most of the left embankment. A bentonite silt/sand/gravel mix backfill was placed into the slurry cutoff trench. The backfill mixture remained soft, long after it was dozed into the trench.

The embankment above the top of the dumped fill was constructed entirely of rolled earth materials. The zones of the embankment include: "impervious fill" core constructed of very fine sandy silt; "pervious fill" shells constructed of cobbles, boulders, sand and gravel; "dumped rockfill" at the upstream and downstream toes areas; and "transition zones" separating the core from the shells and "filter material" separating the pervious fill and the dumped rockfill.

Construction documents and subsequent investigations indicate that good construction practices were followed in the construction of the left embankment of Wanapum Dam. The monitoring of construction, including soundings of the various stages of underwater construction, was thorough and confirmed that the intent of the design was accomplished. Recent investigations have also confirmed that material placement and specification requirements were obtained.

A1.14.1.3 Slurry cutoff trench

Steps 7 and 8, Figure 2, illustrate the construction of the slurry trench through the densified dumped gravel fill. The top portion of the slurry trench was widened to the width of the impervious core zone and the slurry trench was cleaned to bedrock. Silt, sand, and gravel at the bottom of the trench were removed from the bedrock surface and a concrete surfacing was placed in the bottom of the trench. The slurry trench was then backfilled with a bentonite silt/sand/gravel mix.

Three different slumps occurred during construction of the slurry trench in the river crossing section. Two slumps occurred over a 90 meter length; the third, and largest, occurred over about 75 meters. The slumps were cleaned and backfilled with the soil-bentonite mix. This resulted in cutoff trench widths of 12 or more meters at the slump locations. Figure 3 illustrates the cross-sections of slumps at several locations. Figures 4, 5, and 6 are construction photos that illustrate the slumps, backfill, and repair. The backfilled material at the location of the slumps remained soft and wet at the surface.

A1.14.1.4 Impervious core placement

The impervious core was placed and compacted over the slurry cutoff trench transition using conventional rolled fill techniques. Moisture content of the fill and compaction effort were controlled and adjusted as necessary to achieve at least 90% of modified Proctor compaction. The maximum thickness of each lift prior to compaction was one foot and the minimum number of passes of a sheepsfoot roller was set at six.

Because the top of the cutoff trench backfill was too soft and plastic to support compaction equipment for initial lifts, an initial layer of silt was first pushed onto the cutoff trench transition backfill to allow a dozer to push a second lift. This second lift was compacted with a sheepsfoot roller. The third layer generally had sufficient support so that the dozer and sheepsfoot rollers could follow established placement and compaction procedures.

A1.14.2 Quality of construction

Subsequent to construction, surveys of crest settlement indicated that substantially more settlement had occurred at the river closure section and specifically at the locations where the slurry trench had slumped. In view of the difficulties experienced during construction and the settlement history at the crest of the dam over a 30-year period, a comprehensive site and construction history investigation was undertaken. A review of construction documents, field investigations. and laboratory testing has confirmed that material descriptions and properties within the left embankment conform to the intent of the specifications and drawings. Good construction practices were followed in the placement of materials and quality control testing and monitoring is well documented.

A1.14.2.1 Core

The cone penetrometer (CPT) investigation demonstrates that the impervious core of the left embankment is consistent with materials as described on the construction drawings and in the construction records. CPT results in the left embankment indicate that the core has consistent soil properties, is in good condition, and has a dense to very dense consistency. The CPT test results do not indicate any bridging or cracking of the core of the embankment. No unusual seepage conditions were identified. The geophysical and thermal anomalies identified in previous reports do not appear to have any significance due to high cone tip resistances and low seepage observed through the core. Pore pressure dissipation tests in the core indicate that seepage through the core of the embankment is consistent with steady state seepage conditions.

A1.14.2.2 Cutoff trench

The cutoff trench transition materials are composed of a granular matrix with clayey material filling the void space between the aggregate. Cone penetration resistance in the cutoff trench transition indicates that the matrix has high strength. The results of the pore pressure dissipation testing indicate that the cutoff trench transition materials have a very low permeability. No unusual seepage conditions were observed in the limited area of the cutoff trench investigated.

In summary, after more than 40 years of operation, the dam continues to perform well and in accordance with the original design.

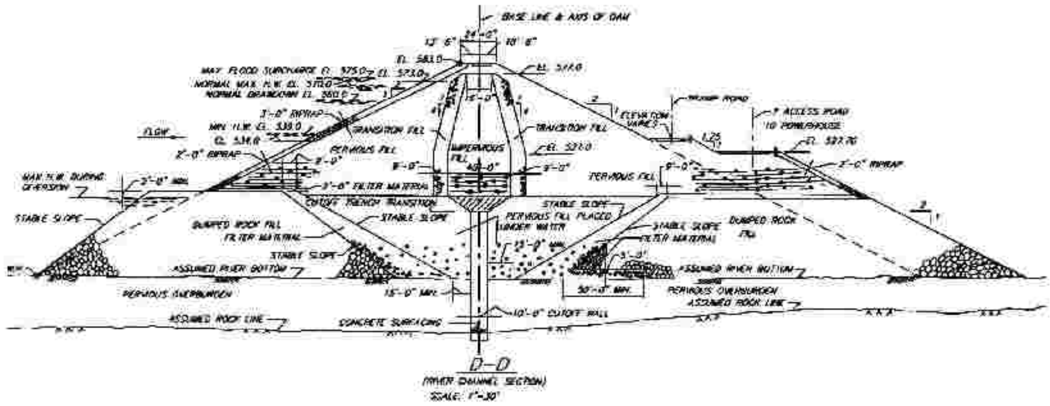

Figure 1 Cross section at Columbia River crossing (dimensions in feet and inches)

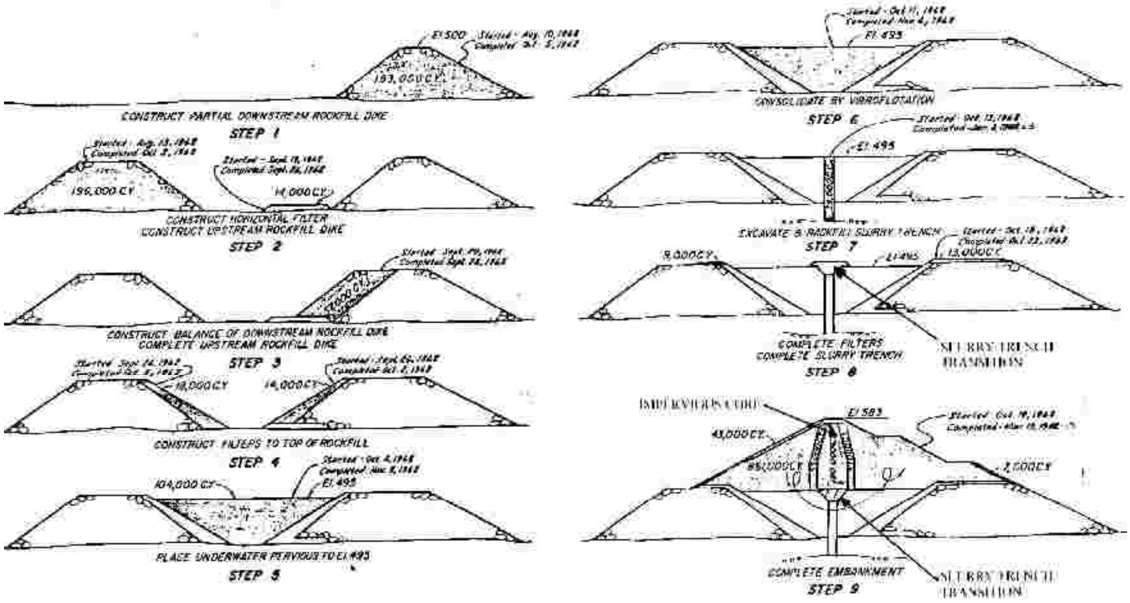

Figure 2 Construction stages at river crossing

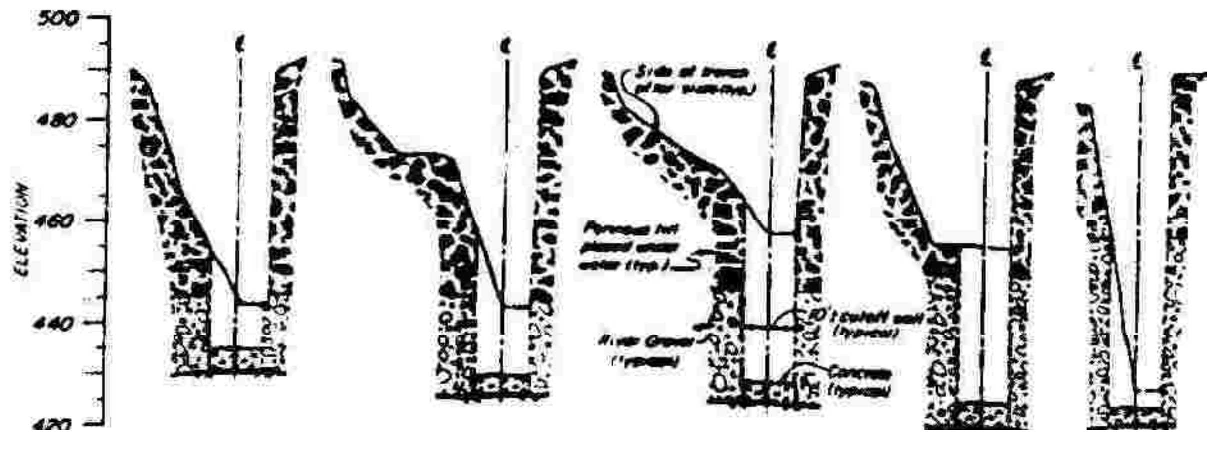

Figure 3 Typical sections of slumped slurry trench (natural scale, in feet)

Figure 4 Slumped slurry trench

Figure 5 Slumped slurry trench, note use of draglines and clam-shell excavators

Figure 6 Fill placement, cleaning, and repair of slumped slurry trench

A1.15 Manasquan Dam (U. S. A.)

A1.15.1 Project description

The 1495 m long Manasquan embankment dam with a structural height of 16.4 m is located on Timber Swamp Brook in Monmouth County, New Jersey, USA. The reservoir is part of a pumped storage water supply system managed by the New Jersey Water Supply Authority. The dam was constructed between 1987 and 1990. A detailed description of the case history can be found in Khoury et al. (1992).

A1.15.2 Foundation conditions

The dam is underlain by Coastal Plain sediments. There are three formations relevant to this project, namely the Kirkwood and the Manasquan Formations which consist of layers of sand, silt and clay. The upper unit of the Manasquan Formation forms the base clay layer beneath the dam and the reservoir. Below the Manasquan formation is the predominantly sandy Vincentown Formation, which is a regional aquifer. A longitudinal section along the axis of the dam is shown in Fig. 1and it can be seen that the base of the dam is in contact with the Upper and Lower Kirkwood Formation along most of the alignment. In the river bed, around the central part of the embankment, the clay had been mined in earlier times. This mined area was filled with soft organic sediments which had to be excavated and replaced with a blanket of imported clay within the affected area. Fig. 2 shows a typical cross section.

A1.15.3 Foundation treatment and seepage control

The embankment was constructed of predominantly non-plastic fine sand from the Kirkwood Formation (SP-SM) with 5 to 35 % passing #200 sieve. For seepage control through the dam body and the foundation, a soil-bentonite (S-B) cutoff wall, was constructed in two stages. This type of wall was selected because it turned out to be the most technically feasible and cost effective method for this embankment. The S-B wall was preferred over a cement bentonite (C-B) wall because the S-B method allowed the formation of a filter cake along the trench walls. This was believed beneficial to the overall performance of the wall, its hydraulic conductivity and the stability of the excavated trench (reduction of slurry loss). However, the main disadvantage of an S-B wall over a C-B wall is associated with the occurrence of differential settlements between the backfill and the trench walls in the embankment.

Design criteria for S-B wall adopted:

- Long-term hydraulic conductivity of wall shall be less than $1x10^{-8}$ m/s, i.e. about 1000 times smaller than $k_{horizontal}$ of the embankment material.
- Design of backfill must maximize the use of on-site soils.
- The wall has to be designed to withstand the development of concentrated leaks by hydraulic fracturing under the prevailing hydraulic gradient.
- The open slurry trench has to be stable during construction in order not to impair the integrity and safety of the embankment.

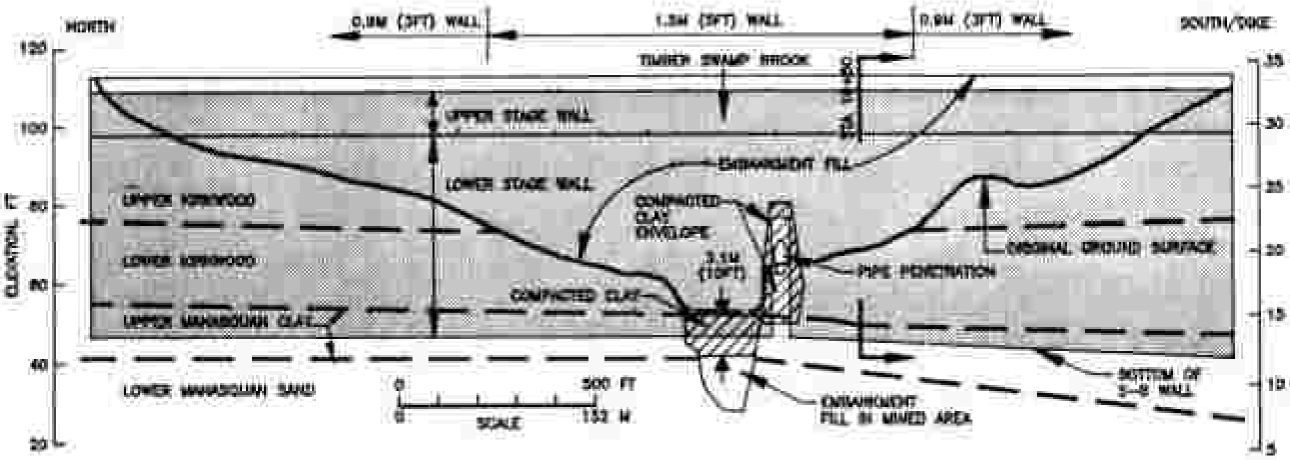

Fig. 1 Profile along axis of dam (Khoury et al., 1992)

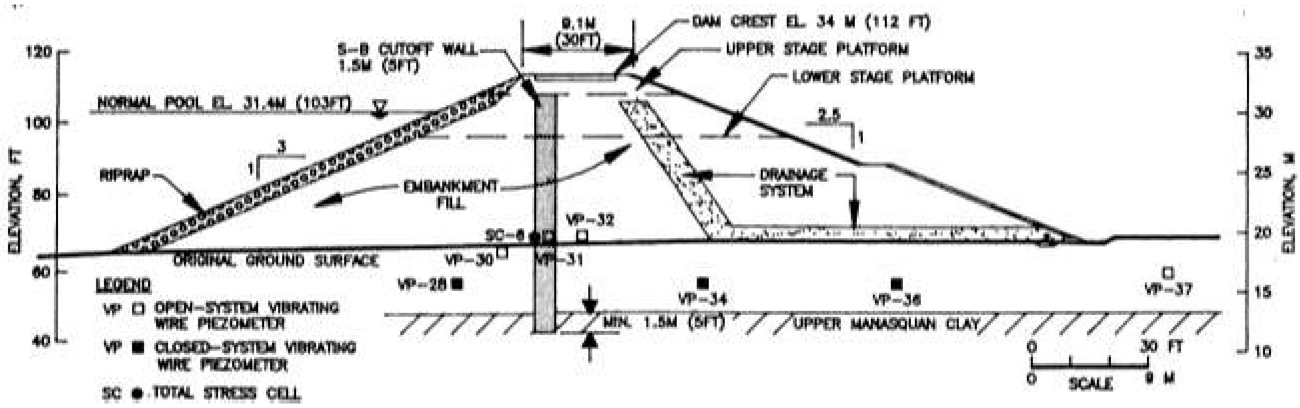

Fig. 2 Typical dam section at Sta. 19+50 (Khoury et al., 1992)

A1.15.4 Backfill mix design

The backfill material in a S-B wall consists of soil, usually excavated from the wall trench, and bentonite. The soil is expected to be of different texture since the wall penetrates different strata. The soil must have sufficient fines in order to achieve the required k-value, otherwise material with a higher fines content or bentonite has to be added. Relevant S-B backfill mix characteristics are summarized in Table 1.

At the required percentage of fines, however, tests showed that the material has a considerable compressibility which would lead to large settlements. A substantial amount of gravel would have to be added to reduce the volumetric strain to reduce the potential for large settlement. This was considered not economical.

Table 1 S-B backfill characteristics for Manasquan cutoff wall

Soil-bentonite mix characteristics	Range of five mixes
General mix data	
▪ Slump (cm)	12.1-13.3
▪ Percent bentonite by dry weight	0.87-1.65
Index and physical properties	
▪ Passing #200 sieve (%)	8.7-72.5
▪ Water content (%)	23.8-62.1
▪ Total unit weight (kN/m3)	15.7-19.0
▪ Undrained shear strength (by lab vane) (kPa)	1.0-2.0
▪ pH	3.2-6.6
Engineering properties	
▪ Hydraulic conductivity (m/s)	
at 48 kPa	$1.2x10^{-7}$-$1.9x10^{-9}$
at 96 kPa	$9.2x10^{-9}$-$1.2x10^{-9}$
at 287 kPa	$6.4x10^{-9}$-$4.5x10^{-10}$
▪ Consolidation	
Compression ratio, C_R	0.077-0.137
Swell Ratio, C_S	0.005-0.010
Coeff. of consolidation, c_v (cm^2/s)	20.6-85.9

Blow out test

Such tests can be carried out in the laboratory to establish the gradient at which the S-B fines pipe into the downstream part of the embankment. According to the US Corps of Engineers the width of the trench should be equal to 0.1 times the maximum differential hydraulic head acting on the S-B wall. This recommendation is based on a factor of safety of 3 and a blowout gradient of 30.

For this project a blow out test was performed to observe the gradient which would cause a sharp increase in flow through a 10 cm thick layer of S-B backfill overlying a 10 cm thick layer of compacted embankment sand. No blowout occurred for hydraulic gradients up to 160. It was also expected that the compacted embankment sand would act as a filter in case a concentrated flow developed. This sand had a D_{15} of less than the required 0.7mm recommended by Sherard & Dunnigan (1989).

A1.15.5 Construction of the S-B wall

Based on the blowout considerations a 0.9 m thick S-B wall was selected for hydraulic heads up to 9 m and a 1.5 m thick wall for hydraulic heads greater than 9 m. The cutoff wall was executed in two stages. The lower stage had a maximum depth of 19.5 m and was keyed 1.5 m into the Manasquan clay. The upper stage was constructed from elevation 33 m and had an average depth of about 5.5 m. It was keyed a minimum of 0.9 m into the lower stage wall. The upper stage wall was constructed two months after the completion of the lower stage wall. The two stage construction was preferred because a reduced excavation depth provided enhanced trench stability and created more room on top of the embankment fill to mix the backfill with the bentonite along the side of the trench.

Both stages were excavated with a backhoe and the trenches were supported with bentonite slurry until backfilled. The excavated material was stockpiled on top of the embankment adjacent to the trench. Backfill material consisted of excavated soils mixed with bentonite slurry. The mixing was performed on the top of the embankment using a bulldozer. Backfilling started at the south end of the trench by pumping the backfill through a tremie pipe. Once a slope of backfill had been formed for the full depth of the trench, the remainder of the trench was then backfilled by pushing the S-B material into the slurry-filled trench at the top of the slope and allowing the material to slump into the open portion of the trench. Backfilling by tremie pipe was also used on the other end of the trench. The bentonite was mixed in a 54,500 liter tank with a high shear mixer. Open ponds were used for hydration, circulation and storage of the bentonite–water slurry before pumping it into the open trench.

Trench excavation proceeded in 6 m to 9 m long sections with the toe of the excavation kept at least 6 m ahead of the leading edge of the backfill slope. The following average progress rates in wall construction were achieved:
Lower stage wall: 455 m^2/day for the 0.9 m and 440 m^2/day for the 1.5 m walls
Upper stage wall: 670 m^2/day for the 0.9 m and 640 m^2/day for the 1.5 m walls.

Important was the regular monitoring of the sand content in the bentonite slurry, especially at the bottom of the trench and also at the connection between the lower and the upper trenches to avoid the formation of a more pervious zone. An efficient slurry recirculation method was implemented and occasionally the slurry had to be sent through a de-sanding unit.

A1.15.6 Quality control

Procedures and scope of the quality control testing program were laid down in the contract documents and included control of trench geometry and testing of the bentonite-water slurry and the S-B backfill. The trench was tested at 3 m intervals for depth and every 6 m for penetration into the keying stratum and for deviation from the planned alignment. The deviation from verticality did not exceed 1.5 % of the trench depth.

The bentonite-water slurry was tested for:

- Viscosity
- Density
- Filtration
- pH

both at the location of the pump (mixing point) and in the trench. A submarine sampler was employed to collect slurry samples from different depths within the trench.

The S-B backfill was tested for:

- Slump
- Density
- Fraction passing #200 sieve
- Hydraulic conductivity (in flexible and fixed wall triaxial cell)

A grab sampler was used to collect the samples. Table 2 presents a summary of the results obtained from the quality control on slurry and backfill samples. Also shown are the specified values.

Table 2 Summary of quality control testing

Component	Frequency	Lower stage trench		Upper stage trench		Specified value
		Range	Average	Range	Average	
Bentonite-water slurry in pump						
Viscosity (s)	2/day	40-50	43.3	37-47	42.3	40-45
Density (kN/m3)	2/day	10-11.6	10.2	10.1-11.5	10.5	min. 10.1
Filtrate loss (ml)	2/day	10.5-19	13.5	12-12.5	13.1	(max 25 ml in 30 min. at 4.8 kPa)
pH	2/day	7-9	7.3	7-8	7.3	7-10
Bentonite-water slurry in trench						
Viscosity (s)	2/day	43-105	58	41-78	57.8	35-60
Density (kN/m3)	2/day	10.2-13.5	12.1	11.6-13.7	12.6	10.1-14.2
Filtrate loss (ml)	1/day	11.5-23	16.1	13-18	15.4	N/A
pH	2/day	6.4-8.6	7.2	6.3-7.4	6.7	N/A
API sand content (%)	-	9.0-34	22.8	18.5-36	25.5	N/A
Soil-bentonite backfill						
Slump (cm)	2/day	9.5-17.8	13.5	9.5-16.5	13.5	10.2-15.2
Density (kN/m3)	2/day	17.1-18.1	17.7	18.1-18.7	18.4	3.1 above slurry densisty
Passing #200 (%)	1/week	17.4-50.3	39.3	31.9-47.1	40.0	N/A
Flexible wall hydr. conductivity, (m/s)	1/week	2.0y10-7 5.2x10-8	9.4x10-8	2.8x10-7 1.0x10-7	1.5x10-7	max.5x10-7
API Fixed wall hydr. Conductivity (m/s)	1/day	2.1x10-6 3.8x10-8	8.5x10-7	1.5x10-6 2.9x10-7	9.2x10-7	N/A
Backfill slope	1/day	10H:1V for 0.9 m trench and 19H:1V for 1.5 m trench				5H:1V – 10H:1V

API = American Petroleum Institute Specification API RP 13A and 13B

A1.15.7 Performance of the wall

The post-construction performance of the wall was comprehensively monitored because of the potential for hydraulic fracturing caused by differential settlements of the wall. Settlements and pore pressures during and after impounding were the main concern.

Settlement observations

During construction, temporary settlement plates were installed 15 cm below the surface of the lower stage backfill, as shown in Fig. 3. These were steel plates, 610x910 mm in size and 13 mm thick, connected to a riser pipe inserted in a plastic sleeve. Wooden settlement plates, 300x300 mm in size were also installed on the top of the upper stage backfill immediately after completion of backfilling. In addition, Borros points were pushed through the upper stage fill and were anchored in the lower stage backfill to monitor the settlement of this fill during construction of the upper stage wall.

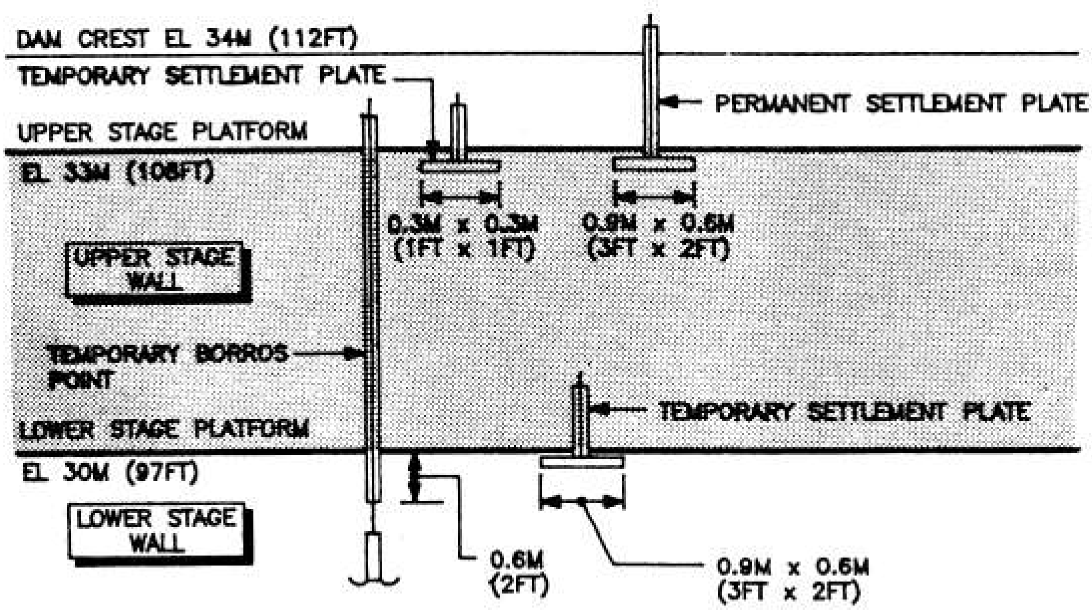

Fig. 3 Location of settlement devices in the S-B backfill (Khoury et al., 1992)

Data interpretation indicated that (Fig. 4):

- Most of the settlements of the lower stage backfill took place during the first two months and three weeks after backfilling of the trenches.
- The measured settlement in the 1.5 m trench was about 2 to 3 times the measured settlement in the 0.9 m wide trench.
- Vertical strains measured in the S-B backfill were about 7 to 9 % and 3 to 4 % in the 1.5 m and the 0.9 m wide trench, respectively.
- The settlement in the lower stage backfill following the construction of the upper stage backfill was negligible.

Construction of the embankment above the completed S-B wall resumed after measurements indicated that most of the backfill settlement had taken place. The top of the backfill in the 1.5 m trench wide upper stage wall was reinforced with geogrids to allow the use of heavy equipment over the trench.

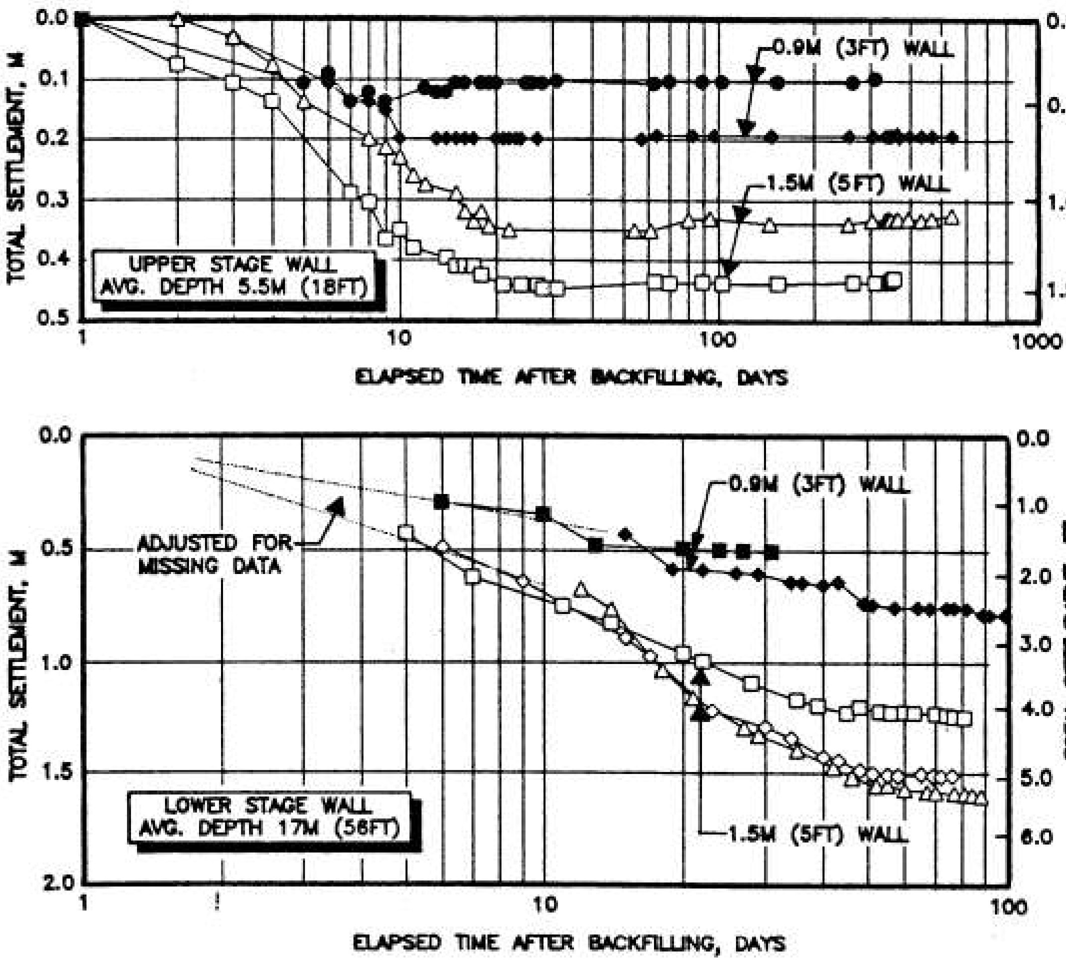

Fig. 4 Observed settlement of S-B backfill from temporary settlement plates (Khoury et al., 1992)

Performance evaluation from permanent instrumentation

Instrumentation to monitor the performance of the S-B cutoff wall included:

- Four permanent settlement plates on top of the S-B backfill
- Vibrating wire-type piezometers (VP) inside the backfill and in the embankment upstream and downstream of the cutoff. These were installed inside a conventional 3.2 cm diameter well point tip pushed into the backfill.
- Five vibrating wire stress cells (SC) inside the S-B backfill at two different elevations to measure total horizontal stresses parallel and perpendicular to the alignment of the cutoff wall.
- Two flow weirs located at the discharge points of the dam's toe drain. These drains had a total discharge of about 45 liters/minute already before reservoir impounding

Fig. 2 shows the locations of these instruments along a typical section of the dam.

From the instrumentation the following information on the behavior of the wall could be deduced (Fig. 4 and Fig. 5):

- No appreciable settlements of the backfill occurred after completion of the upper stage cutoff wall.

- The total heads upstream of the cutoff wall increased with reservoir filling and remained at, or slightly below the reservoir levels. The total heads downstream of the wall were controlled by the blanket drain (or were slightly above the top of the blanket. Inside the cutoff wall the total head was intermediate between those upstream and downstream of the cutoff. No significant changes in the water pressure regime or seepage rates have occurred over the lifetime of the structure (i.e. over a period of about 30 years) (Rice, 2007).

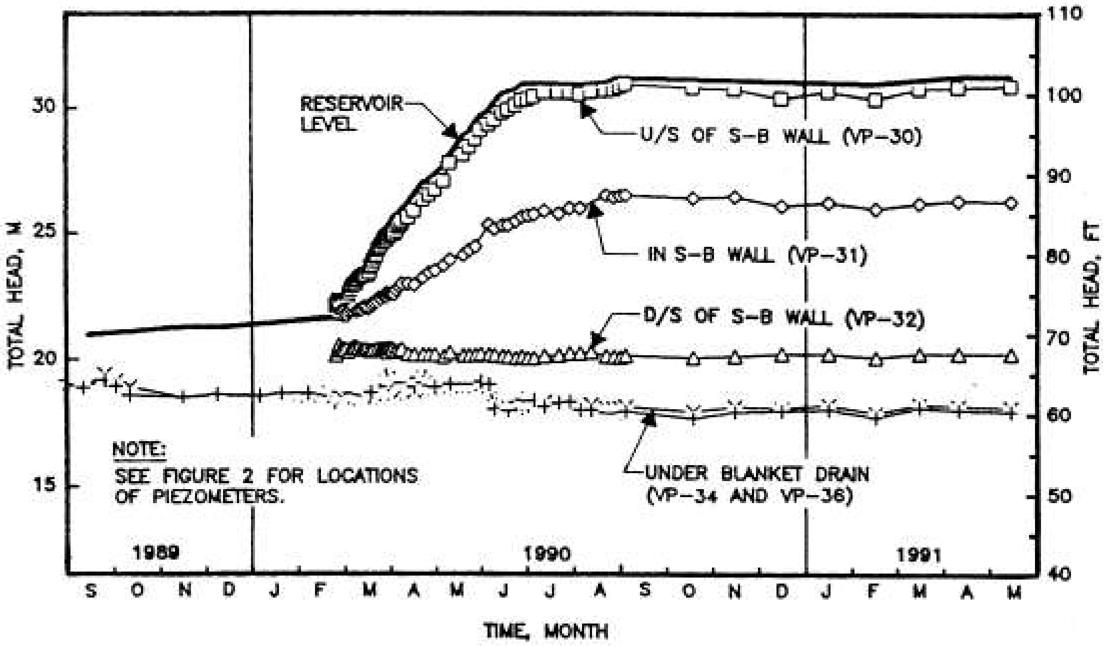

Fig. 5 Piezometric heads at Sta. 19+50 (Khoury et al., 1992)

The efficiency of the cutoff wall can be calculated from the headwater elevation, the piezometric head elevation downstream of the barrier and the elevation of the blanket drain outlet. Based on data from five cross section one obtains (Rice, 2007):

Headwater elevation: 31.4 m
Average piezometric head elevation upstream of wall: 22.75 m
Average elevation of blanket drain outlet: 21.35 m

Efficiency = (31.4-22.75)/(31.4-21.35) = 0.86 or 86 %

- The total stress cells indicated an increase in total stress in the backfill which followed the reservoir level increase. This increase was approximately equal to the increase in total head experienced in the backfill. The stress cells installed parallel to the dam axis, i.e. facing the reservoir, measured higher horizontal stresses than cells perpendicular to the dam axis. The total horizontal stress increased with depth.

A1.15.8 Lessons learnt

The designers of the S-B cutoff wall were aware of the potential of horizontal crack development in narrow walls caused by differential settlement between the backfill and the embankment fill. The presence of such crack could lead to hydraulic fracturing when the reservoir is impounded and finally to piping. The design provided not only a safety margin in the width of the wall but also two lines of defense in case a leak should develop, namely the fine sand of the embankment soil with a D_{15} less than 0.7 mm that would retain particles washed out from the barrier and a chimney drain that would act as a crack stopper.

A rigorous and carefully implemented quality control program during construction greatly contributed to a wall structure of high quality with an almost negligible probability for the existence of "windows" or other defects in the wall.

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Appendix 2. CASE HISTORIES ON VIB WALLS

A2.1 Isar River dykes – Pielweichs Weir

A2.1.1 Project description

As part of the morphological rehabilitation of the Isar River in the State of Bavaria (Germany) a weir with a hydro-electric power station and a stabilizing storage pool were constructed between Pielweichs and Niederpöring, shortly before the river joins the Danube River, (Fig. 1). This is the last of 17 small hydro-power plants on the Isar river. The lateral embankments (dykes) along the river were sealed by conventional vib walls, 12 km in length and with a depth of up to 20 m. Only in one short section the vib wall had to be replaced by a diaphragm wall excavated by a grab for structural reasons. Works commenced in spring of 1990 and were completed in October 1993. The Contractor, Bauer Spezialtiefbau GmbH, proposed the vib walling technique as the optimum solution, both technically and economically.

Fig. 1 Location of Pielweichs Weir

A2.1.2 Foundation conditions

The subsurface conditions along the dykes consist of quaternary alluvial deposits and tertiary impervious clays (Fig. 2). The vib walls had to be keyed into the impervious strata. Due to the complex stratigraphical conditions the clay horizons were found to be extremely variable and considerable technical difficulties were experienced.

A2.1.3 Construction of the vib walls

The equipment used by the Contractor for the installation of the thin diaphragm cutoff wall comprised a crawler crane fitted with a 35 m long lattice type leader and a hydraulic vibrator MS 200 H operated by a hydraulic power pack (**Fig. 3**). The stationary slurry batching and mixing plant had a capacity of 20 m^3 per hour (**Fig. 4**). The slurry consisted of limestone powder, Solidur (Dyckerhoff) and water, designed to give high plasticity and low strength.

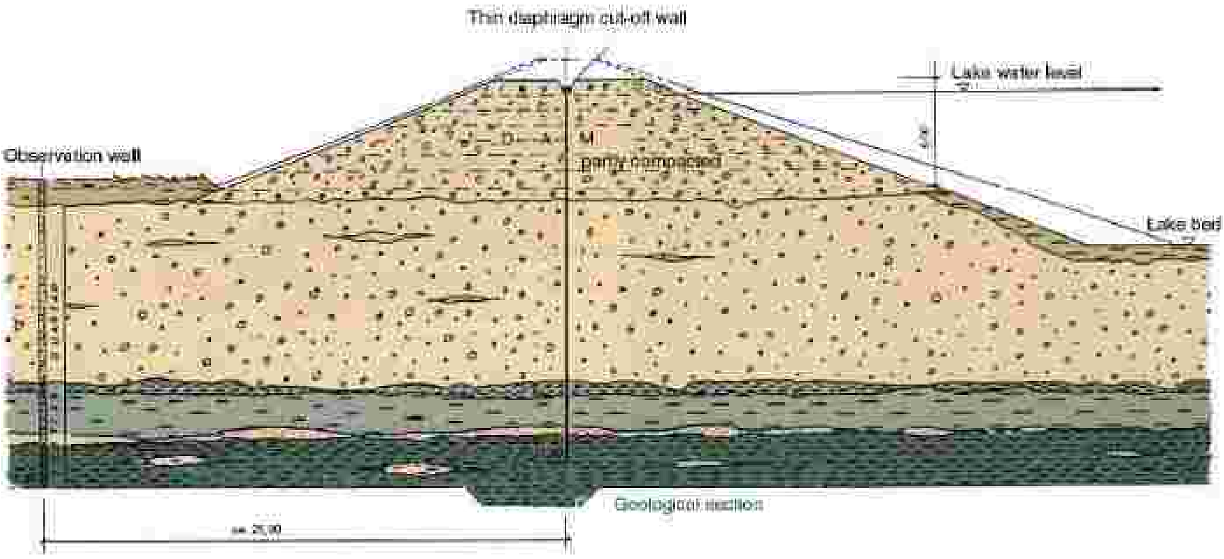


Fig. 2 Cross-section through embankment after installation of vib-wall type cutoff

Fig. 3 Crawler crane SW 311 with hydraulic vibrator MS 200 H

Fig. 4 Stationary slurry batching and mixing plant

A2.1.4 Quality control

During construction the following parameters were continuously monitored:

- Position and inclination of the vib wall beam
- Oil pressure
- Frequency of the vibrator versus penetration of the steel beam for all cuts
- Flow of slurry and pressure versus penetration depth

The quality of the slurry was monitored continuously by testing of samples taken from the mixing plant and the vib-wall slot. Tests included: uniaxial strength, density and permeability. **Fig. 5** shows the grain size distribution of the aggregates used for the supporting slurry and the vib-wall concrete.

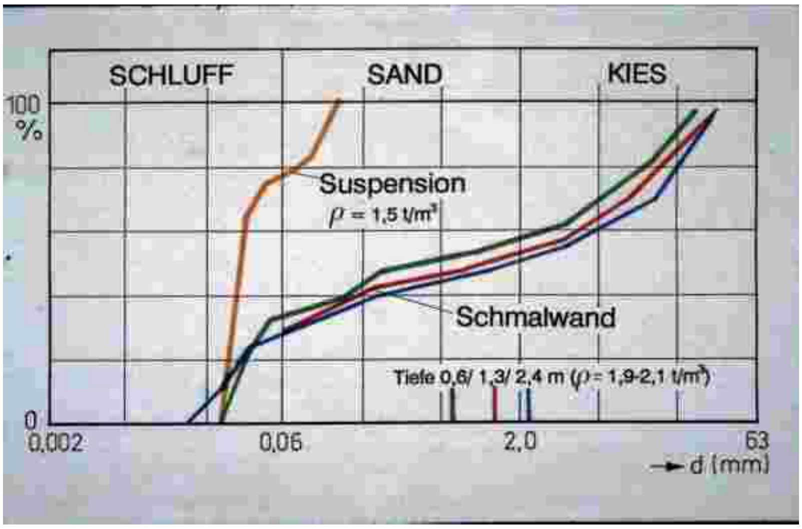


Fig. 5 Grain size distributions for slurry and wall concrete

Finally, six months after completion of wall construction, a section of thin cutoff wall was exposed to a depth of 10m, surveyed and inspected (Fig. 6).

Fig. 6 Exposure of a section of completed vib wall for quality control

Long-term monitoring of the performance and efficiency of the cutoff walls is by observation wells.

Appendix 3. CASE HISTORIES ON PILE WALLS

A3.1 General

The case histories presented here illustrate cutoff pile walls in soil and in rock. Pile walls in soils have all but disappeared from the construction scene because they have been replaced by plastic concrete diaphragm walls which, by means of trench cutters, can be constructed in practically any type of overburden and to large depths, as described in Chapter 3. Pile walls are still used for cutoffs in difficult rock foundations, such as karstic limestone and volcanic rocks.

A widely used technique to construct pile walls in overburden was the I.C.O.S. method. (I.C.O.S stands for Impressa Costruzioni Opere Specializzate). Such a wall consists of alternating circular and bi-concave cast-in-situ concrete piles. The principle of the construction process is illustrated in

Fig. **1** and consists of the following steps:

- Holes are drilled by means of bit A at alternate positions using a thick bentonite slurry. The slurry will penetrate the alluvial soil to some distance and form a cake along the borehole wall.
- Using bit B the filter cake is cut away from the borehole wall
- Concrete is placed into the borehole by means of a tremie pipe (with or without reinforcement)
- Holes are drilled between the concreted primary piles using bit A and the verticality of the holes is checked
- The remaining soil wedge and filter cake are removed by means of bit C
- The space is filled with concrete (with or without reinforcement)

The wall can be built from a working platform located above any maximum water level. The equipment to drill the holes for the piles is usually less bulky and lighter in weight than the rigs used for clamshell excavation or trench cutting. Hence, the construction of the piles can be accomplished also on steeply sloping ground as it often occurs on dam abutments.

There is also the possibility to key the wall into bedrock. In this case performance of the cutoff can be enhanced by cement grouting the underlying rock through vertical holes left in the wall during its construction.

Backfilling of the excavated wall elements is generally by conventional concrete of high slump. Additional watertightness is provided by the bentonite-penetrated zone of soil adjacent to the pile elements.

If needed, the wall can be reinforced on its entire length or in its top part.

Zoccolo dam in Italy is a case history involving an I.C.O.S. type cutoff wall. There are considerable number of dams where this type of cutoff has been employed either as the main cutoff connecting to the impervious element of the embankment or below a cofferdam. Table 1 lists selected examples.

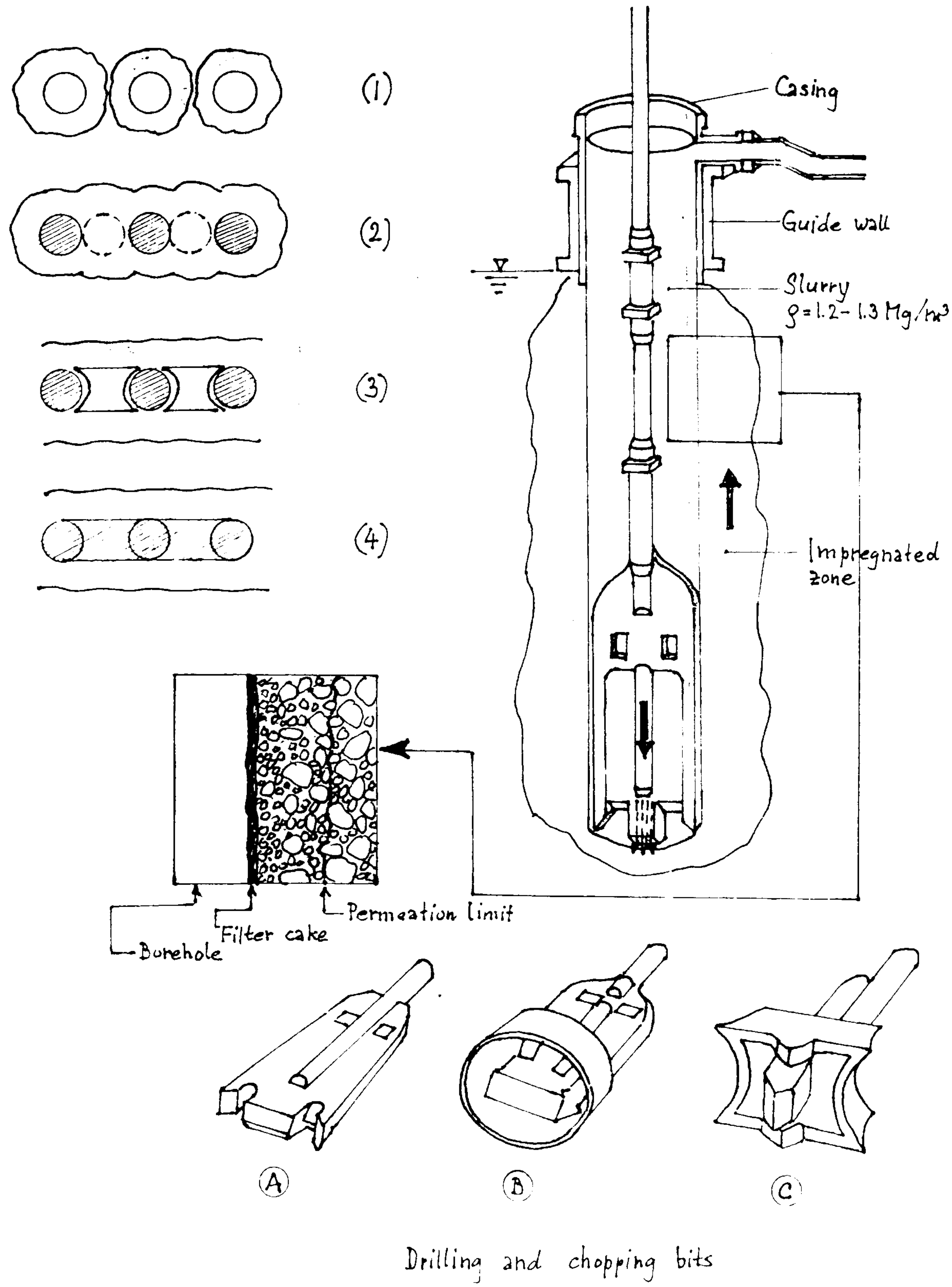


Fig. 1 Steps illustrating the construction of an I-C-O-S pile wall

Table 1 Examples of pile walls using the I.C.O.S. method

Name of dam	Country	Depth of pile wall (m)	Location of wall
Santa Rosa	Mexico	30	Below upstream and downstream cofferdams
Manicuagan 5	Canada	80	Inside upstream and downstream cofferdams and foundation
Manicuagan 2	Canada	25	Inside main dam below steel sheeting and through alluvial foundation
El Infernillo	Mexico	14 to 20	Below pre-cofferdams
Guatavita	Colombia	64	Main dam, below inclined impervious core
El Novillo	Mexico	32	below core of upstream and downstream cofferdams keyed into rock
Guatemare	Venezuela	18	upstream end of impervious clay blanket
Kinzua dam	USA	50	below upstream cofferdam
Hatanagi dam	Japan	22	below 8m high concrete gravity dam
Perdika dam	Greece	20	below central core of main dam
La Villita dam	Mexico	76	below central core of main dam, keyed into rock
Nera	Italy	9	below core of main dam, using plastic concrete, keyed into clay

An important issue is the connection of the wall with the impervious element of the embankment, which is either an impervious core or an upstream impervious facing. Several designs have been constructed and some examples are shown in Fig**. 2** to Fig. 4.

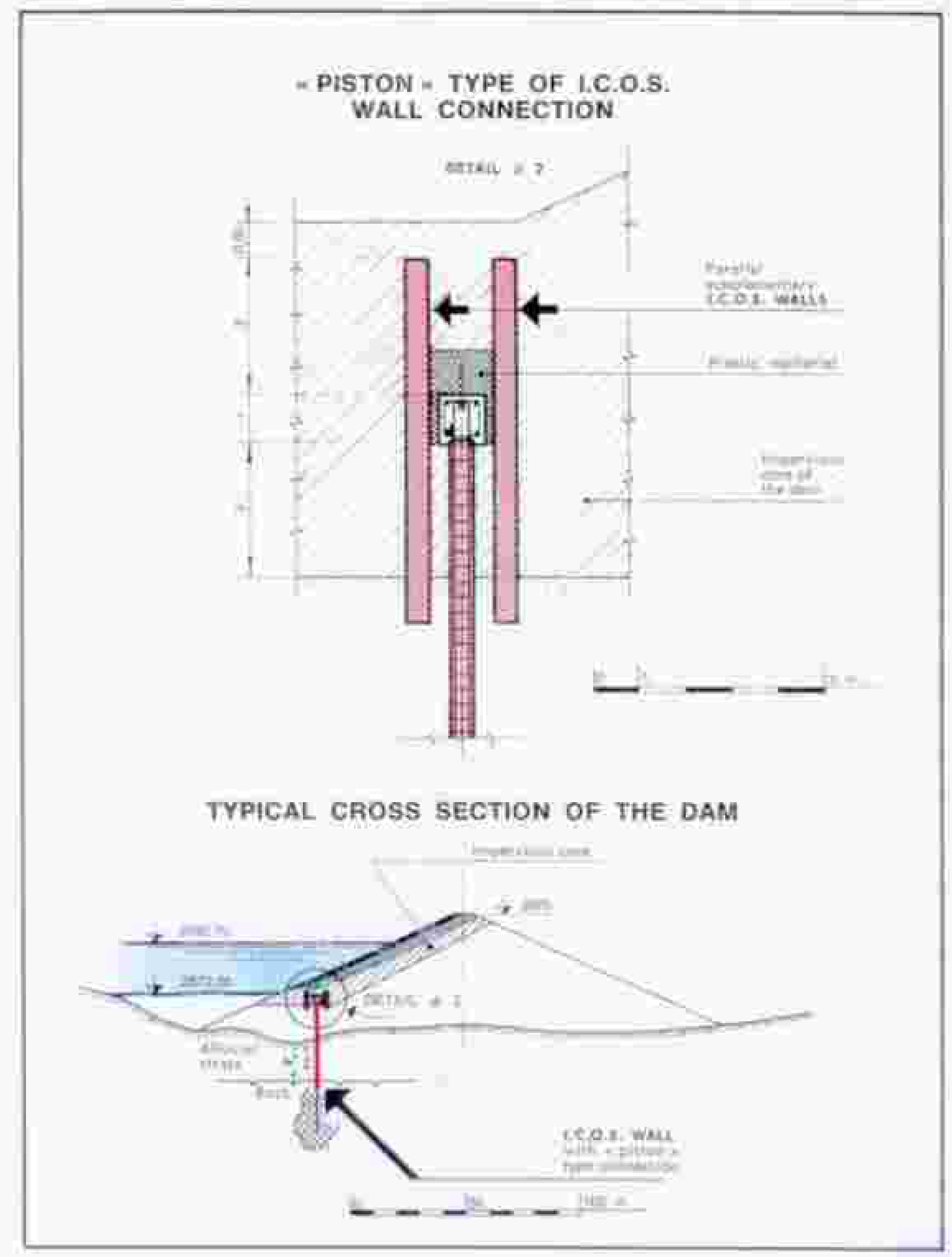


Fig. 2 Dam on the Tominé River, Colombia: Connection between pile cutoff wall and impervious element of embankment

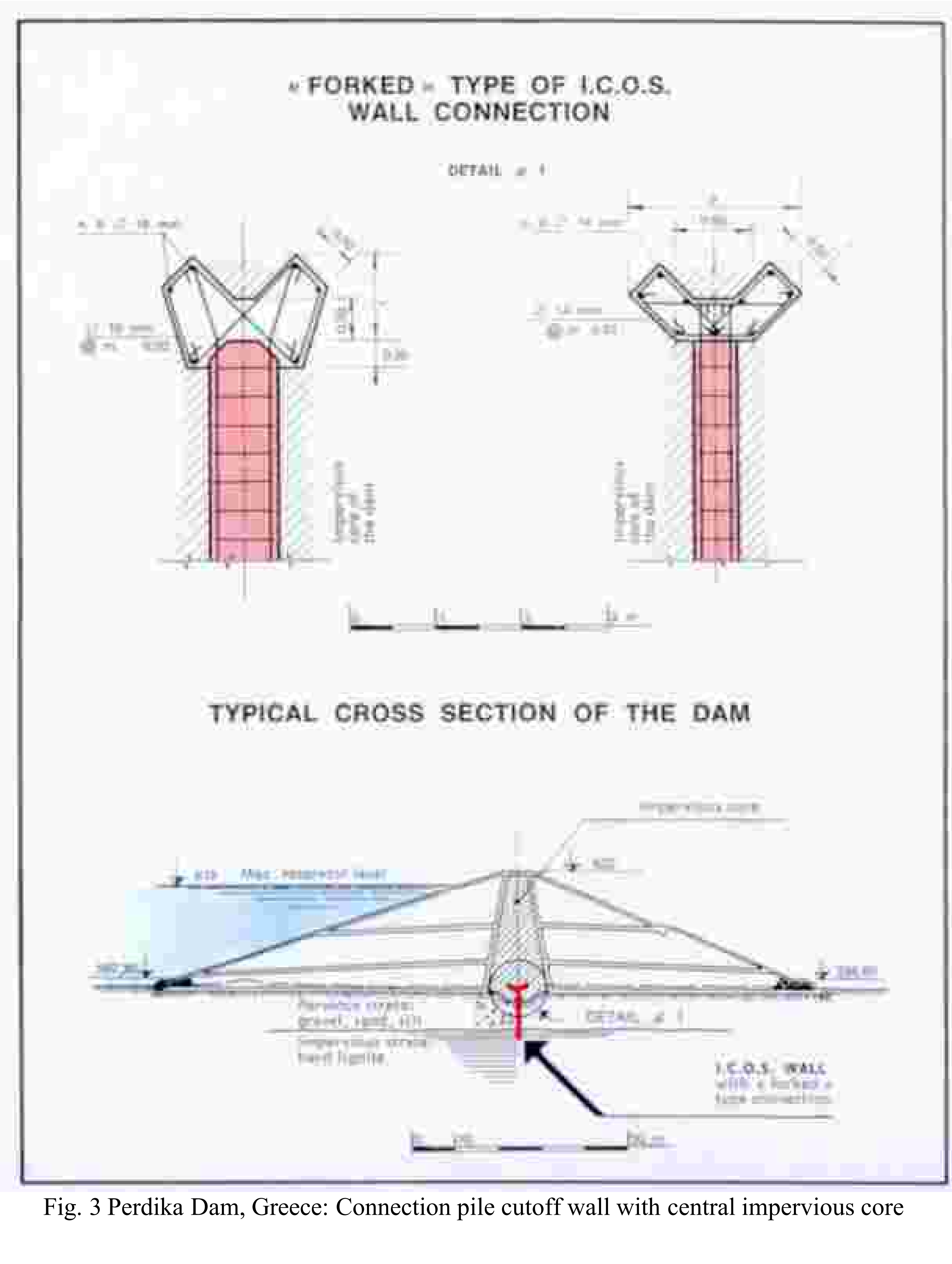


Fig. 3 Perdika Dam, Greece: Connection pile cutoff wall with central impervious core

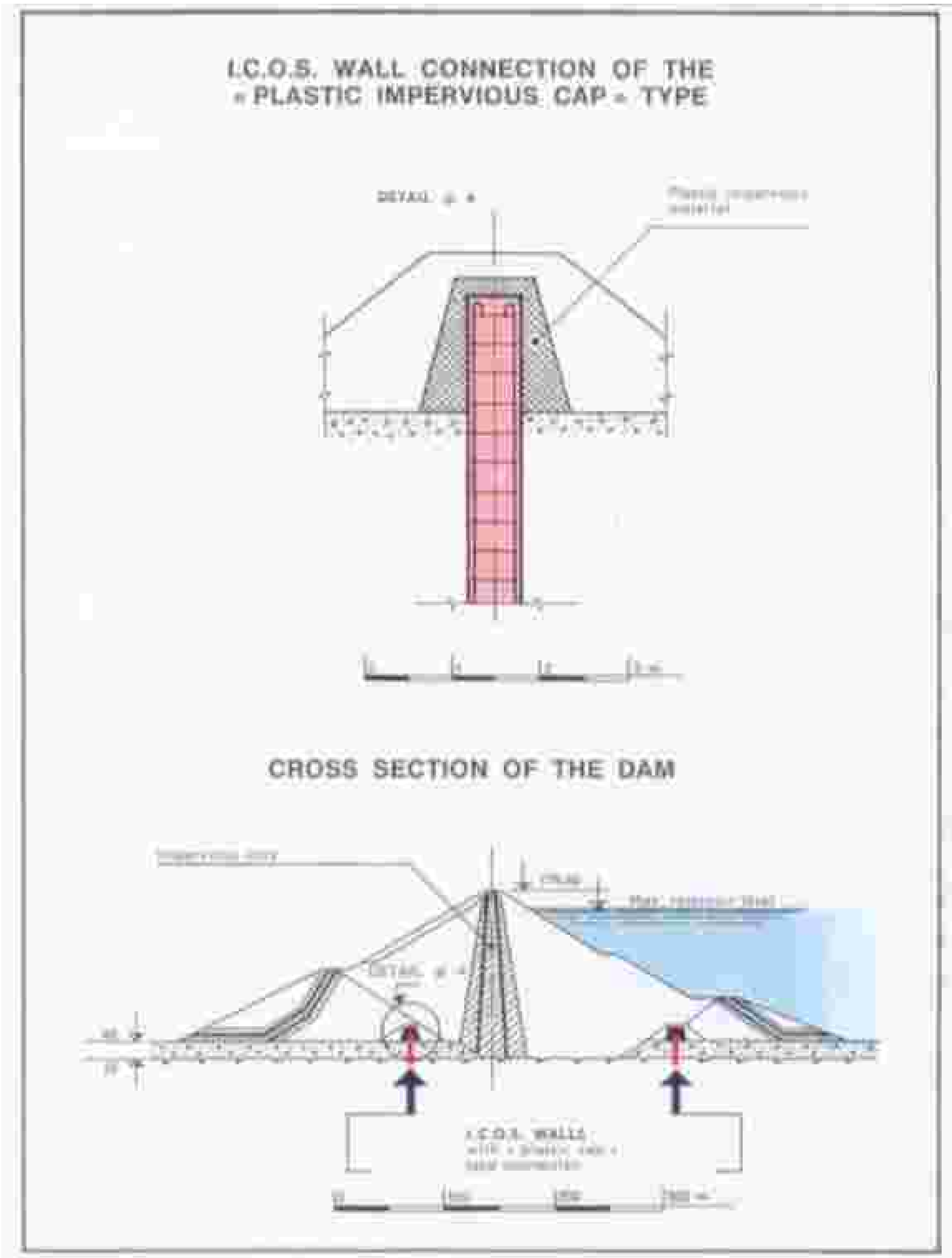


Fig. 4 El Infernillo Dam, Mexico: Connection of pile cutoff wall to pre-cofferdams to enable excavation of alluvium to place core of main dam on rock

The ICOS method of constructing cutoff walls has been widely used also in Japan. An example presented here is the Yagisawa wing embankment dam, completed in 1965 with a maximum depth of 41 m and a thickness of 0.60 m. Fig. 5 shows the maximum cross section with the ICOS wall in the in the centerline of the core and penetrating 1.8 m into the bedrock. The dam crest is at El. 859.00 and the reservoir level at 854.5.

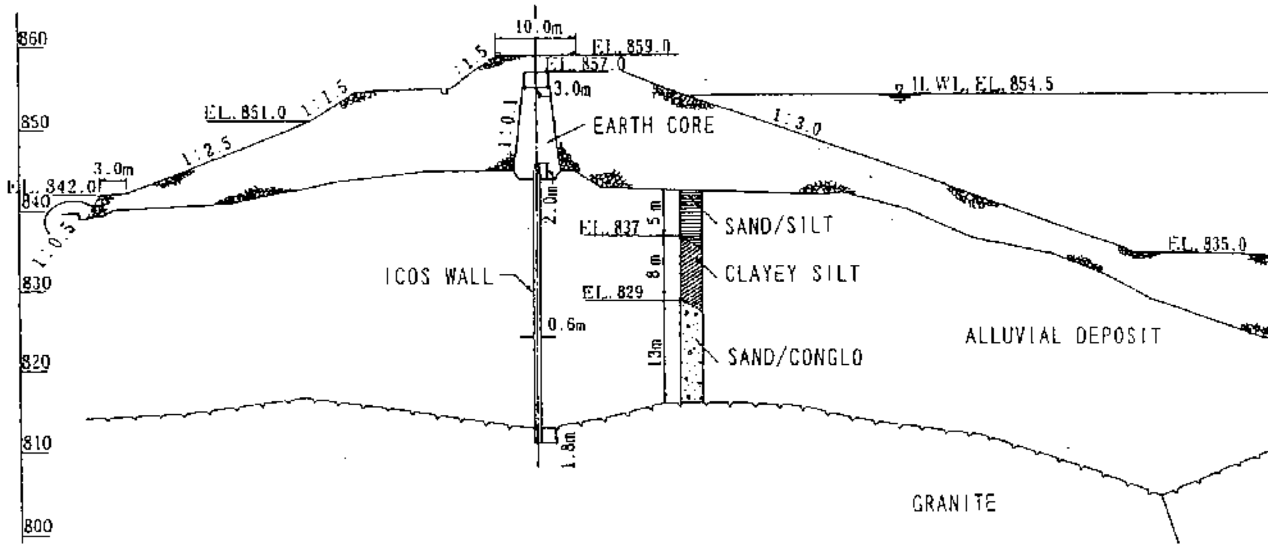


Fig. 5 Cross section of Yagisawa wing embankment dam with ICOS type pile wall

A longitudinal section of the ICOS wall is given in Fig. 6. The earth core only extends over a length of about 26 m, i.e. over the deepest zone of the valley. The top of the core is at El. 857.0. On both sides of the core there are concrete walls embedded in clay (see Fig. 6)

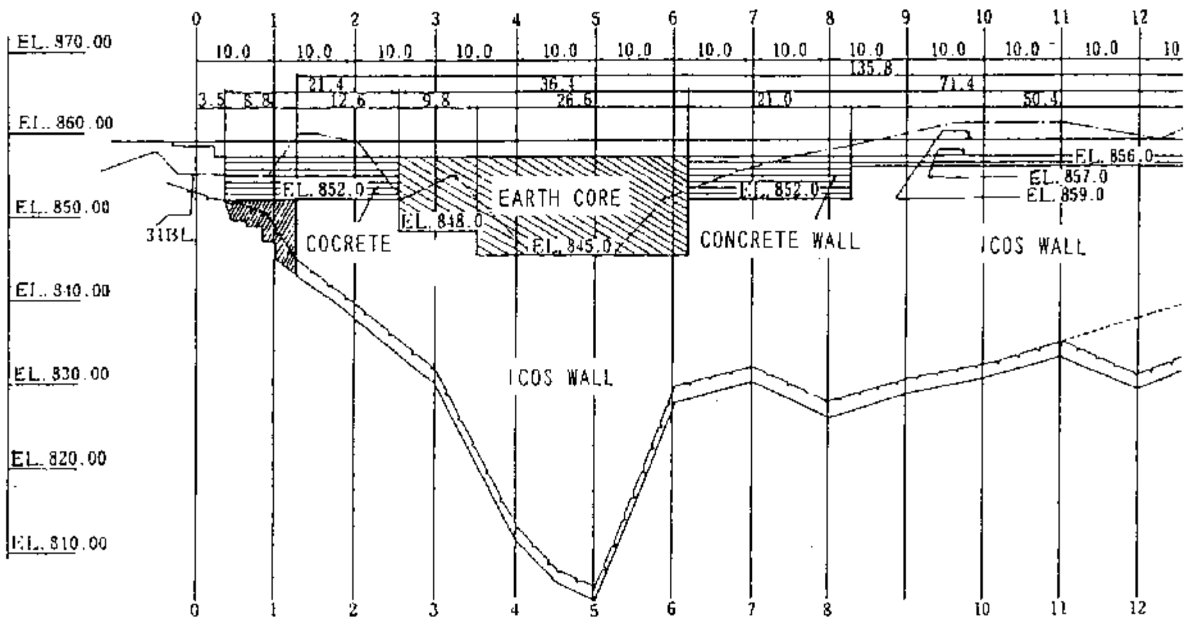


Fig. 6 Longitudinal section along part of ICOS cutoff wall showing location of earth core section flanked by concrete walls on either side

Details of the core zones are displayed in Fig. 7 and details of the wall elements are visible from Fig. 8.

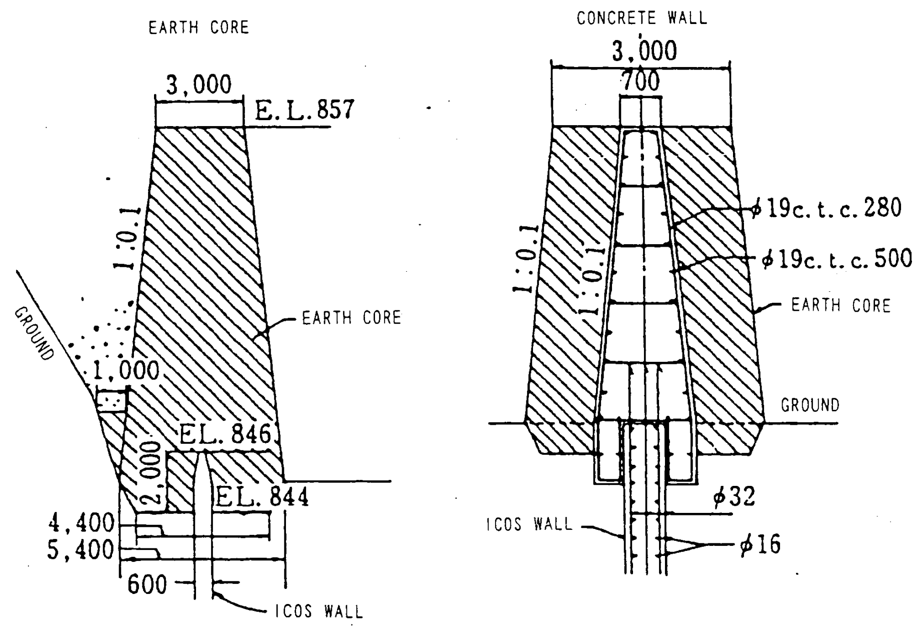


Fig. 7 Two types of cores connected to the ICOS wall. Left: Full clay core with ICOS wall penetrating by 2 m. Right: Core consisting of concrete wall embedded in clay. ICOS wall is connected to concrete wall.

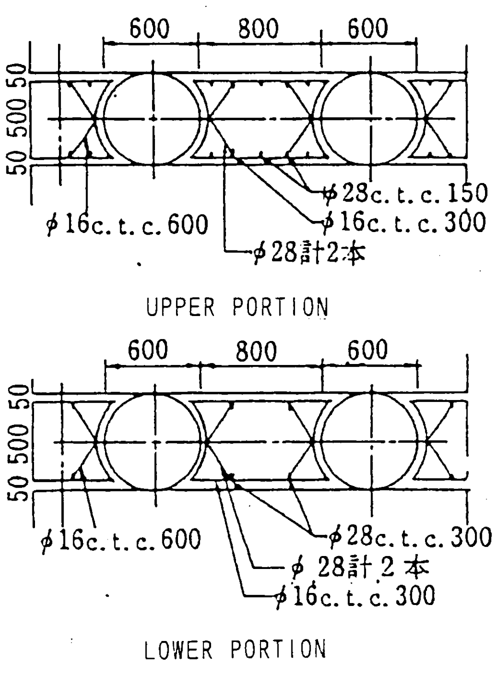


Fig. 8 Shape and arrangement of ICOS wall elements (cylindrical pile and bi-focal panel). Panels are reinforced.

A3.2 Zoccolo dam (Italy)

A3.2.1 Project description

Zoccolo dam is located on the Ultimo River which is a tributary in the upper course of the Adige River in the Province Bolzano, Italy. Construction was completed in 1964. The dam is a homogeneous earthfill structure consisting of compacted gravelly sands of morainic origin. The sealing of the earthfill is accomplished by a three-layer asphaltic concrete upstream facing, which is underlain by a thick layer of filter-drainage material. Seepage control is by a cutoff wall at the heel of the dam and a toe drain at the downstream foot, together with a row of six relief wells. A filter blanket on the upstream face of the toe drain and below it prevents the loss of fines from the foundation. Water passing through the impervious face into the filter zone is conveyed to a manifold in the heel gallery by means of pipes provided with valves. A layout of the embankment is shown in Fig. 1 and a section in Fig. 2.

The main characteristics of the dam are:

- Maximum height: 66.5 m
- Crest elevation 1144.50 m asl
- Crest length: 516 m
- Earth fill volume: 1.4×10^6 m^3

A3.2.2 Foundation conditions

The bedrock in the area of the dam site is lying at a depth in excess of 100 m. It consists of mica-schists, phyllites and paragneiss. The right wing of the embankment is founded on a morainic mound (compression wave velocity, v_P = 1900m/s and hydraulic conductivity $k=10^{-1}$ to 10^{-3} cm/s), which is covered by a relatively thin layer of slope talus. On the left bank, the first 40 to 50 m consist of outwash materials (v_P = 1350 m/s) followed by fluvio-glacial deposits (actually a remolded and sluiced moraine). In the river bed there are a few layers of lacustrine silt interbedded with alluvial materials. These deposits thin out towards downstream until they practically disappear just downstream of the dam axis. Fig. 3 shows a geological section through the foundation materials.

A3.2.3 Foundation treatment

The main element to control seepage is a concrete wall consisting of alternating circular and bi-concave piles. This system is normally known under the name of I.C.O.S. piling system (see Section 5.8.1). The pile diameter, and thus the thickness of the wall, was 0.60 m. The depth of the piles varied between 45 and 55 m but nowhere did they reach the bedrock. The choice of an I.C.O.S. type pile wall, i.e. continuous concrete piling, was based on several technical and economic considerations, corroborated by in-situ and model tests. Other sealing methods considered were an upstream blanket and a vertical grout curtain. The upstream blanket was abandoned because of difficulties in connecting it with the silt layers in the alluvial deposits next to the dam. The silt layers were sensitive being under artesian pressure and any disturbance could have triggered liquefaction phenomena. Alluvial grouting was considered to be of little efficacy because of the presence of silt in some parts of the granular material which would inhibit the penetration of the grout.

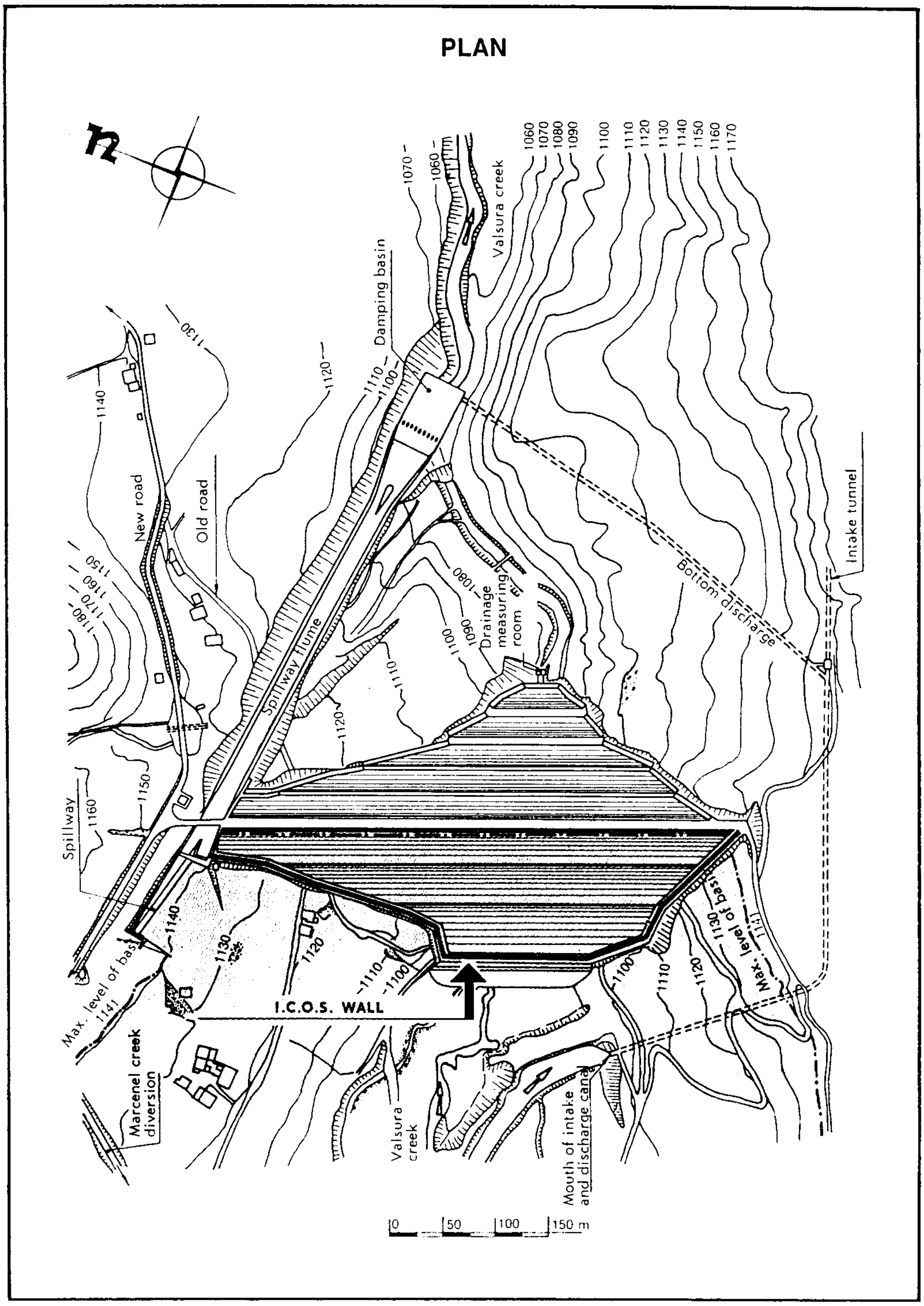


Fig. 1 Zoccolo dam: Layout of embankment and apurtenances

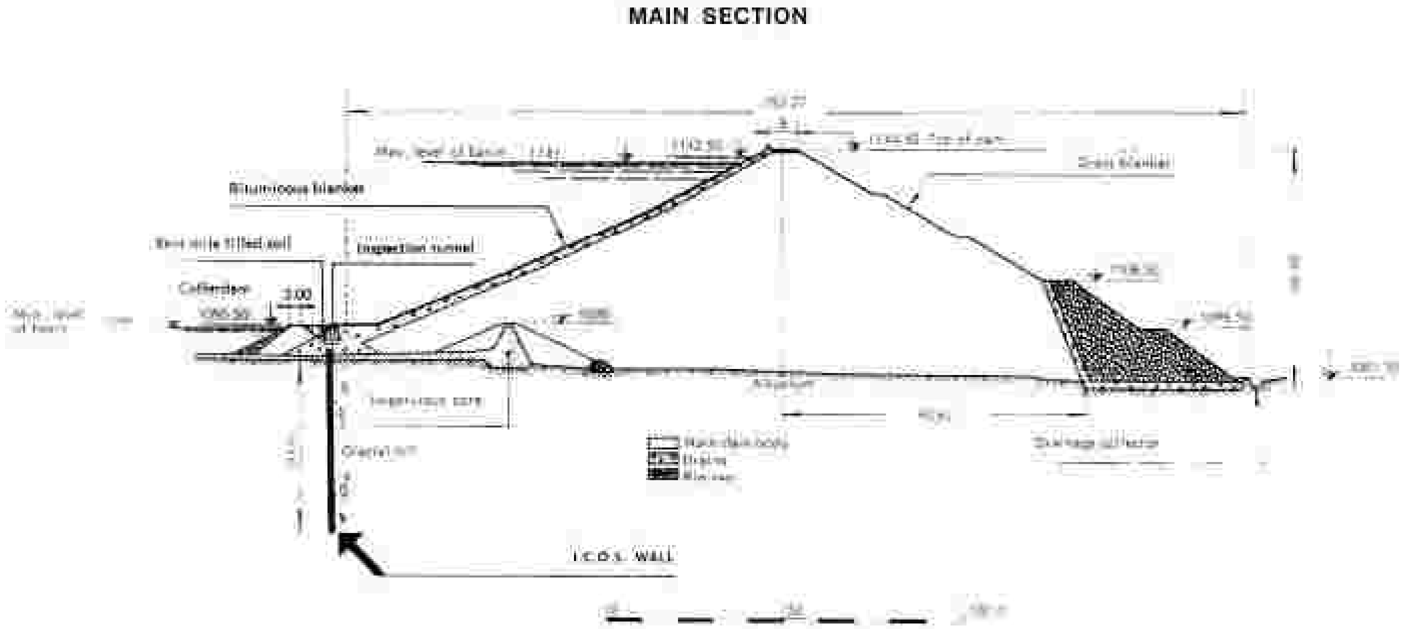
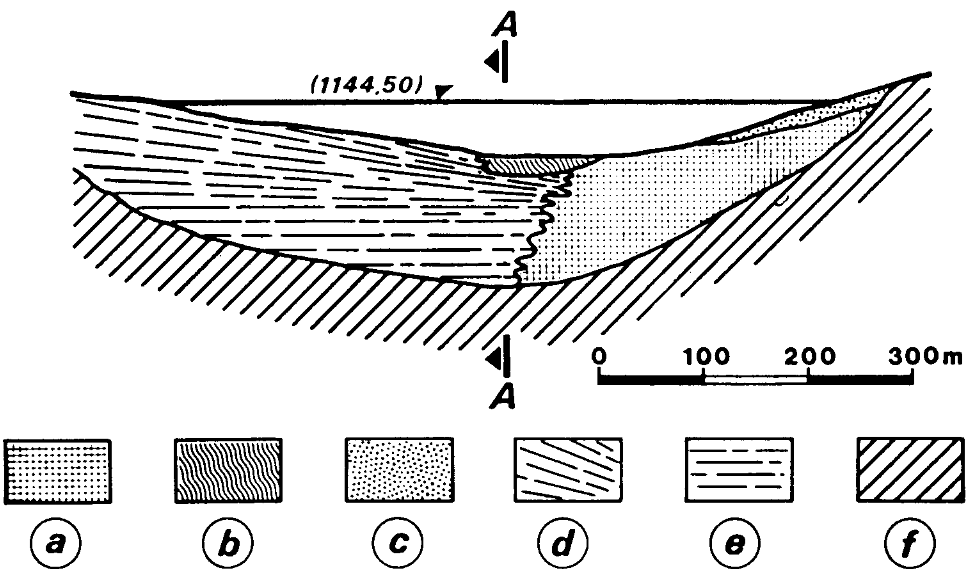


Fig. 2 Zoccolo dam: Maximum section



(a) moraine
(b) alluvial
(c) talus
(d) fan outwash
(e) fluvio-glacial
(f) bedrock

Fig. 3 Geological section along dam axis

The I.C.O.S. wall follows in plan the heel of the embankment with the elevation of its top remaining more or less constant in the central part of the valley and raising on the abutments. The wall is reinforced at the top over a variable depth of 2.7m to 7.3 m, with six vertical bars (Ø 20 mm) on the interlocking secondary elements and only with construction dowels in the primary elements. The reinforcement was needed to connect the wall to an inspection gallery that followed the wall along the heel of the dam. Construction of the pile wall started in 1961 and was completed, after 36 months, during the summer of 1964. Thirteen drilling machines were used to execute this job. The ground contained boulders with a diameter of up to 2.7 m. There entire wall consists of about 1100 pile elements making up a barrier of 33,100 m^2 in area. The price for 1 m^2 of wall during the period 1961/64 came to about 70 US$. Before awarding the piling job the owner decided to construct, on the right bank close to the heel of the dam a test pit of polygonal shape, 1.85 m inside diameter, and 38 m deep. The wall consisted of eight primary and eight secondary elements reaching to a depth of 45 m. The water table was at a depth of 22 m. The wall turned out to be perfectly watertight under this hydraulic head and piles did not show any cracks.

In order to estimate the hydraulic gradients that would exist during operation of the dam, model tests were carried out which demonstrated that in no case these hydraulic gradients would approach those critical for a 50 m deep cutoff. Seepage loss from the soil below the cutoff was estimated to range between a few hundred liters per second to 0.5 m^3/s.

A3.2.4 Wall construction

Primary piles have a center to center spacing of 1.20 m. They serve as guides for the secondary piles. The principle of construction is described in Section A3.1.

The concrete used for the piles at the Zoccolo dam site had the following mix proportions:

- Max grain size 35 mm
- Cement 300 kg/m3
- Water/cement ratio, w/c ~ 0.8
- Additive: Plasticizer

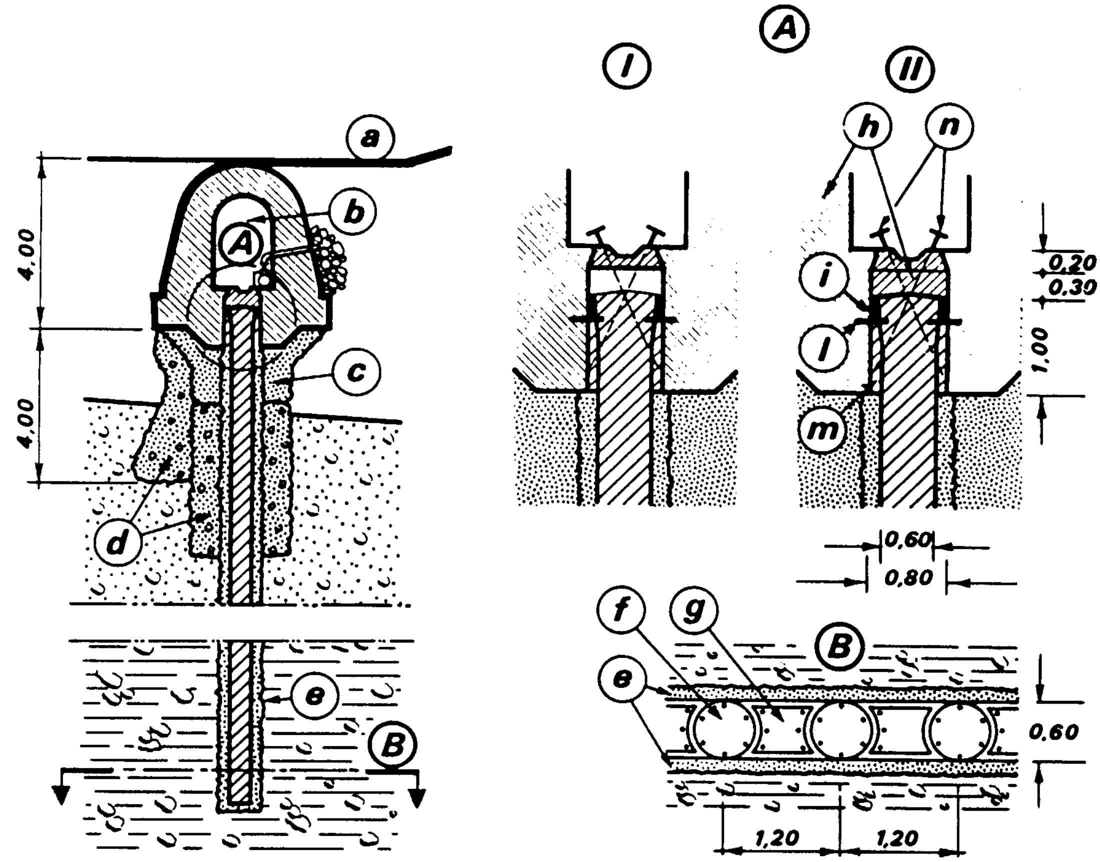
Bentonite consumption was about 280 kg per m^2 of wall. This corresponds to about 2 m^3 of slurry. A good proportion of the bentonite stayed in the ground as a result of penetration into the alluvium adding an additional barrier to the flow of water.

A3.2.5 Connection of wall to impervious upstream facing

The connection of the pile wall with the upstream impervious face was accomplished by means of a head gallery, as shown in Fig. 4. This gallery also collects the drainage water from the filter layer below the impervious facing. The idea was to separate completely the gallery and the pile wall such they can move independently under the imposed water load and the earth pressure. The gallery was expected to move downwards relative to the cutoff wall and to tilt towards the downstream side.

The connection between wall and gallery was accomplished in stages such that displacements could take place before the gap between wall and the gallery floor was fully sealed. Geodetic points were installed on the gallery walls and on the top of the pile wall.

These points were connected to an exterior triangulation net through a gallery leading to the toe outlet of the embankment. PVC water stops sealed the gap between gallery walls and the pile wall head when the first trial impounding with about 20 m of water head was started. The water stop was overlain by a neoprene mastic that adhered to the gallery wall providing an additional line of defense. Differential movement of gallery and pile wall occurred and it was decided to close the gallery floor by pouring slabs of reinforced concrete in a checkerboard pattern.



(a) asphaltic facing
(b) Inside of gallery
(c) Cement grouted zone
(d) Cement and bentonite grouted zone
(e) Bentonite and penetrated zone
(f) Primary pile
(g) Secondary pile
(h) Concrete
(i) Neoprene mastic
(j) PVC water stop
(k) Grouted zone
(l) Injection pipe

Fig. 4 Gallery structure along the heel of the dam

The second trial impounding nearly reached the normal storage level and further movements of gallery and pile wall took place. The missing reinforced concrete slabs were then poured and the space between the floor of the gallery and the top of the pile grouted.

A3.2.6 Performance

Deformations

Gallery and pile wall continued to settle up to the fourth cycle of impounding when the full storage level of 1141 m asl was reached. The gallery settled about 6 to 7 cm, depending on the location, and the wall about 5 cm. The horizontal movements in downstream direction amounted to 8 to 12 cm for the galley and 6 to 12 cm for the pile wall. In addition there was also a slight rotation of the gallery.

Seepage and leakage

The pore water pressures around the cutoff wall were measured by means of standpipes embedded in the piles or along the walls of wells built adherent to the pile cutoff. Under normal storage level of 1141 m asl the water level downstream of the cutoff was at about 1080 m asl, i.e. about one meter below the ground surface at the dam toe. Seepage developed from flow under the cutoff wall and from flow around the dam in the abutments.

Leakage was monitored at two points, one at the toe of the dam in the river bed collecting the water from the filter and the relief wells and the other 250 m downstream collecting also the water from drainage pipes in the abutments downstream of the dam. Under maximum storage conditions the total leakage was measured initially as around 450 l/s. This value decreased to about 350 l/s one year after the first full impounding. The content of fine solids in the leakage water was lower than 0.1 g/liter and considerably less than that of the reservoir water.

However, during the course of operation, leakage from the filter layer below the asphaltic face increased and the position of the phreatic line next to the upstream face moved upwards. Based on investigations this was attributed to a deterioration of the upper part of the diaphragm wall. Repair works were carried out in 1976/1977 and consisted mainly in the construction of a two-row grout curtain with 25 m and 15 m deep holes spaced at 1m, immediately upstream of the diaphragm wall and along its entire length. After refilling the reservoir, the position of the phreatic line was lower again and has remained more or less stationary ever since. (Dolcetta et al., 1999).

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A3.3 Khao Laem dam (Thailand)

A3.3.1 Project description

Khao Laem dam, on the Quae Noi River, is located about 300 km northwest of Bangkok, Thailand. It is a concrete face rockfill structure, rising 130 m from the lowest foundation level, with a crest length of 1019 m. Construction started in 1979 and was completed in December 1984. The main purpose of this facility is hydropower generation with an installed capacity of 300 MW.

A3.3.2 Foundation conditions

The plinth and the dam body are founded on a complex series of sedimentary rocks with variable degrees of karstification. In addition, a major fault zone and several minor faults are crossing the dam axis. In order to ensure watertightness, several methods of foundation treatment were applied. A section along the plinth line is shown in Fig. 1 indicating the different geological zones and the treatment performed. Table 1 gives a brief summary of the geological features of the various zones encountered along the plinth line.

The rock formations encountered at the dam site can be divided into two groups separated by the Three Pagodas Fault Zone, a major transcurrent geological structure of Early Pleistocene age passing through the foot of the right abutment. It has a width of about 30 m and separates the Ordovician Thong Pha Phum Group from the Permian Ratburi limestone. It consists of graphitic shale with frequent clay gouge.

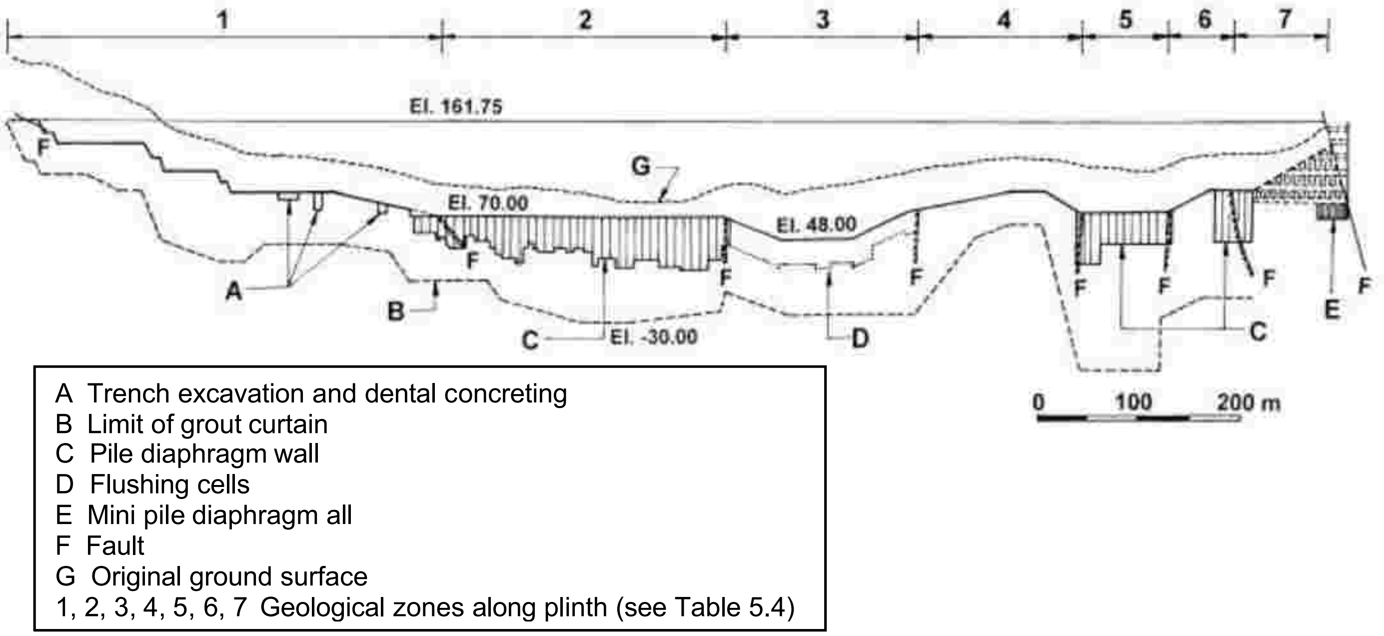

Fig. 1 Methods of foundation treatment along axis of plinth and locations of pile diaphragm walls

Table 1 : Geological conditions along plinth foundation and treatment methods

Project features		Thong Pha Phum Group (Ordovician)						Ratburi Group (Permian)
		Zone 1	Zone 2	Zone 3	Zone 4	Zone 5	Zone 6	Zone 7
Geology	Rock types, Stratigraphy	Calcareous shales, interbedded with some limestone	Argillaceous limestone interbedded with minor shale	Calcareous sandstone, weathering to sandy silt (decalcified sandstone)	Black, non-calcareaous shale	Limestone interbedded with minor shale	Argillaceous limestone interbedded with siltstone and black shale	Limestone
	Geological structures	Bedding and cleavage closely spaced	Bedding closely to moderately spaced; cleavage and jointing moderately spaced	Bedding indistinct, jointing moderately spaced. Steeply dipping fault between Zones 2 and 3	Very closely spaced bedding; intensely sheared and jointed, some clay gouge	-	Very closely spaced bedding, moderately sheared and jointed	Bounded on the left by Three Pagodas Fault and on the right by planar fault; contains zone with very closely spaced calcite veins, termed as "microfractured" limestone;
	Karst features	Occurring locally along geological structures; several large clay-filled cavities	Extensive karst, commonly occurring as solution cavities, partly filled with clay and river alluvium	Minor karst in the form of solution joints, partly infilled with sandy silt	-	Extensive karst along joints and bedding planes, extending to about 50 m below plinth level. Karst weathering up to 150 m below plinth	Minor karst	Extensive karst, many cavities infilled with red earth
Treatment method(s)		Grout curtain; dental concreting	Pile wall; deep grout curtain	Flushing and low pressure grouting followed by high pressure grouting	Grout curtain	Pile wall; deep grout curtain	Deep grout curtain	Superimposed galleries; mini pile wall along "microfractured" limestone;
							Pile wall across Three Pagodas Fault zone	

The Thong Pha Pum Group, which forms the major part of the dam foundation, consists of shales, interbedded with argillaceous limestone, and calcareous and non-calcareous sandstone/siltstone, locally interbedded with limestone strata. This group has been divided into six zones all separated by steeply dipping faults. There is extensive karstification, commonly occurring as solution cavities up to several meters across and partially infilled with silty clay and river alluvium. Karst weathering was found during construction to reach a depth of up to 150 m below the plinth elevation.

The Ratburi Group to the right of the Three Pagodas Fault zone forms the right abutment and consists mainly of massive to thickly bedded limestone with numerous solution cavities and caverns partly or fully filled with clay and sand. Solution features could be observed as deep as 200 m below foundation level. This group is denoted as Zone 7 in Fig. 1 and Table 1.

A3.3.3 Methods of foundation treatment

The full extent of the karst and its character became known only during execution of the foundation treatment work. The original design concept proposed a concrete diaphragm (pile) wall below the plinth in Zone 2 where cavities where known to exist and a grout curtain in the other zones. The thickness specified for the pile wall was 600 mm to a maximum depth of 50 m. The design of the grout curtain envisaged a depth 100 m with a maximum grout pressure of 2.5 MPa and permeability equivalent to a water loss of 3 Lugeons.

The persistence of solution features and infilled cavities to a much greater depth than originally anticipated made it necessary to extend the treatment by a pile diaphragm wall also to the karstic features in Zone 5, and to the fault zone between Zones 6 and 7. The total wall area became thus 15,500 m^2. Similarly, the grout curtain was extended to a maximum depth of 180 m in Zone 5 with a maximum grout pressure of 3 MPa.

Foundation treatment at the Khao Laem dam site can basically be grouped into three types of cutoff methods:

- Positive cutoffs: These included the following types:
    - concrete diaphragm wall by the overlapping pile method, in Zones 2, 5, 6, 7
    - mini diapghragm wall by overlapping piles, in Zone 7
    - concrete diaphragm wall by excavating a short trench in the rock through karst and backfilling it with concrete, in Zone 1
    - continuous concrete cutoff wall between grouting galleries in the right abutment, in Zone 7
- Cavity treatment (e.g. cleaning and filling with concrete)
- Grouting: There were two approaches:
    - Flushing treatment in boreholes with high pressure air and water to remove decalcified infill (silt/sand) before grouting
    - 3-row deep curtain grouting with row spacing of 1.5 m

A3.3.4 Pile walls

In Zones 3, 5, 6 and 7 a Superdrill machine consisting of an Ingersoll Rand down-the-hole pneumatic hammer, model DHD 130, with a 30 inch (762 mm) diameter bottom bit mounted on a Soillmec EC-80 rig was used to construct a continuous wall by the overlapping pile method. The Superdrill was chosen because of its capability of drilling in the hard Thong Pha Phum limestone which had an unconfined compressive strength of up to about 190 MPa.

Construction of the wall was started by drilling a series of primary holes, diameter 0.76 m, at 1.23 m center to center spacing (see Fig. 2). These holes were then filled with concrete using the tremie method. After the concrete had hardened (setting time at least 36 hrs), the intermediate (or secondary) holes were drilled to remove the remaining rock and filled with concrete. In this way, a diaphragm wall with maximum and minimum thicknesses of about 760 mm and 450 mm respectively was created. The average drilling rate was 3.6 m/hr with peaks up to 6 m/hr. All drilling was in water because the water table was about 1 m below working level.

The depth of the pile wall was determined by drilling and water pressure testing 64 mm diameter grout holes along the centerline of the cutoff at 6 m spacing. The maximum depth of significant karst was found to be about 55 m.

Numerous infilled cavities were intersected during drilling the holes for the wall, often resulting in hole collapse and bit jamming. To overcome this problem, the cavities when first intersected, were flushed clean with an air lifter, then filled with concrete and finally re-drilled after 72 hours.

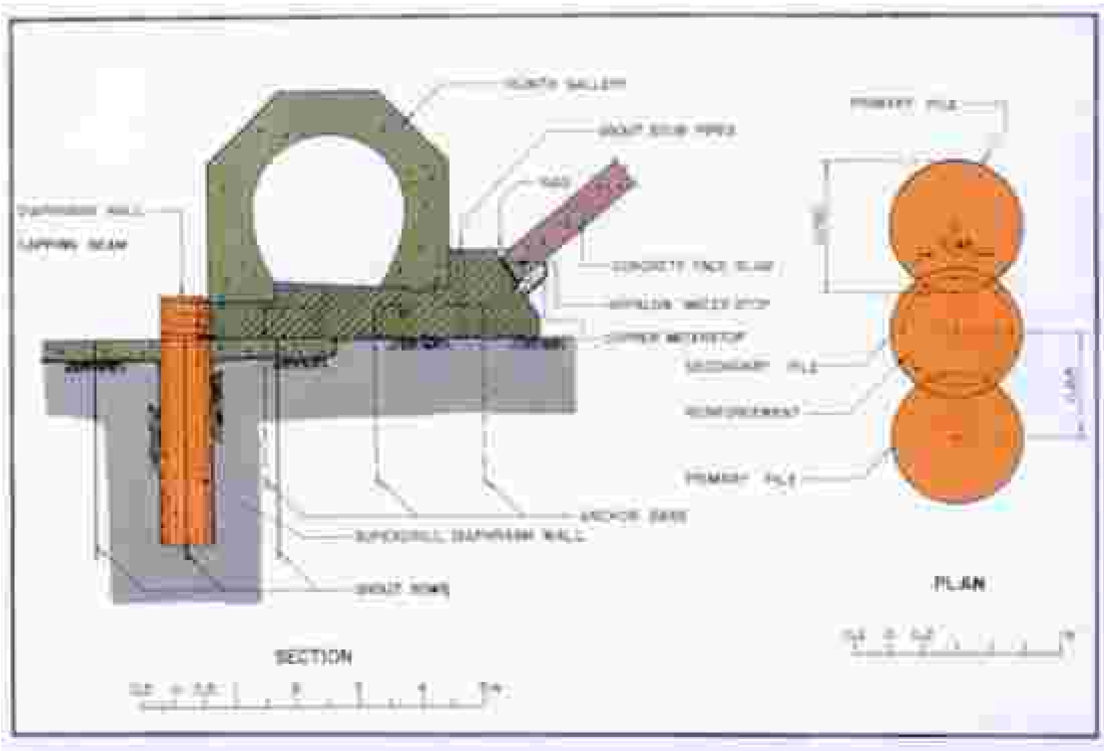

Fig. 2 Plan and section pile wall drilled by Superdrill

Adequate overlap over the full length of each pile was ensured by specifying a tolerance of 0.2 % in verticality. This was checked after drilling using a neatly fitting metal cage suspended on a wire from a large steel tripod set over the hole. The deflection of the hole was then determined by measuring the wire movement for varying cage depths on a scale at the base of the tripod.

The top six meters of the secondary piles were reinforced to connect the wall structurally to the capping beam which was connected to the plinth slab with a hypalen waterstop.

The superdrill was not suited for use on a sloping ground surface because of problems in maintaining stability of the 28 m high guide mast. Zones of isolated major karst on slopes were therefore treated using either the method of trench excavation or by grouting.

A pile wall with a smaller thickness, denoted as "mini" diaphragm wall, was constructed in Zone 7 below the lowest galleries (see Fig. 3). Three zones of major karst occurred in this part of the cutoff and in addition the rock was highly fractured. Because of groundwater problems, it was found most suitable to construct a pile wall with 300 mm overlapping piles to a depth of 15 m. Construction was by a 300 mm down-the-hole hammer. The hole spacing was selected as 225 mm to ensure a continuous wall thickness of 200 mm. Initially, only every seventh hole was drill to reduce the possibility of interconnection. Prior to being filled with concrete, the holes were cleaned by high pressure water followed by air lifting to remove any material remaining at the bottom of the hole. Verticality was maintained at not more than 30 mm in 10 m and checked using the technique developed for the superdrill.

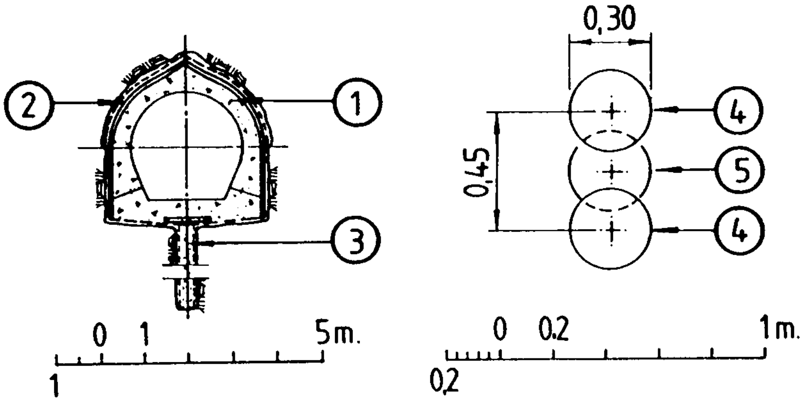

Fig. 3 "Mini" pile wall

If cavities were encountered or a hole collapse occurred, the section of the borehole was consolidated with tremie concrete and then re-drilled after 72 hours. Completed drill holes were backfilled with 20 MPa tremie concrete.

After completion of the pile wall, pressure grouting using conventional split spacing and packer grouting techniques was applied to the rock beneath the pile wall.

The method of overlapping piles for the cutoff wall turned out to be the most effective solution for the very difficult rock conditions encountered at the Khao Laem dam site. The efficiency was verified from a pump-out test carried out about 100 m from the riverbed. It demonstrated that the upstream and downstream water levels had clearly been isolated from each other.

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A3.4 Walter F. George dam (U:S.A)

A3.4.1 Project description

Walter F. George dam spans the Chattahoochee River between the States of Georgia and Alabama, U.S.A. Dam and lock were constructed between 1955 and 1963. The purpose of the facility was to improve navigation and generating power. Even before completion of construction the dam developed seepage problems which subsequently have plagued the dam for over 40 years, in spite of several rehabilitation attempts. Details were reported by Ressi di Cervia (2003) and Simpson et al (2006).

The facility consists of the following components (see Fig. 1):

- a 456 m long concrete structure containing the powerhouse with four generating units of 150 MW capacity, the spillway, a non-overflow section and the 25 m by 137 m lock
- a 1771 m long earthfill embankment (or dike) along the Georgia side
- a 1868 m long earthfill embankment along the Alabama side

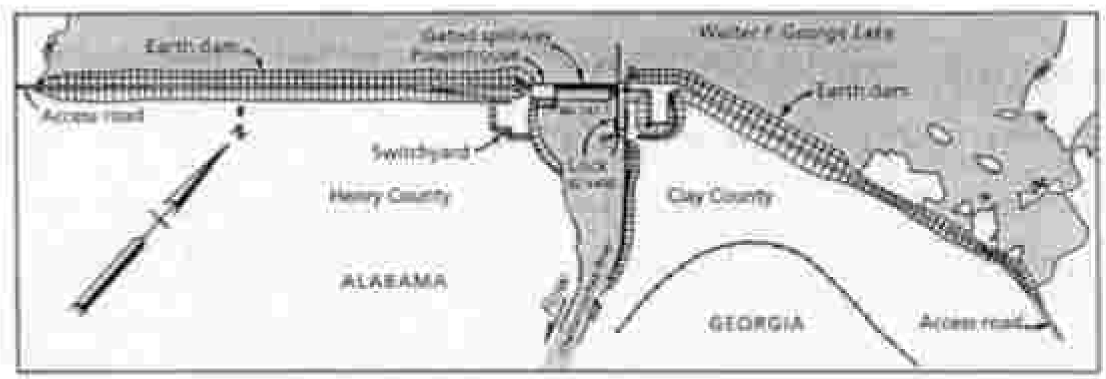

Fig. 1 Walter F. George dam: Layout

A3.4.2 Foundation conditions

The concrete section is founded on rock while the wing dams are placed on alluvium (see Fig. 2). The foundation rock is the Clayton formation which consists of three sub-units, as listed in Table 1 and depicted in Fig. 2). The Clayton formation exhibits karst features with solution channels and cavities, whereas the Providence sand is impervious, at least in its upper part. It appears that the original project did not foresee any foundation sealing measures except probably for some grouting work.

A3.4.3 History of seepage problems

October 1961: two sinkholes developed during the final stages of construction of the Alabama dike, indicating the existence of seepage problems

1962: Boils developed along the downstream toe of the dikes on both the Alabama and the Georgia side. Relief wells were installed and the foundation beneath the dikes grouted. Subsequently, a large spring developed near the downstream lock guide wall.

Table 1 Stratigraphy of the foundation rock at W.F. George dam site

Formation name	Name of sub-unit	Thickness (m)	Characteristics	Compressive strength (MPa)
Clayton formation	Upper earthy limestone	15	Chalky to earthy material containing shell fragments but without bedding planes. Well developed joint system in N-S direction	1.4-2.8
	Middle shell limestone	12	Moderately hard, porous gray coquina type limestone	2.1-6.9
	Sandy limestone	10.6	Variable, with hard solid layers and non-cemented highly compacted sand layers	0.7-131
Providence sand formation			Highly consolidated black marine clay, impervious. Sand content increases with depth	

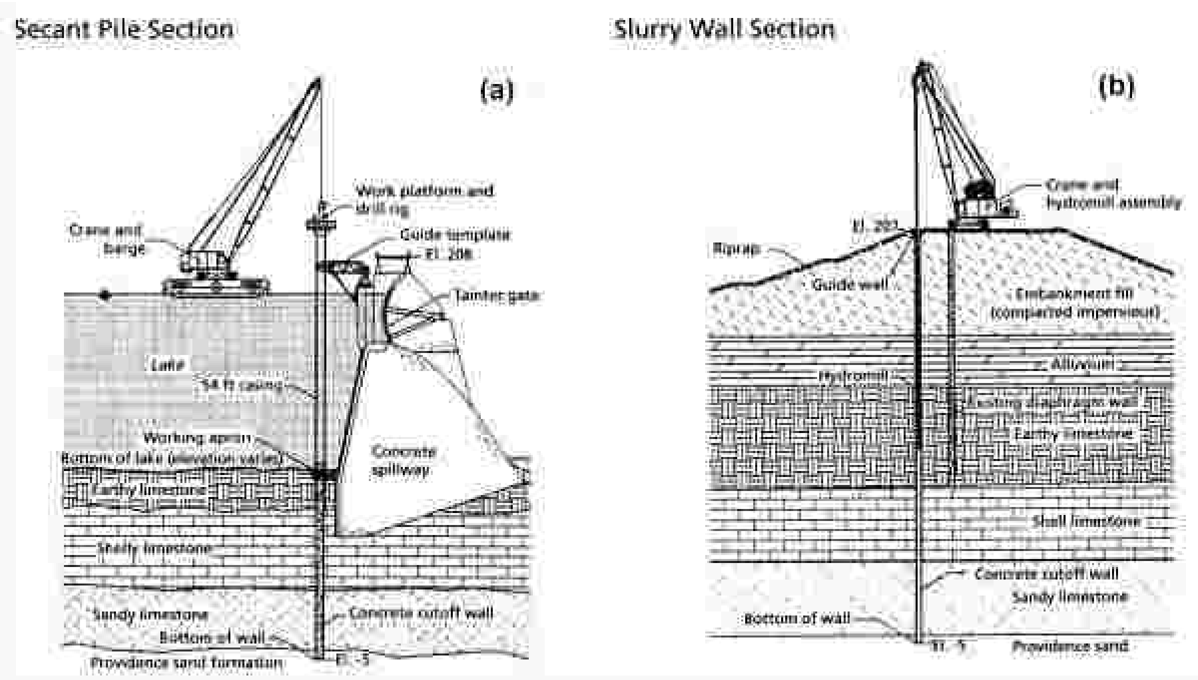

Fig. 2 (a) Walter F. George dam: Installation of secant pile wall immediately upstream of the concrete section of the dam; (b) Installation of plastic concrete diaphragm wall in embankment sections

1968: Sinkhole activity increased. Remedial work included: (1) filling the sinkholes with sand and grouting around their peripheries, (2) installation of piezometers to obtain information on solution channels and fissures in the soluble rock, and (3) construction of a sand-filled filter trench downstream of the lock guide wall to intercept water entering a nearby spring through solution channels.

1970: Digging and grouting of holes downstream of the Georgia dike to reduce seepage.

1971: Several small sinkholes were discovered at the toe of the Alabama dike and pin boils were found in the collector ditch of the relief well located in the same area.

1978: Recommendation to install a test section with a 100 m long positive cutoff in the Alabama wing embankment. This was a 610 mm thick concrete wall reaching to the bottom of the shall layer within the earthy limestone (see Fig. 2), which corresponds to a depth of about 30 m from the top of the embankment. In some sections the wall went even deeper, i.e. to the bottom of the shell limestone.

1981: A cutoff wall similar to the one in the Alabama dike was also constructed beneath the Georgia dike. The main purpose of the cutoffs was to stop the seepage in the alluvium and in the underlying earthy limestone.

1982: A boil appeared immediately downstream of the powerhouse. This boil was caused by the water entering the shell limestone through an old construction piezometer that had been poorly grouted.

Early 1990s: Lake water gains access to the lower strata of the shell limestone through joints in the earthy limestone causing the shell limestone to erode. Thus the situation worsened and threatened to jeopardize the stability of the concrete structure.

1997: The Owner (US Army Corps of Engineers) realizing that (1) grouting would not solve the problem permanently, (2) the cutoff walls installed earlier were not sufficiently deep, and (3) that there was no cutoff stopping the seepage underneath the concrete structure, decided to construct a positive cutoff wall upstream of the main concrete structure to remedy the problem once and for all. There were, however, three restrictions to observe, namely:

- The reservoir could not be drawn down
- Power generation could not be interrupted
- Operation of the lock could be halted for only short periods

A3.4.4 Method of foundation treatment

A 50 million US dollar design/build contract was awarded to a joint venture to construct a 610 mm thick cutoff wall consisting of the following two wall types:

- a diaphragm wall built mainly over land, 93 m long on the Alabama side and 50 m long on the Georgia side. This type of wall was constructed by a hydromill with reverse circulation using the two-phase method with primary and secondary panels. The concrete was plastic with a 28-day compressive strength of 6.2 MPa.
- a secant pile wall built in 30 m deep water immediately upstream of the concrete section of the dam reaching down to Elev. 1.50 m below mean sea level, i.e. about 30 m below the bottom of the reservoir, penetrating into the impervious Providence sand. The pile wall extends laterally through the lock structure, through an underwater retaining wall, and also through the remnants of a steel coffer cell left in place from construction time, before entering the earthfill sections on both sides. Secant piles were considered to be the most reliable and economical method for the

construction of the cutoff in deep water. Emphasis in this case history is on the construction of this pile wall.

The pile wall is 376 m long and consists of 469 overlapping 1320 mm diameter piles installed an average of about 3 m upstream of the concrete structure of the dam. Center to center spacing is 838 mm.

A3.4.5 Pile wall construction

Construction of the secant pile wall was accomplished in seven phases, namely:

- Exploratory drilling and grouting
- Construction of a working apron
- Installation of templates and casings
- Drilling and construction of the pile shafts
- Pouring concrete for the pile shafts
- Construction of the wall through the lock structure
- Tying the pile wall to the concrete structure of the dam by means of a concrete cap

Exploratory drilling and grouting along the planned axis of the cutoff wall was carried out to better define the ground conditions and to fill any large voids in the path of the wall that could impair the progress of work. This preliminary grouting and drilling campaign was considered a valuable activity providing vital information facilitating the planning of the subsequent pile construction phases.

Construction of an apron or platform was needed for the creation of a solid working surface to install the pile wall. This activity was preceded by cleaning the lake bottom removing all the obstructions along the wall axis, as far as this was possible. The Contractor used a remotely operated vehicle (ROV) to determine the position of underwater features. It could also be used for high resolution video and sonar surveys. The construction of the apron was accomplished by underwater excavation of a 1.8 m deep trench directly in front, i.e. 1.5 m from the concrete monolith, and parallel to the dam. The trench was then filled with flowable low-strength (approx. 3.5 MPa) concrete using tremie pipes for placing

To drive and install the temporary casings into the apron, steel templates were used. The position and alignment of the templates were checked using reference points previously installed on the dam during the initial survey.

The cranes used to install the casings were mounted on barges anchored in front of the dam. Powerful cranes picked up the steel casings and lowered them to the bottom of the lake. Verticality was checked by survey instruments. The casing was then vibrated into the working apron. First a series of primary casings were installed followed by a series of secondary casings.

Drilling of the pile shafts was accomplished by a Wirth PB 612 reverse circulation drill rig positioned on top of the casing. The shafts were drilled to a depth of about 70 m below the top of the casing. The cuttings produced by the reverse circulation procedure passed through a 250 mm discharge pipe and were then delivered to a hopper barge. From this barge the cuttings were deposited on the bottom of the lake. When drilling was complete, the Contractor checked the shaft for verticality using a bi-axial inclinometer.

For placing the concrete into the drilled shaft, a tremie pipe was employed to form the pile. The concrete used was a plastic mix with a 28 day unconfined compressive strength of 6.2 MPa.

A difficult task was the construction of the pile wall through the lock and the underwater retaining wall. A diamond wire was used to cut slots into the upper lock guides to maintain structural integrity. After the cut portion was lifted out by crane, the hydromill was engaged to finish cutting through the lock guide walls.

Finally, to provide a seal between the top of the pile wall and the concrete structure, a concrete cap beam was placed on top of the wall. This cap beam is 0.6 m high above the pile wall and 1.8 m between the cutoff wall and the concrete wall of the dam.

The project was completed in June 2004. Piezometer readings downstream of the cutoff wall indicated that the objective of an efficient foundation sealing has been achieved.

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A3.5 Beaver dam (U.S.A.)

A3.5.1 Project description

Beaver dam is located on the White River in Carroll County, Arkansas, U.S.A.). The dam was constructed between November 1960 and June 1966. It consists of a 406 m long and 69 m high, straight, gravity-type concrete structure flanked to the north by a 379 m long main embankment dam and three saddle dikes. The water from the reservoir is used for flood control, power generation and water supply.

Saddle Dike 1 has been experiencing a general increase in seepage rates since the initial reservoir filling. The location of this dike relative to the other structures can be seen in Fig. 1. It measures about 300 m in length and is 9 m high, with the crest elevation at 348.4 m asl. This case history reports on the rehabilitation of Dike 1, based on accounts given by Llopis et al (1988) and Bruce & Stefan (1996).

A3.5.2 Foundation conditions

The Beaver dam site lies near the northeast end of a very gentle, shallow elongate SW-NE trending sedimentary basin, known as the Price Mountain syncline. This basin is often faulted in areas where down-folding is most pronounced. Dike 1 is founded on overburden with a thickness of between 5 and 12 m overlying a down-faulted block or graben structure of the cherty Boone formation. This block extends over a total distance of about 370 m, i.e. from about Station 63+00 to Sta. 75+00 (see Fig. 2). The graben is bounded by steeply dipping normal faults on either side. The Boone formation can be divided into two distinct sub-units. The upper, approximately 30 m thick, part consists of weathered chert, a spongy, vuggy, chalklike, residual material. The lower sub-unit of approximately 18 m thickness is slightly weathered to fresh limestone with silica. Joints in this rock have enabled the passage of water and developed a network of inter-connected cavities locally extending down to Elev. 297 m asl. Usually the Boone formation is above the reservoir water level, except in places where it has been down-faulted.

The Boone formation is underlain by the St. Joe limestone which is non-cherty and contains numerous thin shale seams. The St. Joe limestone, in turn, is underlain by the Chattanooga Shale which is considered impervious.

A3.5.3 History of seepage problems

The seepage problem was recognized before construction started and a two-row grout curtain with hole spacing of 3 m, extending to a depth of 1.5 m below the top of the unweathered rock (except for a 60 m long section where the curtain was deeper) was constructed. The grout was placed by gravity flow.

April to June 1966: During initial filling, seepage was detected in a small gulley (Area SA-1) about 90 m downstream of the centerline of Dike 1 (see Fig. 3). The discharge increased with the rising reservoir level and when it had reached elevation 340 m amsl, eight additional seeps developed with a total discharge of 25 liters/s.

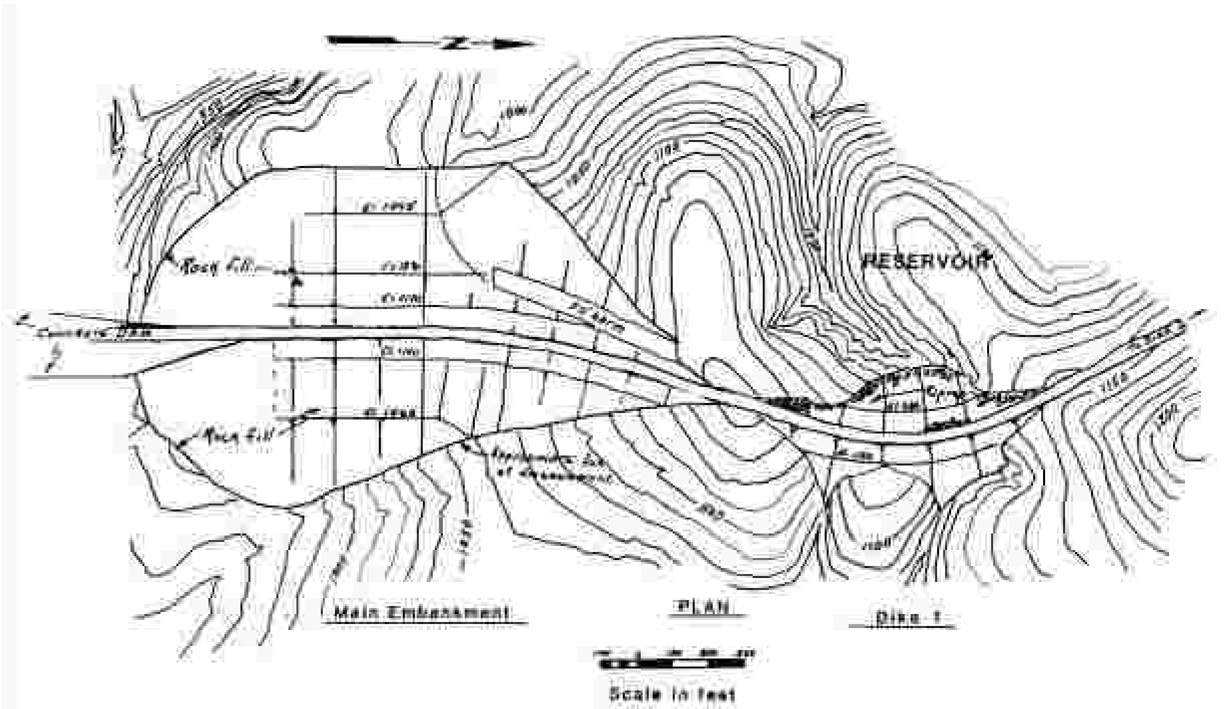

Fig. 1 Beaver dam: Location of Dike 1 relative to concrete dam and main embankment (Bruce & Stefani, 1996)

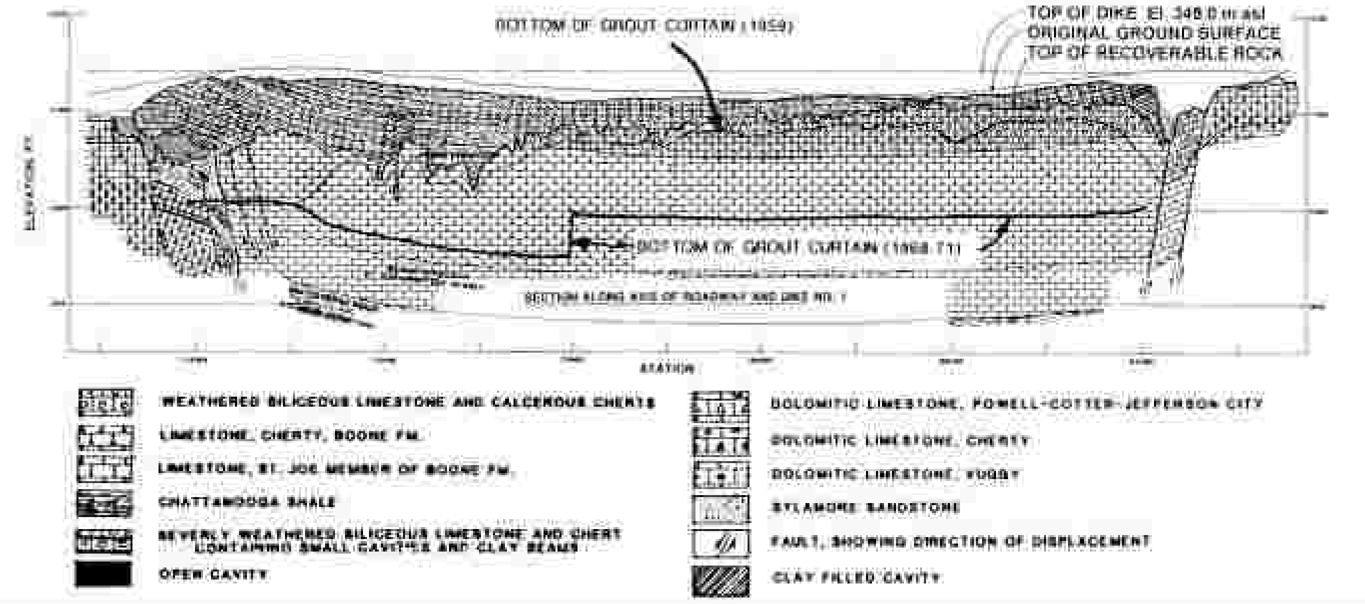

Fig. 2 Beaver dam: Foundation conditions at the site of Dike 1 (Llopis et al., 1988)

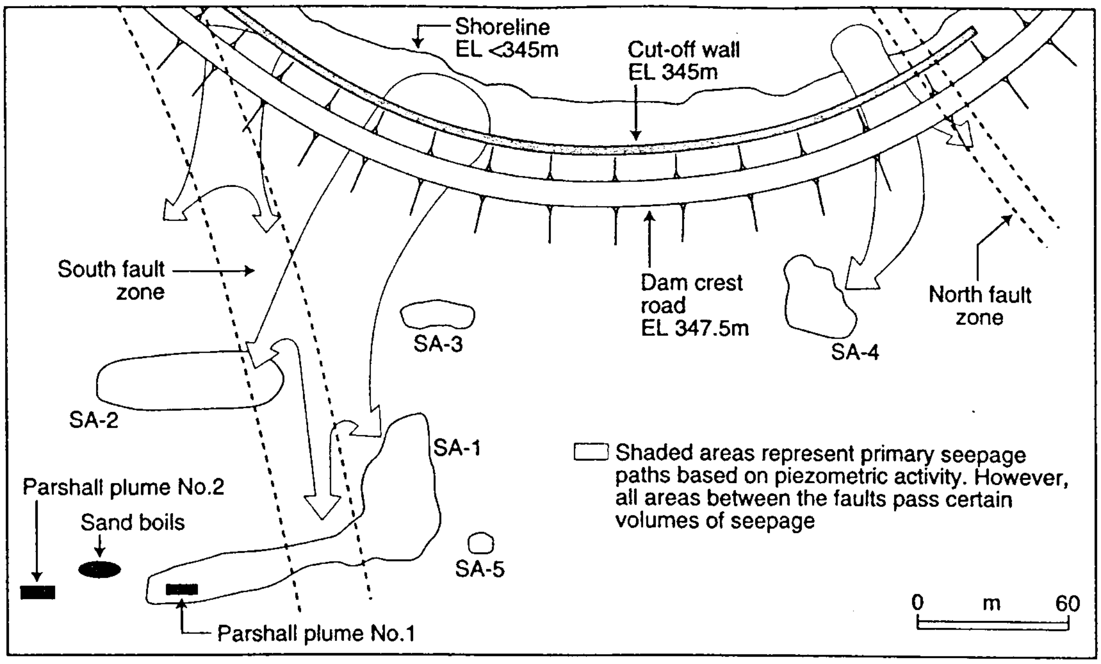

Fig. 3 Beaver dam: Location of major seepage features and of cutoff wall
(Bruce & Stefani, 1996)

Spring 1968: The total discharge from the seeps had doubled to 50 liters/s. From flow measurements and dye tracing it was inferred that the seepage flow took place either through inter-connected cavities below the grout curtain or along the top of the rock. The highest concentration of flow occurred in the vicinity of Sta. 71+00 near the southern portion of the dike (Area SA-1).

July 1968 to December 1971: An extensive grouting program was conducted with the purpose to abate seepage flow in the foundation of Dike 1. Highest grout takes were measured at Sta. 70+50 and 72+00. Grouting, however, was not without problems; these were mainly cave-ins along borehole walls and the failure of grout pumps to grout larger cavities to refusal. This additional grouting program reduced seepage by about 30 to 35 percent (i.e. about 30 to 35 liters/s).

December 1984: A record flood hit the dam site rising the reservoir water level to nearly elevation 345 m amsl. A new exit of seepage was discovered about 150 m downstream of Dike 1. It had a flow rate of 1.6 liters/s and was discharging large amounts of detrital material, ranging from clay to gravel size. The water discharged was muddy. The water of all previous seeps had remained clear.

January 1985: Another new seep was found near the left dike/abutment contact. The Owner decided, for safety reasons, to lower the flood control pool level, but this had little effect on the discharge of the muddy spring.

March 1985: The Owner started a geophysical investigation campaign, supplemented later by exploratory borings, at Dike 1, with the purpose to investigate the seepage regime. The findings from this investigation concluded that the foundation beneath Dike 1 was in an advanced stage of deterioration and that there was a risk that during high reservoir levels (flood), seepage might seek new exit paths and that even a blowout could not be excluded in the overburden downstream of the dike. Studies for watertightening measures of the Dike 1 foundation using a positive cutoff were then initiated.

A3.5.4 Method of foundation treatment

By February 1988, the Owner had designed a concrete diaphragm wall, proposed to be constructed by the slurry trench method. For excavation of the trench, the French-built hydrofraise was mobilized, but it apparently failed because certain beds of the unweathered rock had an in situ unconfined strength of over 170 MPa and could therefore not be excavated economically.

Subsequently, the method of overlapping piles used previously at Khao Laem dam in Thailand (see Section 8.3) was adopted, drilling the bedrock with down-the-hole hammers with drill bit of 864 mm in diameter. Construction of such a wall with a total length of 464 m started in October 1992 and lasted 22 months.

The axis of the wall is offset 20 m upstream of the embankment centerline (see Fig. 3). The wall depth varies from 25 to 56 m (except for one pile with 66 m) (Fig. 4). The total wall area is 19,300 m^2. The individual piles are spaced at 610 mm center to center yielding a nominal chordal joint of also 610 mm. Execution of the wall was in two stages, namely:

- Stage 1: drilling and concreting of a series of primary piles
- Stage 2: installation of intermediate or secondary piles to complete the cutoff (see Fig. 5)

For installation of the piles, the following general rules were observed to avoid disturbing nearby piles being drilled or which had been concreted recently:

- Drilling was only permitted beyond a distance of 9 m from an adjacent open pile not entirely in rock;
- A minimum elapse of 48 hours after completion of concreting in a primary pile before drilling the next successive primary pile;
- Drilling of a secondary pile only when the concrete of the two adjacent primary piles had reached at least an unconfined compressive strength of 14 MPa

A3.5.5 Pile wall construction

Site preparation. A work platform had to be constructed along the axis of the pile wall. Then a trench was excavated through the work platform, the embankment, and the overburden materials down to the top of the weathered rock using slurry wall techniques. The trench was filled with concrete of a strength 21 MPa and was then intended to serve as an "in situ" 1.2 m thick casing for the piles when they would pass through these upper layers. Fig. 4 shows the recorded profile of overburden depth and the wall which was subdivided into four sections based on different sub-surface and construction conditions subsequently encountered.

Equipment. Two drill rigs were used for drilling the pile holes. Each rig was able to drill 21 m in a single pass. Different models of air-powered down-the-hole hammers were used. The main types were the Ingersoll Rand DHD130A and a Sandvik XL24. Each hammer was equipped with an internal check valve to allow it to operate when submerged. Drill penetration rates for primary piles ranged from 2.4 m/h to 6.3 m/h (average 4.3 m/h) and for secondary piles they varied from 4 to 7 m/h.

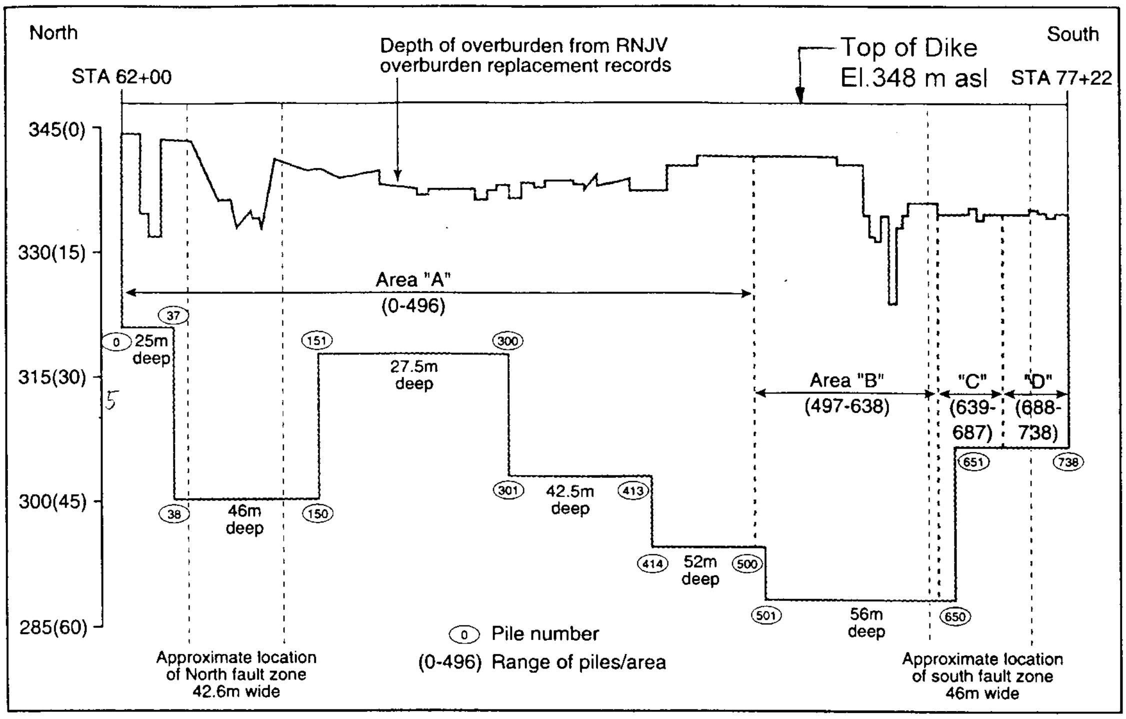

Fig. 4 Beaver dam: Elevation of cutoff wall, showing depth of overburden and pile depths (Bruce & Stefani, 1996)

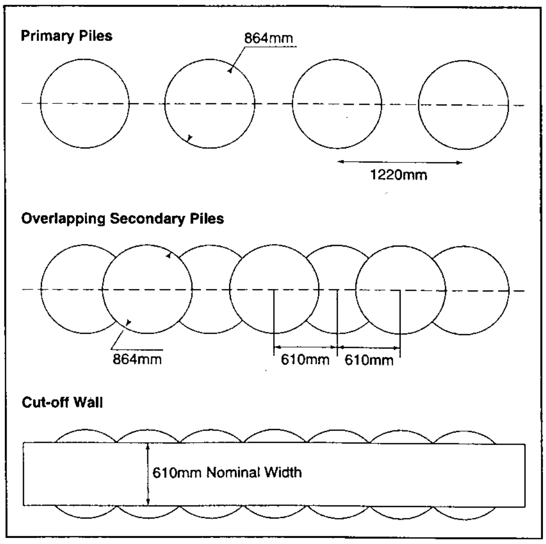

Fig. 5 Beaver dam: Pile arrangement and construction sequence

Pile construction. The following standard steps used in pile construction were applied:

- Setting up the drill rig, using theodolites and lasers
- Drilling using air pressure commensurate with the local geological conditions. Constant monitoring and adjustment if necessary of mast verticality, including after each rod change.

- Extraction of rods, and sounding of the exact hole depth
- Removal, if necessary by airlift, of any soft debris accumulated at the pile toe
- Verticality of the hole verified by a device called a submersible reverse plumb bob (SRPB).
- Placement of concrete via 254 mm diameter tremie pipes fed by a 1.1 m^3 hopper with screen. These pipes were progressively withdrawn during filling, with the tip always embedded 3-6 m in the concrete.

In certain sections of the pile wall with difficult geological conditions caused by the weathered sub-unit of the Boone formation, it was difficult to drill the holes for the piles and special pile installation techniques had to be developed. These were:

- Down staging
- Pre-treatment by pressure grouting
- Hole stabilization by normal grouting

The method of *down staging* was applied in sections with instabilities caused by loose material in the weathered rock below the concrete replacing the overburden and the sound rock. These instabilities prevented the continuous drilling to full depth. To overcome such instabilities, the drill rods were extracted and if the loose zone was more than about 0.6 m thick, the hole was backfilled with concrete and later re-drilled through the unstable zone. Some holes required up to three successive treatments of this kind.

In one section of the pile wall a layer of coarse gravel was encountered. A sub-section of 36 m length was then used as a test section for *pressure grouting*. Steel casings of 178 mm in diameter, spaced at 1.2 m, were driven in two rows to the level of competent rock. During their withdrawal cementitious mixes were injected. Piles could then be installed through this grouted zone without the need for down-staging.

In sections A and B of the pile wall (see Fig. 4) settlement of the work platform occurred and sinkholes appeared. The sinkholes were excavated, examined and then back-filled with concrete. Grout treatment was then applied to work platform, embankment, overburden, weathered rock, and the top 0.5 to 1.5 m of the sound rock. This was achieved by percussion drilling to the depth of sound rock. Then the drilling bit was changed to a rock roller bit and the rods re-introduced. Grout was then pumped through the rods and the bit and simultaneously mixed with the unstable material while also filling possible voids. In this way the drill holes for the piles could be stabilized (*stabilization by normal grouting*). The method was repeated until stable bedrock was reached. Because of the large amount of cavities in these sections of the wall, the volume of concrete used for the pile wall was much in excess of the theoretical value.

Concrete mixes. The concrete mixes were prepared in a 153 m^3/h capacity automatic batching plant in the vicinity of the dam site and from there transported to the cutoff wall by 7 m^3 truck mixers. The mixes produced varied during the course of work in response to the experience gained. The most commonly used mix had the following proportions:

- Coarse aggregate 950-990 kg/m^3

- Fine aggregate 760-810 kg/m^3
- Cement 290-240 kg/m^3
- Fly ash 60-80 kg/m^3
- Water 160-136 liters/m^3
- Reducer N 0.4-0.5 kg/m^3
- Reducer 1 0.3-0 kg/m^3
- Air entraining agent approx. 0.1 kg/m^3
- Calcium 0-1.0 kg/m^3

A3.5.6 Performance of pile wall

Of interest is the efficiency of the pile wall in reducing seepage. Data on seepage quantities were obtained from existing monitoring devices. The seepage area of greatest concern was SA-1, about 90 m downstream of Sta 71+00. This was also the first location where seepage could be observed during reservoir filling. Another area with comprehensive monitoring was SA-2.

Instrumentation consisted of a piezometric network in the three embankments and of discharge measuring devices for the seepage exits downstream of Dike 1, as follows:

Seepage area SA-1: Two Parshall flumes, no.1 and no. 2.
Parshall flume no. 1 measures the discharge from SA-1 and
Parshall flume no.2 that of all the seepages from Dike 1.

V-notch weir measures the quantity of surface water from
area SA-1.

Seepage area SA-2: French drain weir connecting to Parshall flume no. 2
Artesian well

After construction of the pile wall the seepage discharges measured at Parshall flumes no. 1 an 2 reduced drastically, i.e. from a maximum of 80 liters/s to 0.3 liters/s. At the V-notch weir the flow decreased from a maximum of 9 liters/s to zero, i.e. the spring dried up.

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A3.6 Wolf Creek Dam (USA)

A3.6.1 Introduction

The following case history is largely obtained from excerpts from Zoccola et al (2009) and Zoccola & Valagussa (2009), the most recent publication. Earlier accounts on the history of this dam and the problems encountered can be found in: Dunn (1977) and Fetzer (1979 a & b)

In 1968, about 17 years after first being impounded, wet areas, muddy flows in the tailrace and sinkholes in the downstream toe of the embankment signaled serious foundation seepage problems at Wolf Creek Dam. The Nashville District, US Army Corps of Engineers, USACE, began an emergency investigation, instrumentation, and grouting program that was generally credited with saving the dam. Data generated revealed an extensive interconnected network of solution features in the limestone foundation and an inadequate foundation treatment measures taken during construction.

It was decided grouting alone could not be relied upon as a long term solution in such geologic conditions. Thus, from 1975 through 1979, a concrete cutoff wall was installed through the embankment and into the rock foundation. Since that time the project has been closely monitored. Based on instrumentation readings, investigations, and visual observations, it is apparent seepage has found new pathways through features left untreated by the first cutoff wall.

The Nashville District is currently installing a new wall upstream of the existing wall to a greater depth and lateral extent to cut off remaining seepage paths.

A3.6.2 Project description

Wolf Creek Dam is on the Cumberland River in South Central Kentucky near Jamestown, Kentucky, USA. It provides flood control, hydropower, recreation benefits, water supply, and water quality benefits for the Cumberland River system. Construction began in 1941 and was interrupted from 1943 to 1946 as a result of WWII. The reservoir was impounded in December of 1950. The 1,750 meter-long dam is a combination earthfill and concrete gravity section.

The concrete portion consists of 37 gravity monoliths beginning with Monolith 1 at the left abutment and extending 550 meters across the old river channel to Monolith 37 at its tie-in with the embankment. It has a maximum height of about 80 meters above founding level. The spillway section has ten 15.2 by 11.3 meter tainter gates. To the right of the spillway section, the power intake section has penstocks feeding six turbines rated at 45,000 kilowatts each in the powerhouse downstream. Non-overflow sections on either end complete the concrete portion of the dam.

The embankment section extends from the end of the concrete gravity portion 1200 meters across the valley to the right abutment. It has a maximum height of 65 meters above top of rock. The non-zoned embankment is composed of well-compacted, low plasticity clays, from the valley alluvium. The embankment wraps around the end concrete monolith number 37. This critical section is termed the "wraparound" section.

Lake Cumberland, impounded by Wolf Creek Dam, is the ninth largest reservoir in the United States and the Corps' largest reservoir east of the Mississippi. At its power pool elevation of 220 meters, the project impounds about 4.940 million cubic meters. At its flood control pool, elevation 232 meters, it impounds over 7,400 million cubic meters. An aerial view of the dam is shown in Figure 1.

Fig. 1 Aerial Photo of Wolf Creek Dam

A3.6.3 Project Features Related to the Problem

Site Geology: The geologic formations beneath Wolf Creek Dam pertinent to the seepage problem are the Catheys Formation and overlying Leipers Formation. Both are hard, thin to massive bedded, argillaceous limestone interbedded with thin, well cemented, calcareous shale. Based on regional observations throughout their outcrop area and confirmed by foundation construction photographs and subsequent investigations, both of these formations are characterized by a well developed and interconnected system of karst features with some cavities up to 12 meters in size. The most severe solutioning is found along two well defined joint sets oriented approximately parallel and perpendicular to the dam axis, along bedding planes in the essentially flat lying rock, and along the contact between the Catheys and overlying Leipers formations. The solution features are partially or mostly filled with soil. Investigations have shown most of the heavily solutioned limestone is generally confined to the upper 12 to 15 meters of the top of rock which, beneath the dam, generally encompasses all the Leipers and about the top 2 meters of the Catheys.

The base of the concrete monoliths vary by location with some founded on the Catheys and others on the Leipers Formations. The majority of embankment rests on the valley alluvium that overlies the Leipers formation. It varies in thickness from about 15 feet near the right abutment to about 40 feet near the old river channel.

Embankment Foundation treatment: The original design and construction techniques of the 1930's and 1940's used at Wolf Creek were inadequate to control seepage beneath the dam in the karst geology. Design considerations did not fully account for the extent or the impact on performance of the karst features. The alluvium was left in place under the majority of the embankment and did not allow the designers the opportunity to inspect the condition of the rock. Except for the cutoff trench, no foundation treatment occurred beneath the embankment. The design depended on a narrow, steep sided cutoff trench with a single line grout curtain to block seepage in the foundation. The cutoff trench was designed to be under the upstream face of the embankment and parallel to the dam axis except at its left terminus where it turned and tied into the last concrete monolith number 37. It was designed to be 3 meters wide at the base with steep 1.5H:1V side slopes. A schematic of the cutoff trench is shown at Figure 2.

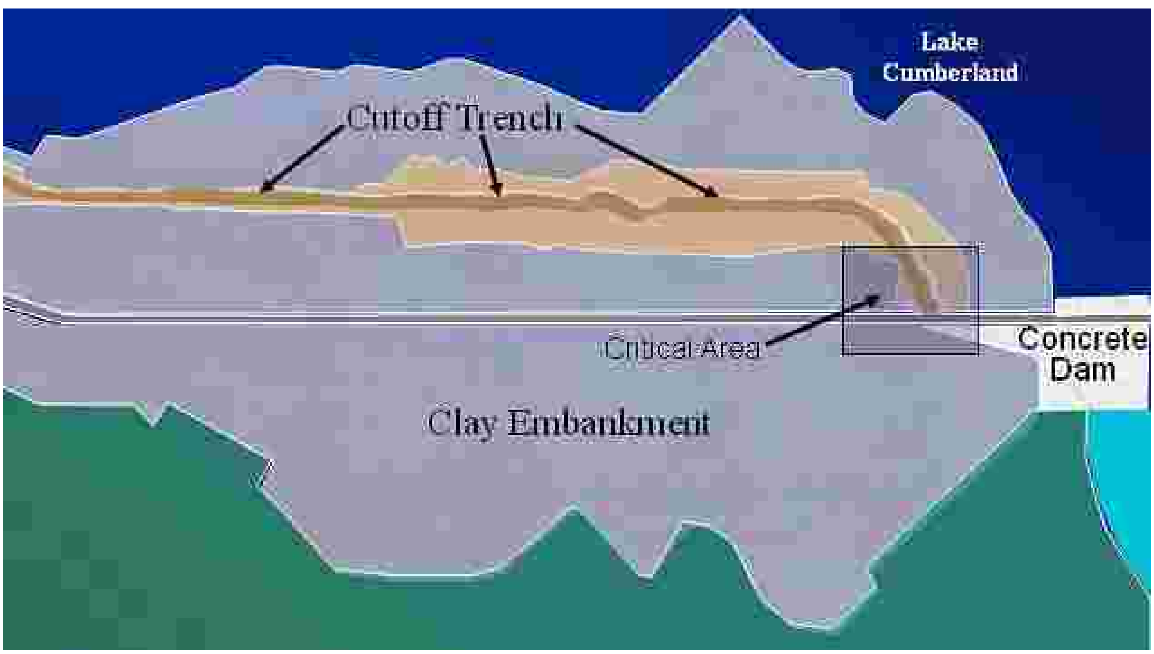

Fig. 2 Location of Cutoff Trench in Relation to Embankment and Concrete

Early during construction of the trench a solution channel was intercepted running generally along the planned trench alignment and the decision was made to clean out and use this feature as the trench. Several large caves and numerous other solution features of varying size intercepted the trench at generally right angles.

Figure 3 illustrates the solution channel/cutoff trench just upstream of its tie-in to Monolith 37. Note the large caves in the trench face. The man standing in the trench bottom gives an idea of the size of the openings.

Fig. 3 View of Solution Channel/Cutoff Trench

Proper placement and compaction of impervious fill against the side walls, with filter protection, and plugging intercepting solution features were not considered important as long as a 3-meter width of compacted material in the center of the trench was achieved. The sidewalls of the trench were therefore left steep, irregularly shaped, and with overhangs that prevented tight contact and good compaction between the fill and rock. Placement and compaction was often by hand in solution features and under rock overhangs. This design philosophy was summed up in a caption on a trench construction photo which stated, "Overhangs and loose rock will be removed only where they cross the line of the trench, since the earthfill in the sides of the trench will have the function only of stability and not of an absolutely uniform tight contact with the trench walls. Tamping will supplement the regular rolling of the fill as required under the overhangs and irregular salients." Figure 4 illustrates the lack of concern about the condition of the trench sidewalls. The figure presents pictures of the same location before and after side wall clean-up in preparation of fill placement. Not much difference is evident.

The cutoff trench can be viewed more as a conduit for seepage than a cutoff. As water moving through the rock intercepts the trench, it is likely moving along the poorly compacted contact between the fill and rock and then crossing the trench at weak points. Bridging of the embankment is likely across the narrow, steep sided trench and cracking and hydro-fracturing of the fill is a possibility. The trench stepped down from the right abutment to its tie-in at Monolith 37 in several high vertical steps or benches. These are potential locations of differential settlement in the trench fill that would cause cracking. Typically, these steps coincided with solution features crossing the trench. Thus, concentrated flow in solution features may occur at cracks in the trench fill which provided an avenue for through seepage. Having no filters, it is likely trench material has been piped into open features.

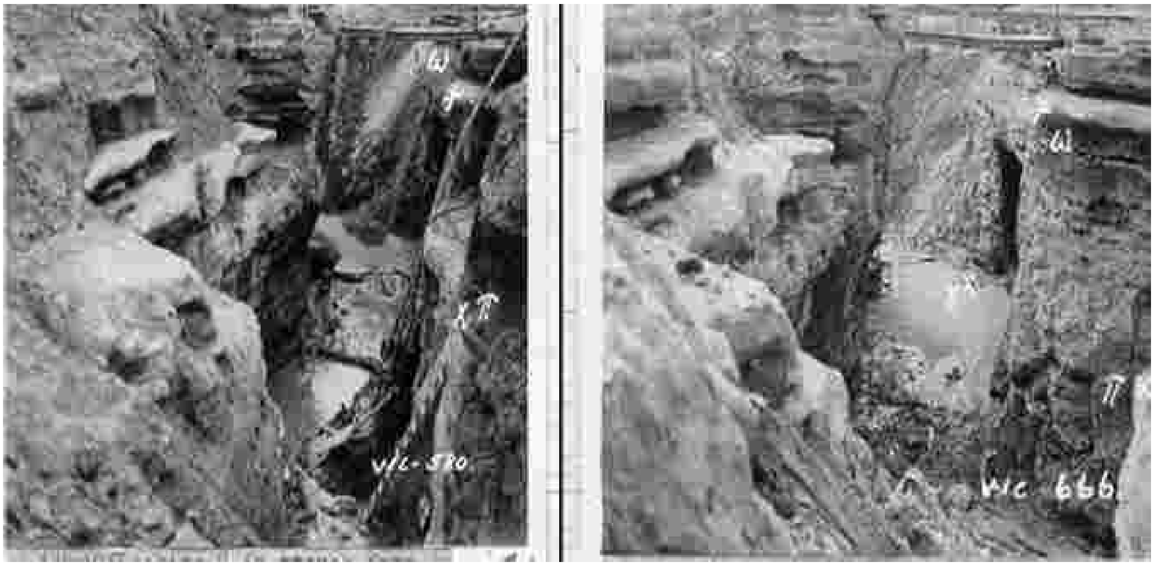

Fig. 4 Before and After Photos Showing Trench Clean-up

Thus the trench was inadequate and resulted in a cutoff that has allowed seepage under it, across it, and along its alignment rendering it ineffective.

Concrete Dam. According to the foundation records, the treatment of the foundation likely left poorly treated or untreated solution features in the rock below the right six monoliths. A modest number of holes were drilled to proof the quality of the rock below the founding elevations. An expanded joint approximately parallel to the dam was encountered in the downstream third of Monoliths 26 through 37 and caves were exposed in Monoliths 37 and 31. Given the numerous solution features exposed in the excavation for the monoliths, additional features are likely to exist below the founding elevation.

A3.6.4 Distress Indicators

The project was operated with few visible distress signs until 1967. This period was prior to the current Dam Safety Program and before any performance monitoring instrumentation was installed in the dam.

Right Downstream Wet Areas: The earliest anomalies observed were wet areas near the downstream toe toward the right abutment. First identified in 1962, these areas became too wet for grass cutting by 1967. In 1967 a small sinkhole was found near the embankment toe in the general vicinity of the wet areas. It was dug out to a depth of about two meters without encountering rock and was backfilled with crushed rock.

Muddy Flows and Sinkholes: Muddy flows were observed in the river about 50 meters downstream of the powerhouse on 7 October 1967. In March and April 1968, two sinkholes developed near the downstream toe above the switchyard in the wraparound area. The sinkholes developed to a maximum size of about four meters in diameter and extended to top of rock about 12 meters below the surface. The second sinkhole was about 8 meters upslope of the first sinkhole. Figure 5 shows the first sinkhole.

Fig. 5 1968 Sinkhole in Toe of Embankment above Switchyard

Emergency Grouting and Cutoff Walls: Immediately following the first sinkhole, the USACE embarked on an emergency exploration, instrumentation, and grouting program. The emergency grouting program lasted from 1968 to 1970. It resulted in about 9000 cubic meters of grout solids being placed in the rock foundation with the majority in the highly solutioned rock in the wraparound area. This grouting program was generally recognized as saving the dam. The investigations indicated that seepage was occurring through and/or under the cutoff trench and through the system of solution features. This seepage was piping material filling these features and subsequently overlying embankment material that collapsed into the voids. Eventually this progression of piping and collapse worked its way to the surface resulting in the sinkholes. Dye tests showed that the system of solution features went through the sinkhole and muddy flow areas.

Because grouting in clay filled features was not considered a permanent fix, a Board of Consultants comprised of Dr. Ralph Peck, Dr. Frank Nickell, and Mr. Francis Slichter was convened to develop a permanent remedy. They concluded a concrete cutoff wall was needed for the long term reliability of the dam. Two walls were recommended and subsequently installed. One was located downstream between the switchyard and river to protect the switchyard foundation from the surging and eroding action during power generation. This wall has performed this purpose well. The second was located along the crest of the embankment. The Board recommended this wall extend the full length of the embankment into the right abutment and extend to a depth at least 5 feet below the Catheys/Leipers contact.

The USACE undertook a pre-installation exploration and grouting program from 1970 to1975 along the alignment of the embankment cutoff wall consisting of borings on 0.75-meter centers. This program served two purposes. It grouted openings along the wall alignment to prevent potential problems with the wall installation. It also provided information on the condition of the rock that the USACE used to select the founding depths and lateral extent of the cutoff wall. Based on these explorations, the bottom of the wall varied in its termination depth. Contrary to the original recommendation, only at two locations was the wall carried below the Catheys/Leipers contact and the majority of the wall terminated in the upper Leipers formation. Laterally the wall tied into the end of concrete Monolith 37 and was carried about two-thirds of the distance toward the right abutment. The designers held the view that grouting would seal the relatively minor openings in the rock indicated in the pre-installation exploratory holes below the selected founding depths and beyond the ends of the wall. A profile showing the limits of the wall is shown in Figure 6.

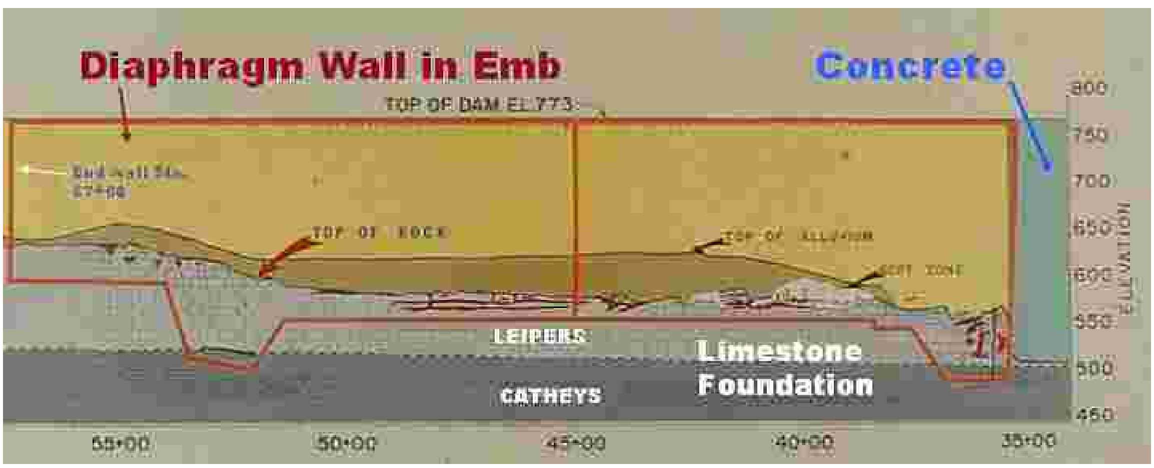

Fig. 6 Profile of Cutoff Wall Looking Upstream Showing Depth and Lateral Extent

<u>Original Cutoff Wall:</u> The first cutoff wall was installed beginning in 1975 and completed in 1979. A typical plan and profile of the wall elements are shown in Figure 7. The wall consisted of circular primary elements spanned by a secondary element. The 0.66-meter diameter steel encased primary elements were installed through the embankment using step down casing and excavation of the material through slurry. Once at design depth the casing was filled using tremie concrete. A boring was extended through each primary element and into the rock foundation 6 to 12 meters. The boring was then grouted to seal any features below the bottom of the wall. Primaries were placed on 1.4-meter centers. Specially designed machinery that tracked between the primary elements was used to excavate the bi-concave secondary elements under slurry. Completion of this secondary element with tremie concrete flanked by two primary elements represented one wall panel.

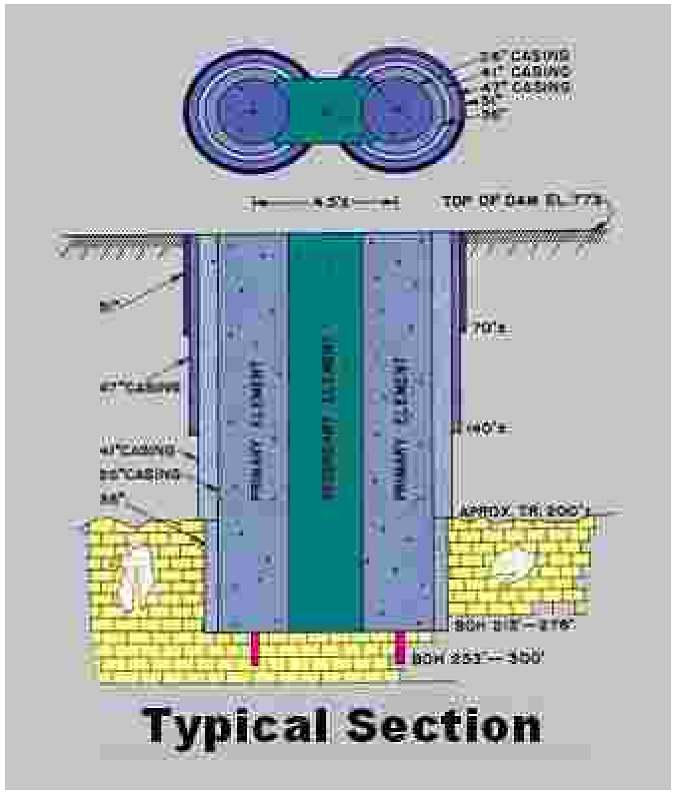

Fig 7 Plan and Section Showing Primary and Secondary Elements of Wall

A3.6.5 Post Wall Construction Performance

In retrospect, the decisions made concerning wall depth and length contribute to the reoccurrence of problems seen today as installation of the grout curtain and partial embankment cutoff wall have failed to adequately cut off seepage. The original wall has worked well in cutting off features it intercepted. It simply did not go deep enough or extend laterally far enough to intercept all the significant features. Subsurface investigations and other indicators of distress confirm features still exist that have not been cut off. Over time, seepage has found these new paths under and around the ends of the wall and is once again increasing.

Since completion of the wall in 1979, the District has been monitoring various indicators of performance. A variety of instrumentation has been installed over the years. These consist of piezometers, displacement monuments, uplift cells, weirs, inclinometers, and alignment plugs. In addition, observation of physical manifestations of the foundation seepage problems in the embankment and downstream areas is done routinely. Project personnel inspect the embankment and downstream areas daily for signs of problems. A brief discussion of some of the performance indicators follows.

<u>Wet Areas Downstream of the Dam:</u> After the grouting program and wall installation, most of the wet areas disappeared. However, over time, persistent wet areas redeveloped primarily near the right end of the dam along the downstream toe. In the early 1980's

drains were installed in wet areas to permit grass cutting as mowers were having maneuverability problems even during the relatively dry summer months. Since 1990 the extent of the wet areas has steadily increased reaching the maximum extent in March of 2004 after a two and a half year interval of sustained high lake levels.

Piezometric Levels: Because of its history, Wolf Creek Dam is extensively instrumented with piezometers. Since 1968, over 300 have been installed. Currently 150 piezometers are being monitored monthly and more frequently as conditions dictate. A select group of 25 in critical locations are read weekly.

Water levels in piezometers immediately upstream of the wall with screened intervals set in the rock foundation are equal to and react with the lake level. This illustrates the lack of head loss across the cutoff trench and its ineffectiveness as a seepage barrier.

Downstream of the wall it was expected foundation pressures would drop to a small percentage of head water levels. However, immediately after installation only a slight reduction occurred and levels remained higher than anticipated. It was concluded the pressures would dissipate over time. However, this has not occurred and in fact several critically located piezometers have risen with 2 piezometers in the wraparound section reflecting a 4-meter rise since 1984. Two embankment piezometers downstream of the wall have high levels and respond to headwater changes. Additionally, five piezometers generally located downstream of the embankment toe read at their top of riser and either flow continually or during high pool events. Flow amounts are slight – approaching a trickle but illustrate the increasing seepage.

Settlement Monuments: Subsidence of the embankment crest as measured by surface monuments is occurring in the wraparound area. A monument in this area settled 46 mm in the period from its installation in 1981 to 1997. From 1997 to September of 2004 it settled another 46 mm which represents both continuing settlement as well as an increase in the rate.

Embankment Investigation: In 2002 and 2003, 12 borings were drilled in the embankment using the resonant sonic drilling method. These holes were at various locations downstream of the wall. Six of the borings encountered soft zones within the embankment. One hole located 1.2 meters downstream of the wall encountered 2.1 meters of very soft, saturated clay at the top of rock. Additionally, two other borings more than 30 meters downstream of the wall in the wraparound section also encountered soft material at the top of rock.

Temperature Survey: In September 2004, a temperature survey of the screened interval of project piezometers was performed. Two cold spots identified in the survey were attributed to foundation seepage. Cold Spot 1 was present at the interface between the embankment and concrete, reinforcing the suspicion seepage is occurring beneath the masonry section. About 37 meters downstream of this is Cold Spot 2, which registered cold temperatures in 2 piezometers with their tips set at different elevations. Overall, the temperature survey confirmed the seepage of cooler reservoir water past the wall and grout curtain.

A3.6.6 Current Fix

Given the substantial consequences due to complete loss or impaired operation of the dam, the history of problems, and the current distress indicators, the Nashville District, USACE, had serious concerns about the long term reliability of the dam. The District prepared a Major Rehabilitation Evaluation Study that, after evaluating several alternatives, recommended a new concrete cutoff wall. The new wall, currently under construction, (see Figure 8) starts immediately upstream of the right most concrete monoliths and runs the length of the embankment into the right abutment. The wall will be constructed to a depth which is deeper than the deepest sections of the original wall and as much as 23 meters deeper than the majority of the original wall. The founding depth will be at least 8 meters into the Catheys formation, well below the zone of solutioning. To verify the founding depth, a drilling and grouting program will be performed 15 meters below the bottom of the wall. Construction began in 2007.

Fig. 8 Repairs (elevations in feet above sea level)

As of mid-2009, the following construction activities are in progress:

- The reservoir is partially drawn down to accommodate construction.
- A platform has been constructed within the upstream shoulder of the dam. The platform was created using a reinforced-earth wall to support the cut slope within the dam and fill was placed on the outer surface of the dam. Concrete pavement was placed over the platform surface to protect the surface during construction.
- Two grout curtains have been constructed, one upstream of the concrete cutoff wall and one downstream, to fill solution cavities so that slurry loss is prevented during construction of the plastic concrete cutoff wall. Cavities are being filled with a low mobility grout under pressure.
- Cutoff wall construction started about May 2009. The cutoff is being excavated using a 1.8-meter-wide hydro-mill cutter. Plastic concrete mix replaces the bentonite slurry.
- Drilling and grouting to a depth of about 15 meters below the newly constructed cutoff wall to verify the founding depth of the cutoff and to treat any features that might accept grout.

Total cost of the remedial treatment at Wolf Creek Dam is on the order of US $500 million.

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Appendix 4. CASE HISTORIES ON SUPERPOSED CONCRETED GALLERIES

A4.1 La Honda Dam (Venezuela)

Comprehensive descriptions of this dam and its foundation treatment were given by Ferrer & Bajetti (1983), Kleiner (1991) and Figuera et al. (19XX)

A4.1.1 Location and general description

La Honda Dam, part of the Uribante-Caparo Hydroelectric development, is located in the Venezuelan Andes, east of the city of San Cristóbal. The dam lies on the Uribante River. The main structure is a zoned earth dam with a core of compacted clayey sand, 139 m high, 600 m long at the crest, and 929 000 m^3 in volume.

A4.1.2 Geology

Rocks from the Aguardiente and Apon formations (Lower Cretaceous), composed of poorly cemented conglomerate sandstones, with lenses of shale and siltstone, are present at the site. The dam is founded in a deep valley formed by a 50 m wide stream bed, with side slopes varying between 1.3 H: 1.0 V on the left abutment and 1.8 H: 1.0 V on the right. The poorly cemented sandstone that crumbles into sand with silty fines is the main fill material for the dam body (Kleiner, 1991).

Three main joint systems were mapped at the site, with frequencies up to 15/20 per meter, and apertures ranging from 0.001 to 0.2 m, sometimes filled with loose sand (Figuera et al., 19XX). Under high water pressure, the foundation rock mass is highly permeable and erodible. The main geological feature, the La Honda Fault, follows the river-channel resulting in different stratigraphy in each abutment.

The unconfined compressive strength for the sandstone ranged from 45 to 84 MPa; for the shales and siltstones it was on average 15 and 105 MPa respectively.

A4.1.3 Foundation treatment

The treatment for the foundation material immediately below the dam was designed to prevent the embankment material from migrating into the foundation. Both, the compacted silty sands, from the upstream shell and the more plastic core material could pipe into foundation cracks. For a complete description of the necessary treatment see Kleiner (1991). In general terms the following works were performed: excavation of overburden to intact rock, dental excavation and concreting of open joints, shallow blanket grouting below the shells and core, construction of a reinforced concrete seepage slab below the core at the left abutment in areas immediately adjacent to the foundation cutoff wall, and placement of a filter layer below the shells and a compacted clayey sand blanket upstream on the left abutment from the core to the abutment blanket.

During the cleaning works of the left abutment, it became obvious that conventional grouting would not be sufficient to reduce the high permeability of the rock mass. Among the studied alternatives to treat the foundation, the following were considered: a cast concrete wall, a grout curtain and a plastic concrete wall. Chemical grouts were discarded because their costs were about equal to those of a concrete cutoff wall and because of the many uncertainties involved.

The final scheme chosen consists of a concrete wall, 380 m long, variable width between 2.20 m (at the galleries) and 1.3 m (at the panels) and a depth ranging from 70 to 120m (**Fig. 1**). The complete treatment on the left abutment comprises: a compacted clayey sand fill blanketing the abutment upstream of the dam, a concrete cutoff from the base of the core trench into the abutment terminating within sandstone that could be treated by conventional curtain, and a blanket filter and drain system below the downstream shell of the dam (**Fig. 2**).

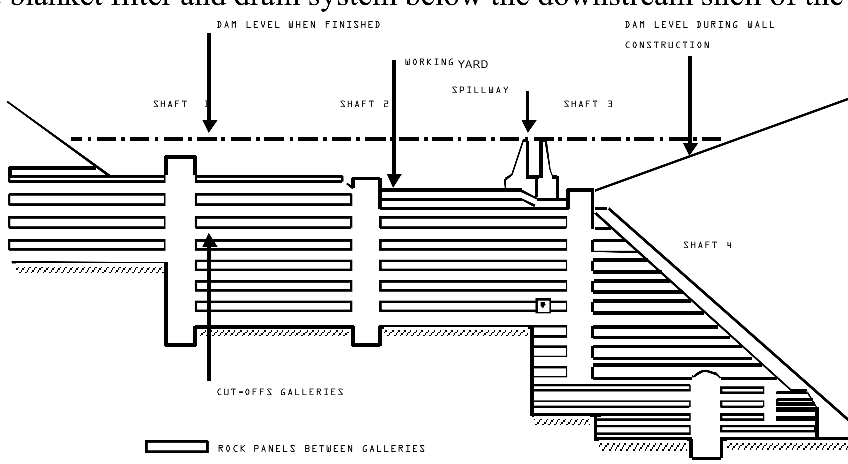

Fig. 1 La Honda dam: Shafts and galleries used for cutoff for wall construction (Cross section)

The construction process for the cutoff demanded the excavation of four shafts, 6.4 m in diameter, 40-90 m deep, separated from each other by 100 m (Figuera et al). The individual shafts were connected to each other by superposed horizontal (multi-level) galleries paralleling the cutoff wall axis at variable intervals depending on the rock quality (**Fig. 1**).

At first, the shafts were excavated using the raise-boring method, but due to frequent collapses, that hindered operations enormously, this procedure was abandoned. The conventional method of mucking from the surface was then adopted. The setup for every level of galleries, and working platform was executed simultaneously with the excavation of the shafts.

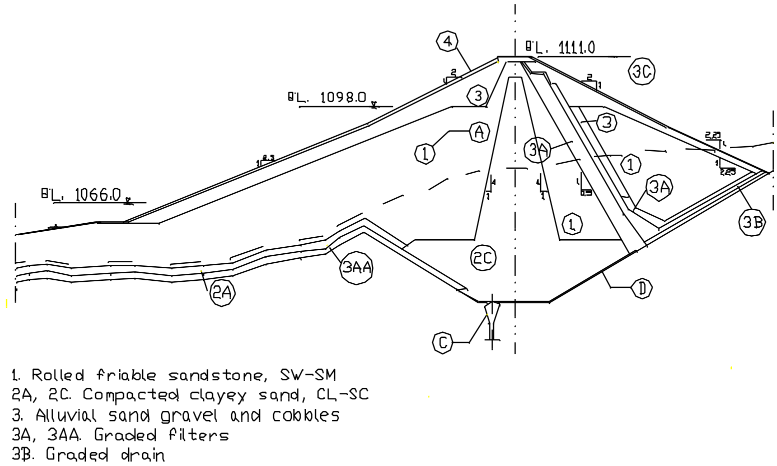

Fig. 2 La Honda dam: Treatment methods at left abutment

Besides the conventional drill and blast, other methods were explored for the excavation of the galleries. Where the rock presented friable characteristics, a high-pressure water jet machine was utilized (**Fig. 3**).

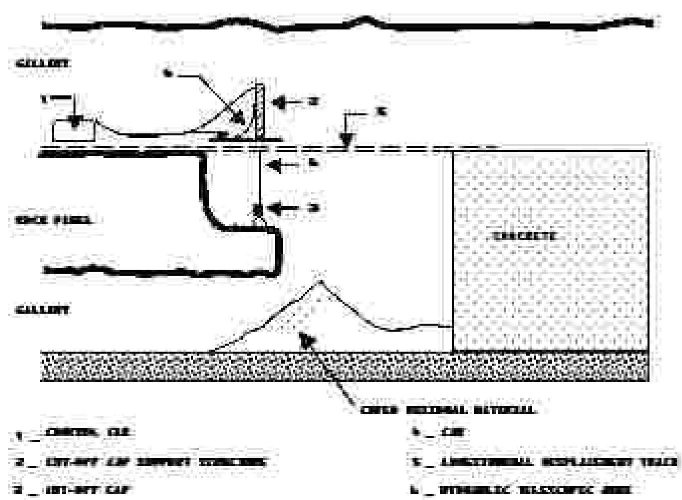

Fig. 3 La Honda dam: High-pressure water jet cutoff procedure to excavate wall panels (longitudinal section).

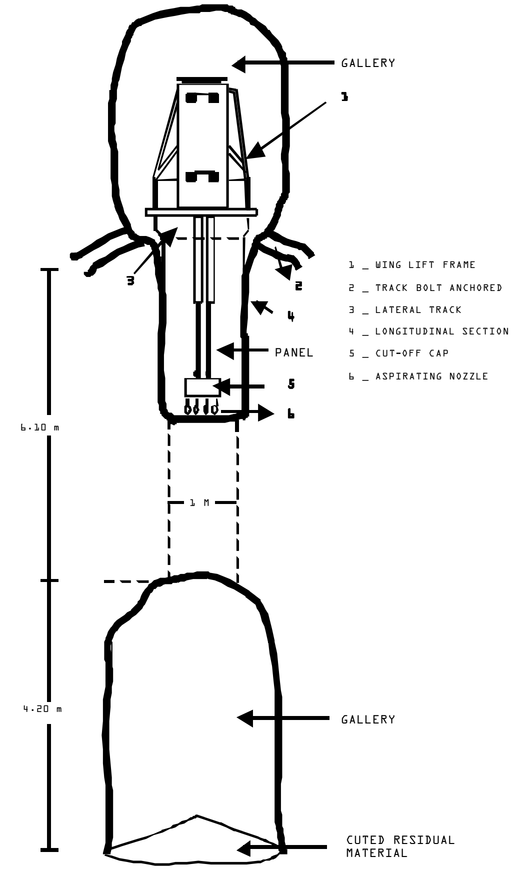

Fig. 4 La Honda dam: High pressure water jet cut-off procedure (cross section)

This technique was very efficient, reducing the specified waiting period (48 hours) between the pouring of the concrete in one gallery and the excavation of the next, thus avoiding damage by blasting vibrations. The mucking rate was also increased by removing the disintegrated and suspended material hydraulically. Over-excavations were also reduced to a minimum, and due to the very minor disturbance of the rock mass no permanent support was necessary.

The average working pressure of the water-jets was about 1000 bars (100 MPa), with an average flux of water of 80 liters/min. The machine was mounted in two platforms that moved on rails (Fig. **3**, Fig. 4 and Fig. **5**). Where the rock was homogeneous the method proved to be very effective, but in the sectors with alternate strata of sandstone, shale and siltstone, the setting of the machine was very difficult. Those zones were excavated by the drill and blast method.

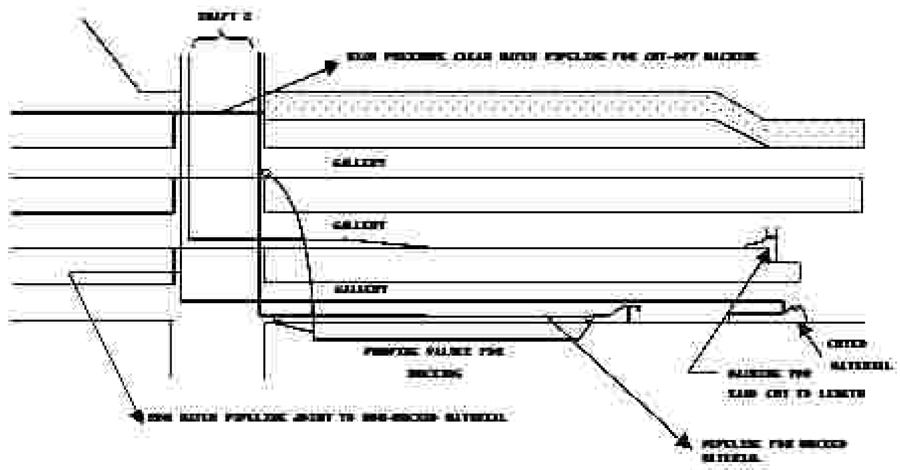

Fig. 5 La Honda dam: High pressure water jet cut-off procedure

All level galleries were excavated with a controlled drill and blast method from the surface. At the panels the blasting was executed in two stages, to avoid exceeding the allowable parameters of intensity, i.e. mainly the particle velocity. The mucking was performed using a rail system to reach a crane, where the material was lifted out.

Table 1shows a summary of the executed project quantities, during a 24-month period:

Table 1 Summary of executed project quantities

Total area excavated (not including the shafts)	22,172 m^2
Level Galleries	2,996 m
Shaft Excavation	8,782 m^3
Excavation of panels with drill and blast	14,440 m^3
Excavation of panels with water jets	2,507 m^3
Concrete poured	43,189 m^3

Even though the technique of excavating with water jets for underground structures is limited to friable, low strength, and relatively homogeneous rock masses, the experience at La Honda Dam to build the superposed galleries proved to be effective; guarantying the verticality of the excavation, reducing over-excavation, and reducing rock disturbance.

A series of galleries, providing access for further treatment and monitoring of the protection system was devised. In this way pore pressures and leakages within the abutment could be supervised.

In general terms, the foundation treatment has performed well; piezometric pressure below the downstream shell within the valley corresponds to about tailwater. Nevertheless, a concentrated leak of about 50 to 70 liters/s, causing the removal of about 60 kg/day of suspended solids, had developed. Apparently, it originated upstream of the abutment blanket, following a system of vertical joints oriented parallel to the river (Kleiner, 1991). However, additional grouting and drainage permitted to control this situation.

A4.2 Sogamoso Dam (Colombia)

A4.2.1 Location and general description

The Sogamoso Hydroelectric Project is located in the northeast of Colombia in the Province of Santander. The concrete-face gravel dam (**Fig. 6**), 190 m high, able to produce 1035 MW, is located in a narrow canyon, where the Sogamoso River cuts through the Serranía de La Paz (Marulanda et al., 1999).

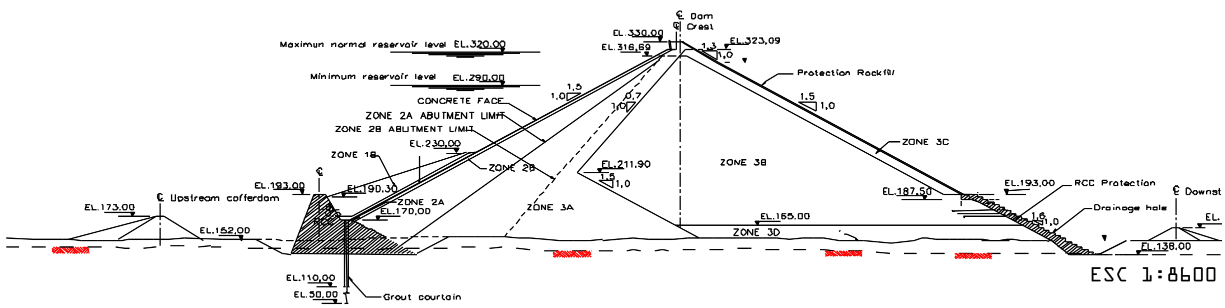

Fig. 6 Sogamoso dam: Maximum dam cross section

A4.2.2 Geology

The geologic characteristics of the dam foundation area were established by a detailed surface geologic reconnaissance (the rock outcrops in most of the area), correlated with results from the following explorations: Fifty holes with a total length of 3100 m were drilled; Lugeon permeability tests were performed and 39 piezometers were installed to register the water level variation in both abutments. Four exploration galleries, two for each abutment, at different levels with a total length of 400 m, and 27 seismic refraction lines with a total length of 1470 m were executed. In addition, systematic point load tests were performed on the recovered rock samples, and in specific sectors other tests like unconfined compression, indirect tension, shear wave velocities, and direct shear in claystone layers were carried out.

The foundation of the dam consists mainly of sandstones with thin claystone interbeds of the tertiary La Paz Formation. Some quaternary deposits fill the lower part of the canyon with a maximum thickness of about 40 m. The sandstone is partly conglomeratic, medium strong to friable, and highly fractured near the surface. Claystone interbeds in the form of lenses, frequently slickensided, and carbonaceous bands up to 0.3 m thick occur.

The result of the geotechnical investigations lead to a rock mass classification for the plinth foundation area, distinguishing five zones based on geomechanical characteristics. (**Fig. 7** and **Table 2**). Rock mass quality notably improves with depth. Good conditions were found below 60 m.

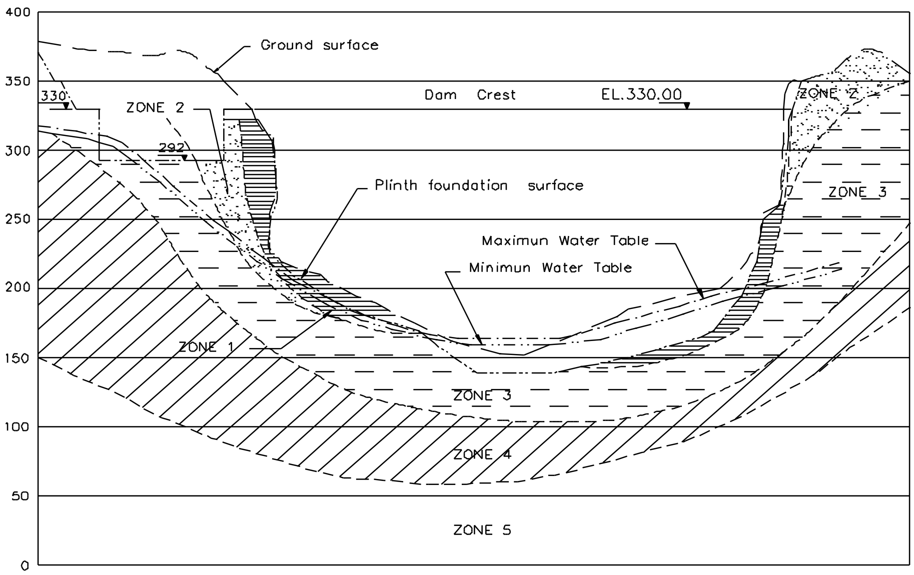

Fig. 7 Sogamoso dam: Geotechnical zones defined based upon field and laboratory investigation

Table 2 Geomechanical properties of geological zones

Zone	Thickness (m)	RQD (%)	Approx. point load strength index (MPa)	Permeability (UL)	RMR Index	Deformation modulus (MPa)
1	15	0 - 30	0.7	30 - 60	15	100 - 300
2	40 - 60	30 - 80	1.0	30 - 60	30	300 - 1,100
3	60 - 120	80 - 90	1.6	10 - 30	45	1100 - 3500
4	20 - 150	90 - 95	1.8	2 - 10	50	3500 - 3700
5	- - -	95 - 100	2.0	0 - 5	60	3700 - 4000

A4.2.3 Foundation Treatment

Competent rock is found at depths below 50 m, with joints principally closed and when open, adequate for grouting. The same situation is observed under the cofferdam, in the lower part of the canyon. However, the high fracture intensity, weathering, and relaxation joints at both of the abutments (the upper 50 m above the cofferdam crest) make conventional grouting treatment not reliable or effective to avoid erosion of the clay-silt fillings found in the open joints, or in the weathered and weakly cemented sandstone mass along the main joints. In both materials grout penetration is very low, making them susceptible to erosion under the high hydraulic gradients that are expected at the plinth.

For this reason, a continuous concrete cutoff wall, 50 m deep, in both abutments was adopted as the deep foundation treatment, complemented with grout curtains only in the foundation sectors where they could be considered effective (Fig. 8).

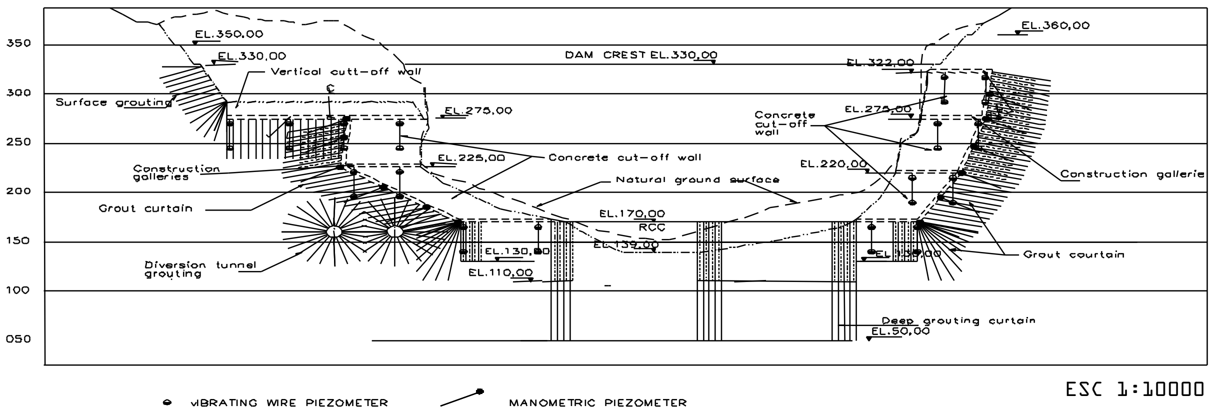

Fig. 8 Sogamoso dam: Foundation treatment: galleries and grout curtain

The cutoff wall is to be developed by concrete walls between horizontal galleries excavated at several levels (multi-level or superposed galleries). Because of the nature of the rock mass and the proximity to the steep canyon, the excavation of the galleries by road header and the cutoff by raise bore equipment has been considered appropriate. The specifications for the project allow the contractor to propose the arrangement of the galleries. Either they can be excavated in the plane of the concrete face (Fig. 9), or from two galleries at the ends, connected by vertical shafts drilled with raise borer (Fig. 10).

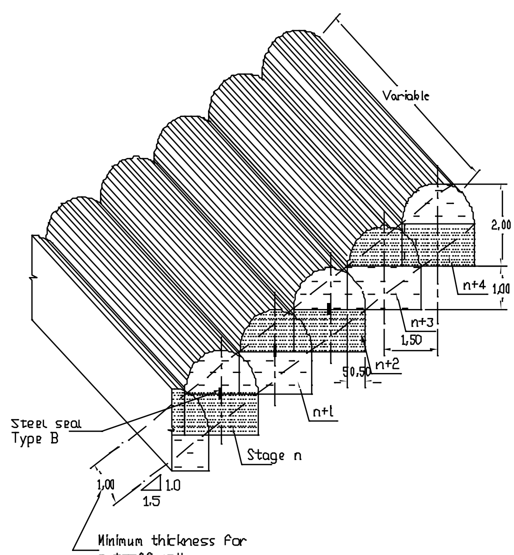

Fig. 9 Sogamoso dam: Construction scheme of cutoff wall and superposed concreted galleries

For the raise-borer alternative the concrete wall should be built from four galleries located at the right abutment at elevations 322, 275, 220 and 170 and three galleries at the left abutment located at elevations 275, 226 and 170. Those galleries are located in Zone 3, near the contact with Zone 4. The concrete wall at the left abutment above elevation 275 will be built from the spillway channel at the spillway intake. The rock mass below elevation 330, between the spillway channel and the dam face, will have a special concrete treatment for protection.

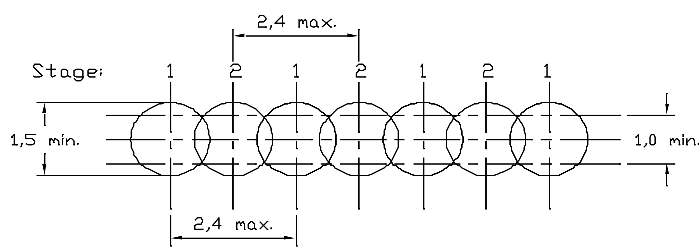

Fig. 10 Sogamoso dam: Construction sequence for the excavation of vertical shafts with raise-borer

As an additional protection measure, the design includes a filter layer placed along both abutments, downstream of the plinth (Fig. 11) in order to avoid progressive erosion of the foundation rock by direct flow under the plinth that could develop as a result of a construction defect.

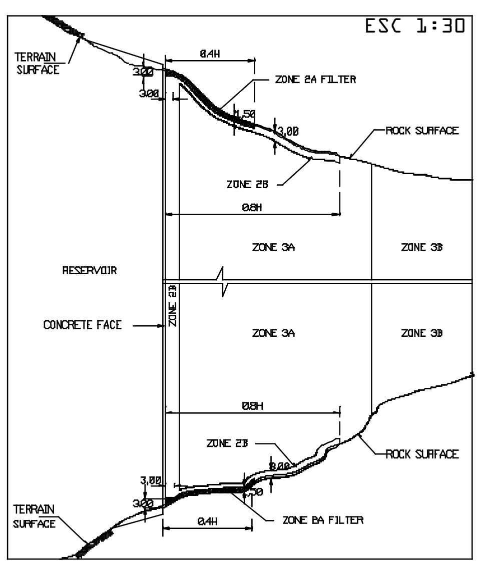

Fig. 11 Sogamoso dam: Additional foundation protection by filter layers

A4.3 Salve Faccha Dam (Ecuador)

A4.3.1 Location and general description

The Salve Faccha Dam is part of the Papallacta water conveyance project for the city of Quito (Ecuador's capital city). It is located about 60 km east of Quito, at 3894 m asl in the eastern range of the Andes, on the Rio Cunuyaco. The rockfill dam with impervious central core (composed of moraine material), and founded on volcanic rocks was constructed by 1999. The dam features a height of 45 m and a crest length of 210 m. Project details were presented by Marulanda et al. (2006).

A4.3.2 Geology

Bedrock at the dam site is of volcanic origin, covered by colluvium and alluvial deposits. The site is located within a crater with steep slopes, shaped by slides in the upper reaches and covered by more recent lava and pyroclastic material. Glacial activity has eroded the crater and opened it to the West, allowing the drainage of the waters of the Cunuyaco Riverto the Amazon Basin. At the dam axis the site is asymmetric. The alluvial channel of the Rio Cunuyaco runs along the right flank, a flat area forms the center and the left part of the valley above which a rock slope rises. Along most part of the dam axis and at greater depth, highly silicified dacite with good mechanical properties is found. At the surface these rocks are weathered.

On the right abutment, a rather special geological feature was observed. An approximately 20 m thick andesitic-basaltic formation of fair geomechanical characteristics covers the higher part of the abutment, followed by two different formations: one, 10 m thick, consisting of dense sand and the other, a moraine, approximately 8 m thick (Fig. 12).

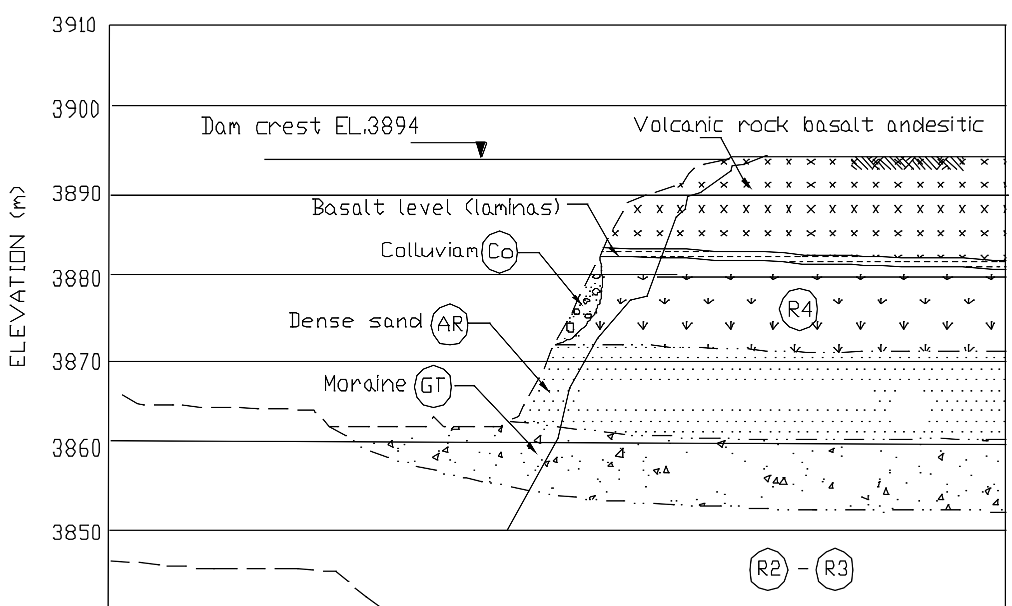

Fig. 12 Salve Faccha dam: Right abutment of dam (longitudinal section) as predicted on the basis of exploration

A4.3.3 Foundation treatment

The foundation treatment foresees a mined diaphragm wall at the right abutment to assure the watertightness and internal stability of the dense sand and moraine, and conventional grouting along the core foundation axis, to reduce water loss through the rock mass under the dam body.

The treatment for the right abutment consists in the construction of a wall formed by hand-dug, semi-circular galleries, two meters in diameter, filled with concrete, beginning at elevation 3860 m asl (one meter below the contact between the moraine with the dense sand) and progressing upwards. The average penetration of the wall into the abutment is 15 m, to hold the seepage gradient within this unit below 0.5. The wall is extended approximately to elevation 3874 m asl, embedding it in the andesitic-basaltic rock (also one meter above the corresponding contact), as illustrated in **Fig. 13** and **Fig. 14**.

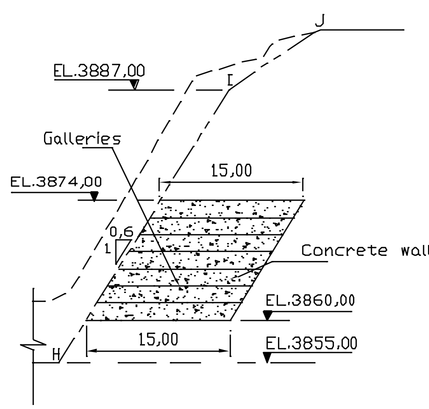

Fig. 13 Salve Faccha dam: Galleries (longitudinal section)

In places where the sand is poorly cemented or water is found, temporary support is required. The support consists of steel ribs type S6x12.5, and corrugated steel sheets (lagging), $^{1}/_{8}$ inch (~3mm) thick. The lower section of the support (the vertical part) is embedded inside the concrete that will form the wall, but the curved section is removed as excavation proceeds and is re-used in the gallery immediately above. Once a gallery is excavated, it has to be filled as soon as possible with pumped concrete (~ 20 MPa compressive strength) to a height of one meter. Only then, excavation of the next gallery is carried out. Construction joints, properly treated (including PVC waterstops) will be provided between the horizontal layers. The final finish of the concrete at the surface of the slope has to be smooth, so that the overall surface is appropriate for the core foundation (**Fig. 14**).

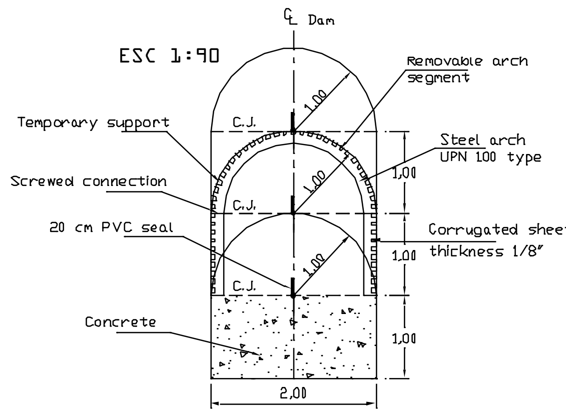

Fig. 14 Salva Faccha dam: Galleries (cross section)

Complementary foundation treatment: Since the dense sand and the moraine layers stretch over the entire width of the dam foundation, there is a possibility of a discontinuity or a crack in the dense materials. This situation was observed at the right abutment close to the dam axis, which presents moderate water flow. Due to the nature of these materials, it is clear, that they cannot be conventionally grouted. To avoid unacceptable erosion in the foundation, the following measures have been proposed to protect the sand level of the right abutment:

- Blanketing of this material at the surface with the same moraine material used for most parts of the contact zone of the core with the downstream shell. This inhibits the migration of this material into the voids of the rockfill. A filter additionally protects this layer. The objective of the reinforcement is to lengthen the path of any possible water flow from the reservoir along the stratification planes, to further reducing the hydraulic gradient.

- Protect the same sandy material downstream of the core with a filter that retains fine particles form the foundation that could potentially be eroded and transported by any concentrated flow of water. This protection also requires a transition within the rockfill (**Fig. 15**).

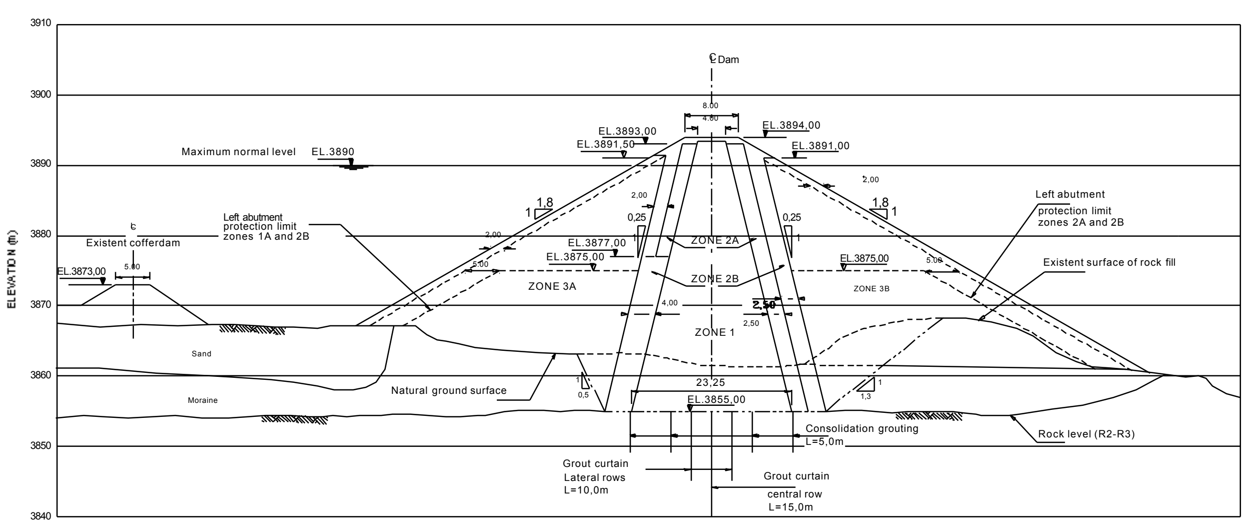

Fig. 15 Salve Faccha dam: Foundation treatment: grouting and filters

During the construction stage, after finishing the excavation for the right abutment, it was found out that within the pyroclastic deposit (sands), there are layers dipping about 40° into the abutment. This condition differs from the horizontal stratification expected according to the explorations performed during the design studies. Consequently, the location and extension of the wall had to be adjusted, maintaining the criteria presented before: (1) an embedment of one meter in the upper and lower levels, and (2) 15m lateral penetration in the pyroclastic level. Fig. 16shows that the lower gallery has to be built at elevation 3853 approximately instead of 3860. This means that an important part of the wall has to be constructed in the moraine.

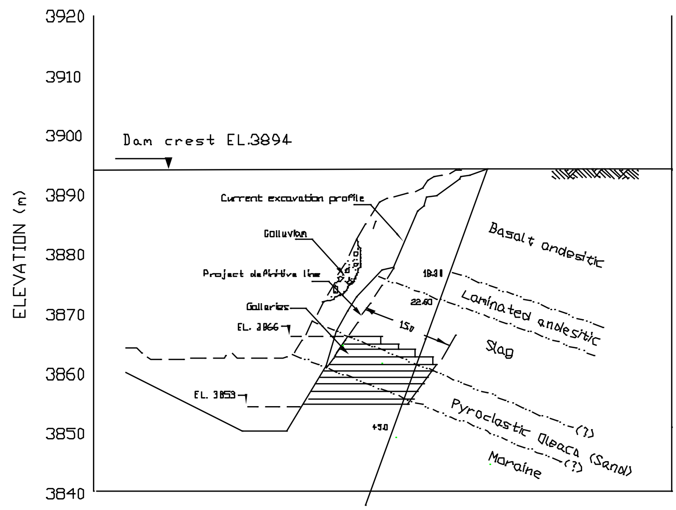

Fig. 16 Salve Faccha dam: Final configuration of the superposed galleries

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Appendix 5. CASE HISTORIES ON JET GROUTING CUTOFF WALLS

A5.1 SM-3 cofferdam (Canada)

A5.1.1 Project description

The Sainte–Marguerite hydro-electric project is located about 90 km northwest of Sept-Îles in the north-eastern part of the province of Quebec, Canada. It includes a 171 m high earth-rockfill dam with a central core of glacial till. For the diversion of the river a 978 m long diversion tunnel had to be constructed and an upstream cofferdam of about 20 m in height. While the core, upstream and downstream filters, and the transition zones of the main dam are founded on rock, the cofferdam is placed on the river alluvium. Sealing of the deep canyon filled with alluvial materials was by means of a jet-grouted cutoff wall, which is the subject of this case history. More details on this project were given by Hammamji et al (1999) and Rattue et al, (1999).

The cofferdam consists of two closure groins of dumped rockfill. The space between the groins is filled with dumped sandy and gravely materials building up a platform from which the cutoff wall could be constructed. After completion of the cutoff, an approximately 20 m high embankment consisting of compacted sand and gravel was constructed with slopes of 1V:2H. A double layer of geo-composite (bentonite between two layers of geotextile) was placed on the upstream slope and acted as the impervious barrier (see Fig. 1).

A5.1.2 Foundation conditions

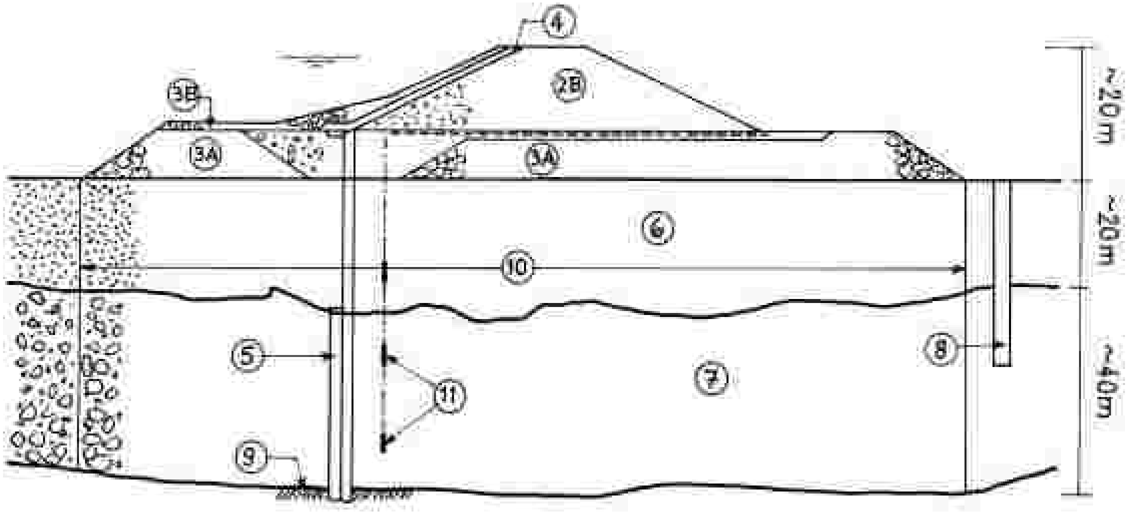
The site for the main dam is located in a narrow, asymmetric and steep-sided valley within a section of rapids. Bedrock outcrops exist mainly at the top of the left abutment and sporadically on the right abutment. The rocks in the region are predominantly gneisses with local intrusions of granite, anorthosite and pegmatite. Bedrock at the SM-3 project site has been identified as an anorthosite batholith.

At the site of the upstream cofferdam, which is upstream of the head of rapids, geophysical investigations and exploratory drilling traced a buried valley, i.e. a 60 m deep and 120 m wide V-shaped canyon filled with about 40 m of coarse alluvial deposits in the lower part and about 20 m of fine to medium sand, occasionally with embedded large blocks, in the upper part (see Fig. 1 and Fig. 2). The bedrock elevation dips towards upstream and at the deepest point below the cofferdam it was about 20 m lower than at the axis of the main dam.

The valley slopes are covered with a substantial amount of scree and talus deposits and large blocks were present throughout the site. An accumulation of large block also existed on the left abutment, at the cofferdam axis, both above and below the river level.

A5.1.3 Foundation treatment for upstream cofferdam

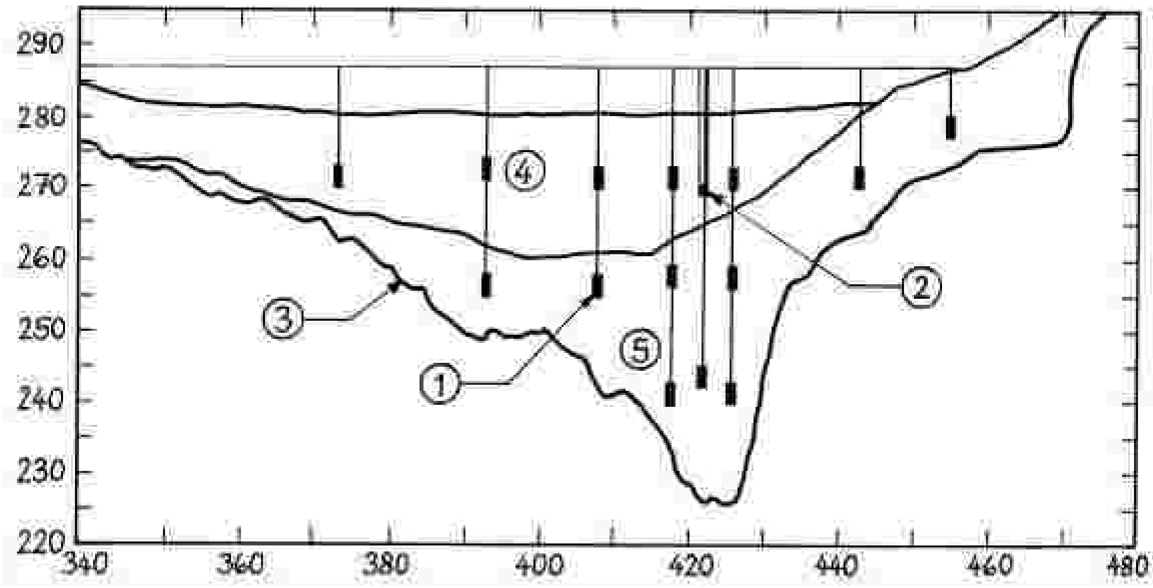
For the treatment of the deep alluvial deposits below the cofferdam several treatment methods were evaluated. Normalized SPT blow counts, $N_{1(60)}$, in the sand layer were as low as 2 to 8 and raised concern for static or dynamic liquefaction. Hence, not only sealing for seepage control but also densification of the alluvium was required. Densification by explosives was selected as the preferred method, as proposed by the contractor, and for the material dumped between the groins dynamic compaction was applied. About 250,000 m^3 of foundation materials were densified by explosives during winter time under an ice bridge prior to the placement of fill for the embankment. Another 71,000 m^3 were densified by dynamic compaction in the space between the groins and at the left and right abutments. This densification work not only increased the liquefaction resistance of the materials but also would reduce deformations of the cofferdam during construction when the water table would be drawn down for dewatering of the construction pit of the main dam.



2A	Sand and gravel dumped and densified	6	Fine to medium sand
2B	Compacted sand and gravel	7	Coarse alluvium
3A	Dumped rockfill	8	Pumping well
3B	Selected rockfill, max. 1 m	9	Bedrock
4	Bentonite geocomposit	10	Limits of densified zone
5	Jet-grouted cutoff wall	11	Piezometers

Fig. 1 Typical section of upstream cofferdam

For the control of seepage various methods were contemplated. The sole use of pumps for de-watering was considered unfeasible because of the high permeability of the sand layer ($k=3x10^{-4}$ m/s) and the underlying coarse alluvium. An infiltration rate of 833 liters/s was estimated. The presence of numerous large blocks in the alluvium precluded the use of sheet piles or any type of positive cutoff. Jet grouting was judged to be the least sensitive treatment for a material with large blocks. Moreover, this method was also competitive to other procedures with respect to costs. Jet grouting was also feasible during the winter season.



1 Electrical piezometer
2 Open standpipe piezometer
3 Profile of bedrock
4 Medium to fine sand
5 Coarse alluvium

Fig. 2 Subsurface conditions and locations pf piezometers

A5.1.4 Cutoff construction

Technical specifications

The following requirements for construction of the cutoff by jet grouting were specified:

- Use of a triple fluid system
- Pumps capable of maintaining a grouting pressure of at least 50 MPa and a grout flow of 150 liters/min.
- Overlapping of two adjacent columns to achieve a continuous wall with a thickness of at least 0.8 m in a single row and with a center to center spacing of the columns of 0.8 m.
- Verticality with a deviation not exceeding 0.5 % in each hole drilled for jet grouting
- Penetration onto bedrock for at least 0.5 m
- Compressive strength of the jetted grout (soilcrete): 500-700 kPa after 7 days and 1500 to 2200 kPa after 28 days, as determined from consolidated undrained triaxial compression tests with a confining (cell) pressure of 120 kPa.
- Permeability under a gradient of 30: $k < 10^{-8}$ m/s
- A complete jet grouting unit must be kept on site for up to six month s after completion and provisional acceptance of the cutoff construction work, for the case corrective action is needed.

In addition, three concepts were introduced, namely:

- Minimizing the hydraulic gradient across cutoff during construction: This was achieved by constructing a temporary dike at the head of the rapids and keep the water level downstream of the cofferdam at about the same level as on the upstream side. To maintain the pool level on the downstream side, water was pumped from upstream across the working platform. Reducing the hydraulic gradient helps to minimize the loss of grout prior to its setting.
- Construction of a test cell and a full-scale pumping test: In order to evaluate the feasibility and efficiency of a jet-grouted cutoff, a 5 m internal diameter test cell was constructed prior to the start of the actual wall installation.
- Second row of jet–grouted columns upstream of the first row with a 1 m penetration into the sand layer to reduce the possible occurrence of windows in the jet-grouted barrier. This secondary row was limited to the central part of the valley and the left abutment.
- Deep pumping wells downstream of the cutoff to (1) reduce pore water pressures in the case of deficiencies (leakage) in the wall and (2) to assist in the dewatering of the excavation pit for the main dam. Altogether six wells were specified downstream of the cofferdam. From calculations, it was estimated that the flow through the cutoff could amount to about 1 liter/s, assuming a permeability of the wall of $k = 10^{-8}$ m/s. The specified capacity of the six wells was 100 liters/s.

Construction sequence

(1) Construction of two trial columns in the deep part of the valley with the purpose to calibrate the equipment

(2) Construction of some columns for the test cell. These columns showed that the setting time for the grout was very long and that grouting next to an already existing column would take longer than anticipated.

(3) Construction of another four test columns, designated as T1 to T4, close to the right bank. These columns were examined to a depth of 2 m and the diameter recorded. The mix proportions and grouting parameters used are listed in Table 1.

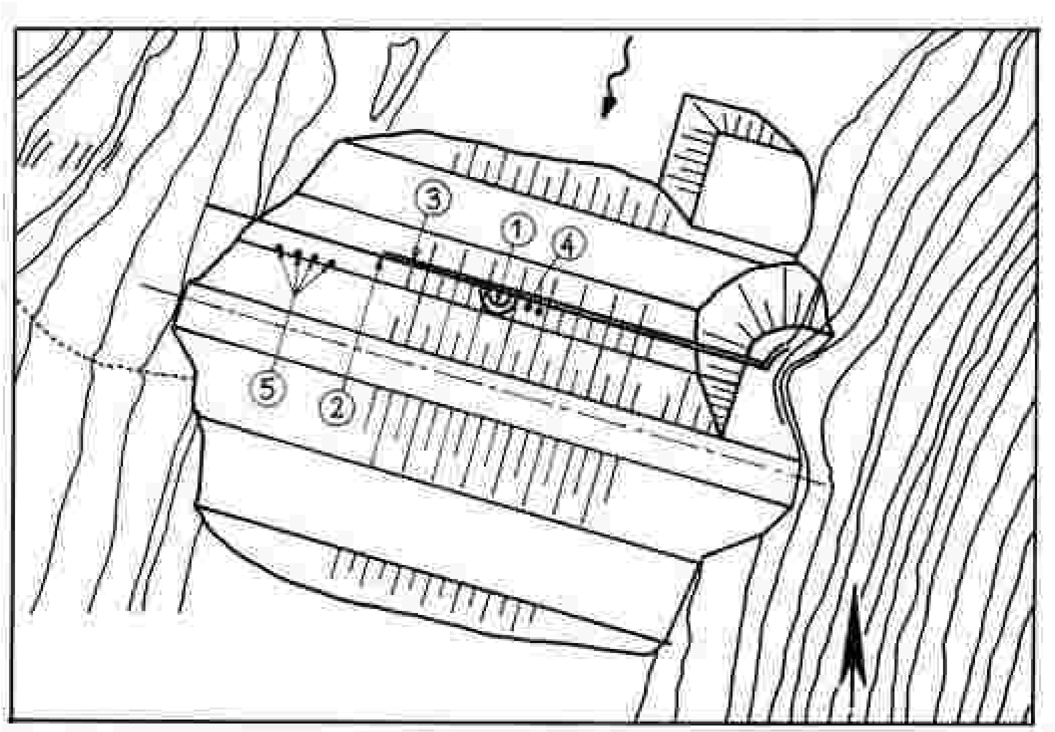
The locations of the test cell, the calibration and test columns are shown in Fig. 3. Also indicated in Fig. 3 are the positions of the axes of the primary and secondary rows of the cutoff wall.

Based on the results obtained from the trial columns, the following amendments were made to the construction program:

- Increase of column spacing from 0.8 m to 1.07 m (see Fig. 4), assuming a minimum diameter of 1.5 m at a lifting speed of 13 to 30 s per 40 mm lift (11 to 4.8 m/hr) and 4 to 5 revolutions per minute.
- The grout mix adopted was with a cement/water ratio (c/w) of 0.6 and a bentonite/water ratio (b/w) of 0.03 (by weight) with the addition of 1.5 % (by weight of cement) of calcium chloride ($CaCl_2$) to accelerate the time of setting of the grout.

Table 1 Characteristics of test columns T1 to T4

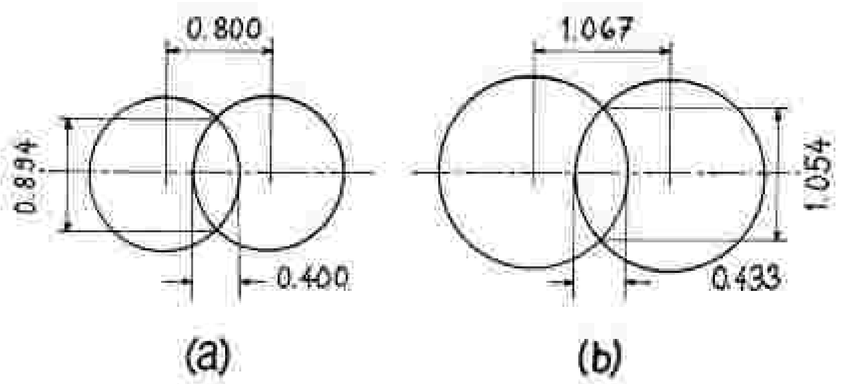
Column no.	Soil type	Grout mix proportions		Lifting speed (m/hr)	Rotational speed (mm/min)	Diameter of column obtained (m)
		c/w	b/w			
T1	2B	0.6	0.03	8	133	2.2
T2	(compacted	0.6	0.,03	6	100	1.8-2.0
T3	sand and	0.8	0.03	4.8	80	2.4
T4	gravel)	0.8	0.03	10.2	133	2.0



1 Test cell
2 Axis of primary wall
3 Axis of secondary wall
4 Calibration columns
5 Test columns (T1 – T4)

Fig. 3 Layout of cofferdam and locations of test cell and trial columns

After completion of a part of the main and secondary walls, selected columns were cored and samples tested in the laboratory for strength and permeability. Permeabilities were marginally higher and strength values somewhat lower than specified. For the remaining columns a higher c/w ratio, i.e. of 1.0, was then adopted.



(a) Phase 1: Column spacing: 0.80 m; Diameter: 1.20 m
(b) Phase 2: Column spacing: 1.067 m; Diameter: 1.50

Fig. 4 Spacing of jet-grouted columns

A5.1.5 Quality control

A number of investigations, observations and analyses provided clues on the possible quality of the jet-grouted columns:

- Pre-drilling of the grout holes provided information on the soil stratigraphy along the axis of the cutoff, based on the return of cuttings and laboratory tests on samples. The verticality of the drillholes was tested by means of an inclinometer in a plastic casing inserted into the drill rod.
- During the jet grouting process information can be obtained from visual inspection of (1) return cuttings, (2) samples of grout before and during jetting, and (3) from automatic readout records provided by the contractor, such as: lifting speed, rotational speed, grout, water and air pressures, and flow quantities.
- Laboratory investigations can be carried out on cuttings from pre-drilling (e.g. grain size), properties of the grout at the mixing plant, return cuttings during jetting, and compression and permeability tests on hardened specimens of grout.
- Tests on cored samples from completed columns of different age: For this purpose the columns of the test cell and selected columns in the cutoff itself were selected.

It turned out that the specified verticality was difficult to achieve. The deviations from the vertical were closer to 1.5 % than to 0.5 %. However, with the larger columns of 1.5 m diameter it was still possible to meet the required wall thickness.

Difficulties also arose with drilling and grouting in the deeper parts of the alluvium when the normal procedure was used. Several columns had to be constructed in two steps as follows: (1) pre-treatment of the alluvium using a high lifting speed to consolidate the column and (2) re-drilling and jet-grouting of the pre-treated column.

The greatest depth reached in the cutoff was 65 m, which at that time was a world record for jet grouting. The total area of the cutoff wall (including primary, secondary columns and test cell), was 6785 m^2. The time for construction, including pre-drilling, all testing and some work stoppage due to adverse conditions in winter, was 7.5 months.

A5.1.6 Performance

Theoretical calculations using the specified values of $k_{wall} = 10^{-8}$ m/s and a wall thickness of 0.8 m predicted a flow through the wall of about 1 liter/s. The flow through the cutoff wall was monitored by 12 electrical piezometers inserted into boreholes at a distance of 5 m downstream of the wall. Pumping tests in the test cell yielded a permeability of $k = 5 \times 10^{-7}$ m/s. Assuming a permeability of $k = 5 \times 10^{-7}$ m/s for the primary wall and 10^{-8} m/s for the secondary wall produced a flow of 14 liters/s. The actual flow was less than 17 liters/s and de-watering of the foundation pit of the main dam was without any problems.

References

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A5.2 Ertan cofferdams (China)

A5.2.1 Project description

The 245 m high double curved Ertan arch dam, located in a steep-sided V-shaped valley on the Yalong River, a tributary to the Jiangtze River, was completed in 1998. For the diversion of the river with a construction design flood of 13,500 m^3/s, high cofferdams were required, namely 60 m upstream and 30 m downstream. While the main dam and the plunge pool were founded on rock, the cofferdams were placed on thick alluvium requiring sealing by a suitable cutoff. A project layout is shown in Fig. 1. The case history of the Ertan cofferdam cutoffs has been described very comprehensively by Sembenelli & Sembenelli (1999). Additional information can be obtained from Chen & Lin (2000).

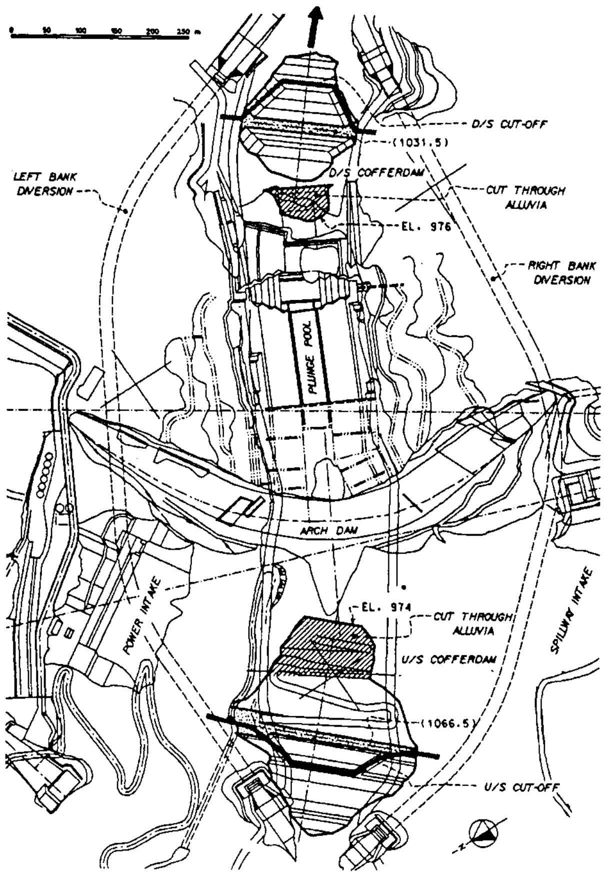

Fig. 1 General lay-out of Ertan project showing position of arch dam and the cofferdams. Hatched surfaces correspond to slopes cut in river alluvium to expose bedrock

A5.2.2 Foundation conditions

The arch dam is founded on an intrusion of igneous rock surrounded by conglomerates and limestone. Overlying the foundation rock are alluvial deposits, 40 to 50 m thick, which can be grouped into the following material types: (1) gap-graded sand and gravel; (2) coarse angular boulder, up to 0.5 m^3 in size, in a sandy matrix (3) uniform, very fine, low plastic sandy silt, and (4) a thin layer of gravelly alluvium covering the rock of the former river bed (only exposed at downstream cofferdam). Details of the foundation conditions can be seen in the valley cross- sections of Fig. 2and Fig. 4 and cross sections of the cofferedams are shown in Fig. 3 and Fig. 5.

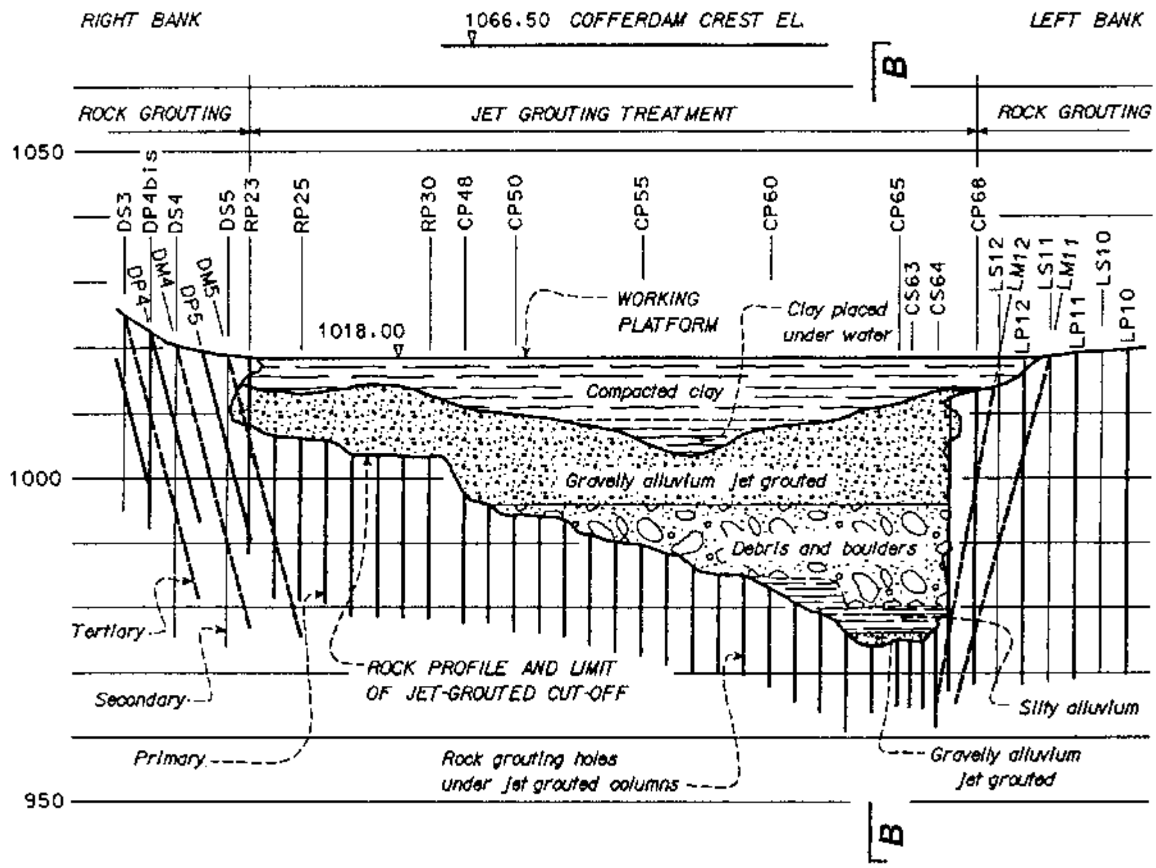

Fig. 2 Cross section A-A of valley at the site of the upstream cofferdam (see Fig. 3 for location), showing rock line, extent of jet grouting treatment, and rock treatment by conventional grouting

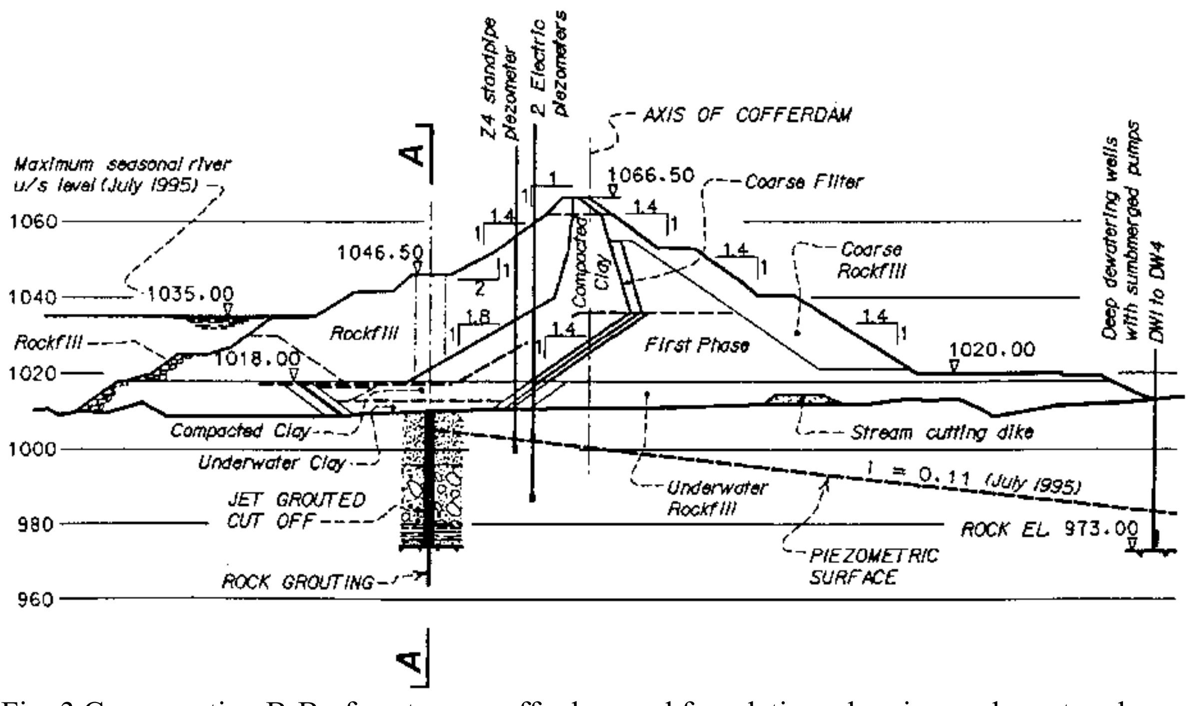

Fig. 3 Cross-section B-B of upstream cofferdam and foundation, showing underwater clay zone, connection of dam core to jet-grouted cutoff and stream cutting rockfill dyke.

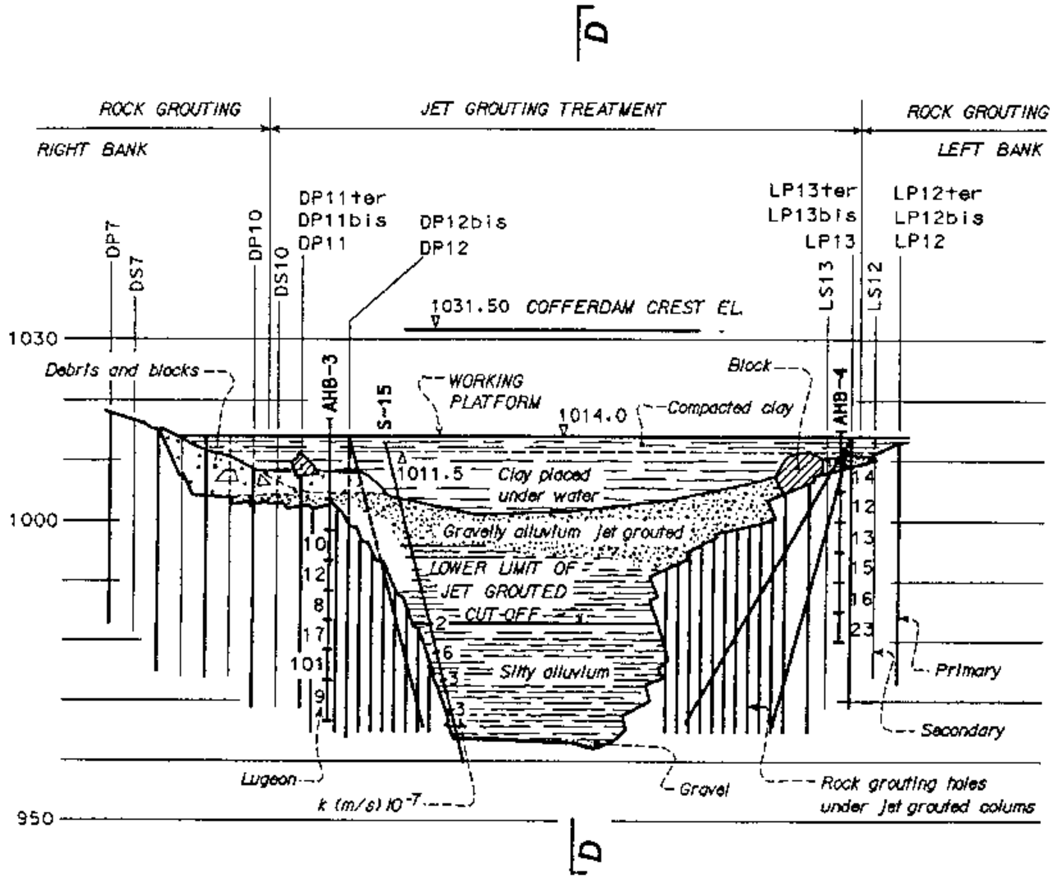

Fig. 4 Cross section C-C of valley at the site of the downstream cofferdam (see Fig. 5 for location), showing rock line, extent of jet grouting treatment, and rock treatment by conventional grouting

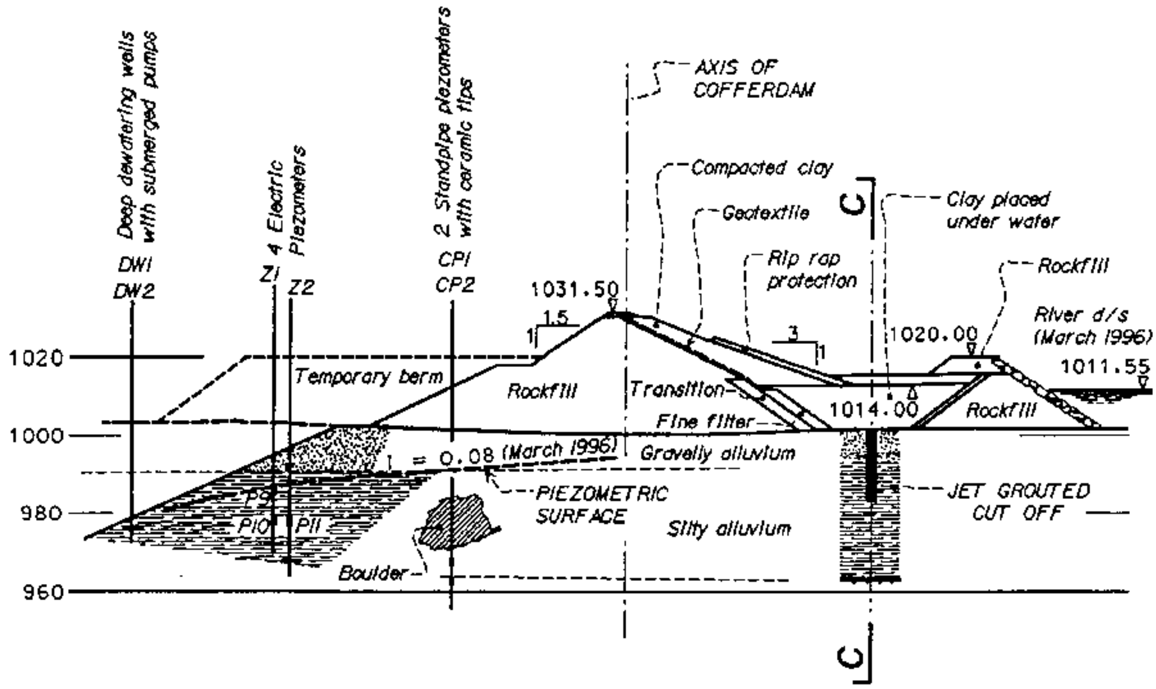

Fig. 5 Cross-section D-D of downstream cofferdam and foundation, showing underwater clay zone and connection of dam core to jet-grouted partial cutoff

A5.2.3 Cutoff design

Design criteria for a cutoff were the following: (1) the cutoff must be able to sustain a permanent water head of 50 m and up to 100 m during floods; (2) allow rapid construction to enable completion within a very limited time frame; (3) be applicable to the alluvial materials encountered in the foundation; (4) be able to reach a depth of at least 50 m, and (5) be compatible with a moderate (~10 m) differential head across the cutoff while being constructed.

For the selection of a watertight barrier, positive cutoffs were eliminated because the presence of large boulders was considered a major unknown in the completion time. Grouted cutoffs were also judged unsuitable regarding the presence of a thick silt stratum in the foundation. Hence, a semi-positive type of cutoff, such as jet grouting, appeared to be the most versatile method to handle the presence of large boulders and to effectively treat fine-grained soils. It was decided to use the double-fluid technique (grout and air) as it promised the construction of columns of a substantial diameter. Because of the tendency of the columns to separate with depth, a cutoff consisting of three rows was chosen. The spacing of the rows was 1.5 m and the column spacing along each row was 1.0 m. This pattern resulted in a 3 m wide jet-grouted curtain.

For the upstream cofferdam, a fully penetrating cutoff, up to 46 m deep, was adopted reaching down to the rock (see Fig. 3). The bedrock itself was treated by a grout curtain to an additional depth of 10 to 25 m. The connection of the cutoff to the clay core of the cofferdam was accomplished by means of a clay plug, which had to be constructed by dumping the clay initially into 10 to 12 m deep still water. After having filled the underwater clay, a conventional compacted clay fill was placed to elevation 1018, which created a convenient working platform for cutoff construction (see Fig. 2). A similar procedure was used for the downstream cofferdam. However the foundation materials there were different and the relatively thick layer of silt was used as a natural blanket and a jet-grouted partial cutoff ending in the silt layer at elevation 983 was adopted (see Fig. 4). Rock grouting at this site was limited to the abutments to prevent by-pass seepage.

A5.2.4 Cutoff construction

Construction of the cutoff involved the following activities: (1) drilling of exploratory holes; (2) drilling the holes for jet-grouting; (3) replacing the steel casing by a fragmenting plastic casing, and (4) jet-grouting.

A line of exploratory holes, spaced at 6 m, was drilled 1.75 m upstream of the cutoff axis with the aim to fill possible open works by gravity saturation grouting, because there was concern that concentrated seepage flow could wash out the freshly jetted columns. In addition the exploratory holes served as a means to define the rock profile across the river bed.

For drilling the holes used for jet grouting, the ODEX rotopercussion duplex system with an eccentric bit was employed. This system consisted of a down-the-hole hammer, a pilot bit, a reamer and a guiding device to keep the bit in the center of the casing. Drill bit and casing are advanced simultaneously. When the depth of the hole exceeded about 20 to 25 m, telescoping casing was installed. The diameters of the ODEX drilling systems used were 115 or 165 mm for the inside allowance of the casing and 142 or 193 mm for the outside diameter.

Drilling penetrated about 1 m into the bedrock. On the average, 2.4 m large boulders per hole were encountered during drilling the jet-grouting holes ('large' was defined as a size exceeding 0.5 m). The verticality of the holes was checked inside the ODEX casing by an inclinometer. The maximum deviation from the vertical was 1.65%.

Jet-grouting was carried out from the bottom of the hole upwards. When drilling of a hole was completed, a PVC fragmenting (slotted) pipe, 106 or 112 mm in diameter and 3 mm thick, was placed inside the steel casing and the steel casing withdrawn. The purpose of this pipe, which was subsequently broken up by the high pressure jet, was to provide support to the hole before and during treatment and allowing easy insertion of the equipment into the hole. The sequence of jetting was upstream row first, followed by the downstream row second, and finally the center row third. Along each row the split spacing procedure was employed starting with a spacing of 4 m for the primary columns (Fig. 6).

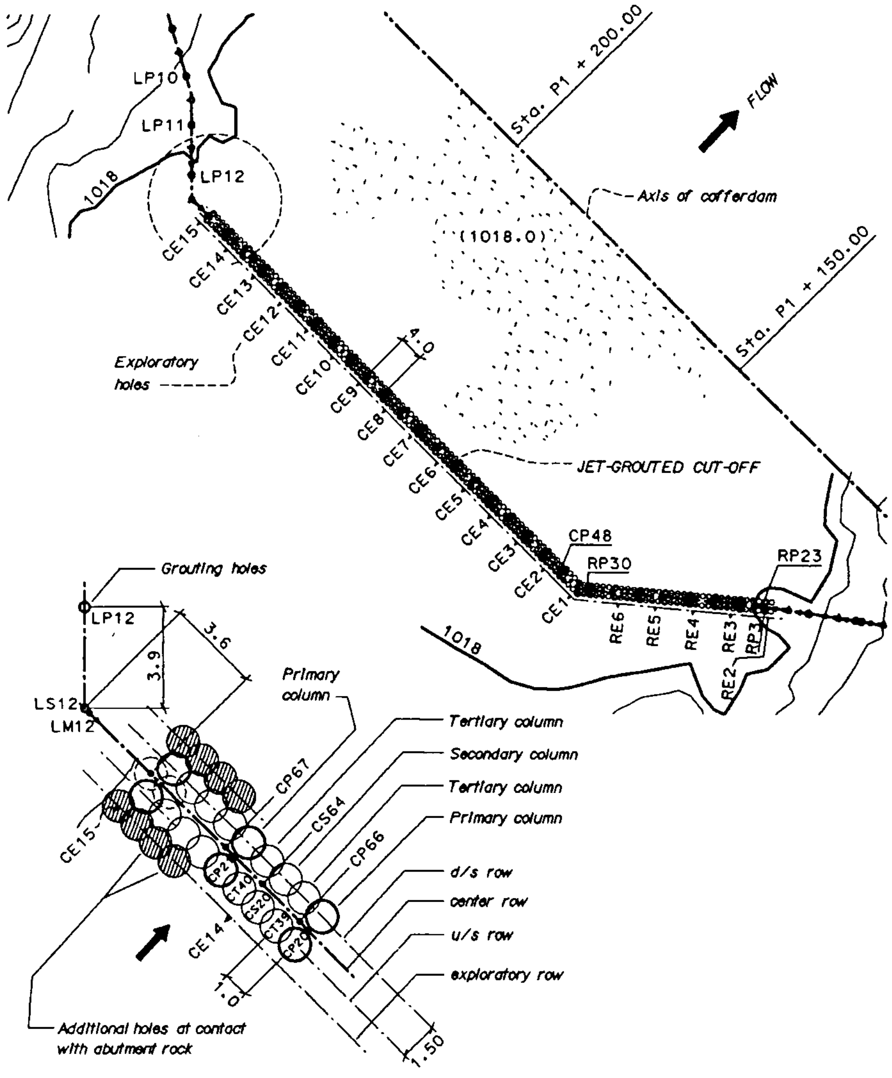

Fig. 6 Lay-out of jet-grouted three-row cutoff for upstream cofferdam. Additional columns at contact with left abutment

Jet-grouting was executed with a Trevijet T1/s double fluid equipment with rods of an outside diameter of about 90 mm. The pumps delivered grout at a rate of 180 to 195 liters/minute at a pressure of 41 to 42 MPa and an air flow rate of 7-10 m^3/min under a pressure of 1 MPa. Head losses in the grout line were estimated on the order of 2 to 3 MPa. The monitor was equipped with two nozzles, set 180° apart which ensured that the grout jet was shrouded by an air jacket thereby reducing the dissipation of energy into lateral turbulence. The air also generates a lifting effect in the annular space between the rods and the casing. The nozzles had an inside diameter of either 3 mm (first row of jet-grouted columns) and 2.8 mm for the two other rows. The rotating speed of the monitor was between 25 and 30 rpm and the lifting speed was in the range of 0.15-0.25 m/min (9-15 m/hr).

Special procedures had to be applied when large overhangs were encountered along the left side abutment of the upstream cofferdam (Fig. 5). Each row was then extended until drilling was proved to be in rock for about the top 30 m. Two additional rows of four columns each were added to increase the length of the cutoff/rock contact.

The grout mix was composed of cement, water and bentonite with the following proportions: water/cement ratio, w/c = 0.85, and bentonite/cement ratio, b/c = 0.6%. It had the following characteristics: a unit mass of 16 kN/m^3, a metric Marsh viscosity between 35-38 s, and a 2-hr bleeding of less than 5%. At the downstream cofferdam, where large shear strains were expected near the contact between the partly penetrating cutoff and the abutment rock, a more plastic mix was used, namely with w/c = 1.12 and b/c = 3%.

The overall areas of the jet-grouted cutoff were 2800 and 2200 m^2 for the upstream and downstream cofferdams respectively. At the downstream cofferdam average production rates with six drill rigs and two jet-grouting units were 165 m/day of cased drill hole and 141 m/day of jet-grouted column, working in two shifts of 11 hours, six days a week.

Grouting of the bedrock below the jet-grouted curtain was accomplished with the following steps: (1) Uncased, non-coring drilling, diameter 150-90 mm, through the jet-grouted columns of the center row at a spacing of 4 m using a down-the-hole hammer. (2) Grouting upstage in 10 m long sections. Along the deepest sections of the bedrock line, the spacing of the grout holes was reduced to 2 m. The abutment rock was grouted on both sides of the cutoff. The grout mix had a w/c ratio of 0.8 and the refusal criterion was based on a limit flow of $Q < 1$ liter/min sustained for 5 minutes.

A5.2.5 Jet-grouting test

Prior to carrying out the actual jet-grouting works, two full-scale jet-grouting tests were performed using the same equipment as foreseen in the production work. The purpose of the tests was to optimize the treatment parameters.

Test 1 consisted of four columns, set 3 m apart and 5 m deep. The location of the test was such that the columns could be excavated later even at high river levels. In this test optimization was focused on the withdrawal rate (or lifting speed) and also on the design of the fragmenting casing. The treatment parameters employed were as follows: rotation speed: 30 rpm; number of nozzles: 2; nozzle diameter: 3 mm; and grout to soil ratio: 0.6. The treatment pressures and rates were: air pressure = 0.7 MPa, air flow rate = 7.5 m^3/min; grout pressure = 40 MPa and grout delivery = 180-190 liters/min. The withdrawal rate was chosen as: 12, 10.2, 15, and 11.4 m/hr for columns 1 to 4 respectively. The grout characteristics were: w/c = 0.83; unit weight = 1.59 t/m3; Marsh viscosity = 37 s, and bleeding after 2 hrs = <2%. After excavation of the columns it was revealed that heavier grouting was needed at all levels where a substantial fine fraction was lacking. It also showed that the columns were quite regular and their diameter always exceeded 1.5 m.

Test 2 involved the jetting of a three-row annular treatment with a bottom plug. The treatment parameters selected were: withdrawal rate = 11.4 m/hr; grout delivery = 186 liters/min; the others were the same as in Test 1. Permeability tests were performed inside the annular shaft before and after treatment. The water losses across the jet-grouted wall of the shaft were practically nil. The test also showed that the presence of large boulders (1) affected the progress of the jet-grouting operations, (2) caused difficulties in extracting the cuttings, and (3) requires large volumes of grout to fill the boulder-related cavities.. Boulders of the order 1 to 3 m were not uncommon. However, with the double fluid equipment used at the Ertan site, it was possible to provide full treatment for smaller boulder (<0.5 m) by reducing the lifting speed near the top and bottom of the boulder and seal the jet-grouted column onto the boulder. When clusters of large boulders were encountered, drilling was stopped and the part above the cluster jet-grouted. Then, after hardening, the column was re-drilled and the hole deepened below the cluster to its final depth. Then the entire column was jet-grouted.

A5.2.6 Quality control

The quality of the completed cutoff was evaluated by means of vertical and inclined check holes, 76 mm in diameter, and drilled with continuous coring. From the inspection of the cores, it was possible to check the continuity of the cutoff at the contacts between columns and between columns and abutment rock. The watertightness was checked by gravity-type water tests, downstage, on sections of different length (0.1 - 9 m). The coefficients of permeability were then calculated as k= $9x10^{-8}$ to $7x10^{-10}$ m/s for the upstream cofferdam and $3x10^{-7}$ to $3x10^{-9}$ for the downstream dam. Drilling and sampling were carried out not earlier than 15 days after completion of the jet grouting work in each of the sites.

A5.2.7 Performance

The performance of the two cutoffs could be assessed during excavation of the riverbed alluvial materials between the two cofferdams. Dewatering wells installed below the cofferdams did not perform satisfactorily and pumping was stopped except for one well at the downstream cofferdam. Seepage into the excavation turned out to be negligible inspite of a permanent gross water head in excess of 60 m and a maximum hydraulic gradient of 0.27 for the upstream cofferdam. Foundation pore water pressures were monitored throughout the excavation and construction period.

At the downstream cofferdam, the seepage passing through and under the cutoff was estimated, based on piezometer readings and a hydraulic gradient of 0.15 - 0.10, as 1.5 liters/s. Actually, it was realized that most of the water flowing into the excavation pit was passing through the rock in the far abutments, where wide open joints had been identified, and then entered the sandy silt below the partial cutoff. This water could then be handled by the installation of ejector wells operated by compressed air. These wells could extract a rate which was much higher than what could flow underneath the cutoff.

A5.2.8 Lessons learnt

This case history demonstrates that jet-grouted cutoffs can be installed to significant depths, i.e. 50 and more meters in layered and widely differentiated river deposits. The presence of large boulders is manageable; however, progress in jet-grouted column construction is slowed down. Multiple rows are an essential feature to ensure watertightness. The use of fragmenting casing proved to be an efficient technique and will gain further application in other large projects. An experienced contractor in foundation treatment is essential to complete such projects successfully.

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A5.3 Thika dam (Kenya)

A5.3.1 Project description

The 65 m high and approximately 500 m long Thika dam on the Thika river is an earthfill dam with a central upstream sloping core located in the foothills of the Aberdane mountains, approximately 50 km north of Nairobi, Kenya (Fig. 1). It is part of a water supply project providing potable water to the city of Nairobi. This case history has been documented by Attewill & Morey (1994) and Harris & Morey (1994).

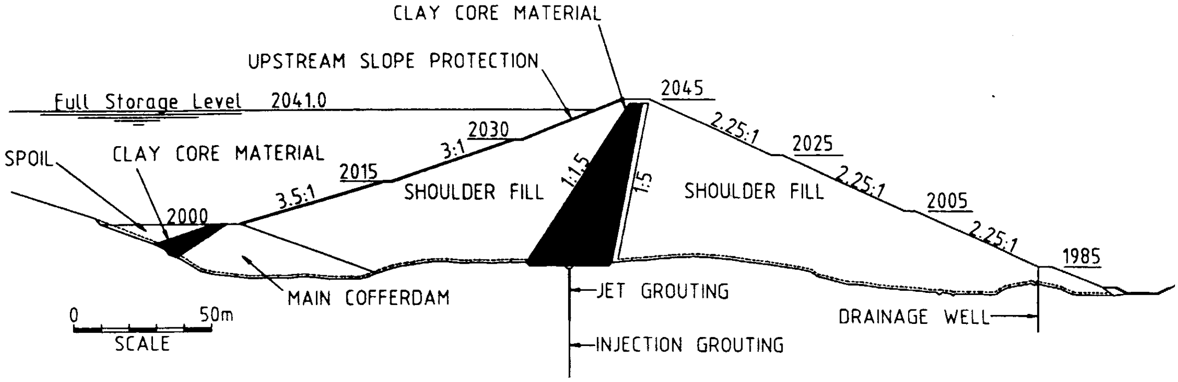

Fig. 1 Cross section of Thika dam showing location of cutoff

A5.3.2 Foundation conditions

The dam site is characterized by rocks of volcanic origin covered by a mantle of highly to completely weathered material and silty clayey residual soils. The volcanic rocks originate from pyroclastic flows which resulted in tuffs of variable grain size and also from flows of phonolite lava. There were several periods of volcanic activity producing a sequence of layers of irregular thickness. The layers are separated by weathered horizons with residual soils which were created during the periods between the flows.

The complex variation of the mechanical properties of the geomaterials was explored by boreholes drilled at 24 m spacing along the axis of the dam. A weathering scale with six grades according to British Standard 5930 was used to classify and interpret material properties in the stratified soil/rock profile.

A5.3.3 Foundation treatment and cutoff design

The depth of weathering that had developed since the last volcanic activity had reached a depth of some tens of meters below the surface of the valley. Because of the greatly differing characteristics along the weathering profile, the concept adopted was that Grade VI materials (residual soil), Grade V materials (completely decomposed rock) and probably also Grade IV materials (highly decomposed rock) would need a different treatment from the less weathered grades (i.e. Grades III, II and I). The top 2 m which contained most of the Grade VI material were stripped. Treatment by grouting is not successful in highly weathered and decomposed rock and a cutoff alternative was envisaged.

The selection of the cutoff was problematic because of the variable strength of the rock in the stratified profile. Another difficulty was the approximately 20 degree steep valley slopes which made access for heavy equipment questionable. Among the alternatives considered for constructing a cutoff were: (1) excavation, (2) diaphragm wall, and (3) jet grouting. Excavation was considered hazardous in view of the stability of the very high slope cuts. The construction of a diaphragm wall requires a large equipment platform and with the machines available at that time there were doubts whether the hard phonolite layers could be excavated without the use of blasting. Hence, the decision was made to employ jet grouting in the residual soils and completely decomposed rock strata and conventional grouting in the other layers (Fig. 2).

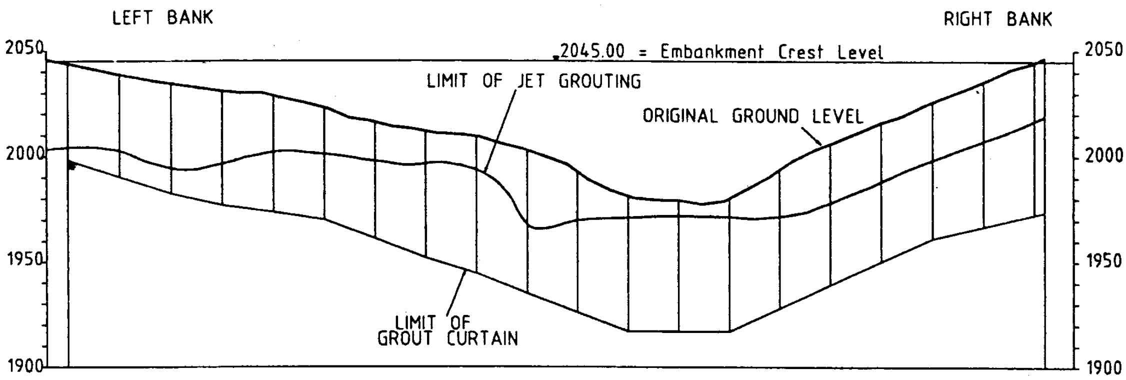

Fig. 2 Longitudinal section of Thika dam showing limits of jet grout curtain and injection grout curtain

The jet grouted cutoff was designed as a curtain of interlocking jet grout columns to a depth of 25 m below the ground surface. The curtain should have a permeability of less than 10^{-9} m/s and a deformation modulus at 1 percent strain in the range of 400 to 700 MPa. For the verticality of the jet-grouted columns a maximum deviation of 1 % of the vertical depth was specified. The actual spacing of the columns had to be determined from a field test since the diameter of the columns depended on the characteristics of the geo-materials to be treated. The aim was to achieve a diameter of at least 1000 mm. The triple fluid method was adopted.

The cement grout mix to be used was determined from laboratory tests with variable proportions of cement, water, bentonite and soil. It was found that the specified characteristics could be achieved with a water/cement ratio of 1:2.3 and with the addition of 10 % bentonite to stabilize the mix. The addition of soil did not have a significant effect on the results.

A5.3.4 Jet grouting test

A trial panel consisting of 16 jet-grouted columns, 20 m deep, was prepared with column spacing and jetting parameters (water pressure, withdrawal speed, rotational speed) being varied. Two problems were encountered, namely:

- The triple fluid method led to a high wastage of cement and bentonite.
- Three columns failed to reach the required depth because of the presence of hard rock lenses in the relatively soft ground

The first problem was solved by a modification in the drilling and jetting procedure. Two additional columns were constructed at a spacing of 1.20 m in which the column was eroded in a descending stage by water and air only. Subsequently, the grout was injected in the ascending stage, as illustrated in Fig. 3. Two benefits resulted: the cement wastage was decreased significantly and the procedure allowed a better control of the final grout mix in the column.

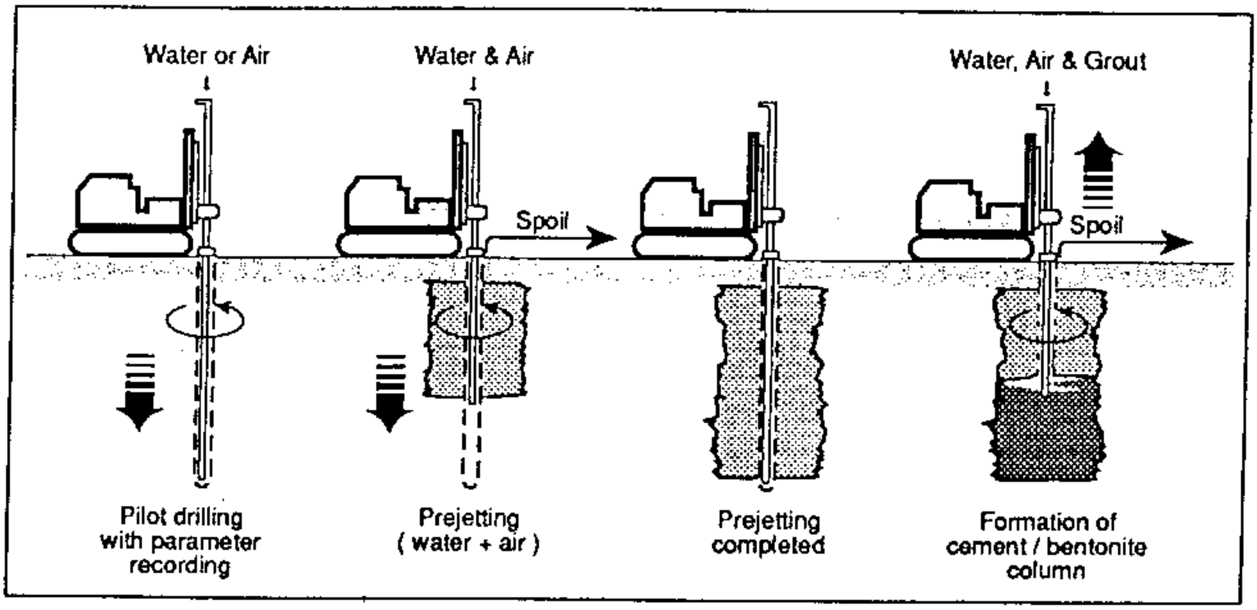

Fig. 3 Jet grouting procedure with pilot hole drilling followed by pre-jetting in the descending stage used at Thika dam

To overcome the problem of hard rock layers, it was necessary to pre-drill the hole alternatively with standard rotary tricone drilling in the soil material and with a down-the-hole hammer in the rock. Two test columns were executed in this way penetrating a 3.5 m thick layer of phonolite located at a depth between 5 and 10 m.

The trial panel, including the two columns where pre-jetting was used, were inspected by inclined check holes drilled diagonally across the panel in the axial direction. In addition, the panel was excavated to a depth of 13 m. The column diameters were measured and related to the different energy input used in the different layers of Grades IV and V. Based on these results energy levels per meter borehole length of 76 MJ/m and 22 MJ/m were selected to enable a 1 m nominal diameter column in material of Grade IV and of Grade V respectively. The erosive energy is the product of fluid pressure (MPa), flow rate (m^3/s) and time (s). In rock-like materials, which are not susceptible to erosion (Grades I to III), a lower erosive energy of 44 MJ/m was used.

The trial panel also revealed that the pre-drill holes satisfied the specified deviation from the vertical of less than 1 % thanks to the precautions taken in drilling, i.e. heavy drill rods and inclinometers on the drill bit. **Fig. 4** shows a histogram of deviations obtained over the first 10,000 m of jet grout drilling. Pre-drilling also has the benefit that it provides advance information on the various strata encountered during penetration of the ground. A drilling monitor furnished continuous data of the drilling parameters (speed of advance, drilling fluid pressure, and torque).

A5.3.5 Jet-grouted cutoff construction

Production jet-grouting of the cutoff was divided into 19 panels, each of 24 m length and each containing 30 jet grout columns. Column spacing was maintained at 800 mm as originally specified although it had been shown that with pre-jetting average diameters of 1200 mm could be achieved. In order to optimize the contact between adjacent columns and to avoid jetting in the vicinity of a recently completed column, primary holes were first installed at 3.6 m centers, followed by secondary holes in between and finally tertiary holes midway between secondaries.

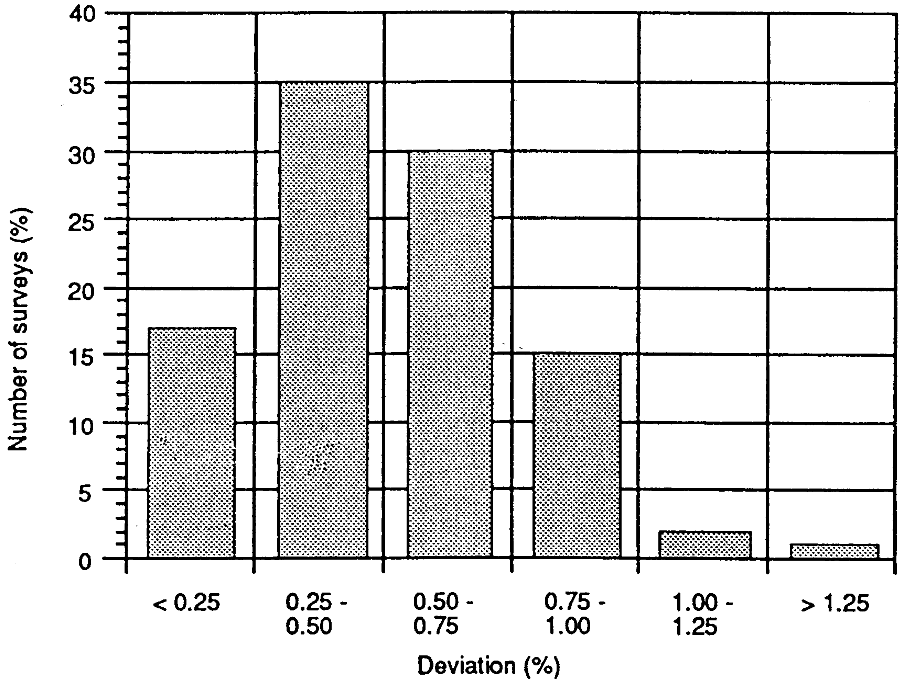

Fig. 4 Results of pilot hole deviation measurements

The execution of a jet-grouted column was accomplished as follows (see Fig. 3):

- A 120 mm diameter pilot hole is drilled to the full length of the jet grout curtain. With sufficient time between pre-drilling and jetting, the hole can be checked for its verticality. Moreover the drilling monitor can detect the locations of harder strata where the jetting parameters have to be adjusted. For pre-drilling, both rotary and down-the-hole percussion drilling is employed according to the hardness of the strata encountered. The open pilot holes are supported by a weak bentonite-cement grout.
- The rig for jet grouting is positioned over the pre-drilled hole and the triple fluid monitor is inserted. While being lowered it is slowly rotated and a a high pressure water jet erodes the the borehole wall and creates a slurry column.
- After having reached the bottom of the pre-drilled hole, the conventional triple fluid jetting procedure is started and the monitor is moved upwards producing a soilcrete column, while the soil-cement-water slurry is driven out.

Access to the relatively steep flanks of the valley was by means of two pairs of railway tracks. Two drilling platforms were erected, one on each pair of rails. The platforms could be moved up and down the slope powered by winches located at the top of the slope. Between the two pairs of rails there was a trench which followed the axis of the jet grout curtain and which also served to dispose spoil by gravity to a collecting pond at the valley bottom. With this construction set-up, an average output of 264 linear meters of column per week could be achieved.

A5.3.6 Permeation grouting

The less weathered zones (Grades I, II and III) were amenable to conventional cement grouting. Hence an injection grout curtain was connected to the completed jet grout curtain to a maximum depth of 35 m below the bottom of the jet grout curtain (see Fig. 2). The injection grout holes were drilled on the same axis as the jet grouted columns with a primary spacing between primary holes of 6.4 m and 1.6 m between tertiary holes. This spacing of the injection grout holes was given by the jet grout column spacing because the injection grout holes coincide with the axis of the jet grout columns, i.e. every second jet grout column was drilled for injection grouting. This arrangement allowed verifying the continuity of the jet grouted columns and ensured the continuity of the watertight barrier at the base of the jet grout column. Downstage grouting was employed with specified stages of 5 m length. The stage length, however, had to be adjusted according to the availability of sufficiently competent rock strata where the single packer could be set.

A5.3.7 Lessons learnt

The jet grouting method was applied successfully in a dam foundation characterized by a heterogeneous sequence of layers of different hardness subjected to tropical weathering. The initially high consumption of cement of bentonite when using the triple fluid monitor was significantly reduced by pre-jetting the pilot holes with a water jet in the descending stage.. Pre-drilling enabled the early localization of hard layers which later were penetrated with a down-the-hole hammer. A trial panel provided significant information to optimize the construction of the jet grout columns

An arrangement of drilling platforms on rails enabled to master the difficulties prevailing in a remote valley with relatively steep slopes. By drilling the grout holes through the completed jet grout curtain, the jet grout curtain could be extended by an injection grout curtain to form a continuous watertight barrier to a depth of about 65 m.

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A5.4 El Tambor dam (Colombia)

A5.4.1 Project description

El Tambor dam forms part of the San Rafael project which was conceived to assure the water supply to the capital city of Santafé de Bogotá. The dam is a 60 m high homogeneous earthfill structure with chimney drain and horizontal drains. On its left abutment the dam is extended by a 15.5 m high and 282 m long auxiliary dike and an open channel spillway. The construction of the dam was completed in March 1994 and impounding started in February 1997. This case history is based on a description given by Marulanda et al (2001)

A5.4.2 Foundation conditions

The area of the dam site is characterized by sedimentary rocks of tertiary and cretaceous age. These rocks are partially covered by quaternary deposits, mainly lacustrine and alluvial sediments, residual and organic soils. Two main formations constitute the foundation of the dam, namely the Guaduas formation consisting essentially of claystones with occasionally interbedded sandstone beds and the Guadalupe formation composed of mainly fine grained sandstones. The rocks have been subjected to intensive tectonic processes, mainly folding, resulting in a system of asymmetric synclines and anticlines. At the dam site the axis of the folds and generally all the beds intersect the dam axis almost perpendicularly. This is a rather unfavorable situation as any permeable stratum can turn into a water conduit from the reservoir. Figure 1 is a section along the dam axis illustrating the nature of the folds.

The shallow quaternary deposits were usually removed within the footprint of the dam , namely at those locations where their properties proved to be poorer than those of the compacted fill of the dam. Only a 4 m thick, colluvial deposit, located at the upper part of the right abutment (see Fig. 1), was left in place because its removal could have caused instability for a large volume of soil outside the dam foundation.

The grayish and reddish Guaduas claystone alternates with siltstone, lenses of coal and with some pockets of yellowish sandstone. This sandstone can be hard and fractured but sometimes also friable and water bearing. This type of foundation exists below the dam and the right abutment.

The left abutment in its upper part, carrying the dike, is made up of the yellowish and grayish Guadalupe sandstone with a variable content of a clayey matrix and with thin intercalations of grayish clay. Cementation of this sandstone is locally often weak and the rock is friable and susceptible to erosion. The sandstone had a higher permeability than the claystone but it was difficult to impossible to determine the coefficient of hydraulic conductivity, k, from a field test, such as the Lugeon test, because of difficulties in drilling in the friable material and extract cores. Local collapses in the borehole also occurred. There was therefore concern that this sandstone could be a source of leakage which may eventually develop into piping.

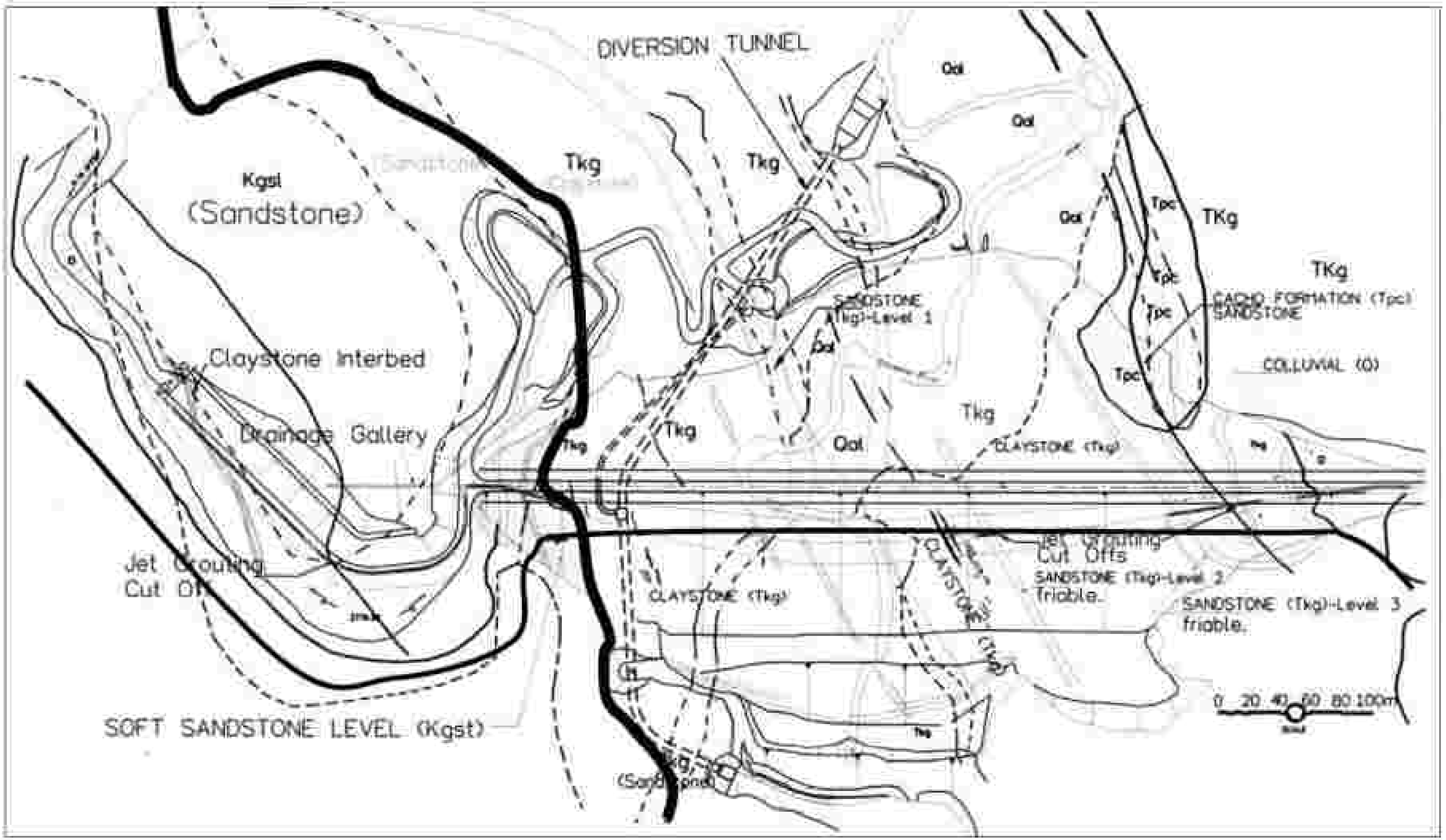

Fig. 1 Layout of El Tambor dam with geology

A5.4.3 Foundation treatment

Leakage through the foundation could take place along two different paths, namely (1) the sandstone intercalations in the Guaduas claystone located below the embankment and on the right abutment, or (2) through the Guadalupe sandstone on the left abutment. Both types of sandstone had in common that their permeability was higher than that of the claystone. But this permeability was still too low for treatment by conventional cement-bentonite permeation grouting.

Under these circumstances it was decided to treat the rock foundation with a combined system of conventional grout and jet grout cutoffs as shown in Fig. 2. Conventional grouting would be executed in most of the foundation where claystone was prevailing. The permeability would be checked and the hard, fractured sandstone intercalations could be treated. Jet grouting was intended to be used in the friable and piping susceptible sandstones, not amenable to treatment by conventional grouting. But even though some specific zones for each kind of treatment were determined, the extension of the treatment depended on the actual conditions found in the ground and on the effectiveness of the treatment achieved with each type of rock.

The most critical zone for watertightening was the contact between the auxiliary dike and the main embankment on the left abutment. Hence, the jet grout curtain was wrapped around the sandstone ridge to connect to the claystone whose highest elevation was at 2754. The cutoff was deepened to elevation 2740 (see Fig. 3) to achieve watertightness. The lowest registered water table at elevation 2745 indicated that the lower rock mass of the sandstone had a lower permeability. The jet grout curtain had to be continuous, impervious, with a minimum thickness of 1.5 m within the friable sandstone susceptible to erosion. For the case of hard and pervious rocks, jet grouting could be substituted by conventional grouting.

A5.4.4 Jet grouting tests

In order to evaluate the efficiency of the jet grouting method on each of the two foundation materials, i.e. claystone and sandstone, field tests were carried out. Several columns were constructed using the triple fluid monitor with the following jetting parameters: Grout pressure 6 MPa, water pressure 40 MPa, air pressure 1.2 MPa, rotational speed 10 rpm, lifting speed 0.4 m/min and 0.145 m/min. Four columns were excavated and the soilcrete properties tested (Fig. 4). The results indicated the following:

- An average diameter of around 1.2 m could be achieved with the claystone and 2.2 m with the sandstone.
- A 20 % increase in the column diameter could be achieved when applying pre-jetting with water downward and upward
- The tests demonstrated that it was possible to produce continuous columns of 2 m diameter (compared to the 1.5 m specified) and install a curtain with columns spaced at 0.9 m centers. Two rows were specified, spaced at 0.80 m.
- The cement consumption was 750 kg per linear meter of borehole with the addition of 1.5 % bentonite.

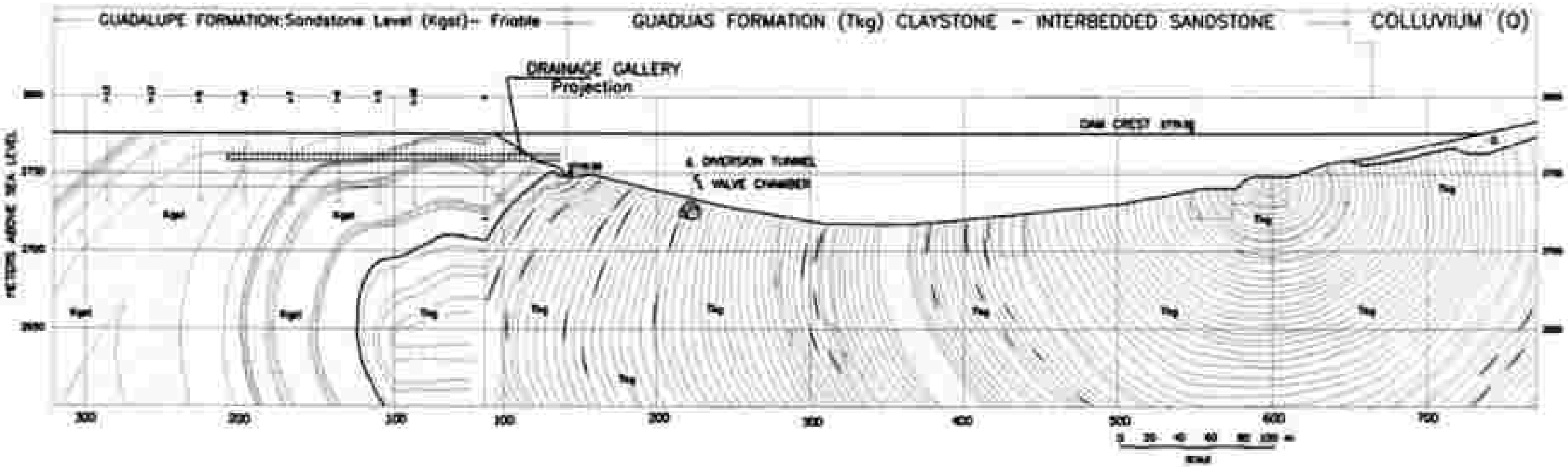

Fig. 2 Geological section along dam axis (Kgst = Guadalupe sandstone; Tkg = Guaduas claystone; Qal = alluvium)

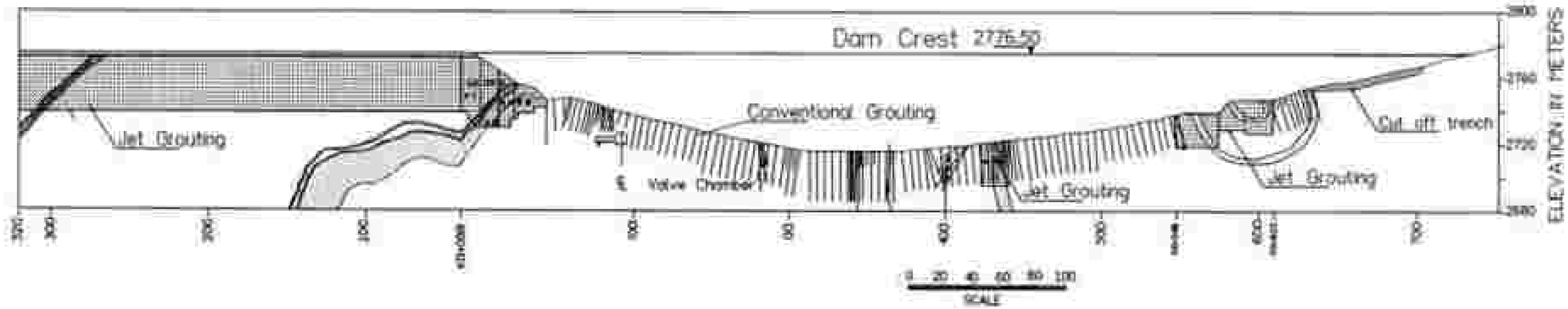

Fig. 3 Foundation treatment along dam axis and left abutment

A5.4.5 Jet grouted cutoff construction

Fig. 5 shows the layout of the equipment used for the jet grouting work. The main components are: air storage and injection, grout mix preparation and storage, water injection, and dosage and boring. The injection equipment consisted of a crane with pole, providing coupling with two rods as the bore advanced. The drill had a diameter of 150 mm. The end of the rod carried the monitor equipped with three nozzles, i.e. two for water (2.4 mm and 3.8 mm) surrounded by a shroud of air, and one nozzle of 6.0 mm diameter for the grout. A water pressure of 40 MPa was used to erode the rock assisted by an air pressure of 1.2 MPa. The soilcrete columns were formed with a cement grout pressure of 6 MPa. Two Tecniwell pumps were employed for the water cutting the rock and another two pumps were used for the grout mix.

Fig. 4 Field test: Excavated jet grout columns

To guarantee that the verticality of the drillholes would stay within the specified 1% of their length, pilot holes were drilled using a Link Belt type CGP 30 drilling machine that used a 30 m high boom to facilitate pipe overlaps. This procedure allowed the use of 90 mm diameter jet grouting pipes instead of the 127 mm diameter pipes planned initially.

The complete treatment of the dam foundation and of the ridge on the left abutment with jet grouting involved the construction of five cutoff sections with a variable depth between 17 m and 34 m resulting in a total area of 12,500 m^2. The locations of the jet-grouted sections are shown in Fig. 3 and the sequence of column construction is illustrated in Fig. 6.

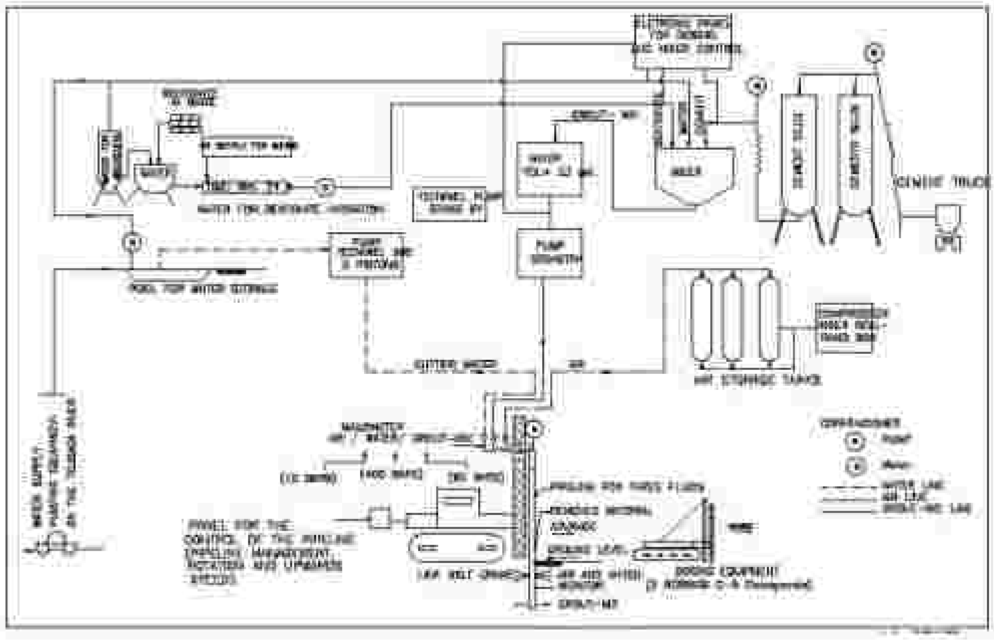

Fig. 5 Equipment used for jet grouting

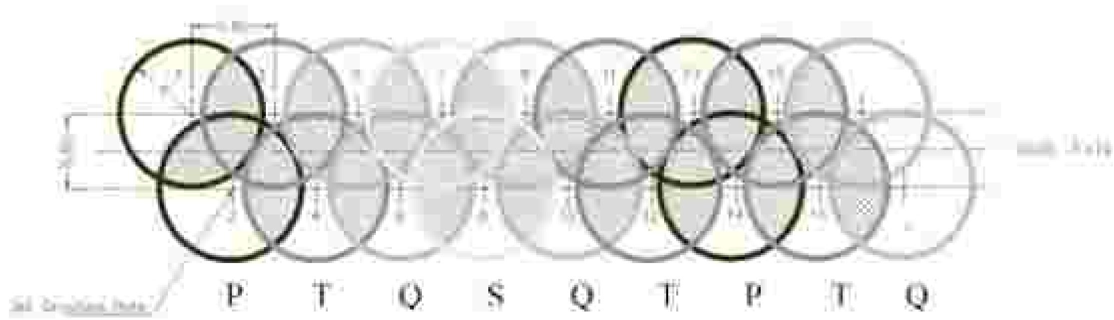

Fig. 6 Sequence of jet grout column installation (P=primary, S=secondary, T=tertiary, Q=quaternary)

A5.4.6 Quality control

Extensive routine controls were performed on the various materials (cement, bentonite, grout mix) and on the completed cutoffs, as summarized below.

Cement: The Portland cement Type I used was checked for Blaine, fineness, normal consistency, loss due to calcinations, unsolvable residues. MgO and SO_3 percentages, and curing time.

Bentonite: Bentonites known under the trade name Arcicol and Colgel were used and tested for Atterberg limits, water content, maximum content of sand, pH, particle size distribution, maximum seepage, maximum residues, and maximum heaving volume.

Grout mix: These tests comprised the determination of the density, viscosity, percentage of sedimentation and unconfined compressive strength. The values of the Marsh viscosity were on average 1.35. The average density was 1510 kg/m^3.

Jet grouted cutoff wall: The quality controls included check boring to verify verticality, measure the permeability by water pressure tests and take samples for unconfined compressive strength tests.

The width of the jet-grouted cutoff could be visualized when the left abutment was stripped from overburden. It was around 2.2 m.

A5.4.7 Monitoring and performance

Monitoring was an essential component in the performance assessment of the jet-grouted cutoff, since the jet-grouted sections had to seal locations of weak friable rock with susceptibility for piping. A drainage gallery was incorporated at the left abutment downstream of the jet-grouted cutoff in order to relieve possible high gradients (see Fig. **7**). Six piezometer stations (boreholes with multiple piezometers) were placed into the gallery. Fig. 8 illustrates the positioning of these piezometers at elevation 2740. Another line with piezometers was located 10 m higher.

The reservoir level rose to a maximum of about El. 2773 while the piezometric head in the drainage gallery increased to El. 2767. The head efficiency, E, calculated at different locations along the dam axis was only about 45 % at the left abutment and at the center of the dam. But it reached a value of over 90 % at the right abutment. The total leakage of the dam on the downstream side with full reservoir was, however, only 30 liters/s.

The total costs of all the jet grouted sections amounted to about US$ 7 million.

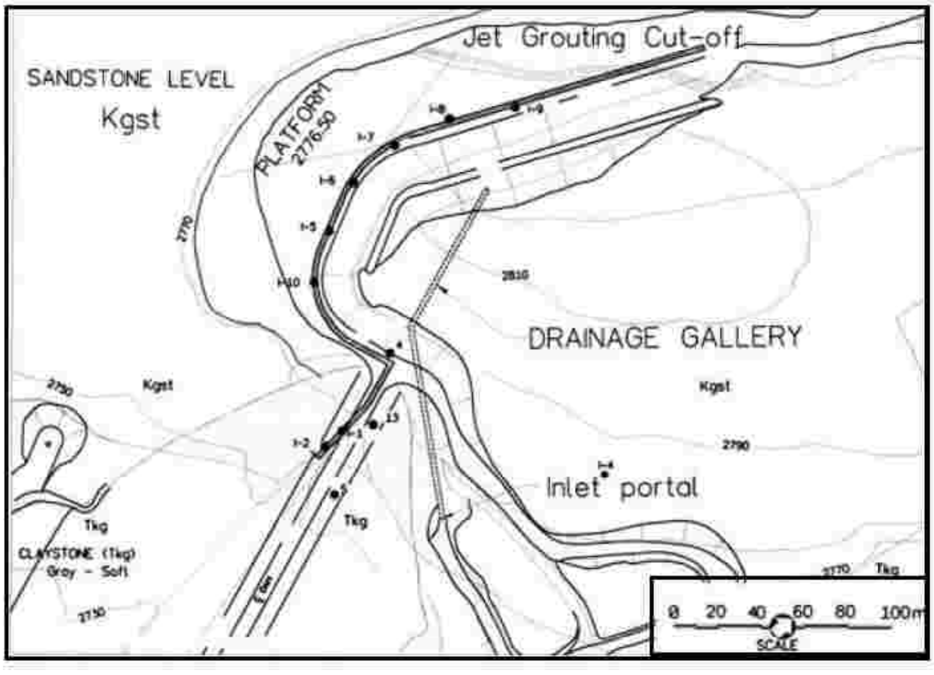

Fig. 7 Left abutment: Jet grout curtain and drainage gallery with piezometer stations

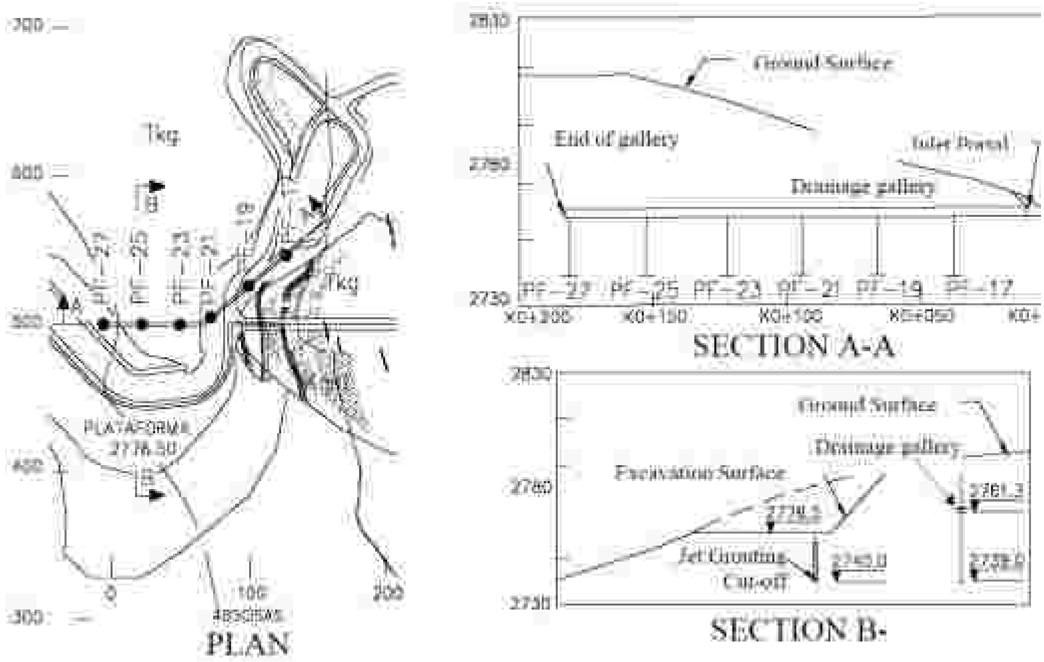

Fig. 8 Piezometer locations in drainage gallery on left abutment

A5.4.8 Lessons learned

The case history illustrates the use of jet grouting in weak rock, i.e. in a claystone and a sandstone with sections of weak cementation. The diameters of the jet grouted columns achieved in these materials are substantial. Pre-jetting with water and air helped to enlarge the column diameter.

Another interesting feature is the side by side use of jet grouting and conventional permeation grouting depending on the conditions of the rock to be treated. In this way an optimal sealing of the dam foundation can be obtained, which has been verified by the very low total leakage measured.

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Appendix 6. CASE HISTORIES ON DEEP MIXING CUTOFF WALLS

A6.1 Cutoff wall at Jackson Lake Dam (USA)

A6.1.1 Project description and remedial measures

The Jackson Lake Dam is located in Grand Teton National Park, Wyoming, USA. It consists of a 1311 m long, 15 m high, hydraulic fill embankment section, a 67 m long concrete spillway spanning the Snake River, and a 45 m long dyke, mainly constructed of rockfill (Fig. 1). Construction took place between 1906 and 1916 whereby the structure was raised twice. The embankments are founded on generally loose, saturated fluvio-lacustrine sediments consisting of a complex, interbedded series of gravel, sand, and fines, extending to a depth of over 30 m.

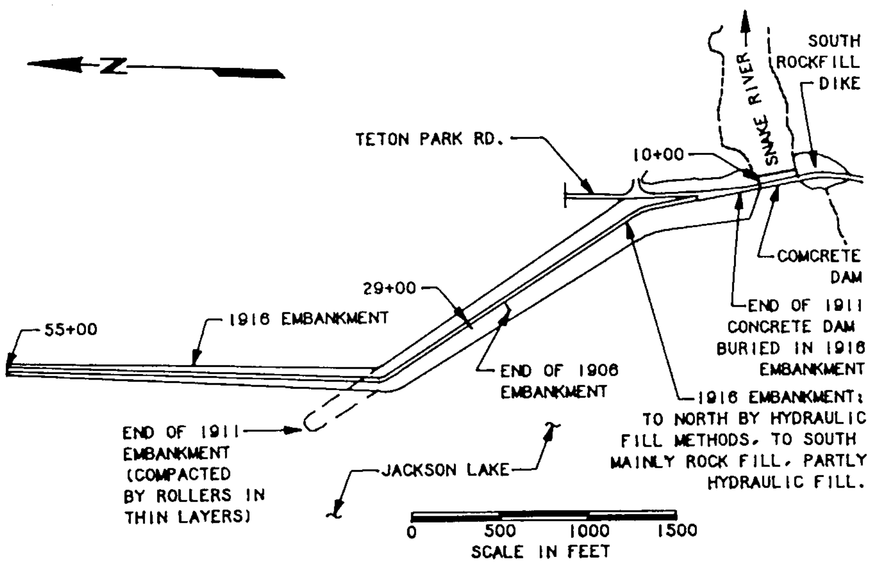


Fig. 1 Schematic plan view of Jackson lake Dam (Pujol-Rius et al., 1989)

The dam is located in a seismically active region, about 15 km from the Teton Fault, which is estimated to be capable of a 7.5 Richter magnitude earthquake. In this seismic environment there is a potential of liquefaction of the foundation soil below the embankment. The loss of strength in the foundation could lead to cracking, sliding and possibly disastrous failure of the embankment. Consequently, considerations of dam safety required remedial modifications. These modifications, their planning and execution, are described by Pujol-Rius et al., 1989; Ryan & Jasperse, 1989 and Taki & Yang, 1991. The following summary is based on these references.

The remedial measures for the foundation consisted of two stages:

Stage I: Excavation of the northern two thirds of the north embankment and densification of the upper 12 m of the foundation by dynamic compaction

Stage II: Removal of the remaining embankment and liquefaction stabilization of the 580 m long southernmost section to maximum depth of about 32 m by deep mixing (SMW-type), i.e. a minimum of 3 m into the silt layer which was considered liquefaction resistant.

Construction of a 1200 m long cutoff wall along upstream toe of the embankment to a maximum depth of about 33 m, i.e. 6m in to the silt layer, to control foundation seepage and decrease uplift pressures under the embankment.

Initially, several methods of stabilization were contemplated. Deep mixing, however, was proposed by the contractor as a possible variation. Since much emphasis was on the system chosen and on how quality could be assured, deep mixing was selected as the most attractive solution. Fig. 2 and Fig. 3 illustrate the treatments performed and the corresponding zones. Liquefaction stabilization was performed in two approximately 15 m wide zones below the upstream and downstream toes of the embankment. The approximately 40 m wide central zone in between was densified by dynamic compaction to a depth of 9 m. The SMW stabilized zones provide containment for potentially liquefiable soil below the depth effectively treated by dynamic compaction. The cutoff wall under the upstream toe was constructed using the same technique. It forms the upstream boundary of the upstream-stabilized zone.

For the stabilized zones the SMW work was carried out by two two-auger machines. These were equipped with 3 feet (910 mm) non-overlapping augers which produced two column panels with each column tangent to the adjacent one. Panels were installed consecutively, with one column of each panel completely overlapping the last column of the previous panel, as shown in Fig. 4a. The augers consisted of hollow shafts with discontinuous flights. At the bottom of each shaft there was an earth auger cutting head.

The soilcrete panels were arranged in a honeycomb-type grid (see **Fig. 5**). This configuration was found to be the most efficient with respect to liquefaction stabilization. A total of 130,000 m of soilcrete columns were constructed.

The cutoff wall was constructed by a three-auger machine with 34 inch (86 mm) diameter overlapping augers. Installation was accomplished by drilling alternate “primary” panels followed by interspersed “secondary” panels (Fig. 4b). Each end column of the secondary panels completely overlapped the end column of the adjacent panel. The cutoff was specified to have a minimum thickness of 610 mm.

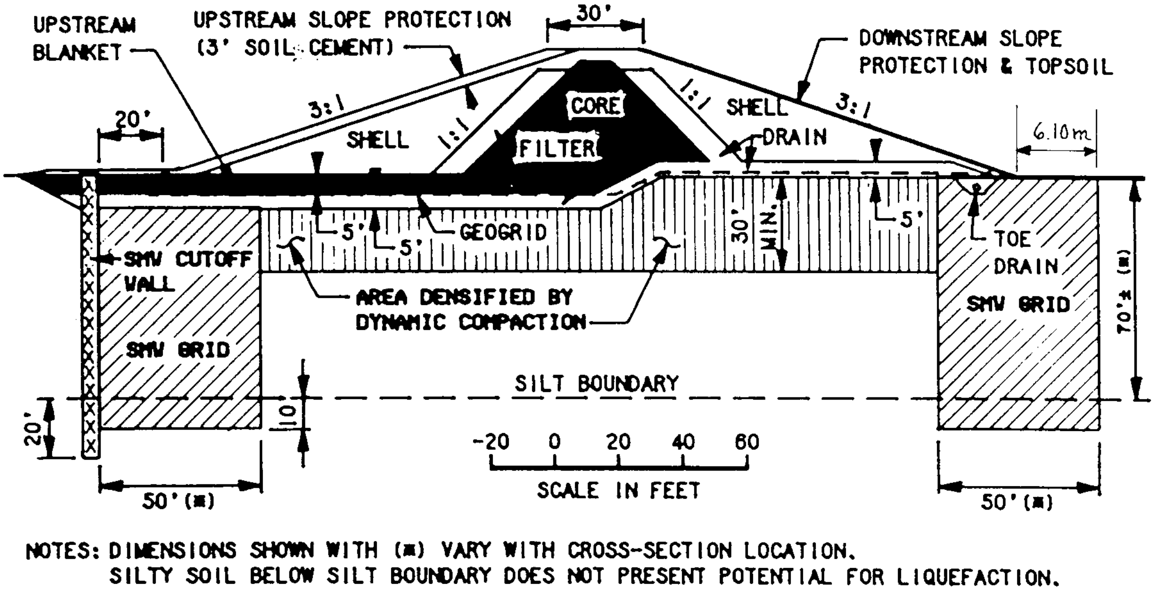


Fig. 2 Jackson Lake Dam: Typical cross-section of embankment and configuration of foundation treatment (Pujol-Rius et al., 1989)

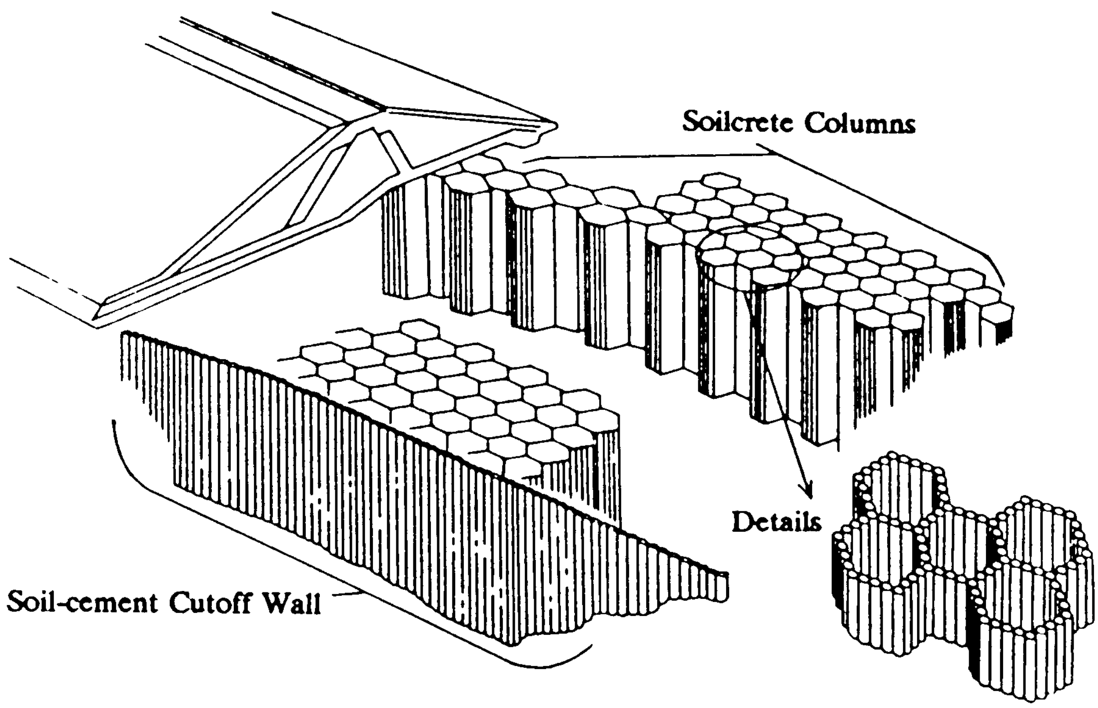


Fig. 3 Jackson Lake Dam: Schematic view of different elements of soil treatment work (Taki & Yang, 1991)

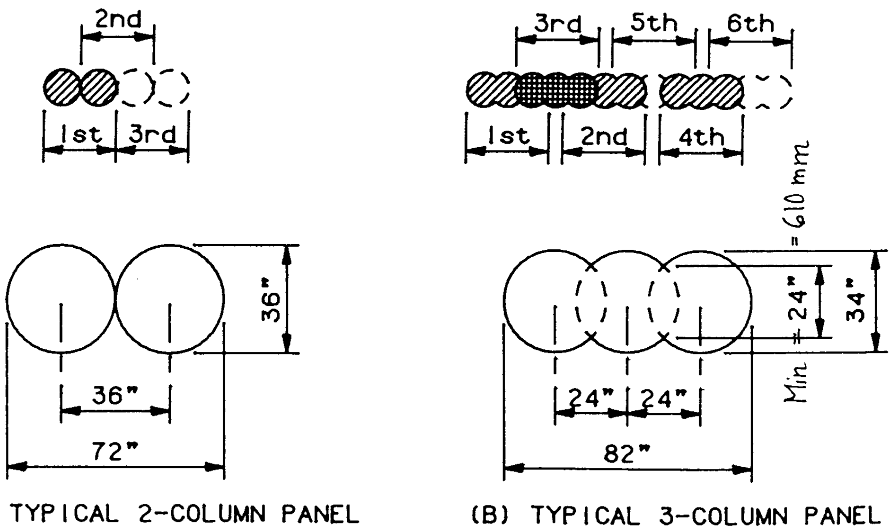


Fig. 4 Jackson Lake Dam: Dimensions and installation sequence of soil-cement panels: (a) non-overlapping 2-column panel used for honeycomb-type grid, (b) overlapping 3-column panel used for cutoff wall (Pujol-Rius et al., 1989)

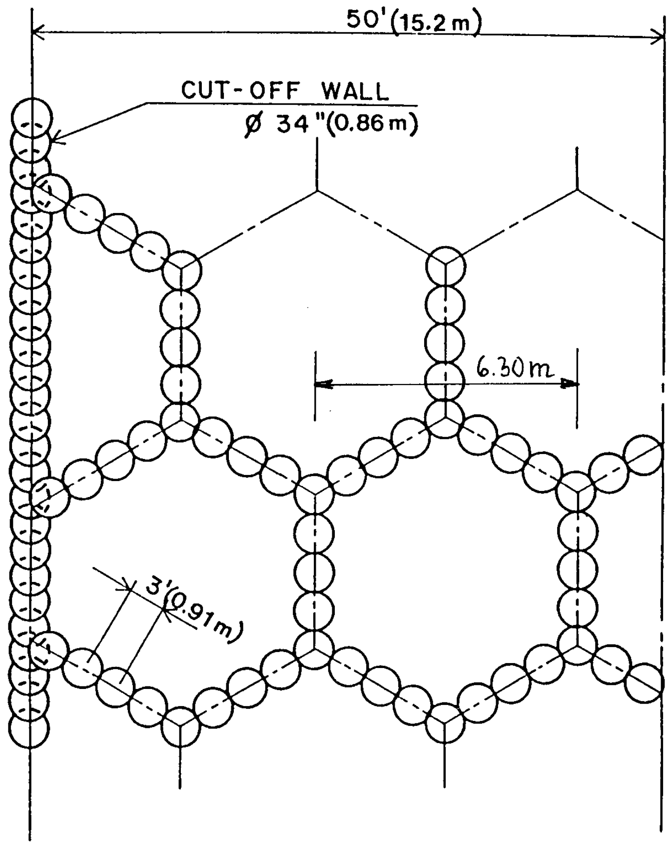


Fig. 5 Jackson Lake Dam: Honey-comb grid of soil-cement columns for liquefaction stabilization (Ryan & Jasperse, 1989)

A.6.1.2 Material requirements

Specifications called for minimum shear strength of the soil-cement mix of 1380 kPa.

A6.1.3 Quality control

The grout flow was controlled by electronic flow metering devices. These devices caused a predetermined amount of grout per meter of drilling to be injected into the column. Digital readout displayed grout pressure, depth of column, verticality, drill rate, grout flow rate, and total grout flow.

For the purpose of controlling the strength of the soil-cement mix, the following tests were performed:

- Sampling and testing of one SMW column from each day's production by taking "wet mix" samples from two depths using an inflatable packer sampler specially developed for the project. More than 2000 samples were taken. These were cured in the laboratory for 7, 28, 56, and 112 days under controlled moisture and temperature.

- Samples taken from core drillings in the hardened soil-cement columns.

The strength of the specimens was assessed from unconfined compression, triaxial compression and direct shear tests. The shear strength determined from triaxial and direct shear tests was estimated conservatively to be about one third of the unconfined compressive strength. Hence, the specified strength of 1380 kPa corresponded to an unconfined compressive strength of 4140 kPa. It was then determined that a 7 day wet sample compressive strength of 860 –1030 kPa would yield a fully cured column of 1380 kPa shear strength. This approach served as a quick strength criterion to assure that the deep mixing process would satisfy the specified strength.

The following results of general validity were observed from the large amount of tests conducted:

(1) The strength of soilcrete is highly time-dependent. The 112 day strength was about 60% higher than the 28-day strength (Fig. 6).

(2) The strength of cored samples was higher than the strength of specimens of the same age sampled field-wet. This is because of better curing conditions in-situ (Fig. 8.3b).

(3) The key factor determining the final strength is the water/cement ratio; it is even more important than the cement content.

The strength test results were used to make adjustments in the mix design. The water/cement ratio was reduced from 1.35 to 1.25 and the cement content from 475 kg/m^3 to 340 kg/m^3. Bentonite was also added in the amount of 4.8 kg/m^3.

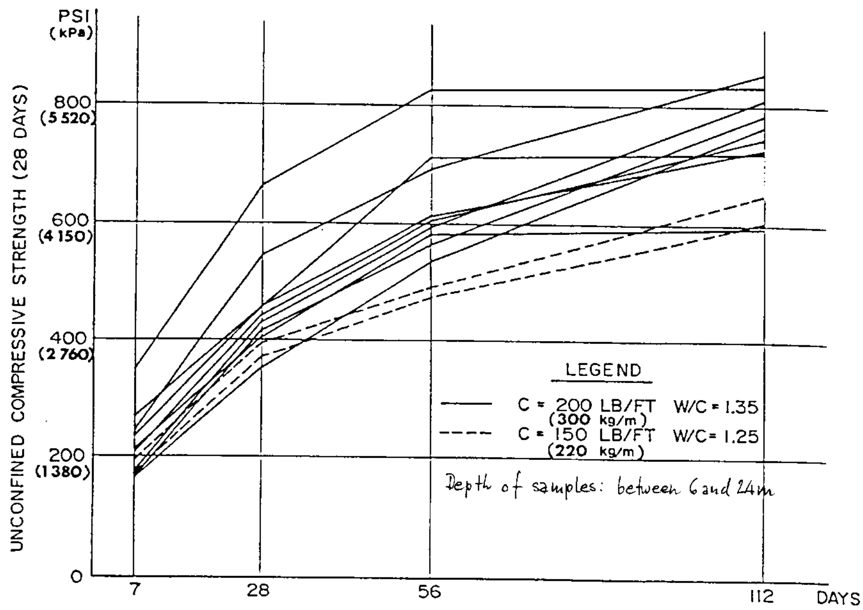


Fig. 6 Increase of soil-cement strength with time (Jackson Lake Dam) (Ryan & Jasperse, 1989)

A6.2 Lake Cushman spillway (USA)

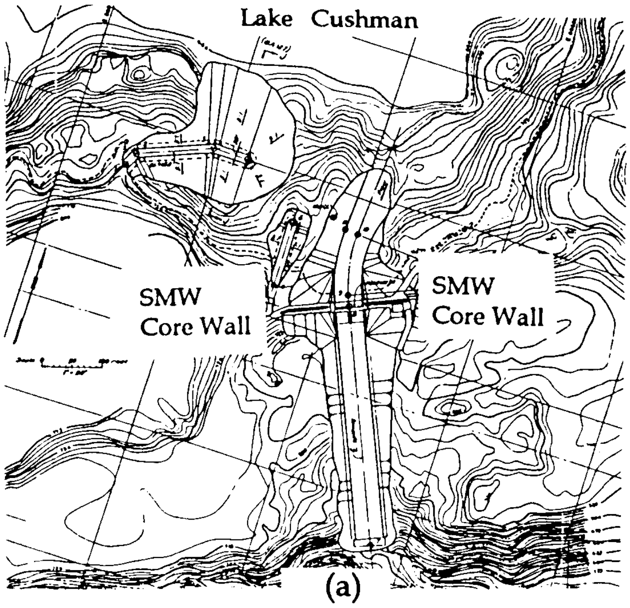
A6.2.1 Project description

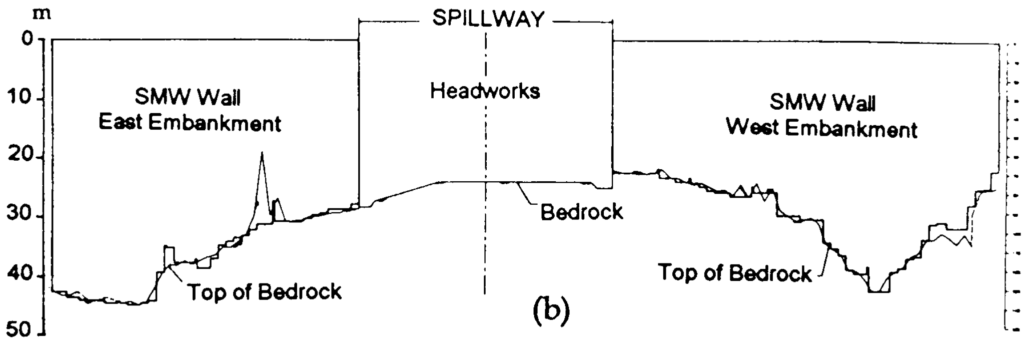
The construction of a new spillway with radial gates at Lake Cushman, Washington, USA, required embankment sections on either side of the spillway headworks (Fig. 7a & Fig. 7b). The foundation conditions consisted of glacial deposits composed of recessional outwash, lacustrine deposits, followed by lodgement till which in turn was underlain by basalt rock with little or no weathering zone (Fig. 7c). The headworks and short sections of the embankments abutting the spillway wall were founded on the basalt. The lodgement till was composed of very dense gravelly sandy silt and silty sand and gravel with SPT blow counts of 50 for less than 15 cm of spoon penetration. The permeability of the glacial deposits ranged from 10^{-4} to 10^{-7} m/s and that of the bedrock from 10^{-6} to 10^{-9} m/s (Yang & Takeshima, 1994).

A6.2.2 Soil-cement wall design.

Both, glacial outwash and glacial till materials had a relatively high permeability calling for seepage control. There were, however, no impervious materials in the vicinity of the site for constructing an embankment core. It was therefore decided to construct a soil-cement wall to serve as cutoff in both the embankment and the foundation. This core wall was extended to the irregular bedrock surface and reached a maximum depth of 43 m (see Fig. 7b).

For preventing leakage along the contact between embankment fill and concrete wall of the spillway headworks, a special design was conceived (Fig. 8). The core wall was embedded into the U-shaped end of a connecting, steel-reinforced concrete wall. The space between the soil-cement wall and the surrounding U-shaped concrete wall was backfilled with a mixture of bentonite and sand.





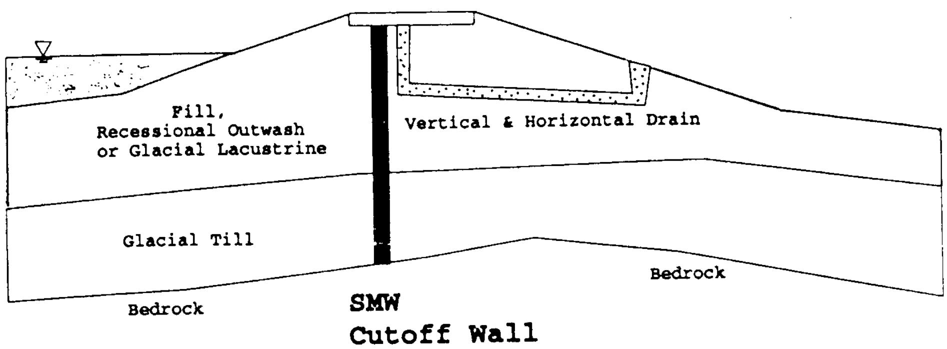


Fig. 7 Lake Cushman spillway: (a) site plan; (b) cutoff wall profile; (c) simplified general cross-section (Yang & Takeshima, 1994)

A 3.4 m long and 4.6 m deep trial section was constructed in the glacial deposits prior to the installation of the actual wall to evaluate the most appropriate mix design. The wall was then excavated for inspection.

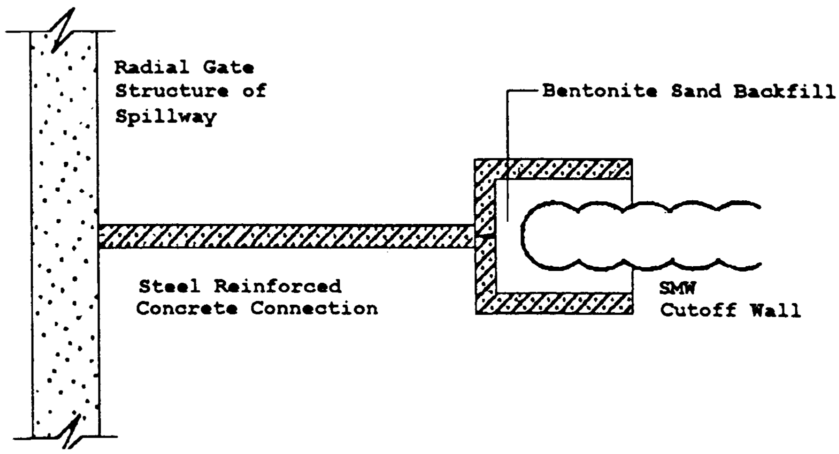


Fig. 8 Lake Cushman spillway: Connection between soil cement wall and spillway wall (Yang & Takeshima, 1994)

For construction of the full-size wall the SMW equipment was positioned on top of the embankment. The width of the working pad was 11 m to 13 m and sufficient for the core-wall installation along an alignment located at 2 m from the upstream shoulder of the embankment. The wall was installed panel by panel. Some difficulties arose with the hard lodgement till stratum. To overcome drilling in this layer, pre-drilling with a single auger was first performed. This was then followed by using a three-axis auger, as shown in Fig. 9. At some locations large boulders were encountered and it was necessary to provide for a by-pass section around the boulder, but maintaining the continuity of the wall. Since the depth of the bedrock was irregular, an important element in quality control was to ensure that the wall had reached the top of the bedrock.

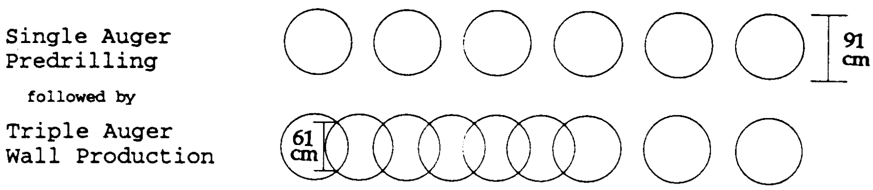


Fig. 9 Lake Cushman spillway: Installation procedures for soilcrete wall

A6.2.3 Material properties.

Mix design used cement dosages from 350 to 550 kg per m^3 of in situ soil. The dimensions of the cutoff walls in the two embankments and the strengths and permeabilities of the soil-cement achieved are shown in Table 1.

Table 1 Geometry and material characteristics of soil-cement cutoff walls

	Area (m^2)	Length (m)	Thickness (m)	28-day strength (kPa)	Permeability (m/s)
West wall	1733	61	0.61	594 - 4396	$2x10^{-7}$ to $6x10^{-9}$
East wall	2219	55	0.61	1800 – 4790	$1x10^{-8}$ to $7x10^{-9}$

A6.3 Sylvenstein dam heightening (Germany)

A6.3.1 Project description

Sylvenstein dam is an earthfill structure located on the Isar River near Bad Tölz in Germany. Its main purpose is flood control but it also stores water for times of low flow. Recent changes in the hydrologic regime required a heightening of the dam crest by 3m (see Fig. 10). To ensure watertightness the top of the impervious element had to be adjusted accordingly.

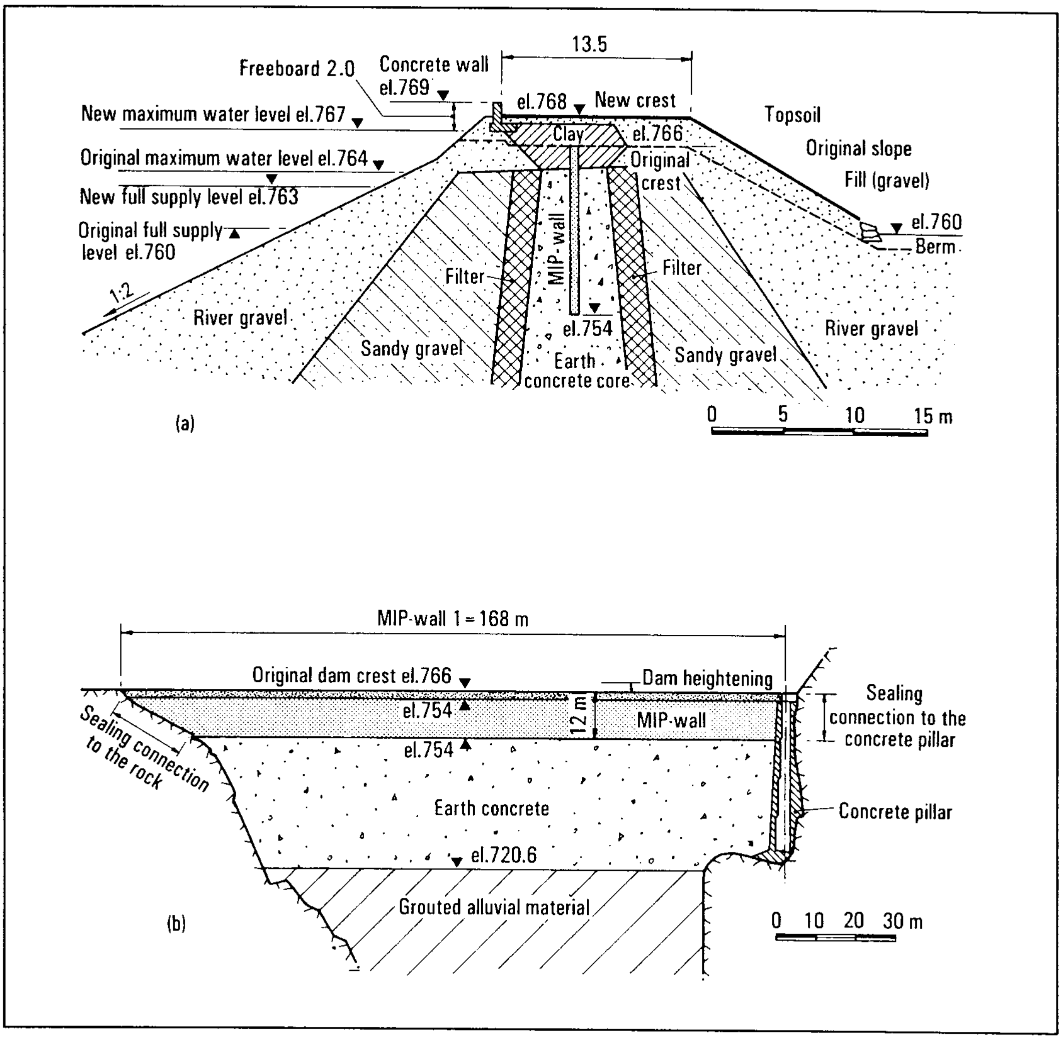


Fig. 10 Heightening of Sylvenstein dam: (a) cross section; (b) longitudinal section along dam axis (Wilder et al., 2002)

Design criteria required the element connecting to the existing core to have a modulus of elasticity of similar magnitude as the surrounding gravel fill in order to prevent, or at least minimize the development of cracks in case of differential settlements. Special attention was given to the connection with the abutments. The cutoff element was to be embedded into the existing core by 12 m. The tender called for a vib wall, however, the contractor proposed a soil-cement mix wall which was accepted due to technical and financial advantages.

A6.3.2 Construction.

A track-mounted drilling machine with triple axis auger shaft, auger diameter 550 mm was used. It enabled a maximum drilling depth of 16.5 m. The sequence of panel installation is shown in Fig. 11. After primary and secondary panels had been installed, supplementary panels were added to ensure better continuity of the wall. However, during sinking of these panels only a small amount of slurry was mixed with the re-drilled soil-cement material.

The width of the finished wall corresponded to the auger diameter. The overlaps specified for the various panels are also given in Fig. 11. This equipment was capable to produce about 200 m^2 of wall per working day. Sink and rise velocities of the augers, the volumes of slurry mixed, and sense of rotation of the augers were recorded automatically by a data logger located in the operator's cabin of the drilling machine.

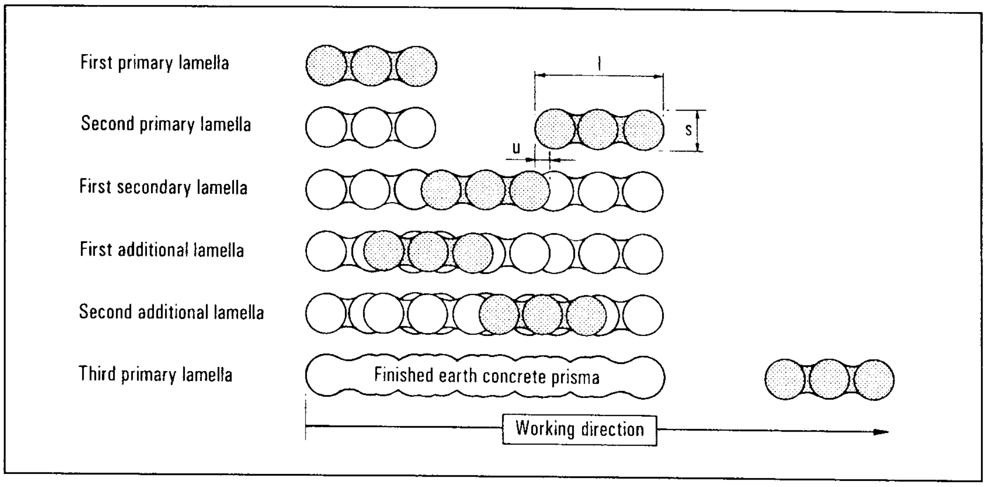


Fig. 11 Sylvenstein dam: Steps in the construction of the soil-cement mixed wall (Wilder et al., 2002)

A6.3.3 Quality control.

Emphasis was on the use of materials with proper characteristics mainly with respect to strength, deformability and permeability. Quality checks ensured that the properties remained within specified limits. Actually, the original composition of the slurry had to be modified as it turned out that setting of the mix was too fast. More bentonite and rock flour were added, but less cement. The final composition was then:

- Cement 300 kg
- Bentonite 50 kg
- Rock flour (from limestone) 300 kg
- Water 780 liter

This change in composition resulted in a lower compressive strength and deformation modulus, but did not affect the permeability.

A6.3.3.1 Temperature control.

Periods of frost could not be excluded during the construction phase, but it was assumed that the temperature inside the earth material would not fall below 5°C. In order to ensure quick hydration of the soil-cement wall, the minimum allowable temperature was specified as 8°C. Temperature measurements were carried out daily at the nozzle of the auger.

A6.3.3.2 Density control.

Density was checked hourly. One of the main reasons for density control was to exclude the use of possibly erroneous mixes. The target density was 1.41 Mg/m^3. With the relatively high solids content it was expected to achieve additional stability against erosion.

A6.3.3.3 Marsh funnel viscosity.

These measurements check the uniformity composition and the degree of mixing. The mean value of Marsh viscosity was 32.5 s.

A6.3.3.4 Strength and deformability of hardened soil-cement in the wall.

A 28-day uniaxial compressive strength of 0.3 MPa was specified to ensure sufficient stability against erosion. The upper limit of strength was set at 1MPa in order to produce a soil-cement of sufficient plasticity that could deform without cracking. This requirement was enforced by monitoring the amount of slurry added per m^3 of wall to remain within specified upper and lower limits.

The elastic modulus was limited to an upper value of 80 MPa, because the modulus of the existing core of the Sylvenstein dam was estimated to range between 15 and 80 MPa.

Measurements of compressive strength and of the stress-strain behavior were carried out on specimens obtained from the finished wall. These were tested after 4, 7, 14, and 28 days after mixing. As an example, the 4-day uniaxial compressive strength ranged between 0.18 and 0.33 MPa, while the corresponding E-moduli were between 13 and 30 MPa. The objective of early tests was to detect panels with possible deficient values. Such panels could then be re-drilled and restored to reach the specified properties.

A6.3.3.5 Permeability.

For the permeability of cylindrical specimens taken from the finished wall, the value of hydraulic conductivity, k, was specified to be equal or less than 10^{-8} m/s under a hydraulic gradient of i = 30. The measured values ranged between 10^{-8} and 4×10^{-10} m/s.

Experience from this case history showed that with a comprehensive monitoring program and well-organized records of measured data all the requirements specified for the wall could be achieved. Hence, a flexible quality control on the basis of pre-determined criteria turned out to be a successful approach.

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