THE PRACTICE AND POTENTIAL OF GROUTING IN MAJOR DAM REHABILITATION

by

Dr. Donald A. Bruce
Nicholson Construction of America
P.O. Box 308
Bridgeville, PA 15017

ABSTRACT

As the average age of our nation's dams rises, and increasing effort is expended to maintain them safe and efficient, the demands placed on specialist geotechnical techniques are becoming ever more severe. For largely historical reasons, the technique of grouting is perceived in certain influential quarters as being somehow but fundamentally inferior to other options in resolving problems relating to seepage control, underpinning, and liquefaction mitigation.

It is easy to challenge this misconception when reviewing attitudes overseas: grouting now less an art than a science, enjoys the highest reputation when properly designed and conscientiously executed. However, in North America, the evidence seems to be more sporadic and less intense, perhaps more a reflection of the far-flung nature of the dam engineering community and its technical outlets, than a true indication of the expertise actually gained.

This paper provides a compendium of summary case histories of dam rehabilitation by grouting. The overall intention of the paper is to reassert the viability and potential of grouting as a reliable engineering tool for dam rehabilitation.

1. INTRODUCTION

Several recent publications (1, 2, 3, 4, 5) highlight the problems posed by the passage of time to the safety and efficiency of our hydraulic structures. They also underline the momentum within the dam engineering community towards analysis, monitoring and rehabilitation. This situation has stimulated the efforts of all interested parties, but especially those of the specialist contractor. Seduced by the prospect - not toally without logic or reality - of "cornering" a particular aspect of the dam rehabilitation market, this group has generated a growing number of novel, and occasionally exotic, solutions for seepage, movement and liquefaction problems. For example, in the U.S.A., very aggressive marketing, backed by the development of highly specialized and expensive construction equipment, has led to the advance of concrete diaphragm walls as a "positive solution" for seepage through and under embankment dams. This has been embraced with a degree of owner enthusiasm evident nowhere else in the world, despite significant cost implications and recent hints emerging of less than "positive" performance (6, 7).

This dynamism in certain directions would appear to contrast strangely with the popular engineering opinion and attitudes to the technique of grouting. It is a sad but realistic observation that the stifling procurement and contractural practices of the last four decades in this continent have led to an ossification of practice and a general ignorance of innovation elsewhere. Not surprisingly, therefore, grouting for dam rehabilitation in North America has generally neither the attraction nor the trust it enjoys and merits in other continents.

While grouting - like any other technique - cannot be regarded as a general panacea - examination of case histories proves that it can be a reliable, respectable engineering tool if properly designed, conscientiously conducted and equitably rewarded. Bad experiences with grouting can usually be traced to inappropriate applications, or poor execution by contractors, or major financial disputes at project's end. Not infrequently, all three factors display a mutual magnetism.

This paper is a positive statement about grouting, articulated by the presentation of details from several case histories. Mostly these projects have involved North American expertise in their engineering and execution: others are cited to illustrate key techniques not yet demonstrated in the U.S.A. or Canada. Between them they describe applications at concrete and embankment structures; in soil and in rock; and for seepage control, underpinning and liquefaction mitigation (Table 1). The list is, of course, not comprehensive, as many excellent remedial grouting projects, such as at Stewart's Bridge Dam, New York, in the 1950's, have not been widely publicized, and no space can be afforded the surficial repair of concrete structures by a wide array of proprietary methods and materials. Nevertheless, it is hoped that this collection will increase awareness and confidence in the technique, which, allied to current advances made by government agencies and contractors in materials and monitoring (8, 9, 10, 11), should reassert the potential of the science of grouting in dam rehabilitation. Further support for this stand may be drawn from the current, relative wealth of grouting textbooks (12, 13, 14, 15), specialty conferences (16, 17, 18), and annual teaching courses, such as at the University of Missouri - Rolla.

2. SEEPAGE CONTROL.

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 (19) 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 originally placed. Davidson (20) notes that "many of the most damaging dam failures in the United States have been caused by seepage induced piping", and named 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 (19) 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:

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Table 1. Classification of Case Histories Reviewed

- 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 (21) discussed the leaching of cementitious compounds from grout curtains as a function of the water-cement ratios, while Houlsby (22) 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, during construction.

Another key factor in trying to account for the need for remedial grouting was touched upon by Benzekri and Marchand (23):

> "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 (24) 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 (19) in this regard was Hoover (Boulder) Dam, a 222m high arch-gravity dam founded an imperfectly cemented volcanic breccia. The original grout curtain was extended from 40m to 130m deep in the valley bottom and 90m into the abutments after initial very high seepage volumes and pressures (Figure 1) were detected after first filling over 50 years ago.

As a final introductory point, Davidson (20) reviewed five case histories of remedial grouting under embankment dams. He drew the following conclusions which should be borne in mind as the details of this paper's projects are considered:

- 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.

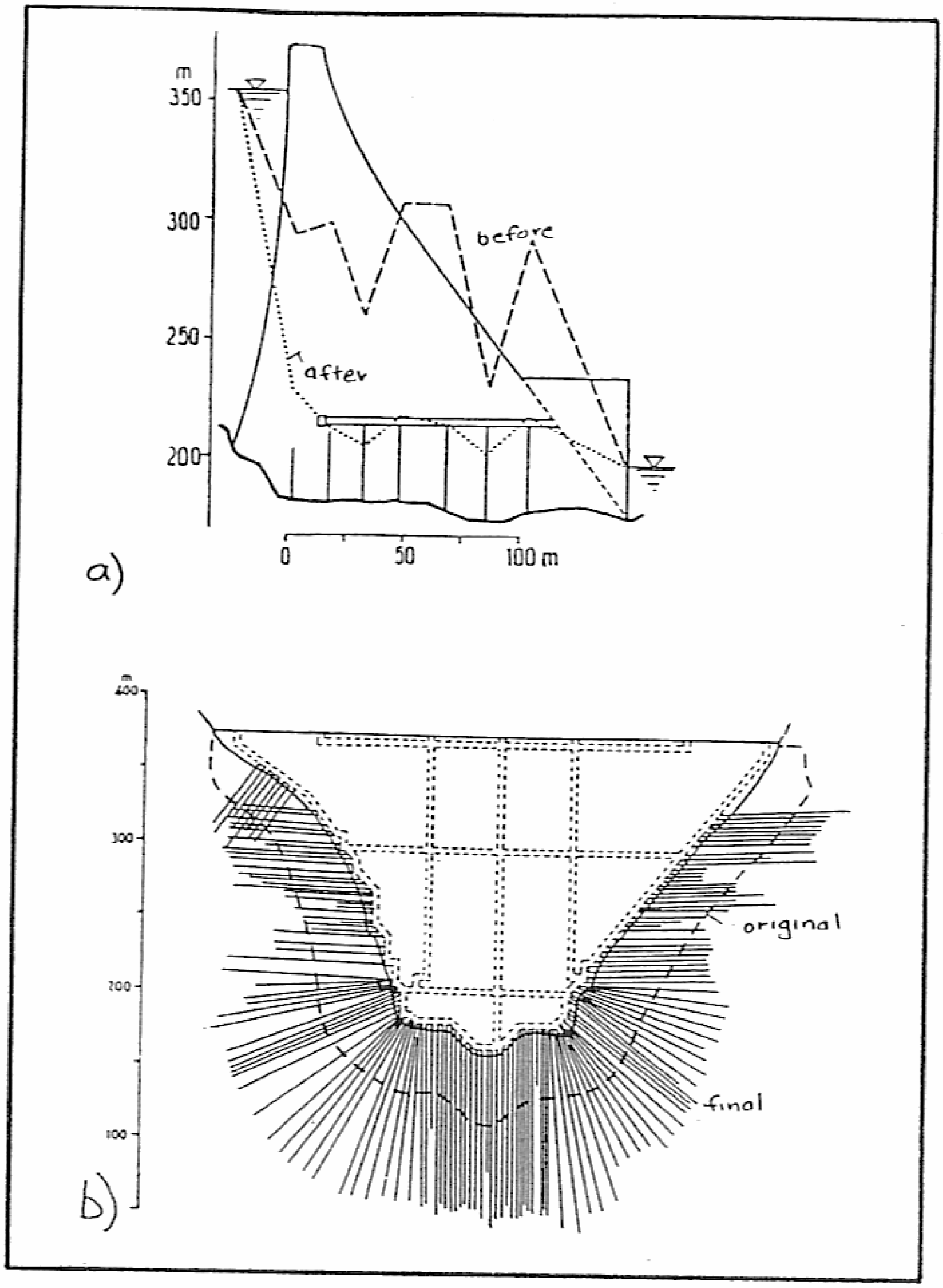


Figure 1. a) Uplift pressure caused by underseepage at Boulder (Hoover) Dam before and after remedial grouting; b) holes drilled for the remedial grout curtain and the limits of the original curtain (dashed line) (19).

- 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.

2.2. Concrete/Masonry Dams

2.1.1. Grouting of the Structure.

The first two examples reviewed in this section describe intensive grouting programs conducted to remedy dams with fundamental and pressing problems. In the first case, the amount of leakage was itself significant while, in addition, it was causing deterioration of the mortar of the masonry. In the second case, basic rebonding of horizontal construction joints was essential to eliminate dangerous uplift pressures and volumes.

On the other hand, the third example highlights another set of issues. There are literally hundreds of older concrete dams, usually in mountainous areas and with north-flowing rivers, which suffer from major degeneration of the concrete in the downstream face. This deterioration - which may extend for as much as 50 cm below the face - is due to freeze-thaw phenomena, fed by seeps of water through the dam's joints. These seeps may also be fueling the alkali-aggregation reaction process commonly found in these structures. Such structures are rarely in imminent danger from these seeps alone, and often the desire to remediate is based on cosmetic grounds. The work is rarely so eyecatching as other rehabilitation projects, and the technology largely revolves around the correct application of proprietary chemical grouts, usually polyurethane foams, either hydrophobic or hydrophylic. Given the widespread popularity of such works, the technique is included herein for completeness.

2.1.1.1. Aswan Dam, Egypt: Sealing with cement based grouts (15).

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 2). 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 calcium carbonate on the downstream face.

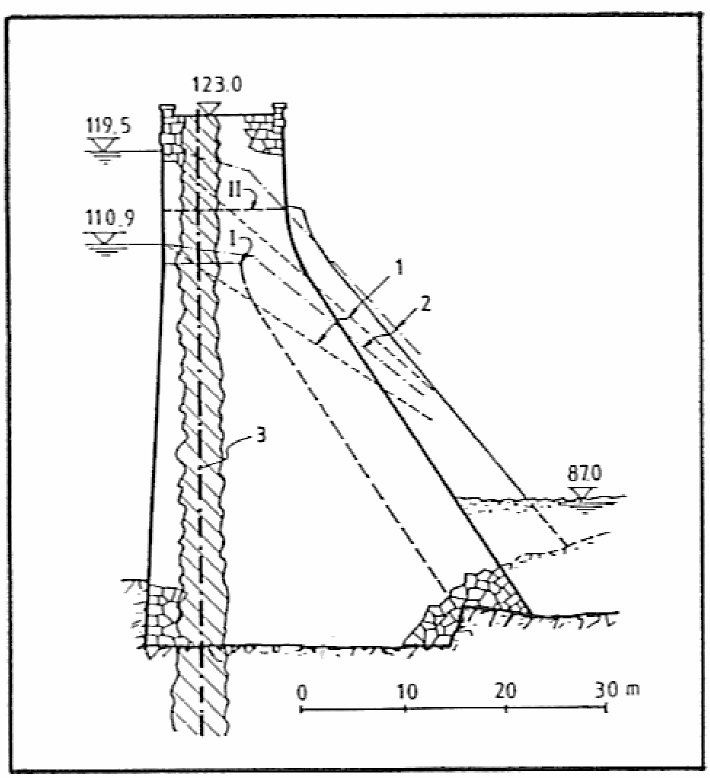


Figure 2. Typical cross section of Aswan Dam and the position of the grout curtain: I and II dam construction phases, 1 linear distribution of uplift pressures in the foundation plane, 2 uplift pressures measured in piezometers, 3 center line of grout curtain (15).

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

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

2.1.1.2. Santeetlah Dam, NC: Rebonding with epoxy resin grouts (25, 26).

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.

The behavior proved to be cyclic, reflecting reservoir levels, but by mid 1987 the seepage had reached record levels and was occurring 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 3). Of particular concern was the observation that seepage was occurring 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

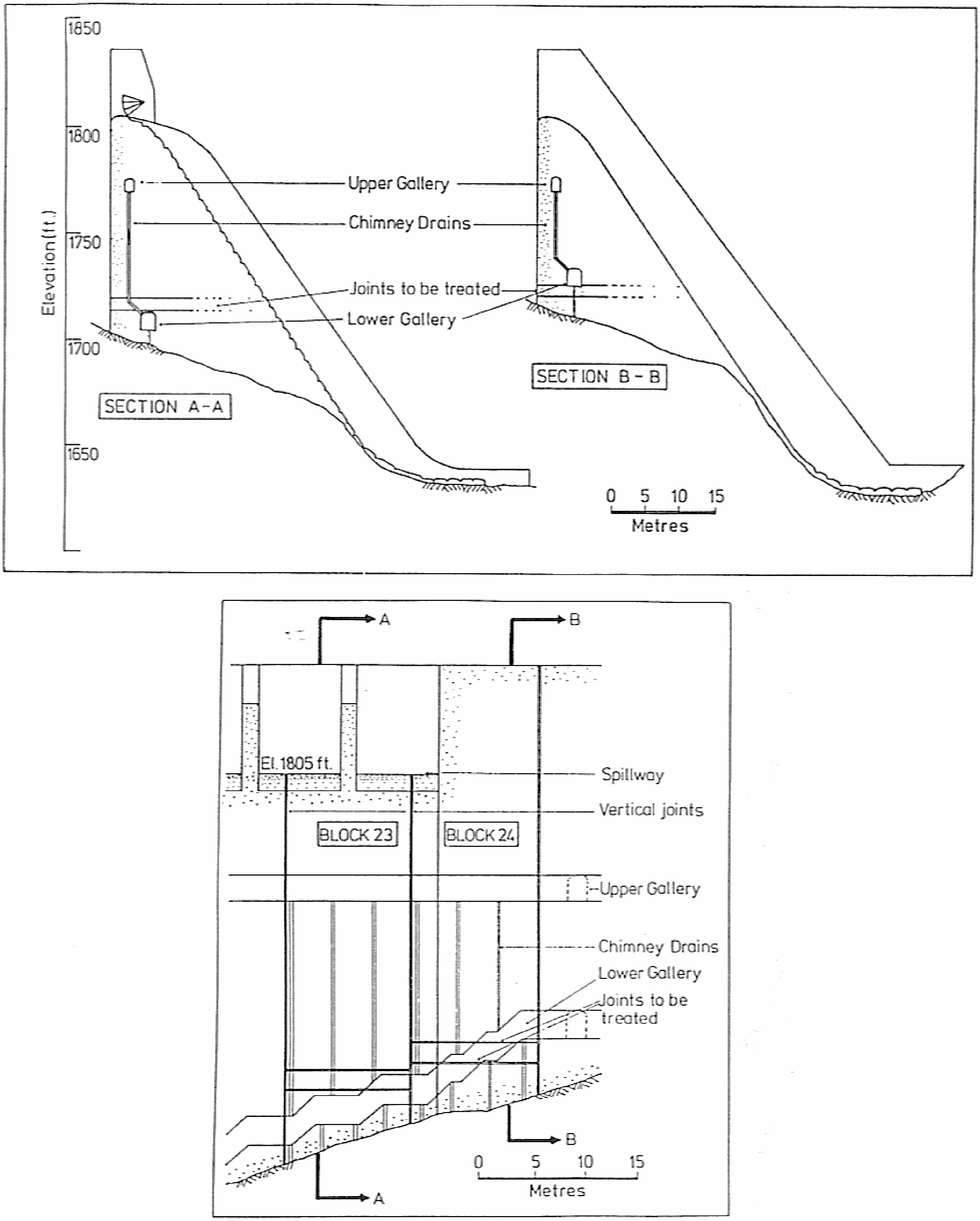


Figure 3. Galleries and horizontal joints for treatment, Santeetlah Dam (25).

- 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, and 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 (Fiogure 4). The special electro-hydraulic drill rig provided holes of 46mm diameter and cores of 36mm diameter.
- 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.

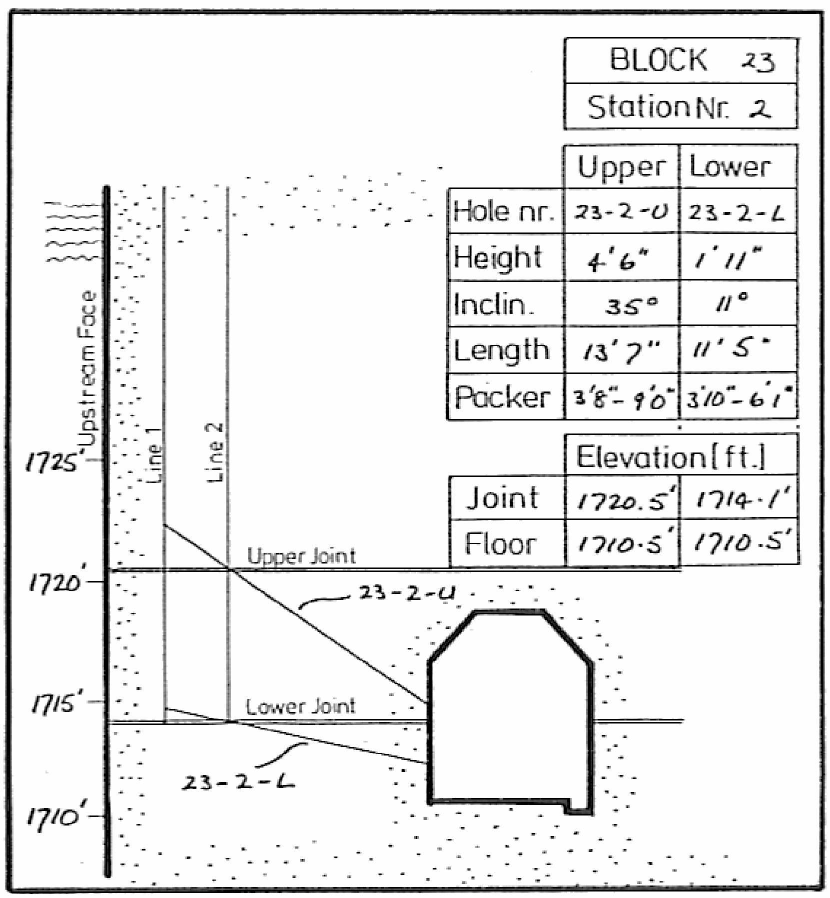


Figure 4. Typical section showing intended joint interceptions at one station, Santeetlah Dam (25).

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.

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.

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 (Generalized)	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

2.1.1.3. Easton Dam, CT: Sealing with polyurethane grout (27).

Built in 1926, this gravity dam is 312m long and 37m high. The downstream face exhibited large areas of surface spalling, joint spalling and efflorescence. Seepage was evident at the 16 vertical construction joints and varied seasonally, with greater volumes in colder weather. Freeze-thaw damage was visible as deep as 150mm, being especially severe in the upper 8m, subjected to fluctuating temperature and lake levels.

A two component polyurethane foam grout was placed into 150mm diameter holes a maximum of 11m deep, drilled vertically down through each joint. The particular grout used was reported as expanding to eight times its volume in contact with water, forming a sponge-like material. It also provided "attractive" adhesion, stretchability, and non-shrink properties. A temporary joint sealant was necessary to prevent the grout from escaping through the joint into the lake.

Seepage volumes were reduced by at least 80% and the balance was accommodated by an elasto-meric joint sealant on the upstream face.

2.1.2. 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 (32). 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 (12).

2.1.2.1 Tarbela Dam Auxiliary Spillway, Pakistan: Conventional descending stage grouting (28).

One of the major concrete structures at the Tarbela Dam Complex is the Auxiliary Spillway (Figure 5). 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

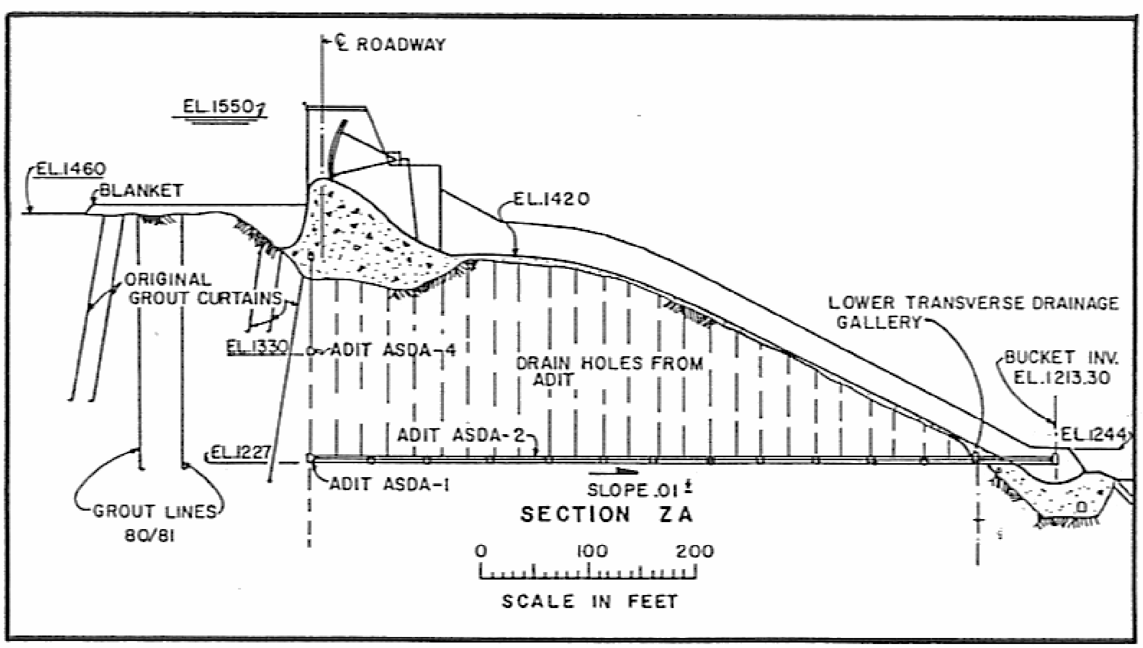


Figure 5. Auxiliary Spillway Section, Tarbela Dam (28).

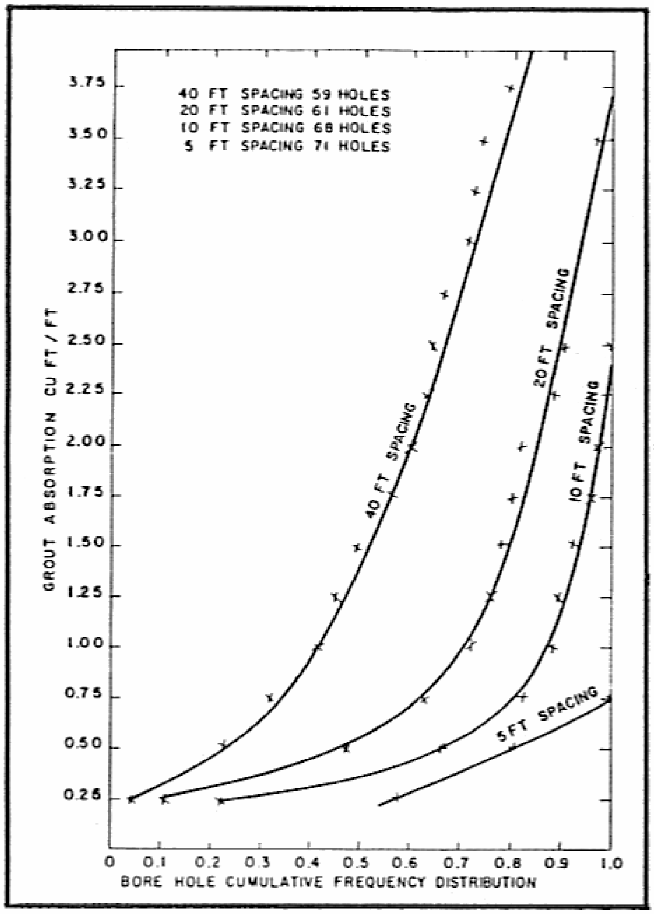


Figure 6. Effect of split spacings on grout takes, Tarbela Dam (28).

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 (by weight)
+ Bentonite = 1.5% weight of cement

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

Conventional descending stage methods (29, 30) 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 this remote 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 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 6). Grout was verified as traveling considerable distances in places by its entry into 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.

2.1.2.2 Tarbela Dam Power Tunnel 1, Pakistan: MPSP rock grouting (31).

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, above, the work is routinely conducted by the descending stage (downstage) method.

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 (Figure 7):

Step 1 - 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.

Step 2 - 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 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.

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

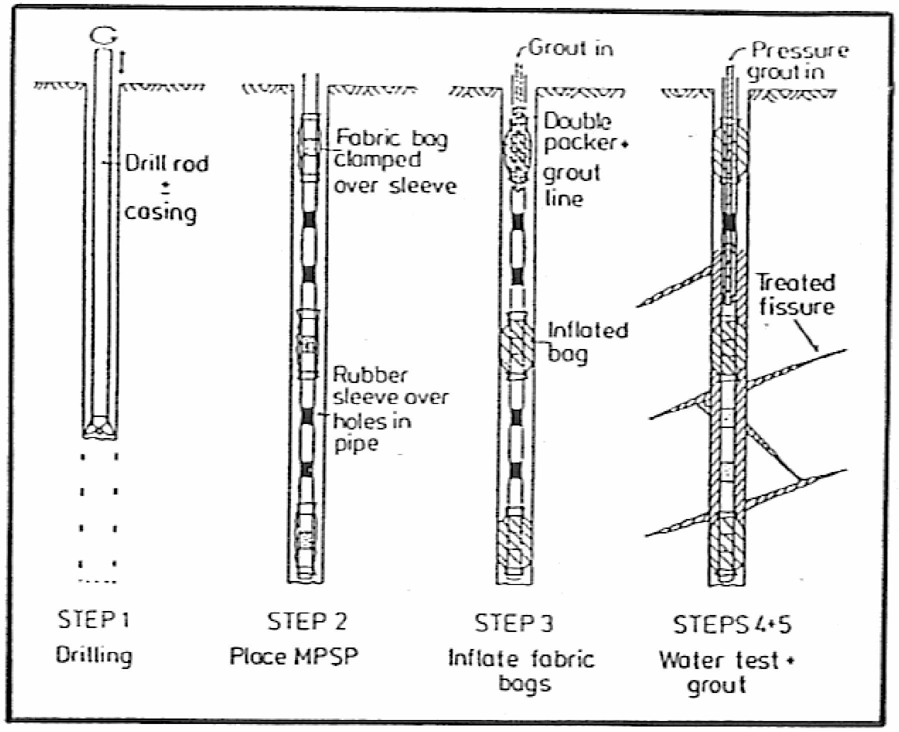


Figure 7. Installation sequence of MPSP (31).

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 8. 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

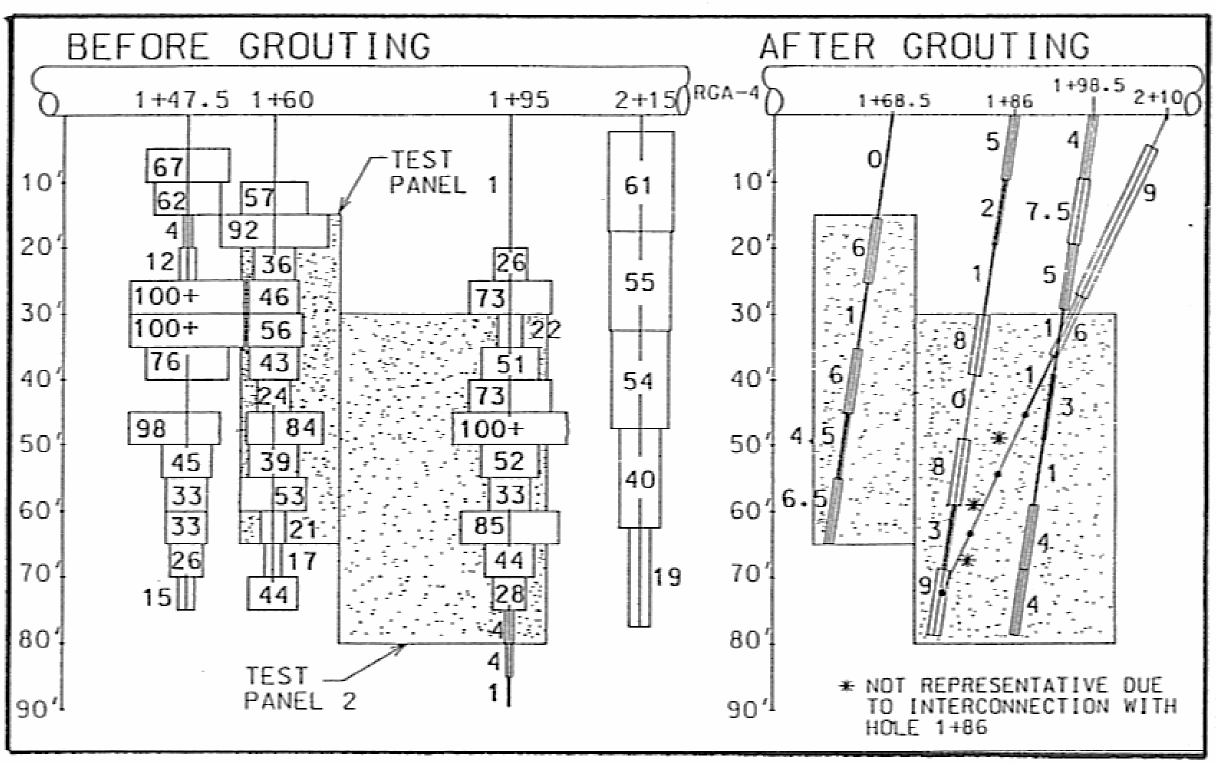


Figure 8. Location of test panels under RGA-4, and results of Lugeon tests, Tarbela Dam (31).

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 holes drilled after grouting.

2.1.2.3 El Cajon Dam, Honduras: Rock grouting from galleries (15).

This recently completed 232m high double curvature arch dam is founded on very karstic Cretaceous Limestones covered by Tertiary and Quaternary 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 9). The drilling and grouting were conducted from over 11.5km of galleries mined into the abutments and under the valley bottom.

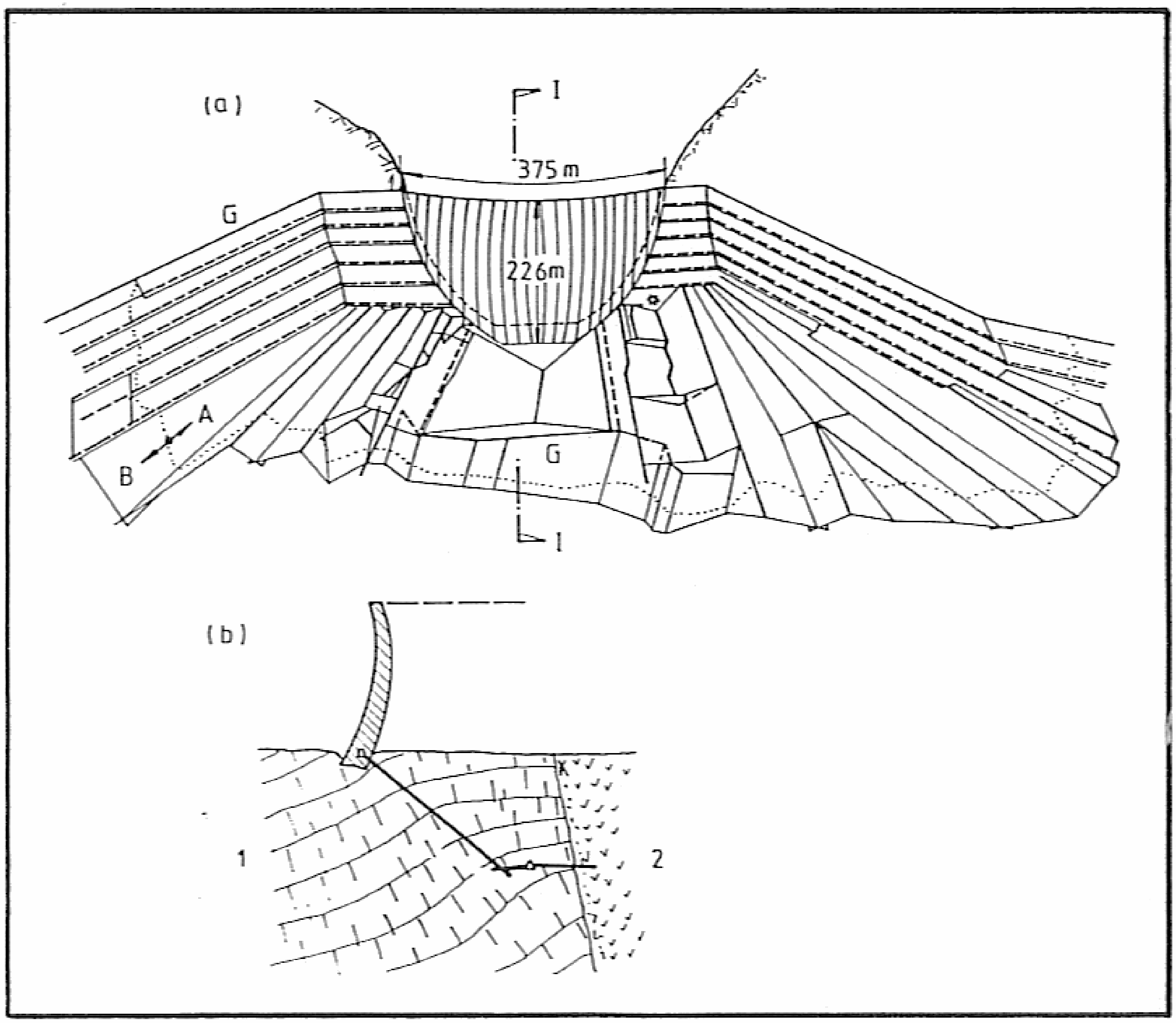


Figure 9. View from upstream on the grout curtain of El Cajon Dam: A-zone of limestones, B-zone of volcanics, (b) sketch of the cross section I-I; 1-limestones, 2-volcanic rock (15).

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.

2.1.2.4 Great Falls, and Tims Ford Dams, TN: TVA practice in asphalt grouting (32).

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.

2.1.2.5 Stewartville Dam, Ontario: Further developments with asphalt grouting (33).

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 10), 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

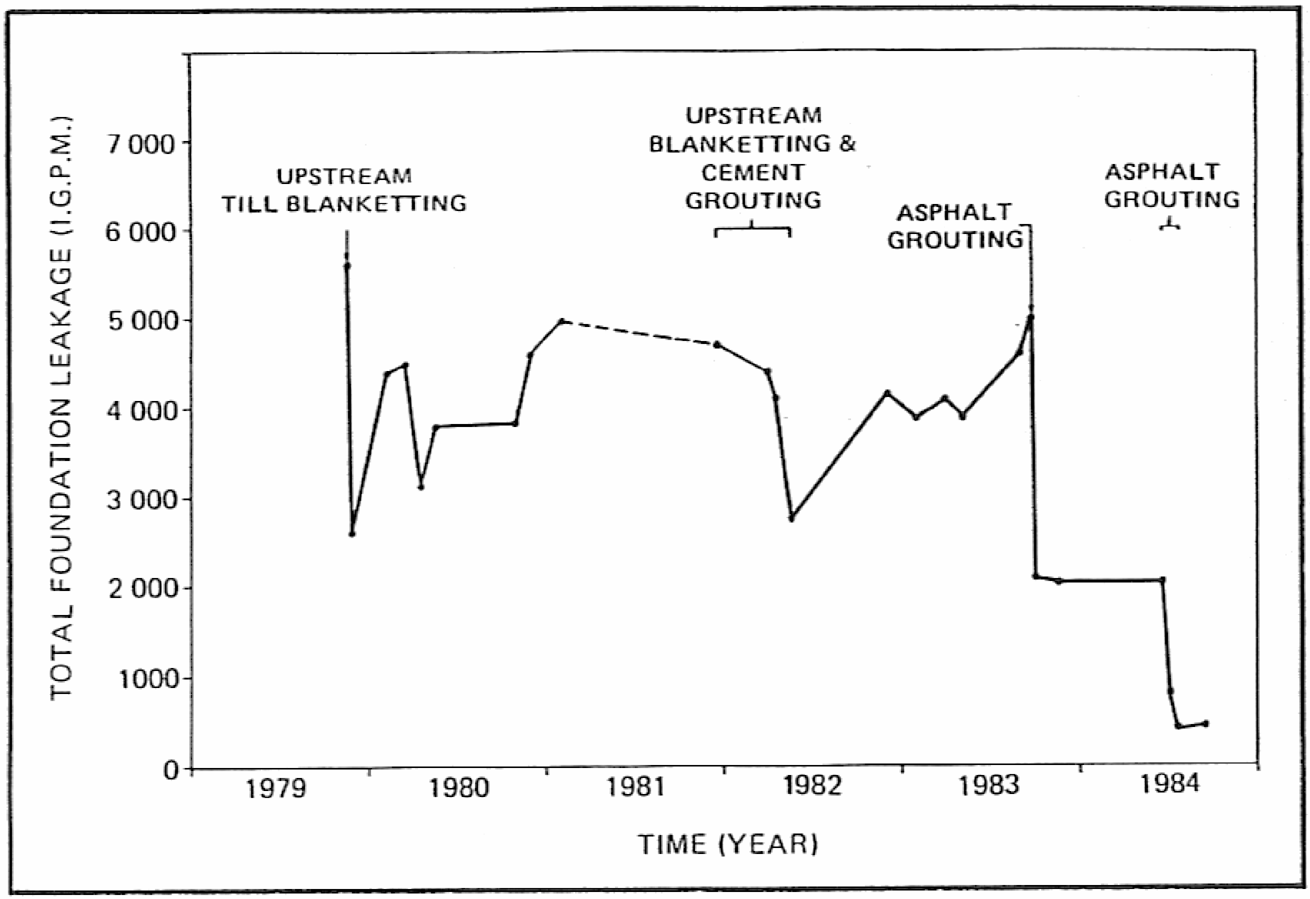


Figure 10. Effect of asphaltic grout treatment on leakage, Stewartville Dam (33).

- North: along the 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 11.

Basically the asphalt provides an initial plug to reduce water flow. The sand-cement grout, following several minites 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.

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.

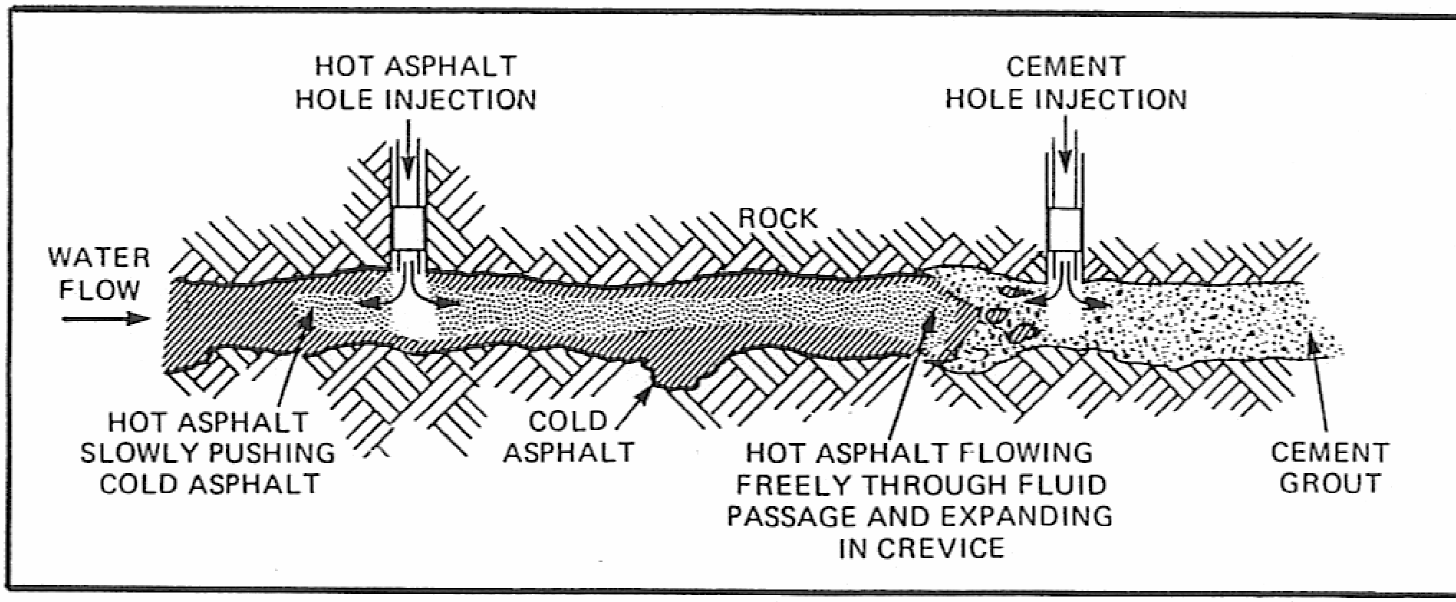


Figure 11. Schematic view of asphalt/ cement grouting (33).

2.2 Embankment Dams

In this section, case histories are reviewed to support the potential of modern grouting as a reputable and reliable remedial technique for embankment dams. However, one should note the caveat made by Von Thun (34) 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 techniques. In some ways, soil grouting is more complex and less precise given the tremendous range in both influential soil properties, and in grouting methods and materials. As a guide, papers by Naudts (35) and Bruce (36) will serve to lend perspective to the myriad of excellent books (e.g. Karol, 14), and conference proceedings (e.g., 16). 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.

The third major category of soil grouting - namely compaction grouting - does not primarily cause reduction in permeability, and so is not discussed in this section. However, its ability to densify soils in situ has been impressively exploited to combat liquefaction potential, and further discussion is provided in Section 4, below.

Overall, it would seem that the question of drilling and grouting the core of an existing embankment dam is a delicate and emotive issue. There are those who fear that hydrofracture of the core will result from the pressure of the flushing media used in the drilling, or from the grouting pressures exerted during injection. These fears typically lie in the minds of engineers who do not appreciate the significance of recent advances in overburden drilling techniques, especially the duplex variants (37), or do not enforce strict interim limitations on fluid grout **volumes** as well as **pressures.** Paradoxically, many of these engineers see no threat when designing concrete diaphragm wall cut-offs, wherein bentonite filled trenches of several hundred cubic meters volume have to be excavated through the very same core materials.

In this context, the recent experience at Mud Mountain Dam, WA, is a fascinating example (20, 38). There was a pressing need to seal the silty sand core of the dam, a structure 210m long and a maximum of 127m in height. The favored solution was a concrete diaphragm wall comprising 67 panels 1m thick and 7m long with a maximum depth of over 130m. However, construction was soon interrupted by massive losses of bentonite into open zones in the core. These 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.

It was then decided to carry out a massive grouting operation to repair the core - to permit the continuation of the diaphragm walling. Tube à manchette techniques (39) were used to "recompact" the core. Two rows of holes, 1.8m apart, and totaling 6000 lin. m of drilling, were installed. Over $3600m^3$ of cement-bentonite grouts were injected, some with silicate to accelerate set in especially severe locations. The diaphragm wall has since been completed without further incident.

In summary, therefore, modern drilling and grouting techniques, methods and materials can be exploited to seal the cores of existing embankments efficiently, economically and without danger to the stability of the structure.

2.2.1. Embankment Dams on Rock

2.2.1.1 Two Dams in Eastern Canada: Conventional sealing of core and rock (40).

Two rockfill dams for electricity generation exhibited worrisome trends within one year of first impoundment: abnormally high seepage volumes, 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 (37) 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.

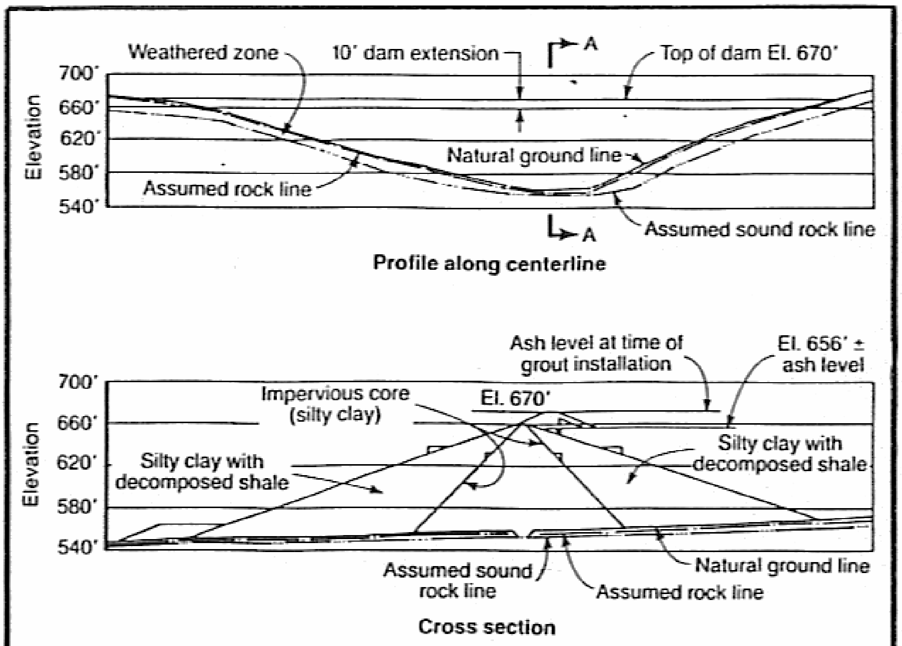
2.2.1.2 Ash Basin Number 2, PA: Advanced sealing of core and rock (41).

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 12).

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, and 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 $1x10^{-5}$m/s while the piezometric level in the dam was flat.



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 12. Layout of Ash Basin No. 2 (41).

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 in a ratio of 5:2, and a water solids ratio as low as 0.4.

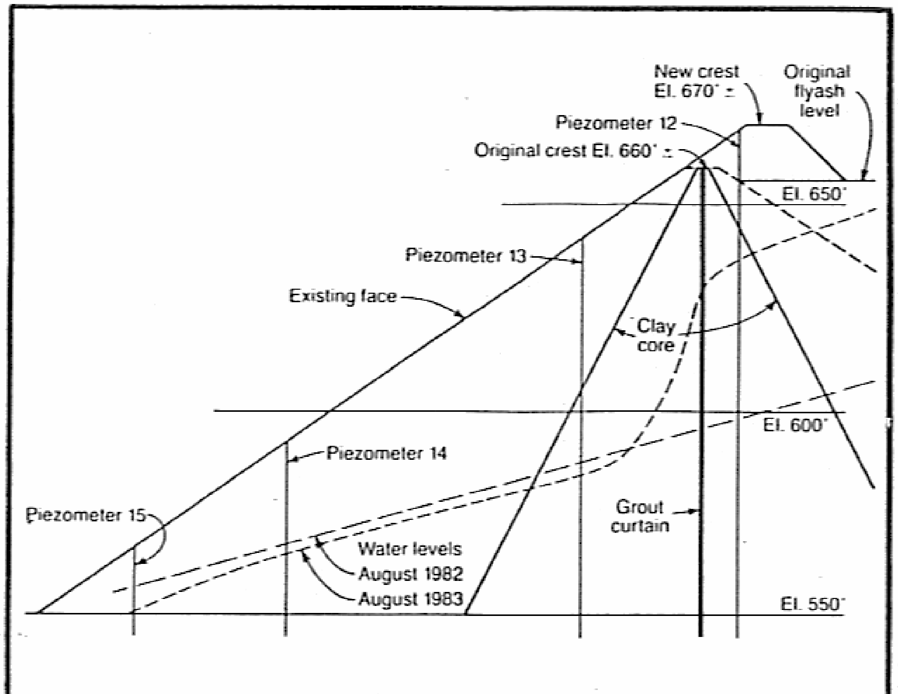
The chemical grout chosen was AC 400 due to its excellent gel time control, stability under flowing water conditions, lack of syneresis, low viscosity and relatively low levels of toxicity. Over 170,000 litres were used - the largest single application to that date - with a formulation of:

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 13). 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 13. Effect of grouting on piezometric level, Ash Basin No. 2 (41).

2.2.1.3. St. Mary Dam, Alberta: Conventional rock grouting (42).

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 (43). 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.

2.2.1.4 Unnamed Dam: Simple chemical grouting (44).

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.

2.2.1.5 King Talal Dam, Jordan: State of practice chemical grouting (45, 68).

King Talal Dam, originally a 100m high earth and rockfill dam on the Zarqa River, was recently raised to 116m. It occupies a deformed U-shaped valley, with the abutments and foundation consisting of gently dipping Mesozoic dolomite, limestone and marl overlain unconformably by sandstones with shale layers. Permeability testing of the sandstone indicated an average of 62 Lugeons, with 67% of tests being over 10 LU. The average permeability of the limestone was 27 LU. These high values were confirmed by injection tests featuring cement-bentonite grout. As a result the original treatment had included an extensive cement grout curtain, and 6800m^2 of concrete cut-off walls, in both abutments, supplemented with drainage curtains and blankets.

As part of the studies for the raising, an evaluation of the performance of the existing dam and its foundations was conducted. Attention was focused on solution phenomena in the carbonates, the high primary and secondary permeability of the sandstones, and their potential for piping, being moderately-weakly cemented and friable. Prior to raising, total leakage through the dam and foundation was within design expectations, but the sources of certain concentrated seepages in the geologically inferior left abutment had not been completely understood. A major study was conducted to this end.

With all these data, engineered solutions primarily to stabilize the potentially erodible sandstone, and secondly to control seepage, were evaluated and compared. These solutions included conventional cement grouting, jet grouting, tangent piles, and state of practice tube à manchette grouting (46). In the event, the last named was selected, and an intensive multirow grout curtain 4-6m wide installed from within the existing grouting galleries in the abutment. Cement based grouts were used to seal preferential seepage paths and larger fissures. A suite of chemical grouts, starting with sodium silicate plus organic ester reagent, were then used in inner rows to seal smaller fissures, and give intergranular cohesion, and sealing.

This excellent illustration of contemporary grouting practice is virtually complete, and all indications to date are of a very successful result. The publication of further data on the grouting and the control instrumentation and monitoring is eagerly awaited.

Similar methods are currently being used to seal ancient alluvials at Piedra del Aguila Dam, in Patagonia. This work, conducted from within galleries by vast resources from European and American firms, features a heavy concentration on electronic monitoring and analysis of drilling and grouting parameters. It is one of the largest and most intensive grouting operations of its type ever undertaken.

2.2.1.6. Unnamed Dam: Backfilling of old mine shafts (44).

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 major raising of Grimwith Dam, Yorkshire (30) where extensive old coal workings in the valley sides had to be grouted up before the reservoir could be raised to its new level.

2.2.2. Embankment Dams on Soil

2.2.2.1 Tarbela Main Embankment Dam, Pakistan: End of casing grouting (28).

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:

Depth of Casing	Excess Pressure (MPa)
>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.

2.2.2.2 New Waddell Dam, AZ: Jet grouting (47, 48).

Even allowing for the fact that jet grouting is tazzzahe newcomer 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 4.2.3 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 several sites, including Brombach Dam, West Germany, and Villanueva Dam, Spain.

One reason for this slow pace of adoption is quite simply the fundamental - and wholly understandable - 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 ± 150mm was permitted, and a jetting pressure of 35MPa was stipulated, at a w:c ratio of at least 1. A horizontal cut-off "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 jetting 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. of seepage. 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.

To redress the balance, however, it is only fair to note the rather more encouraging comments of Guatteri and Altan (49) in relation to the potential of jet grouted cut-offs.

3. SETTLEMENT CONTROL (Concrete Structures on Rock)

It is common in new dam construction to conduct intensive grouting to relatively shallow depths under the "footprint" of the new dam. This "blanket" or "consolidation" grouting is intended to reduce the overall settlement of the superimposed structure, and to ensure that such settlements are uniform and not differential. The blanket grouting also reinforces the effectiveness of any hydraulic cut-off, as it effectively widens it in the zone of greatest sensitivity.

In remedial operations, similar principles can be reapplied where unexpectedly large and/or differential movements have occurred in service, leading to cracking of the strucutre. In Europe, such problems have been noted in certain high concrete dams, usually of the double curvature thin arch type. Repair programs have strongly featured grouting - sometimes with epoxy resins - as a major element in the rehabilitation (25). In the United States, there have been problems with certain lock structures, founded on bedrock permitting unacceptably large deflections during daily or annual loading cycles. This section describes the typical approach.

In addition, structural distress can also be caused by simple washout of founding material. In such cases, the grouting is relatively "low technology" - being intended as a simple void filling operation - even though the execution may involve difficult underwater support efforts by diving personnel. The first example in this section illustrates the scale such applications can reach.

3.1. Old River Low Sill, MS: Void infill under structure (50).

The river control 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 as one of the emergency actions taken. 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, all comprising various amounts of barite, cement, bentonite and water in three basic mixes:

OR13: non sanded, injected in small amounts with the intent to prevent later erosion of the sand in the other mixes.

OR5: sanded, the next mix, also with 170-350kg of cement per cubic meter of grout.

OR23: "topping off mixture" including 700kg of cement per cubic meter 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, inter al., this confirmed the base of the piled cut-off was in fact 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.

3.2. John Day Lock and Dam, WA: Conventional rock grouting (51).

This structure was built on the Columbia River 170km from Portland, OR, from 1958-63. The lock is 206m long and 26m wide with its 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 and spalling were noted near the base of two of the monoliths in the riverside wall (Figure 14). 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. These produced high tensile stresses leading to fracture of the inadequately reinforced structure. By 1979, six of the eleven monoliths were affected over a length of 133m, 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 (to eliminate the suspected cause of the problem), and

- structural repair of the monoliths themselves, without significantly affecting the operation of the facility.

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 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, to an average depth of 22m along the 168m length. The holes were at 3m centers, inclined 5-25° off vertical, and in 6 rows, 1.8m apart. They toed 3m into the lower basalt. Holes furthest from the lock were grouted first, as a curtain.

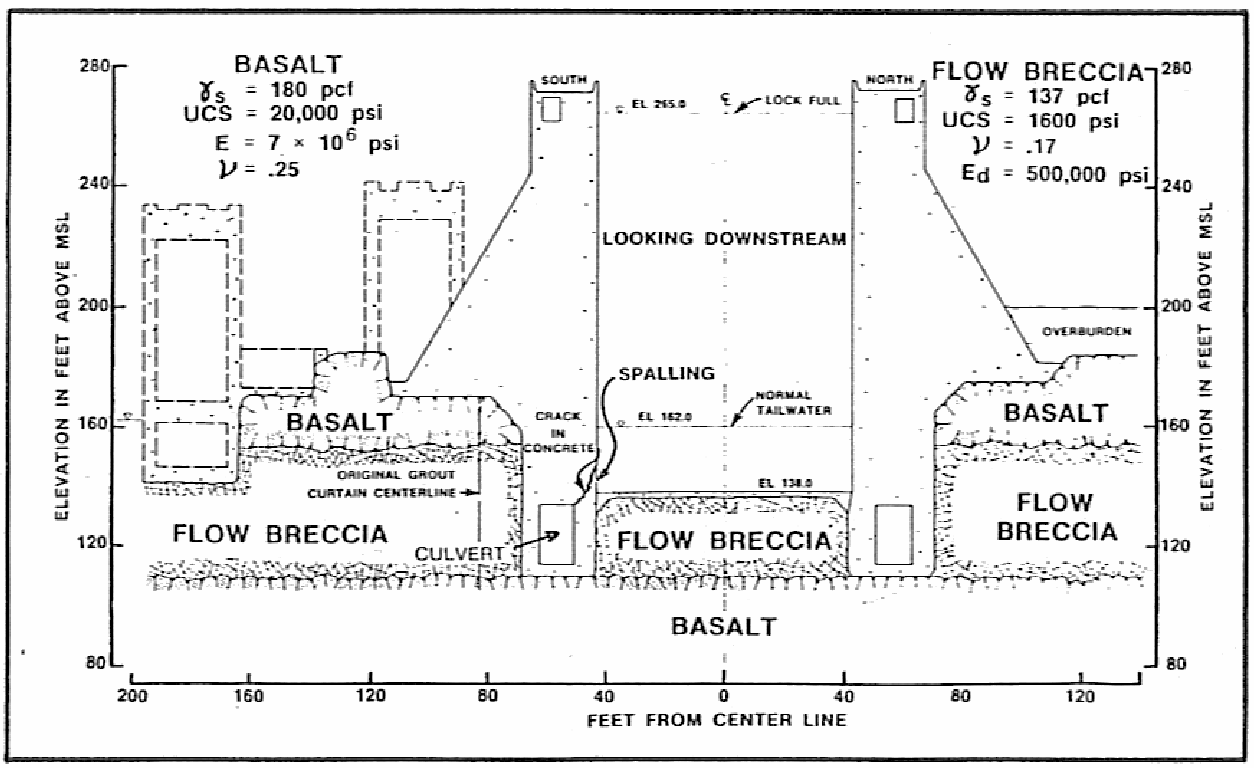


Figure 14. Cross section of lock, John Day Lock and Dam (51).

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 effectiveness of the treatment during its execution.

Post grouting rock modulus was measured 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 reduced by 50-60%. 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 2.1.2.2) would have been ideal.

The later phases of the repair featured 73 prestressed rock anchors and the injection of structural epoxy resin into the fissure network.

3.3. Little Goose Lock and Dam, WA, and Savage River Dam, MD: Further examples of consolidation grouting (10).

Each Corps of Engineers 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. LIQUEFACTION CONTROL

It has been calculated that 650 of the 2000 Federally owned dams are located in highly seismic areas (52). Similar proportions of private and state owned dams are equally threatened: 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 (53) 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 option (viii) is 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 (52) 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 (53):

- Location, area, depth and volume of soil involved
- Soil types, properties and conditions
- Site conditions
- Seismic loading
- Structure type and condition
- Economic and social effects of the structure
- Availability of necessary materials such as sand, aggregates and gravel
- Availability of equipment and skills
- Cost
- 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:

Group 1. Techniques which have a proven record of improvement, but which could not possibly be used in or under an existing dam without prior demolition. Included are dynamic consolidation, compaction piles, stone columns, vibroflotation, and blasting (54).

Group 2. 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.

Group 3. Techniques within the scope of this paper but which have similarly not yet been field evaluated e.g. insitu soil reinforcement (55).

Group 4. 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 methods have been used to provide the necessary aspect of ground improvement or treatment.

4.1. Pinopolis West Dam, SC: Soil densification by compaction grouting (56, 57).

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 principles have been used (58) since the early 1950's in the U.S.A. for settlement control and remediation, but only recently have their potential for densifying soils against liquefaction been 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 (56).

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

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

The distribution of grout volumes injected is shown in Figure 15. 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 ..."

- 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 (56) 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.

4.2. Laboratory Testing: Potential effectiveness of chemical (permeation) grouting (59).

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 (60).

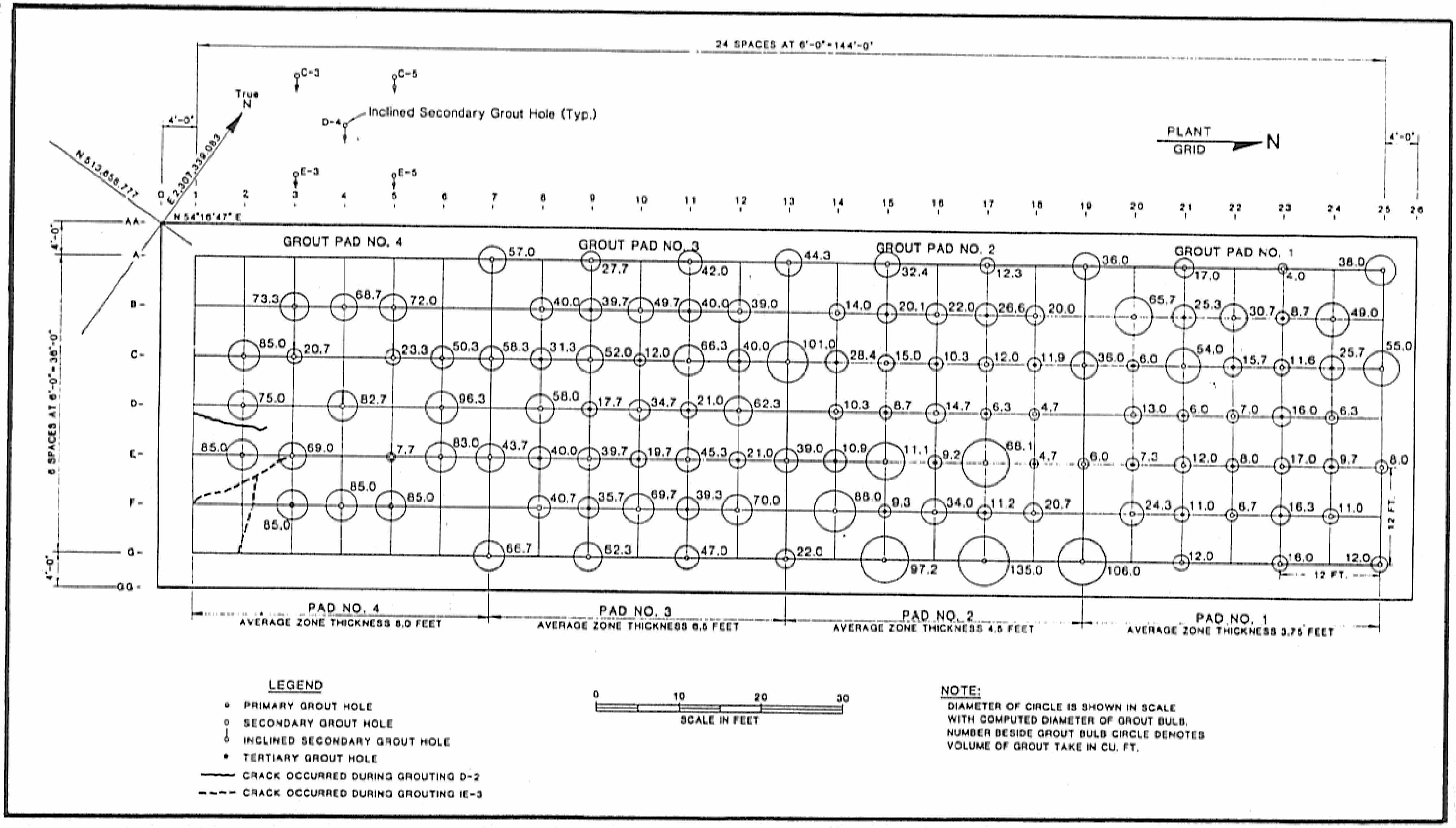


Figure 15. Compaction grout takes, Pinopolis West Dam (57).

4.3. John Hart Dam, British Columbia: Cut-Off to desaturate soils (61, 62).

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 in 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 16). 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.

The one fluid system of jetting (36) 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. This comprised 12% cement and 4% bentonite (by weight of water), with a designed 7-day strength of 3-10MPa, a permeability less than 10^{-8}m/s and the ability to undergo a 10% strain without cracking. 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. As in the other cases (Section 2.2.2.1.), no deleterious effects to the core or the structure were caused by the jet grouting activities.

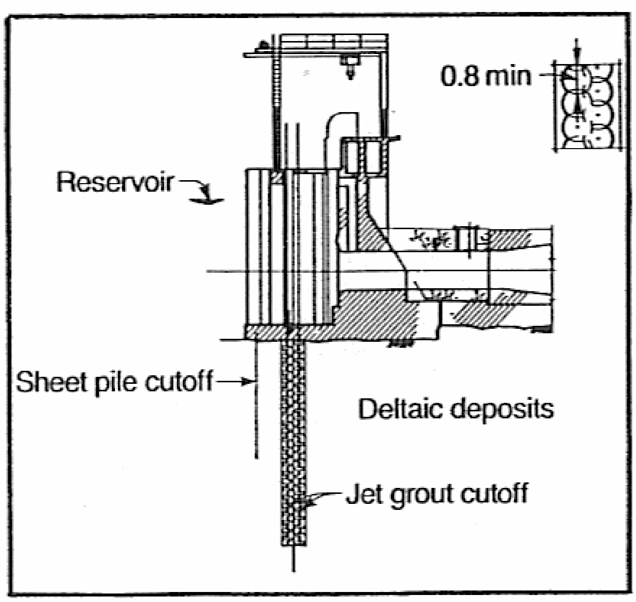


Figure 16. Jet grouting under concrete structure, John Hart Dam (62).

4.4 Jackson Lake Dam, WY: First North American use of SMW Seiko method (63, 64, 65, 66).

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. 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 17). 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 system was used to provide a wall 0.6m wide composed of contiguous columns 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.

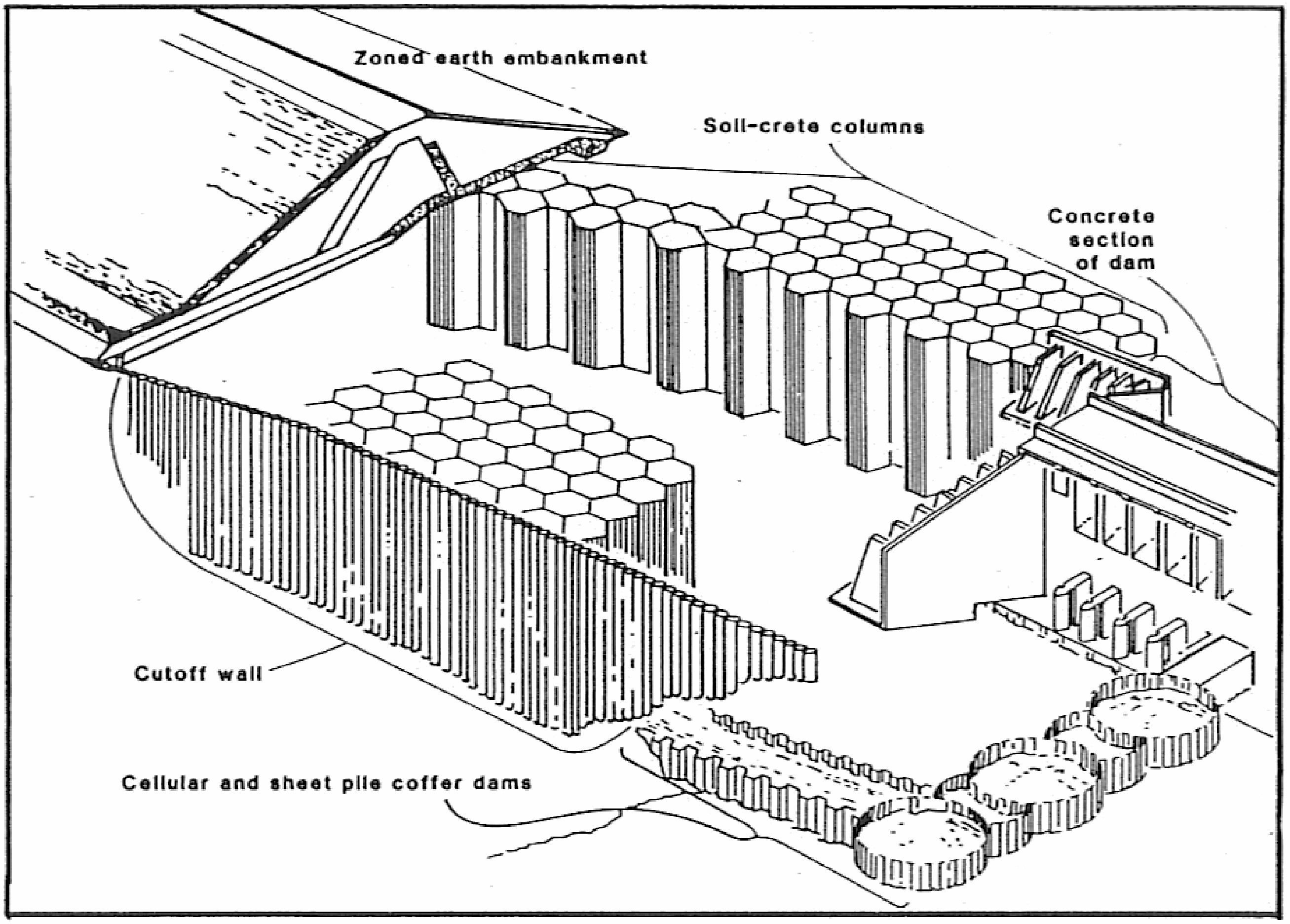


Figure 17. Cut-off wall, and hexagonal containment grid, Jackson Lake Dam (63).

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 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.

Most recently (67), another major application of the method has been announced, in conjunction with the major rehabilitation of Cushman Dam, WA. The SMW technique will be used to form almost $4000m^2$ of cut-off wall adjacent to the spillway, to a maximum depth of 25m through tough glacial till and loose sands and gravel. This option was technically superior to sheet piling, and more economic than conventional slurry trenching.

5. FINAL REMARKS.

This review cites successful case histories involving the proper application of grouting techniques to remedy defects caused by seepage, movement or earthquake susceptibility. The range of rock grouting methods (conventional stage to MPSP system), and soil grouting methods (permeation, compaction, jet) has been illustrated, as has been the range of materials used (cements to chemicals). Examples have covered both concrete and embankment structures.

Even then, this review cannot hope to be comprehensive in terms of listing and describing such remedial case histories. However, it is hoped that the **framework** of the paper will prove to be a durable contribution, which will help put unresearched, or future, case histories in perspective. In this regard, a current task of the USCOLD Foundations Committee in assembling a register of dams rehabilitated by specialist geotechnical construction techniques may well be eased.

Fellow specialists should feel encouraged not only to confidently use grouting to repair dams, but also to publish the results as widely as possible.

Acknowledgements

The author extends his thanks to all 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 providing the facilities to research and write this paper.

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