

EVOLUTION OF ROCK ANCHOR PRACTICE OVER THREE DECADES

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Abstract

In the absence of a formal national standard, the Recommendations of the Post Tensioning Institute (1996) have served the dam industry well regarding the design, construction, testing, and performance of rock anchors. These Recommendations are being updated again for re-issue in 2004, with special emphasis being devoted to the results of recent researches into issues related to epoxy protected strand. Recently, the author has been involved in the performance assessment of high capacity anchors installed in a dam in the Pacific Northwest in 1975. Very comprehensive contemporary records exist from the project, involving multiwire button head tendons. These records clearly illustrate the “State of Practice” as it was for anchors at the time. This paper compares and contrasts these historical data with contemporary practices, illustrating the evolution of certain aspects of dam anchor practice in the U.S. over three decades.

1. Introduction

The history of prestressed rock anchors as a remedial tool to stabilize existing concrete dams in the U.S. extends back to the early 1970s (Bruce, 1989, 1993). In recent years, it is estimated (ADSC, 2002-2003) that between 10 and 15 dams are remediated in this fashion annually, with about one third using epoxy coated strand as the tendon material. This represents an intensity of effort unmatched in any other country, both for quantity and duration: it is more typical to find that a certain country experiences a relatively short-lived phase of dam anchoring on groups of dams, determined by geography, age, and/or design. In this regard, the attention now being focused on the dams operated by Hydro Tasmania is a typical example.

Practice on U.S. dams has understandably evolved over the last 30 years, in response to changes in equipment, materials, and design concepts and philosophies, primarily those relating to corrosion and corrosion protection. In addition, the trend towards specifying progressively higher capacity tendons (a current project in British Columbia has tendons comprising 93 number 0.6-inch diameter strands) has further caused contractors to revise their methodologies in order to satisfy the considerable logistical challenges the handling, installation, grouting, and stressing of such massive tendons pose. The author was recently involved in the reevaluation of 142 anchors installed in 1975 in a major dam in the Pacific Northwest. This study involved not only a physical inspection of the heads of four of the anchors, but afforded the opportunity to appraise the specification, and study the comprehensive as-built construction records. The observations contained in this paper reflect upon various aspects of 1970s rock anchor practice as generalized by Littlejohn and Bruce (1977) and typified by this project, and current practice, which tends to follow closely the existing Recommendations of the Post Tensioning Institute (1996), which will be further enhanced in the upcoming 2004 edition. As is described below, the rate and extent of progress over the last 30 years has not been uniform in all aspects of the technology.

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2. Scope of 1975 Anchor Project

A total of 142 vertical anchors were installed to resist sliding with total lengths ranging from 55 to 168 feet. “Rock embedment” ranged from 27 to 82 feet with first stage grouting lengths (i.e., bond lengths) ranging from 11 to 61 feet. The “effective prestressing force” per anchor varied from 205 to 1490 kips, each anchor comprising multiwire tendons of 240-ksi wire of 11.78 kips per wire Guaranteed Ultimate Tensile Strength. In four anchors, “minitendons” were formed by encapsulating the free lengths of four adjacent wires in a debonding sleeve to allow periodic lift off testing of these wires during service life. This, in itself, was a novel and thoughtful feature.

3. Geotechnical Design Aspects

Then

In terms of the overall prestressing load requirement to resist sliding, the design was classified recently as “exceedingly conservative”, given the anticipated loading conditions, the parameters selected for the rock/concrete contact, and the magnitude of the factors of safety employed. Indeed, the recent reappraisal indicated that the number of anchors actually required using current FERC guidelines would now be considerably less. The key issue of rock-ground bond typified contemporary practice: bond distribution was considered to be uniform and the working bond magnitude (in generally hard volcanics) was conservative (20 to 35 kips/ft or about 100 to 130 psi). Bond lengths and diameters (4 to 8¼ inches) were called out in the anchor schedule, based on these average bond values. Values of 150 to 200 psi for working bond were common in U.S. practice at the time (Littlejohn and Bruce, 1977) for such rock types. Although it is not specifically described, the question of overall stability design is interesting to speculate upon. It is known that the state of practice was to assume that a volume of rock would be engaged by each anchor, being a cone of included angle 60° to 90° and with the apex located at some point along the bond zone, usually the mid point. The uplift capacity was thus the submerged weight of rock in this cone (or wedge of overlapping cones for closely spaced anchors).

Now

Overall anchor load requirements are calculated in accordance with appropriate statutory requirements and using contemporary computational methods and aids. However, given the “consequences of failure” facing a dam remediation program, it is typical that designs remain conservative, if not as excessively so in former years. Regarding rock-grout bond, a plethora of data (Barley and Windsor, 2000) confirms that bond is not evenly distributed and that designs so based provide inefficient load transfer conditions. Nevertheless, despite (or perhaps because of) the great mass of field-generated data, the “uniform load” assumption remains standard practice, and typical bond values, as listed in PTI (1996) remain of the same order of magnitude as those used in 1975 (i.e., ultimate values of 250 to 450 psi). Likewise, the very simple and simplistic “weight of rock in a cone/wedge” remains the typical basis for overall stability design: very few designers consider shear strength contributions, and fewer still employ the more sophisticated equations developed in the 1970s by Czech and German rock mechanics engineers.

4. Construction

Then

Drilling. Diamond drilling was specified through the concrete, whereas rotary or rotary percussive techniques were permitted in the rock. Certain anchor holes were to be fully cored and then

reamed out to final diameter. Hole alignment was to be monitored to prevent total deviation being larger than 1 in 100. Pressure grouting was permitted to combat unstable ground conditions. A “sump”, 18 to 24 inches deep, was to be allowed for at the bottom of each hole to collect any debris which could not otherwise be evacuated by the drill flush. Full and accurate geologic drill logs were to be maintained of all the major lithological and structural variations in the rock mass.

Water Pressure Testing. The full length of every hole in rock was to be tested, the acceptance criterion being a loss of 0.5-gpm at 60 psi. Failure would result in pressure grouting, re-drilling, and retesting. More typical of U.S. practice at the time was a criterion of 0.001 gal/inch diameter/ft/min at an excess pressure of 5 psi, highlighting the very conservative nature of the project’s specifications.

Grouting. A proprietary, presumably non-shrink, high strength pre-blended grout was specified for the first stage, and a Type II grout with non-shrink additive for the second stage. Stressing could commence when the grout reached a strength in excess of 3000 psi. The water/cement ratio was limited to 0.40 to 0.45 with the proviso of “minimal shrinkage”. Pre-construction testing of grout mixes was to be conducted to demonstrate the grouts’ acceptability. The mixer had to be able to measure (accurately) the grout volume, while it was also specified as having to be a high speed, high shear mixer, coupled to a paddle agitated storage tank. Pumping was to be via a Moyno pump. Good grout return at the top of the anchor hole during secondary grouting (with the tremie method) was also required to ensure full and thorough secondary grouting.

Tendon. The contractor was permitted to select the tendon type, in the case of the higher capacity anchors, the choice being strand or wire. The wire was specified to ASTM contemporary standards, and had to perform within reasonable, safe, stressing levels (i.e., at no time being loaded above 80% Guaranteed Ultimate Tensile Strength). No provisions were made for corrosion protection directly placed on or around the tendon, other than the grout itself.

The Specification reflected both the open mindedness of its drafters (in allowing choice of tendon type) and the typical contemporary standards with respect to attitudes towards corrosion protection. In effect, it was tacitly assumed that an efficiently water proofed and grouted anchor in a rock or concrete mass which is naturally (or is rendered) virtually impermeable, without aggressive ground water, will not experience significant corrosion in the hole. This was generally supported by the FIP study (1986) which showed, inter al., that under such advantageous conditions, corrosion was a possibility only at, or under, the head. In the case of fully bonded tendons, as here, this is not a threat to the anchor’s long term performance after final lock off.

Now

Drilling. Except in rare cases where there is a risk of significant embedded steel in the concrete, or where the dam’s “hearting” could be extremely sensitive and voided, coring techniques are not employed as a regular construction tool. Rotary percussion (by down-the-hole (DTH) hammer) is most common, being selected for its cost, speed, and deviation control advantages. Studies have indicated that the vibrations induced by rotary percussion have minimal impact on the structure being penetrated (Bianchi and Bruce, 1993). It is also known that coring rock produces a smooth borehole surface which can inhibit subsequent rock-grout bond development. Contemporary DTH drilling permits holes of up to 15-inch diameter to be drilled to over 300 feet deep with deviations of less than 1 in 150 or better with relatively standard equipment. Sumps are typically 3 to 5 feet deep, and MWD (Measurement While Drilling) recording is common as an indirect guide to rock quality conditions (Bruce, 2003).

Water Pressure Testing. Current practices reflect the knowledge that the permissible water loss calculation should be independent of hole diameter or length, since the critical fissure ($\geq 160 \mu$ wide) can exist anywhere along the hole and flow is not diameter-driven. The 1975 project-specific criterion was very severe, but reflected the fact that grout was the only level of corrosion protection on the steel tendon. The current criterion of 10 gallons in 10 minutes at an excess pressure of 5 psi is much less

onerous, but, for anchors longer than about 80 feet, remains the most conservative of all the international standards (Figure 1). This talks of higher quality in U.S. practice than elsewhere.

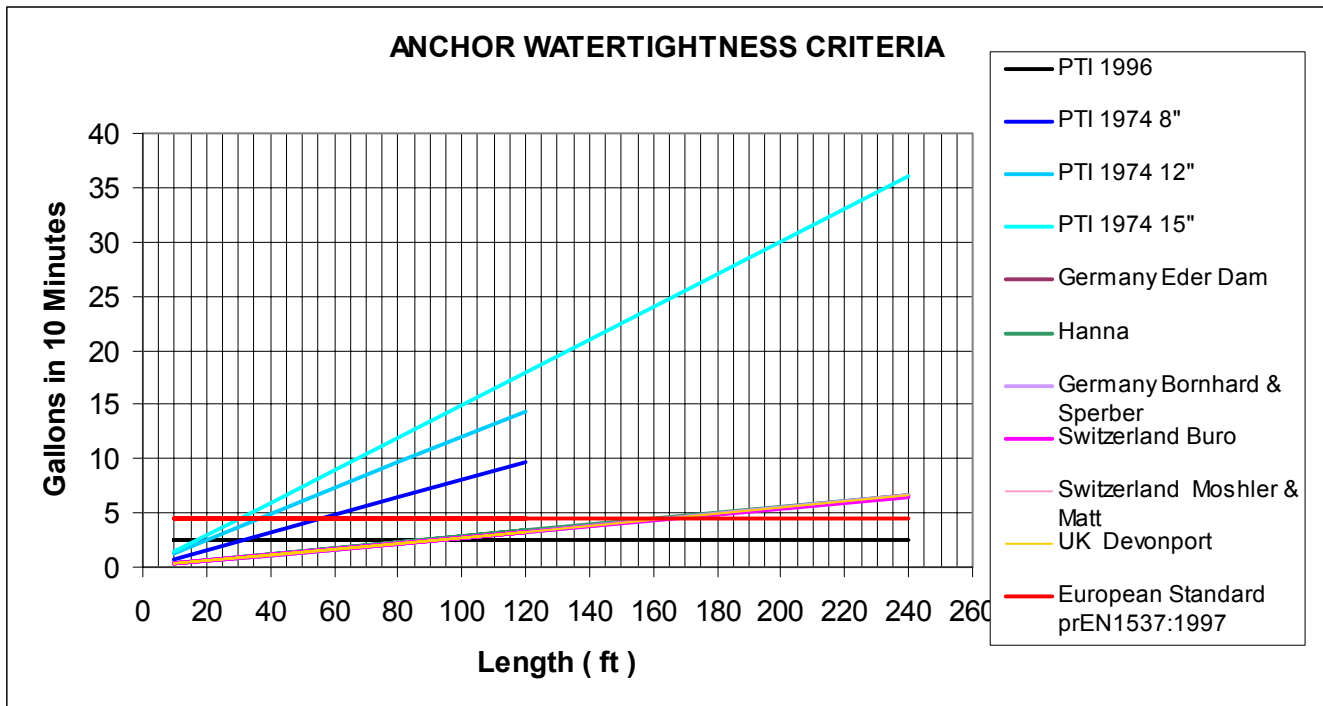


Figure 1. International criteria for water tightness of anchors.

Grouting. Again, in all critical aspects the 1975 specification is excellent and is surprisingly close to current practice. For example, the use of high shear mixers to mix grouts of low water/cement ratio would ensure homogeneous, pumpable, stable grout of high strength and low permeability and so superior durability in situ. The requirement for pre-construction site mix testing is also illustrative both of good practice, and common sense. Furthermore, the decision to use two stage grouting would automatically a) permit the bond zone to be load tested in isolation and b) provide bond to the tendon in the free length (after stressing) thus ensuring that load could not subsequently be lost due to mechanical failure of the top anchorage (i.e., in this case a button head system).

Today, proprietary premixed, preblends are neither necessary nor cost effective. Our advanced knowledge of cement grout rheology, as impacted by admixtures, provides adequate solutions to the problems of stability, pumpability, and durability. Use of Type III cements permits stressing within 3 to 4 days, while ultimate grout strengths in excess of 6000 psi are typical of low water:cement ratio mixes. This can permit higher average grout/rock bond values to be assumed in the design process.

Tendon. The tendon composition (invariably 7-wire strand to appropriate ASTM standards) is specified and the Contractor must provide details of the tendon geometry (i.e., spacers, centralizers, grout tubes, etc.), as well as his plans for shipping, handling, and installing the tendon to minimize damage. Wire tendons are not used while only low capacity anchors (say ≤ 100 kips) of moderate length (say ≤ 50 feet) can be logistically or economically satisfied with bar tendons. The most significant developments in tendon design, however, relate to corrosion protection, and contemporary protection levels for permanent dam anchors are Class 1, as summarized in PTI (1996) Table 5.1.

This superior level of corrosion protection is justified based on service life, consequences of failure, and incremental in place costs, especially. Practice in this regard has improved dramatically in the last 10 years or so.

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 5.1 Corrosion protection requirements (PTI, 1996).

5. Stressing and Testing

Then

The stressing was to be conducted in progressive steps corresponding to 40, 60, 80 and 100% of the “maximum jacking force” (which would, however, be less than or equal to 80% of the tendon Guaranteed Ultimate Tensile Strength). Strict criteria were placed on elastic extension (within 5% of the theoretical) and permanent movement (1 inch allowed). The tendon was to be locked off at the “desired effective prestressed load” or 70% GUTS, whichever was greater. A lift-off test would then be conducted (prior to secondary grouting) no earlier than 2 days after successful initial stressing, with a 5% load variance criterion. In general, the stressing provisions are, by contemporary standards, the weakest part of the specifications in terms of the steps to be taken in reaching the maximum load. However, relatively strict acceptance criteria were applied to the total movement (it would have been impossible to arithmetically separate permanent from elastic movement components without progressive cyclic loading, equivalent to PTI’s “Performance Test” sequence), and on the transfer load retention efficiency after 48 hours. Given the range of tendon types permitted (reflecting the input of the various post tensioning companies who had likely contributed to the specification’s content), the stressing criteria were probably a reasonable consensus of the different methods. In any case, the stressing data, though quite rudimentary, would upon analysis provide a quite sensitive and accurate picture of each anchor’s performance.

Long term performance was to be gauged by “minitendons” within 4 of the anchors, subjected to periodic lift off testing (which would, of course to a certain degree upset the top anchor corrosion protection system).

Now

The 1996 Recommendations corrected certain misconceptions that had worked their way through previous successive editions. In particular, clear distinction was drawn about the measurement and analysis of elastic movements, as opposed to total movements. This is possible if progressive cyclic

loading is used, as in the Performance Test (Figures 8.1a and b), which is conducted on pre-production (“disposable”) anchors and/or on a limited number of production anchors. The use of strand tendons, and a multi-part wedge system of load retention, facilitates such multiphase stressing, which would have been very awkward with the wire tendons and their “button head” top anchor system. Likewise, the hydraulic jacking systems now in use greatly speed the execution of such tests and improve the accuracy of load application. Long term performance is now monitored (if debonded free lengths are provided) via load cell and/or total head lift off testing provided access can be maintained to the head without compromising corrosion protection.

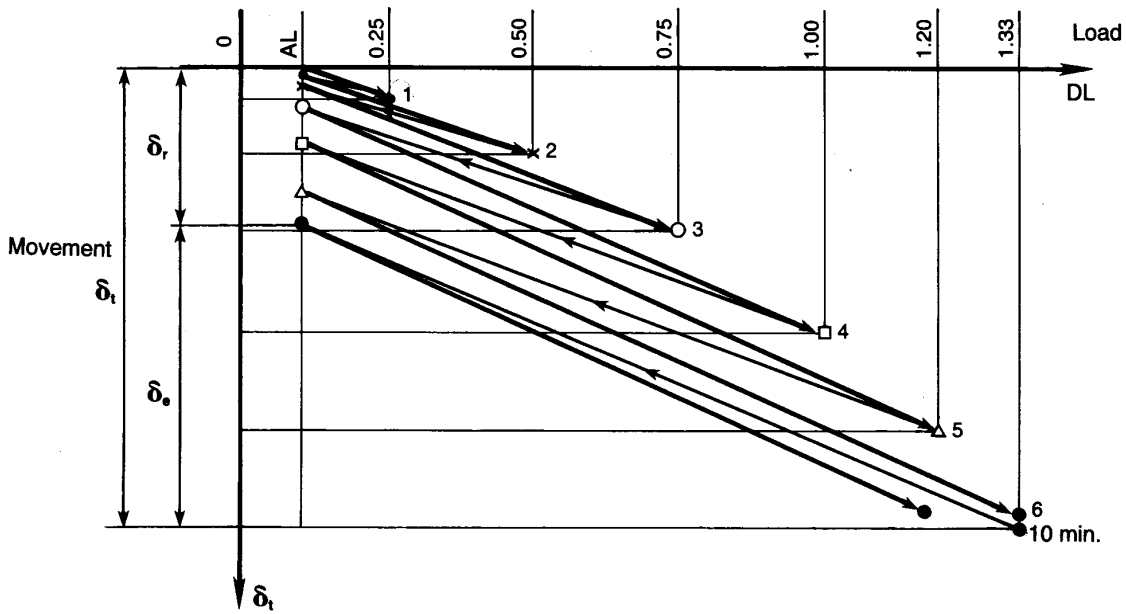


Figure 8.1a. Plotting of Performance Test data.

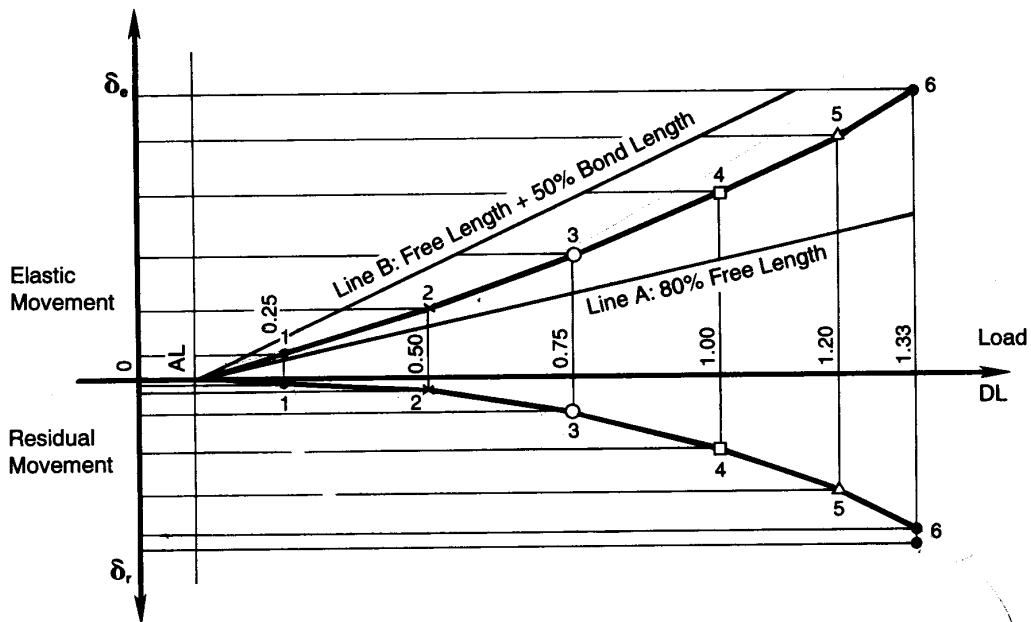


Figure 8.1b. Graphical analysis of Performance Test data.

6. As-Built Anchor Records

Then

Two separate sets of drawings summarized the salient construction data for every anchor, including

- Geological log (for each anchor position) as based on the core results.
- Hole inclination, depth, and diameter.
- Elevations of grout placed, and number of bags injected.
- Water pressure test data.
- Grout rate of gain of strength (Primary and Secondary stages), and similar concrete data.
- Load-extension curve, plus lift-off data.
- Dates for each construction step.

It may be observed that every hole met the drill deviation tolerance and that every stage in rock was water pressure tested to and passed at the 60 psi excess pressure, with the packer placed in the concrete, near its base.

The records are very comprehensive and show that the work was conducted in strict accordance with the Specification. In particular, every hole, prior to tendon insertion and grouting, had a very low permeability (typically less than 0.3 Lugeon) and no hole appeared to show unusual grout takes, also indicative of tight rock conditions. Every tendon provided an extremely linear load-movement performance, indicative of minimal debonding, and so proved considerable bond capacity in excess of that actually mobilized during testing. No anomalies were noted during lift-off to suggest any tendency for creep to have occurred (indeed, the nature of the rock mass would also argue against even the possibility of creep being allowed to occur).

Now

The PTI document requires neat, legible and “suitable for reproduction” records to be provided comprising

- As-built drawings.
- Materials certifications.
- Drilling and grouting records, water testing, grout mix design, laboratory tests on grout cubes.
- Anchor test and monitoring results and corresponding graphs.

These are now computer generated and maintained, supplemented by digital progress photographs. However, it is clear the contents of such reports have not changed over 30 years (except perhaps for details regarding corrosion protection).

7. Overview

This brief review highlights that while progress in the technology has most certainly been made over the last 30 years, it has not been at a constant rate across all the various aspects of the technique. Developments in equipment, technique, and materials have permitted engineers to design increasingly higher capacity, longer anchors. However, the basis for the key elements of their designs remains largely unaltered, with particular respect to overall stability and bond stress magnitude and distribution philosophies. Better understanding of rock and concrete properties, and of the nature of the loads

imposed on structures has – at best – allowed designs to refine from “exceedingly conservative” to merely “overconservative”.

In contrast, the skills of equipment and materials manufacturers and of contractors have been honed on the stones of experience, expedience, and competition, to the extent that the industry can satisfy economically the logistical challenges posed by designers. Of particular relevance are advances in drilling capabilities, understanding of rock mass permeability issues, and developments in the assembly, handling, installation, protection, and grouting of multistrand tendons. In particular, major and significant philosophical changes in attitudes to corrosion protection have been enacted, particularly within the last decade.

A similar picture can be painted of the stressing and testing aspect of the technology wherein the development of high capacity, long extension hydraulic jacks complements the development of multiwire strand systems, and enables sophisticated loading and analysis routines to be conducted, thereby enhancing quality and reliability. Quality assurance is also reflected in the routine maintenance of thorough construction records. The quality of the (manual) records for the 1975 project described herein was astounding: it may be speculated, however, that it was exceptional, not standard, and reflected great credit on all the parties concerned.

Such historical perspectives can have extreme value as well as interest, since it is vital to understand the path of technological evolution in order to predict the course of future needs and development. Throughout the country in dusty boxes in abandoned store rooms there remain the records of dozens of anchor projects covering 30 years of anchor construction. Contemporary MIS techniques assure that it has never been easier or quicker to permanently, electronically archive such data, before they are thrown out, lost, or otherwise destroyed. This is a national need and an unfulfilled initiative. It requires dedicated, funded resources to accomplish it. It must be done before the generation that built the projects is lost to their future service. There is a wealth of data to be collected, archived, and analyzed. The task would benefit future technology and create justified international recognition for the contributions of U.S. engineers to its evolution for over three decades.

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