

Forty Years of Evolution in Specialty Construction Processes for Dams and Levees

2019 LEGACY LECTURE PART 2

BY DR. DONALD A. BRUCE

The Winter 2020 Edition (Issue No 183) of the USSD Bulletin *Dams and Levees* published the first part of this 2-part series memorializing the 2019 Legacy Lecture delivered by Dr. Bruce. The Lecture examined the evolution of four specialty geotechnical construction techniques widely used for dam and levee remediation namely Prestressed Anchors, Deep Mixing, Drilling and Grouting, and “Positive” Cut-off Walls. The first two subjects were described in Issue No 183 (pp 22-36) while the latter two subjects are discussed in this issue. In addition, Part 2 also refers to observations Dr. Bruce made on miscellaneous and non-technical issues ranging from Risk Management through Boards of Consultants to Mentorship.

TECHNOLOGY DEVELOPMENTS

Drilling and Grouting

Perspective and Historical Context

The world of grouting is very wide, as influenced by the explosion of conferences and textbooks which have appeared since the early 1980's. Paradoxically, the grouting techniques used in the U.S. to remediate existing dams and levees are surprisingly few although their annual dollar value has probably reached over \$100 million during the period of intense remediation beginning in the early part of this century. The major techniques may be grouped as follows:

- (i) Remedial grout curtains in rock, under and around existing dams and mainly karst terrains, using High Mobility cement-based Grouts (HMG) as defined by Chuaqui and Bruce (2003).
- (ii) Similar “Quantitatively Engineered Grout Curtains” (Wilson and Dreese, 2003) in rock for new dam construction.
- (iii) Similar curtains in rock installed as the precursor to the installation of a “positive” cut-off wall, known as the “Composite Cut-off” system (Bruce et al., 2008).

(iv) Similar treatments using Low Mobility Grout (LMG), erroneously always referred to as Compaction Grouting, (Byle, 1997), in order to explore and treat large Karstic features either as a stand-alone operation, or as an element of a Composite Cut-off, as has been used with great success at Clearwater Dam, MO, Wolf Creek Dam, KY, Center Hill Dam, TN, and Boone Dam, TN (Stare et al., 2013).

(v) Jet grouting in soils underlying existing embankments to form or complete seepage cut-offs or to improve the foundation's seismic response characteristics. The work currently being conducted at Herbert Hoover Dike, FL, is an excellent example of the former application, and the classic case history of Wickiup Dam, OR (Bliss, 2005), typifies the latter.

(vi) The use of both particulate and solution grouts to seal cracks in concrete structures such as has been carried out extensively at Dworshak Dam, ID, (Smoak and Gularte, 1988) and Stillwater Dam, UT, amongst a host of challenging, admirable, and successful case histories.

Clearly it is not feasible to describe the developments which have been made over the last 40 years in all of these categories, or in the myriad of other less common applications. Therefore, the following review touches mainly on the related applications involving rock grouting (i.e., items (i) – (iii)), while bearing in mind that some of the basic principles such as QA/QC and Verification would also apply to LMG operations. Case histories on jet grouting may be found in Bruce (2013), while the enduring 1990 textbook of Prof. Karol provides detailed background on all types of solution (“chemical”) grouts as used for structural repair. One may refer to papers by Bandimere (1997), Byle (1997), and Warner (1982, 2003, and 2012) for further perspective and guidance on LMG operations.

The use of grouting throughout the nineteenth and twentieth centuries is well documented. Prior to the 1800's, grouting was mainly used to strengthen walls and other manmade structures, but little technical description of these processes remains. Members of the grouting fraternity, notably Houlsby (1990), Weaver (1991), and Littlejohn (2003), have endeavored to document the historical use of grouting. For perspective, it would appear that grouting in the United States dates back at least to 1893, when cement-based grout was injected into the limestone formation of a 290-ft-high dam in the New Croton Project,

New York. The prime goal, according to Glossop (1961), was to reduce uplift pressures, and that therefore no attempt was made to construct an “impermeable” cut-off. The curtain was as much as 100 feet deep. Thereafter, there was a “slow advance in grouting technology throughout much of the 20th Century” (Weaver and Bruce, 2007) for reasons that reflect on technological isolation and parochialism in the face of unprecedented amounts of new dam construction and the popular belief in very prescriptive specifications.

In particular, foundation grouting practices up until the early 1980's – as painstakingly chronicled by Albritton et al. (1984) for USACE dams – would feature:

- Highly prescriptive specifications.
- Almost complete absence of rational design and acceptance processes and widespread use of “rules of thumb” for design and execution.
- Use of:
 - Vertical holes to a predetermined depth beneath the excavation surface.
 - Single row grout curtains.
 - Long downstages of predetermined length.
 - Rotary drilling.
 - Low and conservative grout pressures.
 - “Thin” grouts (i.e., with very high water: cement ratios).
 - “Dipstick, gage, and stopwatch” methods for injection control.
 - Termination of work based on grout takes (and/or cost) as opposed to residual permeability.

To further illustrate this “traditional” mentality, one may consider the opinion of James Polatty, formerly of the USACE, and a prominent grouting engineer of the period. In an invited lecture on U.S. dam grouting practices in 1974, he gave the following synopsis:

“In preparing this paper, I requested copies of current specifications for foundation grouting from several Corps of Engineers districts, the TVA and Bureau of Reclamation. In comparing these current specifications with copies of specifications that I had in my files that are 30 years old, plus my observations and experience, I concluded that we in the United States have not, in general, changed any of our approaches on grouting. AND THIS IS GOOD” (emphasis added).

Interestingly, he then went on to cite “difficulty in having sufficient flexibility in the field to make necessary changes to ensure a good grouting job” as a problem on certain of his projects, while “communications and training” was also listed as a challenge.

This technology inertia in fact widely persisted for at least a further decade, despite the notable exceptions overseen by the USACE and the

Bureau of Reclamation at Ridgway Dam, CO, and Stillwater Dam, UT, and by specialty consultants such as Golder Associates (Jefferies et al., 1982). By then, it was becoming clear that the traditional practices were totally unsuited to the late 20th Century demands on remedial grouting posed by site logistics, geology (especially karst), performance and dam safety considerations. Commencing with the visionary work of Wally Baker, Don Deere, Giovanni Lombardi and Alex Naudts, the “new wave” finally arrived with the association of Advanced Construction Techniques (a contractor) and Gannett Fleming, Inc. (a consultant) around 1996 to conduct the grouting work at Penn Forest Dam, PA. Thereafter, advances have continued both with this team, but also with a relatively limited number of other well resourced and technologically excellent contractors and consultants. It must be noted that these developments occurred within the sustaining environment created by the Federal Agencies’ drift towards more Performance Based Specifications, and the unprecedented challenges posed by safely constructing grout curtains under existing high dams with very adverse difficult hydrogeological conditions.

The resulting design improvements have included:

- Curtains comprising two or more rows.
- Inclined holes in each row.
- Depth of curtain determined by geology and/or by rigorous seepage analyses.
- Stage lengths commensurate with the structural geology, and the use of descending stages in upper horizons, and ascending stages in the lower sections of the hole.
- Use of the highest safe grouting pressures.
- Verification of proper stage refusals.
- Verification of residual in-situ permeability upon closure, using investigatory borings placed between the two rows.

DEVELOPMENTS IN GROUT HOLE DRILLING

Rock Drilling

It was traditional in rock grouting practice in the U.S. that rotary drilling – either “blind”, full face or by coring – was the only acceptable method for grout hole drilling. This paradigm persisted for two main reasons:

- (i) That rotary drilling was synonymous with water flush, while percussive drilling used air flush. (The general theory is that water flush preserves the amenability of the fissures to grout, whereas these same fissures would be blocked off by cuttings

driven by the more violent, expanding action of compressed air).

- (ii) Most of the companies who conducted grouting and, especially for the two main Federal Agencies, were primarily site investigation companies and so were equipped with rotary rigs.

However, from the mid 1970's, European manufacturers started producing top drive percussive hammers (hydraulically powered), which permitted water flush to be used. Rotary percussive drilling is typically reckoned to be several times faster and cheaper than pure rotary drilling, and so the advent of these new top hammers provided economic advantages while preserving the technical benefit of water flush. It is now common for specifications to permit either rotary and/or rotary percussive drilling for grout holes in rock, provided water flush is used. The use of top hammer percussion tends to be restricted to holes less than 100 feet long, since penetration rates drop with depth (energy is lost at each successive rod coupling), and hole deviation can become significant, especially for the inclined holes now preferred for grout curtains.

The other type of rotary percussion drilling in rock is the Down the Hole Hammer (DTH).

Introduced in the 1960's, this air powered tool, located at the bottom - not the top - of the drill string, provided fast and straight drilling potential for holes over 3 inches in diameter. Although its main market was for blast hole, oil field and anchor drilling, certain contractors began to propose it for grout hole drilling, ignoring the fact that the compressed air used to activate the tool then constituted the hole flushing medium. To compound the issue, some practitioners began to introduce water into the air supply to create a "mist", which of course just created a moist sludge in the hole, further obscuring fissures from the later penetration of grout.

Beginning in the Swedish underground iron ore extraction industry in the mid 1990's, a water powered DTH (WDTH) was developed,

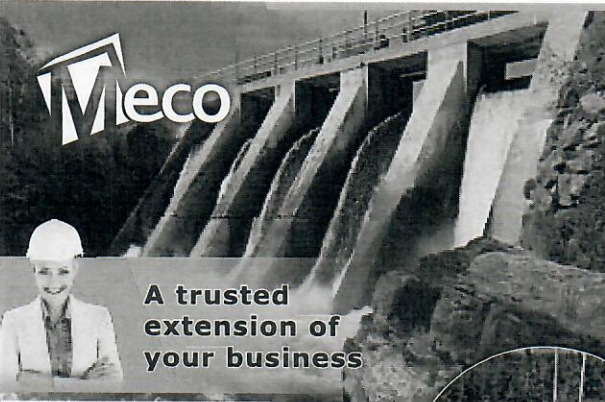
and the reached fully operational stratus soon thereafter. The first application in the U.S. was in the McCook Reservoir Test Program in 2001 where its benefits in drilling deep straight holes in strong limestone were clearly demonstrated. Since then, all the major dam grouting competitors routinely use the WDTH for both new and remedial grout curtains, and for many other applications in heavily karstified terrains.

The air-powered and flushed DTH requires a very high Uphole Velocity to lift the cuttings (Table 1) given the low "density" of air and this can cause scouring of the borehole wall, or in cases where the annulus around the drill rods becomes clogged or unstable softer zones are encountered, pneumatic fracturing of the formation may be induced. In the worst case, this

Table 1. Recommended Uphole Flush Velocities

Drill Flush Medium	Recommended Uphole Velocity
Air or air-water mist	1500 m/min (maximum 2100 m/min)
Water	36 m/min (maximum 120 m/min) (i.e., at least 20 times less than for air)
Medium viscosity mud (35 s Marsh)	30 m/min
Thick mud (more than 50 s Marsh)	18 m/min
Foam	12 m/min

Source: Australian Drilling Industry Training Committee Ltd. 1997, with permission from Taylor & Francis.



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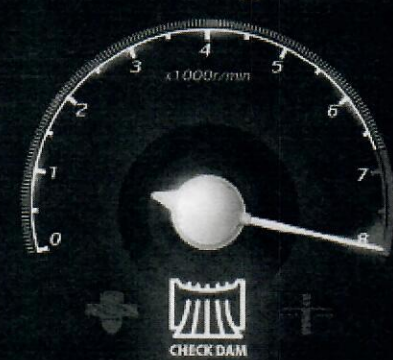
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uncontrolled travel of expanding air may reach up into the embankment itself.

In contrast, the WDTH is powered, and the hole is simultaneously flushed, with the incompressible medium of water (Figure 1). The required UHV of water is far less than for air, and so borehole walls are more gently treated, facilitating subsequent placement (and, hopefully, retrieval!) of down-hole packers. Hydrofracturing is a risk with any water flushed method but is no more likely to occur with a WDTH than with other traditional method such as top drive percussion or rotary drilling. Intense, real-time monitoring of piezometric pressures in rock masses being drilled with the WDTH in dam remediation projects over the last 10 years or so have revealed "spikes" temporarily induced by the close passage of the WDTH. However, in every case known to the author, these spikes have indeed been transient, and in no case has such an event been judged to constitute a dam safety incident of any relevance or scale. It is, of course, interesting to speculate if such transients were always occurring – but never recognized – in previous decades with other types of drill methods. In my opinion, the emergence of the use of the WDTH has been the prime development in rock drilling for grouting practice during the last 20 years.

Overburden Drilling

For the sake of this review, "overburden" is defined as any non-lithified material which overlies the rock mass which is to be grouted. Hence overburden encompasses both man-made materials, such as the fills of an embankment or natural materials such as the underlying alluvial, colluvial, or residual soils. Overburden can therefore comprise the whole range of soils from clay to boulders, with every conceivable mixture in between, especially in urban settings. It is not surprising therefore, that a myriad



Figure 1. Water Powered Down-the-Hole Hammer mounted on a Cubex rig. (Courtesy of ACT Ltd.)

of overburden drilling systems which have been developed over the last 40 years principally in Scandinavia, Western Europe, Japan, and the U.S.

Focusing on drilling through dams and levees, various agencies have in recent years developed their own guidelines for safe drilling methodologies. However, these owe their roots to the overarching USACE ER1110-1-1807 (1997 – revised 2014), which for most cases, precludes the use of air or water flush. The impact of this document on embankment drilling in the U.S. has been major and fundamental.

A review of overburden drilling methods conducted by the author in the early 1980's postulated seven different generic types of

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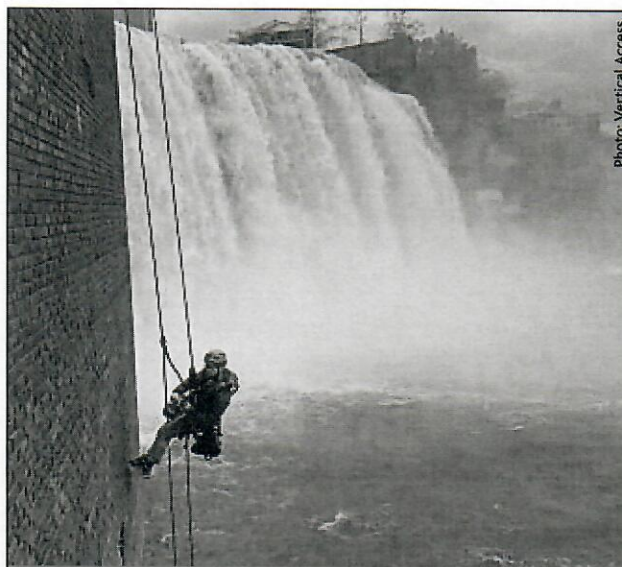
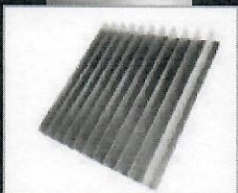


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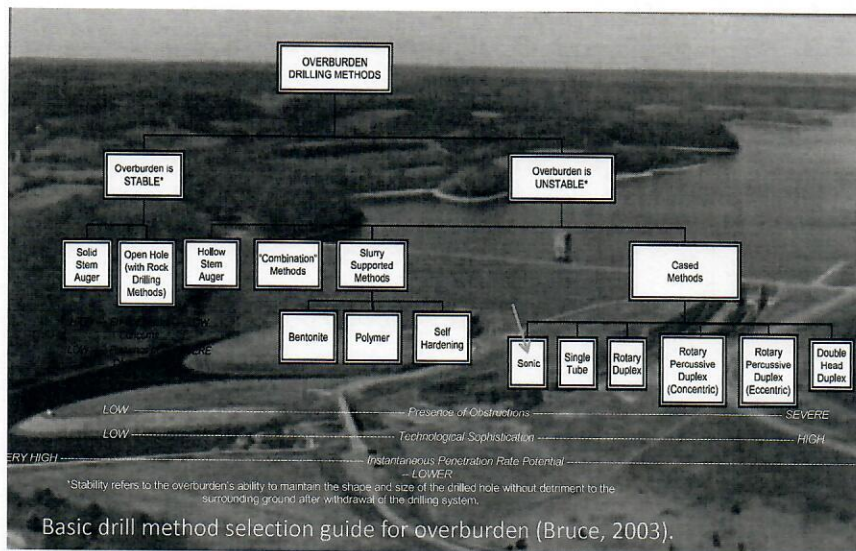


Figure 2. Classification of overburden drilling systems. (Bruce, 2003.)

overburden drilling systems. Although most of them (including drive drilling, different types of duplexing, and hollow stem auger) are still valid in other parts of the specialty geotechnical construction market such as for micropiles and soil anchors, arguably none would now be strictly acceptable for embankment dam drilling. An updated and expanded classification was provided almost 20 years later (Bruce, 2003). As shown in Figure 2, there had been many changes over the intervening period with new and improved systems being developed. However, following the spirit and the letter of ER1110-I-1807, it may be argued that only three methods would be acceptable, on the assumption that the overburden (i.e., the embankment) must be, conservatively, regarded as “unstable” from a

risk driven viewpoint. These three methods are highlighted in Figure 2: hollow stem auger and sonic are obvious choices, whereas rotary duplex/double head duplex systems (Photograph 1) are only admissible if drilled without flush. While the auger method is traditional, tried and tested, it does have depth and geotechnical limitations. The emergence of sonic, or more correctly, rotary sonic, drilling has been the major breakthrough in dam and levee drilling over the last 20 years (Bruce and Davis, 2005) while one of its inventors, Canadian Ray Roussy, in 2002 declared it was “the only true innovation to come to the drilling industry since the Chinese invented cable tool drilling some 3,000 years ago”.

Sonic drilling utilizes a dual casing system with each casing equipped with a hardened casing shoe. The drill utilizes rotation and thrust, as well as intensive vertical vibrations at 50 to 150 Hz from the hydraulically driven oscillator mounted above the casing, to advance the hole once the resonant frequency of the drill pipe is detected (Figure 3). Typically, the inner casing is advanced the full stroke of the rig, then the outer or override casing is advanced to the same depth. During advancement of the inner casing, a continuous but disturbed sample of overburden is obtained. After the inner casing is removed, the sample is extruded by vibrating the casing while pushing out the sample with water pressure. In cohesive soils the samples are minimally disturbed in the interior of the core although samples at the beginning and end of the run may not be representative of the *in-situ* condition. Sonic drilling methods are capable of advancing through bouldery

conditions and even significant depths into bedrock. The vibratory action of the drill string pulverizes the rock at the casing shoe, allowing for reasonable advancement rates in soft-rock conditions. Hard-rock drilling is possible with the method, but the samples recovered are typically significantly disturbed and advancement rates are undesirably slow. Hole depths greater than 200 feet are not uncommon with the sonic drilling method, although telescoping casing is necessary to reduce skin friction below a depth of about 100 feet. Rotary sonic drilling methods typically produce very accurate holes in both vertical and angled drilling applications. The use of dual casings provides a stiff drill string to counteract deviation tendencies resulting from the ground conditions. Casing with sonic drilling also allows for sampling of both soil and rock using standard geotechnical methods such as diamond rotary coring, Shelby tube, and standard penetration testing. Setting of standpipe is simply a matter of removing the inner casing, inserting the standpipe, then retrieving the outer override casing. And, of course, no flushing water – let alone compressed air! – is used during casing advancement.

Monitoring While Drilling (MWD)

Every hole that is drilled is a potential source of information on the properties of the ground and its response as a project progresses. This dictum obviously applies to the (typically limited) number of dedicated site investigation holes cored by specialists and logged by trained technicians and geologists. However, it is equally true for “production” grout holes. Traditionally – if indeed these production holes have been recorded at all for anything other than location, depth, inclination, rod drops, and loss of flush – manual methods have been used. A good driller will have been briefed in advance about the overall details of the ground and will therefore be alerted as to what should be recorded. He will, nevertheless, be on the lookout for “exceptions and unexpected”. Such drillers have been able to greatly enhance the value of a routine production drill hole log by periodic recording of penetration rate, thrust, torque, flush return characteristics (not just total loss), drill “action” or “chatter”, interconnections or surface outbreaks, and hole stability. Given typical penetration rates, these data can readily be observed and recorded at 3 to 5 feet intervals by the driller and/or this helper.

Since the mid 1980’s, developments in industrial software and hardware have allowed drilling parameters to be recorded continuously



Photo 1. Double head, dry duplex drilling system

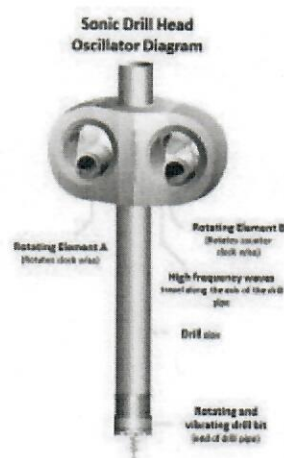


Figure 3. Basic principles of rotary-sonic drilling.

Sonic uses high frequency (50-150 Hertz) mechanical vibration combined with rotation and down-pressure.

Vibrations generated by eccentric counter-rotating rollers in drill head.

Vibrations coincide with the natural resonant frequency of drill pipe.

Drill string produces spring-action and delivers energy to bit face.

Sonic advancement occurs by shearing and minor displacement.

In recent years, it has become increasingly rare on any type of grouting project of substantial scope or complexity to find automated MWD not being used for the mutual benefit of all the stakeholders.

Borehole Imaging

An invaluable gift from the mineral exploration industry has been the use of down hole Optical and Acoustic Televiewers (Figure 5). These can be used in any stable hole and will provide a high-resolution image of the borehole wall as the tool is slowly withdrawn: in effect a "virtual core" is provided. (Figure 6) These images provide data which with proper software and the appropriate interpretative skills can yield:

- rock mass fracture conditions: cumulative thickness, cumulative number, density, and frequency histograms;
- dip plot: arrow plot, stick plot, and rose diagrams;
- borehole verticality: deviation and drift projection onto horizontal and vertical planes and three-dimensional orthogonal projection; and
- stereograms; selectable attributes and depth ranges, including mean-dip and zone-axis calculations.

Such data are invaluable to the grouting engineer in designing, executing, and verifying

and in real-time. One of the first applications was by Rodia SpA when conducting massive permeation grouting projects in the Milan Metro (DePaoli et al, 1987): the data retrieved from each production hole were instrumental in designing the subsequent grouting parameters for that hole. The MWD log of each hole (Figure 4) is shown as the Specific Energy (e), calculated as:

Where:

e = Specific Energy (kJ/m³)

F = Thrust (kN)

A = Cross sectional area of the hole (m²)

N = Rotational speed (revolutions/second)

T = Torque (kN-m)

R = Penetration rate (meters/second)

It is good practice to drill some early MWD holes in close proximity to the traditional cored site investigation holes so that changes in Specific Energy can be correlated to known geological variations. Such data have maximum value when studied in real-time. For example, the characteristics of a Secondary hole may (or may not) be similar to that of its neighboring Primaries: this is another vital piece of information with which to gauge the progressive effectiveness of the grouting program.



Figure 4. The typical output from automated MWD instrumentation.

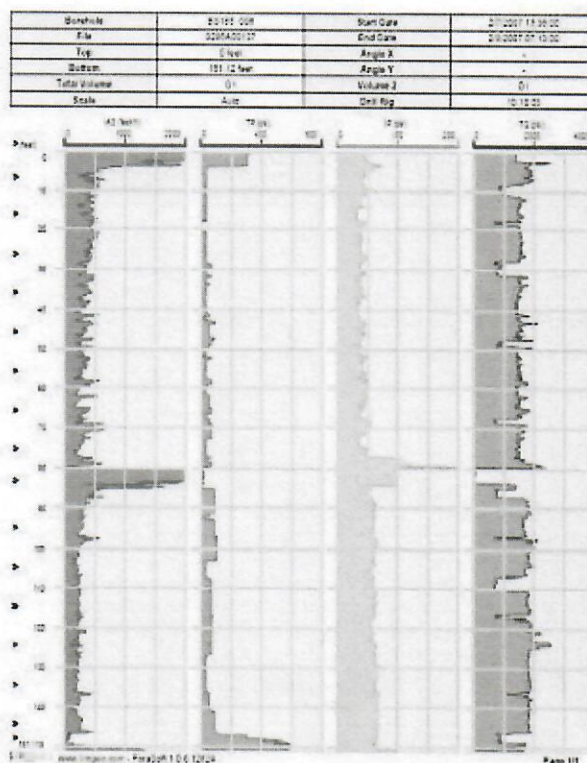




Figure 5. The Optical Televiewer system for high resolution borehole imaging. (Courtesy of Gannett Fleming, Inc.)

his project. Another attraction is that imaging can be conducted in any hole regardless of the mode of its drilling. Hence, it is not necessary to mobilize to site a different verification hole drilling method to that used for the production grout hole drilling.

First used systematically at the McCook Reservoir trial in 2001, borehole imaging has become an integral tool within the grouting industry, to the extent that it has enhanced the quality of the installed product, while at the same time providing project economics.

Design of HMG Grout Mixes

As alluded to above, it was normal to find that "traditional" specifications required cement-based grouting to be commenced with water: cement ratios (by weight) approaching double digits. The hypothesis was that the excess water in the neat grout, (i.e., beyond that required for cement hydration) would carry the cement particles into the finer fissures where they would be deposited, and the excess water would somehow be expelled into the formation. It was also unsaid that equipment suitable for mixing and pumping "thicker" grouts, potentially for long distances did not exist when the original specifications were drafted in the 1920's. However, researches documented by Weaver and Bruce (2007) have now proven without doubt that such "thin" mixes are inefficient in terms of their penetrability (due to their poor pressure filtration resistance) and have poor durability when set due to their low strength and high bleed characteristics. Pressure filtration resistance is a measure of a particulate grout's ability to retain its water during injection under pressure

into a fissure: a grout with poor resistance will simply find its segregated particles smeared over the entrance to the fissure while the water is squeezed out. The actual cement take will be minimal. On the contrary, a grout with superior pressure filtration resistance will retain its water content under pressure and that water will flush particles into the fissure. While it is clear that penetrability is also governed by the grain size of the cement (which is why Type III is preferred over Type I/II), and by cohesion (the lower the better) realization of the fundamental influence of pressure filtration coefficient on grout mix design has been one of the most important advances in the last 40 years.

The greatest majority of dam foundation grouting is conducted by what has been termed "High Mobility Grout" (HMG), to distinguish it from

"Low Mobility Grout" (LMG) or indeed "Medium Mobility Grout" (MMG). The "mobility" term relates to the ease with which the grout can be conveyed and is a product of adjusting the viscosity, cohesion, and/or internal friction of the mix. These rheological properties control a grout's ability to penetrate fractures, and the pressures and processes needed to mix and inject them. For clarification, HMG's are not frictional, but may have high cohesion; MMG's are basically sand-cement grouts; and LMG's are low slump "compaction" grouts with very high internal friction. Although LMG's are becoming more common especially as a pretreatment in Karstic terrains, the improvements in the formulation of HMG's are the story of the last few decades.

As alluded to above, traditional, neat cement grouts (and even those gradually introduced in the 1970's incorporating, bizarrely, a plasticizer) change their rheology during injection due to water being expelled. The consequences are illustrated in Figure 7, which goes a long way to explaining why certain grouting projects in the past did not have a good result even with high order holes drilled at ridiculously close centers. The fundamental breakthrough into HMG design was articulated by DePaoli et al. (1992). The experimental goal of their work was to create a "High Performance" HMG which would be "balanced" with a variety of components to provide low apparent cohesion, (which relates to pumpability and penetrability), minimal bleed, as well as superior pressure filtration resistance. The historical path of development is shown in Figure 8: this is a graph which merits careful study, as it is the key to proper HMG design. Such grouts are formulated with

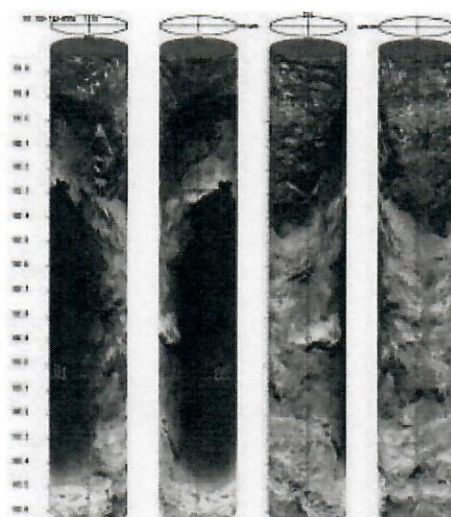


Figure 6. Typical output from the Optical Televiewer.

S36.70U
192.3' - 193.4':
Solution feature in Leipers
Fm.

Wrapped image suggests
feature trends NW-SE,
normal to dam.

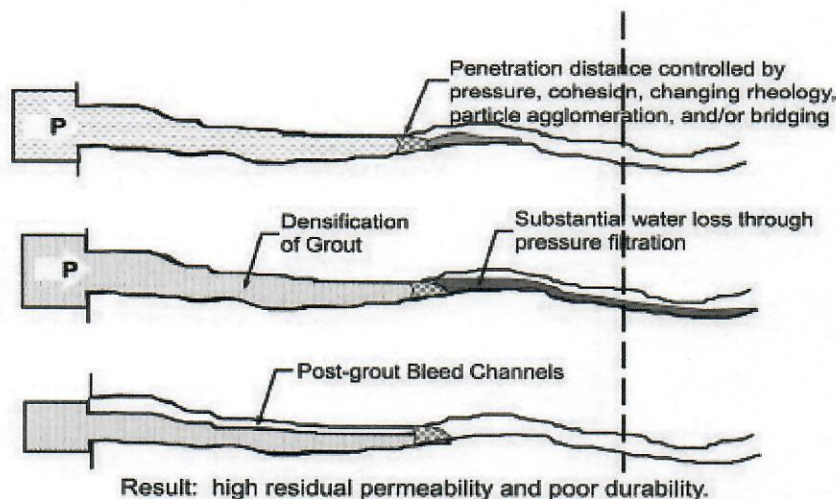


Figure 7. Fissure grouting with an unstable mix.

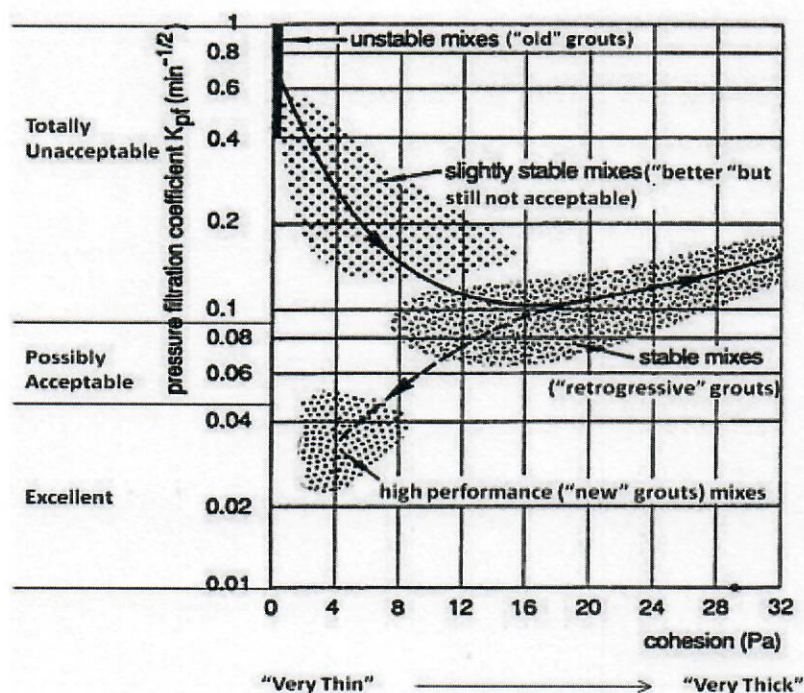


Figure 8. Historical path of development from unstable mixes to contemporary balanced, multicomponent stable mixes (modified from DePaoli et al., 1992.)

Table 2. Typical Fluid HMG Properties ("High Performance" Grouts)

CATEGORY	TEST	REALISTIC LIMITS
Routine Field QA/QC	• Marsh Funnel (Fluidity)	Mix A 30-35 sec Mix B 40-45 sec Mix C 55-60 sec Mix D > 60 sec
	• Baroid Mud Balance (s.g. and therefore WCR)	+ 0.02
	• Bleed (Static stability)	< 2%
Classification/Design Requirement	• Pressure Filtration (Stability during injection)	0.04 to 0.08 min ^{-1/2}
	• Initial and Final Set Times	4-12 hours
	• Cube Strengths	Practically irrelevant

a variety of components designed to mutually compensate for their undesirable effects. For example, superplasticizer is required to reduce the increase in cohesion provided by the otherwise totally beneficial bentonite or polymer stabilizer. It is not unusual to find suites of HMG's involving five or more components (including water and cement). In contrast to the situation illustrated in Figure 7, injection of a stable (but low viscosity) HMG into the same fissure has an entirely different result (Figure 9). Thus, more efficient use can be made of the borehole (which can be more widely spaced) and a more effective and durable product is provided (since the water: cement ratio is far lower, typically in the range of 1.2 to 0.8). Guidance on typical grout mix formulations can be found in Weaver and Bruce (2007) and Stare et al. (2013), while desirable properties are shown in Table 2.

The routine production of such HMG's does require careful and consistent batching, and it is accurate to state that the equipment manufacturers and the Specialty Contractors have together made excellent strides over the last few years to produce both automatic and semi-automatic equipment to consistently satisfy the required HMG properties.

Injection Control, Monitoring, and Analysis

At the dawn of the 1980's in North America, monitoring, recording, and analyzing water and grout injection was a manual labor, exemplified by the dictum of "an inspector at every header" (Photograph 2). Hundreds of major projects were successfully completed with the methods and expectations of the day, and indeed manual control is still common on smaller projects throughout the world. However, by the time of the seminal First International Grouting Conference held in 1982 by ASCE in New Orleans, LA, there were a few - but robust - signs that automated monitoring was coming to age. This was driven by a small number of visionaries - such as Alex Naudts, the contractor Wally Baker, equipment manufacturers such as Hanyunder Reinhard Muller, and a group of consultants at Golder Associates, who in New Orleans presented a strong case (Jefferies et al.): "Modern electronic techniques can be used to successfully monitor cement grouting projects. The potential cost saving to the client is such that electronic techniques should be used.

While electronic monitoring is justified on cost grounds alone, the profession should not overlook the opportunity for improvements in grouting quality, reliability, and marketability

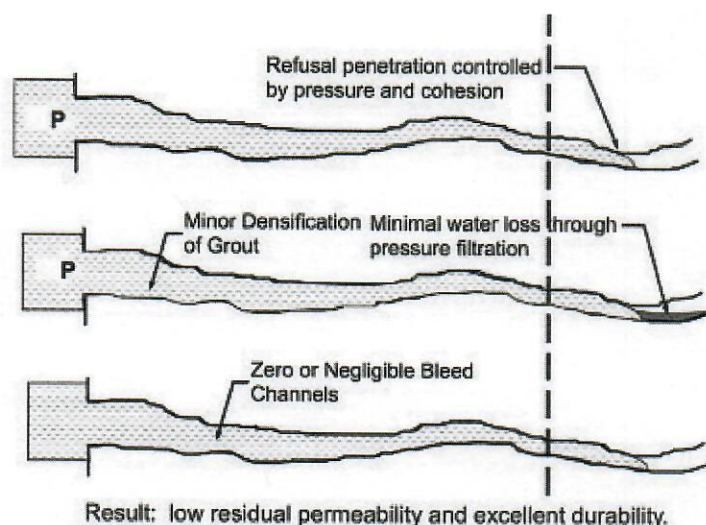


Figure 9. Fissure grouting with a stable mix.

that result from the application of modern electronic techniques. If it can be demonstrated to the civil engineering profession at large that grout injection is an understandable process which can be monitored and which can produce predicted results reliably, then grouting will become more accepted as an engineered ground treatment process. This would benefit both owners and grouting contractors."

The traditional methods are often - bluntly but accurately - referred to as "Dipstick, Gauge, and Stopwatch technology": Wilson and Dreese (2002) and Dreese et al. (2003) of Gannett Fleming, Inc. more rigorously labelled it as the Level 1 approach. Level 1 does not utilize electronic pressure gauges and flow meters that are now readily available in the market, nor does Level 1 Technology incorporate the speed and power of the computer that makes most of the calculations required for monitoring and analysis fast and accurate.

Dreese's team led developments in computer monitoring via their link with Jim Cockburn in Advanced Construction Techniques on a number of very large, very complex remedial grouting projects conducted mostly for USACE, from 1996 onwards. By the time of the Fourth International Conference on Grouting (and Deep Mixing) held 30 years after the first, Hockenberry and his colleagues provided an excellent summary of the state of practice, and one whose messages resonate with the 1982 conclusion of Jefferies et al., referenced above. They began by defining Levels 2 and 3 as follows:

- Level 2: Real-Time Data Collection and Display Systems
- Level 3: Advanced Integrated Analytical Systems

Having considered the limitations of Level 1 Technology, Hockenberry and his colleagues concluded only Levels 2 and 3 Technologies are recommended as they alone are truly

able to produce real-time displays of grouting operations, including water pressure testing. Further, Level 3 was considered the superior level of technology in the contemporary grouting industry (Photograph 3). They continued: "technology, the engineer or geologist operating the system has the tools and capabilities to provide onsite technical support and real-time assessment of the grouting results in addition to monitoring the operations." "Level 3 Technology is comprised of four major components that when combined, produce a unique and powerful monitoring system:

- (1) A real-time data display of information retrieved from field operations;
- (2) A central database to store all collected and calculated information;
- (3) Linked customized CADD functions to automatically display up to the minute information stored in the database on demand; and
- (4) Customized queries to quickly and accurately mine data from the database for daily report generation and up-to-date analytical capabilities. The CADD display allows the project team to visually observe the results as they are obtained and assess the project status in relation to adjacent hole series and lines. The real-time monitoring and analytical capabilities allow the operator to make sound engineering efficiency."

"The linked functionality of the grouting database to both CADD and customized queries is what makes true Level 3 Technology a powerful tool for fast and accurate analysis of grouting results. CADD plots and query analyses can be performed on demand and contain up-to-the-minute information. The rapid nature to which current grouting information can be presented and displayed for analysis by the project team



Photo 2. Traditional injection control and recording.



Photo 3. Contemporary state of practice in injection control. (Courtesy of Gannett Fleming, Inc.)

is of substantial benefit over technologies that require a longer turn-around time of information in the form of drawings and queries, time on the orders of weeks, days or even hours. True Level 3 Technology gives the user the ability to present information on demand in a matter of minutes. This is especially important for projects of a technical nature that require special attention to grouting progress and risk reduction. True Level 3 Technology is also important for projects with widely varied and unpredictable subsurface conditions, such as karst formations, that require careful, and relatively quick analysis and grouting method selection."

Hockenberry and colleagues also described MWD and Borehole Imaging, as reviewed above. That is a strong confirmation of the "all sources" integrated approach to the science of grouting as it now stands in the industry. Other related technologies are Digital Photography and Automated Instrumentation. The former allows geological mapping of areas with difficult accessibility and is capable of producing three-dimensional models including digital terrain models that can be directly incorporated into the project and subsequent geotechnical evaluations. The latter permits real-time, continuous recording and display of data such as piezometric variations and displacements. This is now a routine practice in mitigating risk and assessing system response particularly in remedial grouting projects.

One major outcome of the precision with which grouting can now be controlled is that the highest possible safe injection pressures can be used to increase the penetration of the grout. Any distress in the ground can be predicted or very quickly identified so that injections can be suspended, absolutely minimizing the volume of grout injected in such undesirable occasions.

SOME OTHER VERY IMPORTANT DEVELOPMENTS

Refusal vs. Closure

Each stage of every grout hole must be brought to a true and proper "refusal". This means that the maximum specified pressure has been held over a certain period (say 5 minutes) at a certain maximum flow rate (typically 0.25 to 0.75 gallons per minute, depending on the project's residual permeability goals and the nature of the rock mass). "Closure", on the other hand, is when a section of the curtain has been judged to have been completed with a degree of certainty that the subsequent verification test holes will indeed confirm that the target residual permeability has been achieved. A proper judgement of closure

depends on a holistic analysis of all available and relevant information, such as drilling logs, water tests, grout take analyses, and dam instrument response.

Lax refusal criteria will result in incompletely and inefficiently grouted stages. Given that the cost of drilling is usually the most expensive operation in foundation grouting programs, each and every hole should be viewed as an opportunity to reduce the bedrock permeability and should be grouted to the fullest extent possible. This includes grouting as close to absolute refusal (zero flow) as possible. How close to absolute refusal should the designer set the criterion? According to Housby (1990), "My practice is to go as close as the order of accuracy of the measuring methods allows". In Housby's day, the method of measurement was typically performed with dipsticks and stopwatches to measure differences in volume in the agitator tank versus time, which is why refusal criteria from the era were typically set at one cubic foot or less measured over 10 minutes (0.75 gpm). With the current use of electronic flowmeters and real-time monitoring computer systems, the higher accuracy and complete time-history plot allows the project team to grout to as low as 0.2 gpm or less depending on the application, the target permeability and site logistics and ambient temperature. Many current specifications, however, have not taken advantage of the increased accuracy of these new systems and have kept the refusal flow criterion closer to 0.75 gpm.

Many designers may also view the additional time to grout to a lower refusal flow criterion as unnecessary and expensive. On the contrary, the last amounts of grout injected during the final refusal of a grout stage can constitute the most important time and effort spent to achieve closure, especially if the time spent reduces the number of additional high order holes to be drilled. Designers should consider the costs of drilling additional holes as well as the implications in regard to dam safety (especially for holes that need to be drilled and installed through embankments) when setting refusal flow criteria.

Refusal criteria should also consider the frequency of joint intersections and or the likelihood of not connecting directly to a solution feature or solutioned widened joint. Grouting these higher permeability features through a tighter bedding plane can result in the refusal criterion being "achieved" prior to complete filling of the solution feature. The result is a high residual permeability that will likely go unnoticed

until tested by reservoir filling or a slurry loss during cut-off wall construction.

True vs Apparent Lugeon Values

The value of water pressure testing as a routine and integral part of a proper grouting program has long been recognized (e.g., Housby, 1976, 1990; Bruce and Millmore, 1983; Paisley et al., 2017). In our practice, the unit of measurement is the Lugeon, named after Maurice of that ilk. He defined it as a flow of one l/m/min at an excess of pressure of 10 bars (that being equivalent to the head exerted by a typical dam in the French Alps in 1933). The water pressure test projects a true Lugeon value, although its precision may be sometimes debatable given all the various head flow corrections which have to be made, or indeed simply ignored such as in the robust Modified Lugeon Test Procedure of Housby (1976), widely used.

One of the gifts we accept from the use of stable grouts (i.e., those grouts with minimal pressure filtration) is that they maintain a relatively constant rheology during their period of injection: they do not allow water to be squeezed out of the mix, into fissures, when under pressure. Thus, a grout with constant rheology (i.e., like water has) can be regarded as not only a fissure filler, but also a test fluid. This allows us to calculate at any given time, an Apparent Lugeon Value, calculated in the same way as a Water (True) Lugeon Value, but corrected by a factor which is calculated as Marsh Flow Cone time (grout) divided by Marsh Flow Cone time (water).

Grout curtains should always be grouted to a target True Lugeon Value (proving the residual permeability). Individual stages should be brought towards refusal by observing and controlling the Apparent Lugeon Value, subject to the final "refusal" flow rate discussed above. Grout curtains should not be brought to closure based solely on Apparent Lugeon Values, as the authors have observed on a major recent project in the Pacific North West. Water testing of the final hole series and verification holes must still always be performed to prove the residual permeability of the treated rock mass.

Data Management Systems (DMS)

It is now common practice for an Owner to insist that the Contractor collects, presents, and stores all his construction data in some form of an automated DMS (Data Management System). I have found that Contractors are often overwhelmed (initially at least) by the needs and scale of such a system. This must

not be permitted by the Owner to occur: a fully compliant DMS must be up and running before production work is allowed to commence. From the technical viewpoint alone, it is integral to informing all interested parties on the progress and effectiveness of the work (e.g., via Trend Analyses), and is therefore the only true basis for determining when the work is actually complete or if further work (e.g., higher order holes) is required, and where.

The Contractor's DMS must be controlled by a suitably qualified grouting engineer (not only or necessarily an IT specialist). There must be a similarly-equipped, dedicated, and experienced counterpart on the Owner's side. It is essential that all parties (including a Board of Consultants) are granted access to the single set of information, transcending objections occasionally based on the "proprietary" nature of the data. Historical information on the project, original construction data, and information on dam performance should also be loaded into the DMS prior to the new grouting work commencing, so that new inputs can be compared to these sources, especially when unexpected results are obtained. The requirements of the DMS, the JIMP and long-term monitoring, as described below, obviously apply equally to remediations featuring a "positive" cut-off wall.

Joint Instrumentation Monitoring Plan (JIMP)

It is normally the case that the dam to be remediated will already have some amount of instrumentation which may already have highlighted the actual need for remediation, in concert with visual observations. Further instrumentation is invariably added just before or during the grouting, to target specific "problem areas", address construction PFM's, or simply to ensure a broad deal of coverage without data gaps. The result is a plethora of instruments, typically now configured to provide data in real-time.

It is essential for Dam Safety Assurance during construction that the Owner and the Contractor partner to collect, study, and act upon these data in real-time, regardless of whose contractual responsibility or liability it may be. The most efficient strategy is to create, prior to construction having commenced, a Joint Instrumentation Monitoring Plan (JIMP). This will identify which instruments are to be read, by whom, and at what frequency. The JIMP should also provide Threshold and Action Level guidance for each instrument and identify courses of action when these levels are reached.

A JIMP is implemented most effectively when the Owner and the Contractor can view the data while their representatives are together in a Joint "Mission Control" facility. In this way, the impact of the Contractor's work on the dam and its foundation can be seen in real-time and acted upon accordingly as necessary by the Owner.

Long-Term Monitoring

I have consistently observed that after the curtain or cut-off has been built, little attention is paid to continuing to use the in-situ instrumentation to monitor the long-term efficiency of the cut-off. Even less effort is normally devoted to publishing such data for the benefit of the dam remediation community who can enjoy analyzing a successful (or unsuccessful) case history. Such long-term monitoring is the responsibility of the Owner and should be regarded as an essential cost of doing business. It is equally the source of performance data for future PFMA sessions or "event" investigations.

Overview

Over the last 40 years there has truly been a technological "revolution" in dam grouting, combining various complimentary developments in design, construction, monitoring, and control. During this period, large scale remedial curtains have been built, mainly in heavily karstified carbonate geologies, but all with a satisfactory, quantitatively engineered result. The concept of the "composite cut-off" has been born and is an integral element in securing the safety of high-risk dams for the long-term.

The major Federal Agencies, and organizations such as the TVA, are broadly in favor of performance specifications and innovative procurement vehicles for their big remedial projects. There is now a skilled and deep pool of specialty contractors who have the human and mechanical resources to conduct these works. Consultants have become more entrepreneurial, embedded within the contractors' teams as much as they have traditionally worked for owners. This is a good and sensible outcome for all parties!

Positive Cut-off Walls for Dams

In the first part of this Legacy Lecture, Deep Mixing Methods (DMM) were described. Many of the DMM variants have been used to successfully and economically construct cut-offs, through dams and levees founded on non - or poorly - lithified foundations, to depths of up to 100 feet or so. Such cut-off methods

are described by Bruce (2013) as belonging to Category 2, and the cut-off itself obviously consists of an in-situ blend of the nature soil, or fill, with a cementitious "binder". This blend is usually referred to as soilcrete or soil-cement. However, such methods will not be applicable if the depth through the embankment is over 100 feet and/or the cut-off must reach deep into the underlying rock foundation, such as karstic limestone. Under such circumstances, a Category 1 cut-off is required. This involves the complete excavation of a trench, typically under a temporary stabilizing medium such as bentonite or polymer slurry, followed by the replacement of that slurry with an engineered backfill material. This material can be designed to range from conventional, high strength concrete to "plastic" (or low modulus) concrete, while some earlier case histories have featured the use of soil-cement-bentonite mixtures. Category 1 walls are not built by a continuous trenching method such as by a long reach backhoe for many reasons, first among them being to assure the interim safety of the embankment during remediation. Instead, it is built in discrete but eventually overlapping panels (diaphragm wall method) or by overlapping large diameter secant piles, or some combination thereof. It is typical to find the minimum specified thickness of the cut-off to be in the range of 18 to 36 inches, but this is project specific.

The first U.S. application of a Category 1 cut-off in an existing dam with most of the reservoir head still applying was at Wolf Creek Dam, KY in two phases between 1975 and 1979, and so it is a serendipitous starting point for this 40-year review.

Another concept which has been alluded to in the drilling and grouting section of this review, is the "Composite Cut-off Wall". First articulated by Bruce et al. (2008), a composite cut-off involves first the pretreatment of the rock mass in the location of the subsequent "positive" cut-off by two or more rows of grout holes which:

- Pretreat the rock mass to a certain residual permeability (typically 10 Lu) to prevent sudden and potentially catastrophic loss of stabilizing slurry into the rock mass during element excavation.
- Provide in effect an extremely intense site investigation of the whole potential extent of the cut-off, thus allowing its limits to be defined with precision.
- Allow the effect of the positive cut-off to be extended (usually deeper) by the grouting to

provide an engineered residual permeability into rock judged to be non-erodible.

Consequently, the concept allows the cut-off wall to be built safely, and only through those depths and extents containing potentially erodible materials (e.g., karstic residuum). The subjacent competent rock is sealed (more economically) by the grouting. The concept has been applied in all the major cut-off wall projects in the U.S. since 2004.

A second consideration springs from the Terzaghi Lecture to the Geo Institute of the ASCE which I delivered in 2015. The title was "The Evolution of Specialty Geotechnical Construction Techniques: The Great Leap Theory". Between 1858 and 1865, the great Scottish historian Thomas Carlyle wrote a 6-volume opus on the life and times of King Frederick the Great of Prussia. This work had followed his 1841 masterpiece "On Heroes, Hero-Worship and the Heroic in History". In these publications, Carlyle developed what we now call the "Great Man" theory of history, which postulates that "the history of the world is but a biography of great men". I then extended that theme to reflect the development of the theoretical aspects of geotechnical engineering but challenged that theory when considering the evolutionary drivers in specialty construction techniques.

Instead of the "Great Man" theory, it was clear that the "Great Leap" theory had been at work in these techniques. "Great Leap" theory, put simply, states that the technological developments in specialty geotechnical construction are not incremental, slow, or progressive. Rather, evolution occurs in discrete and often startling leaps, triggered by the demands of one special project or groups of projects. Obviously, there



Photo 4. Major karstic features in the foundation, Wolf Creek Dam, KY. (Courtesy of USACE.)

is a high place of honor for those behind the controls, typically entrepreneurs with the vision, courage, and confidence to try new things. In this category our late friends Arturo Ressi, Alex Naudts, Wally Baker, Ken Weaver, Fernando Lizzi, Harry Schnabel, Renato Fiorotto, Clive Houlsby, and Tony Barley spring to mind.

To constitute recognition as a "Great Leap", I proposed that six successive criteria must each be satisfied:

Criterion 1: The project, group of projects or application must be of exceptional and/or unprecedented scope, complexity, and construction risk.

Criterion 2: There must exist a specialty Contractor who has the ingenuity and resources to devise the solution and there must exist a

manufacturer who can design and build the primary equipment which is to be used.

Criterion 3: There must exist a responsible individual and/or agency on the Owner's side who is prepared to take the perceived risk of deploying a new technology or technique on his project.

Criterion 4: The project must be successful!

Criterion 5: Details of the project must have been published widely in the scientific or professional technical press, and not just as another case history in a trade magazine, regardless of how interesting and well-presented these can be.

Criterion 6: Within a few years, there must be some formal codification or other influence over construction processes, to assure the legacy of the "Great Leap", and to guide and tutor future evolutions, the dubious defense of patents notwithstanding. In our field, such recognition typically comes through the publications of a Federal Agency, such as the U.S. Army Corps of Engineers, the Bureau of Reclamation, or the Federal Highway Administration; a professional society such as USSD; or the efforts of a trade association such as the Association of Drilled Shaft Contractors (ADSC), the Deep Foundations Institute (DFI), or the Post-Tensioning Institute (PTI).

The perfect example of a "Great Leap" has been the two periods of cut-off wall construction undertaken about 30 years apart at Wolf Creek Dam, KY. By considering the technologies used during each phase, evolution in the construction of such positive cut-off walls can be clearly demonstrated, using the framework of the six defining criteria.

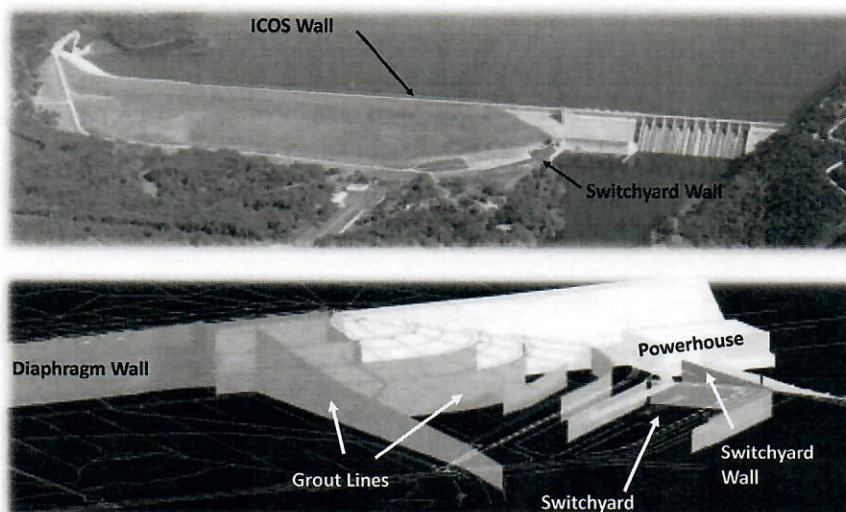


Figure 10. First remediation concept, Wolf Creek Dam, KY.

CRITERION 1: THE EXCEPTIONAL NATURE OF THE PROJECT

The dam comprises a 3,940-foot-long homogeneous fill embankment and a contiguous 1,796-foot-long gated overflow section (Figure 10). Both are founded on Ordovician limestone formations with major karstification. The dam stands a maximum of 258 feet above river level and impounds Lake Cumberland, the ninth largest reservoir by volume in the U.S. and the largest east of the Mississippi. In essence, the dam is a 1930's design, having been authorized in 1938, and built from 1941 to 1943 and 1945 to 1952 with the three-year hiatus occurring during World War II.

Signs of seepage-related distress were noted soon after first impoundment and became more pronounced in the following 15 years. Only extremely intense remedial grouting programs conducted by the USACE in 1968-1970 and again in 1973-1975 arguably saved the dam from a failure mode involving erosion and piping of the weathered material in the foundation, and through the clay fill placed in major karstic features (Photograph 4), extending to over 75 feet below top of rock (Kellberg and Simmons, 1977; Fetzer, 1979; Simmons, 1982; and Mackey and Haskins, 2012). Details of the successive phases of grouting were provided by Bruce et al. (2014). It was, however, recognized by USACE and their Board of Consultants that the grouting operation was but a stopgap, given the capabilities of the grouting technologies of the period, and the certainty that potentially erodible material remaining within the foundation would continue to allow seepage to develop under the high ambient hydraulic gradient.

A competition was arranged to invite technical proposals for the "permanent" solution to the foundation problem. Seven potential techniques were proposed by various contractors, including a wide range of grouting options, and ground freezing. Only two potential options were accepted by the USACE as being appropriate for further development. The competition was won by the ICOS Corporation of America, under the leadership of Dr. Arturo Ressi de Cervia. Their approach was to build a concrete cut-off wall, from the dam crest extending a maximum of 100 feet into the foundation rock. Partly reflecting the mechanical capabilities of excavation equipment of the time, and partly as a consequence of budget considerations, the main wall was designed to extend along about two-thirds the embankment length (a total of 2,237 feet from the concrete section) to a maximum depth of 280 feet below the dam crest. It was

nominally 24 inches thick, and comprised over 531,000 square feet, built in two consecutive phases of work. A smaller wall (600 feet long and 95 feet deep) was constructed downstream to protect the switchyard. Together, the walls were built from 1975 to 1979 at a cost of around 97 million dollars (1970's currency). This was a unique achievement, being the first example of a remedial concrete diaphragm wall installed through an existing, fully operational dam. Hitherto unprecedented levels of quality control, assurance and verification were developed and enforced (Couch and Ressi, 1979).

However, even during these original remedial works, at least one Board of Consultants Member (Dr. Peck) expressed the opinion that the cut-off would really have to penetrate deeper into rock, and to extend further along the embankment to prevent seepages eventually developing under and beyond the cut-off, such was his interpretation of the hydrogeological model. And, of course, he was proved correct.

By 2001, the typical signs of distress had begun to reemerge, such as piezometric anomalies, sinkholes, wet and soft spots, and muddy seepages. Following intensive investigation, instrumentation, and evaluation, the USACE declared in January 2007, that the dam merited a Dam Safety Action Classification I (DSAC-I) status, i.e., a scenario demanding immediate intervention ("Urgent and Compelling"). The USACE therefore reduced the reservoir elevation by 80 feet below maximum capacity as an Interim Risk Reduction Measure and planned other early interventions. An emergency remedial grouting operation was initiated in 2007 to stabilize the situation and to act as the first part of a "composite wall" prior to the construction of a second, longer and deeper cut-off wall.

The new, minimum 24-inch-thick cut-off wall was to be built upstream and independent of the first, and to extend 1,650 feet beyond and 75 feet below the existing wall to provide 980,000 square feet of new cut-off. Strict construction tolerances were specified to control strength, permeability, homogeneity, and continuity. Further practical restraints, driven by dam safety concerns, were placed on operational aspects such as the minimum allowable distance between open individual slurry-supported elements under excavation. The area near the concrete structure was designated the "Critical Area", given its unfavorable geological conditions, original construction details, and previous seepage performance characteristics. The critical nature of the situation demanded that time was of the essence, and a construction period of approximately four-and-a-half years was originally set. Further, the technical specifications had a major "Performance" leaning to encourage bidders to develop innovative and responsive techniques, while at the same time assuring compliance with the extremely rigorous construction compliance criteria. Most importantly, the safety of the dam had to be assured during all activities, including penetration of the embankment, passing through the embankment/rock contact, and excavation into the karstic foundation. It became obvious to all bidders that the technology of the 1970's could not satisfy the numerous and more intense demands posed by the 2008 project.

CRITERION 2: AVAILABILITY OF THE TECHNOLOGY

The original cut-off wall had been built under two consecutive contracts using a clever combination of telescoped, large diameter rotary drilling (to allow the installation of about 990

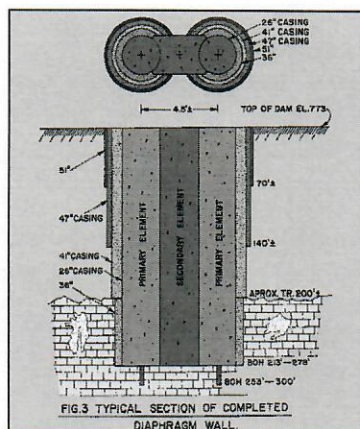
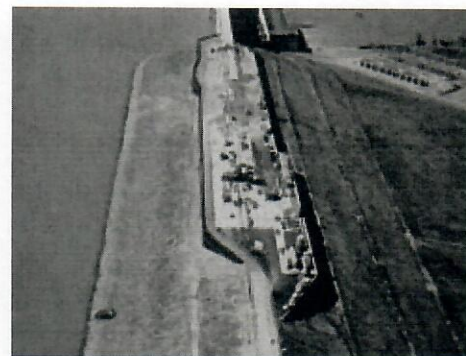
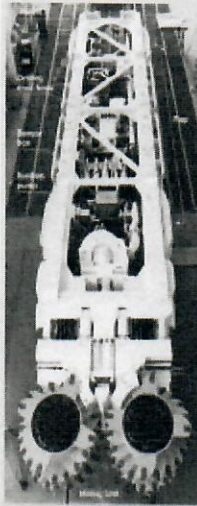


Figure 11. Details of first cut-off wall construction, Wolf Creek Dam, KY.



HYDROMILL TECHNOLOGY



The core of any Hydromill is its trenching/cutting unit, that schematically consists of a heavy steel frame integrating the following components:

- swivel located on top of the frame
- two independent hydraulic engines which allows the rotation of a pair of milling drums located at the bottom of the frame;
- a mud suction pump placed just above the milling wheels;
- front and side hydraulically-operated "steering" flaps;
- a number of built-in sensors and inclinometers.

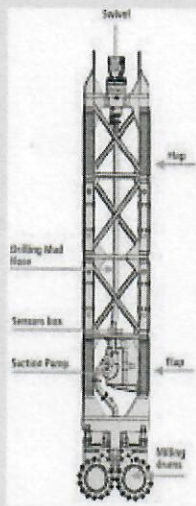


Figure 12. Schematic of a hydro-mill. (Courtesy of Soilmec.)

26-inch diameter steel guide pipes at 54-inch centers) followed by conventional clamshell excavation (to remove the ground between the guide pipes) using biconvex clamshells (Figure 11). Both of these techniques had been used separately on other projects involving deep foundations and support of excavation by the ICOS Corporation, but never to the same depths or to such exacting standards of care of in an active dam environment. Diaphragm wall specialists from ICOS' sister companies in Europe

(and particularly from Italy) were deployed in support of regular North American staff, many of whom had the construction of the diaphragm walls at the World Trade Center site in the late 1960's on their long resums.

However, around the time of this first Wolf Creek cut-off, a technological advance of fundamental impact on diaphragm walling construction was being made in France by Soletanche, now part of Soletanche-Bachy Group. The piece of equipment in question is

termed hydro-fraise by the French and is also known as a hydro-mill or a cutter by other Italian and German firms who have developed their own models. As shown in Figure 12, a hydro-mill comprises a rigid steel frame upon which are mounted cutting wheels and a powerful reverse circulation slurry pump. The hydro-mill is introduced into a minimum 15-foot-deep "starter trench", previously excavated and already filled with bentonite slurry, and the cutting wheels and suction pump are activated. Debris cut by the wheels are removed from the trench in the suspending bentonite slurry by the suction pump. The slurry is "cleaned" at surface de-sanding and/or de-silting plants, and fed back into the trench, supplemented by fresh slurry, to ensure the trench remains topped up and therefore stable. Subsequent technical developments relating to hydro-mills included placing hydraulically activated plates on the frame, which together with adjustments to the rotational speed and direction of the cutting wheels, permit the hydro-fraise to be steered within the trench to satisfy tight verticality tolerances ($< 1\%$ depth). Typical trench widths range from 25 to 55 inches, but special machines have been produced to provide up to 84-inch widths. Rock of up to 20,000 psi unconfined compressive strength can be cut using appropriate teeth or picks on the cutting wheels, and careful operational procedures.

Following the application for Patent by Soletanche in 1972, the first commercial use of the hydro-mill was in Paris, at the Centre Français du Commerce International (Pers Coms, Richards, and Jousselein, 2015). From 1973 to 1974, about 20,000 square feet of load bearing barrettes were constructed, and the success led to the much larger series of projects at the Gare de Lyon, also in Paris, between 1974 and 1978. The first use of a hydro-mill for a new dam cut-off was at Jebba Dam, Nigeria, where in 1981 and 1982, over 364,000 square feet of plastic concrete wall were installed to depths of over 200 feet (Soletanche, 2002). This was followed by similar 24-inch-thick plastic concrete cut-offs at Brombach Dam, Germany in 1983-1984, and again in 1985. These cut-offs totaled over 550,000 square feet and had a reported maximum verticality deviation of only 0.13% (Soletanche, 1999). The first remedial dam cut-off installed with a hydro-mill was at the USACE's St. Stephens Dam, SC in 1984, featuring 78,000 square feet of plastic concrete wall, 24-inch-thick, plus 28,000 square feet of soil-bentonite panels. This was soon followed by the test section at the U.S. Bureau of Reclamation's Fontenelle Dam, WY in

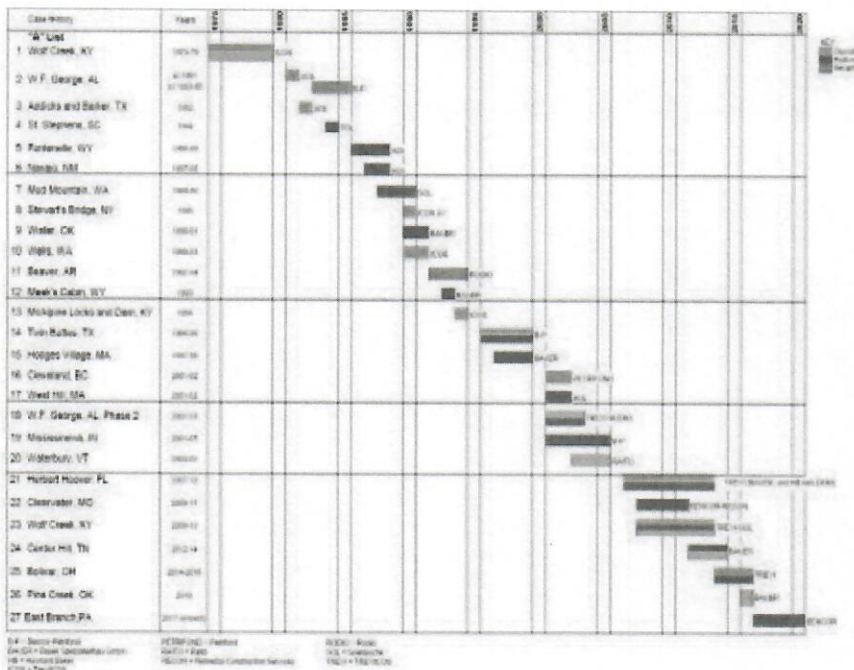


Figure 13. Project listing of remedial cut-off walls in US Dams 1975-2019.

1985, and the subsequent production work of 1987-1988, together totaling 850,000 square feet of cut-off (Bruce et al., 2006). During the same period, three trial panels had been successfully installed to over 330-foot-depth, near Milan, Italy, with deviations *controlled* to within 0.6% (Bruce et al., 1989).

Thereafter, hydro-mills were used to wholly or largely construct similar remedial walls in eight other U.S. dams up to 2007 for a total of about 2.4 million square feet of cut-offs. Since then, further major remedial cut-offs have been constructed at several other U.S. dams (Figure 13) while the technique as deployed by Bauer Construction had been used to build a cut-off wall over 380 feet deep at the new Peribonka Dam, Quebec, (Adnan and Balian, 2008).

By the time, therefore, that bids for the second Wolf Creek project in 2008 were solicited, the "ICOS" method and the hydro-mill technique had become common knowledge and practice in the cut-off wall industry. The problem, however, remained that the risks – technical, quality, dam safety, and schedule – posed by Wolf Creek were unprecedented. There was also the little issue of financial risk on a competition bid project, the overall cost of which was estimated at about 600 million dollars.

The solution adopted by the successful bidder – a TreviCOS-Soletanche JV – combined and leveraged the particular strengths of their respective companies:

- TreviCOS had been formed in Boston in 1997 when the Trevi Group, from Cesena, Italy, acquired the ICOS Corporation of America, the successful builders of the first Wolf Creek cut-off. The new company also acquired the assets of the RODIO Group, based in Casalmajocco, Italy, and who had been an active participant in the Beaver Dam, AR and W.F. George Dam, AL secant pile cut-off walls. TreviCOS' particular corporate strengths were therefore in large diameter drilling especially using Wirth reverse circulation pile top equipment from Germany (Figure 14) and conventional clamshell excavation, while their more recent developments had also advanced skills in directional drilling techniques, as well as with hydro-fraise technology. As example, Chiarabelli and Pagliacci (2014) reported on an 800-foot-deep test panel installed at Gualdo, Italy in 2012. Adjusted by hydraulically-operated "steering flaps", their hydro-mill ("Tiger") was guided to within a verticality of 0.13% at the terminal depth.
- Soletanche were by 2008 long established as a leading specialty geotechnical contractor in

Secant Piles

- Following the directional drilling pilot hole, 1.3 m diameter piles installed at 80 or 90 cm centers
- Ensuring the required overlap and minimum thickness. – Max target depth 85 m

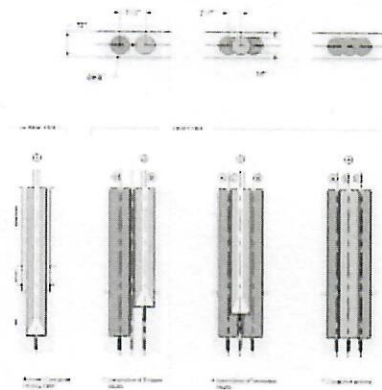


Figure 14. Construction of secant piles, Wolf Creek Dam, KY.



North America and had acquired Nicholson Construction Company in addition to other construction assets. The company had continued to develop hydro-fraise technology, with a focus on improving productivity and verticality control methodologies. In particular, a new generation of hydro-mill had recently been developed, which proved the key for efficiently and precisely excavating a 72-inch-wide, 535,000 square foot "Protective Embankment Concrete Wall" (PCEW) designed by the JV to act as physical protection to the dam embankment and its contact, during its subsequent penetration by the secant piles comprising the cut-off in rock. The PCEW was installed from a berm on the upstream side of the dam. The reverse circulation drilling created the 1,197 secant piles of 50-inch diameter spaced at 35-inch centers required to satisfy the minimum cut-off wall thickness criterion of 24 inches. Eight-inch diameter directionally drilled holes (using Water-Powered Down-the-Hole Hammers) were first used as pilots through the PCEW and the underlying rock to guide each large diameter pile. Eighty percent of these pilot holes subsequently recorded deviations at 282 feet depth of less than three inches (Santillan, 2013), while the balance of the deviations were also compliant.

Combining their respective skills and resources, both in-house and external, the TreviCOS-Soletanche JV was judged by the USACE to be the most responsive bidder and, as such, commenced site operations in the Fall of 2008. Integral support was provided to the JV by equipment suppliers such as Wirth and Wassara and by specialty subcontractors,

principal among were Hayward Baker, Inc. who were responsible for completing the grout curtain in rock, which had been started in 2007 by Advanced Construction Techniques with Gannett Fleming, Inc. Extremely sensitive deviation monitoring instrumentation was specially designed and tailored for the major pieces of drilling and excavation equipment under the direction of Mimmo Sieppi of Trevi.

CRITERION 3: RISK ACCEPTANCE AND MANAGEMENT BY THE OWNER

Faced with an extremely delicate dam safety situation, the USACE and its Board of Consultants in 1975 had made the courageous decision to adopt the innovative ICOS proposal. Given personal knowledge of the man, it may reasonably be assumed that Arturo Ressi's considerable communication and engineering skills were strongly tested. However, no Board including Dr. Ralph Peck and Prof. Mike Duncan would have condoned a method that it felt would pose unacceptable risk to the dam during construction.

The second Wolf Creek project carried in many ways a significantly higher technical risk than the first, especially since it was conceived in a period of rapidly growing understanding and awareness of Potential Failure Modes Analyses. Again, it is to the immense credit of USACE, its Board, and its Consultants that the deep diaphragm wall techniques proposed by the TreviCOS-Soletanche JV were accepted, subject to the requirement to conduct and successfully verify elements comprising of Technique Demonstration Areas before production work was permitted along the balance of the cut-off.

Other major risk management strategies embraced by the USACE included:

- Best Value" contract award basis, with an emphasis on the narrative of the Technical Proposal.
- Very exacting levels of QC/QA and Verification. Implementation of an Intense Joint Instrumentation Monitoring Plan (JIMP), and a comprehensive DMS system.
- Effective, efficient, and frequent technical Partnering sessions, including full involvement by USACE's Board of Consultants, and their counterparts, the Contractor's Internal Advisory Panel.

CRITERION 4: THE SUCCESS OF THE PROJECT

The second Wolf Creek cut-off wall employed nine different specialty construction techniques to assure that the specification requirements for the cut-off's verticality, strength, homogeneity, permeability, and minimum thickness were met, and that the safety of the dam was always assured (Santillan et al., 2013). These nine techniques were sonic drilling, high mobility grouting of the rock, low mobility grouting of the contact, clamshell excavation and hydro-mill excavation of the PCEW, water-powered DTH directional drilling, auger/bucket drilling, reverse circulation drilling, and verification coring, with many of these processes often running concurrently in a very congested site (Photograph 5). Only one of the secant elements was found to be out of tolerance: it had been installed at the beginning of the project in one of the Technique Demonstration Areas, to

depths below the actual designed cut-off toe elevation. The wall was verified as otherwise conforming to the specifications in all regards. Over 380 quality control tasks were conducted each day, and about 500,000 tasks had been conducted by the project's conclusion (Bassola et al., 2013). The wall was completed 9.5 months ahead of the revised target date without a dam safety incident. The personnel safety record was astounding: over 1.4-million-man hours worked without a lost time injury. The early observations of piezometers, and of other signs of distress, confirmed the effectiveness of the wall, under full reservoir conditions.

CRITERION 5: TECHNICAL PUBLICATIONS

In addition to numerous informational and promotional publications by the USACE and the JV, there have been over 20 technical publications detailing various aspects of the project, released between 2010 and 2018. These papers have appeared in the trade and professional magazines, and also in the annual conferences of ASDSO and USSD, in the U.S., and in international conferences worldwide, including ICOLD. Hopefully, there will be further publications dealing with the medium and long-term performance of the cut-off as more data and observations are compiled by the USACE.

CRITERION 6: CODIFICATION

The Risk Management Center of the USACE began producing an Engineering Manual on the design, construction, and evaluation of cut-off walls for dams (and levees). This greatly

enhances the scope of Chapter 9.4 of the existing Engineering Manual EM 1110-2-1901. The DFI Slurry Wall Committee developed a guideline document relating to the selection of specialty techniques for dam and levee remediation. In addition, the Bureau of Reclamation finalized a new Design Standard on cut-off walls, under the guidance of Mark Bliss. These documents provide support and update for the existing ICOLD Bulletin 150, and the European Standard EN1538.

In addition, many of the "lessons learned" during the later works at Wolf Creek, were incorporated into the technical specifications for the subsequent USACE cut-off wall projects at Center Hill Dam, TN, East Branch Dam, PA, and Bolivar Dam, OH, and by TVA at Boone Dam, TN. This trend away from Performance-type specifications towards more Prescriptive specifications, has come as a result of sharing the experiences of Wolf Creek with industry at large, and is contrary to the general clamor in the industry to move towards Performance-type specifications. Doubtless, this technical communism will lead to more competitive bidding between the "usual suspects", but one feared that the extraordinary standards of performance and quality established at Wolf Creek Dam would slip in the face of commercial pressures, on subsequent projects. Based on my personal observations as an advisor on these more recent projects, my fears have been totally allayed: in the field of remedial cut-off wall construction in North America, there has indeed been a "Great Leap" in the period since the first Wolf Creek and the current standards of practice. Owners, Regulators, Consultants, Contractors, and Equipment Manufacturers should all, together, take a big step forward, and share a deep and well-deserved bow (but please still keep focused on your next gig).

MISCELLANEOUS OBSERVATIONS

I took the opportunity to present some observations of a general, non-technical nature to "sow seeds" for the wrap-up of the Lecture, namely a "Question and Answer" Session moderated by Dr. Greene.

Dam Safety Assurance Programs

Every dam owner now has or should have in place a formal Dam Safety Assurance Program (DSAP). Even 20 years ago, such programs were either not always formalized, or in certain cases, were not in existence at all. Such programs represent an extremely positive development, which owes much to Owners' responses to

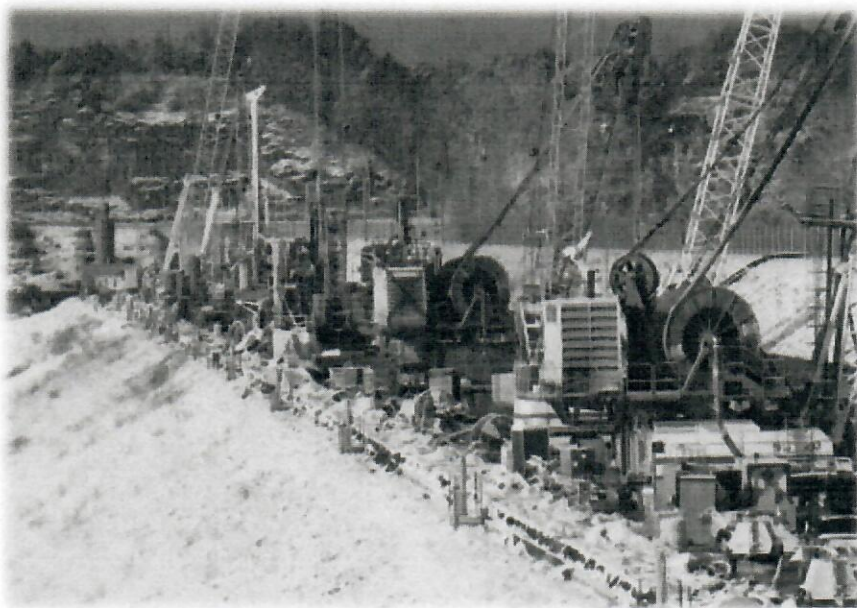


Photo 5. Construction of second cut-off wall, Wolf Creek Dam, KY.

specific project events, stretching back to Teton Dam and beyond, and including Hurricanes Rita and Katrina, and the ash pond failure at Kingston, TN. Classic examples of such programs are those which have been developed by Federal agencies such as the Risk Management Center of the USACE, the Bureau of Reclamation, California DSOD, and TVA. I then took the model of TVA to illustrate how such programs have evolved, and I am indebted to TVA's Dam Safety Officer, Ms. Jennifer Dodd, for the following detailed input.

As background, their DSAP was established in 2010 as a long-range plan to more fully understand issues which could potentially influence the safety of river dams (leading to a potential impact on TVA assets), and also to reduce the risk of a dam safety event.

- drivers to establish the DSAP were the Kingston Ash Spill, after which TVA adopted FERC-based Dam Safety requirements, as well as nuclear facility commitments.
- "ROR (river) dams, while having many years of good performance, require some re-evaluations and rehabilitation. This included updating the design basis to modern industry standards, evaluating dams for potential failure modes, performing risk screenings and analysis, and correcting any identified issues." (TVA Internal document, May 31, 2013).

The DSAP is comprised of 3 parts:

- Operational Improvement Program - "get well"
- Assessment Program - "we don't know what we don't know"
- Rehabilitation Projects

The DSAP was initially established as a 30-year program.

The overall purpose of the DSAP is to protect lives and property by enhancing safety and reducing dam risk by ensuring the dams are designed, constructed, operated, and maintained as safely and as reliably as is practical. This is accomplished by inspecting, monitoring, and performing and documenting the design, construction, operation, maintenance modification, and emergency preparedness for each impoundment and dam to TVA's Dam Safety standards.

The management framework for the DSAP is shown in [Figure 15](#) which illustrates the levels of effort, complexity, and commitment inherent in administering a contemporary DSAP. The Dam Safety Inspection program comprises routine inspections (formal, intermediate, and monthly) to ensure the operational and structural integrity of the water retaining

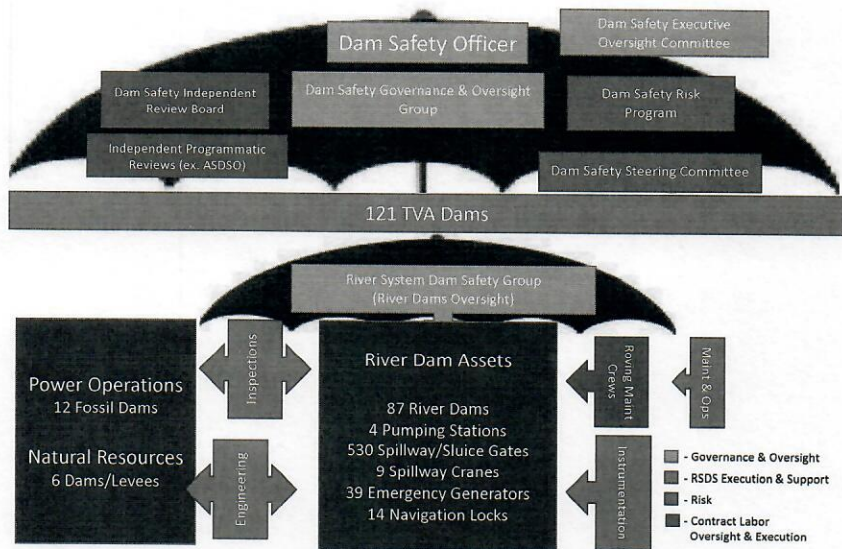


Figure 15. TVA Dam Safety Management Framework. (Courtesy of TVA.)

structures. Special inspections are made after extreme events such as heavy rains, floods, or earthquakes, while emergency inspections are made if a sudden change in performance characteristics is observed, such as a sinkhole or "muddy show" on a project in karstic terrain. Such inspections are in addition to a vast array of structure and foundation instrumentation, much of which is now automated and capable of being monitored remotely. While most inspections are relatively straight forward, depending on keen visual observations by trained personnel, it may also be necessary to conduct inspections of certain structures by rope or cradle access,

the use of a ROV, or by diving. Increasingly, the potential of Lidar is exploited to help detect sinkhole developments or other deflections in and around embankments.

A Dam Safety Maintenance Program provides preventative and corrective maintenance functions, over a wide range of components such as trash racks, gate chains, and riprap. This work is carried out by dedicated, roving crews operating to strict timetables and procedures.

The DSAP is a living, evolving activity, and is updated by analyzing extreme events, and re-evaluating periodic stability analyses (particularly so for embankments under potential

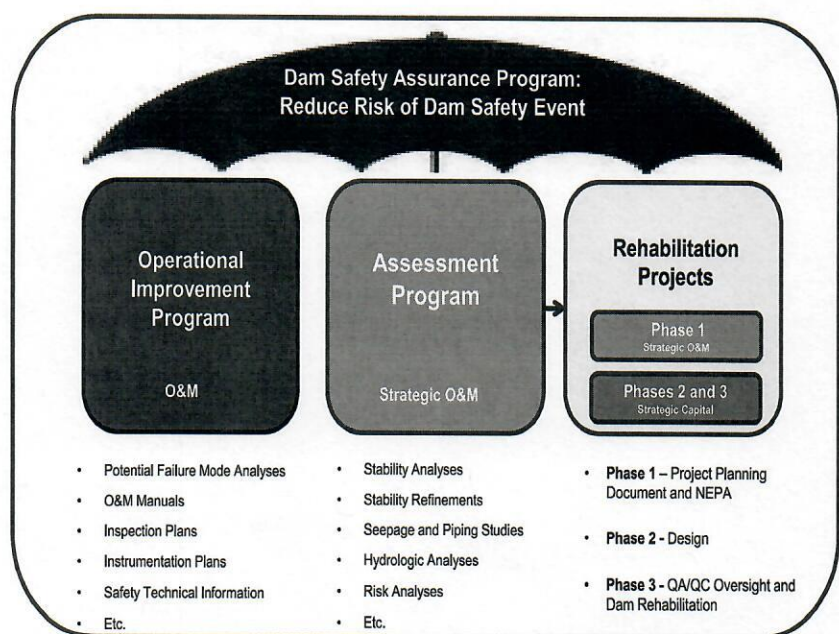


Figure 16. TVA Dam Safety Assurance Program components. (Courtesy of TVA.)

seismic threats). As a result, the need for a major dam modification is identified; the current work at Boone Dam is a prime example, following on the heels of major remediations at Pickwick, Pear Creek and Blue Ridge Dams. All the actions and programs described above ultimately inform the Portfolio Risk Management Framework, and Dam Safety Oversight, as summarized in [Figure 16](#).

Comments on the PFMA Process

Clearly this has been one of the most significant and valuable dam safety tools of the century, having introduced a much-needed rigor and discipline to a subject which had become, well, too subjective: prioritization of remediations has thankfully moved away from the "squeaky wheel" pressures of earlier days. However, the soul searching which has - approximately - been conducted since Oroville has highlighted aspects of the process which merited improvement, and these changes have been implemented by FERC and others. Perhaps after years of uniformly training facilitators, and our frequent individual participations in days-long PFMA sessions, industry had perhaps become a little over-familiar, relaxed, and maybe even complacent about the power and underlying meaningfulness of the PFMA process. My suggestions for the future, which could be either formally or informally recognized, include the following:

- Since the sessions tend to be strongly influenced by the minority of strong, vocal senior personalities present, we must guard against systematic - even if well meaning - influence especially when determining the classification of each PFM which has been proposed. At that time, there should be a "paper vote", not a "show of hands" vote where the opinion of your supervisor is to be ignored at your peril.
- Perhaps it is time to reappraise the descriptions of the four categories of PFM's (i.e., 1-4), and indeed add more classifications if that could help the process. Sometimes it feels that we choose Categories 2 and 3 as if by default.
- Our facilitators are, without exception in my experience, trained and prepared superbly. However, especially in SQRA and QRA sessions, it is essential that common sense and cold logic are still engaged when developing individual PFM's. This is particularly needed when evaluating PFM's associated with internal erosion and piping.
- It is essential for the group to fully understand how the dam was built. There is a world of difference between PFM's which are viable

for a conventional compacted fill dam, and those which would be applicable to an old sluiced "puddle core" structure. In this regard, a comprehensive STID is essential and should be studied by all participants in advance of the meeting.

- If everyone is thinking alike, then somebody isn't thinking" (General George S. Patton).

Site Safety

The changes in attitudes, processes, and - yes - consequences in site safety have been remarkable over the past four decades. Days spent on site, clad in athletic attire, and without a hard hat, are long gone, and rightly so. "Safety" is no longer lip service: it is a culture and the most important aspect of any construction activity. A company's safety statistics are often a prime condition for acceptance of their bid. Specific examples of enhanced safety initiatives include:

- frequent "tailgate" meetings prior to the shift commencing,
- "stretch and flex" routines thereafter (these clear the mind as much as they stretch the muscles),
- a "safety moment" presentation before major meetings such as BOC events,
- development of technique-specific and site-specific safety manuals and operator familiarization courses,
- collection and display of safety statistics,
- zero tolerance on failures to comply with requirements for PPE,
- frequent, random tests for drugs and alcohol,
- rules and restrictions on parking, driving, and use of cellphones,
- the use of the "buddy system" when conducting site walk-downs and inspections (particularly with more mature Board members),
- the training and deployment of Safety Officers having a direct reporting line straight to the overall Project Manager,
- monetary rewards for achieving safety goals.

Assurance of the safest practical working conditions is vitally important in attracting people into the civil engineering workforce, and then keeping them content and productive, as well as safe.

Boards of Consultants

Such Boards, or Panels, or Teams have long played a most important role in the successful planning and execution of major projects. Not only do the Board Members get the opportunity to provide experience-based insight, but the very

act of preparing for such a meeting forces the parties, and especially the Engineer-of-Record, to pause for a while and undertake their own internal reviews and deliberations prior to making the formal Board presentation.

The number of members in a Board varies with the complexity and technical scope of the project but is typically between two and four. So called "standing" Boards who are engaged to provide a long-term oversight role to a certain entity, may have a larger number of participants since issues beyond the purely technical are often within its purview. The frequency of meetings again varies with the nature and urgency of the project: an "emergency" Board may meet every month (with intermediate update conference calls), whereas "routine" Boards for less pressing projects may convene only two or three times per year at most.

Not all Boards function equally effectively or happily. In my experience, the most effective and enjoyable Boards have been following elements in common:

- Strong subject matter experts with up-to-date knowledge and experience, drawn from different organizations. The members respect their colleagues' particular field of expertise while always challenging conclusions in a positive atmosphere of debate, in private deliberation/preparation sessions.
- The members have minimal egos, but strong personalities.
- There is strong personal and professional synergy between the members, who are employed to create an Independent Board of Consultants, not a Board of Independent Consultants.
- The Board report is finalized within a week of the preliminary "read out".
- In meetings, the members are advising and teaching, not pontificating and showboating.

These points should all be considered when the Owner conducts his selection process. Boards should never be involved with the process of procuring a particular Engineer or Contractor but should be prepared to offer perspective on Prequalification requirements. They should never instruct the Engineer as to his business but must offer realistic and practical recommendations for consideration (but prefixed with words like "strongly", for emphasis!). However, Boards should be invited to review the Engineer's Reports and Submittals, especially as these relate to successive phases of a Design Report (e.g., 25%, 50%, Final). Finally, it is for mutual benefit that Boards make regular visits to the site(s) under their purview, not least

because of the opportunities to interface with the personnel "at the sharp end".

Mentorship

Mentorship is essential for encouraging good habits and common sense in younger engineers and is something we value increasingly in our older years as our mentors leave us. Happily, mentorship is something that most of us do, often without realization or formal structure: we encourage technical authorship, and the challenging of paradigms, and we provide perspectives based on years of national and international travel work (a.k.a. "war stories").

However, with the growing - and understandable - reliance being placed on computer skills and discipline, it is becoming more important for the development of our young engineers that the mentoring is conducted in a more dedicated and systematic way. This particularly applies when dealing with staff members from different national, cultural, and linguistic backgrounds.

Many larger companies have formal mentorship schemes in place, but these are frequently unsuccessful given "billable hour" commitments imposed on both the mentor and the mentee: it is simply too easy (even if it is true) to say that "there is not enough hours in a day to do this". However, the most effective mentorship is often provided during participation in a project-related team and does not require a separate dedicated period of time to be carved out of a full schedule. It is for this reason that mentorship is often best in evidence in smaller companies or offices, where the executive lunchroom is the only lunchroom.

What can be done to improve the situation? Simply put, there are two lines of attack:

- Every young engineer should find a mentor - inside or outside of this/her own company.
- Every company should recognize mentorship as a "core value" in corporate business planning and in promotional and recruitment activities.

Let's formalize the informal!

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