ENCLOSURE 2

TECHNICAL EVALUATION REPORT

TENDON EVALUATION

ROCHESTER GAS AND ELECTRIC COMPANY
R. E. GINNA NUCLEAR POWER STATION

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FOREWORD

This Technical Evaluation Report was prepared by Franklin Research Center under a contract with the U.S. Nuclear Regulatory Commission (Office of Nuclear Reactor Regulation, Division of Operating Reactors) for technical assistance in support of NRC operating reactor licensing actions. The technical evaluation was conducted in accordance with criteria established by the NRC.

1. INTRODUCTION

1.1 PURPOSE

Under an assignment from the U.S. Nuclear Regulatory Commission (NRC) [1], the Franklin Research Center (FRC) reviewed documentation provided by the NRC pertaining to the unexpected level of continued prestress force relaxation exhibited by the containment wall tendons of the R. E. Ginna Nuclear Power Plant. In this report, FRC and its subcontractor, Geotechnical Engineers Inc., present an evaluation of the Licensee's investigation and the Licensee's conclusions regarding the tendon prestress loss.

1.2 BACKGROUND

The following description of the containment wall and tendons was supplied by the Licensee [2]:

> "The Robert E. Ginna Station was the first prestressed concrete containment constructed in the United States. It utilizes only vertical prestressing with non-prestressed reinforcement being used to resist forces in the hoop direction and in the containment dome. The vertical prestressing is applied through 160 vertical tendons, consisting of 90 wires (1/4 inch diameter) each conforming to ASTM A421-65, Type BA. Each wall tendon is connected at the containment base to another 90 wire tendon which is anchored by grouting into a 6 inch diameter hole drilled into the base rock. These rock anchors are also prestressed. The prestressing of the rock anchors was done prior to construction of the containment. The prestressed containment wall was placed on an elastomeric pad to provide a hinge at its base."

Prestressing of the rock anchors was performed in the fall of 1966 prior to the pouring of concrete for the containment walls. Stressing of the wall tendons was performed in the spring of 1969. These and subsequent surveillance and maintenance activities are listed in Table 1.

Following installation and stressing of the wall tendons, 23 of the 160 tendons were retensioned at the 1000-hour point. By the time of the 8-year surveillance in 1977, the average force of the tendons had decreased to a value marginally above the design requirement of 636 kips. At this point, the Licensee decided to retension the 137 tendons not retensioned in 1969. This signaled the beginning of a more intensive study by the Licensee and its

Table 1. Wall Tendon Surveillance Activities

Year	Activity
1966	Rock anchor installation and prestressing
1967	Containment wall concrete placement
1969	March, wall tendons stressed April, structural integrity test May, 1000-hr retensioning of 23 tendons Oct., 6-month surveillance of tendon forces
1970	1-year surveillance of tendon forces
1972	3-year surveillance of tendon forces
1977	8-year surveillance of tendon forces
1979	10-year retest
1980	11-year retensioning of 137 tendons
1981	Surveillance of tendon forces
1983	Surveillance of tendon forces

contractors to determine the cause of the tendon relaxation and to initiate corrective action. The Licensee has subsequently considered rock anchor creep and bond slippage, containment wall shrinkage and creep, force measurement system error, temperature effects, and stress relaxation of the tendon wires.

Continued investigation of tendon relaxation by the Licensee included an experimental study of tendon wire stress relaxation, creep phenomena, using spare wires from the Ginna wall tendons. This effort was carried out by the Fritz Engineering Laboratory of Lehigh University, and was extended to include the effects of tendon retensioning when the study indicated that creep of the tendon wire under the concrete prestressing load was greater than the values anticipated in the containment building design.

FRC engaged Geotechnical Engineers Inc. under a subcontract to assist with the review of the rock anchors.

FRC and its subcontractor have reviewed the Licensee's reports provided by the NRC in an effort to determine if tendon creep is the principal cause of the tendon force relaxation or if the presence of tendon creep obscures any possible problems of bond slippage and/or creep in the rock anchors which would yield similar tendon force relaxations.

2. ACCEPTANCE CRITERIA

The following criteria and guidelines were used to determine the adequacy of the tendon installation and surveillance program:

- Robert E. Ginna Nuclear Power Plant Final Safety Analysis Report [3]

- American Concrete Institute, ACI 318-63 [4], Building Code Requirements for Reinforced Concrete

  This is the standard under which the containment wall tendons were designed, installed, and stressed.

- Ginna Technical Specification, Section 4.4.4.2, Criterion for Lift-off Forces [5]

- ASME Boiler and Pressure Vessel Code, Section III, Division 2, Subsection CC, 1983, Concrete Containments [6]

  This code is a joint code combining the ASME and ACI Code requirements for concrete containments.

- Regulatory Guide 1.35, "Determining Prestressing for Inspection of Prestressed Concrete Containments," U.S. Nuclear Regulatory Commission [7]

- Standard Review Plan 3.81, Concrete Containment, U.S. Nuclear Regulatory Commission [8].

3. TECHN CAL EVALUATION

3.1 TECHNICAL OVERVIEW

Section 5.1.2 of the Ginna FSAR [3] presents the following description of the tendons and their levels of loading:

"The prestressing system used for the containment vessel is the BBRV system utilizing 90-1/4 inch diameter wires. The wires are high tensile steel bright, cold drawn and stress relieved conforming to ASTM A421-59T, Type BA "Specifications for Uncoated Stress-Relieved Wire for Prestressed Concrete" with a minimum guaranteed ultimate strength of 240,000 psi. The BBRV system uses parallel wires with cold formed button heads at the ends which bear upon a perforated steel anchor head, thus providing a mechanical means for transferring the prestress force. The buttonheads are formed by cold upsetting to a nominal diameter of 3/8 inch on the 1/4 inch diameter wire.

The effective prestress forces are developed in all tendons in accordance with normal industry practice. All tendons will be initially tensioned to 80% of ultimate stress and then locked-off at 70% of ultimate stress. Basically all tendons are straight. A limited number have a minor curvature where they are draped around small penetrations. The tendons in all cases are located in a relatively large (6 inch diameter) rigid conduit sized to permit the bottom anchor head to pass through. Any wobble and friction losses will be less than 24,000 psi which is 10% of the ultimate stress. The remaining losses consist of elastic shortening, concrete shrinkage and creep, creep of the elastomer pads and steel relaxation. Anchorage losses are negligible for the length of tendon being used."

The Licensee's reports defined all tendon loads with respect to the guaranteed ultimate tensile stress (GUTS). In accordance with the FSAR statement above, GUTS is 240,000 psi.

Based on the tendon and loadings, the following data were tabulated for convenient reference:

Wire Data

Wire diameter, 0.25 inch
Wire area, 0.0490 in^2
GUTS, 240,000 psi
0.8 GUTS, 192,000 psi
0.7 GUTS, 168,000 psi

Tendons (90 Wires) Data

Area, 4.418 in^2
Force at 0.8 GUTS, 848 kips
Force at 0.7 GUTS, 742 kips.

The Licensee's acceptance criteria stipulate that the minimum lift-off force be as follows [9]:

> "Ginna Technical Specification, Section 4.4.4.2 [Ref. 5] provides the acceptance criterion for the lift off forces. The criterion requires that the average stress of the sample tendons not be less than 144,000 psi, which is equivalent to 636 kips. The 636 kip value represents the minimum required average tendon force for the tendons."

In reviewing the Licensee's reports (see References, Section 6), the following topics were selected for review and evaluation:

- Lift-off force measurement
- Creep of concrete
- Creep of elastomeric pads
- Wall tendon stress relaxation
- Rock anchors.

The topics were reviewed by FRC, assisted by Geotechnical Engineers Inc. for the review of rock anchors. The review and evaluation are presented in the following sections. The review of the rock anchors submitted by Geotechnical Engineers Inc. is included as Appendix A.

3.2 LIFT-OFF FORCE MEASUREMENT

In accordance with the FSAR [3], stressing of the tendons was to have been accomplished with hydraulic jacks and pumping units equipped with dial gauges of "not less than 6 inches in diameter," and "which indicates the pressure in the system within plus or minus two percent." However, during the reported surveillance procedures listed in Table 1, there was concern over the accuracy of the hydraulic pressure gauges. Therefore, the stressing jack

tension link was instrumented with strain gauges [10] to rule out possible hydraulic system effects and to improve the accuracy of pressure gauge readings. An instrumented stressing ram tension link was introduced to verify that the stressing ram pressure gauge provided force measurements within ±3% of the load measures by the calibrated strain-gauged stressing ram (±18 kips at a 600-kip load) [10].

Accuracy of the force measurements was not fully documented in the reports reviewed for lift-off measurements prior to 1981. The stressing ram owned by Rochester Gas and Electric was stated to have been used from 1969 through the 8-year surveillance, after which it was recalibrated and used for the 10-year surveillance in 1979 [10]. Two stressing rams were used by Inryco, Inc. for the 1980 retensioning program. Reference 4 indicates that these rams were calibrated before and after these tests and that the ram areas of the two jacks differed "by 1.5% at worst."

In summary, the attention to stressing ram calibration and the accuracy of force measurement improved as surveillance continued. The cross calibration technique discussed for the 1983 surveillance is good [9], wherein the calibrated stressing ram tension link (calibration traceable to U.S. Bureau of Standards) was used to calibrate the load cells and ram pressure gauges [9].

Determination of the point of lift-off was also a problem. In earlier lift-off tests, an acoustical method was used in which the shim was tapped, and the resulting sound was used to determine if the load was removed [10]. In October 1979, a new method was introduced. The new procedure required lift-off to proceed until two 1/32-inch feeler shims (one on either side) could be inserted into the shim stack. The load was then set back on the shims and increasing pressure was applied on the stressing jack until the two feeler shims could be removed, thus defining lift-off [10]. Evaluation of these procedures indicated that the feeler shim method provided greater consistency (and accuracy) as long as the change in load (and pressure) was greater than the hysteresis band.

No methods for measuring tendon elongation and ram extension were described, but it is believed from this review that greater accuracy would be beneficial. While accuracy of tendon force is of primary interest, there are additional benefits of accurate measurements of tendon extension. Since the straight wall tendons have low friction in the tendon sleeves, and since it was reported that short-term load changes such as the 6% load increment above lift-off did not produce instant creep (see Reference 10, Section 3.2.3), then the division of the load increment by the associated tendon extension, or ram displacement, could possibly be used to investigate the state of the anchor tendon wire/grout bond. However, values from the 10-year surveillance [11] and the 1983 surveillance [9] were not sufficiently accurate or consistent for this investigation. The load increments were too small for the accuracy of extension measurements. Improvement of tendon extension accuracy to the current level of force measurement accuracy could possibly enable the recording of data needed to investigate the relative participation of the rock anchor tendon.

The increased sophistication of surveillance instrumentation enabled the Licensee to explain one set of troubling variations. Monitoring of temperature and tendon force from August 1981 to July 1982 identified a seasonal variation in tendon force superimposed upon continued creep [6]. Use of such instrumentation to gather sufficient data in order to explain otherwise confusing values is encouraged. Previous reports had indicated that the investigators had puzzled over a seemingly quick loss in tendon force [10].

3.3 CREEP OF CONCRETE CONTAINMENT WALLS

Creep, elastic shortening, and shrinkage of the containment wall concrete were estimated by the Licensee in Section 4.2 of Reference 11.

Review of these analyses indicated that the Licensee based creep and elastic shortening on the containment wall stress due to the tendon load only. Actually, the wall will shorten and creep from its own dead weight in combination with the tendon load. While elastic shortening under the dead weight of the walls and roof is a constant value and will not affect continued tendon relaxation, the dead weight will affect creep.

Because the relaxation of tendon force due to containment wall creep was shown to be low [11], the additional, but smaller stress due to dead weight of walls and roof would cause only a small increase in the tendon relaxation estimated by the Licensee.

The Licensee noted [11] that constants A and B, normally determined by tests, were not available for the Ginna plant because such tests were not required at the time. In place of actual test data, the Licensee used data from tests for a similar concrete containment [11].

Because the concrete creep loss produced a relatively small part of the total permissible relaxation of the tendon force (total range, 742 kips at 0.7 GUTS lock-off versus the specified minimum of 636 kips), it appears from this evaluation that the Licensee's neglect of dead weight is not important.

3.4 CREEP AND COMPRESSION SET OF ELASTOMERIC PADS

In response to a question from the NRC, the Licensee indicated that data from a 10-year test of neoprene were used as a basis for the estimation of long-term creep of the neoprene elastomeric pads.

The Licensee described the pads as follows [10]:

> "The containment wall is designed to have a hinge at its intersection with the base mat. This hinge is developed with elastomeric pads between these structural components. Each pad consists of two 11/16 inch thick layers of 55 durometer hardness neoprene, separated by and epoxy bonded to a 10 gauge (0.135 inch) steel shim. In addition, a 16 gauge (0.06 inch) steel shim is epoxy bonded to the top and bottom of the composite pad set. The total pad thickness is 1.63 inches, including the steel shims and 9 inch by 42 inch in plan. The combined neoprene thickness is 1 1/38 inches."

The detailed design of the pads and their location at the base of the containment wall is shown in Figure 2.8-1 of Reference 10. A table of elastomeric pad properties is given in Table 5.1.2-5 of the FSAR [3].

In Section 2.8 of Reference 10, the Licensee provided an extensive discussion of the analysis used to estimate the deflection and creep of the neoprene elastomeric pads. The analysis follows the method employing

shape factors for the pad geometry, which is recommended by the duPont Company, manufacturer of the neoprene elastomeric material. No estimate of compression set is noted in the analysis, although this may be reflected in the estimate for creep. Compression set does not act in addition to creep, but serves to denote that portion of total deflection not regained when the load is removed.

The estimated deflection and creep values are small. Even if multiples of these values were to exist, they could not be considered the cause of the observed tendon relaxation. However, the NRC staff comment [10] about checking the conditions of the elastomeric pads to provide adequate support of the containment walls is well taken. Inspection of one pad to confirm that it is satisfactory would provide good assurance of adequate containment wall support as well as assurance that the pad has not contributed to the tendon force relaxation.

3.5 WALL TENDON STRESS RELAXATION

The largest source of tendon force loss appeared to be stress relaxation of the tendon wire. Repeated lift-off surveillance tests confirm the continued gradual loss of tendon force. Laboratory testing of tendon wire established that the creep of tendon wire at the tendon operating temperatures was greater than the design value and appeared to be approximately equal to the observed tendon loss, minus other known losses. However, caution is necessary when making the assumption that tendon wire stress relaxation serves to explain the loss of force without due consideration of the rock anchors which cannot be studied as readily. (Section 3.6 of this report discusses the review of the rock anchors.)

In the original tendon design, the Licensee anticipated a lower value of wire stress relaxation as follows [10]:

> "One test condition of particular interest was an initial stress level of 0.70 GUTS (Guaranteed Ultimate Tensile Strength) and 68°F. These are standard conditions which were generally used by the nuclear industry as a basis for classifying relaxation properties of prestressing wire. Thus, the wire for Ginna was supplied on this basis. The Ginna wire was stated to have a 40 year stress relaxation of 12%; this relaxation grade

being the only grade of wire which was generally commercially available at the time of the Ginna design."

However, faced with unexplained tendon force relaxation, the Licensee investigated the tendon wire properties by heat number. Table 2.4-1 of Reference 10 lists the mechanical properties of six heats of material known to be used in the 160 tendons. The yield point, defined as 1% strain, was shown to have a minimum ratio to the GUTS of 0.90 compared with the tendon loading ratios of 0.80 and 0.70 GUTS. The ratio of the proportional limit to the GUTS (240,000 psi) was shown to have ratios of 0.81 to 0.85 except for heat 22332 and one sample from heat 30091. Ratios for these exceptions were 0.75 and 0.71, and are below the maximum tendon load of 0.80 GUTS. Thus, yield points and proportional limits of the wire materials were generally above the maximum tendon loading point.

3.5.1 Experimental Tendon Wire Creep Investigation

To investigate the actual creep characteristics of the tendon wire, the Licensee contracted with the Fritz Engineering Laboratory of Lehigh University where testing was initiated in March 1981 [14].

The source of test specimens and the conditions of the test program were described by the Licensee as follows [10]:

> "Each of the 160 tendons contains an extra (sacrificial) wire, 115 feet long, which is unstressed. The extra wire was removed from three tendons (numbers 76, 51, and 150) and was cut into 23 feet lengths on site for shipment to Fritz Engineering Laboratory for stress relaxation testing in April 1980. A total of 14 specimens was tested at temperatures of 68°F, 78°F, and 104°F and at initial stress conditions of 0.70 GUTS and 0.75 GUTS (GUTS is the Guaranteed Ultimate Tensile Strength of 240 ksi). During these tests, seven specimens were retensioned."

The test conditions for each wire were shown by the Licensee in Table 3-1 of Reference 10.

Testing was in accordance with ASTM A328-78 in controlled environment cabinets previously constructed for similar tests on other cable [15].

The objectives of the tests were described by the Licensee as follows [10]:

"One objective of the tests was to determine the stress relaxation property, at a condition of 68°F and 0.70 GUTS, of the production wire provided for Ginna since its relaxation grade of 12% at 40 years was based on these conditions. A total of four specimens was allocated for these tests, at least one from each of the three tendons.

Another objective was to determine the influence of temperature and initial stress levels which are greater than the standard condition of 68°F and 0.70 GUTS. Investigation of these effects was considered to be important since the tendons in the containment building have been exposed to temperatures greater than 68°F for prolonged time periods; for example, in the neighborhood of 90°F during plant operation. Also, some tendons were originally stressed greater than 0.70 GUTS, a few as high as 0.74 GUTS. Thus, one specimen from each of the tendons was subjected to an initial stress level of 0.75 GUTS; and a total of eight specimens was tested at a constant elevated temperature of 104°F. In addition, two specimens had their temperature increased from 78°F to 104°F at 1,000 hours after initial stressing. The purpose of this test was to determine the effect on stress relaxation of an elevated temperature (104°F) after substantial stress relaxation had already occurred.

One specimen was tested to determine the effect of applying a temporary 6% overstress, similar to that performed on the tendons at previous surveillances. After being under load for 10,000 hours, an incremented force was applied to temporarily stress the specimen to 1.06 times its existing stress level, then the specimen was reseated to its existing value. This so-called overstressing was performed several times at specified intervals."

These tests constituted the "original tests" which were performed prior to the extension of the test program to study the effects of retensioning. Results of the "original tests" were shown by the Licensee in Figures 3-1 through 3-4 of Reference 10. Figure 3-1 indicated that the present relaxation (extrapolated to 40 years) ranged from a low of 13% for specimen 51-C, which was tested at a stress level of 0.70 GUTS and 68°F, to a high of 21% for specimen 76A, tested at 0.75 GUTS and 104°F. The results were distributed from low to high according to test temperature (68°F and 104°F) and level of loading (0.70 and 0.75 GUTS). These data represented all specimens of wires from tendons 51 and 76. Curiously, all specimens from tendon 150, with the exception of 150A, yielded stress relaxation percentages much below the standard theoretical reference wire (0.70 GUTS and 68°F), which served as the

12% relaxation (40-year) reference point. Specimen 150A, tested at 0.75 GUTS and 104°F, was shown to yield 20% stress relaxation similar to the specimens from tendons 51 and 76, using the same extrapolation methods.

The Licensee discussed the anomaly for the specimens from tendon 150 on pages 3-9 through 3-12 of Reference 10, but provided little information to document the differences. However, the consistency of the stress relaxation of wires from tendons 51 and 76 as well as specimen 150A appears to document increased stress relaxation above the original 40-year design value of 12%. With the understanding that the actual temperature of the tendons was in the range of 85°F to 95°F for much of the time, it was evident that tendon wire stress relaxation could account for a large portion of the unanticipated tendon force loss.

Tests for the short-term 6% overstress were reported to have an insignificant effect on overall stress relaxation [10].

Recognizing that the test data appeared to indicate 40-year stress relaxation values on the order of 15% or more, and realizing that all tendons had been retensioned (23 in 1969 and 137 in 1980), the Licensee modified the tests on the tendon wire samples to study the effects of retensioning. Accordingly, seven test specimens were restressed to their initial stress level. The Licensee described the retensioned specimens as follows [10]:

> "The seven specimens were allocated to assess three effects: (1) duration under load prior to restressing, (2) variation between tendons, and (3) temperature. Specimens from tendon 76 were retensioned at three time decades: 100 hours, 1,000 hours and 10,000 hours. Three of the seven specimens were tested at 68°F and four were tested at 104°F. Tendon 76 had four retensioned wire specimens, tendon 51 had two; and tendon 150 had one retensioned specimen. The retensioning test conditions are summarized in Table 3-1 (of Reference 10)."

The Licensee described the results of the retensioning as follows [10]:

> "The results for the retensioned wire specimens are indicated in Figures 3-5 through 3-10. The stress relaxation history before and after restressing is shown. From these results, it is apparent that the effect of retensioning is to cause a much lower stress relaxation in the wire as expected. Figure 3-11 superimposes the retensioned stress relaxations of specimens 8, 9, and 10 for tendon 76. These specimens were all at the same test condition of 0.70 GUTS and 104°F but were retensioned one time

decade apart at 1000 hours, 1,000 hours, and 10,000 hours. These results indicate that the retensioned stress relaxation property decreases as time of retensioning increases. Therefore, wires which have relaxed for a long period of time prior to retensioning will exhibit less relaxation subsequent to retensioning than wires which were retensioned at earlier times. For tendon retensioning, the time of retensioning was 11.4 years or 98,988 hours, which is one time decade beyond the longest test retensioning time (10,192 hours for specimen 8). Thus, the test results must be evaluated to allow a projection of the stress relaxation in a wire restressed 10 years after initial stressing. This projection must extend to 30 years after restressing to be applicable to the actual condition of the containment tendons."

Although the results of the tests did indicate that the rate of continued stress relaxation following retensioning was inversely proportional to the decade of time in which the retensioning was performed, it must be realized that the total wire specimen test time under tension was a little more than a year, compared with 11 years for 137 of the tendons in the containment wall. The real-life decade of 11 years is very large compared with the test decades of 100 hours, 1,000 hours, and 10,000 hours. Acceptance of the extrapolation to the 11-year decade can only be confirmed by continued monitoring of the test specimens and surveillance of the containment wall tendons.

The Licensee arrived at two additional conclusions as a result of the stress relaxation tests [10]:

"The difference among all three tendons in the stress relaxation properties of wires tested at 104°F decreased with duration under load to the point where after ten years under load the maximum difference was within 10%. Therefore, it appears that even though wires from different heats may initially exhibit significantly different stress relaxation properties (at the same test conditions), if the wires are at elevated temperatures and have been under load for 10 years or more the difference in stress relaxation between heats becomes small.

The effect of increasing the temperature from 78°F to 104°F after the wire has been under load at the lower temperature was to cause a sharp increase in the stress relaxation. It also appears that it is possible for the increased relaxation to approach, or even exceed, the constant 104°F data. Therefore, it appears to be possible for a wire initially stressed at an ambient temperature, which increased later, to eventually exhibit the same stress relaxation as that of a wire which had been under the elevated temperature throughout its load history."

In summary, this evaluation of the wire specimen test program acknowledges that the evidence of trends is apparent--trends that have important ramifications in the control of long-term relaxation. However, the numerical values of the data and the actual levels of influence were based upon a very small sample of specimens, and one heat of wire material out of three possible heats provided contrary results.

3.5.2 Factor Method

The Licensee first investigated the use of superposition techniques for predicting tendon stress relaxation following retensioning, but then developed the factor method that appears to be more accurate.

The Licensee describes the factor method as follows [10]:

> "The Lehigh retensioning tests were designed to generate the empirical data for this approach. Three wires from one tendon (76) were retensioned at 100, 1,000 and 10,000 hours to establish a prediction curve. Figure 4-1 [reproduced here as Figure 1] is a composite idealization of these three stress relaxation curves and the prediction curve generated from the empirical data. These four idealizations will be used to explain the basic development of the Factor Method of predicting retensioned stress relaxation values.
>
> Figures 4-1a, 4-1b and 4-1c are idealized stress relaxation curves for wires retensioned at 10,000, 1,000, and 100 hours. Both the original and the retensioned curves are shown with their extrapolations to some future surveillance time (ST). It is at this surveillance time that the stress relaxation information is needed. For each curve the stress relaxation values are determined at the surveillance time; these are the points A through F. Because each original curve is the plot of data from wire specimens at the test conditions, the following equation should exists:
>
> $$A = C = E$$
>
> The points B, D and F are on the retensioned wire curve at the surveillance time. Each of these values is divided by A to obtain the ratio of the retensioned stress relaxation to original tensioning stress relaxation for different retensioning times. These three ratios are plotted as shown in Figure 4-1d and extrapolated to the surveillance time (ST). The value of this ratio (X) at the surveillance time is determined from the extrapolated plot. X represents the factor that the original stress relaxation value at the surveillance time (ST) would be multiplied by to obtain the retensioned stress relaxation value at the surveillance time (ST)."

Review and evaluation of the factor method indicates that the method uses the relaxation data from empirical tests of retensioned tendon wires. However, very few data are available for use by this method.

The whole approach was based upon three test specimens from one tendon wire, in which each test specimen represented one decade of time at the initial load. Consequently, while the method represents a good approach, confidence in the method must come from its application in continued tendon surveillance.

3.6 ROCK ANCHORS

The review and evaluation of the rock anchors was performed by Geotechnical Engineers Inc. (GEI) under a subcontract from the Franklin Research Center. GEI's report of the review is included as Appendix A to this report.

GEI reviewed the rock anchors using documents supplied by the NRC to FRC. Brief commentary on their review follows:

Bond Slip Along the Grout/Rock Interface

GEI reviewed the Licensee's reports and determined, in support of the Licensee statements, that it is unlikely that significant load loss has occurred due to bond slip along the grout/rock interface.

Debonding Along the Wire/Grout Interface

GEI reviewed the Licensee's submittals and used data from the Licensee's reports to estimate that debonding along the wire/grout interface would effectively lengthen the unbonded portion of the rock anchor tendon, and that the elongation under load of this debonded portion of tendon would cause a reduction in the tendon force of approximately 14 kips.

It was noted that bonding stresses on the wire/grout interface were reported based upon the sum of the bonding areas of all tendon wires. This statement of bonding stress was used for direct comparison with stresses reported by the Licensee.

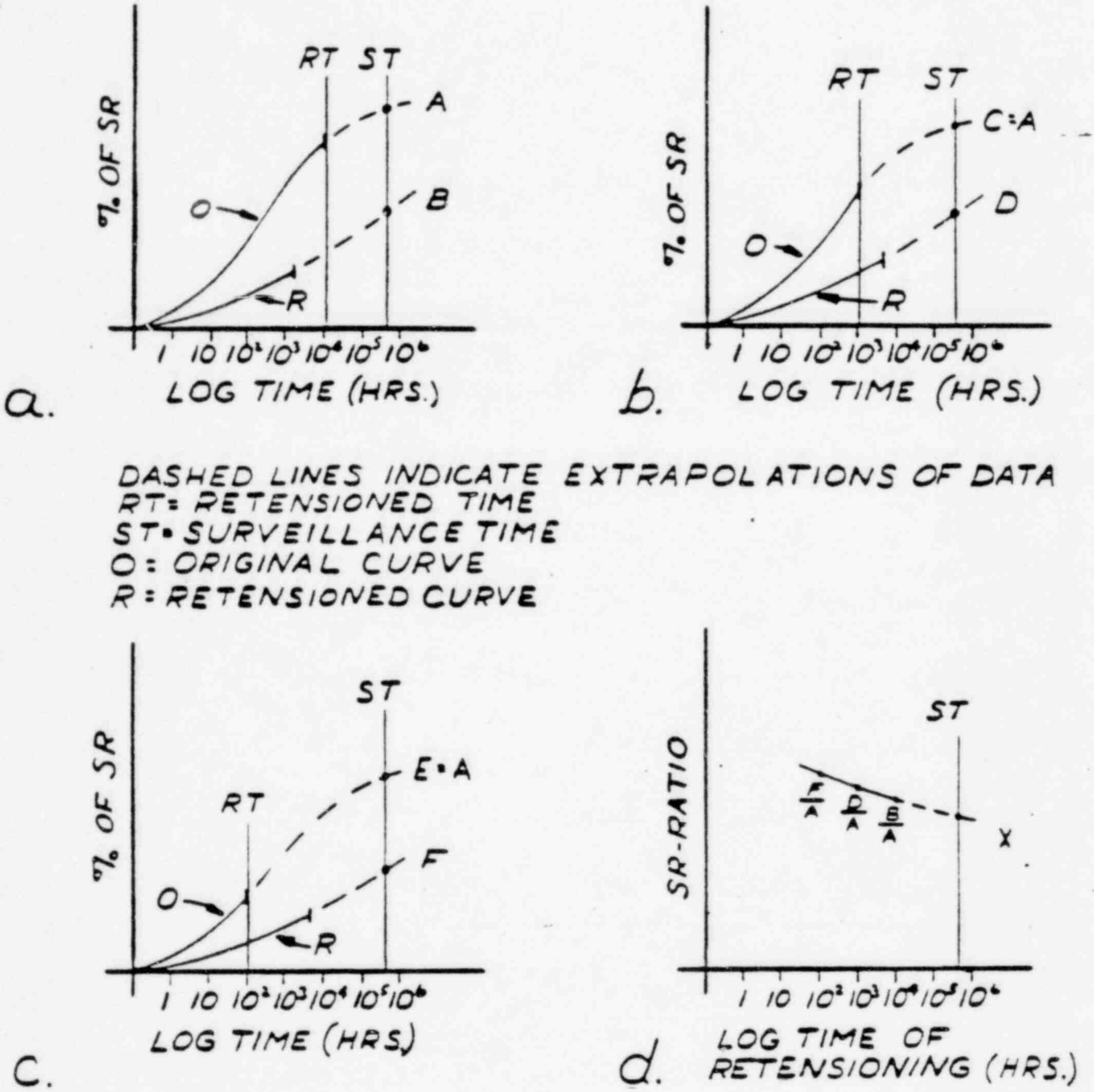

FIGURE 4-1
FACTOR METHOD

Figure 1. Factor Method (from Reference 4)

Note that debonding along the wire/grout interface does not indicate loss of anchor integrity because each tendon wire is attached to the bottom anchor head.

Creep in the Rock

GEI's review of the Licensee's rock creep analysis indicates that it is not correct. The Licensee performed a 4-hour compressive test at 10,000 psi on a rock sample. The results of the test were extrapolated to 40-year creep under 452 psi and indicated that the resulting tendon relaxation would be 0.08 kip. In Appendix A, GEI shows that the direct use of Farmer's parameters for 452 psi could yield between 8 and 122 kips tendon load loss, depending upon other necessary parameters.

In addition, the Licensee considered only creep in compression without mention of the effect of a shear mode along the rock anchor.

Consequently, it is recommended that the Licensee review the analysis methods for rock creep.

Corrosion

GEI reviewed the Licensee's reports in an effort to determine the immunity of the anchor system from corrosion. Although GEI reported that nothing in the information provided suggests a potential corrosion problem in the anchors, it is recommended that the Licensee use modern methods for corrosion protection if such methods are not already in use.

4. CONCLUSIONS

The following conclusions are based upon the review and evaluation reported herein:

- The Licensee has developed new tendon relaxation prediction methods from its efforts to diagnose and control the source of tendon force loss. These methods may be of broader interest.
- The improvement of measurement and reporting methods for tendon displacement, or stressing jack ram displacement, to work in conjunction with the improved tendon force measurement methods would permit comparative investigations of tendon system stiffness between tendons in an effort to investigate debonding of the rock anchors.
- The consistency of relaxation test results of test specimens from two tendon wires representing two material heats may not be sufficiently representative of all six heats of material used in the tendons, especially when specimens from a third heat exhibited inconsistent results.
- Application of the factor method, while encouraged, is based upon the results of only three test specimens from the same tendon wire.
- The methods employed by the Licensee for rock creep analysis are based upon incorrect extrapolation analysis and ignore shear mode creep.

5. RECOMMENDATIONS

Following completion of the review and evaluation reported herein, it is recommended that the Licensee should:

- Maintain lift-off force surveillance of the wall tendons because the data upon which the refined prediction methods are based are very meager.
- Continue the experimental investigation of tendon wire relaxation using a larger and broader sample of test specimens. This would provide a better foundation of knowledge to guide future liftoff surveillance and aid in the explanation of any further unexpected behavior.
- Introduce more accurate measurement and recording methods for tendon elongation and stressing jack displacements to enable comparative estimates of tendon system stiffness in an effort to discern anchor tendon debonding.
- Reexamine its analysis for rock creep and provide analysis based upon more comprehensive methods including shear mode effects and extrapolation of rock test data.

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APPENDIX A

REVIEW OF ROCK ANCHOR EVALUATION, R. E. GINNA NUCLEAR PLANT, BY GEOTECHNICAL ENGINEERS INC.

Review of Rock Anchor Evaluation

R. E. Ginna Nuclear Plant

January 10, 1985

Submitted to

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by

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1. INTRODUCTION

1.1 Purpose

The purpose of this report is to summarize our review of previous studies performed to evaluate the performance and integrity of the rock anchors for the containment vessel wall tendons at the R. E. Ginna Nuclear Power Station. The studies that we reviewed were performed as part of an investigation of prestress losses in the wall tendons which were greater than those predicted during design.

1.2 Scope

Geotechnical Engineers Inc. reviewed reports and documents pertaining to the containment vessel rock anchors provided to us by Mr. Clyde Herrick of Franklin Research Center. The documents we reviewed are listed in Table 1. Our review was intended to:

1. Determine whether all factors have been considered relative to the rock anchors as a potential source of prestress loss in the wall tendons.

2. Determine whether there is sufficient data to properly evaluate all of the relevant factors.

3. Evaluate the potential magnitude of anchor-related prestress losses resulting from various mechanisms, to the extent permitted by the information provided to us.

1.3 Authorization

This work was authorized by Franklin Research Center by Purchase Order No. F73711 dated December 3, 1984.

2. SUMMARY

Based on our review of the information provided to us, the following conclusions have been reached regarding potential load loss in the wall tendons caused by the rock anchors due to the following mechanisms:

1. Bond slip along the grout/rock interface
2. Debonding along the wire/grout interface
3. Creep in the rock
4. Corrosion

Bond Slip Along the Grout/Rock Interface

It is unlikely that significant load loss has occurred due to bond slip along the grout/rock interface. This conclusion is based on analysis of the anchor load history, the estimated bond stress levels in the anchors, and the magnitude of anchor slip required to produce significant load loss in the wall tendons.

Debonding Along the Wire/Grout Interface

The estimated maximum potential load loss due to debonding along the wire/grout interface is on the order of 14 kips. The tendon elongation data from the original tensioning and June 1980 retensioning of the wall tendons do not indicate a significant load loss due to wire/grout debonding; however, these data are subject to signficant measurement errors. More accurate tendon elongation data from performance of a complete unload-reload cycle during future tendon surveillances would provide data for a better evaluation. Some wire/grout debonding should be expected and does not indicate loss of anchor integrity.

Creep in the Rock

In our opinion, the analysis of potential load loss due to anchor creep presented by Gilbert Associates is not correct. There is insufficient data to evaluate the actual magnitude of creep in the anchors and the potential load loss in the wall tendons from this source. In our opinion, the only meaningful way to evaluate the potential magnitude of creep deformation in the anchors would be to perform full-scale constant load creep tests on anchors as nearly identical to the actual containment vessel anchors as possible. Although we are not able to quantify the potential contribution

of anchor creep to load loss in the wall tendons, there is no reason to suspect that anchor creep would affect the ultimate load capacity of the anchors.

Corrosion

There is nothing in the information provided to us for our review to suggest an existing or potential corrosion problem in the anchors. Chemical tests to classify the aggressiveness of the anchor environment are required to determine whether the corrosion protection provided for the anchors is in accordance with current standards of practice.

3. REVIEW OF POTENTIAL SOURCES OF ANCHOR-RELATED PRESTRESS LOSSES

3.1 General

The concrete walls of the Ginna containment vessel contain 160 vertical post-tensioned steel tendons. Each wall tendon is approximately 115 ft long and is connected at the containment base to a vertical rock anchor grouted into a 6-in. drill hole extending approximately 34 ft into the underlying sandstone bedrock. Both the wall tendons and rock anchors consist of 90 1/4-in.-diameter high strength steel wires with buttonhead end connections (BBRV system). The containment vessel and rock anchors are shown in Figs. 1 and 2, which are reproduced from Gilbert Associates' February 1982 report.

The rock anchors were tested to 849 kips (0.8 f_u) and locked off at 742 kips (0.7 f_u) in the fall of 1966. The initially unbonded upper portions of the anchors were grouted after post-tensioning (second-stage grouting). The wall tendons were attached to the rock anchors, tested to 849 kips, and locked off at 742 kips in the spring of 1969. Lift-off tests were performed on the wall tendons immediately following lock off to verify the actual lock-off loads.

The wall tendon loads were periodically monitored by lift-off tests on selected tendons as part of a planned surveillance program. The lift-off tests performed over the first ten years indicated that the load losses in the wall tendons exceeded the predicted losses by an average of about 20 kips with a maximum recorded loss of 65 kips greater than the predicted value. These unpredicted losses occurred as soon as 1,000 hours after initial stressing of the tendons. As a result of the greater-than-predicted load losses, the wall tendons were retensioned to an average load of 760 kips in June 1980 (11 years after initial tensioning). Lift-off tests performed during the June 1980 retensioning indicated that the load losses in the wall tendons exceeded the predicted losses by an average of 39 kips prior to retensioning.

An investigation of the possible causes of the greater-than-predicted wall tendon load losses was performed by Gilbert Associates Inc. Their investigation included evaluation of the wall tendons, rock anchors, and possible measurement errors. From their investigation they concluded that the greater-than-predicted load loss was due to greater stress relaxation of the steel tendon wires than assumed in the original design.

The following sections contain a discussion of the analyses and conclusions by Gilbert Associates relative to the rock anchors as a potential source of the greater-than-predicted prestress losses in the wall tendons. Potential mechanisms of anchor-related prestress loss are discussed and conclusions regarding the potential magnitude of prestress loss resulting from each mechanism are presented, to the extent permitted by the information provided to us. A list of the documents provided to us for our review is presented in Table 1.

3.2 Debonding (Slip) Along the Grout/Rock Interface

3.2.1 Magnitude of Slip Required for Tendon Load Loss

Slippage of the rock anchor results in the following elastic deformations of the containment wall and foundation:

1) Shortening of the rock anchor (ΔL_A)

2) Shortening of the wall tendon (ΔL_T)

3) Elongation of the concrete containment wall (ΔL_W)

4) Rebound of the rock below the wall footing (ΔL_R)

The resulting load loss in each component is proportional to the elastic deformation of that component.

A conservative estimate of the magnitude of tendon load loss for a given magnitude of anchor slip can be made by neglecting the elastic deformations of the anchor, containment wall and rock foundation. In this case, the elastic shortening of the tendon (ΔL_T) equals the anchor slip (δ_A), and the tendon load loss (ΔP_T) can be computed as follows:

$$\delta_A = \Delta L_T = \frac{\Delta P_T \text{ (115 ft x 12)}}{(4.42 \text{ in}^2)(29 \times 10^3 \text{ ksi})}$$

ΔP_T kips = 93 x δ_A inches

The compensating effects of the elastic deformations of the anchor, containment wall, and rock foundation can be evaluated by assuming that:

1) The slip occurs at the lower end of the anchor.

2) The anchor wires are completely unbonded down to the lower anchor head.

In this case, the elastic shortening of the tendon (ΔL_T) is equal to:

$$L_T = \delta_A - \Delta L_A - \Delta L_W - \Delta L_R \qquad (1)$$

and the load losses in the tendon (ΔP_T), anchor (ΔP_A), and wall (ΔP_W) are related as follows:

$$\Delta P_A = \Delta P_T$$

$$\Delta P_W = \frac{\Delta P_T}{2.13 \text{ ft}} \text{ (load per lin. ft of wall)}$$

The elastic deformations of the tendon (ΔL_T), anchor (ΔL_A) and wall (ΔL_W) are equal to:

$$\Delta L_T = \frac{\Delta P_T \text{ (115 ft x 12)}}{(4.42 \text{ in}^2)(29 \times 10^3 \text{ ksi})} \qquad (2)$$

$$\Delta L_A = \frac{\Delta P_T \text{ (34 ft x 12)}}{(4.42 \text{ in}^2)(29 \times 10^3 \text{ ksi})} \qquad (3)$$

$$\Delta L_W = \frac{\Delta P_T \text{ (115 ft x 12)}}{(2.13 \text{ ft})(555 \text{ in}^2/\text{ft})(4 \times 10^3 \text{ ksi})^*} \qquad (4)$$

The elastic deformation of the rock foundation can be calculated from the equation for surface loading on an infinite elastic half-space as follows:

$$\Delta L_R = \Delta P_W \left(\frac{1 - \mu^2}{E_R}\right) I \doteq \frac{\Delta P_T}{2.13} \left(\frac{1-(0.25)^2}{4 \times 10^3 \text{ ksi}}\right)(2.5) \qquad (5)$$

where the Poisson's ratio (μ) and Young's modulus (E_R) were obtained from available lab test data on the rock and an influence factor (I) for a rectangular loading with a length to width ratio of ten was assumed for the ring footing.

Combining equations (1) through (5) and solving for ΔP_T yields the following relationship between anchor slip and tendon load loss:

$$\Delta P_T \text{ kips} = 69 \times \delta_A \text{ inches}$$

The above analyses show that anchor slippage on the order of 0.2 to 0.3 inches would be required to produce a wall tendon load loss of 20 kips. This is roughly equal to 20% of the total elongation measured during stressing of the anchors to the initial 742 kip lock-off load.

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*555 in^2/ft is the transformed wall area per foot of circumference (see page 3, GAI Report No. 2074, Gilbert Associates, Inc., Nov. 20, 1979)

3.2.2 Bond Stress Level at the Grout/Rock Interface

Gilbert Associates present a comparison of the average bond stresses in the rock anchors to the average bond stresses at which "slip" was first observed in the three small-scale anchor tests performed to confirm bond capacity of the anchors. "Slip" in these tests was defined as "the first point in the application of the load where an increase in rock anchor displacement was observed without an accompanying increase in load." First slip in the small-scale anchor tests reportedly occurred at an average bond stress of 340 psi.

Figure 3, reproduced from Gilbert Associates' February 1982 report, shows idealized distributions of the grout/rock bond stresses during stressing of the rock anchors and subsequent stressing of the wall tendons. The average bond stress in the first-stage grout at the maximum anchor test load (0.8 f_u) was 163 psi, assuming that the load was distributed over the entire length of the first-stage grout. A conservative estimate of the load in the second-stage grout during testing of the wall tendons to 0.8 f_u was obtained by assuming that the load in the anchor had decreased to 0.6 f_u prior to stressing of the wall tendons. This results in a load of 212 kips (0.2 f_u) in the second-stage grout and an average bond stress of approximately 100 psi, assuming that the load is distributed over the entire length of the second-stage grout. The average bond stress of 340 psi at which slip occurred in the small-scale anchor tests is about twice the maximum average bond stress of 163 psi in the first-stage grout and more than three times the maximum average bond stress of 100 psi estimated for the second-stage grout. Based on this comparison, Gilbert Associates conclude that the bond stresses in the full-scale anchors were not large enough to produce slip at the grout/rock interface.

Even when the average bond stress along the grout/rock interface is less than the ultimate bond stress, progressive bond slip can still occur due to the fact that the initial distribution of bond stresses along the interface are generally not uniform, except for anchors in very soft rocks. Theoretical studies and field measurements performed on instrumented anchors (Ref. 3, pp. 3-8) indicate that the bond stress at the upper end of the anchor may be 5 to 10 times the average bond stress. Thus the bond stress at the upper end of the anchor may exceed the ultimate bond stress, even though the average stress is well below the ultimate stress. As local bond slip occurs at the upper end of the anchor, load is redistributed toward the lower end of the anchor until equilibrium is reached. The redistribution of load toward the

lower end of the anchor results in elastic deformation of the anchor which appears as "slip" at the upper anchor head.

The progressive debonding mechanism described above is considered unlikely to contribute significantly to load loss in the Ginna wall tendons because of the second-stage grouting performed prior to attachment of the wall tendons. The second-stage grout would restrain slip at the top of the first-stage grout, where the bond stresses are highest. Significant progressive slip in the second-stage grout is unlikely because the estimated average bond stress level in the second-stage grout following tensioning of the wall tendons is very small (less than 50 psi). Also, based on the small-scale anchor tests, debonding along the wire/grout interface is expected to occur prior to debonding along the grout/rock interface. This would reduce the stress concentration at the upper end of the anchor by transferring load farther down the anchor.

3.2.3 Comparison of Calculated and Measured Wall Tendon Elongations

Debonding along the grout/rock interface in the upper portion of the anchor would increase the elongations measured during stressing of the wall tendons. Gilbert Associates present a comparison of the measured versus calculated tendon elongations during the initial tensioning of the wall tendons and the June 1980 retensioning as evidence that slip has not occurred along the grout/rock interface. Since debonding along the wire/grout interface would also increase the measured tendon elongation, and since debonding along this interface is expected to occur prior to debonding along the grout/rock interface, this comparison is discussed in a later section on wire/grout debonding.

3.2.4 Conclusions

Based on analysis of the anchor load history, the estimated bond stress levels in the anchors, and the magnitude of anchor slip required to produce significant load loss in the wall tendons, we conclude it is unlikely that significant load loss has occurred due to bond slip along the grout/rock interface.

3.3 Debonding (Slip) Along the Wire/Grout Interface

3.3.1 Potential Magnitude of Wall Tendon Load Loss

A conservative estimate of the potential magnitude of load loss in the wall tendon for complete wire/grout debonding along a given length of the rock anchor can be made by neglecting the resulting elastic deformations of the containment wall, rock foundation, and remaining bonded portion of the anchor. These deformations tend to reduce the resulting load loss, but the analysis presented for anchor slip in Section 3.2.1 indicates that the effect of the neglected deformations is small. Neglecting these deformations, the elastic shortening of the wall tendon is equal to the elastic elongation of the anchor wires in the debonded portion of the anchor,

$$\frac{\Delta P_T \text{ (115 ft x 12)}}{(4.42 \text{ in}^2)(29 \times 10^3 \text{ ksi})} = \frac{-\Delta P_{AU} \text{ (}L_{AU}\text{ ft x 12)}}{(4.42 \text{ in.}^2)(29 \times 10^3 \text{ ksi})}$$

$$\Delta P_{AU} = \frac{-115}{L_{AU}} \times \Delta P_T \tag{1}$$

and the final load in the debonded portion of the anchor must equal the final load in the wall tendon,

$$P_T + \Delta P_T = P_{AU} + \Delta P_{AU} \tag{2}$$

Combining (1) and (2) gives the following expression for load loss in the wall tendon (ΔP_T):

$$\Delta P_T = \frac{P_{AU} - P_T}{1 + \frac{115}{L_{AU}}}$$

where P_T = initial load in wall tendon

P_{AU} = initial load in debonded portion of anchor

L_{AU} = length of debonded portion of anchor (ft)

Assuming the anchor load has decreased to 0.6 f_u by the time the wall tendon is locked off at 0.7 f_u (the same assumption used by Gilbert Associates to estimate the bond stresses in the second-stage grout), the expression becomes:

$$\Delta P_T = \frac{106}{1 + \frac{115}{L_{AU}}} \text{ kips}$$

If the anchor wires were to become completely debonded within the approximately 10-ft-long second-stage grout, the approximately 8 ft effective length of the first-stage grout debonded during initial stressing of the anchor would also contribute to the resulting load loss in the wall tendon. For this case, the estimated load loss in the wall tendon is about 14 kips from the analysis presented above. This is considered to be a conservative upper bound for the potential load loss due to debonding along the wire/grout interface.

Debonding of the anchor wires would also increase the unbonded wire length subject to stress relaxation.

3.3.2 Bond Stress Level at the Wire/Grout Interface

The interpretation of the small-scale anchor tests presented by Gilbert Associates suggests that complete wire/grout debonding occurred at an average wire/grout bond stress of 17 to 34 psi in these tests. The estimated average wire/grout bond stress in the second-stage grout of the full-scale anchors during testing of the wall tendons is approximately 25 psi, assuming the load in the anchor had decreased to $0.6\ f_u$ prior to stressing of the wall tendons. This comparison suggests that the magnitude of the wire/grout bond stresses in the second-stage grout may have been great enough to cause significant debonding along the wire/grout interface.

Gilbert Associates also present estimated values of the average wire/grout bond stresses developed in the actual rock anchors, based on the anchor elongations measured during stressing of the anchors and an assumed uniform distribution of the wire/grout bond stress, as shown in Fig. 4a. This analysis indicates that average wire/grout bond stresses in the range of 53 to 83 psi were developed in the actual rock anchors, which is significantly higher than the range of 17 to 34 psi at which complete debonding is interpreted to have occurred in the small-scale anchors tests. The actual wire/grout bond stress distribution is probably closer to the distribution shown in Fig. 4b because laboratory pull-out tests on single wires (Refs. 2 and 4) indicate that the bond stress drops off sharply to a residual value in the range of 25% to 50% of the peak value after initial bond slip occurs. The bond stress distribution shown in Fig. 4b actually yields a shorter bond transmission length and higher average bond stress than the assumed uniform distribution shown in Fig. 4a. A lower bound estimate of about 45 psi for the average wire/grout bond stress developed in the anchors can be obtained by assuming that the wire/grout bond is transmitted over the entire 23-ft grouted portion of the anchor. These results suggest that the ultimate wire/grout bond stress is

significantly higher than the value of 17 to 34 psi interpreted from the small-scale anchor tests.

Recommendations presented by Littlejohn for minimum bond development length of plain prestressing wire (Ref. 3, p. 9) correspond to an average bond stress of about 120 psi for 1/4-in.-diameter wire. Laboratory pull-out tests on single wires reported by Stocker and Sozen (Ref. 4) and Keuning, et al. (Ref. 2) yielded peak bond stresses in the range of 300 to 350 psi and residual values in the range of 80 to 160 psi. The tests on single wires probably represent upper bound values, since group effects may reduce the ultimate bond stress in multiple wire anchors. The ultimate wire/grout bond stress value of 17 to 34 psi interpreted from the small-scale anchor tests is significantly lower than the above values obtained from the published literature.

Based on the comparisons presented above, we suspect that either: 1) errors in the test measurements have produced a misleading interpretation of the small-scale anchor test results or 2) the wire/grout bond in the small scale anchor tests is not representative of the full-scale anchors due to differences in materials or construction procedures. In this context, it is important to note that the value of 17 to 34 psi for wire/grout debonding is actually based on only one of the three small-scale anchor bond capacity tests. It is not clear whether the tendon elongation measurements for the bond capacity tests were referenced to a datum that was independent of the jack reaction or whether an initial seating load was used in the tests. Measurement errors related to these test details could result in erroneously high elongation measurements, resulting in an underestimate of the debonding stress.

3.3.3 Time-dependent Loss of Wire/Grout Bond

Plain prestressing wire is particularly susceptible to time-dependent progressive debonding along the wire/grout interface. Unlike spiral-wound strand and deformed bars, there is no tendon/grout interlocking component to the bond strength of plain wire. Once the initial physio-chemical bond of the grout to the steel is exceeded, there is an abrupt reduction in the bond stress to a residual value which apparently results from frictional resistance between the steel and the surrounding grout. Laboratory pull-out tests subjected to long-term sustained loading indicate that the initial physio-chemical bond may decay with time (Ref. 4, pp. 76-77). This could result in time-dependent progressive bond slip as the load is redistributed farther down the anchor.

Environmental factors resulting in cyclic loading of the anchors, e.g., temperature variations in the wall tendons,

could promote progressive debonding along the wire/grout interface. The lift-off tests and 6% overstressing employed in the tendon surveillance program could also contribute to debonding of the wires.

3.3.4 Comparison of Calculated and Measured Wall Tendon Elongations

Debonding along the wire/grout interface would increase the elongations measured during tensioning of the wall tendons. Thus it should be possible to detect significant debonding of the wires by comparing the tendon elongations measured during the original tensioning of the wall tendons and the June 1980 retensioning.

The tendon elongation data presented in the reports by Gilbert Associates are plotted in histograph form in Figs. 5 and 6. The histographs show the distribution of the differences between the measured and calculated tendon elongations for the original tensioning and the June 1980 retensioning. For the original tensioning of the wall tendons (Fig. 5), the distribution of the measured elongations that were less than the calculated elongation are not shown because this information was not available in the documents provided to us for our review. For the June 1980 retensioning (Fig. 6), the data for straight tendons and curved tendons is separated because excessive friction in the curved tendons could obscure the comparison between the measured and calculated elongations. The information available to us on the original tensioning (Fig. 5) did not permit a similar separation of the data for straight and curved tendons.

The elongation data for the June 1980 retensioning indicates that the measured elongations for the straight tendons were about 4% higher than the calculated values. Although the data for the original tensioning is less complete, the comparison between the original tensioning and June 1980 retensioning suggests there was some increase in the measured tendon elongations.

The tendon elongation data do not indicate a significant load loss due to wire/grout debonding. The 4% excess tendon elongation indicated in Fig. 6 corresponds to an estimated load loss of only 4 kips according to the analysis presented in Section 3.3.1. It should be noted that the tendon elongation data are subject to a load measurement error of ±3% and that the ±1/16 in. resolution of the elongation measurements is about ±3% of the total elongations measured in the June 1980 retensioning.

3.3.5 Conclusions

The estimated maximum potential load loss due to debonding along the wire/grout interface is on the order of 14 kips. The tendon elongation data from the original tensioning and June 1980 retensioning of the wall tendons do not indicate a significant load loss due to wire/grout debonding; however, these data are subject to significant measurement errors. More accurate tendon elongation data from performance of a complete unload-reload cycle during future tendon surveillances would provide data for a better evaluation. Some wire/grout debonding should be expected and does not indicate loss of anchor integrity.

3.4 Anchor Creep

3.4.1 Magnitude of Creep Required for Tendon Load Loss

The analysis presented in Section 3.2.1 is also applicable to creep of the anchor. This analysis indicates that anchor creep deformation on the order of 0.2 to 0.3 in. would be required to produce a load loss of 20 kips in the wall tendons.

3.4.2 Analysis Based on Rock Creep Properties

Gilbert Associates present an analysis of the potential load loss due to rock creep. The analysis is based on laboratory data available from an unconfined uniaxial compression test in which the load was maintained constant at 10,000 psi for a period of four hours. This data was used in conjunction with the following empirical creep relationship presented by Farmer (Ref. 1) to predict the load loss in the tendons resulting from uniaxial creep strain, ε_{cr}, of the rock foundation:

$$\varepsilon_{cr} = (\sigma/E)^n \ln t$$

where σ = uniaxial compressive stress
E = Young's modulus of the rock
t = time
n = empirical creep exponent

The laboratory test data obtained for a compressive stress of 10,000 psi was applied to the 472 psi bearing stress below the containment footing by assuming that the empirical creep exponent, n, is constant with respect to stress level. This analysis indicated a maximum load loss of 0.08 kips due to rock creep at the time at the June 1980 retensioning (11 years).

The creep analysis presented by Gilbert Associates is not correct because the empirical creep exponent is stress-dependent, as discussed in Ref. 1. Figure 7, reproduced from Ref. 1, shows an empirical relationship between the creep exponent and stress level. This figure indicates a range of n = 1.1 to 1.4 for the creep exponent at a stress level of 472 psi. Using this range of values in Gilbert Associates's analysis yields a predicted load loss in the range of 8 to 122 kips.

Another fundamental problem with this analysis is that it ignores time-dependent shear deformation along the anchor, which may be a more significant source of anchor creep movement than uniaxial compression of the rock foundation.

In our opinion, the available test data is not sufficient for a reasonably reliable estimate of the potential load loss due to anchor creep.

3.4.3 Creep Data from Published Literature

A review of the published literature indicates that anchor creep has generally been considered only in soils and in very soft rocks, such as clay shales or chalk. For most applications, anchor creep is generally considered to be negligible in relatively hard, competent rocks such as the sandstone at the Ginna plant site. We were unable to locate any anchor creep test data for anchors in hard, competent rocks.

A basic practical problem with performing full-scale anchor creep tests in hard rocks is that the creep of the unbonded anchor tendon, which may be greater than the creep of the grouted anchorage, must be evaluated and subtracted from the test measurements to determine the anchor creep.

There is a considerable body of data from laboratory creep tests on rock in uniaxial compression. However, the use of this laboratory test data to predict the creep behavior of rock anchors is highly questionable, since the mechanisms by which creep occurs in anchors are poorly understood. Use of laboratory creep data in conjunction with sophisticated analytical methods, such as finite element analyses, could yield misleading results if the anchor creep results from a different mechanism than the creep measured in the laboratory tests.

3.4.4 Conclusions

In our opinion, the analysis of potential load loss due to anchor creep presented by Gilbert Associates is not correct. There is insufficient data to evaluate the actual magnitude of creep in the anchors and the potential load loss

in the wall tendons from this source. In our opinion, the only meaningful way to evaluate the potential magnitude of creep deformation in the anchors would be to perform full-scale constant load creep tests on anchors as nearly identical to the actual containment vessel anchors as possible. Although we are not able to quantify the potential contribution of anchor creep to load loss in the wall tendons, there is no reason to suspect that anchor creep would affect the ultimate load capacity of the anchors.

3.5 Corrosion

3.5.1 Corrosion Protection System

According to the Ginna FSAR, corrosion protection of the anchors is provided by continuous grout cover and standby cathodic protection. No information is available on the aggressiveness of the anchor environment.

A recent state-of-the-art study on permanent tieback anchors (Ref. 5) concludes that a minimum 1/2-in. continuous cover of Portland cement grout will provide adequate corrosion protection for permanent anchors in nonaggressive environments, provided that the anchor tendon is electrically isolated from the structure and the anchor bond length is located in oxygen deficient ground (undisturbed natural soils, rocks, or fills below the water table). A nonaggressive environment is defined as having a pH greater than 4.5, resistivity greater than 2,000 ohm-cm, and no detectable sulfides.

The electrical isolation of the anchor tendon is intended to protect it from stray-current and long-line corrosion mechanisms. The FSAR indicates that the Ginna anchors are attached to a standby cathodic protection system consisting of potential reference cells for monitoring corrosion potential and buried anodes for application of protective current if cathodic protection is found to be necessary. Careful monitoring and maintenance of this system would appear to eliminate the need for electrical isolation of the anchor tendons.

For anchors protected by second-stage grouting, there are two critical locations where loss of continuity of the grout cover is most likely to occur: 1) the contact between the first-stage and second-stage grout and 2) the space immediately below the upper anchor head. The grouting procedures specified for the Ginna anchors include detailed procedures designed to reduce the possibility of poor quality grout at the contact between the first-stage and second-stage grout. Admixtures designed to reduce bleed and produce expansion of the grout are generally recommended for the second-stage grouting to reduce the possibility of voids below the upper anchor head. The FSAR indicates that an anti-bleed expansive admixture was used for the Ginna anchors.

3.5.2 Conclusions

There is nothing in the information provided to us for review to suggest an existing or potential corrosion problem in the anchors. Chemical tests to classify the aggressiveness of the anchor environment are required to determine whether the corrosion protection provided for the anchors is in accordance with current standards of practice.

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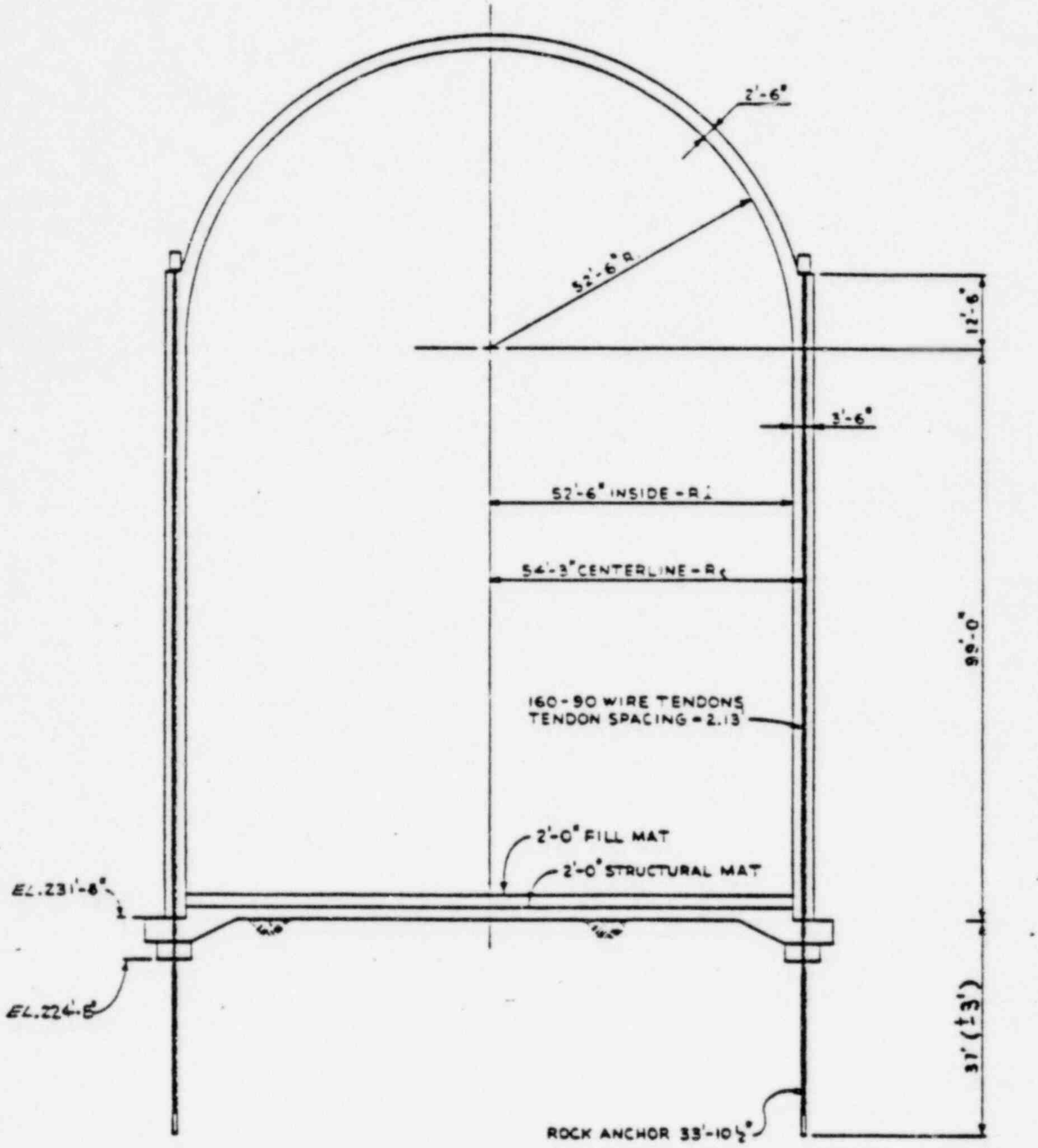
5. Weatherby, D. E., Tiebacks, Federal Highway Adminstration, Report No. FHWA/RD-82/047, 1982, 232 pp.

TABLE 1 - LIST OF DOCUMENTS REVIEWED
ROCK ANCHOR EVALUATION
GINNA NUCLEAR POWER PLANT

Date	Description
June 2, 1966	Dames and Moore report titled "Supplementary Foundation Studies, Proposed Brookwood Nuclear Power Plant, Ontario, New York, Rochester Gas and Electric Corporation"
Sept. 12, 1966	Technical Specifications for Installation of Rock Anchors for the R. E. Ginna Nuclear Power Station - Unit No. 1 (Revision 3)
(unknown)	Following sections from the Ginna Nuclear Plant FSAR: 2.8 Geology 2.9 Seismology 5.1.2 Containment System Structure Design 5.6.1 Subsection on Rock Anchor Tests (pp 5.6.1-4, 4a and 5)
Nov. 20, 1979	Gilbert Associates Inc. report titled "Evaluation of Prestressed Tendon Forces for Robert E. Ginna Nuclear Power Station" (including Addendum I - Tendon Retest)
March 17, 1981	NRC memorandum summarizing a meeting with representatives of Rocheter Gas & Electric and Gilbert Associates held on February 19, 1981 (including viewgraphs from a technical presentation given by Gilbert Associates)
Jan. 21, 1982	Report by Roger G. Slutter of Lehigh University titled "Relaxation Tests on 1/4-in. Prestressing Wire"
February 1982	Gilbert Associates Inc. report titled "Robert E. Ginna Nuclear Power Station Containment Building Tendon Investigation"

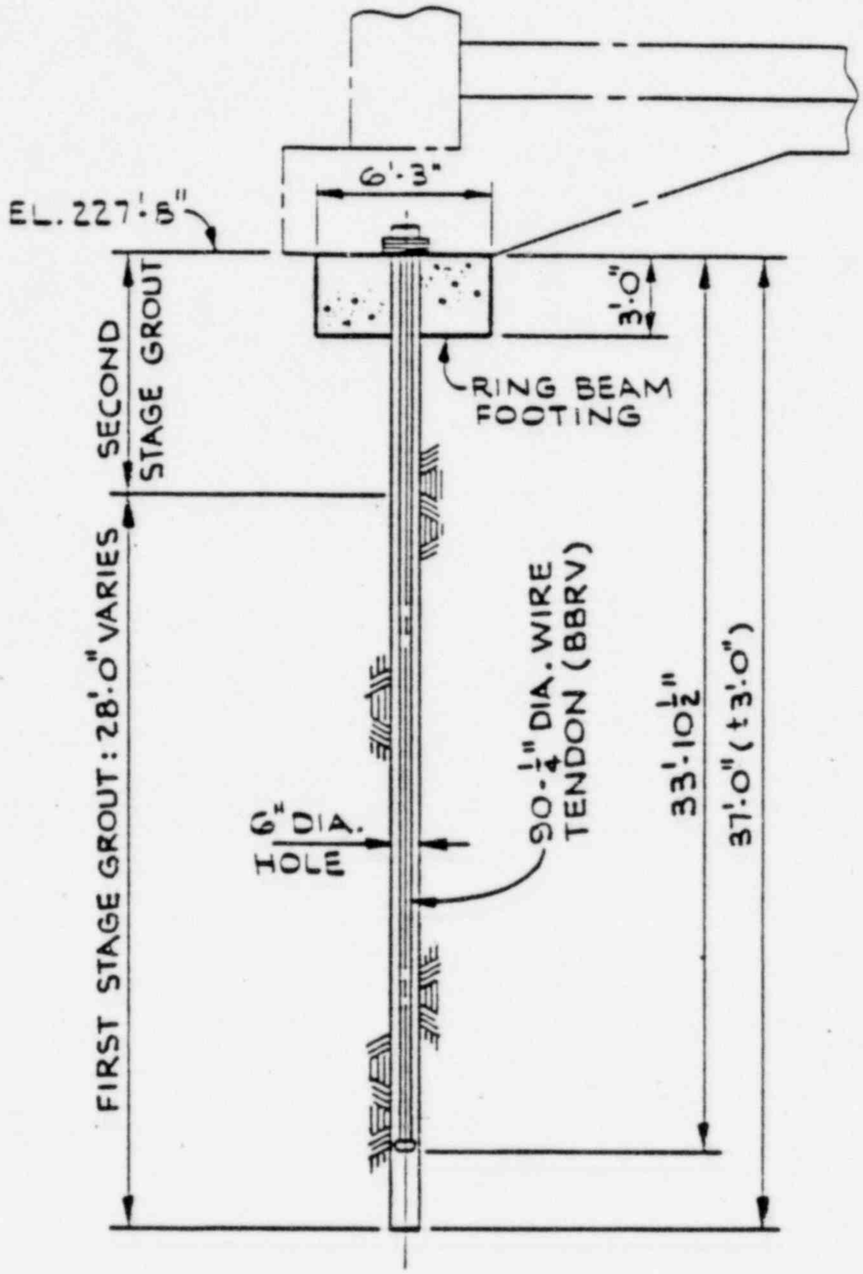
TABLE 1 - LIST OF DOCUMENTS REVIEWED
ROCK ANCHOR EVALUATION
GINNA NUCLEAR POWER PLANT

Date	Description
June 9, 1983	Gilbert Associates Inc. report titled "Robert E. Ginna Nuclear Power Station Containment Vessel Tendons - Response to USNRC Review Comments on Tendon Evaluation"
January 1984	Gilbert Associates Inc. report titled "Robert E. Ginna Nuclear Power Station Containment Vessel Tendons - 1983 Surveillance Final Report"
March 1984	Gilbert Associates Inc. report titled "Robert E. Ginna Nuclear Power Station Containment Vessel Tendons - Load Cell Evaluation"
July 31, 1984	Letter from Rochester Gas & Electric Corporation to NRC regarding tendon 75 which was damaged during the 1983 surveillance



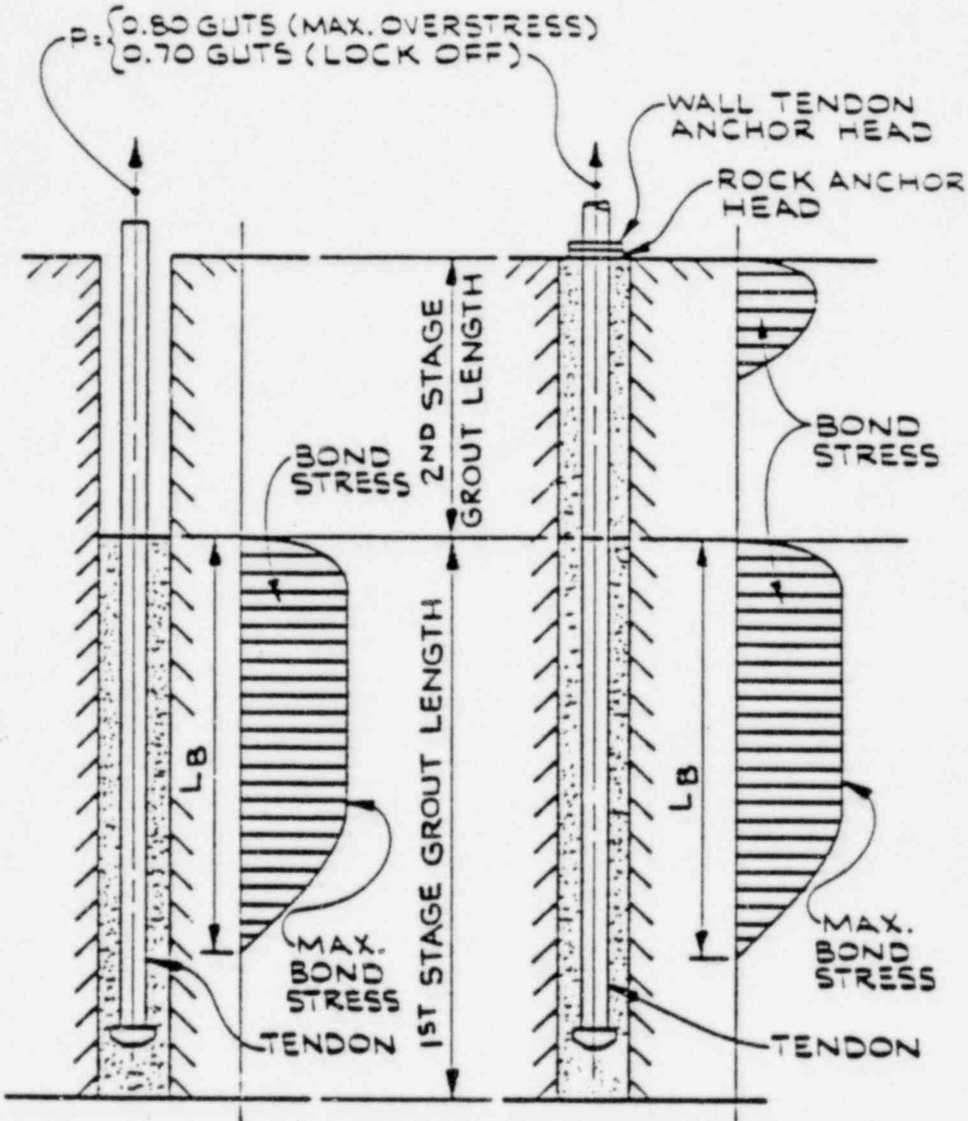
Reproduced from: Gilbert Associates Inc., "Robert E. Ginna Nuclear Power Station Containment Building Tendon Investigation," February 1982.

Franklin Research Center Philadelphia, Pennsylvania	Rock Anchor Review Ginna Nuclear Plant	CONTAINMENT VESSEL
GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Project 84460	January 10, 1985 Fig. 1



Reproduced from: Gilbert Associates Inc., "Robert E. Ginna Nuclear Power Station Containment Building Tendon Investigation," February 1982.

Franklin Research Center Philadelphia, Pennsylvania GEOTECHNICAL ENGINEERS INC WINCHESTER • MASSACHUSETTS	Rock Anchor Review Ginna Nuclear Plant	ROCK ANCHORS
	Project 84460	January 10, 1985 Fig. 2

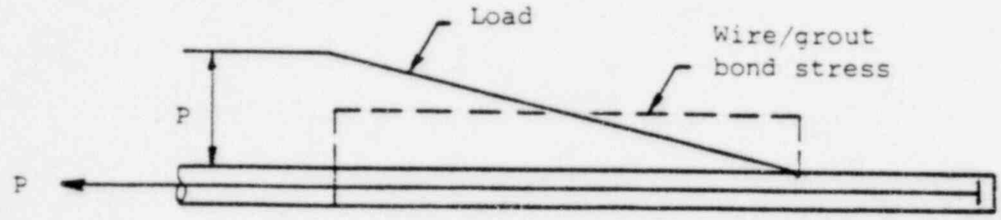


A. Rock Anchor Stressing

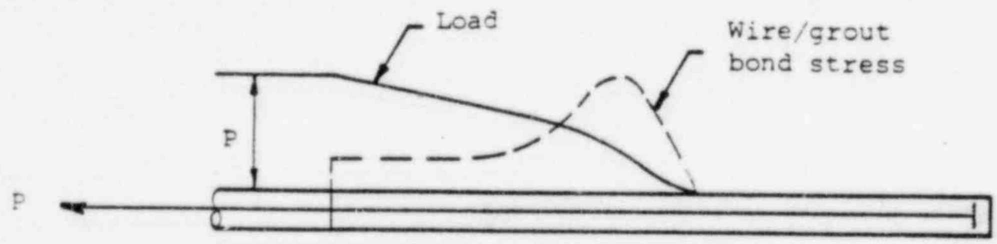
B. Wall Tendon Stressing

Reproduced from: Gilbert Associates Inc., "Robert E. Ginna Nuclear Power Station Containment Building Tendon Investigation," February 1982.

Franklin Research Center Philadelphia, Pennsylvania	Rock Anchor Review Ginna Nuclear Plant	IDEALIZED DISTRIBUTION OF GROUT/ROCK BOND STRESSES
GEOTECHNICAL ENGINEERS INC. WINCHESTER • MASSACHUSETTS	Project 84460	January 10, 1985 Fig. 3

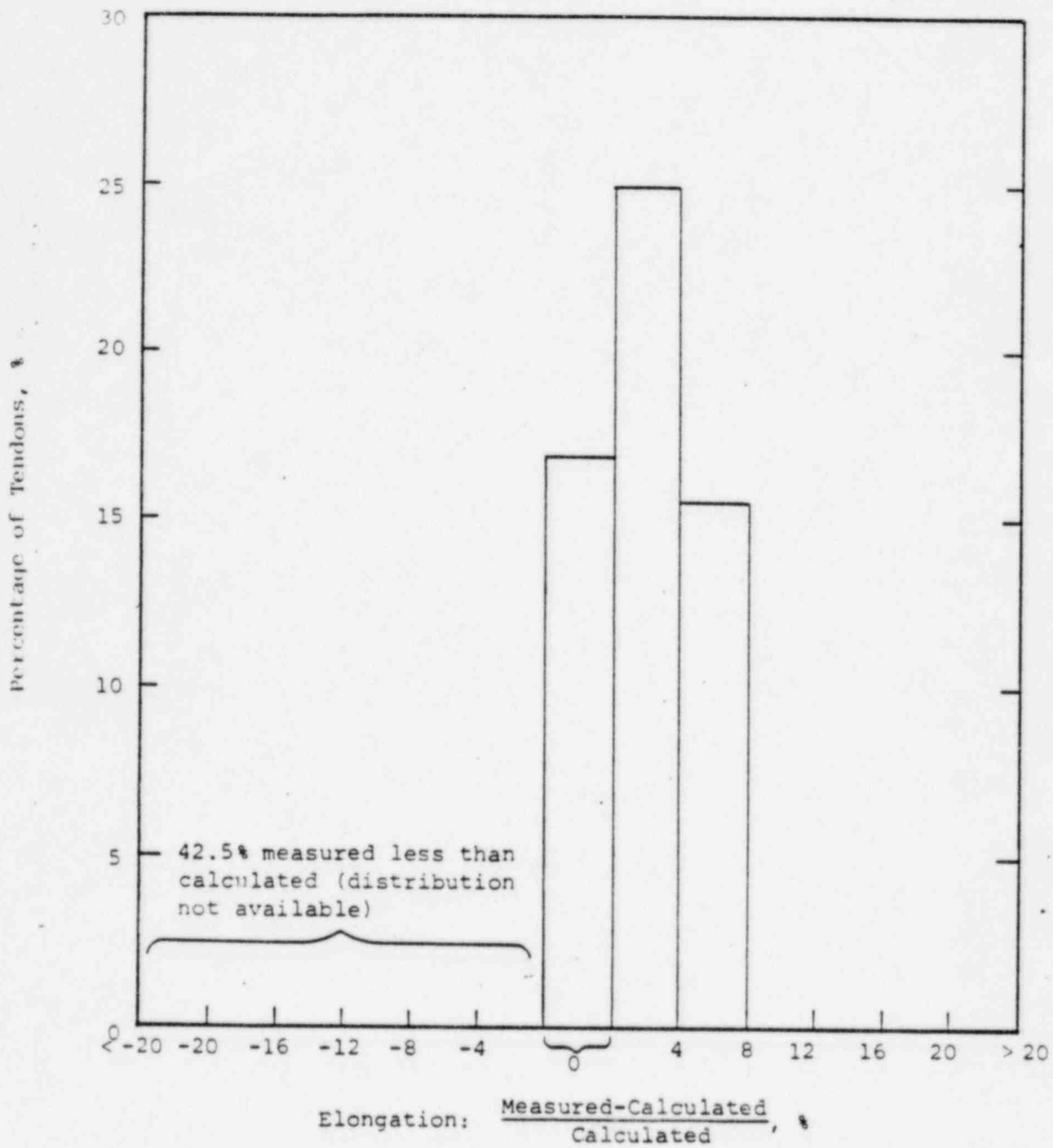


a. Assumed Wire/Grout Bond Stress Distribution

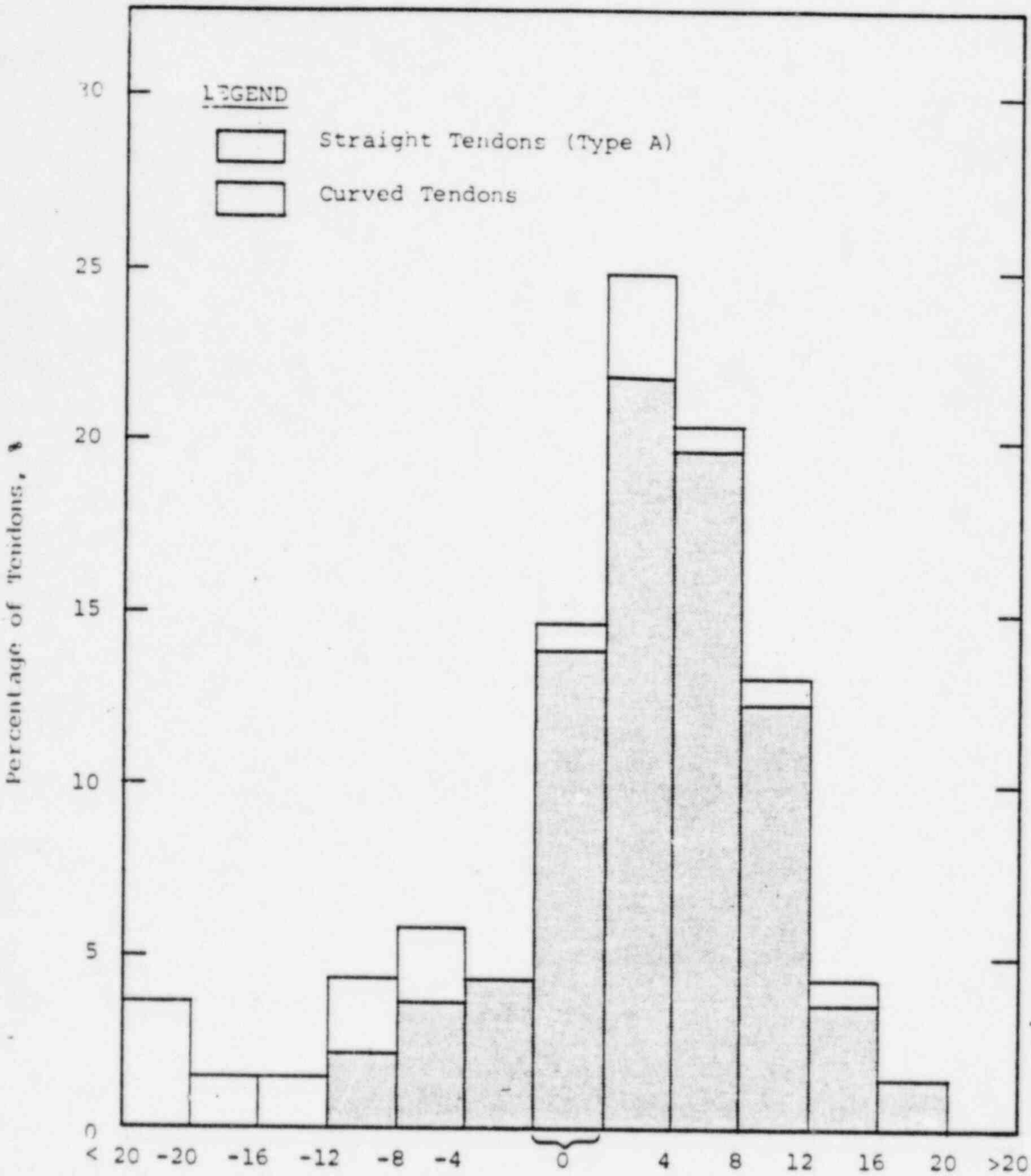


b. Probable Wire/Grout Bond Stress Distribution

Franklin Research Center Philadelphia, Pennsylvania	Rock Anchor Review Ginna Nuclear Plant	WIRE/GROUT BOND STRESS DISTRIBUTION
GEOTECHNICAL ENGINEERS INC. WINCHESTER • MASSACHUSETTS	Project 84460	January 10, 1985 Fig. 4



Franklin Research Center Philadelphia, Pennsylvania	Rock Anchor Review Ginna Nuclear Plant	MEASURED VS CALCULATED TENDON ELONGATIONS ORIGINAL TENSIONING
GEOTECHNICAL ENGINEERS INC. WINCHESTER • MASSACHUSETTS	Project 84460	January 10, 1985 Fig. 5



Elongation: $\frac{\text{Measured-Calculated}}{\text{Calculated}}$, %

Mean Values: +4.1% for straight tendons (137 total)

-8.9% for curved tendons (23 total)

+2.0% for all tendons (137 total)

Franklin Research Center Philadelphia, Pennsylvania	Rock Anchor Review Ginna Nuclear Plant	MEASURED VS CALCULATED TENDON ELONGATIONS JUNE 1980 RETENSIONING
GEOTECHNICAL ENGINEERS INC. WINCHESTER • MASSACHUSETTS	Project 84460	January 10, 1985 Fig. 6

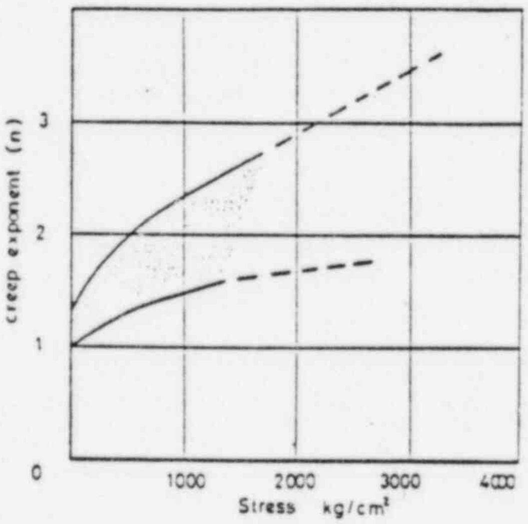


Figure 4.5 Relationship between creep exponent and stress.

Reproduced from: I. W. Farmer, Engineering Properties of Rock, 1968

Franklin Research Center Philadelphia, Pennsylvania GEOTECHNICAL ENGINEERS INC. WINCHESTER • MASSACHUSETTS	Rock Anchor Review Ginna Nuclear Plant	CREEP EXPONENT VS STRESS LEVEL
	Project 84460	January 10, 1985 Fig. 7