THE SEALING OF CONCRETE DAMS AND THEIR FOUNDATIONS: TWO NEW TECHNIQUES

by:

Dr. Donald A. Bruce, Technical Director
Nicholson Construction of America

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NICHOLSON CONSTRUCTION of AMERICA
P.O. BOX 308
BRIDGEVILLE, PA 15017 (412) 221-4500

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ABSTRACT

Grouting methods have been used since the nineteenth century to repair dams and improve their foundations. A wealth of national experience, plus a recent increase in the number of textbooks available, ensures that the general principles are well known. This paper, however, describes two new grouting techniques which may not be so well known. One, the MPSP System, is essentially a rock grouting method which provides for high quality treatment of rock masses which, on account of their geology or structure, can be regarded as "difficult". The other is the RODUR^{sm} method of epoxy resin injection. This can be used in certain rock foundations but its prime application and value is in the sealing and bonding of major concrete dam structures. This is often accomplished under the most arduous hydrologic conditions. The principles of each method are discussed, and illustrated with reference to major case histories.

1. INTRODUCTION

The technical literature has never been richer on the subject of the repair of concrete dams and their foundations. The recently published book by Jansen (1989) provides an excellent overall survey, while theme conferences sponsored by professional bodies have been particularly relevant and informative. For example, in the field of grouting, the so-called "New Orleans Conference" of February 1982 is regarded worldwide as a milestone achievement, while the 1985 ASCE session on "Issues in Dam Grouting" at Denver provided a comprehensive update on materials and operations. Within the last few years, textbooks on rock grouting have appeared from Germany (Ewart, 1985), Yugoslavia (Nonveiller, 1989), Czechoslovakia (Verfel, 1989), and Australia (Houlsby, 1990).

* Technical Director, Nicholson Construction of America
P.O. Box 308, Bridgeville, PA 15017

It is the purpose of this paper to highlight some important new developments which have escaped general attention in this country due to their novelty, or foreign origins or their promotion by or in only one segment of the market, e.g. specialty geotechnical contractors. So, with respect to grouting foundations, the MPSP system is described. This is a new method for grouting particularly difficult rock formations, although it is completely compatible with the current trends in materials, monitoring, and instrumentation described elsewhere. For the grouting of existing concrete structures, the RODUR^{sm} method is introduced. Although cement and chemical injection of concrete dams has been conducted for years, this epoxy resin system not only can seal water flows under full hydrostatic conditions, but it also bonds the structure together again. First North American applications in each of these techniques are reviewed.

2. THE MPSP SYSTEM FOR ROCK GROUTING

2.1 Background

The fissure grouting of rock masses is typically conducted by some type of stage grouting procedure (Figure 1). In "downstage grouting," grout holes are advanced by drilling a certain length, usually 3-5 m, grouting it, and then repeating the process after the grout has set, until final depth is reached. The packer may be kept at the top of the hole, or moved successively downwards to the top of each new stage in turn. In "upstage grouting," the hole is drilled to full depth in one pass and injected progressively in successive stages from the bottom upwards through a down-the-hole packer. Houlsby (1982) also describes "circuit grouting, downstage," which is not dissimilar to downstage without packer. This system has proponents in the U.S.A. (e.g. Burwell, 1958) although Ewart (1985) views the method's effectiveness "with some skepticism," being concerned about hydraulic fracturing "somewhere in the upper stages." Houlsby (1982) also describes it as "difficult, prone to blockages and very expensive." The major advantages and disadvantages of the two basic approaches - upstage and downstage - are summarized in Table 1. Their technical or financial balance on any particular site should logically dictate the final choice of method, but it would seem that tradition and bias often prove at least as decisive.

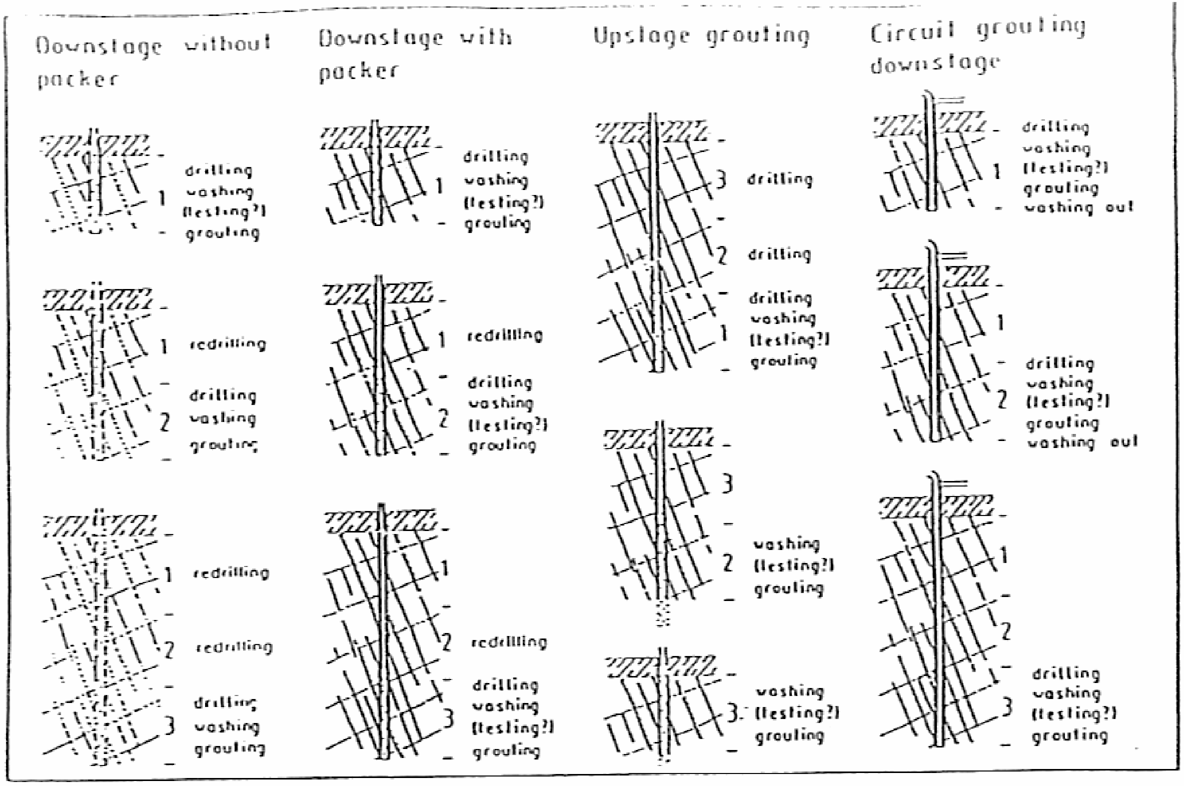

Figure 1. Conventional stage grouting methods for rock (Ewart, 1985, after Houlsby, 1982).

	DOWNSTAGE	UPSTAGE
ADVANTAGES	1. Ground is consolidated from top down, aiding hole stability and packer seating and allowing successively higher pressures to be used with depth without fear of surface leakage.	1. Drilling in one pass.
	2. Depth of the hole need not be predetermined: grout take analyses may dictate changes from foreseen, and shortening or lengthening of the hole can be easily accommodated.	2. Grouting in one repetitive operation without significant delays.
	3. Stage length can be adapted to conditions as encountered to allow "special" treatment.	3. Less wasteful of materials.
		4. Permits materials to be varied readily.
		5. Easier to control and program.
		6. Stage length can be varied to treat "special" zones.
		7. Often cheaper since net drilling output rate is higher.
DISADVANTAGES	1. Requires repeated moving of drilling rig and redrilling of set grout: therefore, process is discontinuous and may be more time-consuming.	1. Grouted depth predetermined.
	2. Relatively wasteful of materials and so generally restricted to cement-based grouts.	2. Hole may collapse before packer introduced or after grouting starts, leading to stuck packers and incomplete treatment.
	3. May lead to significant hole deviation.	3. Grout may escape upwards into (non-grouted) upper layers or the overlying dam, either by hydrofracture or bypassing packer. Smaller fissures may not then be treated efficiently at depth.
	4. Collapsing strata will prevent effective grouting of entire stage, unless circuit grouting method can be deployed.	4. Artesian conditions may pose problems.
	5. Weathered and/or highly variable strata problematical.	5. Weathered and/or highly variable strata problematical.
	6. Packer may be difficult to seat in such conditions.	

Table 1. Major advantages and disadvantages of downstage and upstage grouting of rock masses.

However, there are often conditions in which neither stage grouting method can be relied upon to provide an effective and reliable treatment. For example, in downstage schemes, the presence of very fissured, granular or fragmented rock (e.g. "sugary limestone") may result in caving of the stage after drilling, and before grouting can be executed. Thus in the worst case only the uppermost part of that stage will be treated, and the lower section will remain ungrouted and most probably cause similar problems during later redrilling operations. Likewise, the presence of such strata, voids and/or soft infill zones will prevent upstage grouting being practical: packers will be very difficult to "seat" efficiently, and may permit grout to bypass upwards, leading to ineffective treatment or trapped packers.

Such geological conditions reflect more typically the problems associated with soil grouting, for which the standard "high tech" approach is the tube à manchette (sleeved pipe), shown in Figure 2. A fundamental feature of the tube à manchette operation is the necessity to rupture the "sleeve" grout, thus permitting egress of grout into the surrounding soil. However, in all but the softest rocks, the lateral restraint afforded by a rock mass is sufficient to prevent the sleeves opening (to allow the flow of grout into surrounding fissures). In addition, the nature of the system - involving the use of a stable cement-bentonite sleeve grout - essentially plugs off, for a short but critical distance, those fissures which are intersected by the borehole, thus further circumscribing the possible effectiveness of the system. In this regard, the finely fissured soft shales very successfully treated at Grimwith Dam, Yorkshire (Bruce, 1982) could well be regarded as an upper limit for rock mass quality in terms of effective sleeved pipe grouting.

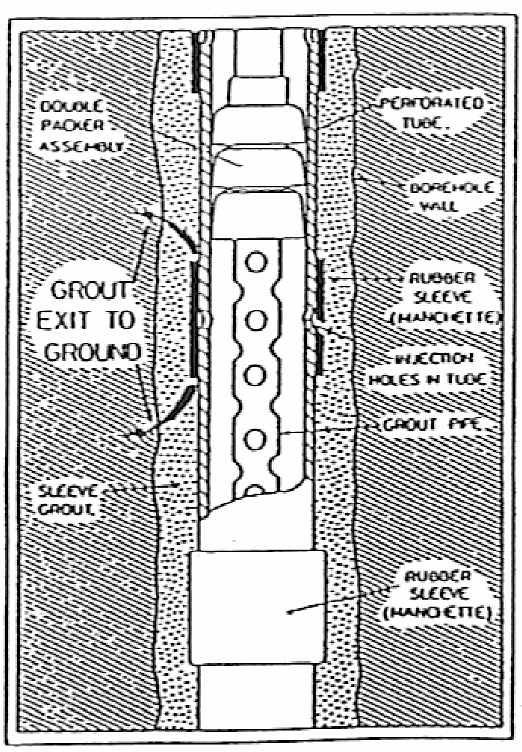

Figure 2. Operating principle of the tube à manchette (sleeved pipe) system.

It was against this background of providing high-quality treatment of ground which would otherwise frustrate the effectiveness of these conventional methods, that Rodio developed the MPSP system.*

2.2 Installation and Operation

MPSP owes much to the principle of the sleeved pipe 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 the sleeved pipe system, 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 grout pipe, either at regular intervals (say 3 to 6 m) 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 3):

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 100-150 mm.

Step 2 - The MPSP is installed. Pipe details can be varied with requirements, but a typical choice consists of a steel pipe, 50 mm o.d., with each length screwed and socketed. Each 5 m pipe has three 80 mm long, rubber sleeves equally spaced along the length, protecting groups of 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 600 mm long (Photograph 1). 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.

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* The MPSP method was devised by the Rodio Group of Companies, which controls the rights to its use. Nicholson Construction Company is the licensee in North America. This is equally the case with the RODUR[sm] system, described later.

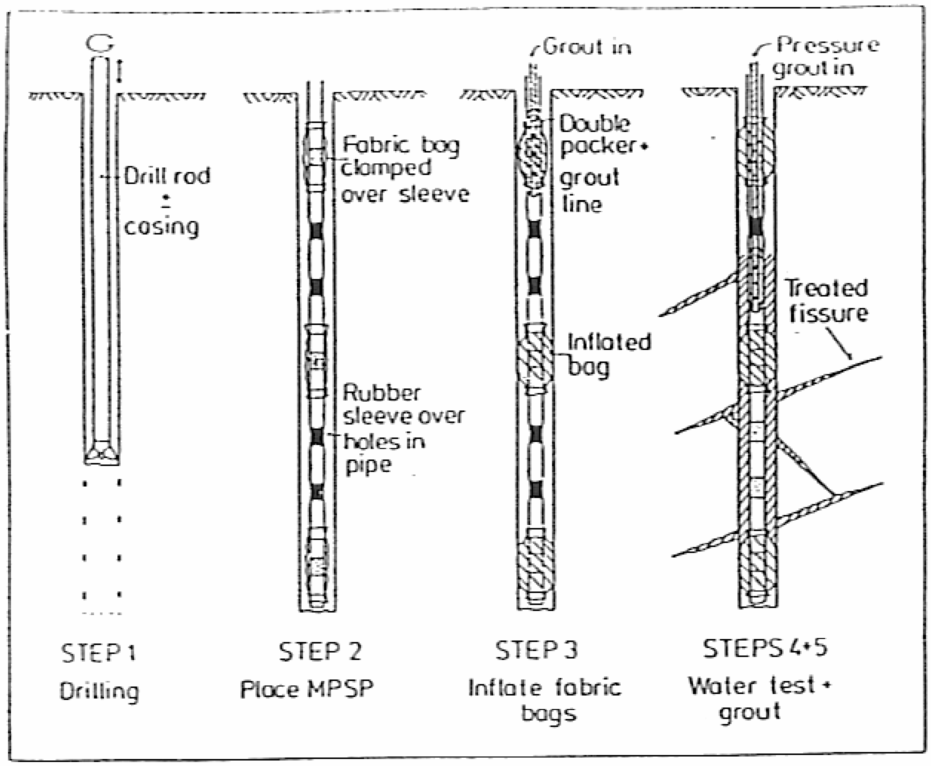


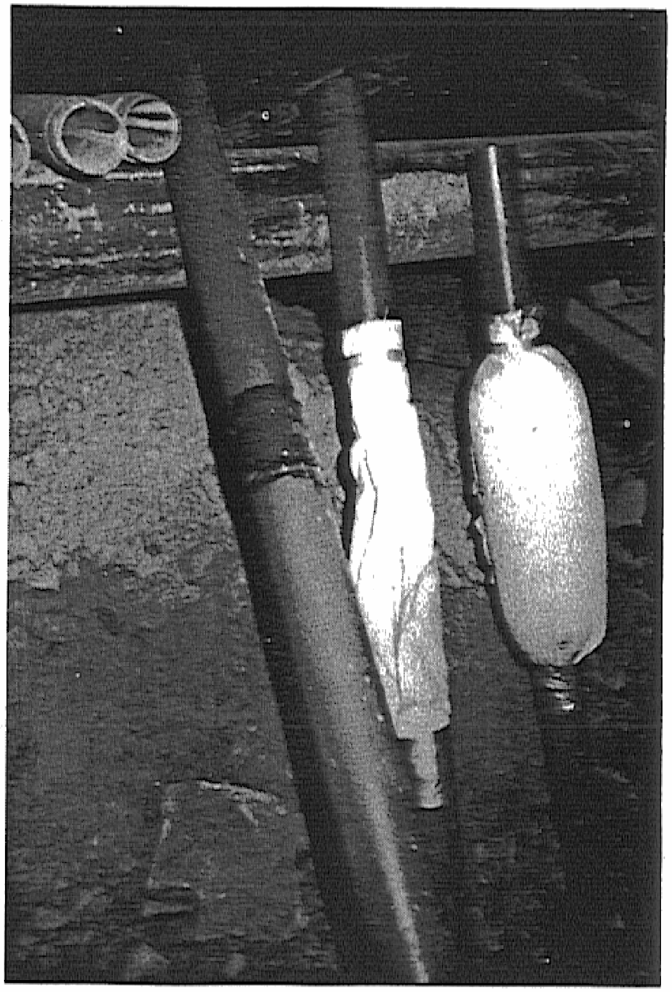
Figure 3. Installation sequence of M.P.S.P.

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 2 bar, 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 later 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 the "free" sleeves between bags, again via a double packer. Tests show that a properly seated fabric collar will permit effective "stage" water testing at up to 4 bar excess pressure.

Step 5 - Grouting is usually executed from bottom up via the double packer (usually of the inflatable type, Photograph 2). 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 pipe and so form a strong "sleeve grout" preventing the



Photograph 1. MPSP grout pipe (76mm dia.) showing (left to right) pipe with rubber sleeve, uninflated fabric bag, inflated fabric bag.

Photograph 2. Inflatable double packer with grout emerging from the central grout pipe between the packers.

opening of sleeves in contact unless a very weak mix was used. (The system does, however, allow different stages <u>in the same hole</u> 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 (<u>Step 1</u>) 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 120 m long was 1.5%, with the great majority being less than 1%.

2.3 Case Histories

The earliest field studies were carried out at Oymapinar Dam, on the Manavgat River, Turkey, in 1982, and were in response to problems incurred in trying to grout unstable limestone formations. The quantities of MPSP actually installed (several hundred meters) were insignificant per se, given the mature stage of the contract at the time of the system development. Illustrative details of works executed are, therefore, limited to three major dam projects (Tarbela Dam, Pakistan; Metramo Dam, Italy; and a large dam being built in Asia which cannot be named for political reasons, and which is referred to as Dam C), and a recent mining application in Canada.

2.3.1 Tarbela Dam (River Indus, Pakistan). Many authors have described different aspects of the long program of development and remedial works undertaken at this site (e.g. Lowe et al., 1979; Lowe and Sandford, 1982; Bruce and Joyce, 1983). It will be recalled that most of the effort was concentrated on spillway and tunnel protection by various means, including rock anchors and roller compacted concrete.

In early 1983, however, 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 betwen Right Grouting Adit 4 (RGA-4) and Tunnel 1, a zone otherwise comprising steeply dipping fissured limestone with phyllitic and carbonaceous schist interbeds. Two test panels were selected, as shown in Figure 4. 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 cement bentonite injection, as a comparison only. Frequent major cave-ins of holes attempted in this way had already confirmed the general 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

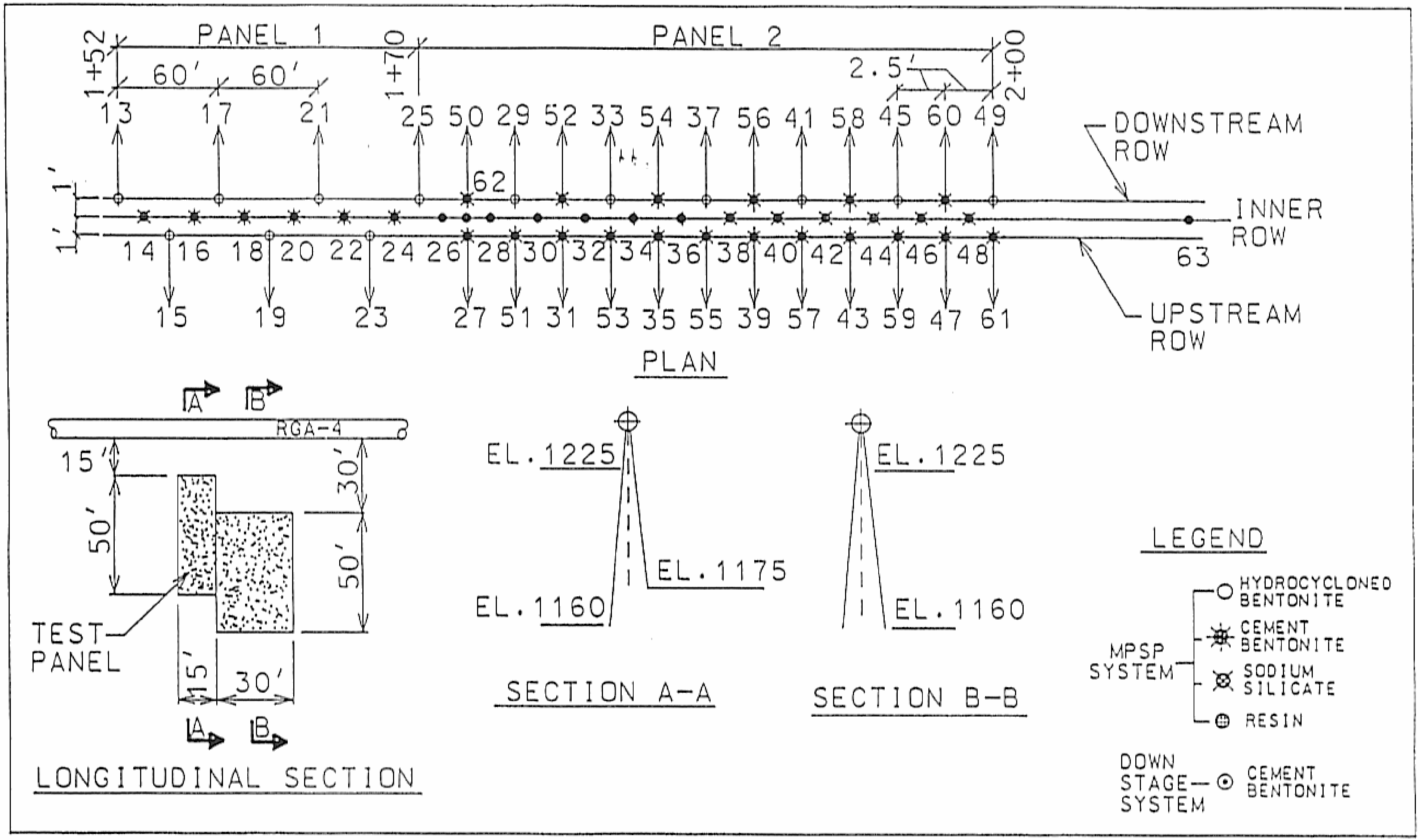


Figure 4. Details of grout hole and arrangement, MPSP Trial, Tarbela Dam, Pakistan

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 were the very low takes in the six downstage holes (25 out of 34 stages, each typically 5 m 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 liters, but many stages readily accepting full target volume).

As a final demonstration of the effectiveness of the grouting program, Figure 5 shows the Lugeon values obtained in water test holes drilled before and after grouting. In addition, hole stability was markedly improved and outflow of artesian water was greatly decreased in those test holes drilled after grouting.

2.3.2. Metramo Dam (River Metramo, Italy). This 600 m long earth and rock fill dam, almost 100 m high, is located in western Calabria. Given the high resource value of the water to be impounded in this arid area, a deep multi-row grout curtain was designed to provide an average permeability of 1 Lugeon compared to the virgin permeability of over 10 Lugeons.

The site is on the outcrop of the crystalline "Le Serre" formation, consisting of a number of largely grandioritic lithologies. They feature numerous major pegmatite veins and the whole mass is severely fissured and fractured in response to tectonic and cooling stresses. In addition, the bedrock is very deeply and heavily weathered by geothermal percolations. There are also substantial thicknesses of alluvial and colluvial materials increasing away from the river bed to over 50 m. The site is in an active earthquake region, and the dam design allows for a maximum seismic acceleration of 0.33g.

Both blanket (consolidation) and curtain grouting were required. The former featured holes at 0.5 m centers, to 10 to 15 m below the core trench. The curtain itself had three rows, the outer two being 50 m (downstream) and over 100 m deep (upstream) while the middle averaged 25 m. The curtain extended laterally for about 70 m on the Left Bank and 120 m on the Right Bank. Under the dam, 20-30 m below the core trench, a system of galleries and shafts was constructed, within the grouted zone, to permit monitoring of the performance of the structure in service and allow additional regrouting, if deemed necessary.

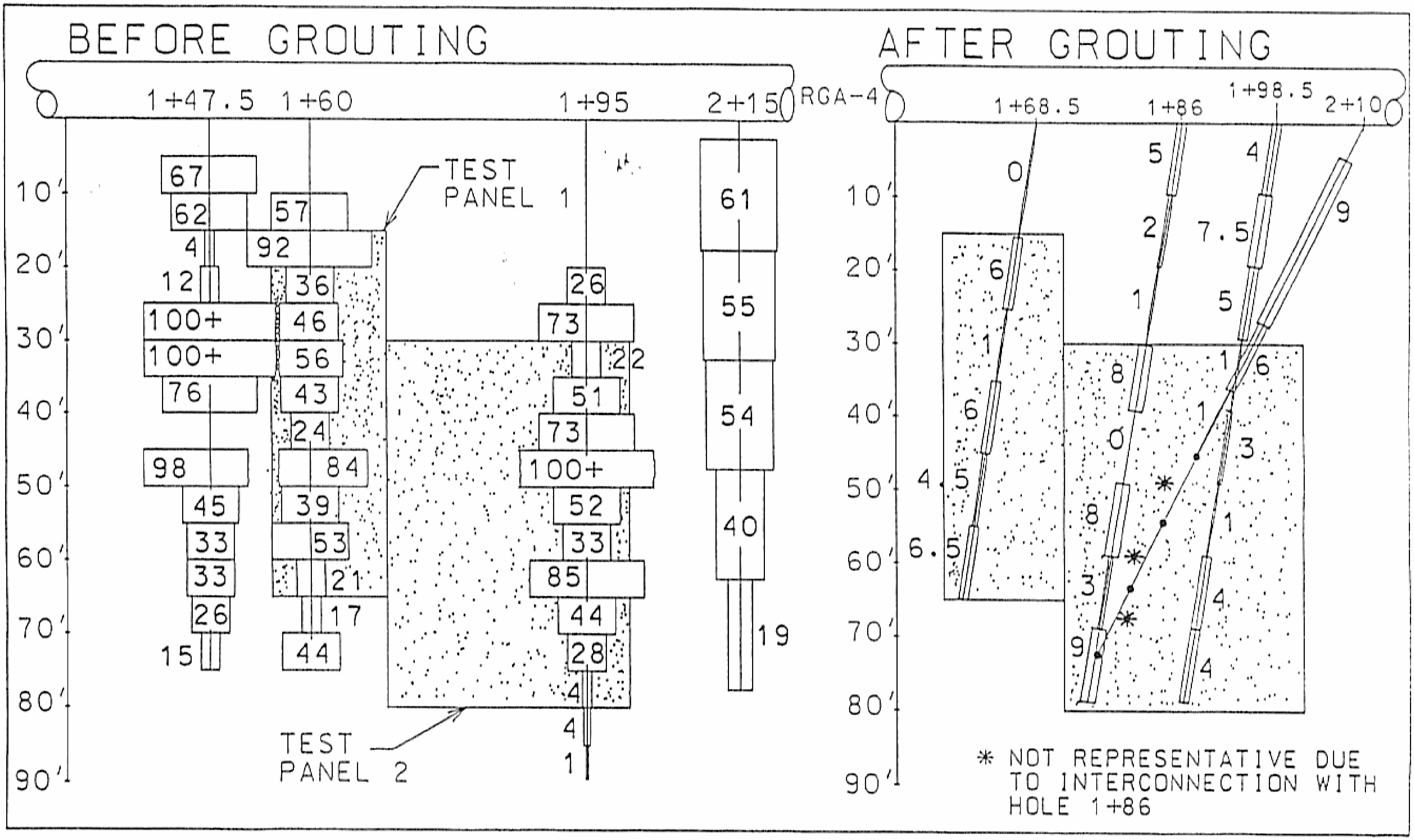


Figure 5. Comparison of water test results (in Lugeon units) before and after MPSP treatment, Tarbela Dam, Pakistan.

An extensive phase of water testing and grouting involving 57 test holes was conducted in the early 1980's. This confirmed, inter al., that the ground would not permit efficient treatment by conventional stage grouting, due to its inherent instability and variability. The MPSP System was proposed and accepted and over 80,000 m of grout holes were installed and injected in this way. 105 mm diameter holes were drilled and cased by diesel hydraulic rotary percussive duplex methods, with water flush. Steel pipes 50 mm in diameter had bags at 5 m centers, while the interhole spacing in the curtain ranged from 1.5 to 3.0 m. For the chemical grouting, a long chain compound diacid ester reagent was used to react with the sodium silicate solution, to enhance the long-term performance of the gel in place. Typically, cement grout injection rates varied from 150 to 450 liters/hour, while those for the chemical grout were somewhat higher, depending always on grout take characteristics. Maximum injection flow pressures range up to 15 bar. Average cement takes of around 300 kg of mix on the Right Abutment were about three times those on the Left. The work has now been completed and the curtain target permeability has been achieved.

2.3.3 Dam C, Asia. The bedrock formations of marls, chalky limestone, and gypsum were found to render impracticable the upstage grouting system originally specified. Similar technical problems seriously compromised the efficiency of a downstage treatment while the very restrictive programming requirements further prevented its general usage. The MPSP system was, therefore, proposed and adopted.

Under the 3600 m long, 100 m high Main Dam, and in the Right Bank extension, the system was adapted in place to provide both a conventional sleeved pipe approach for up to 40 m of very weathered rock (almost a loose, granular soil) lying above the fresher but mechanically unstable sediments (Figure 6).

The pretreatment of the sugary limestone necessary for the excavation of an associated tunnel also proved impractical with conventional methods. A program involving 20 overlapping "umbrellas" of subhorizontal steel MPSP pipes was therefore designed to provide stable and dry excavation conditions despite a 30 m hydraulic head. The holes in each umbrella were 15 m long, spaced at 1 m centers around the ring and were drilled by water flushed rotary duplex methods involving full length casing. Steel MPSP pipes 38 mm i.d. were used thereby providing a further in situ reinforcement benefit to the highly fissured rock mass being consolidated.

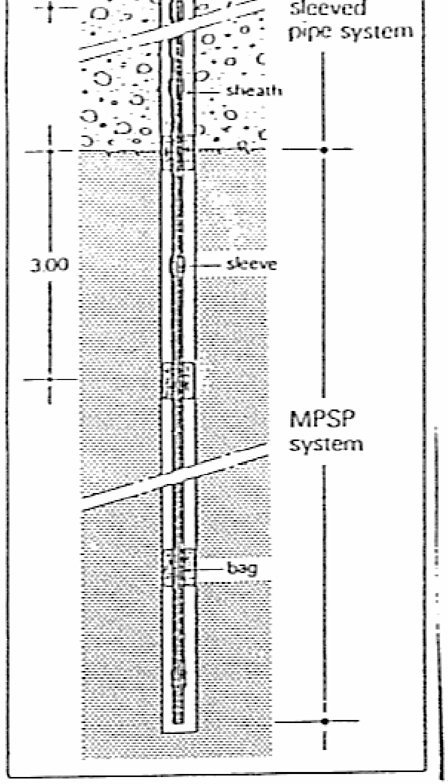
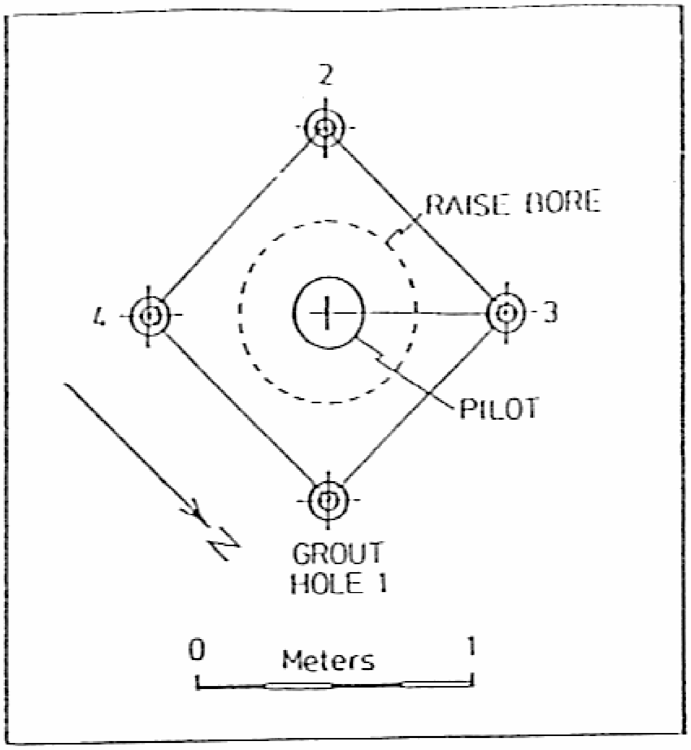


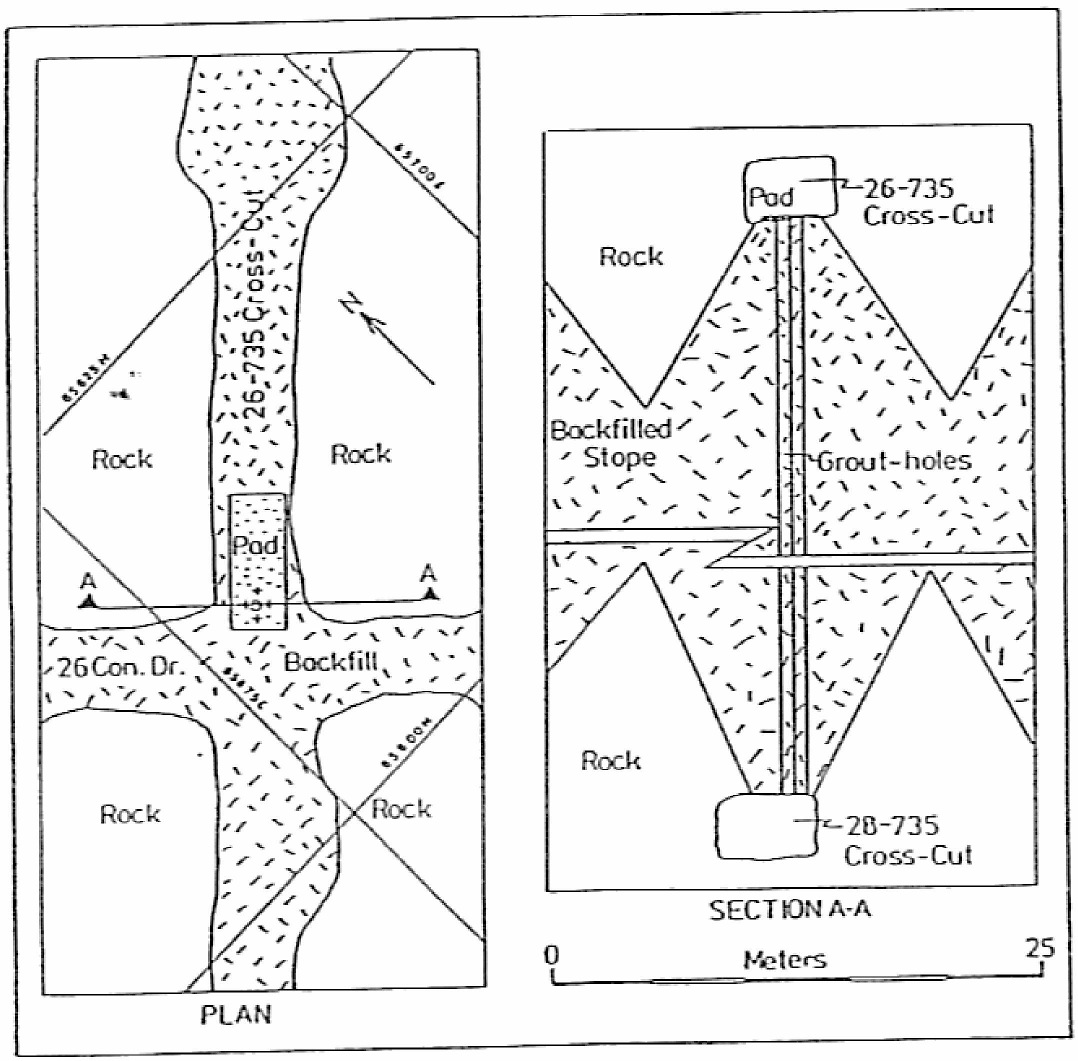
Figure 6. Combined use of tube a manchette and MPSP for treatment of alluvials and fractured rock respectively. Dam 'C', Asia.

Each pipe had bags at 3 m intervals, and each umbrella allowed a 12 m excavated length to be made in one pass. This application in tunnels is expanded upon further by Bruce and Gallavresi (1988).

2.3.4 Kidd Creek Mine (Ontario, Canada). Following years of unsuccessful attempts to drill a 250 mm pilot hole 60 m vertically through mine backfill, some 800 m underground, the Mine decided to use the MPSP System to firstly pregrout the fill (Bruce and Croxall, 1989). This fill body consisted of crushed very hard (3000 bar) and abrasive meta andesite rock, from gravel to room size, very poorly cemented. (The typical "concrete" U.C.S. was 50 bar.) A group of four 133 mm diameter boreholes was drilled from the 800 m level gallery to the 860 m level gallery (Figure 7). Given the extremely onerous drilling conditions and limited headroom (4 m) the holes were drilled by the novel double head duplex system in which the rods and casings are simultaneously advanced while rotating in opposite senses (Bruce 1984, 1989). This system provided fast and reliable penetration, and minimum deviation.

The MPSP pipes were in 3 m lengths and had an outside diameter of 72 mm. Each length had rubber sleeves at 1.5 m intervals, while every fourth sleeve was enveloped within a 600 mm long fabric bag. Grouting was accomplished via an inflatable down the hole double packer, with cement based grouts mixed at the surface and modified prior to injection. The fill proved to be exceptionally open and permeable, and sodium silicate solution had to be used to restrict grout travel away from the target zone. By the conclusion of the four holes, 76 m^3 of cement grout and 9 m^3 of sodium silicate grout had been injected.





<u>Figure 7.</u> Plan and section of grout holes through stope, Kidd Creek Mine, Ontario, Canada

Thereafter, the 250 mm dia. pilot hole was drilled by rotary methods using a Robbins 34-R Raise Borer, at an average industrial production of about 2 m per hour on the hole. Deviation from vertical was measured as 2% upon breakthrough. The raise was then successfully upreamed from the 860 m to the 800 m level to a final nominal diameter of 630 mm. The production rate for this operation was approximately 1.5 m per hour. A television camera survey was then conducted to view the effectiveness of the grouting and the condition of the raise wall. Table 2 summarizes the visual record of porosity.

% Total Raise Length	Estimated % Porosity of Surrounding Fill
48	Less than 15
30	15-30
22	Over 30

Table 2. Visual record of fill around raise bore

These observations suggest that the grouting had reduced the porosity quite substantially in places, but that elsewhere the fill was still relatively open structured, though stable. This confirmed the "feel" for the gluing action of the grout, obtained during the injection phases. In addition, it was possible that the rigid plastic pipes, at relatively close centers, had contributed an in situ reinforcing effect to the larger fill blocks, helping to "stitch" them together somewhat to form a more stable skeleton.

3. THE RODUR^{sm} SYSTEM FOR DAM BONDING

3.1 Background

The cause of cracking in a concrete dam is usually difficult to discern and often incompletely understood. For example, foundation or abutment performance may be different from the projected, or the dam itself may behave in a non monolithic manner. This may happen when dams of novel concept are built on sites of complex geology. There may be transient processes at work such as heat transmission (a consequence of thermal variations or heat dissipation during concrete curing) or pore pressure development (resulting from water filtration through the dam in service). Equally there may have been intrinsic flaws in the construction of the dam itself, such as poor concreting practices (for example, excessive water cement ratios) or the use of aggregates and cements capable of initiating the expansive alkali aggregate reaction process. Depending on their cause, the fissures may appear as early as first impounding, or may develop after years of apparently successful performance. The presence of cracks implies the action of tensile - or, more rarely, shearing - stresses of magnitude greater than the concrete material strength. Such cracks establish planes of discontinuity within the dam, and, if connected to the upstream face, will markedly modify the dam's integrity and performance.

It has long been standard practice to attempt to fill large fissures by injecting cement based grouts, and the smaller fissures with other conventional preparations such as silicates, phenols or acrylates. These attempts have met with mixed results and have often had to be repeated at frequent intervals due to the brittle nature of the grout being incompatible with the design of a structure demanding continuing intra structural movements. There are also major practical difficulties in actually executing the works under conditions in which substantial draw down of reservoir level is simply not possible due to economic, environmental, or operational reasons. These include: 1) the flow of water at high velocity and pressure, 2) segregation and dilution of the grouts, 3) matching grout particle size to the often very irregular fissure width and, 4) the need to avoid using high injection pressures with conventional grouts of long setting times.

Clearly the proper treatment of the fissures must follow an assessment of the causal factors. Decisions can then be made with respect to 1) the ease or practicality of injection, 2) preferred characteristics of the set grout, 3) the desirability of bonding across the fissure, and 4) if it is feasible to exploit further the fissures with the grout, in order to introduce new compressive stresses in the structures to hopefully compensate the tensile stresses that produced the fissures initially. However, in certain cases it may be difficult to eliminate the tensile stresses for all possible combinations of loads acting on the dam. Then, when the treatment is complete, new fissures may well appear nearby (if the causes of the cracking have not been eliminated), or in other locations or directions (if new tensile stresses occur in response to a changed load situation).

Overall, therefore, from the operational viewpoint it is of prime importance to decide the quantity of grout to be used, as well its strength and to a lesser extent its deformability. Such repairs are in a sense "irreversible": an inefficient repair attempt with the wrong material will greatly reduce the success potential of any subsequent attempt at treatment, no matter how conscientiously executed.

Over the last decade the RODUR^{sm} process of concrete dam repair has been applied with remarkable success. In essence, every attempt is first made to try to understand the cause of the problem. This involves careful review of geological, constructional and behavioral data, often as a basis for executing a new campaign of exploration (by cored holes) and additional monitoring. Initial decisions can then be made with respect to the choice of the grouting material. The fluid properties of the grout, the nature of the fissures and the characteristics of the structure then dictate the initial grout hole design and the sequencing of the repair operation. In addition, the performance of the grouting and the structure are continuously monitored during the works, so that adjustments to working parameters can be made in a timely fashion. This intense monitoring and flexibility of response are keystones of the RODUR approach. The RODUR^{sm} process has been used for both water sealing and structural bonding in a variety of high dams including gravity, gravity-arch and double curvature.

The following sections review briefly aspects of material design and injection theory prior to describing a recent major case history in the United States. This particular project featured the technique as a method of repairing the dam itself: RODUR^{sm} can also be used to repair the foundation rock if required.

3.2 Injection Materials

In the majority of cases, the following properties are sought of the grout:

(1) It must be a true Binghamian liquid and not a suspension of particles, in order to have the best change of filling comprehensively the fissures, even though their surfaces may be irregular and the aperture small.

(2) It must be immiscible in water.

(3) It must harden as soon as possible after injection to limit flow from the injection point.

(4) It must have a reasonably constant and controllable viscosity till hardening, suited also to the environmental conditions. This viscosity must also reflect the fissure width anticipated.

(5) It must have minimal shrinkage.

(6) It must have good durability.

(7) It is usually required to bond efficiently to wet surfaces, under high hydrostatic or dynamic heads, often in low temperatures, and so have high tensile and shear strengths.

(8) It is typically advantageous to have a modulus of elasticity significantly less than the concrete.

(9) It must have as low a surface tension as possible in order to ease penetration into fine fissures.

(10) It must be easily handled, with minimal environmental problems.

The distance between the grout holes intercepting the target fissure is then chosen to reflect the grout parameters, the radius of action that can be anticipated, and the injection pressure to be applied. This flow pressure depends mainly on the geometry of the fissure, the grout viscosity and the rate of grout flow anticipated.

The RODUR[sm] process is based on the use of various types of synthetic epoxy resins. Depending on their formulation, such resins can be provided with a wide range of initial viscosities, which remain constant until the point of polymerization. This time of reaction can be preset. Figure 8 illustrates the influence of temperature on the dynamic viscosity, for example.

For any given application, the choice of a particular resin depends on the factors outlined above, as well as the proximity of the grout hole to the open boundary of the fissure (to avoid wasteful and premature loss of resin). Furthermore, after several test programs it became clear that the efficient sealing of a submerged fissure in adverse conditions demands that the injected grout be kept motionless while being subjected to compressive stresses during the setting process. The impossibility of maintaining the pressure mechanically by pumping (because of the short pot life of the resins) and the need to prevent emulsification during the fluid stage further combined to dictate the use of relatively high viscosity grouts.

As a final point, theoretical studies (e.g. Muzas et al., 1985) show that such viscous resins require relatively high pressures to overcome line losses and encourage flow into fine fissures. Under these circumstances, high pressures are typically not dangerous as they act on relatively small areas with respect to the total fissure surface area or to the overall mass of the dam. In addition, these localized injections do not remain fluid for long periods and, upon setting, no longer exert pressure.

When under high pressure, prior to setting, the resin may penetrate the pores of the concrete material. This process can encourage the expulsion of water, so providing dryer and so better conditions for bonding surfaces together.

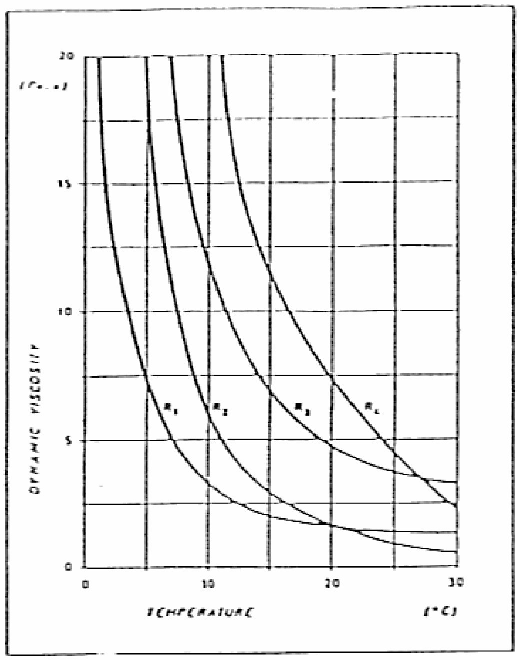


Figure 8. Dynamic viscosity/ temperature curves for four typical RODUR[sm] resins (Muzas et al., 1985)

3.3 Case Histories

Summary details of the major dam projects involving the RODUR[sm] process are provided in Table 3. In addition, there are many other cases where it has been used in the repair of concrete structures for nuclear generation and dam apurtenances such as spillways and penstocks. Three of these dams have already been described in detail in previous publications, namely Atazar (by Echevarria and Gomez, 1982), Zeuzier (by Berchten, 1985) and Cabril (by Portuguese Working Committee, 1985), while in several cases the RODUR[sm] method has been used to treat both the dam and the foundation rock.

The most recent application has been to repair an old concrete dam in the eastern part of the United States. As will become evident below, the repair has proved totally successful (Bruce and DePorcellinis, 1989). However, the Owner currently wishes to retain the anonymity of the project until the current excellent performance over several yearly cycles has been confirmed. It is, therefore, referred to, protem, as "Dam A".

3.3.1. Introduction. The dam is a 320 m long concrete arch structure with gravity abutments and two spillways (Figure 9). It stands a maximum of 65 m above the river bed, and impounds water for hydroelectric generation at another installation. Construction began on the dam in January 1926 followed by closure in December 1927, and lake filling in April 1928. It would appear that concerns about aspects of the dam's performance arose soon after, and the spillways and the West Abutment were soon reinforced, and again later, in 1938 and 1950, with additional concrete. It is clear from the plan of the dam that the whole structure is not a true continuous arch, with the departure from the classical most marked on the West (Left) Abutment. By far the greatest problems relating to seepage and movements have, not surprisingly, been encountered on this side.

YEARS	Site and Location	Structure	Dam Crest (m)	Height (m)	Problem/ Application	Grout (kg)	Owner	Notes
1978 -1980	El Atazar Dam, R. Lozoya, Near Madrid, Spain	Double curvature Arch Dam, for Water Supply	484	134	Failure of upstream joint sealant leading to development of massive fissure (3200m² - 60% of dam area) through dam, 3.5 to 8.5mm wide.	90,000	Canal De Isabel II, Madrid	Sealed under 90m of waterhead and high (
1980	Paradela Dam, River Cavado, Portugal	Concrete faced Rockfil Dam for Hydroelectric Generation	540	110	Sealing of joints between facing panels.	1,000	Electicidade de Portugal, Porto, Portugal	
1980 -1983	Mequinenza Dam, River Ebro, Catalonia, Spain	Concrete Gravity Dam for Hydro-electric Generation	470	146	Differential settlement of foundation rock due to power plant structure led to cracking of dam, encouraging further deterioration	50,000	Empresa Nacional Hydroelectric Del Ribargozana, Barcelona, Spain	Opstrene layer of co[n] grouted from vertical
1981 -1983	Zeuzier Dam, River Lienne, Switzerland	Double curvature Arch Dam, for Hydroelectric Generation	256	156	Settlement of foundation rock caused major concrete fissures especially in lower part of structure.	150,000	Electricite de La Lienne SA, Sion, Switzerland	
1981	Cabril Dam, River Zezere, Portugal	Very thin Double Curvature Arch Dam, for Hydro-electric Generation	290	132	Multiple fissuring of upper section of dam resulting from nature of design of crest works.	18,000	Electicidade de Portugal, Porto, Portugal	Cracks and vertical contraction joints r[…] grouted.
1982	Koelnbrein Dam, River Malta, Kaernten, Austria	Double Curvature Arch Dam for Hydroelectric Generation	626	200	Multiple fissuring in bedrock and near foundations due to design in U-shaped valley	11,000	Oesterieichische Draukraftwerke AG Klagenfurt, Austria	Resin grouting of co[…] and bedrock. One of highest dams of the
1982 -1983	Zillergruendl Dam, River Ziller Austria	Double Curvature Arch Dam for Hydroelectric Generation	506	186	Fissuring of dam due to compression of bedrock.	3,000 rock + 6,500 in dam	Tauernkraftwerke AG Salzburg, Austria	Resin used in rock […] test, and for dam fi[…]
1986 -1987	San Esteban Dam, River Sil Orense, Spain	Concrete Gravity Dam for Hydro-electric Generation	295	116	Exploitation of vertical and horizontal construction joints promoting Alkali Aggregate Reaction.	100,000 (hor.) + 88,000 (vert.)	Iberduero SA, Bilbao, Spain	5,300m² in 22 joints treated. Also Opstre[ne] membrane applied.
1988 -1989	Karakaya Dam, River Furat Turkey	Double Curvature Arch Dam for Hydroelectric Generation	462	175	Differential compressibility of rock in abutments leading to massive tear fissure through the dam.	55,000	Devlet Su Isleri, Aukara, Turkey	Treatment of new dam
1988	Dam 'A' USA	Concrete Gravity Dam	320	65	Exploitation of vertical and horizontal construction joints leading to severe seepage volumes and uplift pressures.	7,200	N/A	First North America[n] Application

Table 3. Summary details of major dam repair projects involving RODUR[sm].

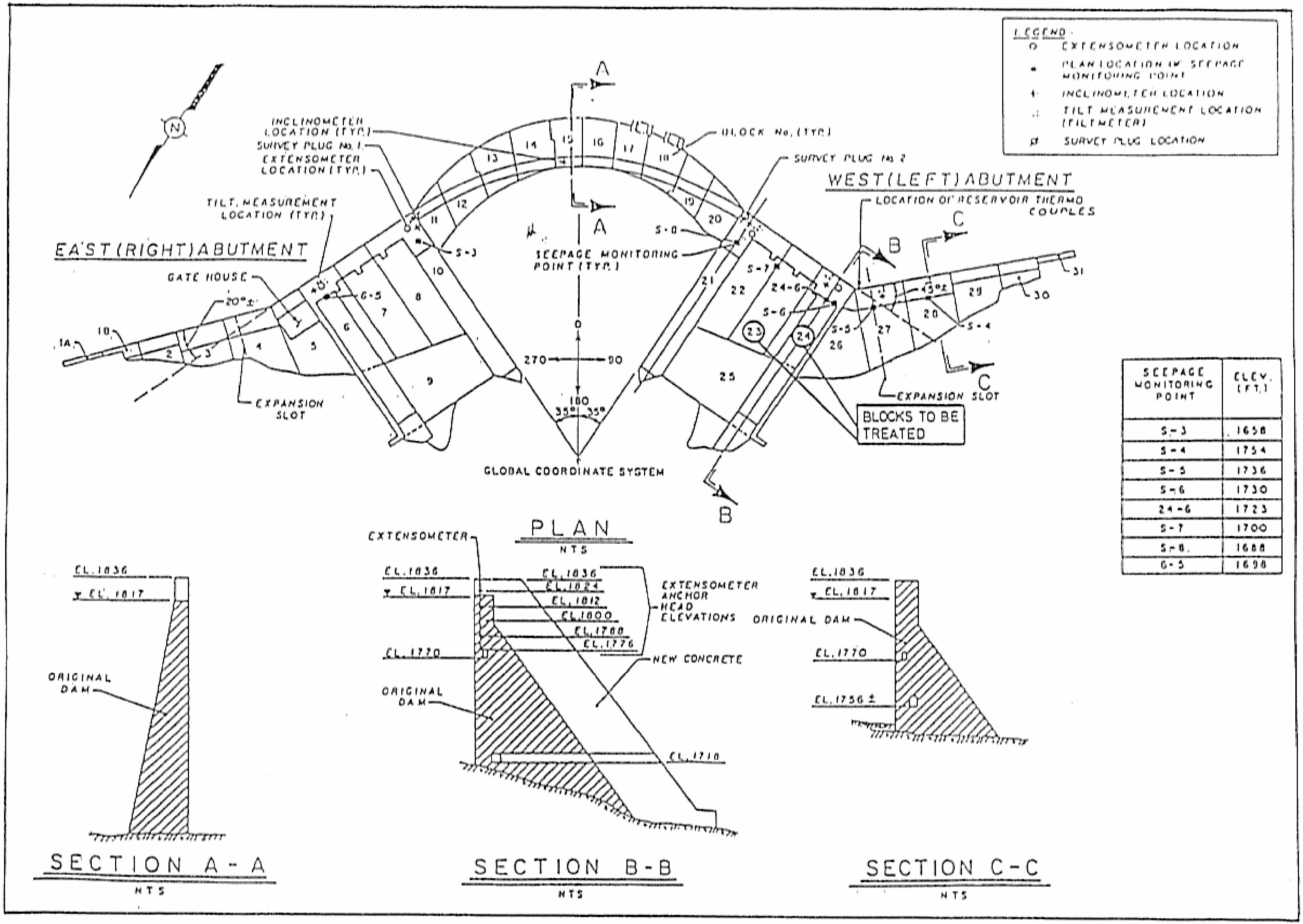


Figure 9. Plan and typical sections of Dam A, showing instrumentation locations, and blocks to be grouted (dimensions in feet).

Comprehensive seepage records were historically maintained, and were carefully reviewed during the early and mid eighties. They showed that the amount of seepage was indeed cyclic, being highest in the summer months, coincident with high reservoir levels. By July 1987, however, the seepage had reached its highest volume since the 1940's and was under higher pressure over a greater area of the dam than previously observed. These three factors led to a further study by the Consultants to assess the situation and to define remedial measures which could be applied quickly. They found that the bulk of the flow was entering the Lower Gallery of the Left Abutment in two blocks - 23 and 24 - between distinct elevations (1730-1700'). The water chemically was very pure, and there was considerable deposits of calcite on both upstream and downstream sides of the lower Gallery. By 1987, there was also considerably more seepage from longitudinal secondary fissures in the roof, and from the downstream side of the Gallery than previously.

The Consultants concluded that the cyclic nature of the flow was most strongly influenced by reservoir head, and to a lesser extent by temperature variations. The latter, especially near the crest, would have the potential to reduce compressive forces on the upstream face, thereby allowing easier exploitation of existing construction and lift joints by higher lake levels. The very pure lake water also had a high potential for solution which would gradually increase the fissure aperture and further reduce the frictional characteristics of the joints.

The seepage entering the downstream side of the Gallery indicated that hydrostatic pressures had developed which had not been fully controlled by the drains and galleries. The development of such pressures across the dam section would further reduce compressive stresses in the dam leading to increased seepage and concerns for decreased overall stability.

The construction records of the suspect zone were carefully reviewed and evidence was found from irregularities in construction sequence, concrete composition, and construction practices to suggest strongly that the water could be flowing primarily through the horizontal lift, and vertical construction, joints.

<u>3.3.2. Concept of Remediation.</u> It was decided to immediately commence efforts to:

- o Reduce uplift due to seepage pressures within the dam.
- o Reduce the rate of deterioration of the concrete along the seepage paths.
- o Reduce monitoring and maintenance requirements associated with the seepage.
- o Increase confidence in the integrity of the dam.

Following a consideration of various options, including prestressed anchors and face sealing membranes, the prime methods recommended were the grouting of the lift joints, and the drilling of additional drainage holes from within the galleries. (Not described in this paper.) Treatment was focused on two lift joints in each of two adjacent blocks, 23 and 24. The Consultant listed the following desirable grout properties:

- to ensure maximum penetration, the grout should be a true liquid and not a suspension of solids.
- the grout should be immiscible in water.
- the grout should have a short hardening time (to minimize washout).
- the grout should have a nearly constant viscosity until setting.
- the grout should not shrink after hardening.
- the material should have a low compressive modulus, but high sheer strength.
- the material must be durable.
- the material should be chemically stable and non toxic during preparation, and in service.

The grout was also required to be placed as close to the face as possible, and at full summer pool when the fissures would be most open. In this way no tension would later be applied to the grout/joint interface in the event of a significant structural movement later occurring at lower pool.

The contractor's Technical Proposal incorporated the following fundamental assumptions:

- Given that very little was known about the actual sources of the flows, and their mode of occurrence (i.e. through horizontal joints, vertical joints, induced fissures, or pores), it was the declared intent to examine and treat only the two prescribed horizontal joints in each block.
- For the same reasons, this whole program of treatment was to be regarded as exploratory in nature.
- For reasons consistent with the proper application of the RODUR^{sm} System, all drilling was to be conducted from the Lower Gallery with a small diameter core drilling system.
- Appropriate epoxy resin formulations were to be used to seal the joints.
- The work in each block would be conducted in a "step by step" way to optimize the exploration of the structure and the verification of the treatment.

<u>3.3.3. Construction.</u> To obtain maximum information about the dam, the water flow patterns, the joints and the effectiveness of the grouting, every hole was cored full length. To improve directional control, and to economize drilling quantities, all grout holes were drilled from the Lower Gallery, approximately 2.1 m wide and 2.5 m high.

For each horizontal joint in each of the two blocks (23 and 24) being treated, holes were designed to intersect it at regular intervals (Figure 10). The two Primary rows were drilled and grouted before locating and drilling Secondary and later holes. The elevation of the Lower Gallery was not constant and so its position relative to the joints being drilled varied constantly. Therefore, to ensure the correct points of intersection of the joint, a scaled section had to be drawn for each hole location to calculate hole length, inclination and gallery wall entry point (Figure 11). At each station two holes were drilled -- one into each joint -- and these were oriented orthogonal to the Gallery wall and so the dam's face. Each hole was designed to terminate slightly beyond the target joint so that the concrete both above and below it could be studied.

Drilling was conducted with a modular frame mounted electrohydraulic rig, providing holes of 46 mm and cores of 36 mm diameter. Details are provided in Table 4. After all the holes in one phase had been drilled, they were fitted with mechanical packers terminating close to the joint. Consistent with the Consultant's wishes for the properties of the grout, the epoxy resin range was the only viable option. Its ability to not only seal the joints under extreme seepage conditions, but to bond the concrete surfaces together offered an attractive potential to structurally improve the dam. As the set resins typically have moduli of elasticity that are far lower than that of even poor concrete, this bonding action would not necessarily produce a structural "hard spot" in the dam. Thus, while it would be hoped that movements in the vicinity of the grouted area would be reduced, the resin's elasticity would still allow it to withstand strains without losing adhesion and allowing the return of the seepage. The bonding effect would also enhance the seismic performance of the dam.

Due to the initial uncetainty regarding aperture width and extent, two resins were brought to site - RODUR 1 and RODUR 2. They differed only in their rheological properties in the liquid phase, the former being less viscous. Their chemical and mechanical properties were similar. Each comprised a two part formulation in which "Components A and B" were thoroughly mixed, usually for 3-5 minutes, prior to injection. Some of the mechanical properties are shown in Table 5.

As described below, even the RODUR 2 proved too thin by itself to combat the extreme flow conditions encountered initially in the wide, open fissures intersected. As standard practice, talcum powder as added to the resin prior to injection to increase initial viscosity. At the concentrations used (generally less than 20% by weight) there was no significant influence on set grout properties. The

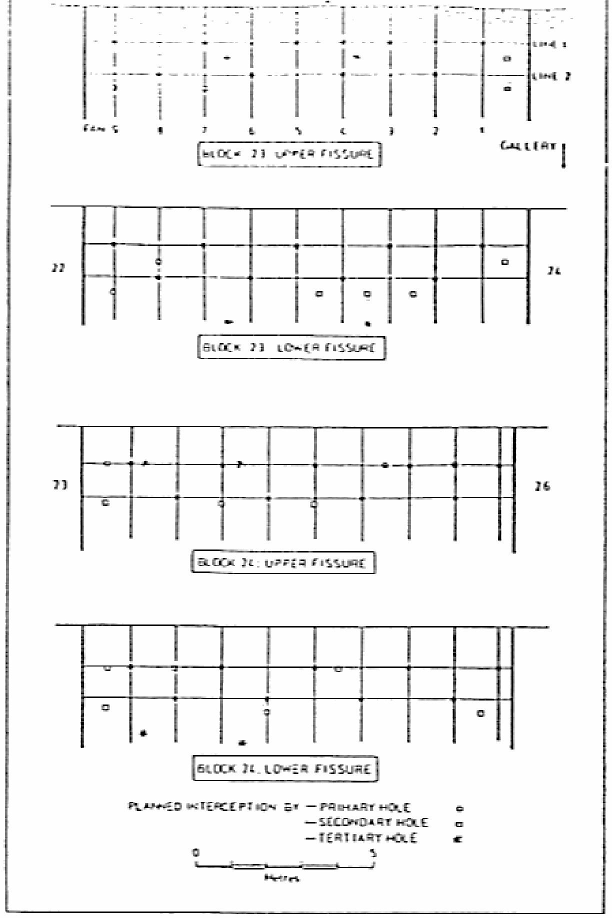


Figure 10. Planned interceptions of joints by drillholes.

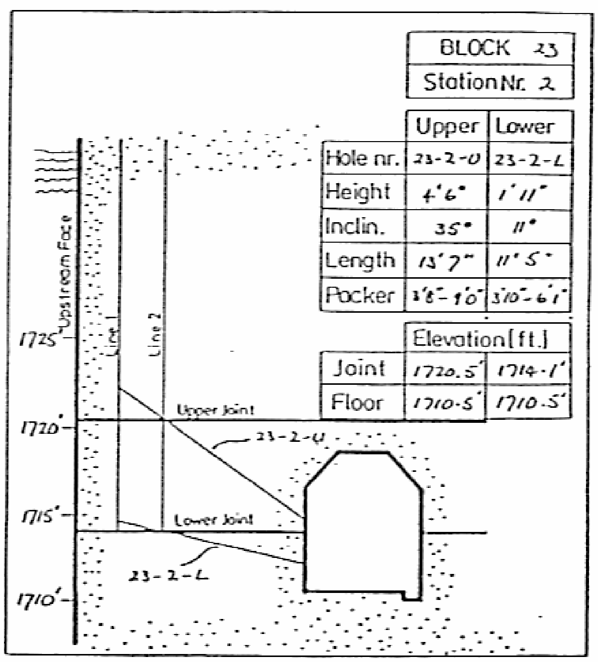


Figure 11. Typical section showing intended interceptions at one station.

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 4. Hole drilling summary. (In addition, two drain holes were drilled downstream, in each block, after grouting: all four were dry.)

average polymerization time decreased several minutes but this merely reflected the extra energy applied by the mixer and the resultant temperature increase in the fluid resin.

The grout was mixed in the supply tubs and transferred to special air powered delivery pumps. Depending on the rate of injection required, these pumps gave maximum output pressure ranges of 24 or 48 times the input air pressure.

Grouting parameters and injection sequences were broadly predetermined, but amended in response to observations on grout travel and pumping characteristics. Generally excellent resin communication was established between and beyond adjacent holes in the same joint, providing encouragement that a comprehensive treatment was being effected. In addition, there were resin connections

- with the large diameter chimney drains intersecting the fissures being grouted
- into vertical construction joints between blocks
- through vertical secondary fissures running between joints
- back into the Gallery where the joints were exposed
- from secondary longitudinal microfissures in the roof of the gallery.

The Primary treatment and the Secondary grouting of especially troublesome zones were conducted with the viscous RODUR 2 grout, with talc. Late stage injection into smaller and tighter fissures and joints was conducted with the less viscous RODUR 1. Close examination of pumping rates and grout consumptions for the successive phases confirmed the "tightening up" of the joints: the pumps had to work several times harder in these Secondary holes, which took barely half the volume of the Primaries at far lower injection rates.

	RODUR 1	RODUR 2
Description	Liquid epoxy resin compound with amine hardener composition, free of solvents, two components.	Liquid epoxy resin compound with amine hardener composition, free of solvents, pigmented, two components. Component A: epoxy liquid resin, plus minerals and pigments Component B: based on a cyclo aliphatic amine
General	Free of solvents and mineral aggregates, its low viscosity allows use for very fine cracks, and in presence of water at low temperatures.	Hardened material has very low creep under compression. Liquid resin has excellent reistance to wash out by flowing water
Fluid Properties		
• Viscosity at 20° C (Brookfield Visc.)	2000/3000 MPa. S.	6500/7500 MPa.S
• Mean Surface Tension (liquid)	50 dyn/cm	50 dyn/cm
• Pot life	Approx. 50 min at 20° C	Approx. 90 min at 20° C
• Lowest hardening temp	4° C	4° C
• Water absorption of hard resin at 23° C	1.98%	1.8%
• Specific gravity at 20°: liquid	1.10 g/cm^3	1.93 g/cm^3
solid	1.19 g/cm^3	1.99 g/cm^3
Mechanical Properties (Values depending on Test Conditions)		
• Compressive Str.	95 Mpa	110 Mpa
• Flexural Str.	40 Mpa	35-40 MPa
• Tensile Str.	28 Mpa	20 Mpa
• Elong. at failure (Tensile)	3%	2%
Adhesion Tests (Cylindrical concrete specimens bonded with resin in different conditions, at 20°C. Simple tensile test.)		
• Dry Concrete	100% of samples break in concrete	100% of samples break in concrete
• Damp Concrete	80% of samples break in concrete	80% of samples break in concrete
• Submerged Concrete	65% of samples break in concrete	60% of samples break in concrete

Table 5. General properties of RODUR^{sm} resins used in Dam 'A'.

3.3.4. Core Observations. Typically the material of the dam appeared to be of good quality. Soft or honeycombed concrete was restricted to the top 80 mm of concrete pours although several smaller isolated and unconnected cases were recorded throughout, and especially in Block 24. No major secondary fissures were encountered in the cores. Evidence of alkali-aggregate reaction was found in many samples but appeared to be of small scale and did not seem to have weakened, cracked or stressed the core samples in question. Old formwork timber and steel grout pipes were also occasionally encountered. In one hole, a zone of washed concrete about 130 mm thick was recorded just above the lower joint, but this extent and type of material weakness was exceptional.

Joint	Summary Characteristics (Generalised)	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 6. Joint apertures, as evidenced by thicknesses of resin infill found.

Plotting the evidence of resin recovery from the Secondary and Tertiary holes provided data on joint widths, as summarized in Table 6. In most of the Primary specimens the resin was poorly bonded to the usually dirty concrete surfaces. The resin either debonded during drilling or could be easily prised off during subsequent handling. However, all of the eight Tertiary hole samples showed good resin adhesion, especially in the five samples containing both Primary (thick) and Secondary (thinner) samples.

				MEASURED FLOW INTO BLOCKS 23/24							
Date	Lake El. (ft)	Cum. flow at S5	Cum. flow at S6	Joints etc. 23	Joints etc. 24	Grout Holes 23	Grout Holes 24	Total in 23+24	Cum. flow at S7	Cum. flow at S8	Total Left Abutment
8/1	1794.1	330	440	210 c	425	-	-	635	1075	1405	1575
8/8	1791.5	280	400	225 c	420	-	-	645	1045	1215	1385
8/15	1789.7	285	405	165 c	400	-	-	565	970	1235	1400
8/22	1787.7	265	395	155 c	345	-	-	500	895	1165	1310
8/29	1785.9	260	370	130 c	330	-	-	460	830	1075	1215
9/6	1786.2	245	330	140 c	310	-	-	450	780	1010	1140
9/12	1786.1	245	310	130 c	285	-	-	415	725 R	905	1030
9/19	1786.2	245	315	130 *	285 *	-	-	415	730 R	925	1040
9/26	1786.6	245	315	130 *	285 *	420	-	835	1150 C	1245 R	1360 *
10/3	1788.3	245	320	115 *	285 *	1320	-	1720	2040 C	2115 R	2190 *
10/10	1788.3	265	340	115 *	265 *	1125	430	1935	2275 C	2380 R	2455 *
10/17	1788.1	265	340	Incl.	160	20	1130	1310	1650 C	1775 R	1890 *
10/24	1787.8	265	340	Incl.	115	115	-	230	570	780 R	870
10/28	1787.6	245	315	Incl.	165	5	25	195	510	610	710
11/7	1789.1	210 *	265 *	Incl.	115	-	-	115	380 C	430	530

* Projected
c Calculated from Owner's data plus Contractor's readings.
R Revised from original data (certain readings at standard locations could not be made accurately or at all during the works)

Table 7. Measured seepage rates (liters/min). Figure 9 shows location of measuring points.

3.3.5. Water Flow Observations. The routine seepage measurements were maintained in the Left side of the dam during the works (Table 7). In addition, twice daily readings were taken of the flow from each borehole open at that time. Holes with particularly strong flows (over 200 liters/min) were fitted with packers and the pressure head noted. Typically this corresponded exactly to the theoretical hydrostatic pressure acting at the level of the fissure.

Primary holes were completely sealed off during Primary grouting. Similar flow records were maintained during the Secondary operations, supplemented by data from other flow sources such as the large diameter chimney drains, vertical roof drains, and fissures exposed in the Gallery. Again complete sealing of grout holes was recorded.

A further observation made during the Primary grouting was the cessation of several small seeps and leaks on the downstream side of the Gallery. The walls of the Gallery, on both sides, and hitherto completely saturated, began to dry out at this time.

Finally, records were maintained of the total seepage into the blocks after Secondary grouting. Water temperatures were also recorded to help locate the level in the lake at which the water was entering the structure. It is noteworthy that by far the greater part of the final flow into the grouted section (105 out of 115 liters) was recorded by a large diameter chimney drain in Block 24. By lowering an inspection lamp down from the Upper Gallery, the major source of the inflow was identified as being a horizontal joint at El 1739' - well above the level grouted in this program.

Since the execution of the work, the dam has passed through a full season, including the heaviest rainfall and highest reservoir levels for many years. The flow through the Left Abutment increased with head but the grouted section remained exactly as before. The Consultant is of the opinion that if the dam had not been grouted with the RODUR^{sm} method at the time it was, it would have been seriously threatened during the flooding season, and an emergency situation would have been declared.

4. FINAL REMARKS

The two techniques described in this paper represent reliable, state of practice solutions to two groups of especially challenging grouting related problems, namely the treatment of "difficult" rock masses, and the sealing/bonding of concrete dams in adverse hydro dynamic situations. The exemplary performance of both techniques in Europe and Asia has been confirmed by recent case histories in North America.

Regarding MPSP, one may speculate that, if this technique had been "picked up" earlier by the dam repair community in the U.S., would there now be a) the same apparent level of distrust of grouting, and b) the increasing use of diaphragm walls - as opposed to grouting - to seal leaking foundations? The use of such walls - strongly promoted and cleverly marketed by one particular European contractor - undoubtedly provides a "positive" solution - but at a tremendous cost implication relative to "new" grouting technologies.

With respect to RODUR^{sm}, it would be an interesting calculation on any one project to compare the cost effectiveness of epoxy resin (expensive) in place, to that of cement grouts or polyurethanes (cheaper) repeatedly washed away and settling out in the spillway plunge pool (or the digestive systems of the local fauna).

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