

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

At the invitation of the Foundations Committee of USSD (Chairman Doug Boyer of FERC), Dr. Donald Bruce, President of Geosystems LP, delivered the second Legacy Lecture to open the Technical Program of the 39th USSD Annual Conference and Exhibition in Chicago, Illinois. The lecture was given on the morning of Monday, April 8, 2019 and was attended by a full house of over 300 registrants. The title of the presentation was "Forty Years of Evolution in Specialty Geotechnical Construction Processes for Dams and Levees," and the bulk of the lecture was broken down into four sections dealing with technological issues, namely Prestressed Rock Anchors,

Deep Mixing, Drilling and Grouting, and "Positive" Cut off Walls. There then followed observations on miscellaneous issues ranging from Risk Management, through Boards of Consultants, to Mentorship. The Lecture concluded with a lively Questions and Answers session, moderated by Dr. Brian Greene of Gannett Fleming, Inc., who had also been nominated to provide the formal introduction of his friend Dr. Bruce to open the session. Dr. Greene's opening remarks follow.

Further details of the technologies described in this paper may be found in the 2013 book "Specialty Construction Techniques for Dam and Levee Remediation," by Dr. Bruce.

DR. GREENE'S INTRODUCTION

Dr. Bruce is only the second USSD professional to be invited by the society to present this distinguished lecture. Having worked with Donald Bruce for some three decades, it was my distinct honor to be asked to introduce my friend and colleague. Besides our mutual interest in dams and dam rehabilitation, Donald and I share Celtic origins in common, which are frequently celebrated in appropriate fashion.

Donald Bruce was born in the city of Glasgow, Scotland. In the late 1960's, he attended Robert Gordon's College in Aberdeen for secondary school where he also played on the Cricket and Hockey Teams. In 1969, while completing his secondary

academic studies, Donald received the prestigious Otaki Scholar Award leading to a three-month visit by sea to New Zealand. He then entered the University of Aberdeen where he majored in Geology and Mineralogy. From 1973 through 1977, Donald worked on doctoral studies in the Department of Engineering at Aberdeen. Donald's dissertation topic dealt with the fundamental behavior of prestressed rock anchor systems and was performed under the guidance of his mentor and advisor, Professor Stuart Littlejohn (Bruce, 1977).

After earning his PhD, Donald joined specialty construction firm Colcrete Ltd. as a Project

Engineer. From 1977 to 1979, he was responsible for drilling and grouting works conducted for four major dams in England as well as several old coal working infill sites in Scotland. From 1979 to 1980, Donald acted as Technical and Planning Manager for the Colcrete/Rodio-Keller Joint Venture for the massive remedial works performed at Tarbela Dam in Pakistan. As you know, Tarbela Dam was at the time the largest earth-fill dam in the world. While at the site, Dr. Bruce oversaw several million dollars of work involving rock anchors, soil and rock grouting, slab-jacking, and drainage systems.

INTRODUCTORY REMARKS

One dictionary definition of "legacy" is "that it is something handed down from an ancestor or predecessor or from past ancient philosophers. As far as I am aware, I am not an ancestor of anyone in this readership, I am still alive, although some may say I am approaching the "ancient" phase, in the philosophical sense. It has been my particular pleasure to observe in my professional career—most of which has now been spent in the United States—the evolution of many of the specialty geotechnical construction techniques which are now regularly used for dam and levee remediation. As a contractor, consultant, researcher, and advisor to Owners and Regulators, I have been granted insights "from all sides." These perspectives have recently led me to advance the "Great Leap" theory of technological advance in our field. This was argued in depth in my Terzaghi Lecture of March 2015 (which can still be accessed on You Tube): in essence, technological advances are not steady and incremental, they are swift, opportunistic, and episodic, driven by specific project demands which challenge the ingenuity of contractors and manufacturers, and test the confidence, insight, and trust of designers and owners.

It is not the theme of this presentation to recite the "Great Leap" theory. Rather, for each of four major dam and levee remedial techniques reviewed, it is the intent to illustrate the advances, regardless of their cadences, in the forty-year period up to 2019. The purpose of establishing this historical record is two-fold:

- i. preserve for posterity the developments made during that period of unprecedented remedial activity for our dams and levees, and
 - ii. create a baseline for future researchers who will chronicle (hopefully) similar, significant advances in the third and subsequent decades of the Twenty-First Century.
- Although our technical literature is wide and rich, it is replete with somewhat unattractive and unhelpful genres, for example:
- a. overly commercial publications, usually in the form of self-promoting case histories (ahead of schedule, below budget, proprietary technique...), and
 - b. revisionist texts, typically authorized by hungry academics attracted to the lure of industry, in attempts to establish a "go to" profile.
- Neither extreme is represented in this presentation.

By now, the numbers of dams and levees and the statistical and historical analyses of these are well established. According to the National Inventory of Dams (NID), there are well over 84,000 dams in the U.S. meeting at least one of the following criteria (Ragon, 2011):

1. High Hazard classification—loss of one human life is likely if the dam fails,
2. Significant hazard classification—possible loss of human life and likely significant property or environmental destruction,

3. Low hazard classification—no probable loss of human life and low economic and/or environmental losses, but the dam:

- Equals or exceeds 25 feet in height and exceeds 15 acre-feet in storage,
- Equals or exceeds 50 acre-feet storage and exceeds 6 feet in height

At least 15,000 other dams have been reported, but which do not meet these criteria. Almost 14,000 dams have been classified as "High Hazard". It is important to note that only about 4% of the dams are Federally owned (e.g., by USACE [United States Army Corps of Engineers], Bureau of Reclamation, and Forest Service) and only a further 2% or so are owned by public utilities and therefore subject to oversight by the Federal Energy Regulatory Commission. Over two thirds are privately owned. Over 87% of the dams are primarily classified as embankments of some form. About 50% of our dams were completed between 1950 and 1979, while the median age in the year 2020 is sixty-nine years.

Whereas one can calculate that all the dams in the U.S., if placed end to end, would form a structure about 18,000 miles long, preliminary estimates put the total length of levees as over 120,000 miles. According to Halpin (2010), there are levees in all 50 states, and in addition to the uncertainty regarding their number and extent, there is general ignorance about their current condition and the threat they pose to the population and property they protect. It is clear, however,

Thereafter, Donald was appointed Managing Director of Colcrete in Hong Kong overseeing major tunneling and grouting work. In 1983, he was advanced to the position of Main Board Director in the U.K., responsible for the operation of Colcrete's Geotechnical Engineering Division. His work included many projects in Egypt and the Middle East.

In 1986, Donald came to the U.S. filling the position of Technical Director with Nicholson Construction Company in the firm's office near Pittsburgh, PA. During his years with Nicholson until 1996, Donald also spent time on

construction sites throughout Europe gaining first-hand experience in innovative areas of ground treatment and ground support. One of his key responsibilities was to transfer "European Technologies" to the U.S. market.

Since 1996, Donald has been President of Geosystems LP, a national and international consultancy focusing on specialty geotechnical construction processes such as drilling and grouting techniques, anchors, micropiles, deep mixing, cutoffs for dams, and in-situ reinforcement. It was during his early years with Nicholson that I first met Donald. The project was

Point Marion Lock Cofferdam, a U.S. Army Corps of Engineers project located on the Monongahela River, 90 miles south of Pittsburgh. We have been working on Board of Consultants for dam projects together ever since.

Donald Bruce's distinguished career of nearly five decades has been one of distinction and accomplishment. It has been said - "A legacy is not someone that has just amassed many years, but more importantly...what one has accomplished during those years." Clearly, Donald Bruce fits this definition!

Dr. Brian H. Greene, April 2019

that tens of millions of people live and work in close proximity to levee systems: originally constructed to safeguard property, they have often inadvertently increased flood risks by attracting greater development to the flood plains they were built to protect.

Until the 1930's, levee construction was sporadic and unsophisticated, without the benefit of systematic engineering or scientific expertise. Levees were considered "simple" structures and so not designed or built to contemporary dam standards. After great devastation and loss of life in the interwar years from the floods in the Mississippi and Ohio River Valleys and in Florida, the USACE was directed, at full Federal expense, to take a more active role in levee design and construction, resulting in the 15,000 miles or so of "robust" levee systems now in place. Many of these levees, which make up the backbone of the nation's levee system, are now therefore well over seventy years old and, even when regularly maintained, may or may not have been brought up to the most recent engineering standards. The balance of the inventory includes municipal, local, and agricultural structures.

Dam or levee failure can arise from one or a combination of factors relating to design, construction, or maintenance shortcomings on the one hand, and fundamental geological and geotechnical controls on the other hand.

In this regard, soluble lithologies immediately underly about 40% of the contiguous United States (Martinez et al., 1998) while there are several regions of extreme seismic activity in the central, south eastern, and western states in particular. To these challenges must be added those transients posed by extreme weather events, causing flooding and overtopping, while the problem in the New Orleans area is exacerbated by the ongoing regional settlement of the delta, estimated at 0.1 to 0.5 inches per year.

Regarding the "human" factors, one may cite designs which would now be judged deficient in many aspects e.g., sliding, and overturning stability, seismic resistance, inadequate or non-existent filters, and through-going penetrations (i.e., culverts) with seepage collars in compacted embankments. Further, one finds uncontrolled or ill-advised construction materials and processes, leading, as examples, to improperly compacted fills, or Alkali Aggregate Reaction in concrete. Then there is the issue of minimal—if any—instrumentation/monitoring and maintenance, especially prior to 1980. It would be inaccurate to ascribe these shortcomings only to lack of funds, although most dams are or were owned by financially limited groups such as farming corporations, flood control or irrigation authorities. Also contributing would be the "if it ain't broke, don't fix it" mindset which

prevailed for decades before regulation and periodic safety analyses were enforced in response to dam and levee failures occurring with worrisome regularity in an ever-aging population of structures. It may be observed that such failures still occur despite all our best efforts: current practices are excellent but not perfect.

TECHNOLOGY DEVELOPMENTS

Prestressed Rock Anchors

Historical Perspective

First deployed as permanent structural elements in Cheurfas Dam, Algeria, in 1934, prestressed anchors have been used in U.S. dams for remedial purposes since 1962. The most common applications have been to arrest sliding and/or overturning, while certain agencies, such as TVA, have used them to combat Alkali Aggregate Reaction expansions, and in western states seismic retrofit has provided further opportunities. There is no Standard—in the strict sense—governing anchor technology, but practitioners would be sorely pressed to not base their design, construction, stressing, and testing procedures on the Recommendations of the Post Tensioning Institute (PTI). As shown in Table I, this document has been through several iterations, reflecting evolutions in practices, and changes in the balance of the drafting committees' affiliations. The first version in 1974 was largely a promotional document forged by the post tensioning materials suppliers. This may be contrasted with the current comprehensive guidance document of 2014, the product of 50 years of experience and a balanced, more inclusive authorship. Within this series, the document of 1996 may be regarded as the "sea change" from earlier years, especially with regards to attitudes towards corrosion and corrosion protection. Although the 2004 and 2014 versions are more voluminous, the basic principles and technologies established in the 1996 version stand.

From 2004 to 2007, the first phase of a National Research Project on Anchors for Dams was conducted by Bruce and Wolfhope (2007 a and b). The three main goals were to:

- Collect and analyze all the successive versions of the PTI Recommendations, to illustrate how the technology evolved;
- To collect and analyze all technical papers written on North American dam anchoring projects, to confirm trends and lessons learned; and
- To compile a database of all dam anchoring projects in North America.

ASPECT	1974	1980	1986	1996	2004	2014
Post-Tensioning Suppliers	6†	4*	4*	2*	3*	4
Anchor Contractors	2	2	3	3	3	3*
Consultants	2	None	1	2	2	2
Owners/Regulators	1	2	1	3	3	3
Sponsor Organizations	1	None	0	1	1	2
TOTAL	12	8	9	11	12	14

* Including the same Chairman from DSI (Post-Tensioning Supplier).

† Chairman from VSL (Post-Tensioning Supplier).

* Chairman from Schnabel Foundations (Specialty Contractor).

ASPECT	1974	1980	1986	1996	2004	2014
Materials	1	2	2	8	10	10
Site Investigation	0	1	1	1	2	2
Design	2	6 ½	6 ½	12— Appendix on grout strand bond.	13	13
Corrosion Protection	1	4	5	10	14	15
Construction	7	9	9	10	15	17
Stressing and Testing	1	6	8	17	18	19
Bibliography/References	0	1	1	1 ½	4	3
Applications	16	18	0	0	0	0
Recordkeeping	0	1	1	1 ½	1 ½	2
Specifications, Responsibilities and Submittals	0	1	1 ½	2	2	2
Epoxy-Coated Strand	0	0	Very minor reference.	Frequent reference but no separate section.	10 (Separate sections.)	10
TOTAL PAGES	32	57	41	70	98	109

Table I: Authorship and contents of the successive PTI "Recommendations" Documents (1974 - 2014).

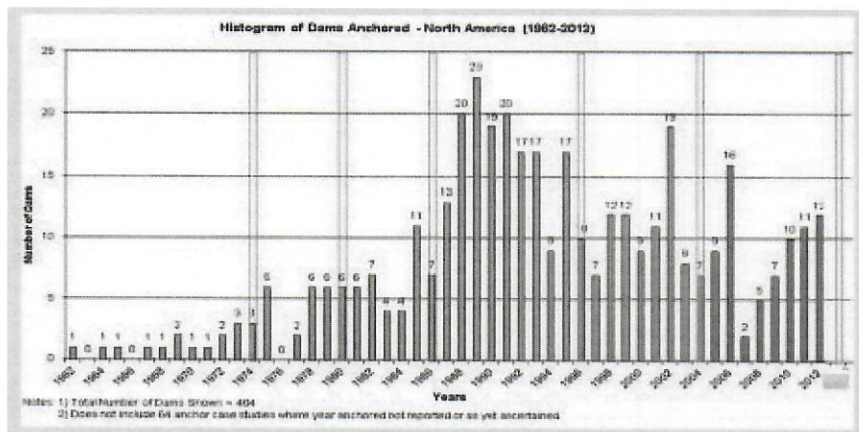


Figure 1: Histogram of dam anchoring projects by year (Bruce et al., 2013), also showing dates of publication of PTI Recommendations.

This work was updated in 2013 (Bruce et al.), and most recently in the CEATI “Best Design Practices” Report of 2020. We now know there have been over 250 technical papers published in the U.S. and Canada on case histories, principally in the annual conferences of groups such as USSD. Further, well over 400 case histories have been identified, some featuring dams anchored two or more times. Figure 1 shows the number of projects conducted each year, bearing in mind that the average since 2014 is of the order of 10 projects per year and shows no sign of declining. Most of the projects have been conducted in direct response to evaluations conducted in accordance with Federal and State regulatory bodies.

DESIGN PHILOSOPHIES

The PTI Recommendations specifically do not cover system design i.e., methods of calculating anchor system load requirements. Instead, PTI covers anchor component design i.e., bond and free lengths (the latter being related to overall stability), tendon components, corrosion protection details, and so on. Overall system design is typically the most challenging design component, now facilitated by advanced software and hardware (and a greater understanding of project-specific strategic and/or seismic demands). In interesting contrast, most component design assumptions have changed little over the years, with the notable exception of corrosion protection requirements. Even these are now closely detailed in PTI and have become standard practice for designers, contractors, and manufacturers alike.

The absence of substantial design change reflects the simplistic—but conservative—assumptions made in the two key areas of anchor design, namely:

- i. overall capacity e.g., conventionally assumed to be the weight of rock in a

CLASS	PROTECTION REQUIREMENTS		
	ANCHORAGE	UNBONDED LENGTH	TENDON BOND LENGTH
I ENCAPSULATED TENDON	1. TRUMPET 2. COVER IF EXPOSED	1. GREASE-FILLED SHEATH, OR 2. GROUT-FILLED SHEATH, OR 3. EPOXY FOR FULLY BONDED ANCHORS	1. GROUT-FILLED ENCAPSULATION, OR 2. EPOXY
II GROUT PROTECTED TENDON	1. TRUMPET 2. COVER IF EXPOSED	1. GREASE-FILLED SHEATH, OR 2. HEAT SHRINK SLEEVE	GROUT

Table 2: Contemporary Corrosion Protection Requirements.

90-degree core, the apex of which is somewhere along the bond zone (top to midpoint).

- ii. load distribution in the bond zone, which is assumed to be uniform.

Neither assumption is correct, but neither is unsafe. The overall stability design complex analyses taking into account the structure of the rock mass as first described by Hobst and Zajic (1977) and largely ignored thereafter. The bond zone assumption ignores the fact that in all, but the weakest rock masses, load is concentrated at the top (proximal) end of the bond zone and it is not equally distributed there below. Barley (1997) discussed this in detail, summarizing that bond lengths over 35 feet long—in most rock conditions—are mechanically inefficient and are artifacts of the “uniform average bond” design assumption.

Although awareness of these two issues is growing amongst enlightened designers, even they usually find that the cost savings potentially arising from more refined assumptions do not outweigh the perceived professional risk and liability of reducing the free or bond lengths.

In contrast, the design approach to corrosion and corrosion protection has changed fundamentally over the years. In

early projects, the bare steel of the tendon was encased in grout and therefore required a two-stage grouting operation. Due to the strain incapability between steel and grout, microfracturing of the grout especially near the proximal end of the bond zone will occur, and so the steel can be exposed to corrosive agents. The commonly adopted practice of pregrouting and redrilling anchor holes to reduce the permeability of the surrounding rock mass, most likely was a Godsend to the protection of many tendons of that time—most of which are still (thought to be) functional today. Contemporary practice for dam anchors

now requires “Class I Corrosion Protection.” The details of which are shown in Table 2. It is noteworthy that rigorous water testing, pregrouting, and redrilling of all dam anchor boreholes is still standard practice. In recent years, water testing of the corrugated duct surrounding the tendon has also become common, before and after the tendon assembly has been placed. The details of anchor head protection—especially for anchors designated to be retestable and/or regROUTable in the long term—are complex, as they should be. However, the details can be correctly designed, properly constructed, and consistently monitored; so providing assurance of reliable anchor service lives in excess of 50 years.

CONSTRUCTION TECHNIQUES

In the early days of anchoring, projects were typically awarded to the post-tensioning companies, who, in turn, subcontracted the “dirty bits” i.e., the drilling, water testing, and grouting, to geotechnical contractors, usually in the site investigation or water well drilling industries. Thus, drilling techniques were typically rotary based, which reflected also a “knee jerk” reaction to the perceived dangers of using percussive techniques

in existing concrete dams. Now rotary percussion is the method of choice, using either air-powered, or water-powered Down the Hole Hammers (DTH) in both concrete and rock. Only for drilling substantial distances in reinforced concrete, or in the hearts of older dams comprising Cyclopean concrete, is rotary drilling still used. DTH has been found to safely provide higher productivities and straighter holes for the diameters common in dam anchoring (i.e., 6-15 inches). No dam safety issues have been ascribed to the use of DTH drilling, unless it must be noted, "artesian conditions" recorded (what a surprise!) by certain contractors when drilling from a dam face where the collar elevation is below that of the impounded reservoir. This is not damage: this is one example—deliberate or otherwise—of a fundamental ignorance of hydraulics.

Regarding hole deviation, although contemporary measurement systems are capable of sensing differences equivalent to 0.25% of depth, they do come with a rapidly escalating cost. In the 1960's, a drill hole tolerance of 1 in 100 was often specified, and apparently, routinely satisfied. Now, even with our new precision toys, such a fine tolerance is rarely actually required, and unless it is a practical necessity (i.e., to avoid internal structures such as galleries, or to assure precise application of load) then it need not be specified. An exception to this refinery would be the use of an Optical Televiwer in at least the first 10% of the holes. The data provided by such instruments allow the dam/rock contact to be observed, together with the structure of the rock mass. Such data inform the subsequent water pressure testing and pregrouting records, and so provide a more confident prediction of the long-term reliability of the anchoring processes. Current practices in the conduct and interpretation of water pressure testing are superior to those of the 1960's and acknowledge the control that fissure width and frequency exert over the potential for tendon grout loss, if not pretreated.

Given the trend for long, high capacity tendons (of over 50 strands, and over 150 feet in length) contractors have developed different methods of placing their tendons in a way that minimizes the potential for damage to the tendon and its corrosion protection. In fact, the means of providing access to the drilling, installation and stressing operations are now the major control over a contractor's price on most projects. Long reach cranes are still used, "uncoilers" likewise, except that there

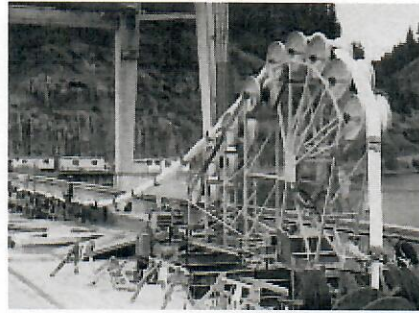


Photo 1: Device used to place long, high capacity tendons.

is awareness that corrosion protection can be damaged in the process, which started the trend from the mid 1990's of placing the corrugated external duct first and separately, followed by the tendon itself. Helicopter-assisted placement (e.g., at Tolt River Dam, WA in 1996) remains a rare sighting, while project-specific "dinosaurs" have been more frequently discovered (Photo 1).

No 1/4 inch multi-wire tendons with button-head stressing arrangements have been used since the early 1980's; but multistrand tendons, comprising 0.6 inch diameter strands, are standard, except where lengths are short (say less than 40 feet) and loads are low (say less than 200 kips) and bars are the logical choice. Regardless of tendon type, working stresses are now limited to 60% guaranteed ultimate tensile strength (GUTS), and test stresses to 80% GUTS. These limits were neither imposed nor universally observed 40 years ago, complicating load-extension data analysis, and long-term performance prediction.

The requirements for tendon grouts have changed little over the years: they must still be stable, pumpable, and capable of reaching target unconfined compressive strengths. The early reliance on special preblended materials has dissipated, while the use of high-speed, high-shear mixers has endured. Due to changes made by manufacturers in the cement chemistry and fineness, target strengths at stressing (at least 3000 psi) are being achieved within several days, even with Type I/II Cement. This gives an additional degree of flexibility to project sequencing. Increasingly over the last 30 years, routine grout quality control has focused on the measurement of grout fluid properties (Specific Gravity, Apparent Viscosity, and Bleed Stability) prior to grout placement. Hence, while cube testing results provide a retrospective indicator of grout quality (assuming always that the curing conditions of the cubes mirror those in the ground), the compliance, consistency, and quality of the grout can be established prior

to placement; so avoiding the possibility of later rejections based on defective cube preparation, curing or testing.

Stressing sequences used in the 1970's were typically very simplistic, thereby reducing the value of the stressing process in judging anchor short-term behavior or predicting long-term performance. For example, it was typical to find progressive loading to 115% Design Working Load, minimal (if any) creep observations, and no return to Alignment Load prior to Lock off. From 1996 onwards, most dam anchors have been stressed cyclically to Test Load (usually 80% GUTS), are subjected to a creep test, are destressed back to Alignment Load (to establish permanent movement), and then Locked-off at < 70% GUTS. Guidance is provided in PTI in the case of one or more acceptance criteria not being met. Fortunately, such failures are extremely rare for anchors conservatively designed and properly constructed in rock. (This is not always the case for anchors founded in soil, tying back relatively flexible retaining wall systems.)

OUR CHALLENGE WITH "LEGACY ANCHORS"

As alluded to above, arguably over 60% of all the tendons installed—i.e., those 25 years or more in age—feature corrosion protection details which would not satisfy contemporary standards. In addition, many of the tendons cannot be Lift-off tested since they have no residual free length, and/or the anchor heads cannot be accessed with a jack. No instrumentation survives to display load, or to measure the integrity of the tendon. As a consequence, the residual load in such anchors is unknown and cannot be measured (Bruce and Wolfhope, 2018). These aging, uncertain anchors are referred to as "Legacy Anchors" and present a risk management conundrum to their Owners and Regulators. Some parties take encouragement from the (limited) number of old tendon observations (including exhumations) and so assume that the anchors are still reliable. Others take the position that exactly because residual load and tendon condition cannot be quantified, neither can the risk of failure, and so they chose to re-anchor the dam using tendons meeting contemporary standards.

This is the most difficult problem facing the dam anchor industry and there is no easy or definitive solution. As a guide, however, an Anchor Portfolio Risk Assessment Process has been proposed (Bruce and Wolfhope, 2020) which provides a qualitative approach to assessing the level of risk posed by Legacy

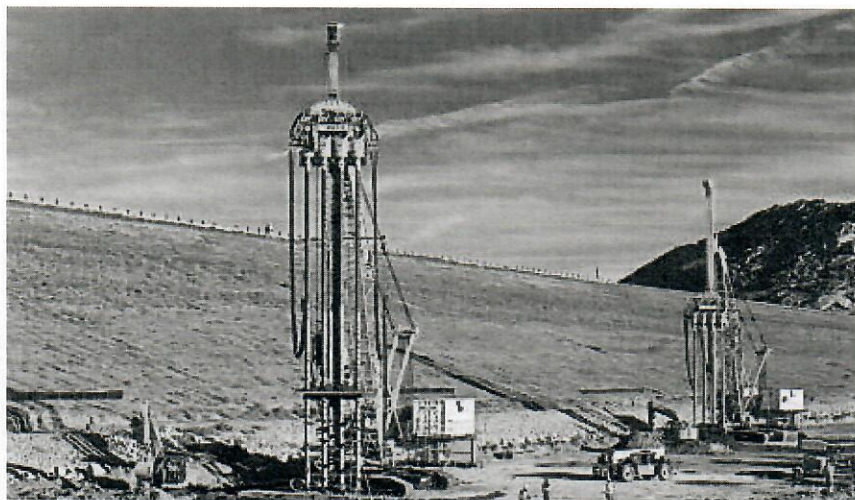


Photo 2: Typical Multi Axis "Conventional" Deep Mixing Rig.

Anchors on a project-specific basis, and therefore helping to prioritize remedial actions if found necessary.

An interesting corollary to this problem is what to do with the anchors now being installed. Do we simply optimize all design construction and short-term acceptance processes and then seal in the anchor head, leaving it undisturbed thereafter? The proponents argue that this would keep the corrosion protection intact and undamaged. The opponents are concerned that we would simply create a new generation of

Legacy Anchors for future engineers to deal with. There is neither national nor international consensus, although it does seem that the philosophy of leaving at least a reasonable percentage of new anchors as retestable and restressable is gaining ground, especially if load cells of guaranteed longevity can be finally delivered. It will be fascinating to see how the U.S. practice evolves in this vital subject in the years to come. Fundamental to this evolution will be seizing full scale research opportunities (e.g., via tendon extractions

and dam decommissioning), and then finding knowledge-sharing platforms at both national and international levels.

THE DEEP MIXING METHODS

Historical Perspective

The Deep Mixing Method (DMM) was introduced into the U.S. in 1986 by SMW Seiko, Inc. operating under license from its Japanese parent (FHWA, 2000, a,b, and 2001). As described below, the first major DMM application in the U.S. was for the foundation remediation of Jackson Lake Dam, WY from 1987 to 1989. This was a particularly important and timely project since it brought DMM to prominence not only as a viable ground treatment methodology, but as one perfectly suited to the demands of dam and levee remediation. Over the past 30 years there has been a steady stream of DMM projects for seepage control, and reinforcement of foundations, especially in California and Louisiana. Many of these projects have been of enormous scale, and these projects have been critical to resolving dam and levee safety problems.

Research and development on deep mixing started with laboratory model tests in 1967 by the Port and Airport Research Institute of the

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former Japanese Ministry of Transportation for the purpose of stabilizing soft marine soils with lime in harbors or below the sea bed before the construction of harbor facilities. In 1974, the DLM method (deep lime mixing method) became feasible for full-scale construction. Based on technology developed for DLM, the CDM method (Cement Deep Mixing) was initiated using cement grout as the binder. Starting from laboratory model tests and progressing through on-land full-scale testing and ocean full-scale testing, the CDM method was developed in 1975 for full-scale application. Parallel to the development of the CDM method for large scale ground treatment, development, and research on the SMW (soil mix wall) method was started in 1972 by Seiko Kogyo Co, Ltd, of Osaka, Japan. In 1976, the SMW method was developed for full-scale application. Other Japanese and European contractors soon followed SMW Seiko's entry into the U.S., and this stimulated U.S. companies to develop their own DMM variants, mainly for earth retention and environmental containment. Indeed, Geo-Con, Inc., of Monroeville, PA, actively participated at Jackson Lake Dam, WY in the late Eighties, and by 1993 they had also built a cutoff wall at Lockington Dam, OH (Walker, 1994).

Basically, all these earlier projects featured "conventional" DM equipment and methods, namely a large crawler mounted base machine carrying one to eight (typically three or four) vertical, hollow shafts with discontinuous flight augers and paddles (Photo 2). A cementitious grout ("binder"), with or without bentonite, is pumped through the ports in the tips of the shafts as they are rotated into the ground and as they are withdrawn. Machines with one shaft create columns, usually three to five feet in

diameter, while multiple shaft machines create panels, usually three feet wide and up to ten feet long. The product of the operation is an in-situ blend of the native soil and the binder ("soilcrete"). The properties of the blend will reflect the nature of the original soil, and the DM parameters selected, such as the volume of slurry and weight of binder injected, and the mixing energy. Design unconfined compressive strengths of the mixed soil of 75–500 psi are common, typically around 200 psi, and the relationships between UCS and shear and tensile strengths, modulus of elasticity, and Poisson's Ratio have been progressively refined over the years, based on intensive lab and field testing. In-situ permeabilities in properly mixed soil are typically 10^{-6} cm/s and less, although lab testing of retrieved samples gives values consistently one or two orders lower.

referred to as TRD. This is basically a very large and powerful chain saw which is pulled vertically through the ground by a base crawler, cutting and mixing the ground with grout injected through ports in the cutting post i.e., the saw itself (Figure 2). The system can cut to depths of 170 feet with a width of 18–34 inches in appropriate conditions, creating a continuous and very homogeneous wall without joints. The cutting chain (and the cutting parameters) can be adjusted to optimize productivity, depending on the ground conditions. TRD has been used in the U.S. since 2005 under license by the Keller Group. A similar system using an inclined cutter and extremely powerful base machines ("One Pass System") has been developed more recently by the DeWind Corporation in Michigan, and again in appropriate conditions,

DAM APPLICATION	DMM PRINCIPLE		
	CONVENTIONAL VERTICAL AXIS	CONTINUOUS TRENCHING	HORIZONTAL AXIS
Seepage cutoff	Original method used, remains viable	Very attractive on big projects	Very popular on projects of all sizes
Seismic remediation	Very common	Not applicable	Common
Foundation strengthening	Very common	Not applicable	Common
Cutoff and buttress	Can be used	Can be used	Can be used

Table 3: Dam applications of DMMs.

Two major competitors to conventional DMM were introduced into the U.S. in the early 2000's, and these have had major impacts on the dam and levee remediation industry. By 1993, Japan had conceived the "Trench Remixing and Cutting Deep Wall Method",

can produce a homogenous mixed wall very quickly and competitively (Photo 3).

Table 3 summarizes the links now clear between the general application and the principle of DMM used for dam and levee remediation.

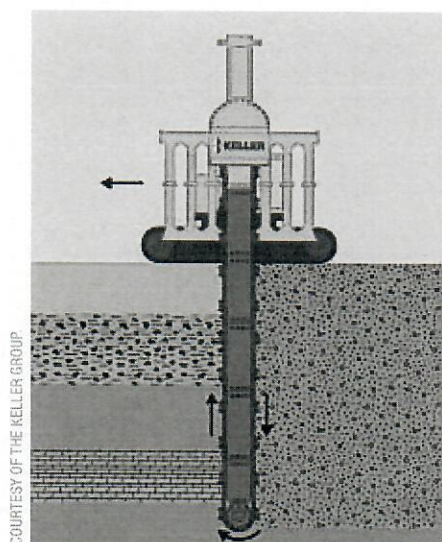
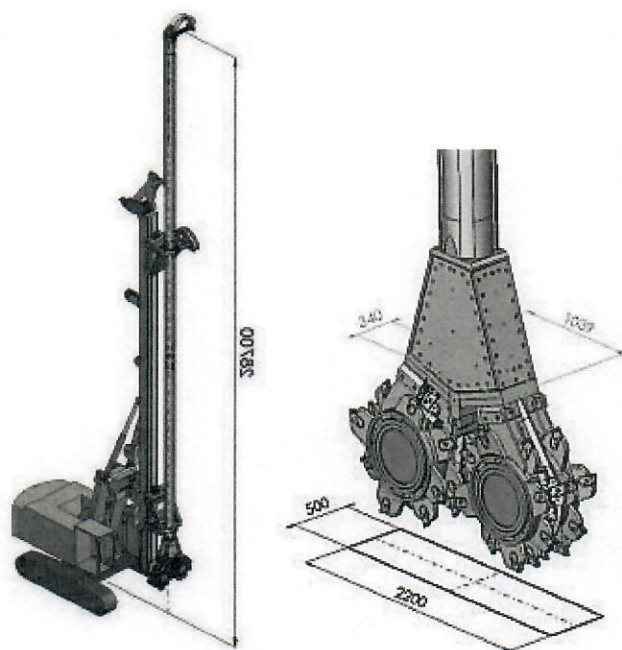


Figure 2: Concept of the TRD Deep Mixing Method.



Photo 3: One version of the "One Pass" System.



COURTESY OF BAUER MASCHINEN.

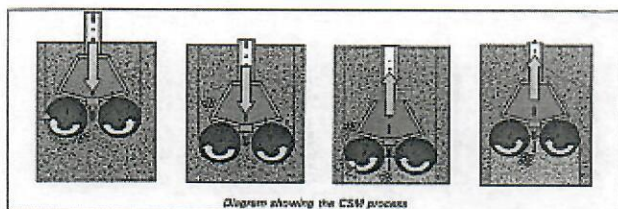


Figure 3: Schematic of CSM System.

The second "newcomer" was the Cutter Soil Mix (CSM) System. Based on experiences gained with trench cutter ("hydromill") technology, and with conventional DMM techniques, Bauer Maschinen of Germany, and the Bachy Soletanche Group from France, jointly developed CSM in 2003. Bauer was granted a patent in 2004, and the first application in North America followed in Vancouver, BC in 2006. As shown in Figure 3, grout is injected from ports located between the cutting wheels during penetration and withdrawal, creating rectangular panels about 8 to 9 feet long and 20 to 48 inches wide. A maximum depth of 200 feet is claimed. The panels are overlapped in a Primary-Secondary sequence to create a continuous cutoff. This has become a particularly popular technique in North America, not least because it can more readily accommodate denser and coarser deposits, and the cutter/mixer can be operated from a non-specialist crawler base. A similar system—CT Jet—has been developed by the Trevi Group in Italy.

Clearly, therefore, there are a number of DMM techniques available for creating soil-cement elements in and under existing dams and levees, and each may hold a particular advantage on any specific site. However, there are practical limits to the application of DMM regardless of the type being considered:

- the ground should be neither very stiff nor very dense
- penetration into materials other than soils (alluvial or residual) and fills should be minimized
- the ground should not contain dense arrays of large boulders
- any indurated layers (e.g., thin caliche or limestone layers) must be amenable to shearing or ripping, otherwise the process will be stalled
- a maximum depth of 120 feet is preferable

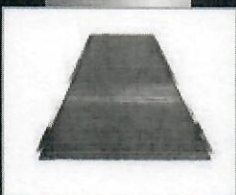
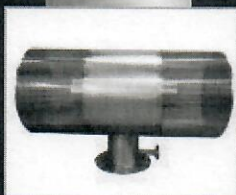
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- there must be adequate overhead access and a wide, competent working platform
- spoil volumes can be tolerated and managed
- treatment volumes should be large (to offset the relatively high mobilization costs)

Further, as noted below, DMM works have been found to favor Performance Specifications provided the acceptance criteria for the treated soil are well defined, and the testing methods clearly specified.

QA/QC AND VERIFICATION

There is one major commonality which now characterizes all the different DMM techniques as used in dam and levee remediation—namely the sophisticated real time monitoring and control of the cutting and mixing processes. Early reliance on manual observations of penetration and withdrawal rates, and on simple instrumentation of grout injection rates and volumes has given way to automated systems which, though proprietary to the respective contractors, all basically provide similar data and control in real time:

- verticality of the mixing tool (shaft, post, or cutter), its depth, and position
- energy draw and penetration rates which reflect changes in the competency of the ground



Photo 4: View of the drill rig operator's screens.

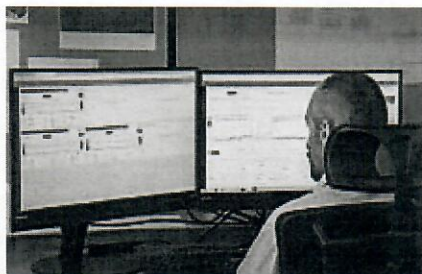


Photo 5: Construction data relayed in real time for monitoring in remote offices.

- grout injection rates and volumes, and grout Specific Gravity. Examples are shown in Photos 4 and 5.

In short, strict quality control over the construction processes is exercised with respect to geometry (position, depth, width, verticality, and continuity), homogeneity, and

target in-situ properties (strength, deformability, Unit Weight and where applicable, permeability). Of course, the in-situ properties must be verified independently from the expectations of the installation processes. Historically in the U.S. samples of the soil-cement were taken from elements prior to setting, either in the form of "bulk" samples (from surface spoils) or from "deep grab" sampling (from discrete depths in the DM elements). An interesting refinement is the method of pushing open-ended concentric tubes into the wet soil-cement. The inner tube is then extracted after initial set of the mixed material, effectively providing an undisturbed full depth sample from which standard cylinders can be cut for material testing in the laboratory. Samples obtained from the wet sampling methods are typically shared between the Contractor (QC) and the Owner (QA). Testing is conducted routinely at 28 days, but valuable information can be gained through earlier (7 and 14 days), and later (56, 112, 180 days) testing. (Bruce et al., 2020)

These "wet" sampling techniques were favored since specialty contractors were initially distrustful of their ability to core mixed soil, and suspicious of the validity of subsequent strength testing given the possibility of coring-induced micro fissuring in the samples. However, by drawing from Japanese experiences and by developing specialty coring techniques and equipment, DMM contractors have accepted coring as a verification methodology for DM materials of confined strengths of 60 psi and above. This progress is illustrated in the case histories of Herbert Hoover Dike, FL and LPV III, LA summarized below.

An added invaluable benefit gained from verification coring is that the holes can then be accessed by Optical Televiewing cameras (to show precisely what the in-situ conditions are) and can be subjected to permeability testing (where this property is an acceptance criterion).

Finally, while the QA/QC and verification testing data do of course talk to compliance and acceptance, they also provide feedback for optimizing the construction processes. The statistical analytical methods developed by Filz and Navin (2010), for example, have allowed the safe reduction in cement factors while still meeting project strength requirements. This can constitute an important cost reduction in major projects and has become a commonly accepted approach in the industry. In this regard, it may be borne in mind that the classic sequence of bench

scale testing—pre-production full scale testing—and more intense verification of early production elements, tends to produce conservative selections of DM parameters; especially relating to cement factors, as a natural response to managing the risk of early noncompliance.

LANDMARK PROJECTS AND EVENTS

It is normal that new specialty geotechnical construction techniques are introduced by contractors, and as noted above, that is exactly what happened with DMM in the U.S. However, what is somewhat unusual—certainly in the U.S.—has been early and active involvements by owners (Federal and State) and consultants. For example, throughout the 1990's, there were frequent technical exchanges between engineers in the U.S. and their counterparts in Japan and Scandinavia in particular. These were conducted to inform the design of particular projects (e.g., the Fort Point Channel section of the Boston Central Artery) or to gain broader appreciation for the potential of DMM in a wider range of transport related projects (e.g., the FHWA Reports of 2000 and 2001).

A full scale DMM demonstration program was sponsored by the USACE in 2000–2003 in New Orleans, very prescient given the events there of Fall 2005, and a States Pooled Fund Research project was in 2000–2005 led by CALTRANS. Major international conferences have strongly featured DMM (e.g., New Orleans 2012, San Francisco 2015, and Honolulu 2017), and Government Agencies and Professional Societies such as Deep Foundations Institute (DFI) sponsored workshops, seminars, and manuals to promote the technique and to provide consistent guidance on design and acceptance issues. Analytical leadership was provided by practical academia at Virginia Tech and at Bucknell University in particular.

This has all created a very positive and supportive base for the promotion and expansion of DMM in the U.S. However, the progress of any technique can best be regarded by considering a number of landmark projects; illustrating significant advances in design, technique, and quality as summarized in the following brief project descriptions. This selection is not intended in any way to detract from the efforts of contractors routinely building levee cutoffs around the American and Sacramento Rivers in California, and in Idaho since the early 1990's, or installing massive seismic remediations in port facilities around San Francisco Bay, or providing foundation

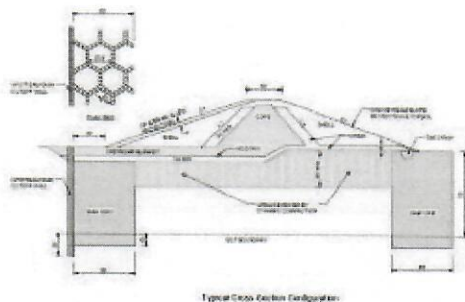


Figure 4: Foundation stabilization of Jackson Lake Dam, WY. (Sources: 1. Foundation Stabilization of Jackson Lake Dam, A. Pujol-Rius, P. Griggin, J. Neal and O. Taki, 12th International Conference on Soil Mechanics and Foundation Engineering, Rio de Janeiro, Brazil, August 1989; 2. Foundation Treatment Methods for Jackson Lake Dam Modification, J. Farrar, K. Wirkus & J. McClain, USCOLD).

reinforcement under levees in New Orleans. Every project has taught us something, but some projects are particularly insightful. Further details are to be found in Bruce (2020).

SEISMIC REMEDIATION OF THE FOUNDATION OF JACKSON LAKE DAM, WY (1987-1989)

This Bureau of Reclamation project, built from 1911–1917, included a 4,300-foot long, 49-foot high northern embankment comprising hydraulic fill. It was founded on fluvio-lacustrine and lacustrine sediments, and located in a highly seismic zone (i.e., the Grand Tetons). Studies in the early 1980's confirmed both dam and foundation were susceptible to liquefaction, and a major modification was designed to replace the dam, and to improve the foundation to depths of 110 feet. As shown in Figure 4, ground treatment was considered for creating an upstream seepage cutoff, and to create hexagonal "cells" to reinforce the foundation under the footprint of the replacement dam. The soilcrete columns were to be in full depth contact and have a minimum shear strength of 200 psi. The project specifications for column construction were largely Performance-based, and a "Best-Value" bidding competition was held. The successful bidder had offered a DMM solution—much to the surprise of his many competitors based on more traditional methods!—and the first major DMM project in North America had arrived. The work was conducted successfully to the best contemporary standards of quality control and assurance. About 70 core holes were also drilled in the soilcrete to allow comparison with data from over 2000 wet grab samples. Most significantly, details of the project were widely published in the technical press by the

participants, the most impactful being the paper by Ryan and Jasperse (1989) who concluded:

"At the Jackson Lake project, deep soil mixing provided an economical, reliable way of satisfying a difficult set of technical parameters and meeting a tight project work schedule.

Obviously, much is yet to be learned about DSM strength parameters at other sites with differing soil conditions. It is apparent, however, that high quality, high strength columns can be constructed using DSM. The different applications that could be used are numerous and the technique appears to have a solid future in the geotechnical construction industry."

PROJECT SIGNIFICANCE: JACKSON LAKE DAM

This was the first large scale DMM project in North America, well conducted and widely publicized. Whereas the DMM construction processes and the testing protocols reflect well on contemporary practices, there is not engineering consensus about the design concept (closed cells), as illustrated in subsequent case histories for similar applications. One very important finding from the intensive testing was that soil-cement strengths could be expected to increase by 100% from 7 to 28 days, and from 28 to 56 days, and by 40–50% between 56 and 112 days. There was still a trend for strength increase beyond 112 days.

SEISMIC REMEDIATION OF CLEMSON UPPER AND LOWER DIVERSION DAMS, SC

Input from Dr. D.S. Yang of JAFEC Inc. is gratefully acknowledged

These dams were constructed in 1960–1961, comprising random fills up to 80-feet high, and lengths of 3,000 and 2,100 feet, respectively. The foundation soils contained loose alluvial silty sands/sandy silts of thickness 7 to 28 feet and N-values of 3 to 30. These overlaid a thin gravel layer over weathered bedrock. Testing and analyses revealed that liquefaction slope failure would occur at the downstream section, given the levels of strain induced in the alluvium by a maximum bedrock acceleration of 0.08–0.1 g.

An intensive remediation alternatives analysis process found that an in-situ remediation using DM shear walls beneath the downstream berm of each dam was optimal (Figures 5 and 6). These walls would be toed into the gravel or into weathered bedrock. To alleviate ground water buildup against the longitudinal wall, a filtered seepage collection system was placed behind it.

Seismic analyses confirmed that the DMM reduced strains in the alluvium to levels below the triggering strain such that significant strength loss in the alluvium would not occur. Bench scale tests in five major strata indicated that the DMM treatment could produce soil-cement with an unconfined compressive

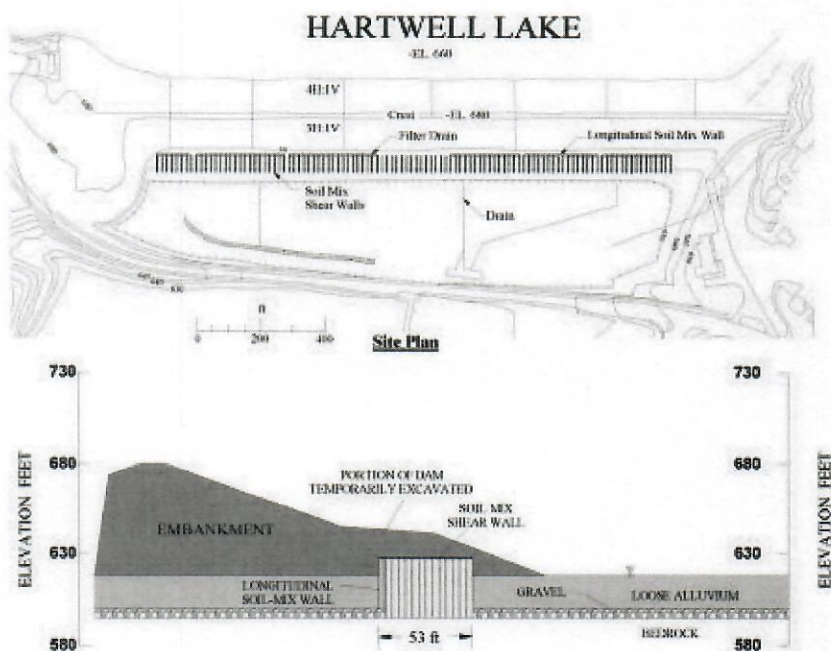


Figure 5: Plan and section of DMM layout at Clemson Dam, South Carolina (After Wooten, R. L. and B. Foreman, Proceedings of the 25th Annual United States Society of Dams conference 2005).

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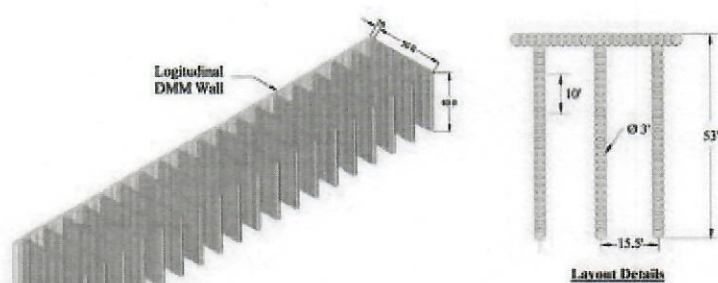


Figure 6: DMM treatment layout at Clemson Dam, South Carolina.

CREDIT: DMM WALLS INSTALLED BY RAITO, INC. IN 2004 AND 2005.

strength of 400 psi, which was required to maintain the seismic stability with factors of safety between 2 and 3. This target strength (f'_{sc}) was developed using 3-foot wide DMM shear walls at a 15.5-foot spacing. To allow for flexibility in using different DMM equipment types, the following equation was developed to permit adjustment of the wall width (W_a) and wall spacing (S):

$$f'_{sc} > 77.4 \text{ psi} \times (S/W_a)$$

In addition, the wall spacing was restricted to no more than 12.5 feet plus the wall width. This formula also provided a mechanism for revising wall spacing if the strength of the soil-cement was lower than the target value due to difficulty during construction in achieving the target strength values. The value, 77.4 psi, is a site-specific value that can be developed during the design of a DMM remediation scheme. For this project, it can be considered as the design average unconfined compressive strength of DMM with 100% treatment ratio, or a W_a/S of one. A six-axis DMM rig was used by Raito Inc to construct the 50-foot long x 2.76-foot wide shear walls, following the successful installation, and testing of two trial sections at each dam. The total DMM volume was around 45,500 cubic yards. The relatively high UCS target (average 435 psi) resulted in the use of a relatively low "water to cement" ratio grout. The soilcrete was therefore very viscous and so inhibited successful wet grab sampling. This necessitated the systematic use of coring to provide in-situ samples at an average rate of one core per two shear walls.

PROJECT SIGNIFICANCE: CLEMSON UPPER AND LOWER DIVERSION DAMS

This project was conducted from February 2004 to February 2005, and set a new standard for design, construction, and verification. It was followed by similar seismic remediation projects such as Sunset North Basin Dam, CA; San Pablo Dam, CA; Perris Dam, CA; and Kingston Ash Pond, TN. Methods of seismic stability analysis continue to evolve, and so design

details for the layout of the DMM buttresses have naturally changed with time. However, the attraction of DMM techniques of all types has become a reliable staple of seismic remediation in cases where "excavate and replace" is not feasible for cost or dam safety reasons.

MIXING IN NEW ORLEANS

The soils in the Mississippi Delta region are in many ways similar to the extremely soft natural deposits of the Nordic countries, and the hydraulic fills in Japan—the birthplaces of DMM. Traditionally, deep foundation construction in the Delta had been served by "tried and proven" techniques. These had been well suited to the challenges of the local conditions, and the related design and construction details had been consistently implemented by the local engineering community.

In 2001, the New Orleans District of the U.S. Army Corps of Engineers (USACE) raised significant funds to carry out a full scale field demonstration of DMM: the potential for its use had been recognized by geotechnical engineers led by Dr. Peter Cali who were engaged in levee evaluation and maintenance. This test proved the practicality of DMM in the soft organic soils, established QC/QA procedures and informed subsequent design procedures and cost estimating (Cali et al., 2005). However, there were no immediate projects which could benefit from this new technique—in either the Federal or private sectors of local construction. This situation changed sadly and dramatically in the form of the devastation wreaked by Hurricanes Katrina and Rita in August 2005. Emergency repairs were implemented under the aegis of "Task Force Guardian," pending a fundamental assessment of the whole flood protection system. During the period 2006–2009, there was a plethora of DMM projects of small–medium scale, each categorized by disciplined testing sequences. A most valuable data base of experience was therefore accumulated by USACE and its consultants as a foundation for the design of the massive works to be associated



Photo 6: View of LPVIII Deep Mixing.

with the definitive Hurricane and Storm Damage Risk Reduction System.

The largest DMM project in the system was for foundation stabilization for over six miles of levee being raised along the north bank of the Gulf Intracoastal Waterway and bordered on both sides by the Bayou Sauvage National Wildlife Refuge (Photo 6). The project was conducted from January 2010 to March 2011 and is referred to as the LPVIII contract. It involved over 1.68 million cubic yards of DM to provide in-situ buttresses to improve the otherwise inadequate bearing capacity of the soft foundation soils. Eight mixing rigs and batch plants operated around the clock to inject over 457,000 tons of cement. 506 of the 18,028, five-foot diameter elements were cored to verify the 100 psi UCS target had been met (the actual average was 292 psi). The buttresses were constructed orthogonal to the levee axis and extended into competent Pleistocene clays at depth. It is particularly noteworthy that the successful DMM contractor—Trevi ICOS—had been procured under the Early Contractor Involvement (ECI) process wherein it had significant input into the design and planning of the project. Another notable point was the contractor's use of their Turbo Mix method, single and double axis, a novel system using elevated grout injection pressures. A minor part of the work was conducted under subcontract by another contractor (FUDO) using a different proprietary method (CI-CMC), but yielding the same excellent and consistent performance (Cali et al., 2012).

PROJECT SIGNIFICANCE: LPVIII

The experience in New Orleans is a classic example of the development of a market and its response to an enormous natural disaster. The LPVIII project remains by far the largest conducted in North America and was done so with a consistency and level of quality

COURTESY OF TREVI ICOS.



Photo 7: Typical core extracted from DM cutoff at Herbert Hoover Dike, FL.

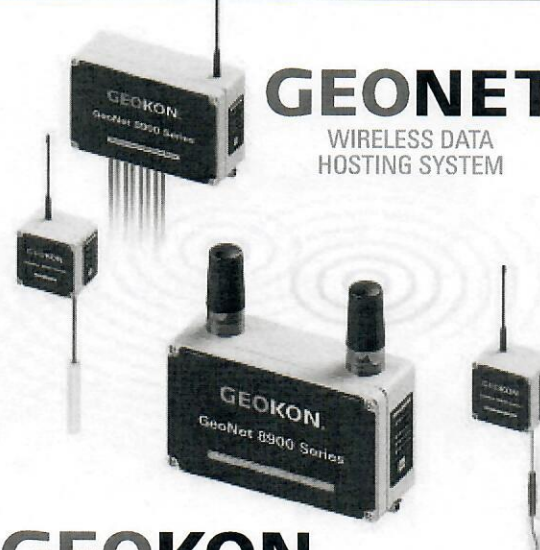
extraordinary given the technical, scheduling, logistical, and environmental challenges confronting it. The Government's use of the ECI process was instrumental in the success of this project. Another major breakthrough was the use of about 500,000 cubic yards of DM spoils ("Reworked Embankment Material") as fill material during the reconstruction of the levee.

A whole session of the 2012 Grouting and Deep Mixing Conference was devoted to this project.

CUTOFF WALL CONSTRUCTION AT HERBERT HOOVER DIKE, FL

Lake Okeechobee in southcentral Florida is a roughly circular body of water surrounded by a very heterogeneous levee known as the

Herbert Hoover Dike. Early construction by local farmers first began around 1915 utilizing mostly sand and topsoil (Garbin et al., 2009). Portions of the original embankments were overtopped by hurricane induced surges in 1926 and 1928, resulting in the loss of over 2,500 lives. As a result, 84 miles of levee were reconstructed by the USACE between 1932 and 1938. A major hurricane in 1947 highlighted the need for additional flood protection, and the current dike system for Lake Okeechobee was completed in the late 1960's. The system now consists of 140 miles of levee with 19 culverts, hurricane gates and other water control structures. Lake Okeechobee has become the third largest freshwater lake in the continental U.S., draining to the ocean through the Everglades. The levee crest is typically at around elevation +36 feet, and the lake is normally at elevation +10 to 13 feet. However, sections of the levee are at risk due to seepage and piping, in particular when lake levels increase beyond certain elevations due to surges caused by severe weather. Therefore, the USACE designed a cutoff wall constructed progressively in prioritized "reaches", each several miles long, to mitigate shallower piping concerns and ensure dike stability during extreme weather events.



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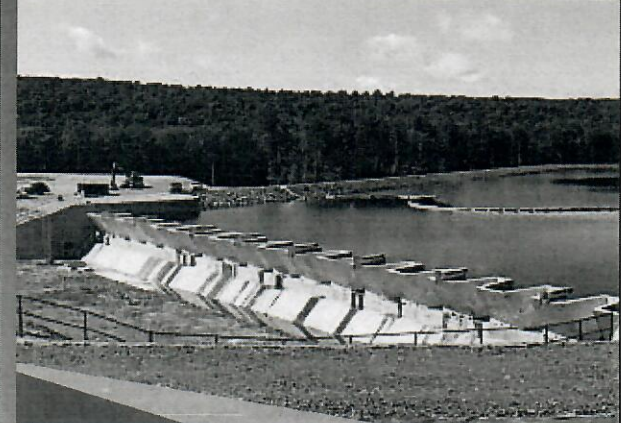


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The foundation conditions are highly variable, and comprise, from surface down, peat, sands, and limestone layers (5–50 feet bgs) and then more sands. The cutoff was to extend five feet below the upper limestone layers given that the depth to the main limestone bed is over 150 feet bgs: it is therefore a “floating wall”.

Given the scale of the project (Phase 1 was over 21 miles long, and Phase 2, currently underway, is over 30 miles long) and the urgency of the remediation, the USACE managed four major risk elements as follows:

1. **Technology Risk:** Prequalification of three different cutoff wall methods (out of eight proposed, each with very detailed statements of prior experience).
2. **Schedule Risk:** Breaking down the whole project into numerous smaller reaches so permitting simultaneous work in several sections, each competitively bid by the three prequalified contractors.
3. **Project-Specific Risk:** Each reach was predicated by a 500-foot long Demonstration Section. Production was only permitted after USACE had accepted the Demonstration Section Report. USACE also employed an external consulting group as continuity/oversight over all individual contracts.

4. **Performance Risk:** Use of Performance Based Specifications, but featuring very clear QA/QC and Verification criteria defining:

- Continuity, and verticality (by analyses of on-board instrumentation).
- 18 to 36-inch wall thickness;
- In-situ permeability $< 1 \times 10^{-6}$ cm/s (in cored verification holes at 200 feet centers);
- UCS in the range of 100 to 500 psi from cores, at 28 days;
- Chemical compatibility with groundwater and soil.
- Homogeneity.
- Core recovery > 95%.

These measures were supported by groups of external DMM specialists, acting as oversight and perspective. Approximately 70% of Phase 1, which began in 2008, was conducted using TRD (Hayward Baker) and CSM (Bauer) methods, the balance being a Self-Hardening Slurry cutoff constructed by traditional cutoff wall methods i.e., grab and hydromill, by Trevi ICOS. The QA/QC program was very intense, with core data being supplemented by testing of “bulk” samples (i.e., taken from the spoils generated at the surface) and deeper “wet grab” samples taken from different elevations of the cutoff prior to initial set. Both DMM contractors elected to use

grouts with a high slag content, in addition to bentonite and other admixtures to extend workability and reduce permeability as required. The contractors used different mixes, but it was common to see at least a 3:1 slag to cement ratio, and at least 2% bentonite. Samples continued to gain strength even beyond 200 days age. It was normal to take special measures, such as chilling the mixing water, to combat very high ambient temperatures over the summer months, so as to ensure the binder was below 94° F upon injection.

Bearing in mind the great heterogeneity of the soils to be mixed, the soil-cement of the cutoff was found by coring and the use of Optical Televiewers to be extremely homogeneous in appearance and in its engineering properties (Photo 7). In the case of the CSM sections of the cutoff, achieving homogeneity was aided by the contractor's decision to predrill the alignment and remove substantial thicknesses of peat and replace them with controlled granular backfill.

PROJECT SIGNIFICANCE: HERBERT HOOVER DIKE

This massive project showcased the newer DMM techniques of TRD and CSM and elevated

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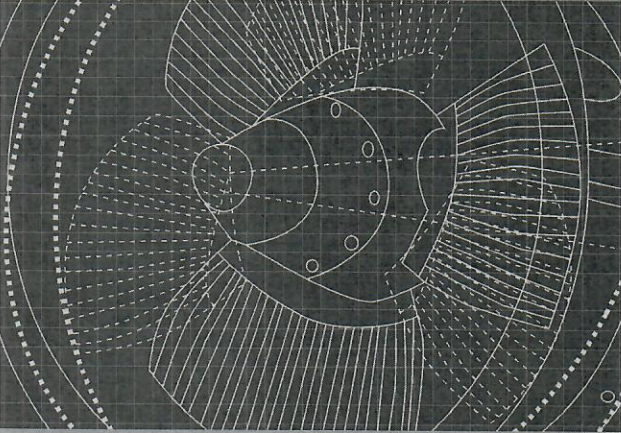
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them to a level of nationwide acceptance in less than two years. Given how variable the fills and soils were, it is remarkable how consistent and homogeneous the DM product was proved to be. The success of this project reflects well on the resilience of the technical specifications, particularly regarding verification and acceptance methodologies and criteria. The prequalification process ensured that the bids for each section were realistic and responsive, as well as highly competitive. The work is ongoing.

REPLACEMENT DAM CONSTRUCTION, BUCKEYE LAKE, OH

Input from D.P. Stare of Gannett Fleming Inc. is gratefully acknowledged

The Ohio and Erie Canal systems began construction in 1825 to permit barge traffic between the Ohio River and Lake Erie, and to promote economic expansion in the region. Buckeye Lake was already a natural reservoir on a topographic high along the route but was expanded in the early 1830's with the construction of encircling clay and shale embankments, faced with masonry. When canal traffic rapidly declined from the late 1880's due to competition from railroads, Buckeye Lake was repurposed as a



Photo 8: The shoreline of Buckeye Lake Dam, Ohio before remediation.



Photo 9: Installation of the DM cutoff using the "One Pass" system.

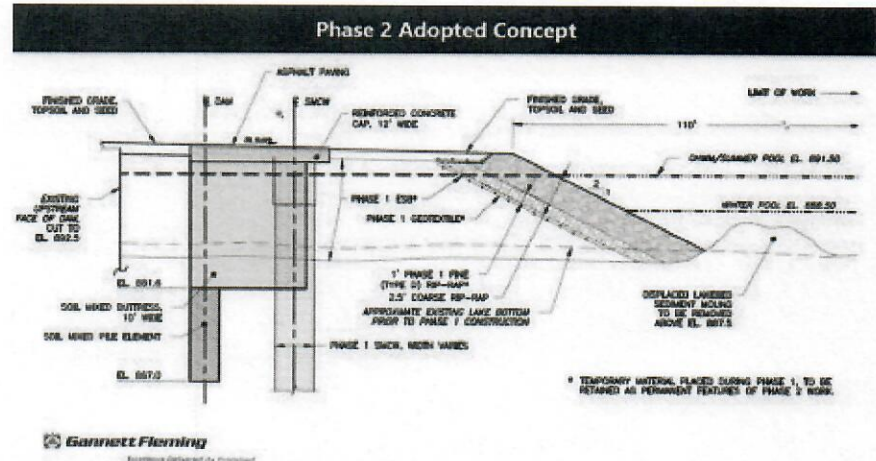


Figure 7: Concept of Phase 2 of Buckeye Lake Dam remediation.

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public park, lots on the dam were sold/leased, and eventually 370 cabins and cottages were built on and into the embankments. Despite sporadic shoreline improvements such as sheet piling installed immediately upstream of the masonry (Photo 8), leakages and deformations continued to the extent that by 2015, the USACE, assisting the Ohio Department of Natural Resources, opined that various interim risk reduction measures be immediately implemented (Stare and Bruce, 2018).

These Phase I measures included the installation of a 40-foot deep DM cutoff wall upstream of the sheeting (Photo 9) and the placement of additional embankment materials. The 90,000 cubic yards of DMM was conducted with both conventional (Raito Inc) and "One Pass" (DeWind) methods.

The definitive Phase 2 works involved construction of basically a new embankment reaching into the lake, but founded in part on a 10-foot wide DM stability buttress, tied into the Phase I cutoff and supported on the upstream side by another row of DM elements (Figure 7). A further 130,000 cubic yards of DM was installed as the buttress. The soils were glacial outwash and lacustrine deposits overlying till. Bedrock (Mississippian) was found over 100 feet below grade.

PROJECT SIGNIFICANCE: BUCKEYE LAKE DAM

This project has received national recognition for innovation and excellence in design, and indeed the concept has been duplicated on other projects in the region. It has secured the safety of a vulnerable, aging embankment against several Potential Failure Modes, and has done so while minimizing disruption to homeowners, their properties, and the environment. Tourist and recreation amenities have been enhanced while the roadway located above the DM buttress allows easy access for inspection and maintenance of the replacement structure.

It is a most timely reminder that DMM is not just for mega projects of immense scale conducted away from the gaze of the general public. 🐼

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