Dam Remediation by Anchors and Cut-Offs: A Summary of Two National Research Programs

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Abstract

During the last three years, the author has acted as Principal Investigator in two National Research Programs dealing with the remediation of existing dams. The first program relates to the use of prestressed rock anchors for concrete structures, including dams, spillways, abutments and other appurtenances. The project has generated three deliverables:

- A comparative analysis of the five successive versions of the "Recommendations" documents (which have acted as defacto standards) published between 1974 and 2004.
- A compendium of all the technical papers written on North American dam anchoring projects. To date, over 235 papers have been collected.
- An interactive database providing details of every case history in North America. So far, over 400 projects have been found.

The second initiative deals with "positive" concrete cut-offs for existing embankment dams. This study has so far revealed over 20 case histories reaching from 1975, and comprising millions of square feet of cut-offs ranging to over 400 feet deep. Of particular interest has been the "lessons learned" regarding the design, construction and performance of these massive remediations.

Given the ever-pressing and growing need to remediate our infrastructure as related to dams and levees, these two studies should be valuable reference sources for practitioners from all parts of the industry. The techniques have particular relevance to structures in the Ohio River Valley and its environs.

1. Introduction

For over 40 years, existing concrete and masonry dams have been stabilized in North America by the use of high capacity, prestressed anchors. These dams were found to have been structurally deficient in one or more modes including sliding, overturning and seismic resistance. In addition, anchors have often been required to stabilize appurtenant structures including powerhouses, spillways and abutments.

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For over 30 years, existing embankment dams have been remediated in North America by the use of cut-offs installed through the dam and into the underlying or abutting bedrock. These cut-offs are often referred to as "positive" since they involve the total excavation and replacement of in situ materials (i.e., fill, soil and rock), with a continuous diaphragm comprising a carefully engineered material, namely some type of concrete. Seepage cut-offs of this nature are required when the dam is judged to be progressing along the continuum of failure to the extent that failure of the embankment through piping into its foundation is a probability that cannot be ignored and must be rectified.

This paper provides a brief review of North American practice in both anchors and cut-offs, as collated in two national research projects initiated by the author. The value of these studies will be as a reference source for practitioners involved in the design, construction and monitoring of major dam remediations using these particular techniques. More information on the anchor program may be found in Bruce and Wolfhope (2005, 2006, 2007a, 2007b, 2007c), while Bruce et al. (2006) provides further data on cut-offs. This current paper describes only concrete cut-offs, whereas the paper by Bruce et al. (2006) details case histories involving cut-offs constructed by the Deep Mixing Method (FHWA 2000) and by backhoe as well. This decision has been made since the bulk of the work anticipated by the USACE, for example, will involve cut-offs comprising concrete only (Halpin, 2007).

2. Rock Anchors

2.1 The Goals of the Program

During the period 2005-2006, Phase 1 of the National Research Program was undertaken. This had three goals:

(i) conduct a comparative review of the five successive versions of the national "Recommendations" documents which have guided (and reflected) U.S. practice since 1974,
(ii) conduct a biography of all technical papers published on the subject of dam anchoring in North America, and
(iii) create a database containing as much information as possible on each structure anchored in North America.

The program was funded by a consortium of U.S. and Japanese interests. The investigations relied heavily on the cooperation of specialty contractors and specialist post-tensioning suppliers who provided access to historical records.

2.2 <u>The Recommendations</u>

2.2.1 *General Statement*

Current research indicates that the first U.S. dam to be stabilized by high capacity prestressed rock anchors was the John Hollis Bankhead Lock and Dam, Alabama (first 6 test anchors and 16 production anchors installed from 1962). This project was completed for the U.S. Army Corps of Engineers who had sufficient confidence in the technology (and, presumably, a pressing need for it!) that they were the sponsor for most of the half dozen or so similar applications in the six years that immediately followed. The U.S. Bureau of Reclamation began using anchors to stabilize appurtenant structures at dams in 1967. The Montana Power Company was also an early proponent. In those days, the technology was largely driven by the post tensioning specialists, employing the same principles and materials used in prestressed/post tensioned structural elements for new buildings and bridges. The "geotechnical" inputs, i.e., the drilling and grouting activities, were typically subcontracted to drilling contractors specializing in site investigation and dam grouting in the west, and to "tieback" contractors in the east.

Recognizing the need for some type of national guidance and uniformity, the Post Tensioning Division of the Prestressed Concrete Institute (PCI) formed an adhoc committee which published, in 1974, a 32-page document entitled "Tentative Recommendations for Prestressed Rock and Soil Anchors." It is interesting to note (Table 1) that half of the document comprised an appendix of annotated project photographs intended to illustrate and presumably promote anchor applications, including dam anchors at Libby Dam, MT, and Ocoee Dam, TN.

After publication of its document, the Post Tensioning Division of PCI left to form the Post Tensioning Institute (PTI) in 1976. Successive editions of its "Recommendations" were issued in 1980, 1986, 1996 and 2004. As general perspective to the development of concepts, Table 1 provides an analysis of the relative and absolute sizes of the major sections in each successive edition. It is immediately obvious that the original documents stressed "applications" – in an attempt to promote usage – while the most recent edition provides very detailed guidance (and commentary) on the "big five" in particular (i.e., Materials, Design, Corrosion Protection, Construction, and Stressing/Testing).

Table 1. Number of pages in major sections of successive U.S. "Recommendations" documents.

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

2.2.2 *Detailed Comparison by Technical Topic*

The structure of each successive edition has changed in the same way that the content has, although there are comparatively few structural differences between the 1996 and 2004 versions. The following comparison therefore is based on the structure of the 2004 version.

<u>Scope and Definitions (Chapters 1 and 2)</u>
The scope has remained relatively constant, and focuses on the anchors themselves (as components) as opposed to the analysis and design of the overall anchored system. A total of 72 technical terms are now defined, which represents a major expansion even over the 1996 edition: the first edition had 24 definitions, most of which, incidentally, remain valid and little changed.

<u>Specifications, Responsibilities and Submittals (Chapter 3)</u>
Whereas 1974 provided no insight into specifications and responsibilities, certain records were required to be maintained on the grouting operations. By 1980, however, specifications had been addressed, reflecting the need to tailor procurement processes to “experienced” contractors, “thoroughly experienced” and match the innovation of the technique with alternative procurements methods. It is notable that the three types of specification outlined in 1980 (namely open, performance and closed) have endured, although “closed” is now referred to as “prescriptive.” Building on a 1996 innovation, the responsibilities to be discharged during a project — regardless of type of specification — were summarized in 2004 as shown in <u>Table 2</u>. Clear guidance is also provided on the content of preconstruction submittals and as-built records. The former also include the requirement for the contractor to prepare a Construction Quality Plan. Emphasis remains on the need for “specialized equipment, knowledge, techniques and expert workmanship” and for “thoroughly experienced” contractors. The obvious, but often ignored, benefit of “clear communication and close co-operation,” especially in the start up phase of a project, is underlined.

<u>Anchor Materials (Chapter 4)</u>
The 1974 document very briefly refers to wires, strand, and bars, and to protective sheathing. In stark contrast, the current version has built to 10 pages providing definitive detail on materials used in each of the 10 major anchor components, with particular emphasis placed on steel, corrosion-inhibiting compounds, sheathings and grouts (cementitious and polyester). Strong cross-reference to relevant ASTM standards is provided as a direct guide to specification drafters.

Table 2. Tasks and responsibilities to be allocated for anchor works (PTI, 2004).

1.	Site investigation, geotechnical investigation and interpretation, site survey and potential work restrictions.
2.	Decision to use an anchor system, requirements for a pre-contract testing program, type of specification and procurement method, and contractor prequalification.
3.	Obtaining easements, permits, permissions.
4.	Overall scope of the work, design of the anchored structure, and definition of safety factors.
5.	Definition of service life (temporary or permanent) and required degree of corrosion protection.
6.	Anchor spacing and orientation, minimum total anchor length, free anchor length and anchor load.
7.	Anchor components and details.
8.	Determination of bond length.
9.	Details of water pressure testing, consolidation grouting and re-drilling of drill holes
10.	Details of corrosion protection.
11.	Type and number of tests.
12.	Evaluation of test results.
13.	Construction methods.
14.	Requirements for QA/QC Program.
15.	Supervision of the work.
16.	Maintenance and long-term monitoring.

Site Investigation (Included in Chapter 6 – Design)

An issued not referred to in 1974, recommended first in 1980 and completely revised and expanded in 1996 and 2004, clear guidance is now provided on the goals and details of a site investigation program. "Minimum requirements" are recommended. However, this remains an area where the anchor specialist often has less "leverage" to influence since the costs associated with such programs typically exercise strong control over the scope actually permitted by the owner.

Corrosion and Corrosion Protection (Chapter 5)

Given the major significance and relevance of this topic, this subject is discussed separately, Section 2.2.3, below.

Design (Chapter 6)

Judging from the relatively short and simplistic coverage of this aspect in 1974, it is fair to say that not much was really then *known* of the subject. Core drilling was considered absolutely necessary and preproduction pullout tests were "strongly recommended." However, two enduring issues were faced:

– The safety factor (on grout-rock bond) "should range from 1.5 to 2.5", with grout/steel bond not normally governing.
– A table of "typical (ultimate) bond stresses" was issued as guidance to designers.

Today even despite superior and often demonstrated knowledge of load transfer mechanisms (i.e., the issue of bond stresses NOT being uniform), the same philosophy prevails:

– The safety factor (reflecting, of course, the criticality of the project, rock variability and installation procedures) is normally 2 or more.
– The table of "average ultimate" bond stresses which is presented is basically identical except for typographic errors, to the 1974 table.

However, the current edition does provide very detailed guidance on critical design aspects, including allowable tendon stresses; minimum free and bond lengths;

factors influencing rock/grout bond stress development; anchor spacing; grout cover/strand spacing; and grout mix design.

Construction (Chapter 7)

As noted above, there was a strong bias in the 1974 document towards construction, largely, it may be assumed, because practice far led theory. Furthermore, much of what was described in 1974 remains valid, especially with respect to issues relating to grouts, grouting and tendon placement. Certain features, such as a reliance on core drilling, the use of a "fixed anchorage" (i.e., the use of a plate) at the lower end of multistrand tendons, and specific water take criteria to determine the need for "consolidation grouting" are, however, no longer valid.

The 2004 version expands upon the 1996 guidance, itself a radical improvement over its two immediate predecessors, and is strongly permeated by an emphasis on quality control and assurance. For example, practical recommendations are provided on the fabrication of tendons (including the pregrouting of encapsulations) and storage handling and insertion. Drilling methods are best "left to the discretion of the contractor, wherever possible," although specifications should clearly spell out what is not acceptable or permissible. In rock, rotary percussion is favored, and the drilling tolerance for deviation of 2º is "routinely achievable," while smaller tolerances may be difficult to achieve or to measure. Holes open for longer than 8 to 12 hours should be recleaned prior to tendon insertion and grouting.

The acceptance criterion for water pressure testing is adjusted to 10.3 liters in 10 minutes at 0.035 MPa for the entire hole. Technical background is provided on the selection of this threshold (based on fissure flow theory). Holes with artesian or flowing water are to be grouted and redrilled prior to water pressure testing. The pregrout (generally WCR = 0.5 to 1.0 by weight) is to be redrilled when it is weaker than the surrounding rock. When corrugated sheathing is preplaced, a water test should be conducted on it also, prior to any grouting of its annulus.

The treatment of grouting is considerably expanded and features a new decision tree to guide in the selection of appropriate levels of QC programs. Holes are to be grouted in a continuous operation not to exceed 1 hour, with grouts batched to within 5% component accuracy. The value of testing grout consistency by use of specific gravity measurements is illustrated. Special care is needed when grouting large corrugated sheaths; multiple stages may be required to avoid flotation or distortion. The cutting of "windows" in the sheath (to equalize pressures) is strictly prohibited.

Stressing, Load Testing and Acceptance (Chapter 8)

Given the professional experience and background of the drafting committee, it is surprising, in retrospect, to note the very simplistic contents of the 1974 document:

- "proof test" every anchor to ≥ 115% "transfer" load (to maximum 80% GUTS),
- hold for up to 15 minutes (but no creep criterion is given),
- lock-off at 50 to 70% GUTS,
- alignment load = 10% of Test Load, with movement only apparently recorded at this Test Load (115 to 150% transfer load). "If measured and calculated

elongations disagree by more than 10%, an investigation shall be made to determine the source of the discrepancy,"
- a lift-off test may be instructed by the Engineer "as soon as 24 hours after stressing."

Despite significant advances in the 1980 and 1986 documents, reflecting heavily on European practice and experience, significant technical flaws persisted until the completely rewritten 1996 version. The 2004 document was little changed in structure and content, the main highlights being as follows:

- Practical advice is provided on preparatory and set up operations and on equipment and instrumentation including calibration requirements.
- Alignment Load can vary from 5 to 25% of Design Load and 10% is common. This initial, or datum load, is the only preloading permitted prior to testing. On long, multistrand tendons, a monojack is often used to set the Alignment Load, to ensure uniform initial loading of the strands.
- Maximum tendon stress is 80% F_{pu}.
- Preproduction ("disposable," test anchors, typically 1 to 3 in number), Performance and Proof Tests are defined, the latter two covering all production anchors.
- For Performance Testing, the first 2 or 3 anchors plus 2 to 5% of the remainder are selected. The test is a progressive cyclic loading sequence, typically to 1.33 times Working Load. A short (10 or 60 minute) creep test is run at Test Load.
- Proof Tests are simpler, requiring no cycling and are conducted to the same stress limits. The option is provided to return to Alignment Load prior to lock-off (in order to measure the permanent movement at Test Load), otherwise this movement can be estimated from measurements from representative Performance Tests.
- Supplementary Extended Creep Tests are not normally performed on rock anchors, except when installed in very decomposed or argillaceous rocks. A load cell is required and the load steps and reading frequencies are specified.
- Lock-off load shall not exceed 70% F_{pu}, and the wedges will be seated at 50% F_{pu} or more.
- The initial lift-off reading shall be accurate to 2%.
- There are three acceptance criteria for every anchor:
  - Creep: less than 1 mm in the period 1 to 10 minutes, or less than 2 mm in the period 6 to 60 minutes.
  - Movement: there is no criterion on residual movement, but clear criteria are set on the minimum elastic movement (equivalent to at least 80% free length plus jack length) and the maximum elastic movement (equivalent to 100% free length, plus 50% bond length plus jack length).
  - Lift-Off Reading: within 5% of the designed Lock-Off load.
- A decision tree guides practitioners in the event of a failure in any one criterion. The "enhanced" creep criterion is 1 mm in the period 1 to 60 minutes at Test Load.
- The monitoring of service behavior is also addressed. Typically 3 to 10% of the anchors are monitored (if desired), by load cells or lift-off tests. Initial monitoring is at 1 to 3-month intervals, stretching to 2 years later.

Epoxy-coated strand (Supplement)
This material and its use was first discussed systematically in 1996, although minor references had been made in 1986. The 2004 document contains a separate supplement dealing with specifications, materials, design, construction and testing, being a condensed and modified version of a document produced by the ADSC Epoxy-Coated Strand Task Force in November 2003. The Scope (Section 1) notes that anchors made from such strand "require experience and techniques beyond those for bare strand anchors." The supplement is a condensed version of the "Supplement for Epoxy-Coated Strand" as prepared by the ADSC Epoxy-Coated Strand Task Force (November 2003). It supplements the recommendations provided in the general recommendations with respect to specifications/responsibilities/submittals; materials; design; construction; and stressing and testing.

2.2.3 *The Issue of Corrosion Protection*

1974. Figure 1 illustrates the very simple approach to tendon protection, i.e., cement grout or nothing. "Permanent" is defined as "generally more than a 3-year service life." Sheathing is only discussed as a debonding medium, not a corrosion protection barrier. For permanent anchors "protective corrosion seals over their entire length" are to be provided (but are not defined). For two stage grouted tendons, *sheathing can be omitted*, the implication being that cement grout alone would be acceptable.

1980. The same Figure 1 is reproduced (as it was also in 1986). The term "permanent" is now reduced to 18 months or more, and growing attention is drawn to the requirements of permanent anchors: sheathing is for debonding "and/or to provide corrosion protection," as is secondary cement grout. Corrugated protection, and epoxy coating for bars, are discussed.

The type and details of corrosion protection are to be based on longevity, anchor environment, consequences of future and in-hole conditions/length of time before grouting. For the bond length, cement grout is considered "the first level of corrosion protection," and plastic corrugated sheathing ("for multiple corrosion protection schemes") or epoxy are permitted. Such protection is to extend at least 2 feet into the free length. The free length is to have, as a minimum, a sheath with cement grout or grease infill. A full length outer sheath is regarded as "good practice."

1986. The emphasis is placed on first investigating the chemical aggressiveness of the soil and ground water: "Permanent anchors placed in environments where any one of these tests indicate critical values must be encapsulated over their full length." Thus, even up until the next set of Recommendations (1996), it was considered acceptable to allow anchors for dams to be installed without any protection for the bond length other than cement grout, depending on the results of laboratory tests on small samples. Encapsulation was not detailed.

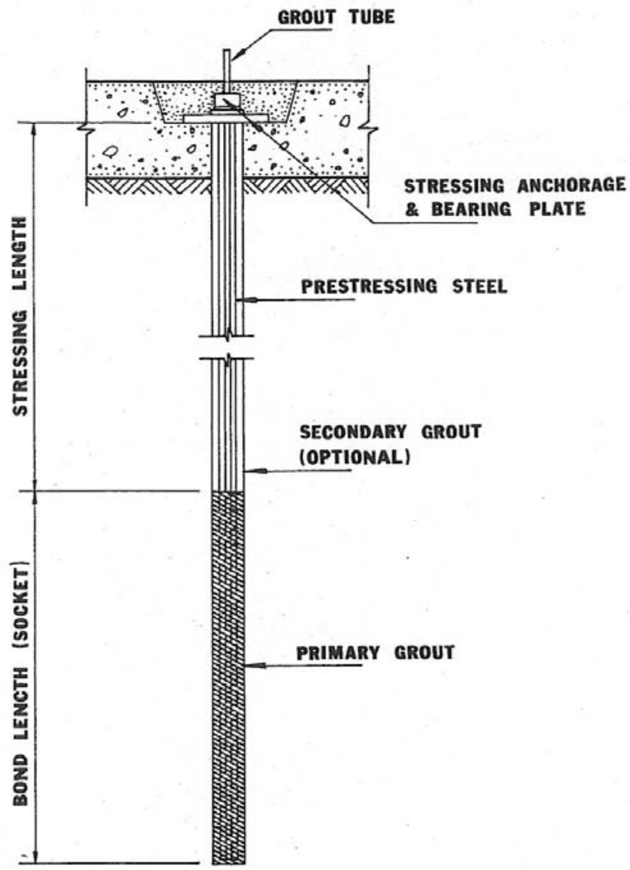


Figure 1. Rock Anchor Components (PCI, 1974).
(Note the lack of protection to the steel other than cement grout.)

1996. Permanence is now defined as a minimum of 24 months in a completely revised set of Recommendations. A wider spectrum of issues than simple chemistry now have to be considered when selecting corrosion protection principles. A major breakthrough was to identify two classes of protection (Class I and II) for permanent anchors to replace the poorly defined and loosely used "double" and "single" corrosion protection systems offered by various tendon manufacturers. The details are summarized in Table 3 and a "decision tree" was provided for the guidance of designers (Nierlich and Bruce, 1997).

2004. The 1996 Recommendations were revalidated while it is stated that, for permanent anchors, "aggressive conditions shall be assumed if the aggressivity of the ground has not been quantified by testing." Table 3 was revised, as shown in Table 4, mainly to clarify the acceptable Class I status of epoxy protected steel in a "water proofed hole." The sophistication of contemporary tendons is shown in Figure 2. A long supplement is devoted to epoxy protected strand.

Table 3. Corrosion Protection Requirements (PTI, 1996).

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 4. Corrosion Protection Requirements (PTI, 2004).

CLASS	CORROSION PROTECTION REQUIREMENTS		
	ANCHORAGE	FREE STRESSING LENGTH	TENDON BOND LENGTH
I ENCAPSULATED TENDON	Trumpet Cover if exposed	• Corrosion inhibiting compound-filled sheath encased in grout, or • Grout-filled sheath, or • Grout-encased epoxy-coated strand in a successfully water-pressure tested drill hole	• Grout-filled encapsulation, or • Epoxy-coated strand tendon in a successfully water-pressure tested drill hole
II GROUT PROTECTED TENDON	Trumpet Cover if exposed	• Corrosion inhibiting compound-filled sheath encased in grout, or • Heat shrink sleeve, or • Grout-encased epoxy-coated bar tendon, or • Polyester resin for fully bonded bar tendons in sound rock with non-aggressive ground water	• Grout • Polyester resin in sound rock with non-aggressive ground water

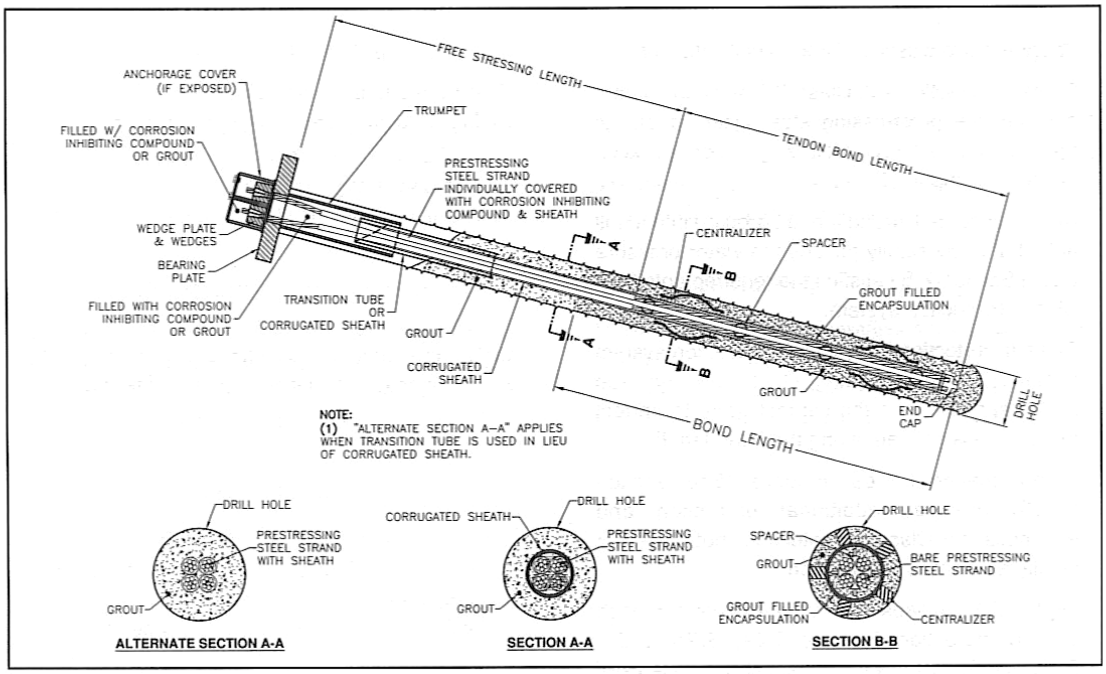


Figure 2. Class I Protection – Encapsulated Strand Anchor (PTI, 2004).

Overall, therefore, one is impressed that between 1974 and 2006 (i) extremely sophisticated corrosion protection systems were developed and (ii) the latitude offered to designers relative to choice of corrosion protection intensity and details was severely restricted: to install a permanent anchor in a dam without Class I protection is now not only impermissible, but unthinkable.

It must also be noted that the philosophy of pregrouting and redrilling the hole ("waterproofing") if it were to fail a permeability test was reaffirmed from 1974 onwards: indeed the early "pass-fail" acceptance criteria were, in fact, very rigorous and led to most anchors on most projects having to be pregrouted and redrilled several times. Although laudable, this was often, in fact, "extra work" since the criterion to achieve grout tightness is really much more lax than the criterion needed to provide the specified degree of water tightness. The saving grace of many of the early anchors was doubtless, therefore, the somewhat erroneous drill hole "waterproofing" criterion under which they were constructed.

As a final word on corrosion and corrosion protection, an analysis of the case histories in the database further illustrates the evolution of systems and philosophies, as shown in Figure 3. This figure again underlines the fact that, even until the late 1990s, anchors were being installed with arguably vulnerable corrosion protection systems, which are practically inconsistent with the concept of "permanence."

2.3 The Bibliography

A comprehensive literature survey was completed to identify published dam anchoring case studies and various publications documenting the evolution of North American dam anchoring practices and construction methods. Over 235

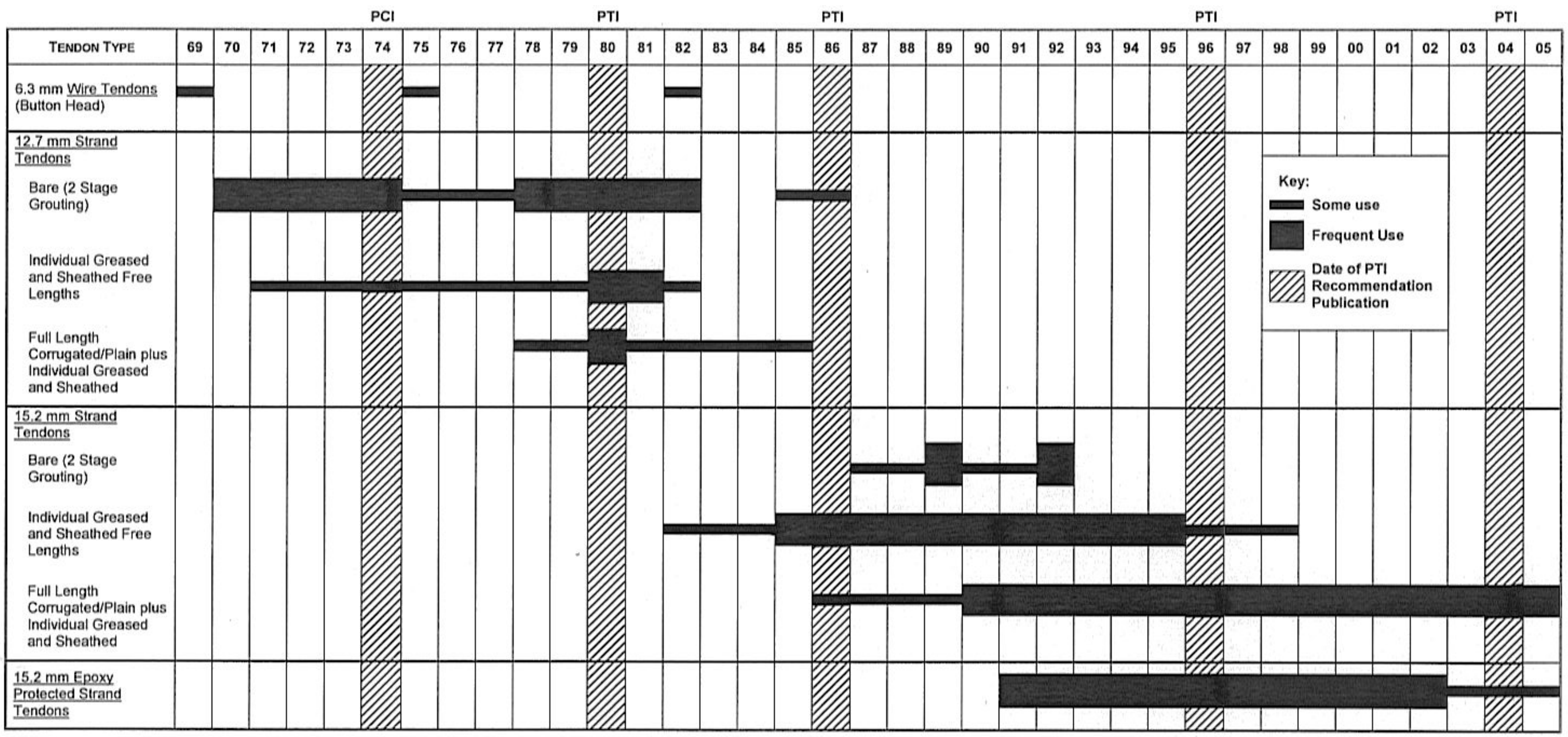


Figure 3. Illustrating how the types of Corrosion Protection have Evolved (Data Drawn from Authors' Research Database).

technical papers have been compiled relating to North American post-tensioned rock anchoring projects. Hard copy and electronic versions of each have been collected. Figure 4 shows the number of publications by year indicating that over the first five years of the twenty-first century, industry has been publishing at a rate of about 13 papers per year. These papers relate to over 200 dams.

2.4 The Database

Given that anchoring is conducted in dams other than earth embankments, Figure 5 presents a histogram of U.S. dam construction involving concrete and masonry structures.

Phase 1 of the database preparation has revealed 239 projects whose anchor details are essentially "complete," a further 50 projects classified as "nearly complete," and a further 104 case histories regarded as "incomplete." (This will clearly drive the nature of the Phase 2 studies.) Nevertheless, a total of 318 anchor projects do have sufficient data to allow year of commencement to be plotted (Figure 6). Over the 40-year period, well in excess of 20,000 tendons were installed, with the peak years, driven by Federal Regulatory demands, being in the period 1988-2002.

A geographic distribution of dams anchored in the U.S. and Canada is provided in Figures 7 and 8, respectively.

3. Cut-Offs

3.1 Scope

The study by Bruce et al. (2006) identified thirty case histories of embankment dams remediated by some type of "positive" cut-off. That list included nine projects wherein Deep Mixing Methods or backhoe had been used. The former technology creates a cut-off comprising "soilcrete," a mixture of in situ soils, and grout. The latter has been used in relatively shallow applications (less than 80 feet) involving soil-cement, or soil-cement-bentonite diaphragms which have low permeability but also low strength. In order to focus the scope of the study, cut-offs involving soil or rock grouting methods (as the sole hydraulic barrier) were omitted. For guidance on grouted cut-offs, the reader is referred to Weaver and Bruce (2007).

Given the nature of the cut-off work anticipated in the next decade on major dams in the U.S., the scope of this paper has been further restricted to deep cut-offs, constructed by the panel or secant pile methods and comprising some type of concrete backfill. For information, Figure 9 illustrates panel wall construction (by clamshell or hydromill), and Figure 10 illustrates the principle of secant wall construction.

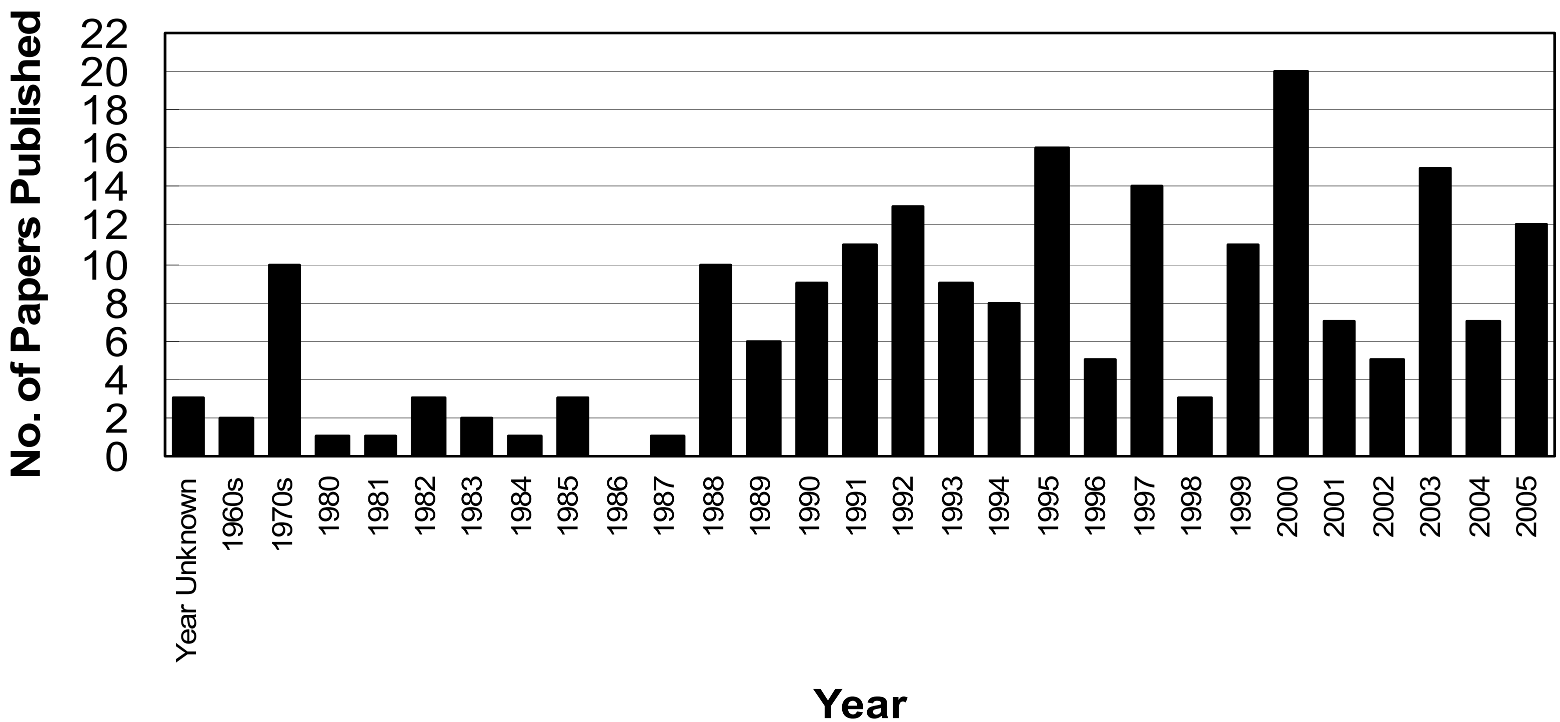
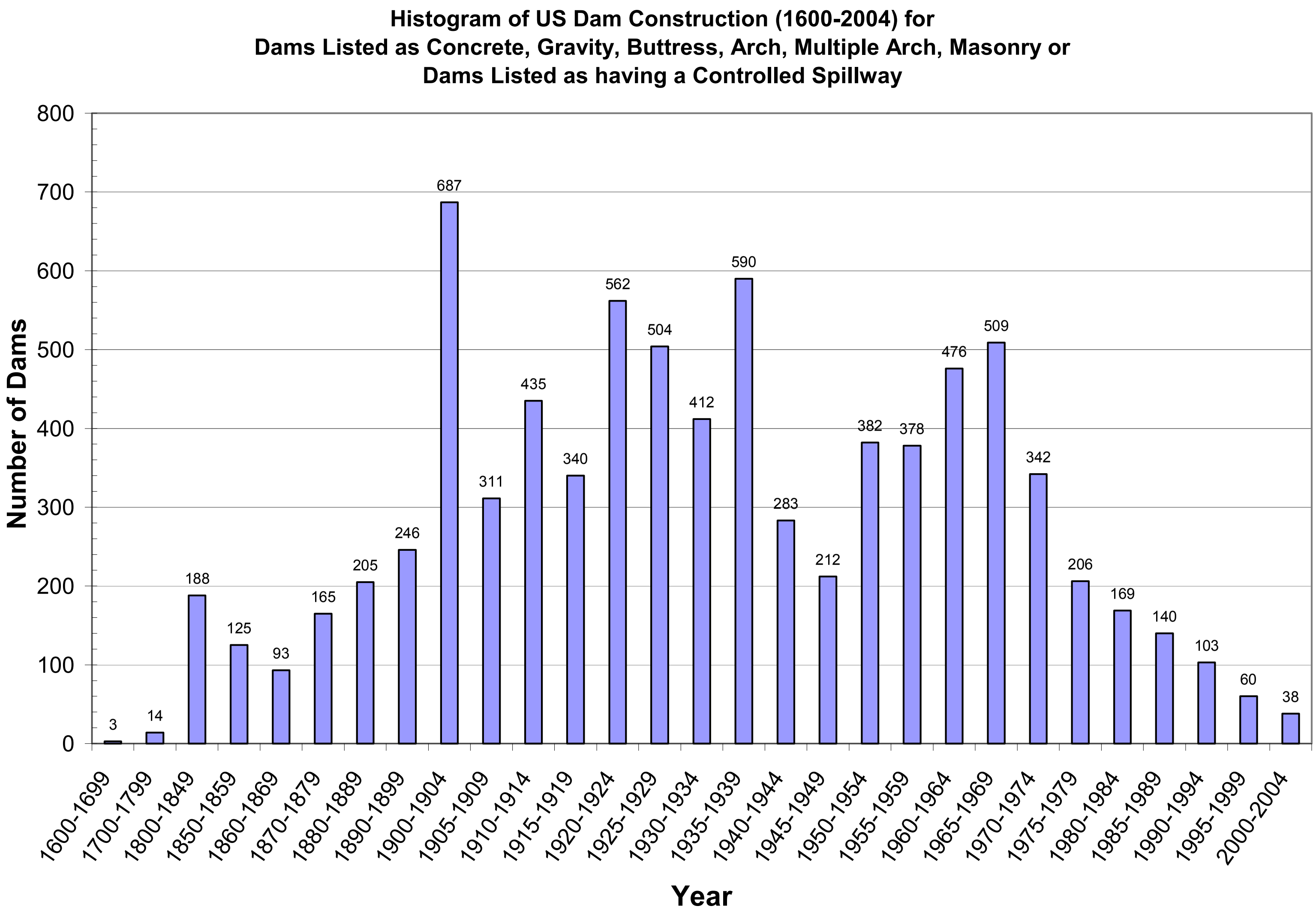
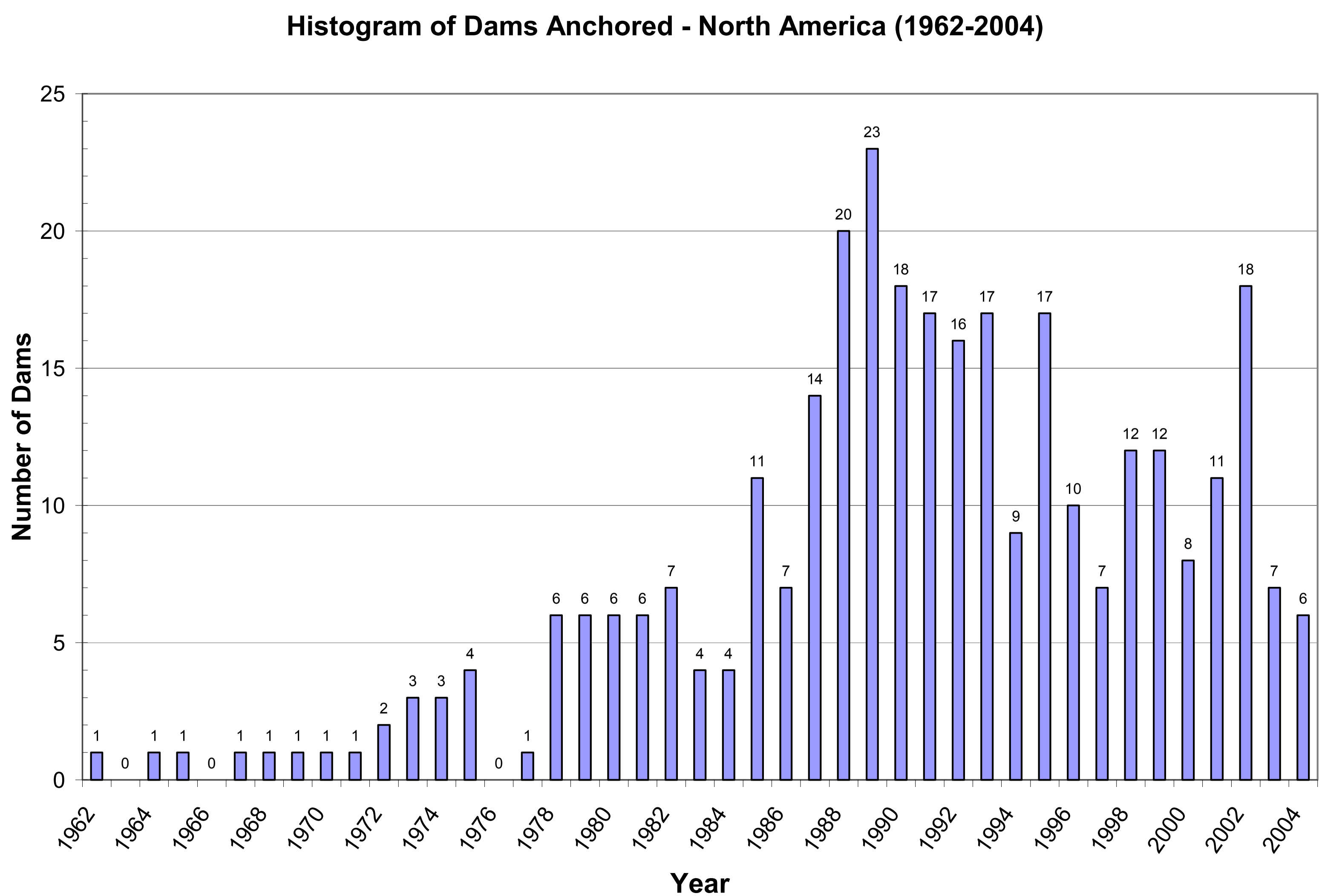


Figure 4. Numbers of technical papers on dam anchoring published per year.



Notes: 1) Source of Data - National Inventory of Dams, USACE, 79777 Dams Total
2) Does not include 9500 dams where the year construction completed is not reported or invalid
3) Total Number of Dams (not including 9500 with unreported/invalid data) = 8178

Figure 5. Histogram of U.S. dam construction by type of dam.



Notes: 1) Total Number of Dams Shown = 323
2) Does not include 70 anchor case studies where year anchored not reported or as yet ascertained.

Figure 6. Histogram of dams anchored by year (1962-2004).

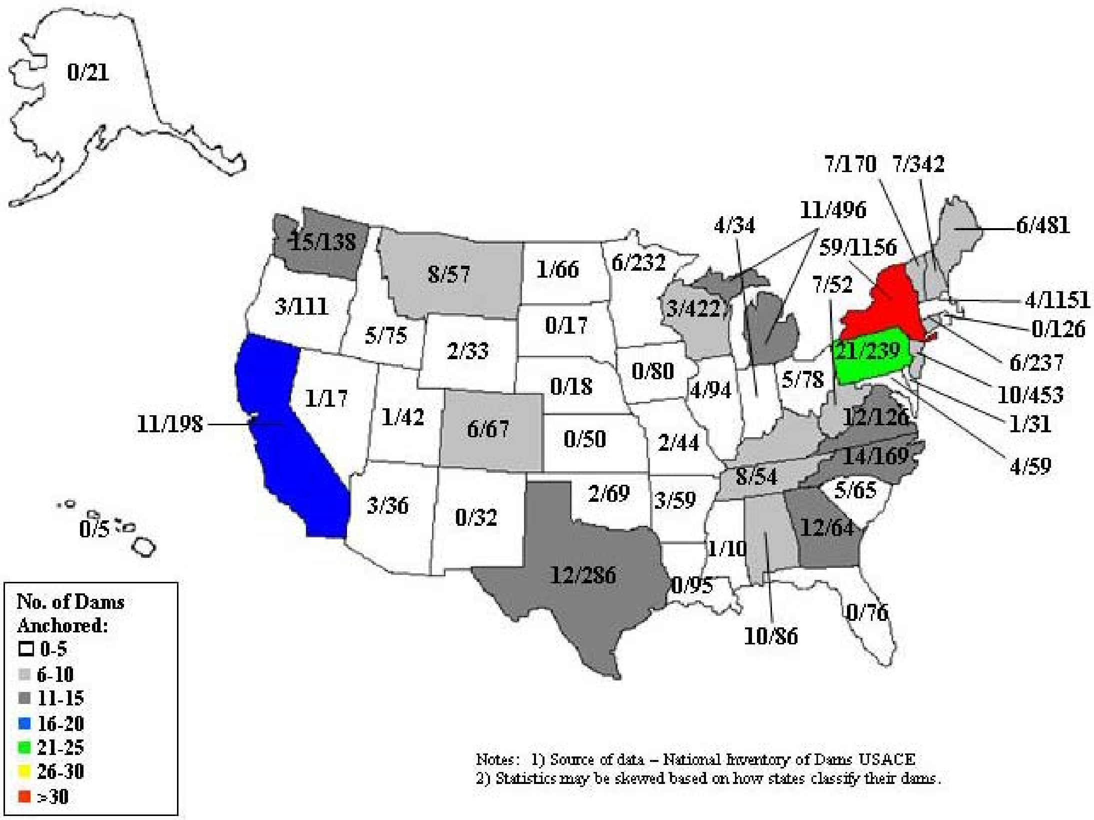


Figure 7. Geographic distribution of U.S. dams which have been anchored.

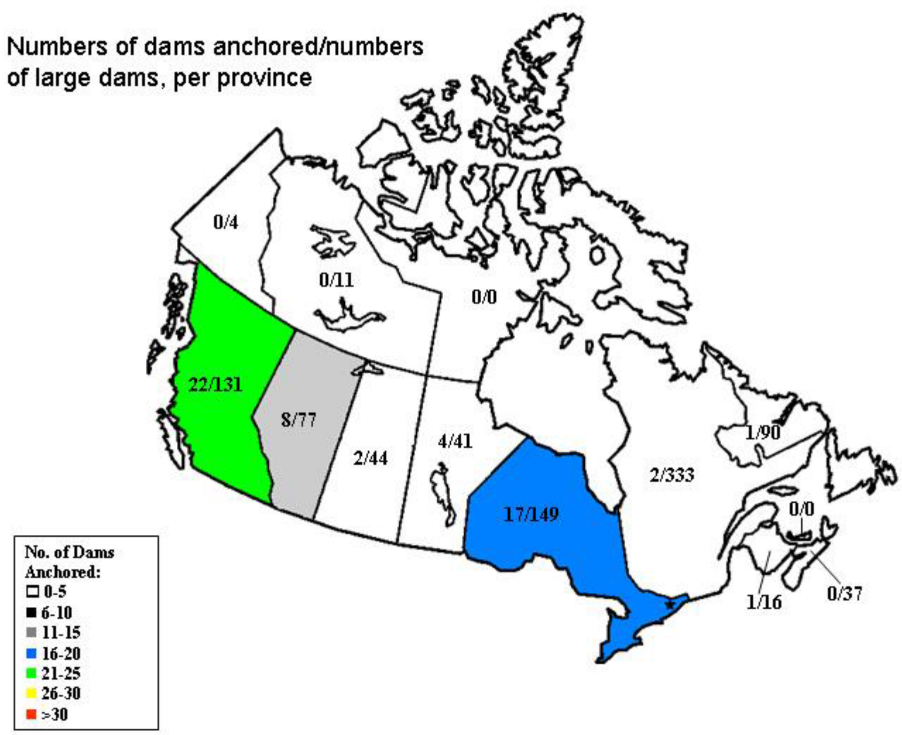


Figure 8. Geographic distribution of Canadian dams which have been anchored.

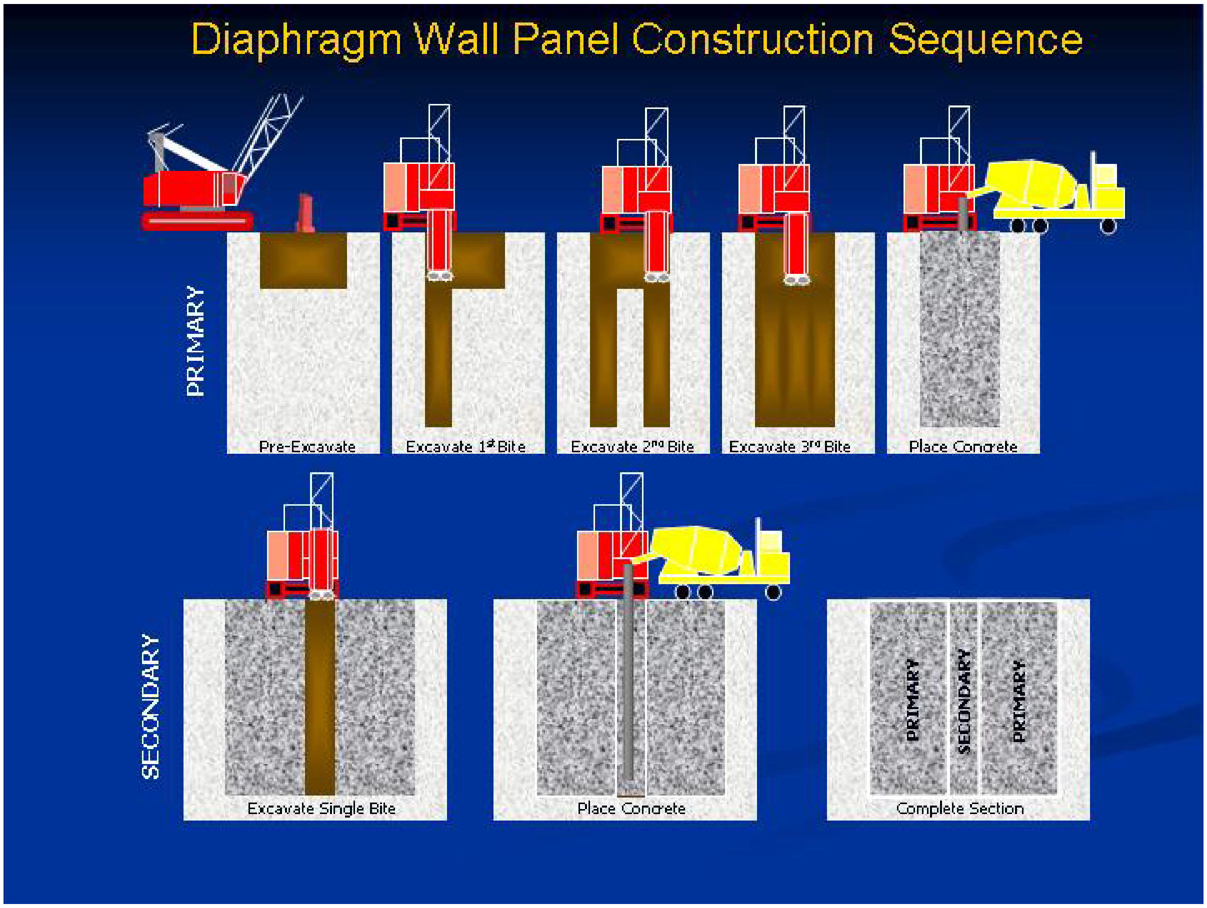


Figure 9. Diaphragm wall panel construction sequence.

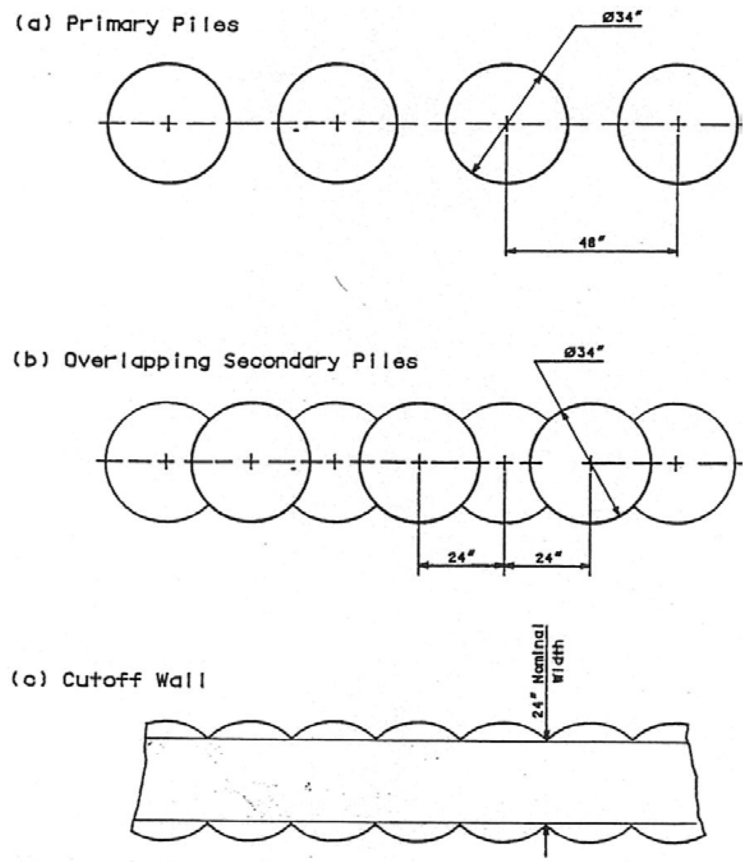


Figure 10. Construction sequence for a secant cut-off wall (Bruce and Dugnani, 1995).

3.2 Case History Data

Table 5 provides summary data on 18 projects conducted in North America (all but one in the U.S.), featuring seepage remediation by concrete cut-off walls. Details from a further 3 projects (Sam Rayburn, TX; W.F. George, AL, Phase 1; and Stewart Bridge, NY) remain to be acquired. Table 6 summarizes the size of the cut-offs and the total square footage installed to date. It may be observed that the cumulative value of these 18 projects, conducted over a period of 32 years, could well be matched on a few USACE dams alone over the next 5 years. Figure 11 shows how these 18 projects have been distributed over the years and the different techniques and contractors involved. There is a small, but highly qualified, group of specialty contractors capable of doing such work.

Table 5. Details of remedial dam cut-off projects using panel or secant concrete cut-offs.

Dam Name and Year of Remediation	Contractor	Type of Wall	Composition of Wall	Ground Conditions	Purpose of Wall	Scope of Project				References
						Area	Min. Width	Depth	Length	
1. WOLF CREEK, KY. 1975-1979	ICOS	24-inch diameter Primary Piles, joined by 24-inch wide clamshell panels. Two phases of work.	Concrete.	Dam FILL, and ALLUVIUM over argillaceous and karstic LIMESTONE with cavities, often clay-filled.	To provide a "Positive concrete cut-off" through dam and into bedrock to stop seepage, progressively developing in the karst.	270,000 sf (Phase 1) plus 261,000 sf (Phase 2)	24 in	Max. 280 ft	2,000 ft plus 1,250 ft	• ICOS brochures (undated) • Fetzer (1988)
2. ADDICKS AND BARKER, TX. Completed in 1982 (Phase 1 took 5 months)	Soletanche*	36-inch thick panel wall with clamshell excavation using Kelly.	Soil-Bentonite.	Dam FILL over CLAY.	To prevent seepage and piping through core.	450,000 sf (Phase 1) plus 730,000 sf (Phase 2)	36 in	Max 66 ft typically 35 to 52 ft	8,330 ft plus 12,900 ft	• Soletanche website.
3. ST. STEPHENS, SC. 1984	Soletanche	24-inch-thick concrete panel wall, installed by Hydromill. Plus upstream joint protection by soil-bentonite panels.	Concrete and soil-bentonite.	Dam FILL, over sandy marly SHALE.	To provide a cut-off through dam.	78,600 sf (concrete) plus 28,000 sf (soil-bentonite)	24 in	Max. 120 ft including 3 ft into shale	695 ft	• USACE Report (1984) • Soletanche (various) • Parkinson (1986) • Bruce (1990)
4. FONTENELLE, WY. 1986-1988	Soletanche	24-inch-thick concrete panel wall installed by Hydromill. Minor soil-bentonite panels.	Concrete and soil-bentonite.	Dam FILL over horizontally bedded SANDSTONE.	To prevent piping of core into permeable sandstone abutment.	50,000 sf (LA test) plus 100,000 sf (RA test) plus 700,000 sf (Production)	24 in	Max 180 ft including 16 to 160 ft into rock	Approx. 6,000 ft	• Cyganiewicz (1988) • Soletanche (various)

* Soletanche have operated in the U.S. under different business identities over the years. "Soletanche" is used herein as the general term.

Dam Name and Year of Remediation	Contractor	Type of Wall	Composition of Wall	Ground Conditions	Purpose of Wall	Scope of Project				References
						Area	Min. Width	Depth	Length	
5. NAVAJO, NM. 1987-1988	Soletanche	39-inch-thick panel wall installed by Hydromill.	Concrete.	Dam FILL over flat-lying SANDSTONE with layers of SILTSTONE and SHALES. Very fractured, weathered and permeable with vertical and horizontal joints.	To prevent piping of core into permeable sandstone abutment.	130,000 sf	39 in	Max 400 ft including over 50 ft into rock	450 ft	• Davidson (1990) • Dewey (1988)
6. MUD MOUNTAIN, WA. 1988-1990, (Mainly over period December 1989 to April 1990)	Soletanche	33- and 39-inch-thick panels wall installed by Hydromill. Extensive pregrouting of core.	Concrete.	Dam FILL silty and sandy, over very hard, blocky cemented ANDESITE (UCS over 20,000 psi).	To prevent seepage through the core.	133,000 sf	33 in in abutments, 39 in in center	Max. 402 ft	700 ft	• Soletanche brochures • Eckerlin (1993) • ENR (1990) • Davidson et al. (1991) • Graybeal and Levallois (1991)
7. WISTER, OK. 1990-1991 (6 months)	Bauer	24-inch-thick panel wall installed by Hydromill.	Plastic concrete.	Dam FILL, over 30 feet of ALLUVIALS overlying SANDSTONE and SHALE.	To prevent piping through the embankment.	216,000 sf	24 in	Approx. 54 ft	Approx. 4,000 ft	• Erwin (1994) • Erwin and Glenn (1991)
8. WELLS, WA. 1990-1991 (7 months, 208 working shifts)	Bencor-Petrifond	30-inch-thick panel wall installed by clamshell and joint pipe ends.	Concrete.	Dam FILL with permeable zones over miscellaneous ALLUVIUM and very dense TILL.	Prevent piping through permeable core materials, in gap between original cut-off and rockhead.	124,320 sf	30 in	80 to 223 ft	849 ft	• Kulesza et al. (1994) • Roberts and Ho (1991)

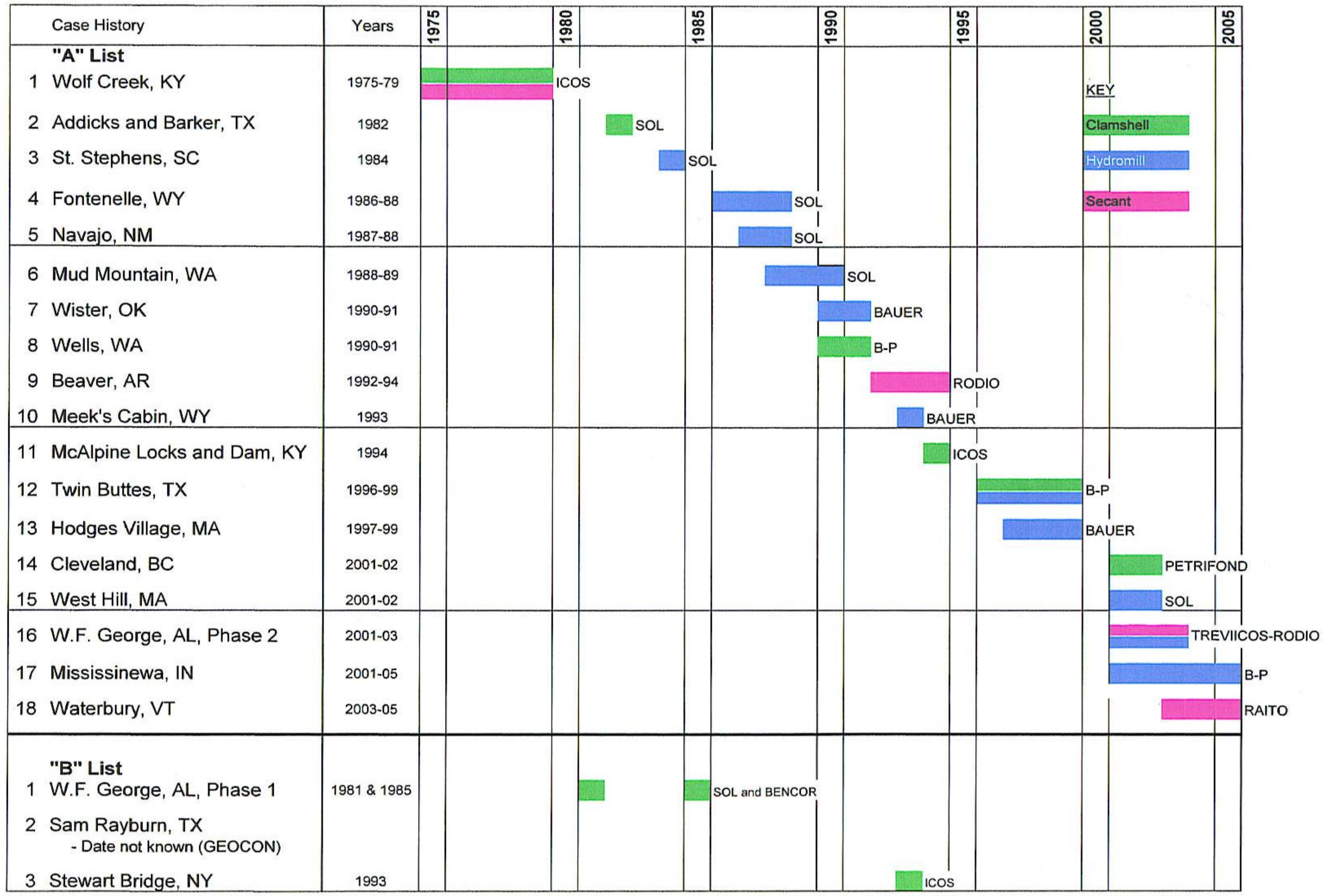
Dam Name and Year of Remediation	Contractor	Type of Wall	Composition of Wall	Ground Conditions	Purpose of Wall	Scope of Project				References
						Area	Min. Width	Depth	Length	
9. BEAVER, AR. 1992-1994 (22 months)	Rodio-Nicholson	24-inch-thick wall created by 34-inch secant columns at 24-inch centers.	Concrete.	Dam FILL over very variable and permeable karstic LIMESTONE with open and clay-filled cavities. Some sandstone.	To prevent seepage through karstic limestone under embankment.	207,700 sf	24 in	80 to 185 ft	1,475 ft	• Bruce and Dugnani (1996) • Bruce and Stefani (1996)
10. MEEK'S CABIN, WY. 1993	Bauer	36-inch-thick panel wall formed by Hydromill.	Plastic concrete.	Dam FILL over very variable glacial TILL and OUTWASH comprising sand, gravel, cobbles, and boulders.	To prevent seepage through glacial outwash deposits.	125,000 sf	36 in	130 to 170 ft including minimum 10 ft into lower glacial till	825 ft	• Pagano and Pache (1995) • Gagliardi and Routh (1993)
11. McALPINE LOCKS AND DAM, KY. 1994 (6 months)	ICOS	24-inch panel wall formed by clamshell and chisel. Upper portion pretrenched with backhoe and filled with cement-bentonite.	Concrete.	Very variable FILL, with rubble, cobbles, and boulders over silty CLAY over SHALE and LIMESTONE.	To prevent seepage through dike and alluvials.	51,000 sf	24 in	30 to 90 ft plus 5 ft into bedrock	850 ft	• Murray (1994)
12. TWIN BUTTES, TX. 1996-1999	Granite-Bencor-Petrifond	30-inch wide wall formed with panel methods (Kelly and cable suspended grabs, plus chisels.) Hydromill also used.	Soil-cement-bentonite.	Dam FILL over CLAY and ALLUVIAL gravel often highly cemented (up to 15,000 psi) and SHALEY SAND-STONE bedrock.	To prevent seepage through dam foundation causing uplift or blowout.	1,400,000 sf	30 in	Max 100 ft deep including at least 2.5 ft into rock	21,000 ft	• Dinneen and Sheshkier (1997)

Dam Name and Year of Remediation	Contractor	Type of Wall	Composition of Wall	Ground Conditions	Purpose of Wall	Scope of Project				References
						Area	Min. Width	Depth	Length	
13. HODGES VILLAGE, MA. 1997-1999	Bauer of America	31.5-inch-wide panel wall formed by hydromill plus use of grab for initial excavation and boulder removal.	Concrete.	Homogeneous permeable sand and gravel FILL over stratified glacial OUTWASH sands, gravels and cobbles over MICA SCHIST.	To prevent seepage through the dam and foundation.	185,021 sf (dam) plus 75,486 sf (dike)	31.5 in	Max 143 ft (avg. 89 ft) including 5 ft into rock (dam) plus Max 77' (avg. 57') including 5 ft into rock (dike)	2,078 ft (dam) plus 1,315 ft (dike)	• Dunbar and Sheahan (1999) • USACE (2005)
14. CLEVELAND, BC. 2001-2002 (4 months)	Petrifond and Vancouver Pile Driving	32-inch panel wall constructed by cable suspended clamshell.	Plastic Concrete.	Heterogeneous glacial sediments including SILT, SAND, GRAVEL and TILL with hard igneous boulders.	To prevent seepage through glacial and interglacial foundation sediments, especially 20-foot sand layer.	55,000 sf (est)	32 in	20 to 75 ft	1,004 ft	• Singh et al. (2005)
15. WEST HILL, MA. 2002-2002	Soletanche-McManus JV	31.5-inch panel wall constructed by hydromill plus clamshell for pre-excavation.	Concrete.	Zoned random and impermeable FILL over permeable stratified sand and gravel. Glacial OUTWASH over GRANTITE GNEISS.	To prevent seepage through the foundation.	143,000 sf	31.5 in	Max 120 ft including 2 ft into rock (avg. 69 ft)	2,083 ft	• USACE (2004)
16. W.F. GEORGE, AL. 2001-2003	Treviicos-Rodio	24-inch-thick secant pile wall (50-inch diameter at 33-inch centers) plus hydromill through concrete structures.	Concrete.	Over 90 feet of water over LIMESTONE with light karst, and very soft horizons (rock strength over 14,000 psi in places).	To prevent seepage through karstified bedrock under concrete dam section.	Approx. 300,000 sf including hydromill wall (50,000 sf)	24 in	100 ft of excavation (under 90 ft water)	2,040 ft	• Ressi (2003) • Ressi (2005)

Dam Name and Year of Remediation	Contractor	Type of Wall	Composition of Wall	Ground Conditions	Purpose of Wall	Scope of Project				References
						Area	Min. Width	Depth	Length	
17. MISSISSINEWA, IN. 2001-2005 (including shutdown for grouting)	Bencor-Petrifond	18-inch thick panel wall, using 30-inch hydromill, with clamshells through dam.	Concrete.	Dam FILL over karstic LIMESTONE (to 25,000 psi), very permeable and jointed.	To prevent piping into karstic limestone foundation.	427,308 sf (330,127 embank-ment plus 97,181 in rock)	30 in	123 to 230 ft (including max 148 ft into limestone) av. 180 ft	2,600 ft	• Hornbeck and Henn (2001) • Henn and Brosi (2005)
18. WATERBURY, VT. 2003-2005	Raito	18-inch-thick secant wall using 79-inch piles at about 66-inch centers.	Concrete in lower 100 ft, permeable backfill above.	Zoned EARTHFILL dam over permeable GLACIAL DEPOSITS with detached rock slabs and MICA SCHIST.	To prevent seepage and piping in the glacial gorge.	20,000 sf designed but only 13 of 22 piles completed	18 in	125-180 ft (avg. 163 ft) with "anomaly" to 222 ft, including 10 ft into rock	124 ft designed	• USACE (2006) • Washington et al. (2005)

Table 6. Concrete Cut-Offs for Existing Embankment Dams

Type of Construction	Number of Projects	Square Footage		
		Smallest	Largest	Total
Mainly Clamshell	5	51,000	1,400,000	2,810,320
Mainly Hydromill	9	104,600	850,000	2,389,415
Mainly Secant Piles	4	12,000	531,000	1,050,700
Total	**18**			**6,250,435**



ICOS = TreviICOS
SOL = Soletanche Bachy
BAUER = Bauer Spezialteifbau GmbH
B-P = Bencor-Petrifond
RODIO = Rodio
PETRIFOND = Petrifond
RAITO = Raito, Inc.

Figure 11. Project listing showing chronology, type of cut-off and specialty contractor.

3.3 Observations and Lessons Learned

3.3.1 *Investigations, Design, Specifications and Contractor Procurement*

(i) Intensive, focused site investigations are essential as the basis for cut-off design and contractor bidding purposes. In particular, these investigations must not only identify rock mass lithology, structure and strength, but also the potential for loss of slurry during panel excavation. This has not always been done, and cost and schedule have suffered accordingly on certain major projects.

(ii) Special considerations have had to be made when designing cut-offs which must contact existing concrete structures, or which must be installed in very deep-sided valley sections, or which must toe in to especially strong rock.

(iii) "Test Sections" have proved to be extremely valuable, especially for the contractor to refine his means, methods and quality control systems. Such programs have also given the dam safety officials and owners the opportunity to gain confidence and understanding in the response of their dams to the invasive surgery that constitutes cut-off wall construction. Furthermore, such programs have occasionally shown that the foreseen construction method was practically impossible (e.g., a hydromill at Beaver Dam, AR) or that significant facilitation works (e.g., pregrouting of alignment at Mississinewa Dam, IN) were required.

(iv) Every project has involved a high degree of risk and complexity and has demanded superior levels of collaboration between designer and contractor. This situation has been best satisfied by procuring a contractor on the basis of "best value," not "low bid." This involves the use of RFP's (Requests for Proposals) with a heavy emphasis on the technical submittal and, in particular, on corporate experience, expertise and resources, and the project-specific Method Statement. These projects are essentially based on Performance, as opposed to Prescriptive Specifications . Partnering arrangements (which are post-contract) have proved very useful to both parties when entered into with confidence and enthusiasm.

3.3.2 *Construction and QA/QC*

(i) The specialty contractors have developed a wide and responsive variety of equipment and techniques to assure penetration and wall continuity in all ground conditions. More than one technique, e.g., clamshell followed by hydromill, has frequently been used on the same project and especially where bouldery conditions have been encountered.

(ii) Cut-offs can be safely constructed with high lake levels, provided that the slurry level can be maintained a minimum of 3 feet higher. In extreme geological conditions, this may demand pretreatment of the embankment (e.g., Mud Mountain Dam, WA) or the rock mass (Mississinewa Dam, IN) to guard against massive, sudden slurry loss.

(iii) In less severe conditions, contractors have also developed a variety of defenses against slurry losses of smaller volume and rate by providing large slurry reserves, flocculating agents, fillers, or by limiting the open-panel width.

(iv) Very tight verticality tolerances are necessary to assure, and especially in deeper cut-offs, but have been not only difficult to satisfy, but also difficult to measure accurately. Such deviation tolerances have been measured to be less than 0.5% of the wall depth.

(v) The deepest panel walls have been installed at Wells Dam, WA (223 feet, clamshell) and at Mud Mountain Dam, WA (402 feet, hydromill). The hydromill has proved to be the method of choice for cut-offs in fill, alluvials and in rock masses of unconfined cmpressive strengths less than 10,000 psi (massive) to 20,000 psi (fissile, and so, rippable).
(vi) Secant pile cut-offs are expensive and intricate to build. However, they are the only choice in certain conditions (e.g., heavily karstified, hard limestone rock masses) which would otherwise defeat the hydromill. The deepest such wall (albeit a composite pile/panel wall) was the first — at Wolf Creek, KY in 1975 — which reached a maximum of 280 feet.
(vii) A wide range of backfill materials have been used, ranging from low strength plastic concrete, to conventional high strength concrete.
(viii) The preparation and maintenance of a stable and durable working platform has proved always to be a beneficial investment, and its value should not be underestimated.
(ix) The highest standards of real time QA/QC and verification are essential to specify and implement. This applies to every phase of the excavation process, and to each of the materials employed.
(x) Enhancements have progressively been made in cut-off excavation technology, especially to raise productivity (and particularly in difficult conditions), to increase mechanical reliability, and to improve the practicality and accuracy of deviation control and measurement.

3.3.3 *Performance of Cut-Offs*

Little has in fact been published to date describing the actual efficiency of cut-off walls after their installation: most of the publications describe design and construction and have usually been written soon after construction by the contractors themselves. Although there is some evidence (e.g., Davidson, 1990) that the walls have not always functioned as well as anticipated, it can be reasonably assumed that the majority of the remediations have been successful, <u>provided</u> the wall has been extended laterally and vertically into competent, impermeable and non-erodible bedrock. It may also be stated that the capabilities of the technology of the day have not always been able to satisfy this depth criterion.

3.4 <u>Recommendations for Future Projects</u>

3.4.1 *Site Investigation and Assessment, and Design*

(i) Remediations involving concrete cut-offs are conducted in existing dams about which there is typically a large amount of historical information stretching back to the conceptual design and the original site investigation and through the subsequent design and construction phases. These data can be invaluable in the creation of the geological model upon which the remediation must be logically and responsively designed. In addition, this model will provide a guide to the extent of any further site investigation which ought to be conducted prior to final design. This desk study should include an analysis of any and all previous drilling and grouting records which may still exist. An excellent example of the nature of such a model is provided in Spencer (2006), in preparation for the new phase of remediation at Wolf Creek Dam, KY.

(ii) The new site investigation must establish the in situ properties of the embankment and the rock mass, and confirm the provisional geological model. In particular, it must provide clear guidance to bidders on the potential for sudden loss of slurry and any other difficulties which can be anticipated during excavation. In this regard, the in situ strength of the rock mass and its structure are very important guides to rippability, and hence the choice of equipment and its likely productivity and operating costs.

(iii) The entirety of the site investigation data — historical and new — must then be used to establish the location, and lateral and vertical extent of the cut-off. A concrete cut-off is essential in horizons which contain significant amounts of potentially erodible material, e.g., epikarst and discrete, heavily karstified zones at depth. In other areas where fissures and features do not contain such potentially erodible materials, a hydraulic cut-off can be effectively and reliably constructed using state-of-practice drilling and grouting methods at a fraction of the cost of a concrete cut-off. This concept of a "positive cut-off" emplaced through erodibles, flanked and/or underlain by a grout curtain, has been termed a "composite cut-off" (Bruce and Dreese, 2008).

(iv) At this juncture, the adequacy of the existing dam instrumentation must also be assessed, and the need for new instrumentation identified. Protocols should be established for regular readings and observations consistent with the location and progress of the remediation relative to the key performance indicators such as piezometric levels, settlements, seeps, turbidity, sinkholes, and wet spots.

(v) Similarly, the designer must arrive at a conclusion on the need for a systematic pretreatment by grouting along the proposed alignment of the cut-off. Such pretreatments may well prove to be costly and relatively time consuming. However, the downside of not conducting this work must be carefully balanced in such situations: in heavily karstified terrains, pretreatment is invariably a most cost-effective risk management policy.

3.4.2 *Preparation of Contract Documents, and Contractor Procurement Methods*

(i) The Specification should be of the Performance type (and not Prescriptive) to best encourage the commitment and interest of the bidders. However, these documents must also be very clear as to what is not acceptable in terms of construction techniques or equipment, and what the minimum acceptable measures of success of the project are. These measures most typically relate to the integrity, continuity, extent and strength of the wall. In certain projects, it would be conceivable to set overall performance goals for the cut-off such as a reduction in seepage, or a specified difference in piezometric levels. In any event, clarity is essential.

(ii) The procurement process must feature "best value" concepts, as noted in Section 3.3.1 above. This will value the technical capabilities of the bidders above their ability to calculate a low price, and will typically involve the submittal of very detailed technical proposals which will be carefully evaluated by the Owner's Technical Proposal Evaluation Committee (TPEC). It will always be the case, however, that the best qualified technical proposer must also be in the financially "responsive range" previously established by the Owner.

(iii) Regarding the scope of the project, pregrouting and other facilitating/miscellaneous tasks (such as undertaking office extensions, and utility relocations) should be separated from the cut-off wall contract itself. However, the preparation of the working platform should be included in the specialty contractor's scope since it is completely integral to the efficiency of his work.

(iv) The Specifications must be clear and unambiguous regarding the QA/QC and verification goals, but not necessarily the exact processes to be employed. The document must also specify clearly the nature, number and qualifications of the key personnel who <u>will</u> be present on site, or located principally at the contractor's head office.

(v) As a minimum, Partnering must be mandated. However, on especially difficult or otherwise challenging projects, the new concept of a full "Alliance" should be considered (Carter and Bruce, 2005). This has the potential to eliminate financial and/or contractual disputes and has been found to be an extremely valuable and pragmatic approach to dam remediation (Amos et al., 2007).

3.4.3 *Aspects of Pretreatment by Grouting*

The need for pretreatment is preferably identified prior to wall construction, and not found as a surprise during the Test Panel phase. The following points of guidance are provided with regard to pretreatment by grouting:

- If flush water has been lost during investigatory drilling, slurry will certainly be lost during wall excavation, without pretreatment in those same areas.
- The minimum treatment intensity features two rows of inclined holes, one either side of the subsequent wall location. The rows may be 5 to 10 feet apart, and the holes in each row will typically close at 5- to 10-foot centers. The inclination (15º off vertical) will be different in each row.
- The curtain should be installed to at least 50 feet below and beyond the designed extent of the cut-off to assure adequate coverage and to search for unanticipated problems. Pretreatment must be regarded as an investigatory tool equally as much as a ground treatment operation.
- "Measurement While Drilling" principles are to be used, the philosophy being that every hole drilled in the formation (not just cored investigations) is a source of valuable geotechnical information (Bruce and Davis, 2005).
- Special attention must be paid to the epikarstic horizon, which will typically require special grouting methods such as MPSP (Bruce and Gallavresi, 1988) or descending stages.
- A test section at least 100 feet long should be conducted and verified to allow finalization of the Method Statement for the balance of the pretreatment work. A residual permeability of 10 Lugeons or less should be sought. Conversely, a falling head test in vertical verification holes, using bentonite slurry, is an appropriate test. Verification holes should be cored, and observed in situ with a televiewer to demonstrate the thoroughness of the pretreatment.
- In terms of the details of execution, the principles detailed by Wilson and Dreese (2003) to create Quantitatively Engineered Grout Curtains should be adopted. Thus, one can anticipate stage water tests, balanced, modified, stable grouts, and computer collection, analysis and display of injection data. When drilling the verification holes (at 25-foot centers between the two grout rows), particular care must be taken to assure that no drill rods have to be abandoned in the alignment of the wall since this steel will adversely impact subsequent wall excavation techniques.

3.4.4 *Construction*

(i) The work must be conducted in accordance with the Contractor's detailed Method Statement which, in turn, must be in compliance with the minimum requirements of the Specification unless otherwise modified during the bidding and negotiation process. At the same time, modifications to the foreseen means and methods can be anticipated on every project, in response to unanticipated phenomena. Prompt attention to, and resolution of, these challenges are essential.

(ii) As noted above, special attention is merited to the details of the design and construction of the working platform. The Contractor's site support facilities (e.g., workshop, slurry storage and cleaning, concrete operations) can be completed and the utilities extended along the alignment (water, air, light, slurry) during the building of the work platform.

(iii) The Test Section should be established in a structurally non-critical area, which does not contain the deepest extent of the foreseen wall. The Test Section can, however, be integrated into the final works if it is proved to have acceptable quality.

(iv) The excavation equipment must have adequate redundancy, and must be supported by appropriate repair/maintenance facilities. A variety of equipment is usually necessary (clamshell, hydromill, chisels, backhoe) to best respond to variable site conditions and construction sequences. Standard mechanical features, such as the autofeed facility on hydromills, must not be disabled in an attempt to enhance productivity.

(v) The site laboratory must be capable of conducting accurately and quickly the whole range of tests required. In addition, the Contractor's Technical/Quality Manager, who is a vital component in any such project, must be fully conversant with all the principles and details involved in the monitoring of the construction, and of the dam itself. In particular, expertise with panel or pile verticality and continuity measurement is essential.

(vi) Emergency Response Plans must be established to satisfy any event which may compromise dam safety.

3.4.5 *Assessment of Cut-Off Effectiveness*

The protocols established for observations and instrument readings during remediation must be extended after remediation although usually at a somewhat reduced frequency. The data must be studied and rationalized in real time so that the remediation can be verified as having met the design intent. Alternatively, it may become apparent that further work is necessary, a requirement that becomes clear only when the impact of the remediation of the dam/foundation system is fully understood. Finally, Owners and Designers should publish the results of these longer-term observations so that their peers elsewhere can be well briefed prior to engaging in their own programs of similar scope and complexity.

4. Final Remarks

The decision to conduct these two research programs arose from the observation that, as the years go by and key personnel retire, records of past projects become progressively more difficult to locate. This unfortunate fact of life is highly ironic in our electronic age, at a time that archiving of material has never been easier or more convenient to carry out. Invaluable information on past projects has, however, simply vanished, within the space of one or two generations of engineers.

These programs will continue, with the prime goal being to fill in the historical "gaps," especially in the field of rock anchors prior to 1990. It is of value to all in the dam rehabilitation business to have access to such studies and databases. Designers will be able to draw conclusions on project feasibility and reasonable performance expectations, and will be able to obtain details of the design processes themselves. Contractors will draw guidance on special "tricks of the trade" which have been developed, often at considerable corporate cost, over the years. Owners will be able to draw comfort from a weight of successful precedent practices and will win a fair estimate of the time, cost and disruption likely to be involved with such remediations.

There is an excellent but relatively small pool of specialty contractors in North America who undertake the type of works described herein. Given the volume of dam remediation anticipated over the next few years, it is clear that the resources (both human and mechanical) of these contractors will be severely strained. Furthermore, many of these companies are foreign-owned and the parents are also extremely busy in other parts of the world. The author hopes that, at this critical time, studies such as these national research programs will help industry meet the challenges by facilitating information exchange.

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