Major Dam Rehabilitation by Specialist Geotechnical Construction Techniques: A State of Practice Review

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1. Introduction.

As we move into the last few years of the century, the emphasis in dam engineering is firmly on the monitoring, reassessment and rehabilitation of existing structures. Considering US statistics alone, State governments regulate more than 80,000 dams, and the Federal Government owns 3,000 more. According to a Corps of Engineers' inventory, nearly one quarter of all US dams are now at least 50 years old, compared to 5% in 1980 and a projected 33% in 2,000(1). In 1981, the Corps completed inspecting about 8,800 non-Federal dams classified as high hazard. It found one third unsafe by current design and performance criteria. By 1987 only 133 had had remedial action(2). With respect to concrete dams, the ICOLD Committee on the Deterioration of Dams and Reservoirs reported in 1984 that 28% of cases of deterioration were due to foundation problems, while 70% of those were related to hydraulic behavior. In addition, 83% of accidents were associated with foundation problems(3).

In this highly charged atmosphere, major steps are being made in the dam safety research effort. For example, the Corps launched a 6 year, $35 million research program in 1984, while the Bureau of Reclamation's current $650 million evaluation and repair program covering its 350 dams is expected to peak in 1992(4). Universities are collaborating with Government agencies and independent research groups such as EPRI, while bodies such as ASDSO, CDSA, and USCOLD are increasingly turning their attentions to these matters. In addition, FERC regulates over 2,000 dams, 500 of which are concrete(5). Their experience in analyzing these dams led to the publication of "Engineering Guidelines for the Evaluation of Hydropower Projects" in 1988.

These important guidelines made special allowance for older dam designs, originating 30-75 years ago, when the understanding of uplift mechanisms, for example, was not advanced. In many other cases there is critically little information available at all, about the particular contemporary design and construction methods.

Notwithstanding all these essential efforts at monitoring and reassessing our dams, it would appear that the conclusive challenge - that of actually effecting the repairs - has fallen to specialist contractors. Today, the role of the best geotechnical and structural repair companies has expanded considerably as the practicality of actually carrying out the required repair frequently depends on the ingenuity and resources of such firms.

Such has been the pace of developments in this actual construction phase in the past decade that it is virtually impossible for the individual engineer to keep up to date. This is especially true in the dam community, where environmental, economic and political matters often consume at least as much attention. The engineer may, therefore, be forgiven for thinking that our abilities to repair lag behind our capabilities to monitor, analyze, project and pacify.

It is the purpose of this paper to review and lend perspective to the range of proven specialist techniques for repair and rehabilitation. The paper concentrates more on the nature of the repair method itself and on its application than on the details of its execution. Each technique is illustrated with respect to published case histories, and so the interested reader can pursue the original reference for additional data.

Two categories of processes are described:

- those which have already been used in existing dams. Typical examples are anchors, grouting and diaphragm walls.

- those which have already been used in new dam construction, but which could conceivably be used in insitu remedial applications. An example would be the SMW soilcrete method, as used to safeguard the foundation soil of the replacement Jackson Lake embankment dam, Wyoming, against liquefaction.

On the other hand, certain other geotechnical processes, which can clearly improve the soil, such as vibroflotation, blasting, compaction piles, and dynamic consolidation, are not described, as it is inconceivable that they could ever be applied to the foundations of an existing dam, insitu. In addition, the well known principles of improvement by drainage (curtain and blankets) is not addressed separately(6).

Some of the techniques apply to only one particular type of dam: rock anchors, for example, can only be installed through a concrete structure whereas diaphragm walls can only be installed through embankment dams. Grouting for seepage control is common to both kinds of dam although remedial foundation grouting for concrete dams is invariably in rock, while such grouting for embankment dams usually features soil treatment methods. These same methods are used to combat foundation soil liquefaction which is a problem faced exclusively by embankment dams.

Table 1 summarizes the techniques and case histories reviewed. In certain categories, there were numerous case histories from which to chose, and the choice was influenced by the desire to

TABLE 1 -- Summary of Case Histories Reviewed

SECTION	DAM	TYPE	REMEDIATION	REFERENCE
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3.2.2.2.	Tarbela, Pakistan	Embankment w/ Concrete Structures	MPSP Rock Grouting for Seepage Control (Concrete Structures)	39
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5.2.1.	Pinopolis West, SC	Embankment	Compaction Grouting for Liquefaction Control	82, 83
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provide the most representitive and recent examples, preferably from North American practice.

It is assumed that the reader has knowledge of what these techniques actually involve: fundamental descriptions are not generally provided, except briefly for some of the more novel methods.

2. Stabilization of Concrete Structures Using Prestressed Rock Anchors.

2.1 General Aspects.

The use of prestressed rock anchors in dam engineering is as old as the technique itself: the first recorded use of anchors was to stabilize Cheurfas Dam, Algeria in 1934(7). Since then, anchoring has gained worldwide recognition and acceptance not only in connection with dams but for a multitude of other purposes (8).

The classic applications for dams are to provide additional resistance to overturning (Figure 1) and restraint to sliding(9). The trend towards raising existing dams to increase storage capacity, and the ongoing process of fundamental PMF reevaluation have generated a great increase in activities of this sort in the U.S. while a similar atmosphere now exists in Canada.
In addition, there are countless examples of anchors being used as tiedowns for spillway construction, and for straight forward rock mass stabilization for abutments, portals and excavations(10,11). Perhaps the most famous example of remediation was in the Service Spillway Plunge Pool of Tarbela Dam, Pakistan where almost 2,000 anchors of 430 tonne ultimate tendon capacity were installed to provide, in effect, a zone of "compressed rock" to further resist the tremendous dynamic forces exerted by the water during operation of the Spillway/Flipbucket structure. Most recently a new concept has been created during the repair of Stewart Mountain Dam, AZ: the anchors are designed to provide overall stability and continuity to the arch dam and the Left Thrust Block during a projected major seismic event.

There is wealth of literature available to the engineer who wishes to design and test anchor systems (10, 11, 12, 13, 14, 15, 16). Equally, there is generally a high degree of all round experience and expertise to exploit from within the ranks of the specialist contractors. Such work always involves a very high engineering content, and frequently provides unique or exceptional facets, such as those arising from difficult access restrictions, extreme construction tolerances, or tendons of great weight or length. Rock anchor practice in the U.S.A. in general compares favorably with that anywhere in the world, and indeed has some unique aspects stemming from the scale and complexity of many of the projects.

Two case histories are reviewed. The first, at Shepaug Dam, CT, is the classic application of anchors to resist overturning. At the time it featured the heaviest tendons ever installed in the U.S., having been only recently surpassed by similar work executed by the author's company at Lake Lynn Dam, Pennsylvania.

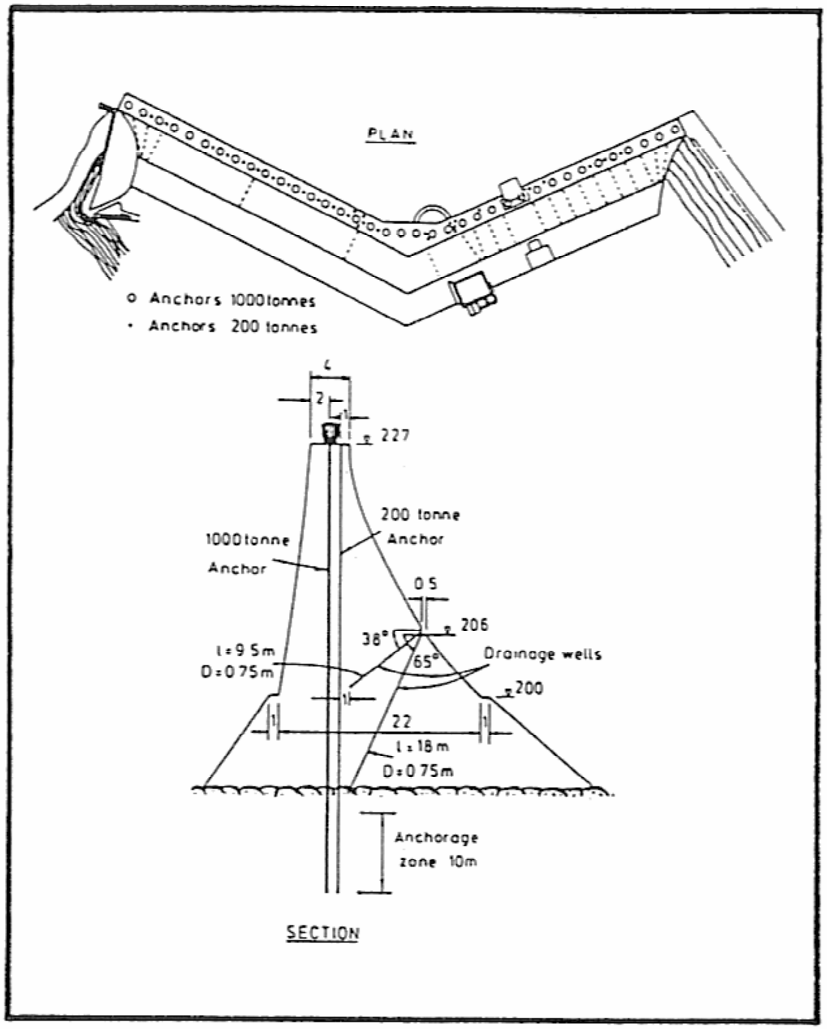


Figure 1. Anchors to resist overturning (7).

The second example is Stewart Mountain Dam. In this case, the application of the anchoring is thought to be unique, while there are several innovative and special features of the construction - not necessary applicable in most cases - which are exceptional.

2.2 Case Histories.

2.2.1. Shepaug Dam, CT: Anchors for Overturning (17).

Shepaug Dam is a 36-year-old concrete buttress/gravity dam on the Housatonic River near Sandy Hook, CT. The purpose of the 40m high, 430m long structure is flood control and power generation. The stability of the dam was reassessed, with the PMF estimated as equivalent to 6m above the present crest. High capacity prestressed rock anchors were selected to safeguard against overturning.

A total of 83 anchors was designed for installation from the 3m wide crest. The 250mm diameter bond lengths (8-15mlong) were permitted to begin 1.5m below the dam/rock interface, and assumed average rock (micaceous granite gneiss) - grout bonds of 0.7MPa at design working load. Free lengths were largely dictated by the thickness of the concrete drilled (8 - 44m). Individual design working loads of 450 - 850 tonnes were calculated, requiring tendons comprising up to 53 Nr. 15.2mm diameter seven wire strands. These crest anchors varied in inclination from vertical to 2.3° upstream and were located at 1.8-4.2m centers.

In addition, there were a further 14 anchors (inclined 49°down) installed from an elevation 24m below the crest, on the downstream face of the Spillway. These anchors, intended to resist sliding, had free lengths of 18 - 22m, bond lengths of 12 - 15m and tendons comprising up to 52 strands. The holes were spaced from 6.5 to 7m apart.

The strands in the bond lengths of all tendons were spaced and centralised to provide nodes at 3m centers.

Major features of the construction were as follows:

- Drilling was conducted with the rotary percussive down the hole hammer system: this remains the fastest, cheapest and safest way of drilling holes over 100 mm in diameter through competent materials. Each hole was overdrilled by 1m to form a "sump".

- Hole alignment was measured by an Eastman Whipstock R instrument, at top, midpoint and bottom. Deviations were less than 1°, in line with data previously recorded on Nicholson contracts.

- Water Testing was conducted on every hole to an equivalent Lugeon value of about 10. No hole failed and so no separate stage of pregrouting and redrilling was necessary. [This contrasts with common experiences with dams on horizontally bedded sediments, and in karstic limestone terrains].

- Tendons must always be placed with care, and especially in this case where they were exceptionally long, and heavy (up to 5 tonnes) with restricted access at their hole entry point. A special mechanical uncoiler was used to permit controlled placement at rates of about 6m per minute.

- Grouting was conducted with neat cement grout of w/c ~ 0.4 - 0.45 by weight, prepared in a colloidal mill mixer. The anchorage grouting was conducted to 3m above the bond length and a minimum of 5 days was specified before stressing was permitted.

- Corrosion protection to the free length was provided by individual strand greasing and sheathing.

- The design working tendon stress of 60% GUTS (Guaranteed Ultimate Tensile Strength) allowed a test overload of 1.33 on each anchor (ie. not in excess of 80% GUTS).

- Prior to multijack tendon stressing, each strand was individually stressed to Alignment Load (5% Test load) with a monojack to ensure evenness of loading and so prevent potentially dangerous overstressing of individual strands at high average test stress levels. This operation revealed that some strands needed as much as 150mm of ram extension to get them straight and holding the Alignment Load.

- The Performance Tests indicated an apparent debonding of about 2.5m in the bond zone, and the total permanent displacement was less than 12mm compared with total extensions of up to 350mm. Creep was negligible, and 7 day lift-off checks revealed minimal relaxation losses after lock off.

- In all 14 spillway anchors, and in 8 crest anchors where the free length was less than 15m, a conventional top anchorage lock off assembly featuring gripping wedges, was used. For all the other anchors, however, the load was maintained by bond between the bared upper part of the free length (about 11 - 17m long) and the secondary grout. This scheme saved the considerable cost of coring large diameter recesses in the dam crest to accommodate the conventional top anchorage hardwear. The 75 anchors of this type were referred to as Secondary Bond Anchors, and this concept is being specified more frequently in recent projects.

2.2.2 Stewart Mountain Dam, AZ: Anchors for Seismic Stability(18).

Stewart Mountain Dam is located on the Salt River 65km northeast of Phoenix, Arizona, in Maricopa County. During the 1920's, when this 60m high double-curvature, thin-arch dam was being built, the significance of effective construction joint clean-up was not fully appreciated. As a result, horizontal construction joints at 1.5m intervals have a thin layer of laitance and are very weak, exhibiting little or no cohesion. The arch section is currently not a monolithic structure, as designed, but a series of unbonded blocks held together by gravity and natural arch dam action. Individual concrete blocks would therefore be unstable high in the arch during a major seismic event because of poor construction joint bond, large inertial forces, and separation of the arch along vertical joints. Alkali-aggregate reaction has also attacked the concrete, leading to concrete expansion, surface cracking and permanent upstream crest movement of 150mm between 1930 and 1968. More recent material studies indicate that the concrete still has enough strength and stiffness to support normal loading conditions and, indeed, interior concrete shows trends of healing and gaining strength.

The Government considers the MCE (maximum credible earthquake) to be of magnitude 6.75, 15 km away, producing an estmated site acceleration of 0.34 g. This seismic event will be sufficiently major to significantly shake the structure and cause instability of the blocks high in the arch.

In order to restore the structural monolithicism and increase factors of safety during normal and seismic loadings, post-tensioned rock anchors have been judged to be the least expensive and most viable solution. A total of 62 anchors is foreseen at 2.5 - 3m. centers along the 178m long dam crest, with individual design loads of 250 - 340 tonnes. They range from 2°30' downstream to 8° 40' upstream, and from 22 - 75m long. Each will have 22 strands.

Three dimensional finite element studies show that these anchor forces have a very positive effect on the arch by providing additional normal forces across the weak horizontal construction joints and by horizontally compressing the arch (Figure 2). They also add stability by increasing the lateral support. These analyses did show, however, that arch action and lateral suport were still lost in the top 12m, at times, during the MCE, and that this zone warranted special attention.

The major implications of the site, the structure and the design on crest anchor construction are as follows:

- drilling: high accuracy drilling is essential as the dam is 2.4 - 10m thick and contains drains and other structures.
- grouting: potential grout flow paths through concrete lift joints and in the foundation rock have to be located and sealed.
- tendon assembly: to be conducted on site to optimize response to poor rock areas. Bond lengths to be confirmed by special preproduction tests.
- stressing: tendons have to be restressable to allow for various causes of load loss during a 100-day monitoring period. Anchor grouting in two phases.
- stressing sequence: special sequence specified to uniformly load the arch.

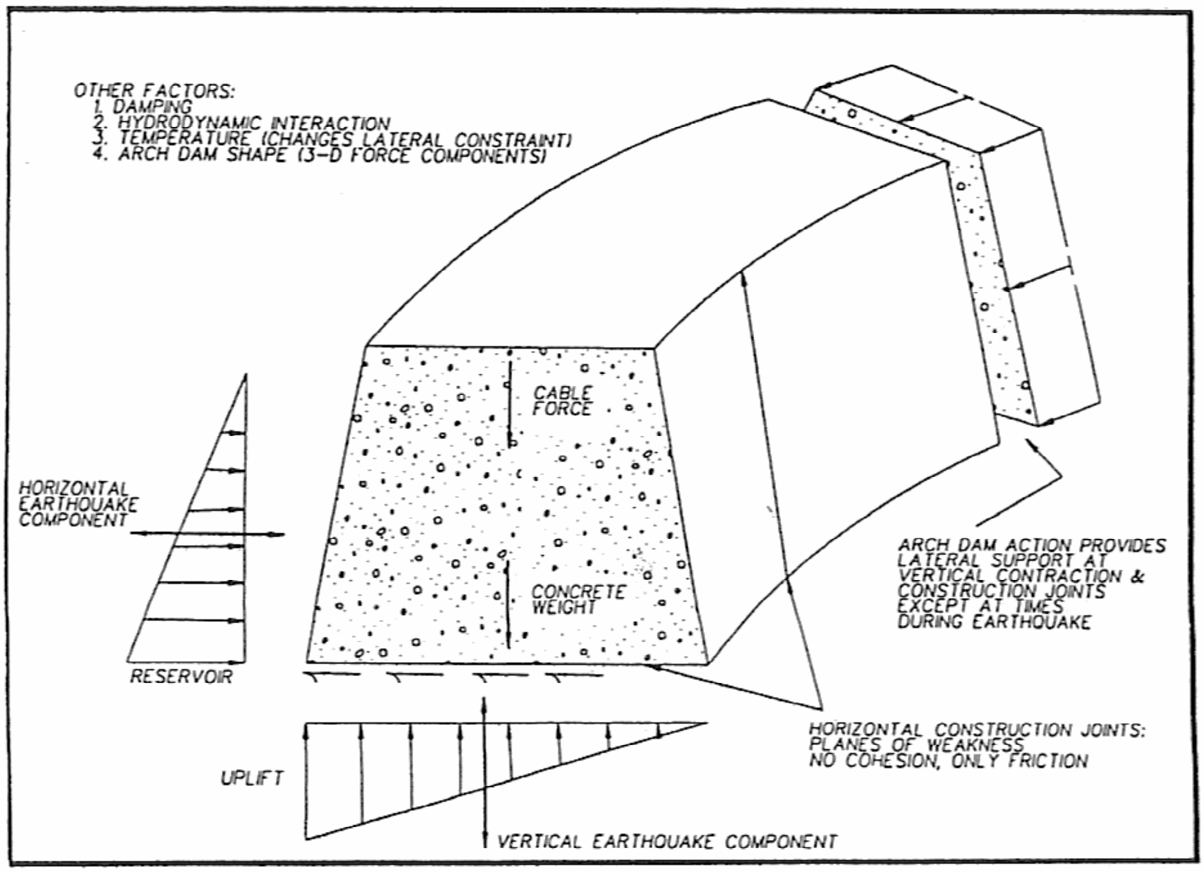


Figure 2. Force components on Arch Blocks (18).

- corrosion protection: to protect the bare strand of the free length during the load monitoring period, but to still allow it to be bonded to the upper 15m of the dam thereafter, epoxy coated strand is specified.

In addition, the analyses have shown a requirement for a further 22 anchors through the face of the Left Thrust Block to guard against sliding at, or just below, the concrete/rock interface. Each anchor will have 28 strands, and be inclined 60° below horizontal. Anchor lengths will vary from 24 - 50m. Similar contruction demands will be observed, except that there is no requirement for the 100-day monitoring period.

Work is due to commence in the summer of 1990, and be completed by the fall of 1991.

3. Seepage Control And Structural Repair By Grouting Techniques

The principles of grouting have been applied since 1802 to improve certain properties of rock and soil masses(19). The old connotation with "black magic", "smoke and mirrors" and similar illusions to engineering deficiencies and contractor exploitation has been buried in the advances this art has made in the last decades. This is reflected in the treatment of the subject in textbooks (19, 20, 21, 22), conferences (23, 24, 25) and at annual courses such as at the University of Missouri-Rolla.

Throughout the world, grouting is recognized as, and used routinely as, an engineering construction technique, and not just a last resort when all else has failed. Unfortunately, this new awareness has not dawned as positively in the United States as elsewhere: the memories of bad experiences arising from unscrupulous contractors and archaic, dictatorial contracting procedures, have still not been wholly expunged in certain, influential circles. In new construction, the efforts of Government agencies (6, 26, 27, 28) and specialty contractors have strongly readvanced the cause, and reaffirmed grouting's true potential and reputation. Major dams and tunnels throughout the country regularly incorporate grouting as an integral activity, enabling construction, and guaranteeing long term performance.

In remedial construction, however, there seems less evidence of this current confidence. For example, leakage through or under embankment dams is being addressed by the "positive" solution of diaphragm walls (Section 4) - at a significant cost premium - as opposed to contemporary grouting methods. In such cases, we hear "Well, grouting didn't work the first time!" Grouting can be made to work, if properly designed, conscientiously constructed and equibly rewarded.

This section commences by making some general statements about the particular demands of remedial grouting. Thereafter, case histories are reviewed under three categories:

1. Seepage control: Concrete Dams (Concrete and Rock Grouting)
2. Seepage control:Embankment Dams (Rock and Soil Grouting)

3. Miscellaneous applications (including void filling, consolidation grouting and slabjacking)

3.1. General Aspects of Remedial Grouting.

The loss of water through or under a dam is rarely just a question of volume, although in especially arid or marginal areas, losses may be significantly large to compromise storage, or generating capacity. More usually, the seepage is a threat in terms of the foundation uplift pressures it may cause, or the suspended dam or foundation material it may transport.

For example, foundation scour resulting from piping was blamed (29) for the 1989 failure of the St. Anthony Falls lower dam powerhouse on the Mississippi River, Minneapolis. The bedrock was a very friable and erodible Silurian sandstone: the structure had survived for 90 years before suddenly failing.

For embankment dams, seepage through the dam, at its contact, or in the immediate bedrock can be equally dangerous, especially in cases where insufficient internal filters were placed. Davidson (30) notes that "many of the most damaging dam failures in the United States have been caused by seepage induced piping", and cited Teton Dam, and Quail Creek Dam as prime examples.

At the former, the joints in the ignimbrite were sufficiently open to permit movement of the erodible loess embankment material into the foundation bedrock. Goodman (29) also cited the case of erosion of canal linings into open fractures in ignimbrite which caused two major failures of power canals in New Zealand. Further geologically based hazards involve bedrocks featuring:

- erodible seams or karsts
- soluble minerals (eg. gypsum)
- expansive minerals (eg. anhydrite)

However, it would seem that in many of the cases where remediation has been required, the cause has been the manner in which the initial grouting was conducted. Inappropriate selections with respect to drilling and grouting parameters and procedures lead to curtains which are inefficient at inception and progressively less effective with time. For example, Petrovsky (31) discussed the leaching of cementitious compounds from grout curtains as a function of the water-cement ratios, while Houlsby (32) also addressed the problem. This question of grout permanence is even more acute in the case of curtains in alluvials, formed with the earlier silicate based chemicals alone. Fortunately, this is not one of the problems facing the U.S. community, as such technologies were not routinely applied in U.S. dams.

Another key factor in trying to account for the need for remedial grouting was touched upon by Benzekri and Marchand (33):

> "There is no precise and accurate way of checking how effective the drainage and grouting will be prior to filling the reservoir. Water tests in boreholes are useful in guiding the work as it proceeds and give an overall indication of how much permeability has been reduced; however, the results must not necessarily be

taken at their face value. The only conclusive test is to fill the reservoir and observe seepage pressures and flow rates during, and for some time after, this period. A large number of piezometers is required. The foundation must be divided into zones for flow measurements so that any concentrated seepage paths can be traced. Any unacceptable subsurface conditions can then be treated provided that a grouting plant is held in readiness during reservoir filling and that convenient access has been provided beforehand to critical points."

Weaver (34) used this as a strong argument in favor of incorporating grouting galleries in dams and abutments, especially for higher structures on poorer foundations, built with less than optimum local materials. The classic example cited by Goodman (29) in this regard was Hoover (Boulder) Dam, a 222m high arch-gravity dam founded on imperfectly cemented volcanic breccia. The original grout curtain was extended from 40m deep to 130m deep in the valley bottom and 90m into the abutments after initial very high seepage volumes and pressures (Figure 3) were detected after first filling over 50 years ago. As a final introductory point, Davidson (30) reviewed five case histories of remedial grouting under embankment dams. He drew the following lessons which merit consideration as the details of the following projects are reviewed:

- grouting may be successful in reducing seepage volumes but may still not significantly reduce piezometric levels, especially in soils or rocks with open but ungroutable pores or fissures.
- grouting may be only a temporary solution if the real cause is solution of a soluble horizon.
- attacking the seepage problem upstream with a clay blanket may be more cost effective.
- although state of practice methods and materials may have been originally used, they may now be judged as ineffective and inappropriate, in the light of current knowledge.
- the extent of knowledge of the foundation and embankment materials, and the construction history will greatly influence the accuracy of the analysis of the seepage problem.

In each of the categories of case histories presented below, there is a general progression from the simple to the more complex.

3.2. Concrete Dams.

3.2.1. Grouting of the Structure.

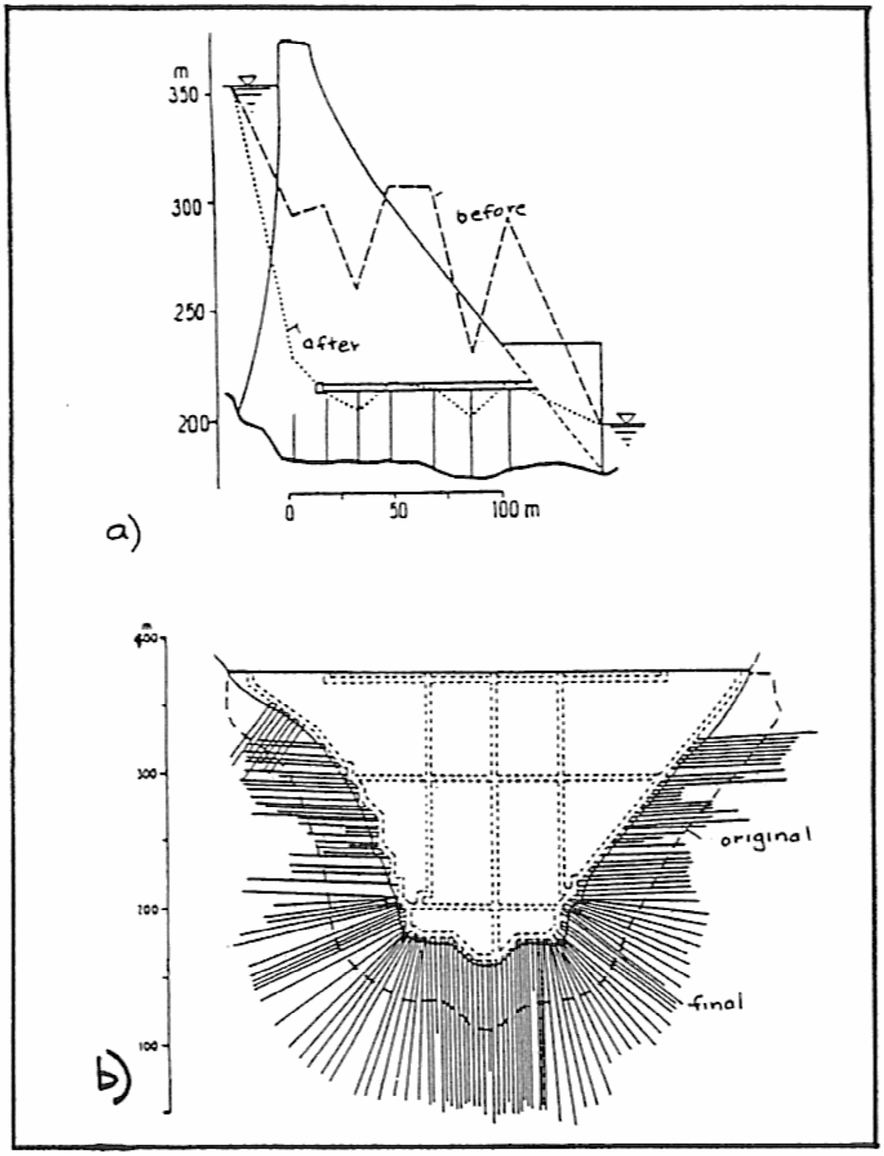


Figure 3. a) Uplift pressure caused by underseepage at Boulder Dam (Hoover Dam) before and after remedial grouting; b) holes drilled for the remedial grout curtain and the limits of the original curtain (dashed line) (29).

3.2.1.1. Aswan Dam, Egypt(22).

This 1982m long masonry dam was built to regulate the R. Nile in 1900. Originally 34m high, it was raised to 39m, and later to 48m (Figure 4). It consists of granite blocks and flagstones set in cement mortar, of low chemical resistance to the soft river water. Seepage through the structure increased progressively over the years, leading to leaching of the mortar and deposition of calcum carbonate on the downstream face.

The remedial work consisted of an internal grout curtain, installed in 1960-61, and featuring:

- 2000m of exploration drilling, with piezometers along the axis
- 44,000m of grout holes, treated with cementitious grouts containing slag.
- 100m of final permeability check holes.

Initial tests showed the overall permeability to be 2.5-9 Lugeons, with flow occuring at the granite/mortar contacts.

Water cement ratios varied from 3:1 to 0.6:1 and were injected at pressures of 1.5 - 2.0MPa in the rock, 1.0 - 1.5MPa in the masonry (to 15m from the crest), and 0.3 - 0.5MPa above. Grouting of each ascending 3m stage was conducted to leakage or a maximum take of 500 kg/m. Split spacing reduced Primary spacings of 7.0m to final Tertiary spacings of 1.75m. Primary grout takes averaging 92 kg/m reduced to 58 kg/m in the Secondaries and 36 kg/m in the Tertiaries.

The effectiveness was demonstrated visually by the drying up of downstream seepage. Piezometer data confirmed that basal uplift pressures had been reduced substantially.

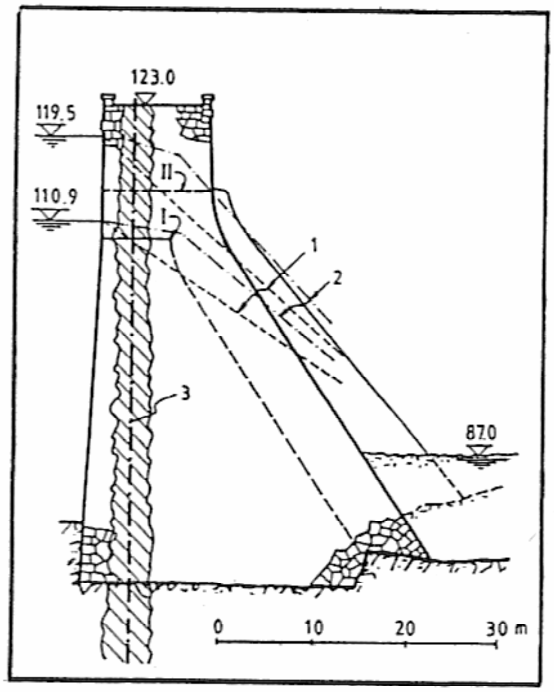
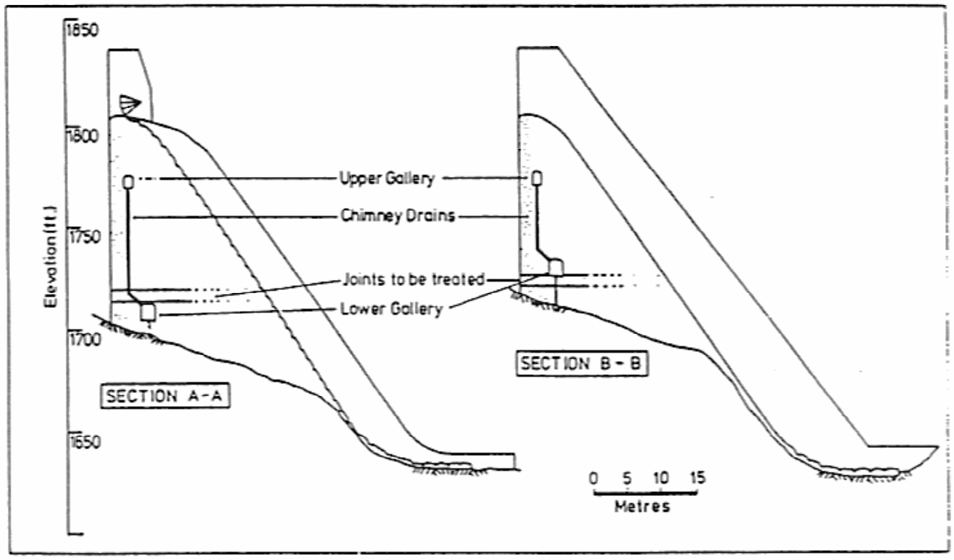


Figure 4. Typical cross section of Aswan Dam and the position of the grout curtain, I and II dam construction phases, 1 linear distribution of uplift pressure in the foundation plane, 2 uplift pressure measured in piezometers, 3 center line of grout curtain (22).

3.2.1.2. Santeetlah Dam, NC(35, 36).

The dam is a 320m long concrete arch structure with gravity abutments and two spillways. It stands a maximum of 65m above the bed of the Cheoah River in western North Carolina and serves to impound water for generation at another installation. From its first impounding in 1928, there appear to have been concerns about aspects of the dam's performance - especially seepage and movement - and the spillways and West Abutment were reinforced with additional concrete in 1938 and 1950.



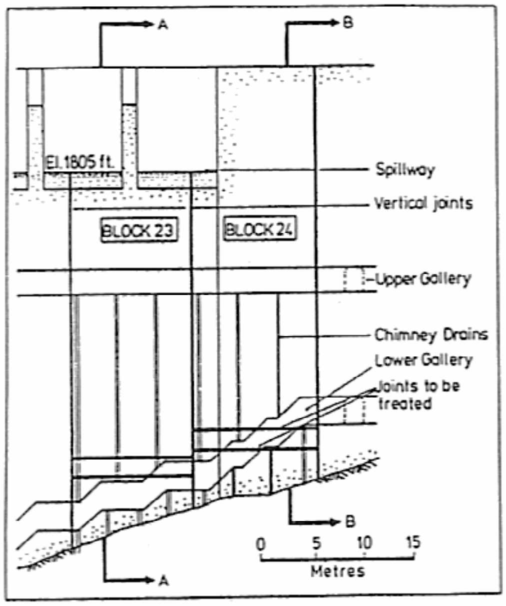


Figure 5. Galleries, and horizontal joints for treatment, Santeetlah Dam (35).

The behavior proved to be cyclic, reflecting reservoir levels, but by mid 1987 the seepage had reached record levels and was occuring over larger areas and at higher pressures than ever before. For example, almost $4m^3$/min was recorded flowing into the Lower Gallery (3m above the contact) through old lift joints, vertical chimney drains, and secondary longitudinal fissures in the roof of the Gallery (Figure 5). Of particular concern was the observation that seepage was occuring into the Gallery from its downstream face. It was feared that the flow of the very pure lake water - which had already caused massive carbonate leaching from the concrete - would gradually increase the fissure aperture, further reduce the frictional characteristics across the joints, and so further reduce the overall stability of the structure as a whole.

Review of the original construction records and locations of maximum flows and pressures highlighted two blocks in particular needing immediate treatment. Each block was about 12m long and contained two especially suspect horizontal lift joints, 1.5m vertically apart. The Consultants specified the purpose of the treatment - namely to stop the seepage and rebond the blocks together - and specified the generic properties of the grout to be used:

- to ensure maximum penetration, the grout had to be a true liquid and not a suspension of solids
- the grout had to be immiscible in water
- the grout had to have a short hardening/polymerization time, to minimize washout
- the grout had to have an almost constant viscosity till setting
- the grout had to have minimal shrinkage
- the grout had to have high sheer strength and adhesion, but low elastic modulous
- the grout had to be durable
- the grout had to be chemically stable and non-toxic during preparation and in service.

In addition, the grout was to be placed as close to the upstream face as possible, and at full summer pool when the fissures would be opened to their widest extent.

Nicholson Construction won the design-build contract in early 1988 using the RODUR^{sm} system of structural repair, with a program as follows:

- Core Primary holes upstream from the Lower Gallery (approximately 2.1m wide x 2.5m high) to intersect each fissure in a regular pattern close to the lake. The special electro-hydraulic drill rig provided holes of 46mm diameter and cores of 36mm diameter. (Figure 6)
- Observe water flows and establish flow paths, pressures, interconnections, etc.
- Inject Primary holes with relatively viscous RODUR^{sm} resin
- Core intermediate Secondary holes in a regular pattern with specific "problem areas" given additional holes.
- Grout Secondary holes with thinner resin
- Core a limited number of Tertiary holes, to demonstrate uniformity and effectiveness of treatment, and to act as long-term drainage holes.

Table 2 summarizes the drilling conducted, while Table 3 summarizes the grouting quantities. These data confirmed that the concrete material itself was in good condition and that all the water flow was in the discontinuities, both preexisting and Secondary. Water flows which had peaked at almost 2m^3/min from these four fissures (often at full hydrostatic pressure) had been almost completely eliminated after the Secondary grout treatment. Complete filling of the four major lift joints was confirmed, while excellent penetration of the secondary microfissures by the viscous, but low surface tension, RODUR^{sm} grouts was noted throughout both blocks.

A further observation made during the Primary grouting was the cessation of the several seepages from the downstream side of the Gallery. Thereafter, the walls of the Gallery, on both sides, hitherto completely saturated, began to dry out.

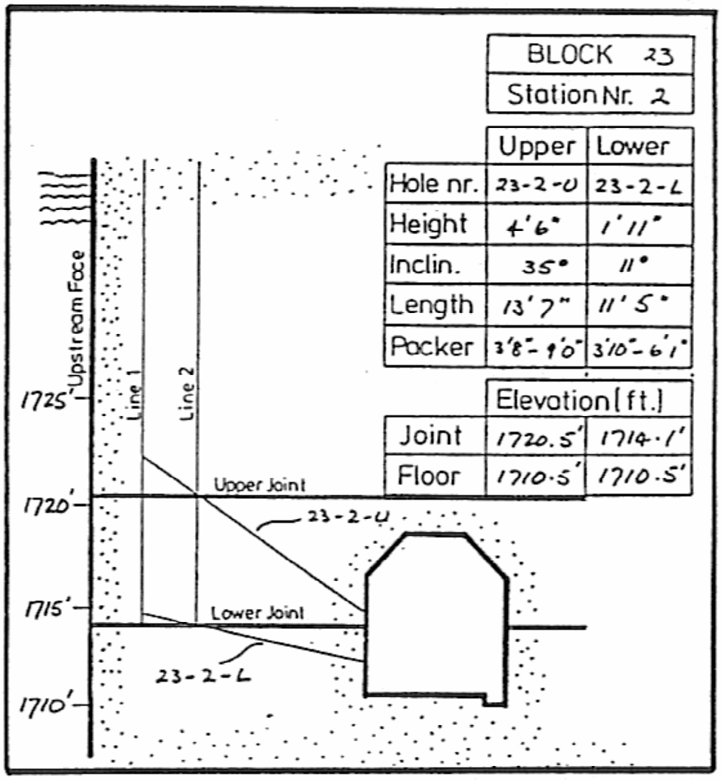


Figure 6. Typical section showing intended joint interceptions at one station, Santeetlah Dam (35).

Order Drilled and Grouted	Block and Phase	Holes (Nr.)	Total Drilling (m)	Inclination
1	23 Primary	18	77	9-50° up
3	23 Secondary	11	42	10-56° up
5	23 Tertiary	2	9	40.5° up
2	24 Primary	18	71	39° down to 45° up
4	24 Secondary	12	46	48.5° down to 49° up
6	24 Tertiary	2	9	33.5-46° up
Total		63	254	

TABLE 2 -- Hole drilling summary, Santeetlah Dam, NC (In addition, two drain holes were drilled downstream, in each block, after grouting: all four were dry.)

Joint	Summary Characteristics (Generated)	Total Resin (kg)
23 Upper	Prim. 3mm irregular	713
	Sec. 1-2mm irregular	508
23 Lower	Prim. 1-6mm irregular	752
	Sec. 1-2mm regular	736
24 Upper	Prim. 4-5mm regular	1803
	Sec. 1-2mm regular	663
24 Lower	Prim. 4-10mm irregular	1430
	Sec. 1-2mm regular	564
Note	Prim. - Primary grout thickness	
	Sec. - Secondary grout thickness	

TABLE 3 -- Joint apertures, as evidenced by thicknesses of resin infill found, Santeetlah Dam, NC

It is noteworthy that the RODUR^{sm} concept of epoxy resin bonding has been used on major dams throughout the world during the last decade with conspicuous success. Its success owes as much to the meticulous way in which each project is individually analyzed and planned, as it does to the remarkable properties of the RODUR^{sm} series of epoxy resins.

When repairing structures by grouting, there is typically only one chance at success - an abortive attempt with inappropriate materials will seriously compromise the chances of a later attempt proving successful. This realization had often been a key factor in the decision to rely on the RODUR^{sm} method.

3.2.2. Grouting of the Foundation Rock.

It would seem that case histories dealing with remedial rock grouting for seepage control under concrete dams are not so common as those describing similar treatments under embankments. If this is in fact a true reflection of the relative levels of activity, possible explanations could include:

- grout curtains for concrete dams are usually an integral part of the design, and the only primary defence against seepage. They are therefore executed intensively to the highest engineering standards.
- Excessive uplift pressures can be alleviated by drainage curtains drilled from galleries within the dam. This is not so easily conducted under embankments.
- Site selection and preparation initially tend to be more critical to accomodate the higher stresses imposed by concrete structures.

- Seepage under or around a concrete dam may be more easy to tolerate (assuming uplift is not a problem), as there is no danger of piping fines from the core or contact.

In any event, case histories from American practice are rare, and if the work on Boulder Dam is excluded, appear to concentrate on the problem of seepage induced in karstic limestone terrains (37). Examples from overseas are therefore also reviewed to illustrate current practice and expertise. For a comprehensive state of practice review of rock grouting technology, the reader is referred to the recent book by Houlsby (19).

3.2.2.1 Tarbela Dam, Pakistan: Conventional Descending Stage Grouting (38).

One of the major concrete structures at the Tarbela Dam Complex is the Auxiliary Spillway (Figure 7). It is founded on a complex sequence of weathered dolomitic limestone, and limestone, interbedded with phyllite and beds of cohesionless marly silt. It is folded, faulted and fissured, and contains some small karstic features. Soft erodible material exists in some faults and karsts.

The Spillway was originally protected by an upstream triple row grout curtain, a drainage system and a connecting surface blanket. Over the years, some movement of fines into the drainage system was noted, and as a first step, was rectified by replacing the original drain liners with special filtered liners at 3m centers along the line of the drainage curtain. Thereafter, the systematic grouting of "preferred paths" and voids in the rock mass upstream of these new drains was planned.

To effectively seal these long water passage ways of relatively small cross section, the required grout had to be fluid, but stable during and after injection. On grounds of bleed, pumpability, stability and set strength (14MPa at 28 days) the following mix was selected:

$$w/c = 1.0 \text{ (by weight)}$$
$$+ \text{ Bentonite} = 1.5\% \text{ weight of cement}$$

During injection, the mix design was kept constant, both to encourage flow and help site quality control.

Classical descending stage methods were used due to the poor mechanical quality of the rock mass especially in the upper reaches. The average hole depth was 79m, and the maximum 117m. Rotary drilling was used simply because percussion equipment was not available on site which had this depth capacity. Stage lengths were:

Up to 6m from 0 - 31m deep
9m from 31 - 69m deep
and 12m below 69m.

A maximum pump pressure of 0.02 MPa/m was exerted except for a maximum of 0.1 MPa in Stage 1 and 0.2 MPa in Stage 2. Grouts were mixed in colloidal mixers and pumped by fluctuating pressure pumps. Refusal was defined as a take of less than 20 litres in 10 minutes. Redrilling of each successive stage was conducted

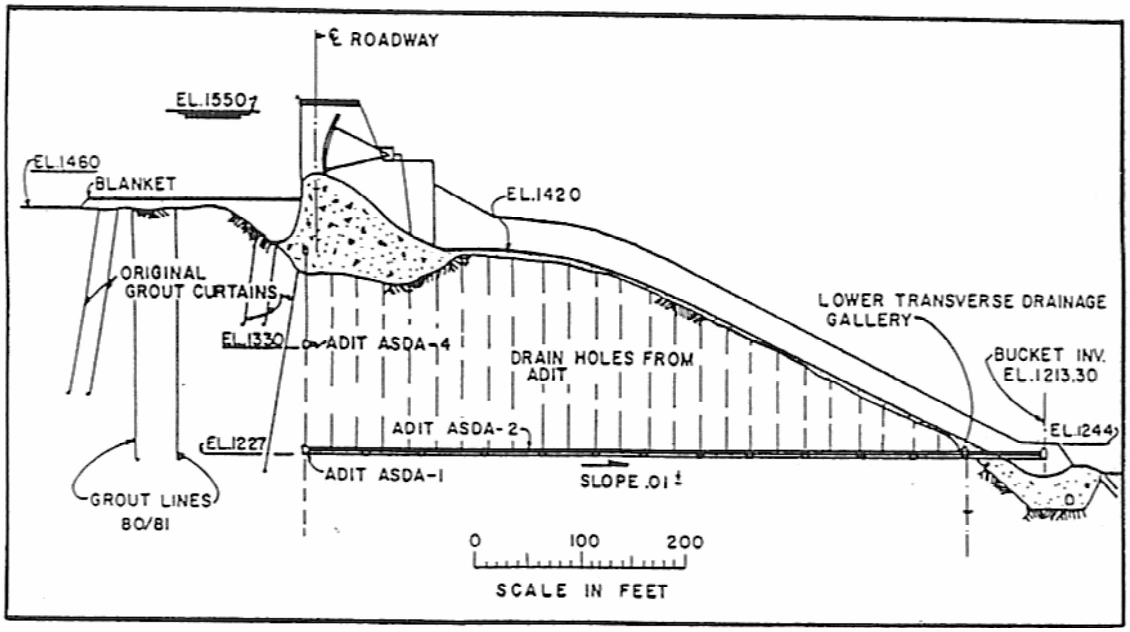


Figure 7. Auxiliary Spillway Section, Tarbela Dam (38).

3 - 6 hours after grouting, as confirmed by gelimeter tests. The original row of holes featured Primaries at 12m centers, finally closed by Quaternaries to 1.5m.

Maximum stage takes reached $6.4m^3/m$ but excellent reduction ratios were achieved (Figure 8). Grout was verified as travelling considerable distances in places by its entry into

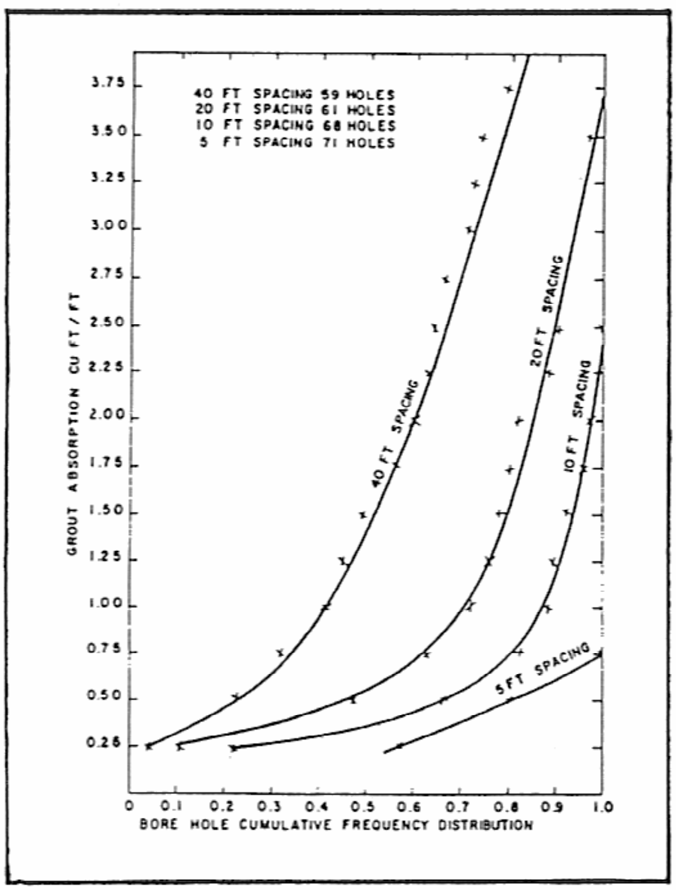


Figure 8. Effect of split spacings on grout takes, Tarbela Dam (38).

some old, unfiltered drains. Some later holes were commenced 31m downstream of the first, and takes were of the same order as in the first row, indicating no en masse grout travel. This remedial work has since proved effective in controlling seepage and eliminating migration of fines.

<u>3.2.2.2 Tarbela Dam, Pakistan: MPSP Grouting</u> (39).

In conventional rock mass grouting, the most expedient method is often the ascending stage (or upstage) method whereby the hole is drilled to full depth in one pass and then grouted in successive ascending stages via a down-the-hole packer. This method clearly depends for its success on the ability of the grout hole to remain open and stable during injection. When it is anticipated that rock mass instability will be a problem, as in the case of the Tarbela Auxiliary Spillway, the work is conducted by the descending stage (downstage) method. Here, the hole is advanced in discrete drill stages, each of which is grouted - and hopefully consolidated - in sequence from the surface to full depth (40, 41).

However, there are occasions when even this approach cannot be pursued successfully as even the shortest stages will collapse after withdrawal of the drill rods and commencement of the grouting. To combat these conditions, the MPSP* (Multiple Packer Sleeved Pipe) system was developed.

MPSP owes much to the principle of the tube à manchette system, in that grouting of the surrounding rock is effected through the ports of a plastic or steel grout tube placed in a predrilled hole. However, unlike tube à manchette, no sleeve or annulus grout is used. Instead, the grouting tube is retained and centralized in each borehole by collars - fabric bags inflated in situ with cement grout. These collars are positioned along the length of each pipe, either at regular intervals (say 3 to 6m) to isolate standard "stages", or at intermediate or closer centers to ensure intensive treatment of special or particular zones. The system permits the use of all grout types, depending on the characteristics of the rock mass and the purpose of the ground treatment.

The typical construction sequence is as follows (<u>Figure 9</u>):

<u>Step 1</u> - The borehole is drilled by fastest available method (usually rotary percussive) with water flush to full depth. Temporary casing may be necessary to full depth also, as dictated by the degree of instability of the rock mass. Typically borehole diameters are 100 - 150mm.

<u>Step 2</u> - The MPSP is installed. Pipe details can be varied with requirements, but a typical choice consists of a steel pipe, 50mm o.d., with each length screwed and socketted. Each 5m pipe has three 80mm long, 4mm thick rubber sleeves equally spaced along the length, protecting groups of 4mm diameter holes drilled in the pipe. A concentric

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* The MPSP System was developed by the Rodio Group of Companies and is conducted under license in North America by Nicholson Construction

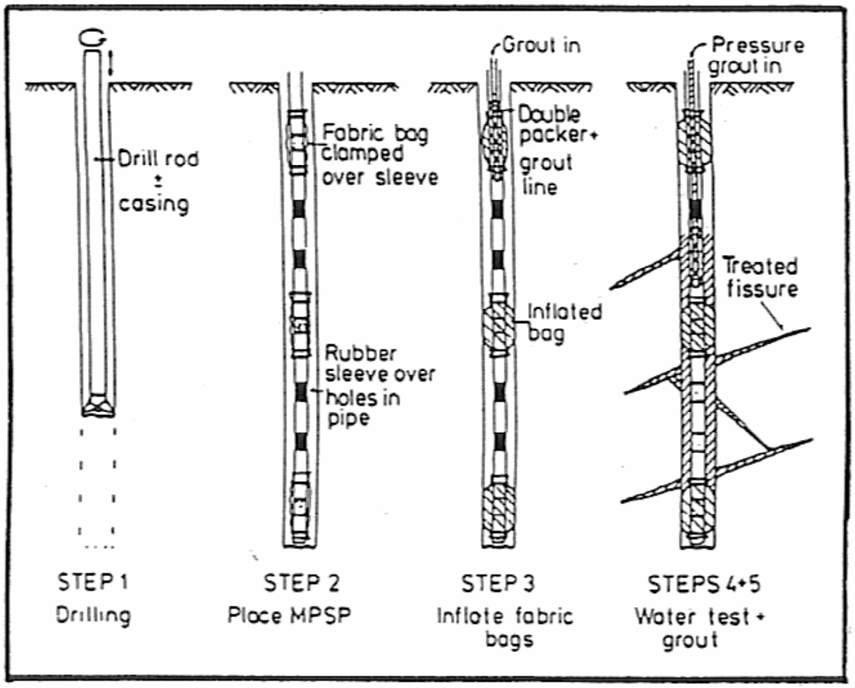


Figure 9. Installation sequence of MPSP (39).

polypropylene fabric bag is sealed by clips above and below the uppermost sleeve in each length and is typically 400 to 600mm long. For short drill holes, plastic pipes of smaller diameter may be used. The temporary drill casing is then extracted, and any collapsing material simply falls against the outside wall of the MPSP tube.

Step 3 - Starting from the lowermost pipe length, each fabric bag is inflated via a double packer positioned at the sleeved port covered by the bag. A neat cement grout is used at excess pressures of up to 0.2MPa, to ensure intimate contact with the borehole wall. The material of the bag permits seepage of water out of the grout, thus promoting high early strength and no possibility of shrinkage. Clearly the choice of the bag material is crucial to the efficient operation of the system: the fabric must have strength, a certain elasticity, and a carefully prescribed permeability.

Step 4 - Water testing may be conducted if required, through either of the two "free" sleeves per length, again through a double packer. Tests show that a properly seated fabric collar will permit effective "stage" water testing at up to 0.4MPa.

Step 5 - Grouting is executed in standard tube à manchette fashion from bottom up via the double packer (usually of the inflatable type). The grouting parameters are chosen to respect target volumes (to prevent potentially wasteful long-distance travel of the grout) and/or target pressures (to prevent potentially dangerous structural upheavals).

The following additional points are especially noteworthy regarding the MPSP System. Firstly, it is clear that, if a hole has been grouted once, it generally cannot be regrouted: some of the pressure grout will remain in the annulus outside the tube and so form a strong "sleeve grout" preventing the opening of sleeves in contact unless a very weak mix was used. (The system does, however, allow different stages in the same hole to be treated at different times.) Thus the MPSP system adopts the principles of stage grouting where "split spacing" methods are used: the intermediate Secondary holes both demonstrate the effectiveness of the Primaries and intensify the treatment by intersecting incompletely grouted zones. Analyses of water test records, grout injection parameters, "reduction ratios" and so on will dictate the need for further intermediate grouting phases.

Secondly, in addition to the technical advantages of the system, there are significant logistical and work scheduling attractions. For example, the drilling and installation work can proceed regularly at well-known rates of production, without requiring an integrated effort from the grouting crews (as in downstage grouting). In addition, the "secure" nature of the grout tube prevents the possibility of stuck packers, which is an unpleasant but unavoidable fact of life in upstage grouting in boreholes in most rock types. Grouting progress is therefore also more predictable and smoother, to the operational, technical and financial advantage of all parties concerned.

A third point relates to the straightness of the borehole and thus the integrity and continuity of the ground treatment. The temporary drill casings used in the hole drilling operations (Step 1) are typically thick-walled and robust. They therefore promote hole straightness, whereas the uncased boreholes common in stage grouting in rock, and drilled by relatively flexible small-diameter rods, are known to deviate substantially, especially in cases where fissures and/or softish zones in the rock mass are unfavorably located or oriented. By way of illustration, at Metramo Dam, Italy, the maximum deviation recorded in a test block of 150 holes each 120m long was 1.5%, with the great majority being less than 1%.

Referring to the example of Tarbela Dam, in early 1983, an intensive chemical grouting program was carried out in the Right Abutment of the Main Embankment Dam. The purpose was to verify a practical method to reduce seepage mainly through the notorious "sugary limestone" present between Right Grouting Adit 4 (RGA-4) and Tunnel 1, a zone otherwise comprising fissured limestone with phyllitic and carbonaceous schist interbeds. Two test panels were selected, as shown in Figure 10. Four different grout mixes were considered (based on hydrocycloned bentonite, cement-bentonite, sodium silicate, and resin, respectively) while all but six of the holes were equipped with plastic MPSP pipes. The other six holes (Panel 2, downstream row) were formed by the downstage method and used cement-bentonite injection, as a comparison only. Frequent major caveins of holes attempted in this way had already confirmed the unsuitability of that method in these ground conditions.

The grouting station featured mixing plants located above the entrance to RGA-4, and pumping stations set up near the grout holes in the Adit. The mixing plant incorporated electronic volumetric batching, while the injection plant provided electronic pressure, volume and flow rate monitoring in real time, and hard copy records for later use.

The treatment was undertaken in strict sequence. The injection of the outer rows with hydrocycloned bentonite and sodium silicate mixes (Panels 1 and 2, respectively) was followed by sodium silicate and resin grouting in the middle row. Analysis of the grouting records indicated generally low takes of cement-based grouts, inconsistent with the high permeabilities, but consistent with the microfissured nature of the rock mass. Equally, the analyses indicated that these outer rows had limited the travel of the (expensive) chemical grouts of the center row. Of special interest was the very low takes in the six downstage holes (25 out of 34 stages, each typically 5m long, consumed less than 15 liters of grout) compared with adjacent holes in the same row injected via MPSP (19 out of 41 stages less than 15 litres, but many stages readily accepting full target volume).

By the end of the treatment, pregrouting Lugeon values of 4 to over 100 (typically 20 - 70) had been uniformly reduced to 0 - 9 (typically 1 - 5). In addition, hole stability was markedly improved, and out flow of artesian water was greatly diminished in those test hole drilled after grouting.

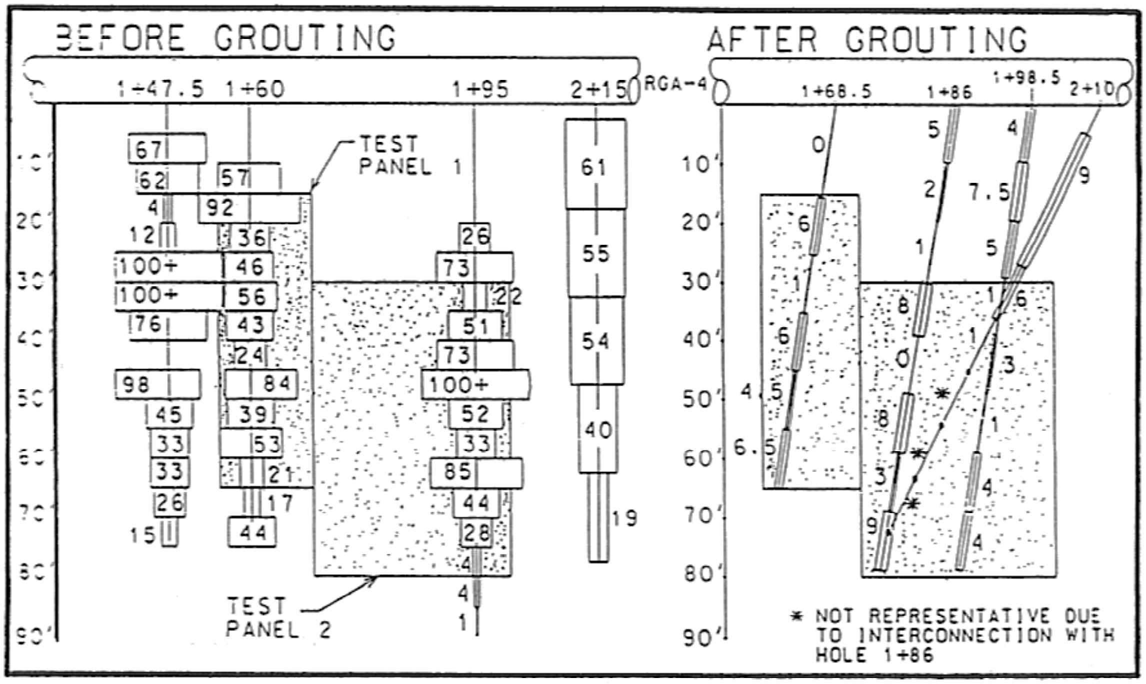


Figure 10. Location of test panels under RGA-4, and results of Lugeon tests, Tarbela Dam (39).

3.2.2.3 El Cajon Dam, Honduras: Grouting from Galleries (22).

This recently completed 232m high double curvature arch dam is founded on very karstic Cretaceous Limestones covered by Tertiary and Quaterary lavas. At an early stage in the works, it was discovered that the intensity of the karsticity did not decrease with depth and so a conventional vertical grouted cut-off would be ineffective as it would have no tight stratum to "toe" into. The concept was therefore changed from curtain to "bath tub": a grouted basin to the dam and its lower reservoir, extending upstream to a convenient vertical impermeable zone striking across the valley (Figure 11). The drilling and grouting was conducted from over 11.5km of galleries mined into the abutments and under the valley bottom.

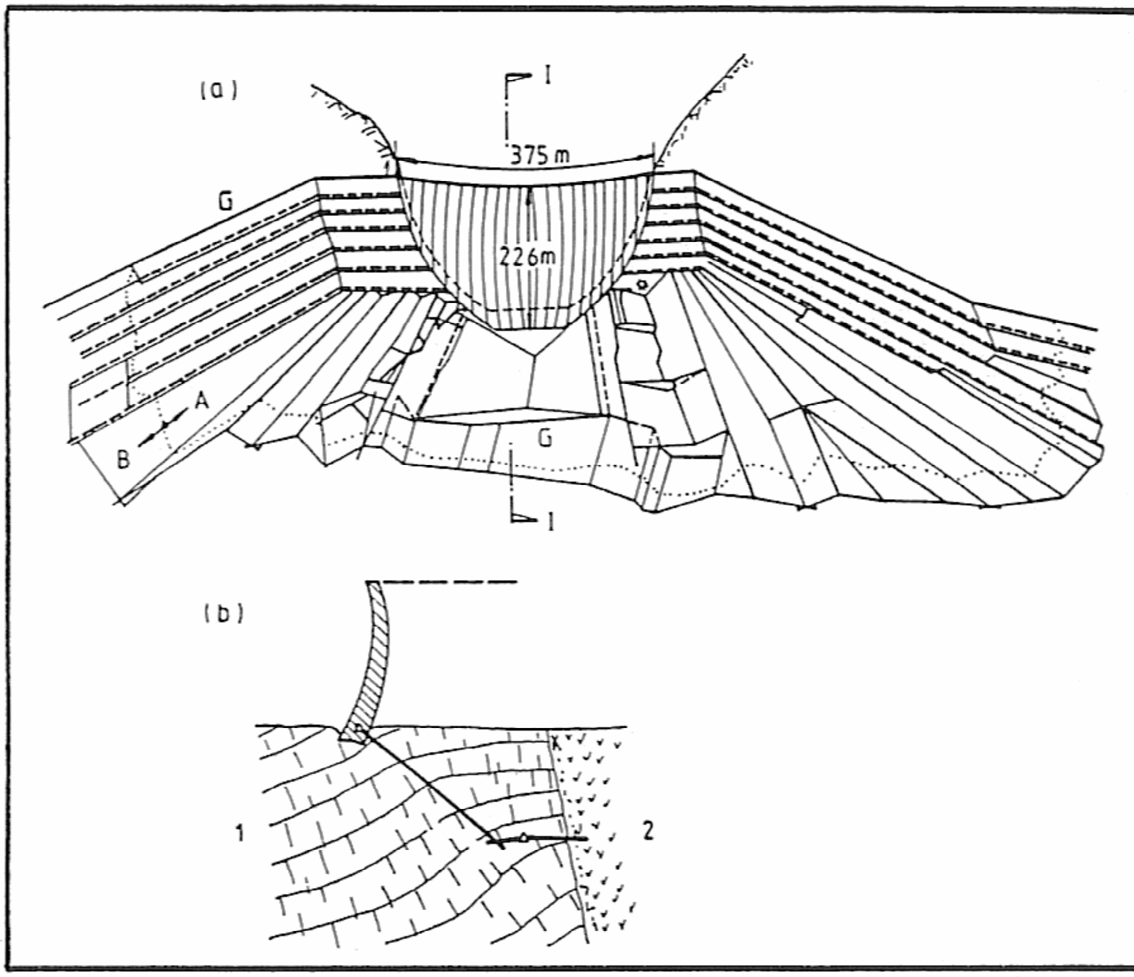


Figure 11. View from upstream on the grout curtain of El Cajon Dam, A zone of limestones, B zone of volcanites, (b) sketch of the cross sectionI-I, 1 limestones, 2 volcanic rock.

Over 100,000 tonnes of cement were injected by stage grouting methods into 485,000m of grout holes, to form the 530,000m^2 of bath tub curtain. Regarding mix design, stability and strength (to resist the massive hydrostatic pressures) were critical factors. The standard mixes (by weight) were:

Cement	Bentonite	Water	Sand	
1	0.02	0.6	-	(Normal Conditions)
1	0.02	1.0	0.15	(Large Voids)

Pressures of up to 5MPa were used on the 5m stages, with a refusal criterion of 50kg/m.

3.2.2.4 Great Falls, and Tims Ford Dams, TN: TVA Practice in Asphalt Grouting (37).

Twenty-one of the Tennessee Valley Authority's thirty dams are built on carbonate formations, in which karstic features are common. Every effort is taken during construction to hopefully eliminate the impacts of such features immediately under the dam. However, "rim treatment" is now conducted after reservoir filling once the actual leakage paths are determined.

At Great Falls Dam, constructed in 1914, seepages occurred through horizontal and vertical cavities, and were treated in 1940 with cement and asphalt injection. In general, asphalt was injected through holes close to the outcrop of the water bearing stratum, with cement grout a short distance upstream. Totals of $4432m^3$ of asphalt, and $7064m^3$ of cement grout were used to seal 96 specific leakages, some being waterfalls up to 18m high as far as 1.6km downstream of the dam.

Tims Ford Dam was completed in 1970. At first maximum pool, six points of leakage on the left abutment and left reservoir rim totalled over $30m^3$/min. On the right rim two seeps totalled over $5m^3$/min. All flows were stopped by the cement based grout/hot asphalt techniques.

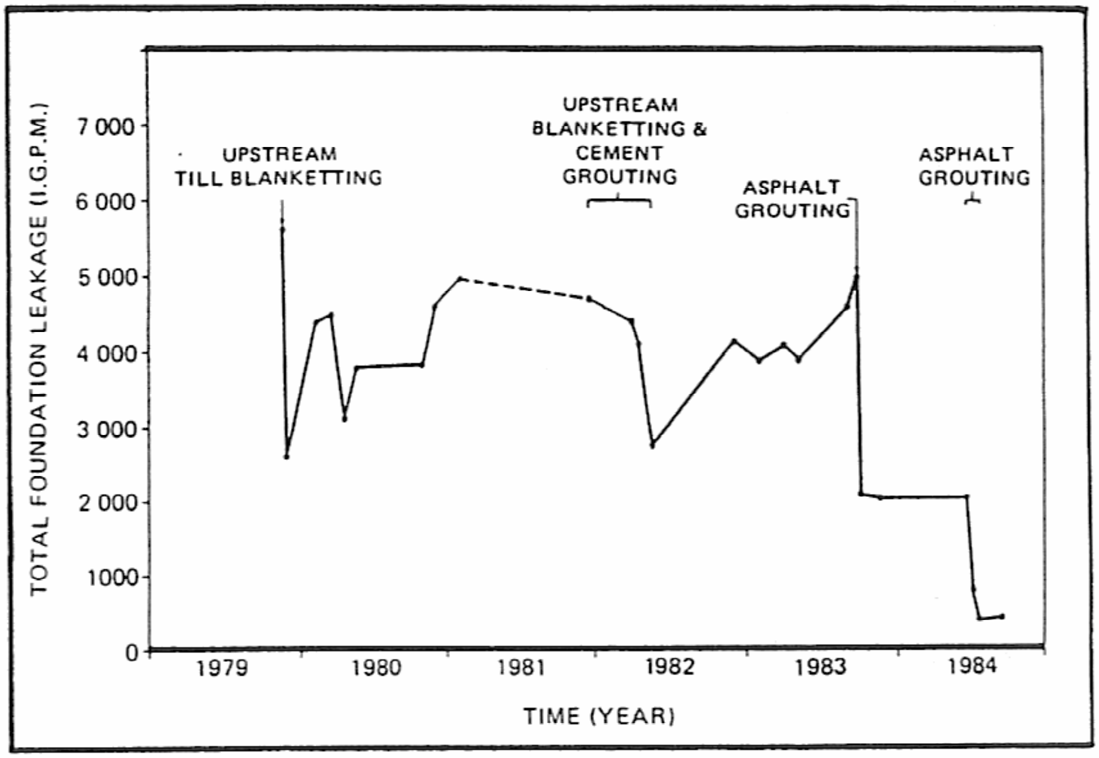
3.2.2.5 Stewartville Dam, Ontario: Further Developments with Hot Asphalt (43).

Stewartville Generation Station was built in 1948 on the Madawaska River, Eastern Ontario. The 63m high 1248m long main dam is founded on generally massive competent crystalline bedrock which contains zones of weathered decomposed micaceous limestone along some bedding and joint planes. Initial foundation preparation was insufficient to treat all the zones, which proved susceptible to erosion by moving water. As a result, several major phases of cement grouting had to be undertaken during the first 20 years after impounding to combat seepage.

By 1974, despite periodic upstream blanketing, the flow into the inspection tunnel had exceeded $4.5m^3$/min. Further testing identified two major areas of leakage and additional conventional cement grouting was conducted, but without success (Figure 12), and flows increased. A program was established to determine the fissure geometry, and seepage velocity, and to investigate alternative grouting materials. Core holes, borehole T.V. and tracers helped pinpoint the conditions in the two zones:

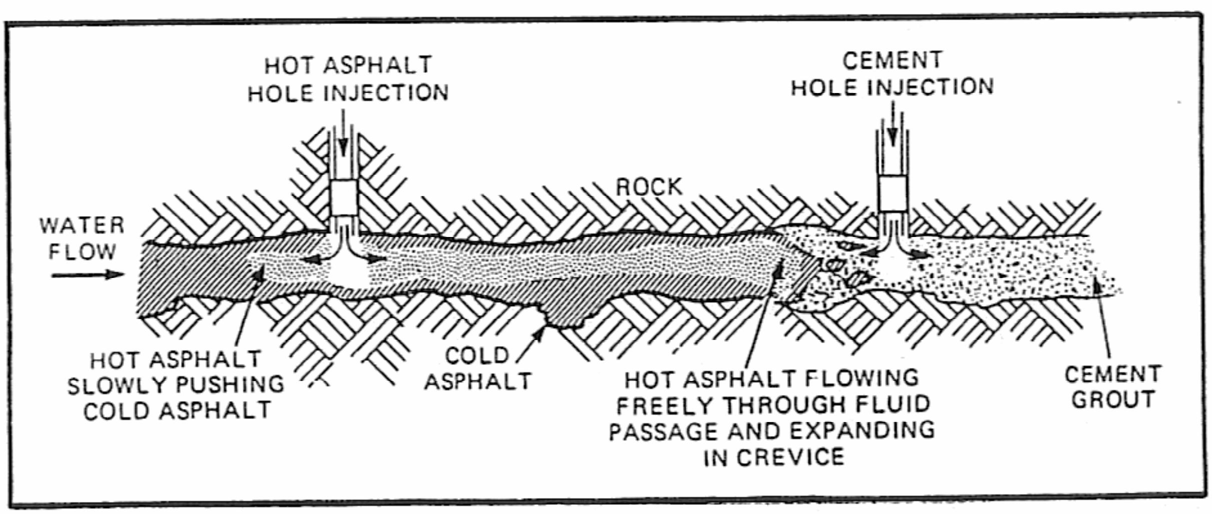
- South: bedding plane in the upper bedrock, 3m wide with 200mm aperture
- North: along concrete - bedrock contact and through the fractured rock surface, 5m wide with apertures of over 100mm.

The water was flowing at high velocity at a temperature of about 10°C. The lateral distance between the practical grout injection points and the foundation underdrain - which had to be kept open - was barely 8m. Grouting was still judged to be the best remedial option, assuming a material could provide fast set, high viscosity and resistance to erosion. Both cement based and urethane grouts were rejected for failure to meet these criteria and operational practicalities. Again, hot asphalt, injected with cement based grout was found to have the best potential for localized sealing, using the concept illustrated in Figure 13.



<u>Figure 12.</u> Effect of asphaltic grout treatment on leakage, Stewartville Dam (43).

Basically the asphalt provides an initial plug to reduce water flow. The sand-cement grout, following several minutes later, then provides the permanent seal by permeating smaller fissures and stiffening the asphalt mass against long-term creep under the high hydrostatic pressures. The asphalt was injected at temperatures of over 110°C, at rates of over 18 litres/min and back pressures of about 0.3MPa.



<u>Figure 13.</u> Schematic view of asphalt/ cement grouting (43).

By the end of the only day's treatment, under full reservoir head, at the southern zone, flows were reduced to an acceptable $1.3m^3/min$ by injecting $7m^3$ of asphalt. A later similar treatment of the northern zone with $4m^3$, virtually eliminated the preexisiting $9m^3/min$ seepage. The quantities of cement grout used totalled 15% of all the previous phases, lasting 2 months. Subsequent exploration holes revealed good bond between the asphalt - cement - rock system, and the sealing is regarded as a permanent solution.

3.3 Embankment Dams

In this section, case histories are reviewed to support the potential of modern grouting as a reputable and reliable remedial technique. However, a general caveat was made by Von Thun (44) in conjunction with his analysis of the Quail Creek Dike failure on January 1, 1989. This was an occasion when grouting was not effective: although it remedied the symptom of the problem (excess seepage), it did not resolve the real problem (the potential for embankment materials to pipe excessively due to lack of contact treatment during original construction). This must always be a critical point in evaluating the nature of the remediation proposed.

From the grouting viewpoint, embankment dams may involve both rock grouting, as referenced above, and soil grouting. In some ways, soil grouting is more complex and less precise given the tremendous range in both determinant soil properties, and in grouting methods and materials. As a guide, papers by Naudts (92) and Bruce (88) will serve to lend perspective to the myriad of excellent books (e.g. Karol, 21), and conferences (23). In connection with seepage control, the two basic types of soil grouting involved are:

- permeation: where grout is placed into the pre-existing pores while preserving the virgin soil structure, and
- jet grouting: where cement grout at very high pressure is used to simultaneously erode and mix with the soil. The cutting effect of the grout is often enhanced by combinations of air and/or water.

3.3.1 Grouting of the Core

Drilling and grouting the core of existing dams is a delicate and emotive issue. However, there is no doubt that, correctly designed and carefully conducted, it can be an extremely effective option. Such treatment is usually continued down into the foundation horizons, but in this section, case histories are presented which concentrate on the grouting of the core. The case of Mud Mountain Dam is addressed in Section 4.2.6. At this point it may be reiterated that whereas grouting was not trusted as a means of sealing the core at the site, it was turned to in an emergency situation, to treat the core to permit the safe installation of a diaphragm wall, judged to be the definitive cut-off.

3.3.1.1 Banbury Dam, England (45)

In south eastern England there are many older storage reservoirs, consisting of an encircling dike with a puddle clay core, founded on the London Clay. The puddle is silty alluvial clay, prone to fissuring over long service. At Banbury Dam, various options for a protective cut-off were considered, but the "thin grout screen" method was selected. In such cases, the grout is required to have:

- an early shear strength not less than that of the clay core (say 0.035 MPa at 7 days)
- a permeability of about 10^{-9}m/s
- stability and durability
- flexibility
- pumpability (without segregation)
- resistance to chemical attack

Following experiments, the following mix was selected:

Type 1 Cement:	4.2% by weight
P.f.a.:	16.8%
Bentonite:	5.6%
Kaolinite:	16.8%
Water:	56.0%
Sodium tripolyphosphate:	0.3%
Methyl acetate:	0.3%

A single I-section was vibrated into the core, and the grout introduced from the base (Figure 14). Overall, 450m of the core was sealed in this way to an average depth of 5-6m and a minimum thickness of 50mm. Excellent piezometric performance has been recorded since (over 15 years). The authors ascribed this both to the remoulding effect of the vibrator and the action of the grout.

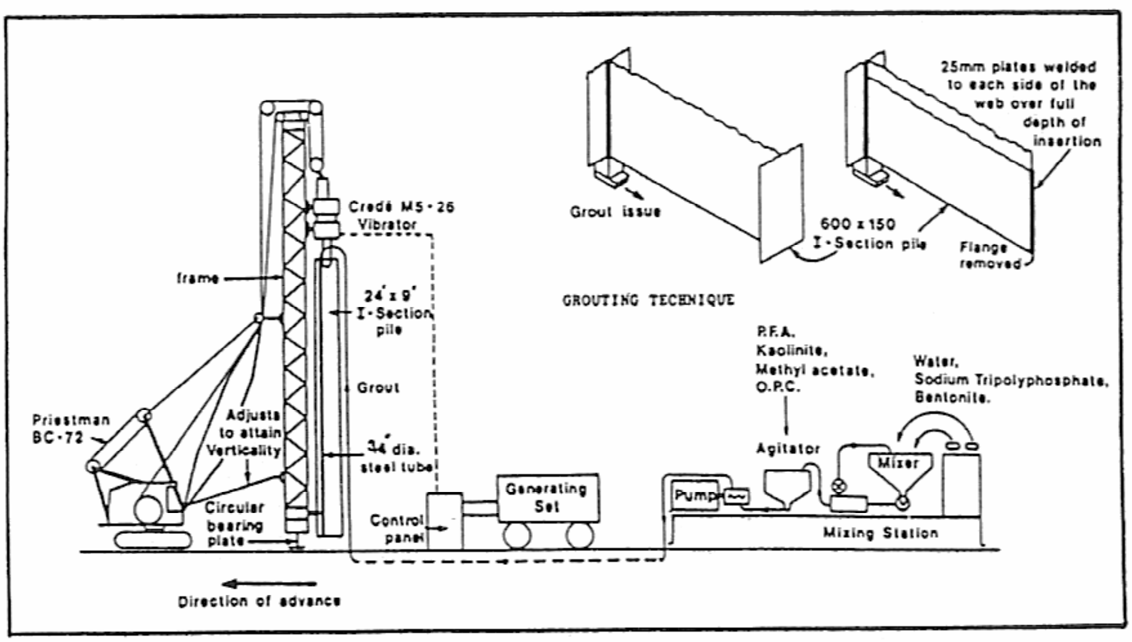


Figure 14. Schematic view of grouting equipment, Banbury Dam (45

3.3.1.2 Two Dams in Eastern Canada (46).

Two rockfill dams for electricity generation exhibited worrisome trends within one year of first impoundment: abnormally high seepage, crest sinkholes, and surges of muddy water. Each dam's core consisted of well graded glacial till. Exploratory programs showed that there were distinct zones in the core from which loss of material had occurred due to early piping. A glacial till blanket proved ineffective, the concept of a diaphragm wall was rejected, and grouting was selected.

A modified rotary percussive duplex system (47, 47a) was used to advance holes through the cores without the danger of hydrofracture. The underlying rock was then cored or percussed. In advance, lake levels had been reduced 2.44m, the lowest practical pool level for generation. "This reduced the buoyancy of the dams, thus increasing the earth pressures within the dam core and, therefore, reduced the risk of hydro-fracturing of the dam during drilling and grouting operations."

After experimentation, two mixes were used:

Core Grouting	Cement - Type 30 Bentonite - 20% by weight of cement Sodium Silicate - 5% by weight of cement w:c 8:1 - 2:1 (by volume)
Rock Grouting	Cement - Type 30 Bentonite - 3.4% by weight of cement w:c - 6.3:1 (by volume)

Conventional split spacing staged methods were used. The rock had uniformly low takes, but the core material either had large takes or no consumption. Grouting was conducted in 3m ascending stages through the casing at very low excess head. (0.6m head of grout)

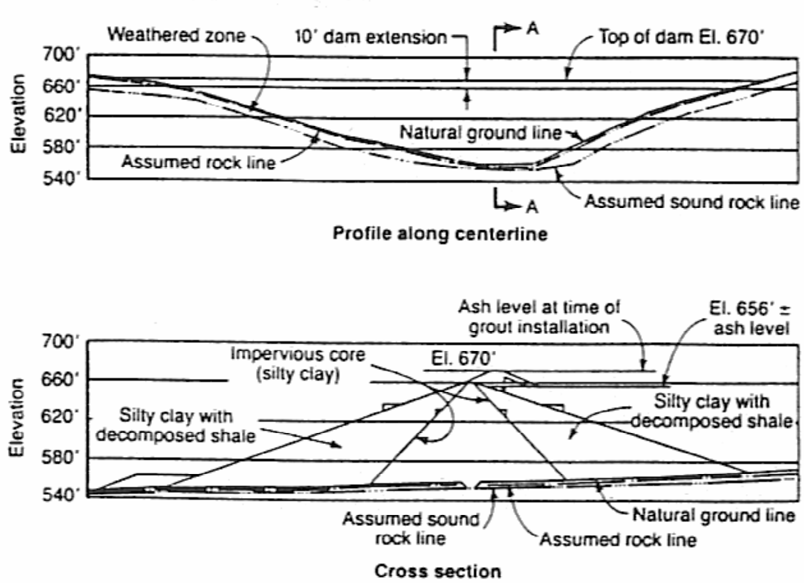
Dam "A", 413m long by 21m high was grouted first. Weir flows of 1.31 - 1.58m^3/min were reduced to 0.16 - 0.19m^3/min, in 3 areas. Dam "B", 1850m long by 24m high had seepages in 5 places, mainly through the rock, reduced by 26-97%.

Much of the grout injected in the core escaped into the shell: there were observations of grout flowing from the downstream toe during injection of certain "high take" stages. Analysis of the grouting data indicated that the washed out areas were not generally interconnected and were of local extent. The majority of stages accepted only their nominal hole volume.

The record of both these structures has been uneventful since treatment.

3.3.1.3 Ash Basin Number 2, Pennsylvania (48)

The basin was built in 1955 to store residual flyash. Due to rapid infilling, it was raised twice - by 3m and by a further 9m. However, by 1982 a seepage problem had developed, with attention focused on the weathered rock zone just below the dam (and the contact), the lower embankment, just above the interface, and the upper embankment (Figure 15).



The core of the 870 ft long, 106 ft high dam consists of silty clay materials on native, slightly weathered shale; the remainder of the embankment is made of silty clay with decomposed shale.

Figure 15. Layout of Ash Basin No. 2 (48).

The grouting work carried out illustrated several good features "not widely practiced" at the time of publication, including:

- thorough site exploration
- duplex drilling (percussive eccentric) with air and foam to protect the embankment against hydrofracture
- low pressure, short duration water tests as a precursor to grout mix design
- field selection of grout types, from stable cementitious mixes for high takes, to rapid set low viscosity acrylate (AC-400) for tighter conditions
- colloidal mixing of low w/c grouts to promote stability
- continuous mixing and injection of acrylates to provide stage pumping times in excess of the gel times.
- split spacing of holes
- careful, real time monitoring of injection pressure and flow data to detect the possibility of embankment fracturing
- piezometric monitoring of water levels, before and after grouting, in the embankments and in the foundation

Prior to grouting, seepage had been identified at two locations, one with 140 litres/min, the other a general seepage on the lower embankment. The average foundation permeability was about 1×10^{-5} m/s while the piezometric level in the dam was flat.

The treatment consisted of a single row of holes, the Primary spacing of 6m quartered by the Secondaries and Tertiaries. These holes were continued 1.5 - 3m into the shale bedrock to permit treatment of it, the contact and the lower 6m of silty clay embankment. Holes ranged from 8 - 36m in depth.

The cementitious grout - very stable, pumpable, economic and nonshrink - comprised cement and flyash with a ratio of 5:2, and a water solids ratio as low as 0.4.

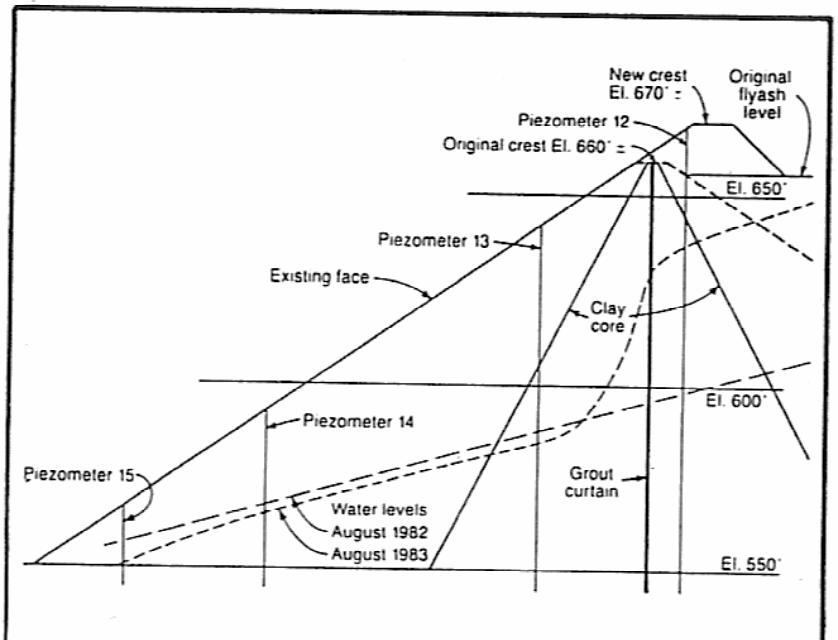
The chemical grout chosen was AC 400 due to its excellent gel time control, stability under flowing water conditions, lack of syneresis, low viscosity and relatively low levels of toxicity. Over 170,000 litres were used - the largest single application to that date - with a formulation of:

25%		AC 400
0.3%		Triethanolamine
0.3%		ammonium persulfate
Balance	-	water

This gave a 10 minute gel time at 15°C, and was used in flowing water conditions and where the permeability was less than 1.4 x 10^{-5}m/s (100 Lugeons).

Grouting pressures were limited to 0.01MPa/m at flows of less than 15 litres/minute, through the casing during its withdrawal. Excellent reduction ratios were recorded.

After grouting, seepages totalled less than 8 litres/min, and the piezometric surface in the dam was strongly modified (Figure 16). During the work there had been no evidence of fracturing in the dam and no grout seepage was observed.



Water-level profiles show a general lowering of the water level downstream of the grout curtain and a drastic rise in the groundwater level as recorded in piezometers immediately upstream of the curtain at the rock-embankment interface. Permeabilities calculated from tertiary grout stage water tests show a reduction in permeability of the embankment soils from the original 1×10^{-3} cm/s to values approaching 1×10^{-5} cm/s.

Figure 16. Effect of grouting on piezometric level, Ash Basin No. 2 (48).

3.3.2 Grouting of the Foundation.

3.3.2.1 St. Mary Dam, Alberta: Classical Cement Grouting (49).

When completed in 1951, this 62m high structure was the highest earthfill dam in Canada. Superficial deposits were removed and a 3 - 13m deep cut-off trench was blasted into the flatlying sediments as a key trench. With initial filling, numerous seeps along the embankment toe and the abutments developed. There was thus concern about the possibility of piping and high pore pressures. It was suggested that the blasting had further opened fractures in the shale-sandstone foundation, decomposed and weathered in places.

A single row curtain was formed, incorporating 15,000m of 101mm diameter holes and 3,000m of 76mm diameter holes, all drilled vertically. Many had complete flushing water loss. Split spacing reduced the Primary spacing of 12.2m to 1.5m in the Quaternaries. Grout pressures were not allowed to exceed 0.02MPa/m. Average takes were around 180kg/m, but very erratic in distribution: some 10m stages took in excess of 22 tonnes.

By 1957-58, the total seepage had been halved, to 1.6m^3/min, and the owners "... considered grouting to be an effective, if rather costly, solution to the seepage problem". In addition, piezometers in the downstream half of the core still indicate no pore pressures, and a new piezometers close to the centerline of the dam, with tips near the bottom of the embankment, are all dry.

Similar techniques were more recently used to combat piping, which had induced sinkholes on the upstream face of Gurley Reservoir Dam, CO (50). On this occation a three row curtain was constructed to high engineering standards to give a "very successful" result and security to the dam and its appurtenant structures.

3.3.2.2 Unnamed Dam: Simple Chemical Grouting (51).

This zoned fill dam, 28m high, was built in 1958-59 for domestic water supply. The bedrock was volcanic, including blocky tuff under the core, and flow rock under the shell. An initial grouting program was abandoned in favor of a deeper cut-off trench. Upon first filling in February 1960, seepage occurred adjacent to the 23m long left Groin, high on the downstream face. This totalled 150 litres/min but was left untreated as there appeared to be no danger of piping.

However, a later need to raise the dam 2m demanded that treatment be carried out. The (now banned) acrylamide grout AM 9 was used in a total of 23 holes, 9 - 25m long, split spaced 1.2m apart. Takes varied from 90 to 2600 litres per hole, at nominal excess pressures on the casing, and totalled 12,000 litres. Analyses confirmed that while some grout was in fact placed in the embankment and at the contact, "most" permeated the foundation tuff.

The treatment was successful-leakage dropped to 12 litres/min, seepage dried up on the face, downstream piezometric levels decreased, and no cases of neurotoxic effects were recorded.

3.3.2.3 King Talal Dam, Jordan: Contemporary State of Practice Chemical Grouting. (52)

King Talal Dam spans the Jordan River, and mainly acts as a source of irrigation water. Recent plans to raise the embankment by 16m to 115m involve treatment of a sandstone and limestone formation in the Left Abutment. The sandstone is Lower Cretaceous, fine to coarse grained, medium to thick bedded and cemented with iron oxide and some clay minerals. It is therefore weak and easily weathered, friable and porous. Some joints contain loose sand infill. Seepage water moves easily, at a maximum of 5.4m^3/min. There is a high primary and secondary permeability.

An intensive program of grouting is therefore being conducted from within galleries in the abutment, as there is great concern about the posibility of piping of the sandstone into the underlying karstic limestone. The treatment is designed to stabilize a 4-6m wide zone, primarily to prevent erosion: reduction of seepage is a secondary benefit.

State of practice tube à manchette methods (53) are being used, featuring a variety of materials including cement-bentonite for voids and the gallery/rock contact, sodium silicate and organic ester reagent in outer rows, and other low viscosity chemical grouts to provide inner row tightening.

3.3.2.4 New Waddell Dam, AZ: Jet Grouting. (54,55)

Even allowing for the fact that jet grouting is the new comer to soil treatment methodology, its use in dam rehabilitation has been uncommon to date. Apart from the example - for liquefaction control at John Hart Dam, described in Section 5.2.3 and the imminent use - largely for structural underpinning at Deerfield Dam, MA, - the only other North American example remains the large scale test conducted at New Waddell Dam, Arizona. Overseas, the situation is somewhat more positive, with major seepage contol remediation having been conducted at Brombach Dam, West Germany, and at Villanueva Dam, Spain.

One reason for this slow pace of adoption is quite simply the fundamental - and wholly understandible - retiscence of the engineering community to permit grout jetting at up to 50MPa within the core and foundation of often highly sensitive and delicate dams. The other reason is less laudable, and stems from the somewhat indecisive results obtained from the New Waddell test. In this case, the difficulties inherent in an unfavorable geology appear to have been compounded by certain unfortunate operational decisions made at site level.

The Government commissioned the full scale test at New Waddell Dam as a chance to examine the potential of a cut-off wall formed by a new technology. The intent was to construct a shaft through 12m of younger alluvial sands and gravels overlying an older, very dense sand gravel-cobble and boulder horizon with a clay matrix. The natural ground water level was about 2m below surface. The specification called for a circular ring of 18 jet grouted columns, 2.4m in diameter, 45m deep, with each column being a minimum of 0.6m in diameter. A drill hole deviation of $\pm$ 150mm was permitted, and a jetting pressure of 35MPa was stipulated, at a w:c ratio of at least 1. A plug 3m thick at the base of the recent alluvials was requested.

The contractor treated the first 12m, then redrilled through this to gain access to the lower 33m. A pressure of 40-46MPa was used, with a rotational speed of 25 rpm and a withdrawal rate of 0.25m/min. Monitoring of drill hole deviations indicated that holes wandered in the region of 1 in 75 to 300. Prior to grouting, this soil volume had yielded 1-1.4m^3/min. After grouting the flow was about 230 litres/min, but it was impossible to tell if this flow was coming through "windows" in the shaft wall, or up from the (unsealed) base at rockhead.

After excavation into the grouted shaft, and as confirmed by cores taken in the ring, it became apparent that the grouting method used (grout jetting only: no air or water enhancement) had not produced columns of uniform diameter in these conditions. Given these uncertainties and mistrusts, the alluvial cut-off was subsequently conducted by diaphragm wall (see below).

To redress the balance, however, it is only fair to note the rather more encouraging comments of Guatteri and Altan (56) in relation to the potential of jet grouted cut-offs.

3.3.2.5 Tarbela Dam, Pakistan: End of Casing Grouting of Alluvial Foundation (38).

During the construction of the Main Embankment Dam, grouting techniques were used as an exploratory tool to investigate aspects of the foundation alluvials. In particular, the concerns about the potential for piping dictated the need to check if the openwork gravel would accept alluvial sand, and if these open, highly permeable zones were laterally continuous.

The significance of this work has wider relevance to dam remediation, however. Work is currently underway on a major dam in Washington State where it is intended to remedy major seepage problems - which already having induced substantial sinkholes - with a diaphragm wall. However, there are major concerns that the impared dam's structure and foundation alluvials will prove so permeable that massive and potentially catastrophic slurry losses may occur during wall construction. An exploratory/sealing program has therefore been instigated prior to wall construction. Of particular interest is the use of rotary duplex drilling techniques to over 70m through the core and alluvials, with the reservoir elevation at full pool (differential head 25m).

At Tarbela Dam, end of casing injection was conducted in progressive ascending stages with a mix of composition:

cement:	bentonite:	sand:	water	
1.0	0.2	5.0	11.0	by weight

Maximum excess pressures were:

>18m	0.21MPa
9-18m	0.14MPa
3-9m	0.07MPa
0-3m	Gravity head only

Refusal was set at a stage take of less than 14 litres/min.

Takes varied from 75 - 230m^3/m and careful analysis of their charateristics yielded invaluable information on the openwork gravel, used beneficially during the subsequent construction of the embankment and its protective upstream blanket.

3.4 Miscellaneous Applications.

3.4.1 Unnamed Dam: Backfilling of Old Mine Shaft (51).

The brief case history describes a smallish 11m high embankment with a clay core, and shells, and a 13m deep core trench. It dams a creek where gold sluicing and dredging had been conducted. Upon first impounding, the downstream seepage was 120 litres/min, but after 2 months there was a sudden loss of about 18m^3/min. The embankment itself appeared to be working efficiently, but a flow of 4m^3/min was recorded 150m downstream, and an old mine shaft 800m away began to fill up.

It was reasoned that reservoir water was finding its way into the old workings, and the reservoir was emptied. Inspection confirmed the presence of old shafts in the river bed, the caps of which had been breached under the reservoir head. These shafts were then properly backfilled and the whole complex functioned perfectly thereafter.

A similar problem was addressed during the raising of Grimwith Dam, Yorkshire (41) where extensive old coal workings in the valley sides had to be grouted up before the reservoir could be raised to its new level.

3.4.2 Old River Low Sill, MS: Void Infill under Structure (58).

The structure, 240km upstream from New Orleans on the Mississippi River, was completed in 1960. During the 1973 flood, a large scour hole was produced in front of Gate Bays 8, 9, 10 and 11 to a maximum depth of 20m. Some 85,000 tonnes of riprap were placed. Further site investigation confirmed the existence under the structure of a cavity from 0.6 to 16m high under Bays 6-11 with a total volume of 23,000m^3. Uplift pressures downstream under the stilling basin almost equalled the headwater and so it was assumed that the original sheet pile cut-off was ruptured or undermined.

25,000m^3 of "self levelling" grout mixes were used, comprising barite, cement, bentonite and water in 3 basic mixes:

OR13: non sanded , first injected in small amounts with the intent to prevent further erosion of the sand.

OR5: sanded, the next mix, also with 170-350kg per m^3 of grout.

OR23: "topping off mixture" including 700kg cement per m^3 of grout, to ensure good concrete/grout underslab contact, and to provide a "hard capping".

Occasionally accelerators, and fillers were used to restrict flow. The travel of the grout was monitored electronically, and, interal., this confirmed the base of the piled cut-off was intact under Bays 8-10.

Since the repair, further exploratory drillling of the treated zone has been conducted, and this, together with data from piezometers and relief wells, confirms the proper performance of the structure/foundation system.

3.4.3 John Day Lock and Dam, WA: Traditional Consolidation Grouting of Foundation Rock (59).

This structure was built on the Columbia River 170km from Portland, OR from 1958-63. The lock is 206m long and 26m wide with a single lift of 35m making it one of the largest of its type in the world. Each concrete monolith was designed as a gravity structure, founded on grouted rock.

During a routine dewatering and inspection in 1975, a major fissure was noted near the base of the riverside wall (Figure 17). This was considered to be due to excessive foundation deformation at full condition, and hydraulic surge pressures within the culvert due to the initial filling procedure. By 1979, six of eleven monoliths were affected, and a two part repair scheme was initiated as a permanent solution:

- cement grouting to increase the foundation rock mass modulus by filling open joints and voids, and
- structural repair of the monoliths themselves.

The grouting was intended to double to 7000 MPa the modulus of a weak, weathered and variable flow breccia 8-12m thick, and sandwiched between upper and lower basalt horizons. This breccia

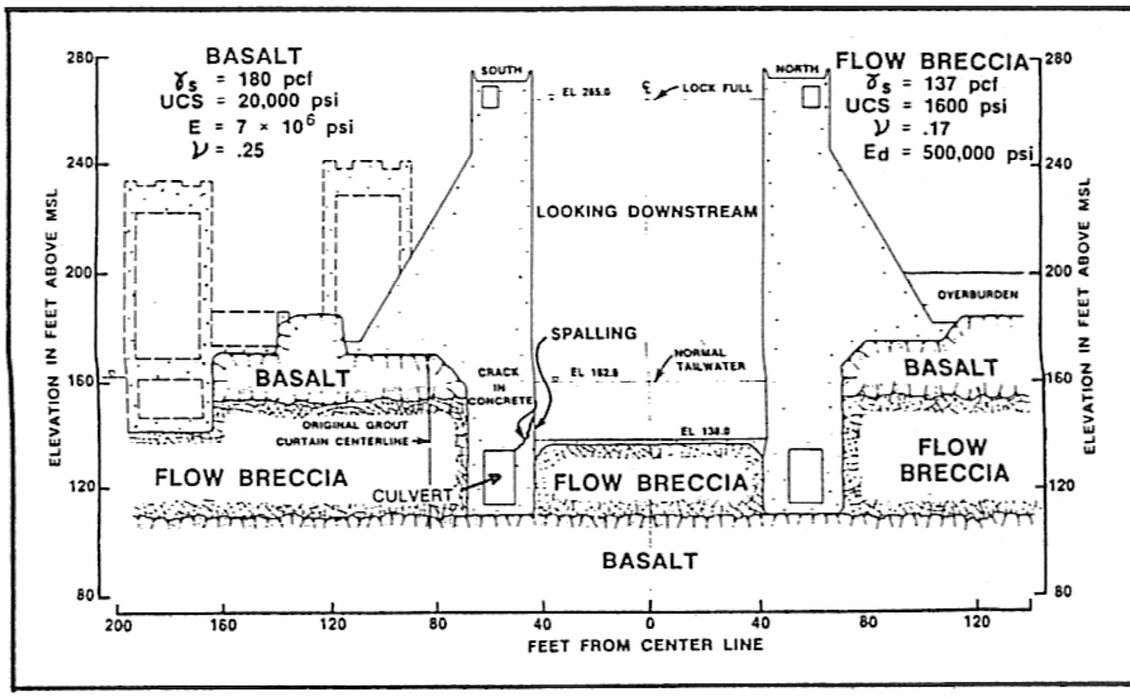


Figure 17. Cross section of lock, John Day Lock and Dam (59).

had a very variable permeability, averaging over 150 Lugeons and permitting a through flow in the 1979 outage of about $7m^3/min$.

According to then standard Corps of Engineers' practice, a total of 569 Primary and Secondary holes each 40mm in diameter was cored, averaging 22m deep over the 168m length. The holes were at 3m centers, inclined 5-25° off vertical, and in 6 rows, 1.8m apart. These holes toed 3m into the lower basalt. Holes furthest from the lock were grouted first, as a curtain. The upper basalt was grouted in one 9-10m long stage, and the flow breccia was treated in 1.5m long descending stages. Intensive water testing and washing was conducted in each hole prior to *grouting. Grout mixes featured Type 1 cement of w:c ratios of* 3.5 to 2:1 (by weight) plus fluidifiers. Low pressures were used (0.2MPa) to avoid potential uplift in adjacent blocks.

Overall, including a few Tertiary holes, the upper basalt involved 7450m of drilling and 150 tonnes of cement, and the breccia had 5800 m of drilling and 1240 tonnes of cement. Excellent reduction ratios confirmed the progress of the treatment.

Post grouting rock modulus was measure by three methods:

- Goodman Jack
- 3 - D Accoustic Velocity Logger
- Crosshole Seismics.

The latter two were considered most appropriate and they confirmed the target modulus had been achieved. Subsequent structural deflections measured during the annual cycle were halved.

It was noted that the stage grouting method, though successful, understandably slowed the progress and necessitated double shifting of the resources. Under such conditions, the MPSP system (Section 3.2.2.2) would have been ideal.

3.4.4 Little Goose Lock and Dam, WA, and Savage River Dam, MD: Further Examples of Consolidation Grouting (27).

Each Corps of Engineer's District was contacted in a search for case histories of remedial grouting for rock mass consolidation. Whereas several examples were found of such grouting projects for seepage control, only three cases of consolidation grouting were found, one of which was John Day, as described above. The other two examples were as follows:

Little Goose Lock and Dam, WA is located on the Snake River, 120km upstream of its confluence with the Columbia River. Differential monolith movements during full lock conditions had led to concrete spalling and damage to the water stop. Differential movements of 6-8mm compared with permanent outward deflections of over 18mm. Part of the repair was the treatment of flow breccia under the capping of competent basalt.

Using similar methods to John Day, takes of around 450kg/m were recorded. The results were greatly reduced movements, and "additional stability to the foundation".

The Savage River Dam is 7km upstream of its confluence with the Potomac River, MD. The 55m high, 315m long earth and rockfill dam has a concrete spillway and weir. The bedrock is a limestone with two sets of near vertical joints. Distress to the concrete structure resulted in a consolidation grouting program, involving 49 holes in 3 rows, 2.1m apart, at 6m centers. A total of 640m of drill holes consumed over $50m^3$ of grout (neat type 2 , of variable w/c ratio). It was concluded from observation that the foundation was "effectively grouted to a depth of 3m below the spillway floor over 90% of the area".

4. Seepage Control By Concrete Diaphragms

Concrete cut-offs under new dams built on alluvial foundations have been installed around the world for decades. Such diaphragms have provided positive seepage barriers on projects where technical, logistical, political and economic reasons have weighed against the choice of some other method, such as a grout curtain. Within the last decade, however, the use of concrete diaphragms to repair existing embankment dams has grown remarkably in North America, fostered on the one hand by doubts about grouting, and promoted on the other hand by aggressive marketing by certain foreign based specialist contractors. Despite variations in equipment and material, most of the concrete cut-offs have been constructed by the slurry trench, or diaphragm wall method. Most of this section is therefore devoted to examples from that category. In advance, however, another technique - overlapping large diameter piles - is reviewed. Although so far conducted on a limited scale in new dam construction, the technique has potential in remedial applications where the embankment is founded on voided rock masses such as karstic limestone or gypsiferous sediments.

4.1 Overlapping Large Diameter Piles Khao Laem Dam, Thailand (61).

This concrete faced rockfill dam is a multipurpose structure 114m high and 1020m long. It is located on the Quae Noi River, 300km east of Bangkok, and is founded on Ordovician and Permian limestones. These contain major karstic features to a depth of 60m, and other significant highly permeable zones to over 180m. Conventional grouting was conducted to this depth, but it was decided to reinforce the effectiveness of this effort with a concrete cut-off in three particular places where the solutioning was especially severe. In these areas grouting would have been unacceptably time consuming, labor intensive and expensive to ensure long term protection.

It was orginally foreseen to use the then newly developed hydrofraise (described in Section 4.2) to cut the wall into the rock. However, detailed site investigation confirmed that 83% of the core samples gave strengths in excess of 100MPa: the hydrofraise would also have been too slow and costly. It was then decided to use the overlapping pile method, successful at the R.D. Bailey Dam in the USA from 1977-79. The concept is illustrated in Figure 18 - Primary holes of 762mm were drilled and concreted at 1230mm centers, followed by intermediate Secondaries 36 hours later. The cut-off then varied from 450-762mm in thickness.

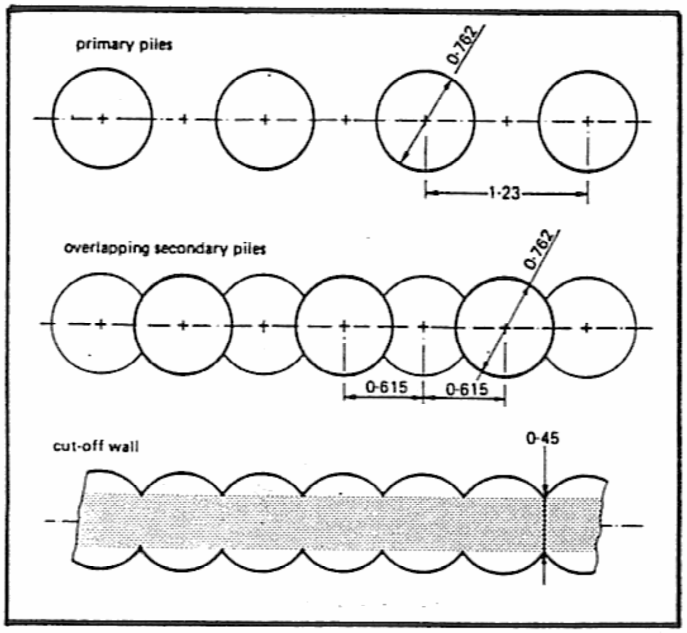


Figure 18. Construction phases of the cut-off wall, Khao Laem Dam (61).

The equipment used (Figure 19) featured:

- a 30m high tubular tower on a 70 tonne crane, as a guide, and support for the head
- a high torque rotary head (2700kgm at 2-5 rpm)
- 686mm diameter down-the-hole hammer with a 762mm bit, mounted on 6m long rods of 686mm o.d. and 152mm i.d.
- compressor station providing 90-100m^3/min. of compressed air at 1Mpa.

This whole mobile unit, complete with 60m of rods, weighed 135 tonnes.

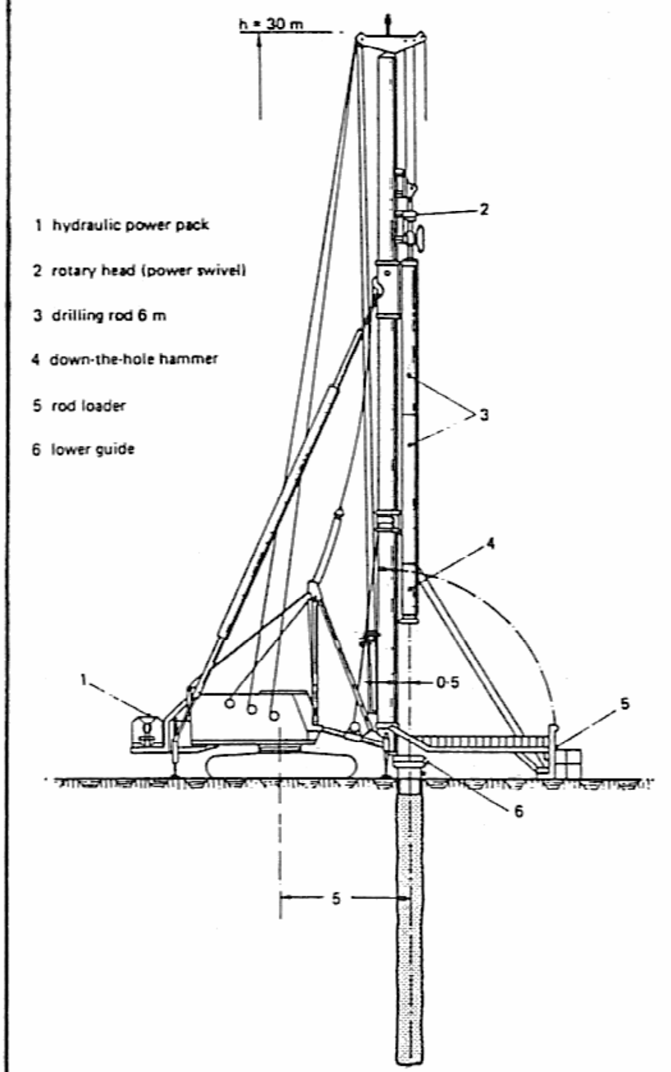
If the hammer found a cavity, a separate rig was used to clean out all soft material, by grab and by jetting with air and water. Such cavities were then backfilled with concrete prior to resumption of the pile drilling. Cavities with volumes as much as 50m^3 were found.

Overall, a total length of 431m was protected in this way, from 15-55m deep, and totalling almost 16,000m^2. Over 18,700m^3 of concrete was used at an average rate of 0.73m^3/m of hole. The overall industrial average was 46m/day, all of this being under the water table. Hole verticality was measured by pendulum and never exceeded 0.3%.

The authors concluded the method has no practical limits regarding the hardness of the rock, whereas depth is limited only by the available air pressure and machine torque - possibly 100m. Since construction, no evidence of piping and erosion has been recorded.

A similar, smaller application was conducted in conjunction with grouting at Mangrove Creek Dam, Australia (62) to positively prevent movement of erodible gouge material from the upstream toe of another high concrete faced rockfill dam.

Although no example has been recorded to date of the use of this technique to repair existing dams, it is surely a contender under similar geological circumstances where slurry trenching techniques are ruled out. It may be speculated that Beaver Dam, AR, (63) could be such a case. In this example, earlier traditional grouting attempts have had limited success, although the use of contemporary techniques (39) would arguably produce the desired result.



The EC 80 Superdrill, where: 1 = hydraulic power pack; 2 = rotary head (power swivel); 3= drilling rod; 4 = down-the-hole hammer; 5 = rod loader; and, 6 = lower guide.

Figure 19. Equipment for 686mm diamter holes, Khao Laem Dam (61).

4.2 Slurry Trench Methods.

As noted by Tamaro (64) there are generically three types of slurry trench cut-offs:

1. Soil-Bentonite - excavated continuously to maximum depth of 30m (15m by backhoe or dragline) under bentonite slurry. This is backfiled with an appropriate soil-bentonite mix to give a permeability of 10^{-8}-10^{-9}m/s. The thickness depends on the hydraulic gradient, which is conservatively limited to 10 to avoid either hydraulic fracturing or migration of fines.

2. Cement-Bentonite - excavated continuously or in panels under a cement-bentonite slurry. However, this mix can undergo shrinkage and cracking if dry and so large piezometric fluctuations can render the cut-off more permeable. This self-hardening mix gives an initial permeability of about 10^{-8}m/s.

3. Concrete - excavated in panels under bentonite in a Primary and Secondary sequence. The backfill may be plastic concrete (say 3-6MPa at 28 days) or a conventional mix design depending on the characteristics of the dam and the cut-off. Material permeabilities range from 10^{-9}m/s to 10^{-12}m/s. This type is the best choice for deep walls, sujected to high gradients, in dense soils and softer rocks. They are typically thinner (from 0.6m) but this depends on the equipment and the construction tolerance.

From the historical viewpoint (6), the first use of the soil-bentonite method was in September 1945 to form a 11m deep partial cut-off in a levee on the Mississippi. The first major dam aplication was at Wanapum Dam on the Columbia River in 1959, and featured a wall 3m wide and 24m deep to resist a 25m maximum differential head. Apart from ancillary applications, eg. at St. Stephen Dam, SC, and Fontenelle Dam, WY, the only major sizeable dam thought to have been repaired with this technique is Manasquan Dam, NJ (30).

Following earlier new applications in Mexico (1970-72), and Michigan (1976), the first remedial applications for cement-bentonite walls were in Addicks Dam, and Barker Dam, Texas, in 1979. (1m wide, 21m deep, 9m differential head). Since then, however, the scale of the repairs, and the advances in excavation technology appear to have curtailed activity of this type in favor of concrete walls. The example of Prospertown Lake Dam, NJ, described below is a rare example, as is the project at Sulphur Creek Dam, Wyoming (30).

The first Corps of Engineers' concrete wall was constructed at Kinzua Dam, Pennsylvania in 1964, and the technique has been developed since at major sites such as Manicouagan 3 Dam, Quebec (66) and New Waddell Dam, Arizona (67). However, the key project in relation to dam remediation was the sealing of Wolf Creek Dam, on the Cumberland River, Kentucky in 1978. As detailed below, a 30MPa concrete wall, 0.6m thick and 90m deep was installed through a 61m high embankment and a maximum of 30m into cavernous limestone to resist a maximum differential head of 56m. Individual panel deviations had to be less than 1 in 600 to ensure an acceptable area of interpanel overlap.

4.2.1 Prospertown Lake Dam, NJ: Cement-Bentonite Wall (68).

Built in 1965 on Lakaway Creek, NJ, this homogeneous earthfill embankment is 65m long and 6m maximum height. In 1966 seepage on the downstream face was first noted, and monitored with increasing concern to stability until 1985. The seepage was found to be occuring through relatively permeable embankment soils and near surface foundation deposits.

A cut-off was designed through the alignment of the dam to control the seepage and improve stability. The target permeability of 10^{-8}m/s was 1000 times lower than the existing condition. With a minimum width of 0.8m it was designed to toe into a relatively impermeable glauconitic clay stratum. It was designed as a simple "one stage" construction rather than the two stage (i.e. excavate and replace) method of soil-bentonite.

The mix design featured a bentonite to water ratio of 0.0445 (by weight), a cement: water ratio of 0.034, and an additive to prevent flash set and reduce overall viscosity.

The 200m long wall averaged 7m in depth (5-10m) and was built in 4 weeks in 1988. The reservoir was lowered before construction. The cut-off was constructed by backhoe in continuous progressive panels, each "biting in" 0.6m into the previous day's work.

The authors reported on the high degree of qa/qc during the construction, but recommended that a permeability of $5x10^{-8}$m/s be regarded as a practical minimum target in future jobs, given mix workability restraints.

4.2.2 Wolf Creek Dam, KY: Deep Concrete Cut-Off Using Conventional Excavation Methods (69).

Constructed in the 1940's, the combination earthfill and concrete structure featured on original cut-off trench 3m wide, and a 15m deep single row grout curtain. In 1967 a muddy flow into the tailrace was observed, together with a small sinkhole near the downstream toe, and other saturated areas nearby. The following year a larger sinkhole about 3m deep by 4m wide developed, promoting concern about the action of the solution freatures in the foundation bedrock. A concrete diaphragm wall was chosen as the remedy, a decision strongly influenced both by the ability to construct it at full lake conditions, and by the excellent deep wall (120m) experience previously recorded by the specialist contractor at Manicouagan 3 Dam, Quebec.

It was constructed in alternating cylindrical Primary elements, and connecting Secondary panels (Figure 20), under bentonite slurry. The Primaries consisted of 0.6m dia. steel casings backfilled with concrete. The project, completed in 1978, took 4 years and two construction contracts. The total length of the dam sealed was 685m, while a further 60m was treated by grouting.

Subsequent monitoring inidicates generally a piezometric drop of 10-20m across the wall. There is still some underseepage through the rock not sealed, but the total flow from the measuring weir is around 4 litres/minute. Inclinometers show slight (but erratic) downstream and vertical movements, of the order of 7-10mm. All data confirm that the wall is acting as a successful seepage barrier.

Walter F. George Dam in Alabama was also similarly repaired using conventional excavation techniques.

4.2.3 St. Stephen Dam, SC: First Application of New Excavation Equipment (70, 71, 72, 73).

The dam is located on the Cooper River, and consists of a concrete power station flanked by two earth embankments. A major seepage problem was associated with the sand marl foundation.

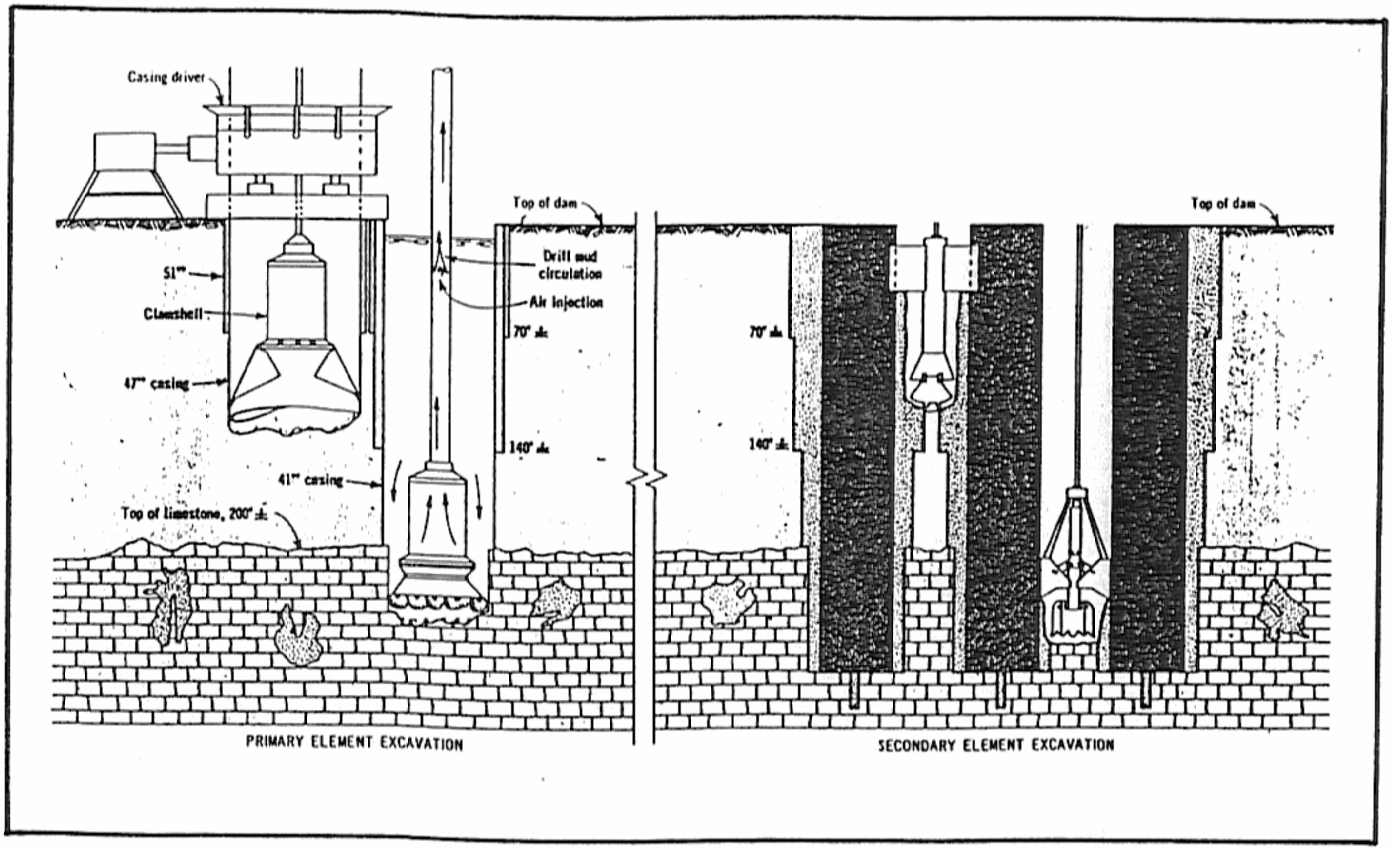


Figure 20. Cut-off wall construction details, Wolf Creek (6).

A 0.65m thick concrete wall was installed 36m deep with as much as 25m in the marl. Primary and Secondary panels were 9m and 2.2m long respectively. Due to the seismic nature of the region, each panel joint was further protected upstream by complimentary 5.5m long soil bentonite panels. The total area of the treatment was 12,000m^2, and the work was completed in 1984.

This project is particularly remarkable, however, for its introduction into the U.S. dam industry of a new excavation tool - the hydrofraise or hydromill.

Previously, the standard method of excavation was to use a grab (or bucket), suspended from a crane or kelly or cables. This grab is lowered into the trench, "bites" the material and then is lifted, full, back to the surface to discharge the excavated material (and a volume of bentonite slurry). However, this system has certain limitations, especially when the special problems of sealing large, existing dams are considered, and from the early 1970's the new machine was developed.

The principal components of the system (Figure 21) are:

- a crane, typically 100-150 tonnes
- hydraulic power pack
- excavating machine, typically 15m high, 25 tonnes in weight and equipped with three hydraulic motors (one to drive each of the milling wheels, the third to power the suction pump to lift to the surface the cut debris suspended in the bentonite slurry).
- an adjacent slurry treatment plant to separate out the debris and return cleaned bentonite to the trench.

The major characteristics of the excavating machine are:

(i) The cutting wheels have high torque and turn at 10-20rpm. The frame has a hydraulic feed cylinder to control effective weight and so the thrust on the wheels.

(ii) Typically, a trench 2.4m long by 0.6-1.8m wide can be constructed, to depths of over 120m.

(iii) It can now cut fissured rock masses of material strength up to 100MPa. However hard boulders 75-150mm in diameter pose a problem as they are too small to be crushed by the wheels but a critical size for blocking off the uplift pump suction.

(iv) It is a shock free excavating technique, without the need to chisel into hard till or rock.

(v) Excavation is continuous, without the need for repeated insertion and withdrawal of the tool. This leads to enhanced trench stability and straightness, and more effective slurry containment and handling.

(vi) It permits continuous desanding of the slurry during excavation.

(vii) It gives excellent interpanel joints, without the need for end stops or pipes. Primary panels are usually 6m (2.4+2.4+1.2) or 9.4m long (2.4+1.1+2.4+1.1+2.4). These are set about 2.2m apart so providing a 200mm overlap by each Secondary panel into each Primary (typical strength 15MPa).

(viii) It provides excellent verticality, and the facility to

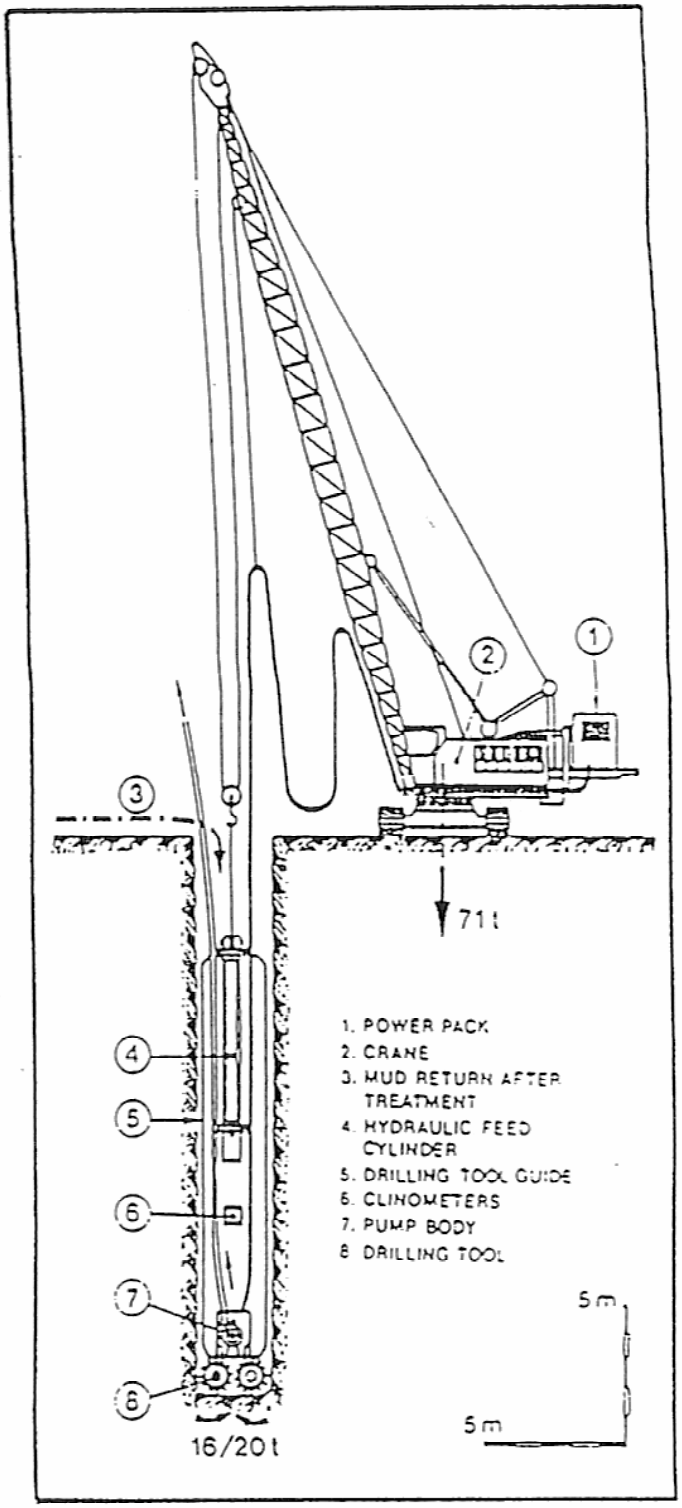


Figure 21. Equipment for rock mill excavation (71).

correct deviation tendencies in either the longitudinal or lateral directions. High precision inclinometers allow continuous monitoring of position while all the other drilling parameters are also recorded. On a recent deep trial in Milan (73) a steerable accuracy of 0.1 - 0.2% in the transverse sense, and 0.2 - 0.6% in the longitudinal sense was demonstrated. No panel torsion was noted. The test also demonstrated the potential of ultrasonic testing to confirm the shape of the excavated panel before concreting, and the use of cross hole seismics to demonstrate the quality of the concrete panels and the joints between them.

Since the St. Stephen Dam repair, three other rehabilitations have to date been completed with this technique, as described below, with a similar number of shallow repairs conducted with conventional grab, or bucket methods.

4.2.4 Navajo Dam, NM (30, 74, 75).

The dam is on the San Juan River and is part of the Colorado Storage Project, providing water for flood control, recreation, municipal and industrial uses. The 115m high embankment, 1200m long, and 9m wide at its crest was constructed from 1958-63. Filling began in 1963 and immediately saturation of the downstream fill was noted, together with seepages of 2.4m^3/min in the Left Abutment and 5m^3/min. in the Right Abutment.

The bedrock consists of flatlying sandstones with layers of siltstones and shales, and was very fractured, weathered and permeable. Evaluations were made of the original curtain grouting which concluded "following the technical specifications of that period, was actually too light".

The 1984 SEED Report found:

- there were open horizontal and vertical joints in contact with the embankment core material on both abutments
- the embankment had erodible core material
- the potential existed for uncontrolled seepage along the contact sufficient to cause piping
- this piping could lead to dam failure

Whereas a drainage tunnel scheme was selected for the Right Abutment, a concrete diaphragm wall (Figure 22) was chosen for the Left.

This extended 135m from the Abutment into the embankment and its maximum depth of 120m (including at least 15m into rock) made it in 1987-88 the deepest wall constructed in an existing embankment dam. With a width of 1m, it covered an area of 13,000m^2 and took 630 days to build.

A special excavator 27m high was used to provide Primary panels 5.7m long, and Secondaries 2.0m long. The design strength of the concrete was 21MPa. At maximum depth, the rig instrumentation confirmed an interpanel overlap of at least 0.65m. During excavation, however, there were five major slurry losses. The worst involved the sudden loss of 400m^3 of slurry and 80m^3 of sand and gravel at a depth of 100m. None was ever seen. Another loss involved 150m^3 of slurry and 40m^3 of sand, but this was noted exiting 120m away in the dam's groin.

Of particular interest on this project was the intensive array of instrumentation installed in the dam to monitor the impact of the wall both during and after construction.

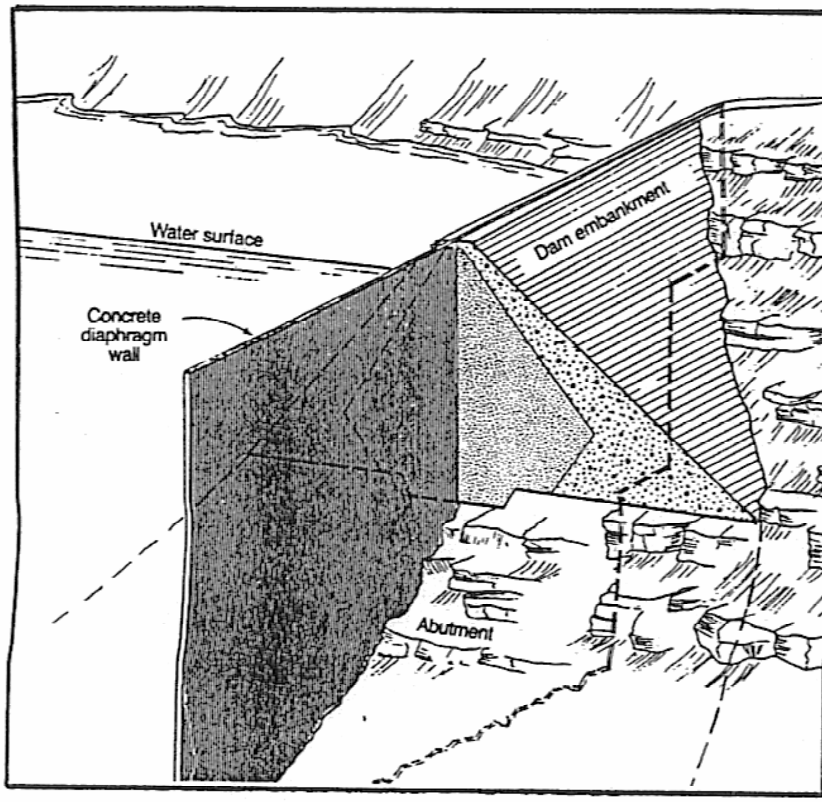


Figure 22. Concept of Left Abutment cut-off wall, Navajo Dam (74).

Piezometric levels downstream of the wall have dropped as much as 3.3m in the Left Abutment, and 9m in the dam whereas these upstream at the embankment end are 1.5-2m above the initial levels, indicating the effect of flow being forced around the end of the wall. Regarding seepage, the weirs indicated a reduction to about 40% of initial levels. Analysis indicates all the dam/rock contact flow has been stopped. Inclinometer data indicate no movements of the wall.

To ascertain the quality of the concrete in the wall, and the condition of the joints, 17 core holes were drilled. Cracking (small aperture) was noted in all panels and was assumed to be caused by shrinkage. No concentration of cracks was apparent at the contact area. The joints varied from concrete, to concrete bonding with a thin bentonite coating, to some with 6mm bentonite seams between the concrete surfaces. Water was lost 105m down in one panel, indicative therefore, of an "open crack" in the wall.

Sonic variable density logging in 13 of these holes then commenced, and led to the following conclusions:

1. The top 3-10m of the diaphragm wall had a lower density than the remainder.
2. Fracture locations correlated well with the logged core.
3. No large open fractures were recorded.
4. Low density zones existed near several of the panel joints.
5. One hole showed a concentration of cracking near the embankment/foundation contact at a depth of 21m.

As a result, Televiewer monitoring in all holes at the contact is now planned to determine if the cracking is increasing. Overall, though , the results indicated that "the amount of seepage stopped by the wall is not, however, as much as had been hoped for", and downstream piezometric levels have dropped "less than anticipated".

4.2.5 Fontenelle Dam, WY (70,76).

The embankment is approximately 2000m long with a 40m maximum height and is located on the Green River. Following completion in 1964, it almost failed in 1965 during initial filling due to piping of embankment material through fractures in the horizontally bedded sandstone in the Right Abutment. The embankment was repaired, additional foundation grouting was undertaken, the reservoir refilled and close monitoring started. In 1983, increasing seepage in areas downstream of the dam, anomolous instrumentation behavior, and evidence of piping into a piezometer standpipe led to further concerns, and a reduction in lake level by two thirds. Back analyses of original design and construction data confirmed the need to eliminate the potential for piping of the highly erodible silt core into the shell and the foundation.

A concrete diaphragm wall was chosed as the remedial measure, and two abutment test sections totalling 15,000m^2 were completed in 1986 to a maximum depth of 55m (including 5-50m into bedrock, using the rockmilling machine). Thereafter, the rest of the dam was similarly protected in a second contract involving over 70,000m^2 of wall.

Grouting and soil bentonite walls were used (Figure 23) to ensure adequate hydraulic seals at contact points between the embankment materials and existing concrete structures.

4.2.6 Mud Mountain Dam, WA (30, 77).

This major flood control structure is situated on the White River and was built between 1939 and 1942. It consists of rockfill with a silty sand core, and is 127m high and 210m long. The dam is built in a narrow steep sided canyon, and the geometry of the structure led to excessive differential settlement, cracking, and arch induced hydraulic fracturing. In addition, the upper 45m had never been subjected to reservoir loading.

By 1985, seepage and piezometer data indicated that there were direct hydraulic connections between the core and the reservoir, while the dam had inadequate internal filter zones to prevent piping. It appeared the flow was occuring through especially loose zones in sandier portions of the core, while in addition reservoir fluctuations had led to washed out zones in certain lower gravelly sections.

Remediation was necessary to ensure that the structure could withstand the maximum credible earthquake, and safely withstand the probable maximum flood. The favored solution was a concrete diaphragm wall extending down through the core and 5m into the volcanic bedrock the strength of which reached over 160MPa.

A total of 67 panels up to 7m long and 1m thick were installed, the Primaries being 2.4m apart. The concrete was color coded to confirm proper overlap by the Secondaries. The rockmilling equipment was virtually a "new generation" of the Navajo machine,

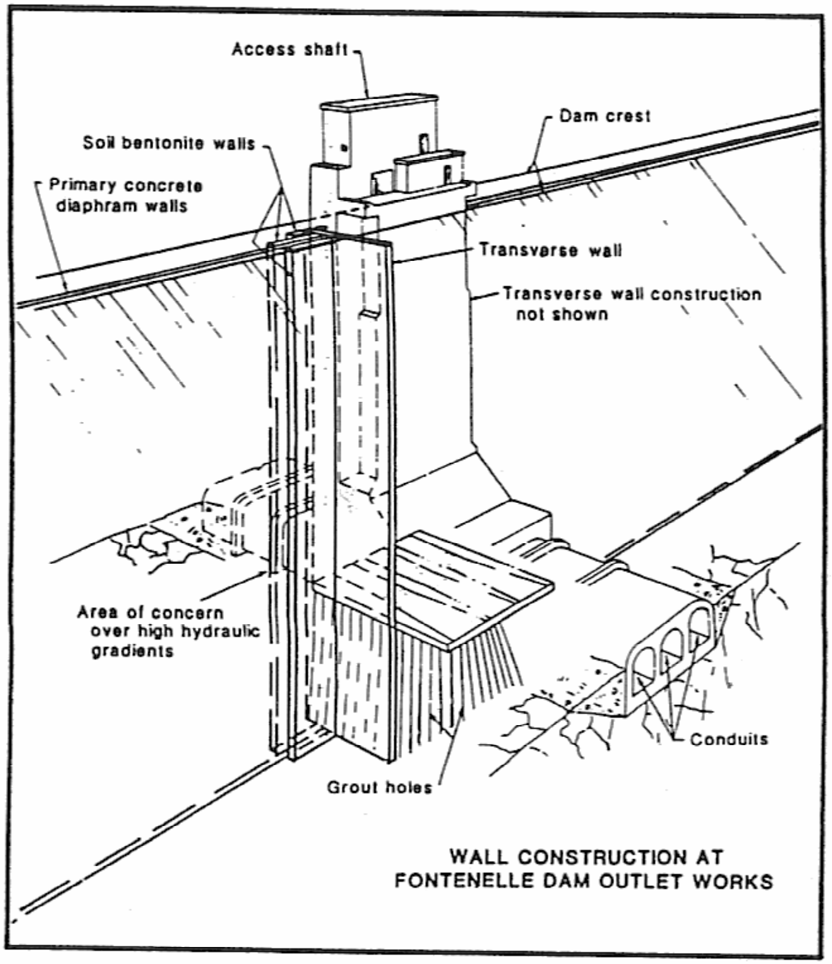


Figure 23. Cut-off wall, and back-up soil-bentonite panels, Fontenelle Dam (76).

weighing 45 tonnes, having three times more torque, and operating off a 51m boom and 25m long frame. At contract award time, the wall was the deepest installed.

However, construction was soon interrupted by massive losses of bentonite into the open zones of the core, which fundamentally cracked the core longitudinally, and created a network of other fractures. Slurry losses exceeded 4000m^3, with as much as 800m^3 being lost in a matter of a few minutes in certain panels.

The diaphragm walling was therefore suspended, and - ironically - grouting techniques used to repair the core in advance of further construction. Tube à manchette grouting (47) was conducted in two rows, 1.8m apart, to "recompact" the clay. This involved 6000m of drilling and 3600m^3 of cement-bentonite grouts, including sodium silicate to accelerate set in particularly severe conditions.

This remedial, remedial grouting program proved entirely successful at sealing the core, at an additional cost several times that of the original saving apparently generated by selecting the rock milling method. The diaphragm wall was then installed, as planned,through this treated zone without significant slurry loss.

5. Liquefaction Mitigation

5.1 General Aspects

It has been calculated that 650 of the 2000 Federally owned dams are located in highly seismic areas (78). Similar proportions of private and state owned dams are similarly located: for example in Illinois alone there are now over 30 dams in the major earthquake zone where the New Madrid events occurred in 1811-12. Well constructed earth dams built on competent foundations, and embankment materials generally perform well. However, embankment dams built of hydraulic fill or founded on loose saturated sands have the potential to liquefy.

The Corps of Engineers (79) reviewed the major successive possible courses of action for such structures:

(i) No action
(ii) Regulate access to the structure and to areas not affected by failure
(iii) Restrict reservoir level (or empty entirely)

(iv) Construct buttresses of a) Earth or
b) Retaining Walls above and below ground
(v) Increase height
(vi) Construct a downstream detention dam

(vii) Construct a replacement structure at either the same or a new location.
(viii) Improve the liquefiable soil.

From the viewpoint of this paper, only options (iv) b) and (viii) are of relevance, as basically there are five methods of providing direct insitu remediation:

- increase soil density
- increase effective confining pressure
- provide protection to structures while liquefaction occurs
- remove pore water pressure
- use grouting techniques to increase intergranular cohesion and fill voids, to prevent orientation of soil into a dense state.

No general method or approach is believed applicable for all conditions and structures, and indeed Marcuson and Silver (78) note that " in situ improvements made to dam foundations and embankments are the most challenging aspect of seismic dam improvement".

The selection of potential methods for a site improvement as well as the applications and results of the methods will depend on (79):

a) Location, area, depth and volume of soil involved
b) Soil types, properties and conditions

c) Site conditions
d) Seismic loading
e) Structure type and condition
f) Economic and social effects of the structure
g) Availability of necessary materials such as sand, aggregates and gravel
h) Availability of equipment and skills
i) Cost
j) Program

They further recommend that "at present state of the art, field tests must be conducted to insure that a selected --- method is applicable, --- to verify that the method will perform its intended function, and to show that the method will not threaten the safety and stability of the structure." They also provided excellent data on the verification of the effects.

It is possible to subdivide the different types of ground improvement techniques with respect to dam remediation:

<u>Group 1.</u> Techniques which have a proven record of improvement, but which could not possibly be used in or under an <u>existing</u> dam without prior demolition. Included are dynamic consolidation, compaction piles, stone columns, vibroflotation, and blasting (80).

<u>Group 2.</u> Techniques providing dewatering effects outwith the scope of this paper e.g. pressure relief wells, drains and blankets. Some of these techniques are, in addition, controversial and few have been field tested in actual earthquake conditions.

<u>Group 3.</u> Techniques within the scope of this paper but which have similarly not yet been field evaluated e.g. insitu soil reinforcement (81).

<u>Group 4.</u> Techniques which are within the scope, and either have been used on existing dams or have been used on new dams but could also be used in existing dams.

This section deals with examples from Group 4, and describes case histories where grouting and diaphragm walling methods have been used to provide the necessary aspect of ground improvement or treatment.

5.2 Case Histories

5.2.1. Pinopolis West Dam, SC: Soil Densification by Compaction Grouting (82, 83).

The 21.3m high, 2011m long homogeneous rolled earthfill dam was built in 1940. It is underlain by 1.2 - 2.4m of very loose sand about 3.7m below the original ground surface. Historically the Charlestown area has proved seismically active, and studies showed that the sand could liquefy and render the dam unsafe during an earthquake. Various downstream structures were considered to improve the seismic stability of the dam, as well insitu densification of the loose sand for which compaction grouting was promoted.

Compaction grouting has been used (84) since the early 1950's in the U.S.A. for settlement control and remediation, but only recently has its potential for densifying soils against liquefaction began to be exploited. It features the injection under pressure of very stiff grouts to displace and densify the surrounding soil. In contrast to permeation grouting (in which preexisting pores are infilled with grout) the influence of the grout bulb extends well beyond it, engaging soil volumes up to 20 times the placed grout volume. For embankment grouting, depths are usually greater, grout volumes are larger, and injection rates may be up to 10 times faster. (However, when grouting on a sloping embankment lateral displacements can easily occur, and may limit the treatment's effectiveness.) Concepts of mix design, and grouting methodology, parameters and analyses are detailed by Baker (82).

The grouting was conducted from a special test berm 6.1m high x 13.4m wide x 46.3m long built at the downstream toe of the main embankment. This allowed the actual conditions to be simulated without the need to operate initially under an active structure. Instrumentation was installed to monitor porewater pressures and embankment deformation. At this location, the target horizon was 10-12m below the berm's surface. This horizon was classified as very loose - loose grey silty fine sand, water bearing, with 10 - 20% fines and D_{50} = 0.3 - 0.6mm.

The Primary holes were installed on a 3.7m grid, with intermediate Secondaries and Tertiaries. The grout was pumped through 76mm id. steel casing at rates of up to 60 litres/minute at pressures typically 2 - 4MPa. The limiting criterion to grout volume injection was embankment heave - 25mm at depth and/or 6mm at the surface. The former value was later reduced to 19mm for the production work. The mix design was approximately:

Cement	120kg	per cubic metre of grout
Sand	970kg	
Flyash	790kg	
Water	300 Litres	
Pozzalin 122R	0.5kg	

This provided a U.C.S. of 4MPa at 7 days and 9MPa at 28 days.

Grout volumes injected are shown in <u>Figure 24</u>. Primary takes averaged 760 litres/m, Secondaries 620/m, and Tertiaries 350 litres/m. A total replacement volume of 25% was injected overall.

Post grouting tests indicated the following:

- Electric Cone Penetrometer - tip resistance increased from 2 to 8.5MPa after Secondaries, and to 12.5MPa after Tertiaries.
- Standard Penetration - increased from 4 to 17-25. (Previous studies had shown that a value of 11 was sufficient at the test site to avoid the potential for liquefaction at the downstream toe, and assure a safety factor of 1.25.

These increases alone suggested corresponding increases in resistance to liquefaction, but even then they "do not adequately reflect increased resistance .. due to large increases in lateral stresses ..."

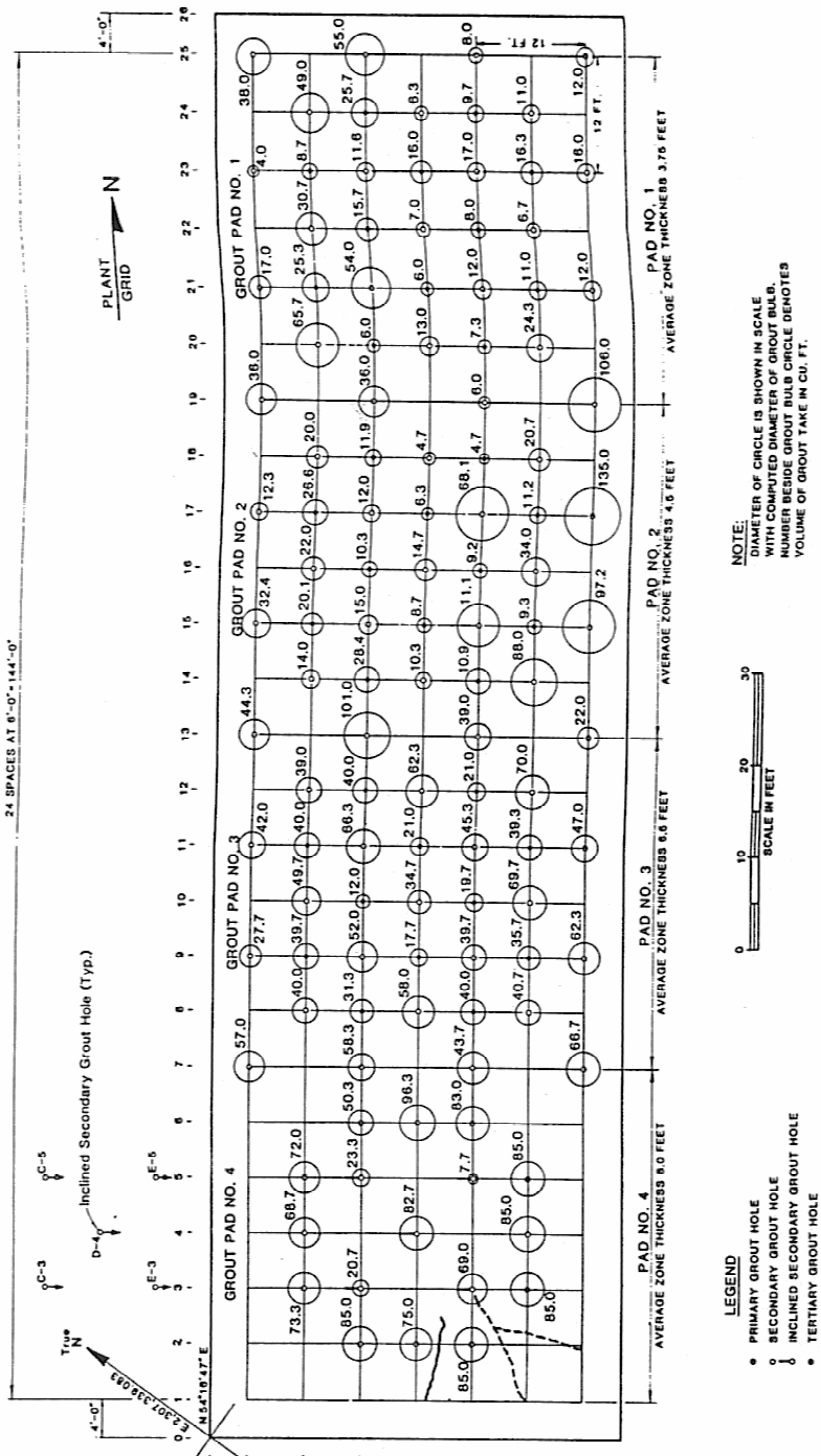


Figure 24. Compaction grout takes, Pinopolis West Dam (83).

- Flatplate Dilatometer Test - showed improvements in Constrained Modulus by 20 - 50 times.

The benefits of compaction grouting were clearly demonstrated in this program, and the technique was used, with minor modifications, in the subsequent full rehabilitation works. As a word of caution, it must be noted that a similar approach was tested at Steel Creek Dam, where Baker (82) ascribes the "reduced effectiveness" to (i) the use of a grout mix in which sand blocked the injection process at too low a pressure, (ii) the effects of highly plastic fines in the soil which restricted rapid densification, and (iii) the unfavorable (sloping) site geometry.

5.2.2 Laboratory Testing: Effectiveness of Chemical (Permeation) Grouting (85).

The authors noted that cementation can exist in a sand naturally, or can be added artificially. In either case it is known to increase the resistance to liquefaction. They experimented with sand weakly cemented by various types of chemical grouts, and concluded that a saturated medium sand with 2% cement content and an U.C.S. of only 0.1MPa was stable to the point that it would require "a very large earthquake loading to liquefy". They also found that that the unit weight of the soil only had a significant impact on liquefaction potential at low strengths (less than 0.4MPa).

The inference is clear, therefore, that permeation grouting with even relatively weak grouts is very effective. However, such grouts are typically unstable in the long term, and may still prove expensive to install, when the drilling and injection costs are included. No case histories of permeation grouting for primary liquefaction control have been found, although the concept is implicit in Bell's description of chemically grouted "thrust blocks" in alluvium at Asprokremmos Dam, Cyprus (94).

5.2.3 John Hart Dam, BC: Seepage Cut-Off to Desaturate Soils (86, 87).

This 40m high dam on the Campbell River, Victoria Island, was completed in 1947. It was not specifically designed for earthquake resistance and the foundation soils were saturated and prone to liquefaction. The original design allowance for 0.1g maximum seismic horizontal acceleration compared with the revised estimated m.c.e. acceleration of 0.6g. The needs to keep the reservoir full (for generation), and to maintain high water quality in the reservoir and the river led to the concept of an insitu cut-off wall to desaturate the fine-medium sands of the embankment and the fluvioglacial sands of the river bed (Figure 25). In addition, some soil replacement and densification was also conducted in some other areas of the scheme.

Whereas most of the 400 lin. metres of cut-off was formed by conventional slurry trench diaphragm wall, jet grouting was used for the remaining 60m. John Hart Dam thus became the first dam in North America to be rehabilitated with the jet grouting technique, used under and around the embedded concrete structures.

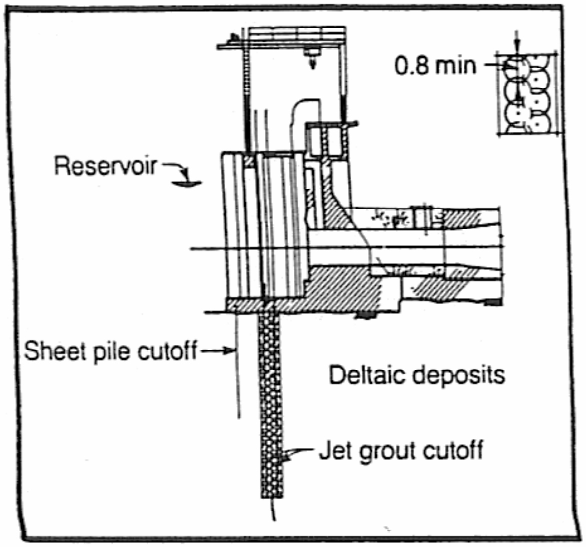


Figure 25. Jet grouting under concrete structure, John Hart Dam (87).

The diaphragm wall specifications called for a permeability no greater than 1 x 10^{-8}m/s, a 7 day UCS of 3-10MPa, and the ability to undergo a minimum of 10% strain without cracking, as verified by the 7 day test. A cement bentonite slurry comprising 12% cement and 4% bentonite (both by weight of water) was designed and used. The wall was constructed using both rope suspended and kelly grabs to a maximum depth of 29m to tie into a suitably impermeable horizon. Exacting quality control and assurance tests were conducted in both field and laboratory of all the materials used.

Regarding the jet grouting, the one fluid system (88) was used, featuring a grout injection pressure of 40MPa. A total of 203 columns, each about 0.8m in diameter, were formed in two rows, 14m deep, using the same cement-bentonite mix as the diaphragm wall. The jet grouting technique required only small diameter holes to be drilled through the base slab of the concrete structure. Prior to the production work, a field test was conducted in which columns were tested insitu for strength and permeability before being exposed and sampled for further testing.

All the verification and quality testing of the cut-off confirmed its construction to the designed standards. Piezometric and pump test data confirmed the effectiveness of the treatment upon completion.

5.2.4 Jackson Lake Dam, WY: First North American Use of SMW Seiko Method (89, 90).

Jackson Lake Dam is situated in Grand Teton National Park and was constructed in several stages from 1906 to 1916. It includes a northern embankment of hydraulic fill 1300m long, and 1.5-15m high, founded on fluviolacustrine and lacustrine sediments comprised of loose saturated gravels and sands with variable fines. Seismotectonic studies confirmed that the Teton fault zone was capable of a magnitude 7.5 event at an epicentral distance from the dam of 7km. The studies conducted in 1975 as part of the Bureau of Reclamation's Safety of Dams Program indicated that the embankment and its foundations were susceptible to liquefaction in this case. A major phase of modifications was put in hand from 1986 to 1988 including demolition of this embankment, and various foundation treatments prior to rebuilding.

Deep dynamic compaction, using wick drains, was successfully and economically used to densify the soil to 12m in the northern half where the subsequent embankment height would be less than 8m. (91) However, at greater depths in the rest of the area, and for 1200m of cut-off to depths of 33m, another technique had to be considered. Originally the idea was that jet grouting could be used to provide both liquefaction resistance, and the cut-off, but a proposal featuring the Seiko SMW (Soil Mixed Wall) Method proved superior in terms of cost and time. Although this work was conducted in essentially a "new site", it does equally have the potential for being conducted through an existing embankment. For the treated soil column method to be effective, the adjacent columns had to be fully contiguous, and to have a minimum shear strength of 1.4MPa.

In principle, grout is pumped down through each of the 2 or 3 hollow stem augers (non continuous) as they are simultaneously advanced and withdrawn to form "soilcrete" columns - 2% of bentonite by weight of cement is used to aid pumpability. The crane mounted augers are electrically driven. Volumetric central batch plants are operated semiautomatically to provide grout at rates and pressures appropriate to each auger's progress.

An inital phase of testing indicated that a cement content of about 300kg/m was necessary for a 1m diameter column at a w:c ratio of 1.35. For the production work, double auger machines were mainly used to provide 1m diameter columns to form contiguous hexagonal "cells" to isolate the soil mass against general liquefaction (Figure 26). Triple augers were used for the upstream cut-off and some of the deeper cells. A template was used to ensure the correct cell shape, and a shallow trench was preexcavated around it to contain overflow or waste.

In the double auger work, all columns were drilled and grouted twice to 100 - 150% theoretical volume to assure intercolumn continuity and contact. With the three auger system, a Primary and Secondary systems was used to provide a wall 0.6m wide composed of contiguous colums with at least 100% grout target volume. In such cases, 70 - 80% of the grout was injected on the way down and the balance on the way up with reversed auger rotation.

Columns as deep as 23m could be formed in one pass. Originally the waste was 15 - 30% of the total grout injected volume, and so early in the work the target volume was reduced to 220kg/m at w:c= 1.25.

Overall, the deep grid was completed over a "foot print" of 22,000m^2, to an average depth of 21m to provide a stabilized foundation block of about 440,000m^3. This involved 130,000m of columns and 35,000 tonnes of cement. The cut-off wall measured 25,000m^2 over a length of 1215m and consumed 7500 tonnes of cement.

Throughout the work, an aggressive qa/qc program of tests was conducted on wet samples, and on cores from set column material. The average core strength was just under 5MPa although it was

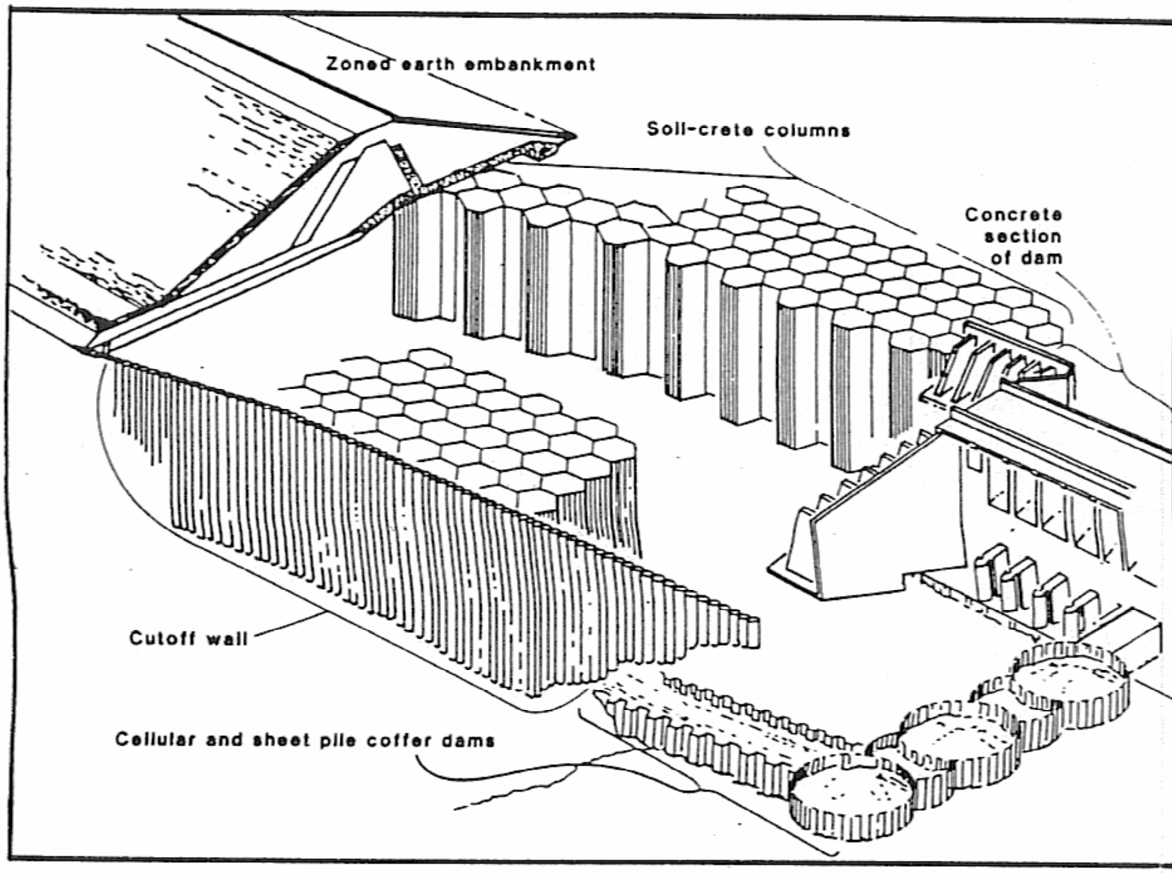


Figure 26. Cut-off wall, and hexagonal containment grid, Jackson Lake Dam (89).

noted that "coring of hardened column material and evaluation of core strength is difficult due to several factors including the length of time and strength before coring can commence, presence of gravel or unmixed soil in a column, problem of staying in a column for entire length, low tensile strength of treated soil, stresses imparted to core during drilling and the effects of handling core material."

Clearly the effectiveness of the honeycombe grid treatment - as opposed to an overall mass grouting - will only be truly demonstrated in the course of a major seismic event, although the work has apparently been constructed to design parameters. The overall effectiveness of the cut-off wall is still being evaluated by full scale areal tests including piezometers and seismic tomography.

6. Final Remarks.

This paper is basically a statement of the specialist geotechnical construction skills which are available to dam engineers involved in major rehabilitation schemes. With few exceptions, each technique has been tried and proven, often in the most arduous conditions. The exceptions are also important as they highlight newer concepts, used already in new construction, but perfectly suited to future remedial works.

With respect to rock anchors, ongoing developments in equipment, materials and techniques are keeping pace with the progressively onerous demands placed by p.m.f. and m.c.e. considerations. The technology of rock anchoring is, after almost 60 years, still growing rapidly, and will remain a vital tool in the repair of major concrete structures against both static and dynamic forces.

It has become fashionable in certain circles to discredit grouting as a reliable construction method, in both new and remedial work. However, given contemporary levels of knowledge of material chemistry and injection theory, there is no doubt that a grouting program can be made to work succesfully if properly designed, correctly constructed and equibly rewarded.

The technique of diaphragm walling to seal the cores and foundations of existing embankment dams has undoubtedly been the beneficiary (and probably one of the causes) of the apparent demise of grouting. Indeed the popularity of the method in the USA - strongly encouraged by developments in deep trenching equipment technology - is unmatched anywhere else in the world. However, closer analysis of the problems and uncertainties of both construction and performance does begin to raise questions as to the real benefits of this method, especially when the cost factor is weighed. It is not unreasonable to predict that in the next decade, the pendulum may begin to swing back to grouting techniques as cut-offs in and under existing structures.

Finally there is growing awareness and concern about the problems of seismically induced liquefaction. The restraints of conducting remediations insitu rule out many of the techniques used successfully in "green field" locations. Nevertheless, various grouting techniques have been proved to be effective in densifying or binding the soil on an areal basis, while grouting and diaphragm walling can be equally effective when forming cut-offs (to allow drying of saturated soils) or isolation grids (to prevent general liquefaction). This aspect of dam remediation may well prove the most challenging and volatile in the next few years.

Acknowledgements

The author thanks all the engineers who have published the case histories cited herein. Without such case histories, there can be no synthesis such as this. He also thanks Nicholson Construction of America for the time and resources (especially Wendy) to research and write this paper.

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