

ATTACHMENT 1

EVALUATION
OF **RETURN TO REACTOR DOCKET**
FILES
PRESTRESSED TENDON FORCES

RETURN TO REACTOR DOCKET
FILES

for

ROBERT E. GINNA
NUCLEAR POWER STATION

50-244
Ltr 12-12-79
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RETURN TO REACTOR DOCKET
FILES



Gilbert/Commonwealth

ENGINEERS/CONSULTANTS Reading, PA/Jackson, MI

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RETURN TO REACTOR DOCKET FILES

EVALUATION OF PRESTRESSED TENDON FORCES

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FOR

ROBERT E. GINNA NUCLEAR POWER STATION

RETURN TO REACTOR DOCKET FILES

NOVEMBER 20, 1979

BY

GILBERT ASSOCIATES, INC.
READING, PENNSYLVANIA

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1.0

INTRODUCTION

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 (see Figure 1). 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 done in the fall of 1966, and prestressing of the wall tendons was done in the spring of 1969. Following this work, there was an initial retensioning of 23 tendons 42 days later. Four tendon surveillances have been conducted since that time as required by the Technical Specification with a total of 42 tendons surveyed at least once.

In August of 1979, Gilbert Associates, Inc. (GAI) was asked to review the tendon surveillance data, respond to a series of NRC questions on the history and behavior of the post tensioning system, and recommend to Rochester Gas and Electric Corporation (RG&E) any appropriate actions relative to the containment and the prestressing system. This report is responsive to that request.

2.0

SCOPE OF WORK

The scope of GAI work covered by this report is delineated in Attachment 1. The responses to the questions raised by the NRC will be discussed in the same sequence as they appear in the attachment.

3.0

COMMITMENTS AND REQUIREMENTS FOR PRESTRESSING SYSTEM

The basic documents surveyed were the FSAR, the Technical Specification, the Project Material Specifications, and the Project Design Calculations. Attachment 2 contains pertinent extracts from these documents. The containment design was generally based on ACI 318-63, for the wall prestressing, and the recommendations of References 1 and 2 for the rock anchors. Tests were conducted to substantiate rock anchor design assumptions. Concrete materials were procured on the basis of ACI 301-66, and the PCI "Tentative Recommended Practices for Grouting Post Tensioned Prestressed Concrete."

Current containments are designed utilizing an ASME Code which was first issued in 1973 for trial use and comment. Several differences exist between the two sets of design rules, that is ACI 318 and the ASME Code. Some of these differences are discussed in later sections.

Attachment 3 contains the material requirements from these two specifications.

4.0

RESPONSES TO NRC QUESTIONS

4.1

QUESTION: Were tests conducted on the concrete to determine the amount of creep to be expected? What is current practice?

RESPONSE: The Technical Specification for Structural Concrete did not require a creep test. The current practice is to perform creep and shrinkage tests. The ASME Code, Section III, Division 2, CC-2231.3, under "Additional Property Requirements" states that these tests should be conducted if they are required by the Construction Specification.



4.2 QUESTION: How were the results of the creep tests used in the design? What is current practice?

RESPONSE: In the original design, creep and shrinkage were considered based on average historical values for concrete similar to that used. Tendon losses caused by creep were determined based on Reference 3. Tendon losses caused by shrinkage were calculated based on Reference 4. The values used in the calculations for the containment, were:

$$\begin{aligned}\text{Creep strain} &= 220 \times 10^{-6} \text{ in/in} \\ \text{Shrinkage strain} &= 100 \times 10^{-6} \text{ in/in}\end{aligned}$$

(See page 5.1.2-8A of the FSAR.)

The current state of the art is defined by the proposed USNRC Regulatory Guide 1.35.1 (Reference 5) for both shrinkage and creep calculations. The following demonstrates the application of these procedures to the Ginna containment.

1) Properties

Elastic shortening, creep, shrinkage, and tendon stress relaxation losses are based on the following:

$$\begin{aligned}\text{Area of one tendon (90 - 1/4 inch diameter wires)} &= \\ &4.42 \text{ in}^2\end{aligned}$$

$$\begin{aligned}\text{Concrete stress on Transformed Net Area at Lockoff} &= \\ (0.7 f_{pu}) &= 628 \text{ psi}\end{aligned}$$

$$\text{Concrete Young's Modulus} = 4 \times 10^6 \text{ psi}$$

$$\text{Tendon Young's Modulus} = 29 \times 10^6 \text{ psi}$$

$$\begin{aligned}\text{Guaranteed minimum ultimate tensile strength of} \\ \text{prestressing wire} &= 240 \text{ ksi}\end{aligned}$$

2) Concrete Shrinkage

For the Ginna site region, the Mean Daily Relative Humidity is in the 70% to 80% range. Therefore, from Table 1 of Reference 5, the 40 year shrinkage strain is 100×10^{-6} in/in. For purposes of plotting on a log-time ($\log t$) scale, shrinkage is taken to start 1 hour (0.00011 yr.) after the date corresponding to the average date of containment wall concrete placement. The containment wall construction and stressing dates are given in Figure 2. The shrinkage strain varies linearly with $\log t$ from zero at $t = 0.00011$ yr. to 100×10^{-6} in/in at $t = 41.4$ years (40 years after initial stressing) and is shown in Figure 3.

Only the containment shrinkage which occurs after initial stressing ($t = 1.41$ years) influences the loss of tendon force. Based on the assumed shrinkage strain variation, a strain of 78×10^{-6} in/in at $t = 1.41$ years is calculated. The tendon force loss at $t = 41.4$ years is predicted to be:

$$\begin{aligned} & (100 \times 10^{-6} \text{ in/in} - 78 \times 10^{-6} \text{ in/in}) (29 \times 10^6 \text{ psi}) \\ & (4.42 \text{ in}^2) = 2.8 \text{ kips} \end{aligned}$$

which is shown in Figure 3.

A variation of $\pm 20\%$ in the predicted values is allowed by Reference 5.

3) Concrete Creep

The recommended specific creep formula from Appendix A of Reference 5 is:

$$\frac{\epsilon_c}{f_c} = A\alpha \quad 1 - e^{-(t-t_o)/30} \quad + \quad B \log_{10} (t/t_o)$$



where:

t = time (after average time of concrete placement) when creep value is desired, days

t_0 = time of loading (after average time of concrete placement), days

f_c = average sustained concrete stress

ϵ_c = creep strain at time t

A, B are coefficients to be determined from tests

α coefficient determined by the degree of hydration that has taken place at time t_0

This expression may be evaluated for a specific concrete once the constants A and B are obtained from short-term creep tests. Since these tests were not required for Ginna, A and B cannot be determined from available experimental data on the concrete. However, Appendix A of Reference 6 provides the general rheological form of the above creep equation. In this form, A and B can be determined from the water-cement ratio of the concrete (by weight) and the volume concentration of cement paste in the concrete. Appendix C of Reference 6 provides specific creep curves for concrete initially stressed at ages ranging from 8 to 180 days. The curves were developed for a 5000 psi concrete with a water-cement ratio of 0.5 (by weight) and a cement paste volume concentration of 0.3. These are close to the values of

0.45 and 0.29 for the 5000 psi concrete used in the Ginna containment. The specific creep curves in Appendix C of Reference 6 were used in the calculations.

From Figure 2, the average age of the containment wall concrete at time of initial stressing was 515 days (1.41 years). A specific creep curve for a concrete age 515 days was obtained by extrapolating from the curves in Appendix C of Reference 6. Based on this extrapolated curve, the concrete in the containment wall is predicted to have a 1 year specific creep of 0.08×10^{-6} in/in/psi and a 40 year specific creep of 0.23×10^{-6} in/in/psi. The corresponding losses of tendon force are:

At 1 year after initial stressing ($t = 2.4$)

$$(0.08 \times 10^{-6} \text{ in/in/psi}) (628 \text{ psi}) (29 \times 10^6 \text{ psi}) \\ (4.42 \text{ in}^2) = (0.08 \times 10^{-6}) (80497 \times 10^6) = 6.4 \text{ kips}$$

At 40 years after initial stressing ($t = 41.4$)

$$(0.28 \times 10^{-6}) (80497 \times 10^6) = 18.5 \text{ kips}$$

The creep loss is zero at $t = 1.41$ years after concrete placement and varies with $\log t$ as shown in Figure 3. A variation of +25% and -15% in the predicted value is allowed by Reference 5.

4.3 QUESTION: Was there a chemical analysis of the groundwater at the site?

RESPONSE: Not to our knowledge. Corrosion protection for the anchor was provided by grouting. This method has been successfully used on a number of projects. See References 7 and 8.



4.4 QUESTION: Was rock flow, creep or squeeze considered in the analysis for relaxation?

RESPONSE: Literature used to evaluate the original design procedure as well as that resulting from a current literature search does not indicate that these phenomena would have a significant effect on the design. A more extensive investigation beyond the scope of this report would be required to confirm this conclusion.

4.5 QUESTION: Were individual relaxation curves developed for specific tendon types? What is the current practice?

RESPONSE: Individual relaxation curves were not prepared during the original design. The following describes the current practice for developing these curves (See Reference 5).

1) Predicted Tendon Forces

It is assumed that each tendon will sustain the same loss of force due to concrete creep and shrinkage and steel relaxation. These force losses are assumed to vary linearly with $\log t$ from 1 year after initial stressing to end of plant life. In addition, tendons will experience a reduction in force from their lockoff values which results from the elastic shortening of the concrete due to subsequent stressing of other tendons. The amount of elastic shortening loss is different for each stressing group depending on its sequence of stressing relative to other groups. Stressing was done in 40 groups with 4 tendons in each group. The stressing sequence schedule appears in Figure 1-A.



2) Concrete Elastic Shortening (ES)

$$\text{Total ES Loss} = \frac{(628 \text{ psi}) (29 \times 10^6 \text{ psi}) (4.42 \text{ in}^2)}{(4 \times 10^6 \text{ psi}) (1000)} = 20.1 \text{ kips}$$

$$\text{ES Loss for the } n^{\text{th}} \text{ stressing group} = \frac{40-n}{40} (20.1 \text{ kips})$$

3) Concrete Shrinkage and Creep

These calculations are shown in Section 4.2.

4) Tendon Stress Relaxation

The tendon wire material conforms to ASTM A 421-65, Type BA, and is estimated to have a 40 year stress relaxation of 12%. A typical stress relaxation curve for 12% wire was obtained from the supplier of the original wire, and was used for the loss calculations (Figure 3A). From this curve, losses at 1000 hours, 1 year, and 40 years after initial stressing are:

$$\begin{aligned} (0.047)(.7 \times 240 \text{ ksi})(4.42 \text{ in}^2) &= \\ (0.047)(742.56) &= 34.9 \text{ kips (at 1000 hours)} \end{aligned}$$

$$(0.07)(742.56) = 52.0 \text{ kips (at 1 year)}$$

$$(0.12)(.7 \times 240 \text{ ksi})(4.42 \text{ in}^2) = 89.1 \text{ kips (at 40 years)}$$

The relaxation loss is zero at $t = 1.41$ years after concrete placement and varies with $\log t$ as shown in Figure 3. Note that 39% of the steel relaxation has occurred by 1000 hours and 58% has occurred by 1 year after stressing. Reference 5 allows a variation of $\pm 15\%$ in the predicted values.



5) Total Tendon Losses

The tendon force losses are plotted over the 40 year plant life in Figure 3. These are the "base values" in the terminology of Reference 5. Applying the percentage variations to the individual losses results in the Upper and Lower Bound values.

The tendon losses at 1000 hours, 1 year, and 40 years after initial stressing are summarized in Table 2.

4.6 QUESTION: Is there any assurance that broken wires can be detected by visual observation of the stressed tendon?

RESPONSE: With the exception of curvature around containment penetrations, the tendons on the project are all straight tendons and were installed without wrapping or twisting. A space of 3/16 inch exists between each wire, this space being filled with grease. It seems unlikely that in this condition enough friction could be developed to restrain a broken wire. For additional comments see Addendum I.

4.7 QUESTION: Is a minimum stress of 60% f_u required by the design? What is the significance of the note "3. Maximum Final Stress = .6 ULT = 144,000 PSI" which appears on the Inland-Ryerson Drawing 159-2 dated 3/16/69?

RESPONSE: The first question will be answered by considering: 1) the present licensing requirement, 2) expected tendon force at 40 years, and 3) minimum design requirements.

1) Present Licensing Requirements

- a) The specified prestress or tendon lock-off force for the containment shell and the rock anchors was:
0.7 x ULTS

$$0.7 (240 \text{ ksi})(4.42 \text{ in}^2) = 168 (4.42) = 742 \text{ kips}$$



- b) The average tendon loss after 40 years based on an average predicted prestress loss of 24 ksi is:

$$24 \text{ ksi} (4.42) = 106 \text{ kips}$$

- c) The required average final prestress after 40 years of operation is:

$$0.6 (240 \text{ ksi})(4.42 \text{ in}^2) = 144 (4.42) = 636 \text{ kips}$$

- d) For original design criteria, refer to the FSAR:

- (1) page 5.1.2-8A and 26
- (2) Table 5.1.2 - 4B
- (3) Table 5.1.2 - 4I
- (4) Appendix 5D

2) Expected Tendon Force at 40 Years

Extrapolation of the lift off data obtained during the last surveillance using the "predicted base value" slope indicates that the average tendon force at end of plant life will be 609 kips which is 27 kips or 4.2% below the present licensing requirements of 636 kips. This is the least conservative approach to the estimate; that is, it provides the highest estimate of the 40 year tendon forces.

3) Design Requirements

From a licensing standpoint, the average tendon force will be below the present requirements by 27 kips. However, during the most severe loading combination which is design accident plus maximum earthquake, the tendon stresses will increase as required to equilibrate the applied loadings.



Investigations could be made using the requirements of the current ASME Section III, Division 2 containment code which permits the tendon stresses to increase to a value of $.9 F_{py}$. There are also other requirements related to acceptable stress and strain limits which would require consideration. Such an investigation might demonstrate the acceptability of the "as is" containment without retensioning.

The note referred to in the second question refers to the requirement of ACI 318 that the effective prestress, used for design purposes, cannot exceed .6 ULT (para. 2606(b)) regardless of the calculated losses.

4.8 QUESTION: Is the current tendon sampling method representative? How does it compare with other statistical methods?

RESPONSE: Of the 160 tendons in the containment, 22 will be tested in the retest. The following table divides the tendons into three categories which demonstrates the sample of 22 tendons is representative of all tendons:

- 1) Order of Tensioning (8 groups)
- 2) Position in Containment (9 - 40° segments)
- 3) Types based on straight, or curved for penetrations

(1) Order of Tensioning			(2) Position*			(3) Types		
Group Number	Number in Group	Number Tested	Group Number	Number in Group	Number Tested	Group Number	Number in Group	Number Tested
1	20	4	1		1	A	129	16
2	18	0	2		1	B	11	2
3	19	4	3		1	C	10	2
4	20	2	4	13 or 14	3	Others	9	2
5	20	3	5	in each	2			
6	19	2	6	group	1			

*based on 30° wedges beginning with Tendon 1



(1) Order of Tensioning			(2) Position*			(3) Types		
Group Number	Number in Group	Number Tested	Group Number	Number in Group	Number Tested	Group Number	Number in Group	Number Tested
7	18	3	7		2			
8	5	1	8		2			
9**	21	3	9		2			
			10		3			
			11		1			
			12		2			

* based on 30° wedges beginning with Tendon 1

**retensioned

Current USNRC requirements would require a test of 7 randomly selected tendons for each surveillance with one retested during each subsequent surveillance. This would require only 28 tendons to have been tested at least once to date rather than the 42 tested to date. Military Standard 105D (Sampling Procedures and Tables for Inspection by Attributes) indicates that for a lot or batch between 151 and 280 using single sampling for normal inspection, a test sample of 13 would be adequate.

4.9 QUESTION: Is there a tendency for relaxation to produce unbalanced forces around the perimeter of the containment?

RESPONSE: To evaluate the potential for unbalanced forces due to relaxation, the tested tendons were grouped into loads concentrated at 12 points 30° apart. Moments were taken about 30° axes and the eccentricities computed. They ranged from 0.2 to 0.52 feet. The maximum eccentricity produces an increase in concrete stress of 0.037% which is insignificant.

4.10 QUESTION: Does there appear to be any relaxation relationship based on location or tendon type?

RESPONSE: As can be concluded from a review of the data in Table 1, there does not seem to be a relation between losses and location or type.



4.11 QUESTION: What relaxation assumptions were used for the rock?

RESPONSE: Relaxation of the rock was not considered in the design since it was felt that the major effects would have occurred prior to the tensioning of the wall tendons. Refer to the response for question 4.4.

4.12 QUESTION: Were temperature gradients across the walls and their variation around the perimeter taken into account in the design? Would temperature effects contribute to variations in tendon surveillance lift-off values?

RESPONSE: The temperature gradients across the wall and dome were considered in the design. Refer to FSAR page 5.1.2-8 and Table 5.1.2-4C/Winter, 4D/Summer, 4F/Accident Temperature for $T = 286^{\circ}\text{F}$; Table 5.1.2-4I, Appendix 5D, 4G/Accident Temperature for $T = 312^{\circ}\text{F}$; and Figure 5.1.2-34. The thermal effect around the perimeter was considered in the design of the wall and dome.

For temperature effects on tendon surveillance lift-off values, see Section 6.0.

5.0 COMPARISON OF TENDON FORCES

For each of the 42 tendons surveyed previously, the tendon force over 40 years after initial stressing was predicted using the following procedure. For a specific tendon, the lock-off force at Initial Stressing is obtained from Table 1. The elastic shortening tendon force loss is calculated for the tendon from the equation $\frac{40-n}{40}$ (20.1 kips). As discussed previously, "n" is the stressing sequence number in which the tendon occurs. This elastic shortening loss is subtracted from the lock-off force, and the result is plotted at $t = 1.4$ years in Figures 4-1 through 4-42. The total long term losses in Table 2 are subtracted from this force to arrive at Upper Bound, Lower Bound, and Base Value



tendon forces. From Table 2, the Upper and Lower long term losses are used to obtain the Lower Bound and Upper Bound tendon forces, respectively. The values obtained for the Upper Bound, Lower Bound, and Base Value tendon forces are plotted at $t = 1.52$, $t = 2.4$, and $t = 41.4$ years after concrete placement; these correspond to 1000 hours, 1 year and 40 years after initial stressing, respectively. Thus, these values in conjunction with the tendon force at $t = 1.4$ years after concrete placement establish the three "predicted" curves in Figures 4-1 through 4-42.

The data points shown in the figures are the actual lift-off forces from Table 1 measured at the 6 month, 1 year, 3 year, and 8 year surveillance periods. Seven tendons in Figures 4-1 through 4-42 were retensioned at 1000 hours after initial stressing. For these tendons, the lift-off force and retensioned force at 1000 hours are also shown. An examination of these figures reveals the following:

- 1) Of the 42 surveillance tendons in Figures 4-1 through 4-42, 7 were retensioned at 1000 hours after initial stressing of all the tendons. The 7 retensioned tendons are 121 (Figure 4-5), 73 (Figure 4-6), 36 (Figure 4-7), 111 (Figure 4-9), 126 (Figure 4-31), 32 (Figure 4-41), and 117 (Figure 4-42). The "predicted" curves for these seven tendons are based on the lock-off forces at initial stressing and do not reflect retensioning.

For the remaining 35 tendons in Figures 4-1 through 4-42, a total of 46 surveillance data points exist. Using this data, the average value by which measured lift-off forces are below those predicted (Base Value) was calculated to be about 18 kips; with a Standard Deviation (σ) of ± 22 kips. Of the measured lift-off forces, 37 are below their predicted value and 9 are

above. Of the 37 which are below, the force discrepancies range from a maximum of 65 kips to a minimum of almost zero.

- 2) Resulting from the observations in Item 1), the data for the 23 tendons retensioned at 1000 hours was considered. For each of these tendons, the force loss at 1000 hours was obtained by subtracting the measured 1000 hour lift-off forces (Attachment 4) from the Initial Stressing lock-off forces after these forces are reduced for elastic shortening. The average force loss for 21 of these tendons (data not available for 2) is calculated to be 56 kips, with $\sigma = \pm 19$. From the Base Value line in Figure 3, the loss for the 1000 hour interval is seen to be predicted at approximately 35 kips (practically all due to tendon stress relaxation). Thus, the actual force loss is 21 kips greater than predicted, with 17 of the 21 tendons exhibiting losses greater than 35 kips.
- 3) Items 1) and 2) indicate an average tendon force loss in the neighborhood of 20 kips greater than predicted. This unpredicted loss occurred as soon as 1000 hours after Initial Stressing. The Standard Deviation is also approximately ± 20 kips; and this might cast doubt on the foregoing conclusion, if it were not for the fact that more than 80% of the lift-off tests have measured forces less than predicted. The unpredicted tendon force loss of 20 kips could be due to a consistent underestimation of the precise point of lift-off using the "acoustical" method of determining lift-off. This is discussed in Section 6.0 - Possible Causes. Note that the point of lift-off would have to be prematurely estimated at only 3% below the tendon force in order to result in a 20 kip



underestimate. The ± 20 kip standard deviation would represent a ± 155 psig inaccuracy in reading the ram pressure gauge.

- 4) For purposes of evaluating the lift-off forces with respect to the design requirements, the horizontal line at 636 kips labeled "Average Design Requirements" is shown in Figures 4-1 through 4-42. This value corresponds to the minimum effective tendon stress of 144 ksi required by the FSAR and Technical Specification. Based on the loss calculations presented previously, the values of tendon lift-off force at the 10 year retest have been predicted and are given in Table 3. The average of the predicted forces is 631 kips which is 5 kips or less than 1% below the required 636 kips.

6.0

POSSIBLE CAUSES

Possible causes for measured tendon forces being less than predicted include rock and elastomeric pad creep, stress relaxation properties of the wire, premature determination of the point of lift-off and temperature effects. Due to time limitations, losses due to rock and elastomeric pad deformation at the base of the containment wall were not thoroughly evaluated, although these effects could account for a significant portion of the approximate 20 kip variation defined previously.

Tendon stress relaxation was calculated on the basis of an assumed 12% relaxation at 40 years. Inryco has indicated that this assumption would be consistent with the properties of typical materials available at the time the wire was procured for the Ginna containment. However, actual test data for the in-place wire is not available. It should be noted that an increase in this property from 12 to 13% would result in a 7.4 kip decrease in 40 year tendon force.

The determination of the point of lift-off of the tendons is influenced by the procedure used. For the Ginna containment, an acoustical method (shim-tapping) has been used, and the lift-off forces in Table 1 and Figures 4-1 through 4-42 are based on this method. However, difficulty in obtaining reproducible lift-off forces using the acoustical method has been experienced on at least one other post tensioned containment. There is reason to suspect that the inaccuracy may consistently give an underestimation of tendon force. A different lift-off procedure which uses 1/32 inch thick shims will be used for the retest at 10 years. This method is believed to be more accurate than the acoustical method. For a comparison of the differences in lift-off forces measured by these two methods, both methods will be used on the 14 tendons last surveyed at the 8 year period. Such a comparison will indicate whether or not the previous lift-off force measurements were underestimated.

Several temperature conditions were investigated for the tendons and containment wall to see if the 20 kip difference could have resulted from thermal effects.

In these investigations, it is assumed that at initial stressing the average temperature across the wall and the tendon temperature were the same. Also, it is assumed that the vertical thermal growth of the wall is a function of this average temperature.

First the small difference in linear coefficient of thermal expansion for the concrete (5.5×10^{-6} in/in/°F) and the tendon (6.5×10^{-6} in/in/°F) was determined to be insignificant. If the wall and tendon were to experience a temperature increase of 40°F, the tendon force would relax 5 kips. This 40°F increase represents the difference between the stressing environment and operating condition temperatures.



The average temperature difference between the tendon and the wall over their entire 115 foot length would have to be 24°F in order to produce a 20 kip relaxation of tendon force. Such a temperature difference over the entire tendon length does not seem likely.

The effect of local hot spots on tendons curved around the main steam and feedwater penetrations was investigated. In order to relax 20 kips, a typical tendon would have to be at a temperature 92°F above its temperature at initial stressing over a 30 foot length in the area of the penetration. The 30 foot length is the tendon length conservatively assumed to be heated by the penetration, without the benefit of a heat transfer analysis. If in fact the affected length were 15 feet, the required temperature would be 184°F. These affected length and temperature combinations would seem to rule out local hot spots as a cause. This conclusion is supported by the fact that the tendons around penetrations (48, 124, and 126) do not exhibit larger losses than the other tendons.

7.0

SUMMARY

Predictions for the results of the retest (10 years) indicate that the plant will be less than 1% below the requirement of the Technical Specification. In examining the design calculations, it is clear that the failure to meet this requirement does not at this time constitute a threat to public safety since the forces necessary to mitigate a posutlated accident are available, and the additional strains required to develop these forces would be negligible.

Calculations indicate that, using current code requirements, it might be possible to avoid retensioning tendons altogether. However, additional engineering calculations would be necessary, as a minimum to have enough information to approach the NRC for the necessary revisions to the Technical Specification.

Since the FSAR and the Technical Specification allow the retensioning of the tendons it would also be possible to simply retension to values stated in the original design.

There appears to have been an approximate 20 kip unpredicted loss of prestress occurring within the first 1000 hours of stressing the tendons. As stated in Section 6.0, there are several possible causes for the differences between predicted and measured average tendon forces. Loss behavior can be investigated further to provide additional support to the conclusion of this report.

8.0

CONCLUSIONS

On the basis of the work reported, it is concluded that the response of the containment does not indicate that there is any danger to public safety. The basic problem appears to be minor difficulties in predicting the early response of the tendons. Additional conclusions must await the data from retests and perhaps further study of the anomaly in loss behavior (see Addendum I).

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TABLES

1. SURVEILLANCE TENDON FORCES
2. 40 YEARS TENDON FORCE LOSSES
3. TEN YEAR LIFT-OFF FORCE ESTIMATE



SEQUENCE	TENDON NO.	TENDON MARK	INITIAL STRESSING*				1000 HR. RE-TENSION*				6 MONTH**				1 YEAR**				3 YEARS**				8 YEARS**			
			DATE	JACK PRESSURE (PSI)	RAM AREA (IN ²)	FORCE (KIPS)	DATE	JACK PRESSURE (PSI)	RAM AREA (IN ²)	FORCE (KIPS)	DATE	JACK PRESSURE (PSI)	RAM AREA (IN ²)	FORCE (KIPS)	DATE	JACK PRESSURE (PSI)	RAM AREA (IN ²)	FORCE (KIPS)	DATE	JACK PRESSURE (PSI)	RAM AREA (IN ²)	FORCE (KIPS)	DATE	JACK PRESSURE (PSI)	RAM AREA (IN ²)	FORCE (KIPS)
1	53	A	-	6100	129.35	789																	6/77	4900	127.6	625.2
1	133	A	-	5800	129.35	750.2																	6/77	4950	127.6	631.6
3	45	A	4/4/69	6000	129.35	776.1																	6/77	5000	127.6	638
5	17	C	-	5750	129.35	743.7								5/5/70	5000	129.3	646.5	5/16/72	5000	129.3	646.5	6/77	4950	127.6	631.6	
8	121	A1	3/28/69	5800	129.35	750.2	5/21/69	6000	129.35	776.1				5/5/70	5550	129.3	717.6									
9	73	B	-	5600	129.35	724.4	5/22/69	5600	129.35	724.4	10/30/69	5525	129.34	714.6												
11	36	HT	-	5500	129.35	711.4	5/23/69	5900	129.35	763.1	10/30/69	5550	129.34	717.8					5/16/72	5400	129.3	698.2	6/77	4970	127.6	634.2
11	75	C	3/25/69	5500	129.35	711.4								5/5/70	4850	129.3	627.1									
11	111	A2	3/29/69	5800	129.35	750.2	5/21/69	5750	129.35	743.7								5/16/72	5200	129.3	672.4					
12	159	A	4/5/69	6000	129.35	776.1												5/16/72	5200	129.3	672.4					
13	83	A	3/29/69	5750	129.35	743.7								5/5/70	5050	129.3	652.9									
14	51	A	3/30/69	5500	129.35	711.4								5/5/70	5150	129.3	665.9									
16	95	A	3/30/69	6000	129.35	776.1																6/77	4820	127.6	615.0	
17	63	A	3/30/69	5800	129.35	750.2												5/16/72	5000	129.3	646.5					
18	67	A	4/1/69	5650	129.35	730.8					10/30/69	5200	129.34	672.6												
21	100	A	4/1/69	5800	129.35	750.2								5/5/70	5100	129.3	659.4	5/16/72	4850	129.3	627.1					
22	18	B	4/1/69	5900	129.35	763.1					10/30/69	5150	129.34	666.1												
22	142	A	4/1/69	5600	129.35	724.3					10/30/69	5150	129.34	666.1								6/77	4800	127.6	612.5	
23	60	A	4/1/69	5550	129.35	717.9								5/5/70	5000	129.3	646.5									
23	96	A	4/1/69	5850	129.35	756.7					10/30/69	5475	129.34	708.1												
23	140	B2	4/1/69	5550	129.35	717.9								5/5/70	5000	129.3	646.5									
24	58	B1	4/1/69	5500	129.35	711.4					10/30/69	5000	129.34	646.7												
24	138	A	-	5900	129.35	763.1												5/16/72	5000	129.3	646.5					
27	8	A	4/2/69	5800	129.35	750.2					10/30/69	5425	129.34	701.6				5/16/72	5200	129.3	672.3	6/77	4800	127.6	612.5	
27	132	A	4/2/69	5650	129.35	730.8					10/30/69	5050	129.34	653.1												
28	6	A	4/2/69	5550	129.35	717.9								5/5/70	5100	129.3	659.4									
28	50	A	4/2/69	5800	129.35	750.2												5/16/72	5000	129.3	646.5					
28	130	A	4/2/69	5600	129.35	724.3								5/5/70	5200	129.3	672.3									
29	48	C1	4/3/69	5500	129.35	711.4					10/30/69	4950	129.34	640.2												
29	84	A	4/2/69	5500	129.35	711.4					10/30/69	5300	129.34	685.5				5/16/72	4800	129.3	620.6	6/77	4600	127.6	587	
30	126	C2	4/5/69	5900	129.35	763.1	5/22/69	5900	129.35	763.1								5/16/72	5600	129.3	724.1	6/77	5400	127.6	689.0	
31	124	C1	4/3/69	5600	129.35	724.3					10/30/69	5050	129.34	653.1												
31	160	A	4/3/69	5650	129.35	730.8					10/30/69	5200	129.34	672.6								6/77	4800	127.6	612.5	
33	40	A	4/4/69	6000	129.35	776.1								5/5/70	5025	129.3	649.7									
33	76	B	4/3/69	5500	129.35	711.4												5/16/72	4800	129.3	620.6	6/77	4750	127.6	606.1	
34	30	A	4/3/69	5800	129.35	750.2												5/16/72	5000	129.3	646.5					
34	110	A	4/4/69	6000	129.35	776.1								5/5/70	5550	129.3	717.6					6/77	5150	127.6	657.1	
34	154	A	4/3/69	5750	129.35	743.7								5/5/70	5050	129.3	652.9									
36	106	A	4/3/69	5600	129.35	724.4					10/30/69	5300	129.34	685.5												
36	150	B3	4/5/69	6000	129.35	776.1												5/16/72	5100	129.3	659.4					
37	32	A3	4/3/69	5650	129.35	730.8	5/23/69	6000	129.35	776.1				5/5/70	5400	129.3	698.2									
37	117	FR	4/4/69	5750	129.35	739.9	5/22/69	5800	129.35	750.2												6/77	5200	127.6	663.5	

* LOCK OFF FORCES

** LIFT OFF FORCES

TABLE 1
SURVEILLANCE TENDON FORCES

TABLE 2

TENDON FORCE LOSSES

AT 1000 HOURS, 1 YEAR, AND 40 YEARS AFTER INITIAL STRESSING

LOSS	TENDON FORCE LOSSES (KIPS)								
	1000 HR			1 YEAR			40 YEARS		
	Base	Upper	Lower	Base	Upper	Lower	Base	Upper	Lower
Elastic Shortening	20.1	20.1	20.1	20.1	20.1	20.1	20.1	20.1	20.1
Shrinkage	±0	±0	±0	0.5	0.5	0.4	2.8	3.0	2.3
Creep	±0	±0	±0	6.4	8.0	5.4	18.5	23.1	15.7
Relaxation	34.9	40.1	29.7	52.0	59.8	44.2	89.1	102.4	75.7
TOTAL	55.0	60.2	49.8	79.0	88.4	70.1	130.5	148.6	113.8
TOTAL LONG TERM (Excludes Elastic Shortening)	34.9	40.1	29.7	58.9	68.3	50.0	110.4	128.5	93.7



TABLE 3
TEN YEAR LIFT-OFF FORCE ESTIMATE

TENDON NUMBER	FIGURE NUMBER	FORCE LAST TEST (kips)	LAST TEST (Year)	PREDICTED FORCE (kips)	REMARKS
53	4-1	625	8	621	All predicted forces are based on the Base Value predicted slope through last test data unless noted otherwise.
133	4-2	630	8	626	
45	4-3	637	8	634	
17	4-4	632	8	627	
36	4-7	632	8	679(627)	Based on predicted slope thru 3 year test data (based on 8 year test data).
95	4-13	615	8	611	
142	4-18	612	8	607	
8	4-24	612	8	655(607)	Based on predicted slope thru 3 year test data (based on predicted slope thru 8 year test data).
84	4-30	588	8	584	
126	4-31	687	8	683	
160	4-33	612	8	607	
76	4-35	605	8	602	
117	4-42	663	8	658	
110	4-37	657	8	653	
60	4-19	647	1	621	
63	4-14	647	3	630	
150	4-40	658	3	640	
158		740		650	Based on Initial Stressing and 10 year losses.
51	4-12	665	1	636	
83	4-11	653	1	624	
100	4-16	627	3	610(575)	Based on actual slope thru 1 and 3 year test data (based on predicted slope thru 3 year test data).
132	4-25	653	6 mo.	614	
				631	Average Lift Off Force in 22 Tendons tested (estimated)

FIGURES

1. TENDON LOCATION
- 1-A. TENDON STRESSING SEQUENCE
2. CONTAINMENT CONSTRUCTION HISTORY
3. PREDICTED TENDON LOSSES (BASE VALUES)
- 3-A. STRESS RELAXATION 12% WIRE
4. COMPARISON OF PREDICTED AND MEASURED TENDON FORCES OVER 40 YEARS
(INCLUDES FIGURE 4-1 THROUGH FIGURE 4-42)

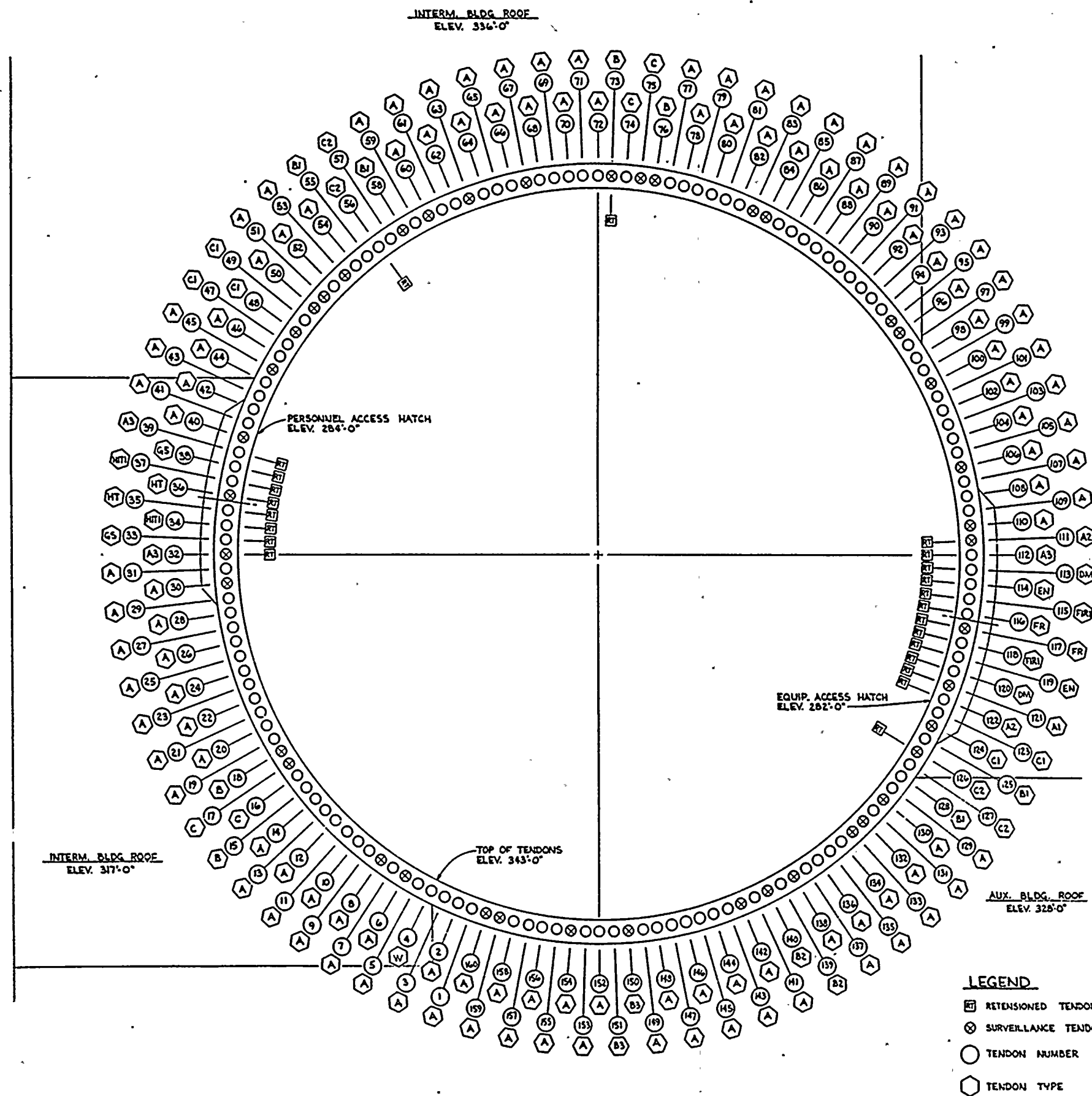


FIGURE 1
TENDON LOCATION

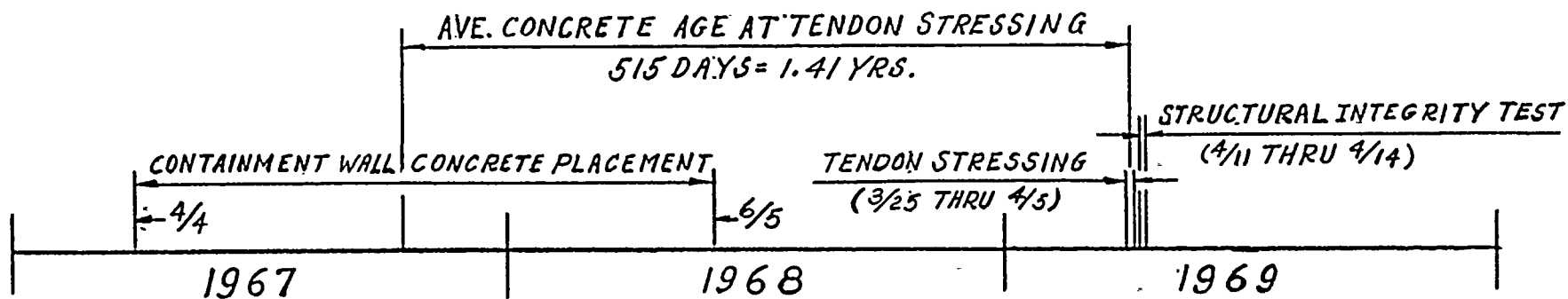
FIGURE 1-A
TENDON STRESSING SEQUENCE

<u>Jack #1</u>	<u>Jack #2</u>	<u>Jack #3</u>	<u>Jack #4</u>
13	53	93	133
149	25	69	105
1	45	81	127
21	57	101	137
17	65	87	145
9	61	89	141
5	49	85	129
157	44	77	<u>121</u>
153	26	<u>73</u>	108
151	27	71	107
185	<u>36</u>	75	<u>111</u>
139	<u>43</u>	79	128
3	47	85	125
7	51	87	131
11	55	91	135
15	59	95	139
19	63	99	143
23	67	103	147
24	68	104	148
22	66	102	146
20	64	100	144
18	62	98	142
16	60	96	140
14	58	94	128
12	<u>56</u>	92	136
10	<u>54</u>	90	134
8	52	88	132
6	50	86	130
4	48	84	123
2	46	82	<u>126</u>
160	41	80	124
158	42	78	<u>122</u>
156	40	76	<u>112</u>
154	30	74	110
152	28	72	109
150	29	70	106
<u>32</u>	31	<u>113</u>	<u>117</u>
<u>33</u>	<u>37</u>	<u>114</u>	<u>118</u>
<u>34</u>	<u>38</u>	<u>115</u>	<u>119</u>
<u>35</u>	<u>39</u>	<u>116</u>	<u>120</u>

___ = Retensioned at 1000 hours.

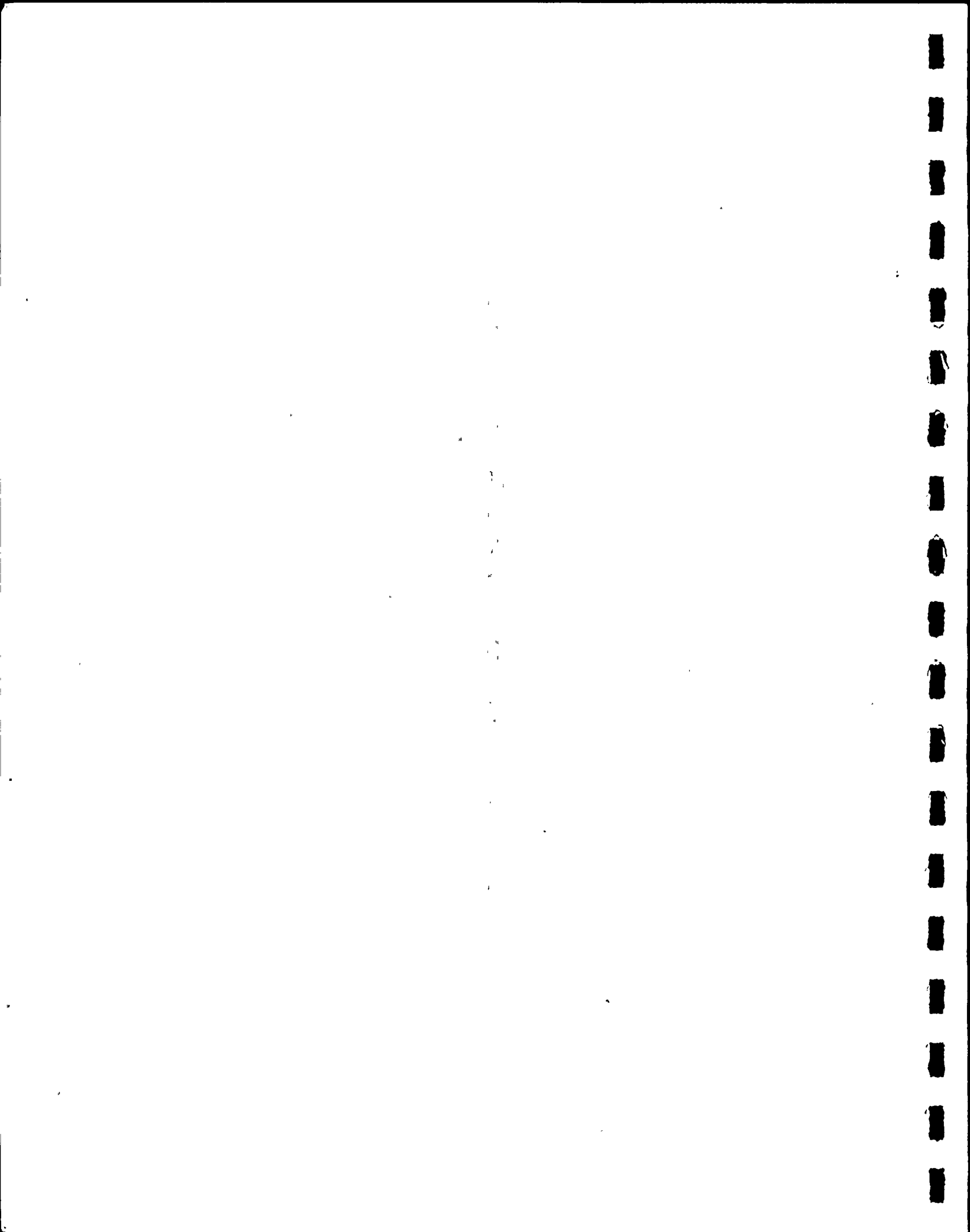
FIGURE 1-A
TENDON STRESSING SEQUENCE

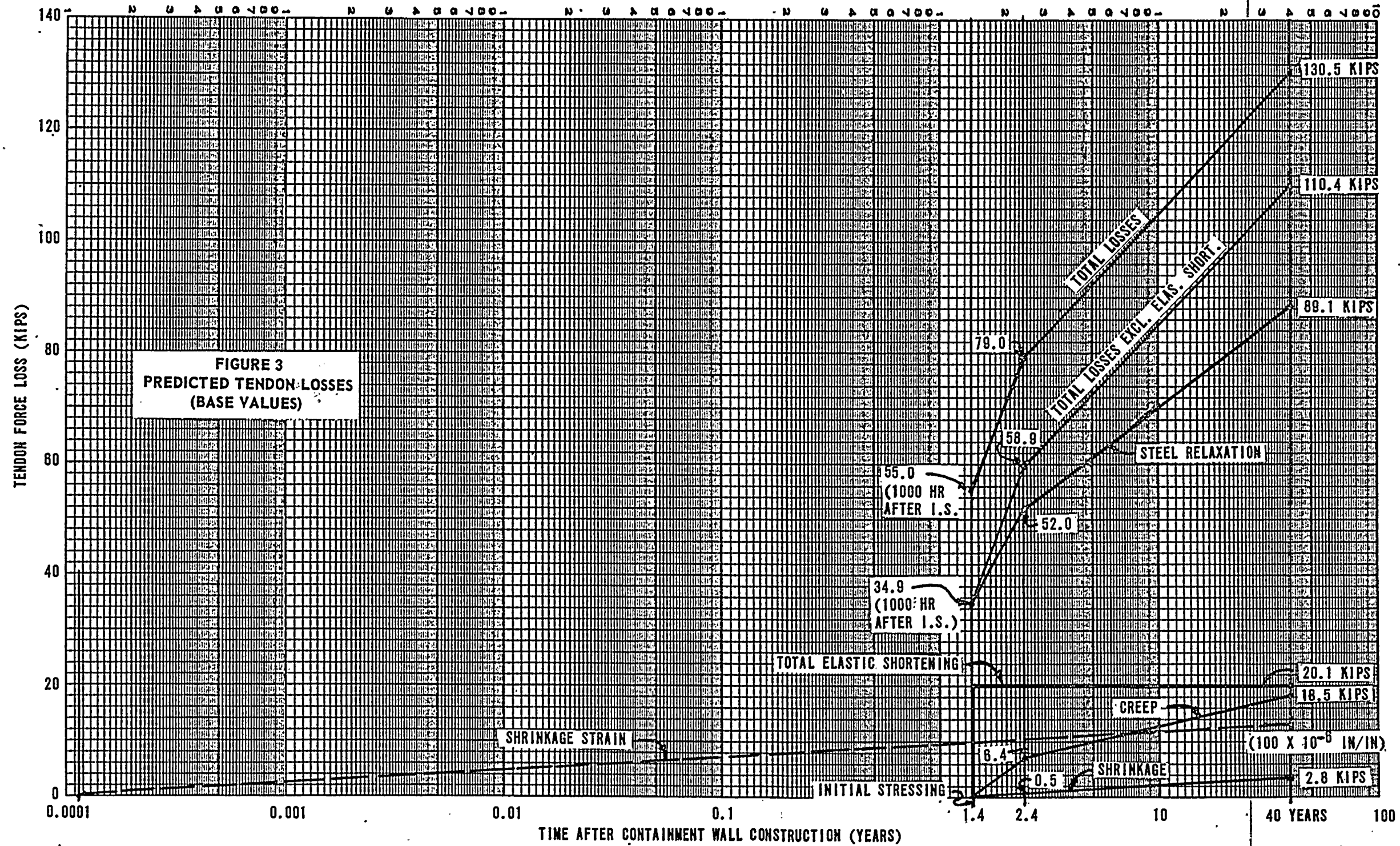




R. E. GINNA STATION
 CONTAINMENT CONSTRUCTION
 CONCRETE PLACEMENT-TENDON STRESSING-
 STRUCTURAL INTEGRITY TEST

FIGURE 2
 CONTAINMENT CONSTRUCTION HISTORY





**FIGURE 3
PREDICTED TENDON LOSSES
(BASE VALUES)**

NO. 340-LS10 DIETZGEN GRAPH PAPER
SEMI-LOGARITHMIC
5 CYCLES X 10 DIVISIONS PER INCH

EUGENE DIETZGEN CO.
MADE IN U. S. A.

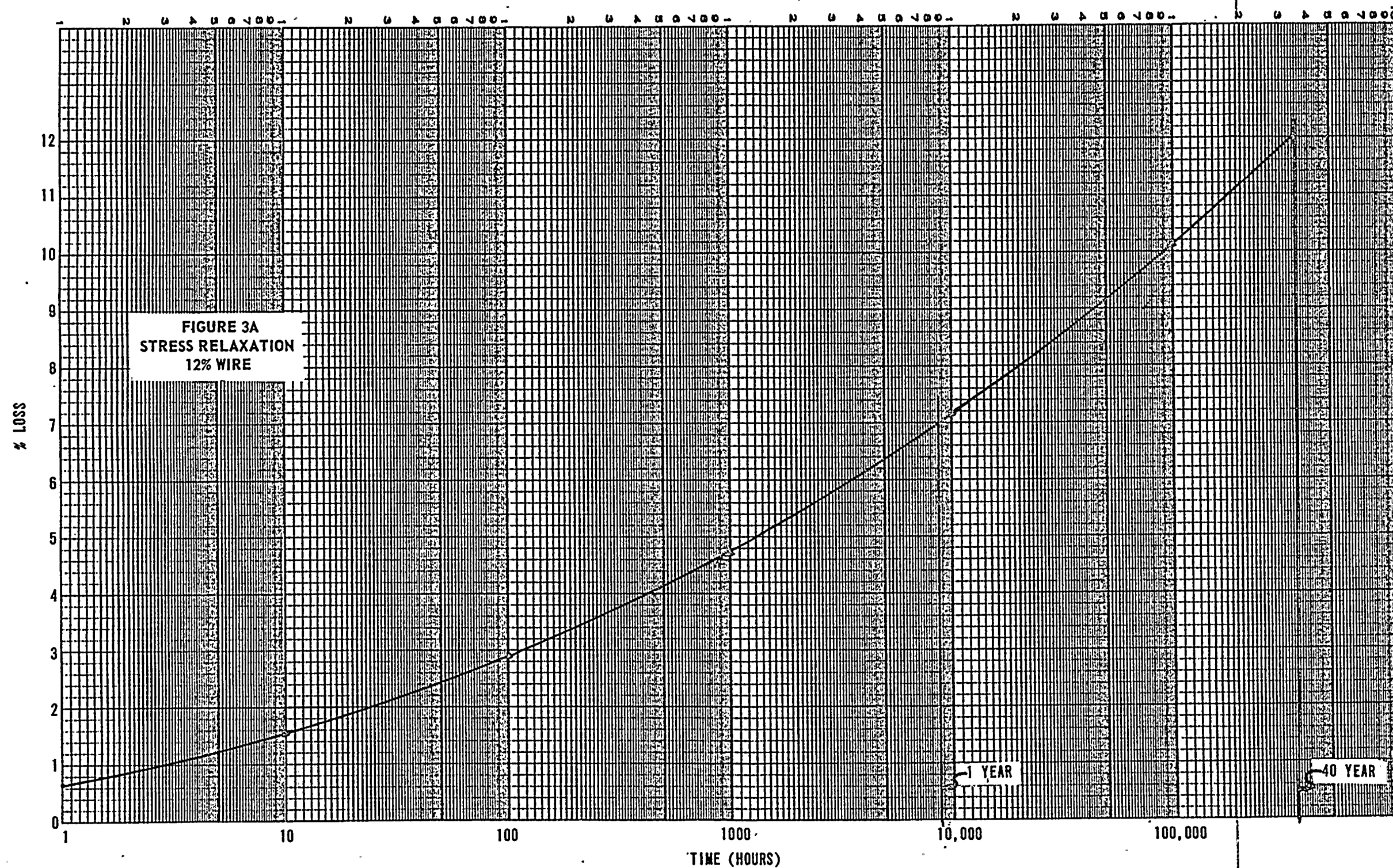
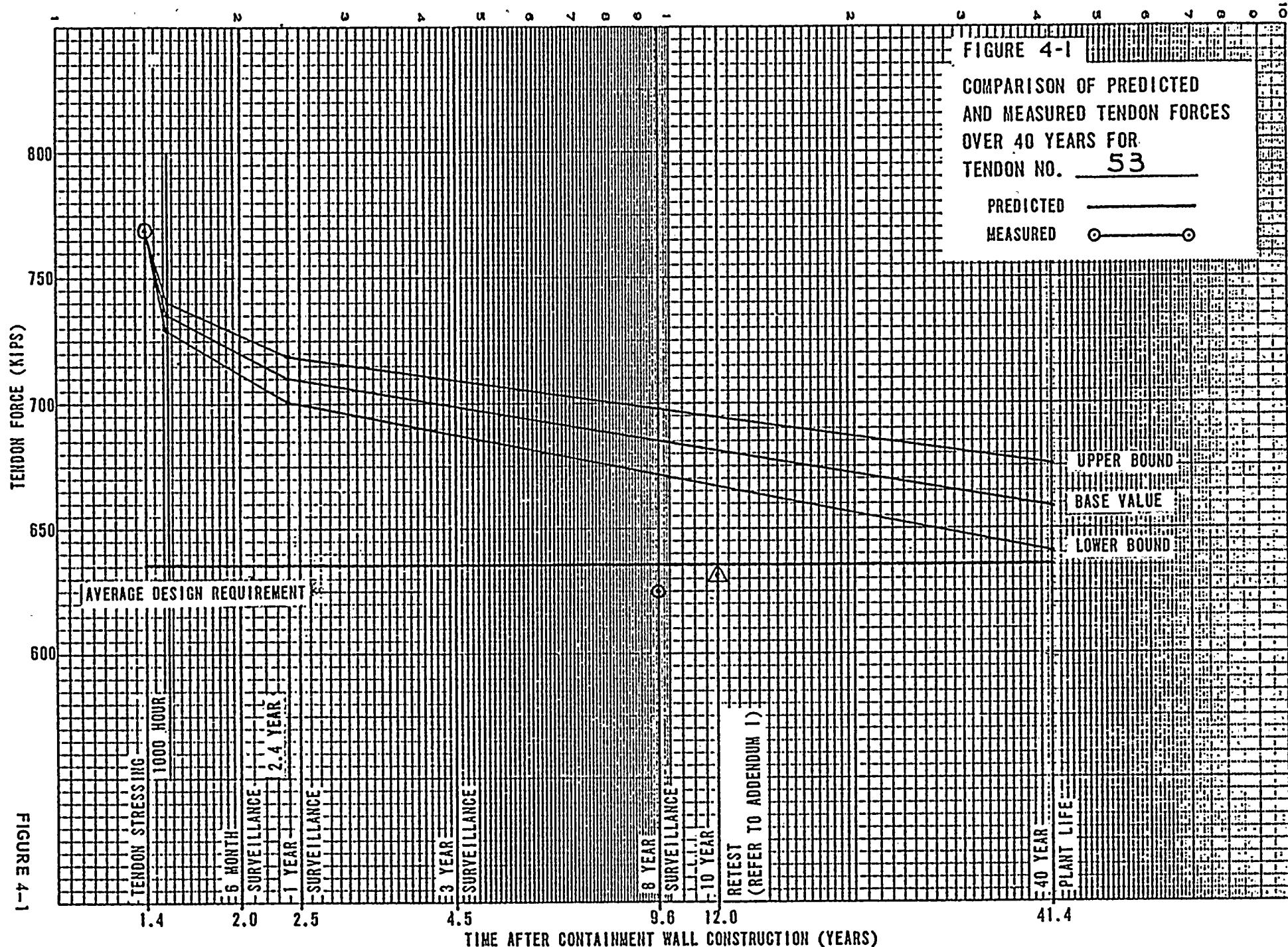


FIGURE 3A
STRESS RELAXATION
12% WIRE





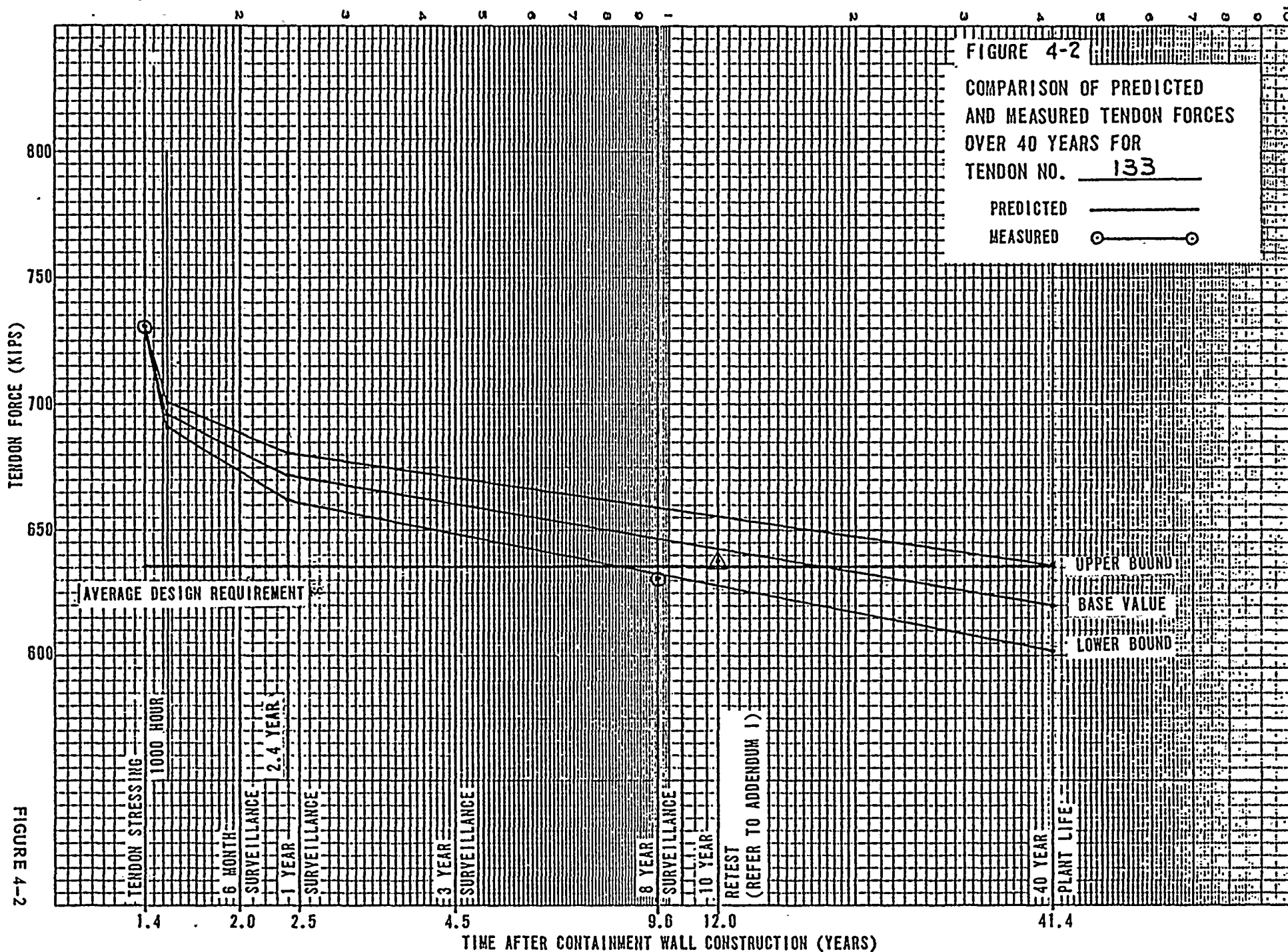
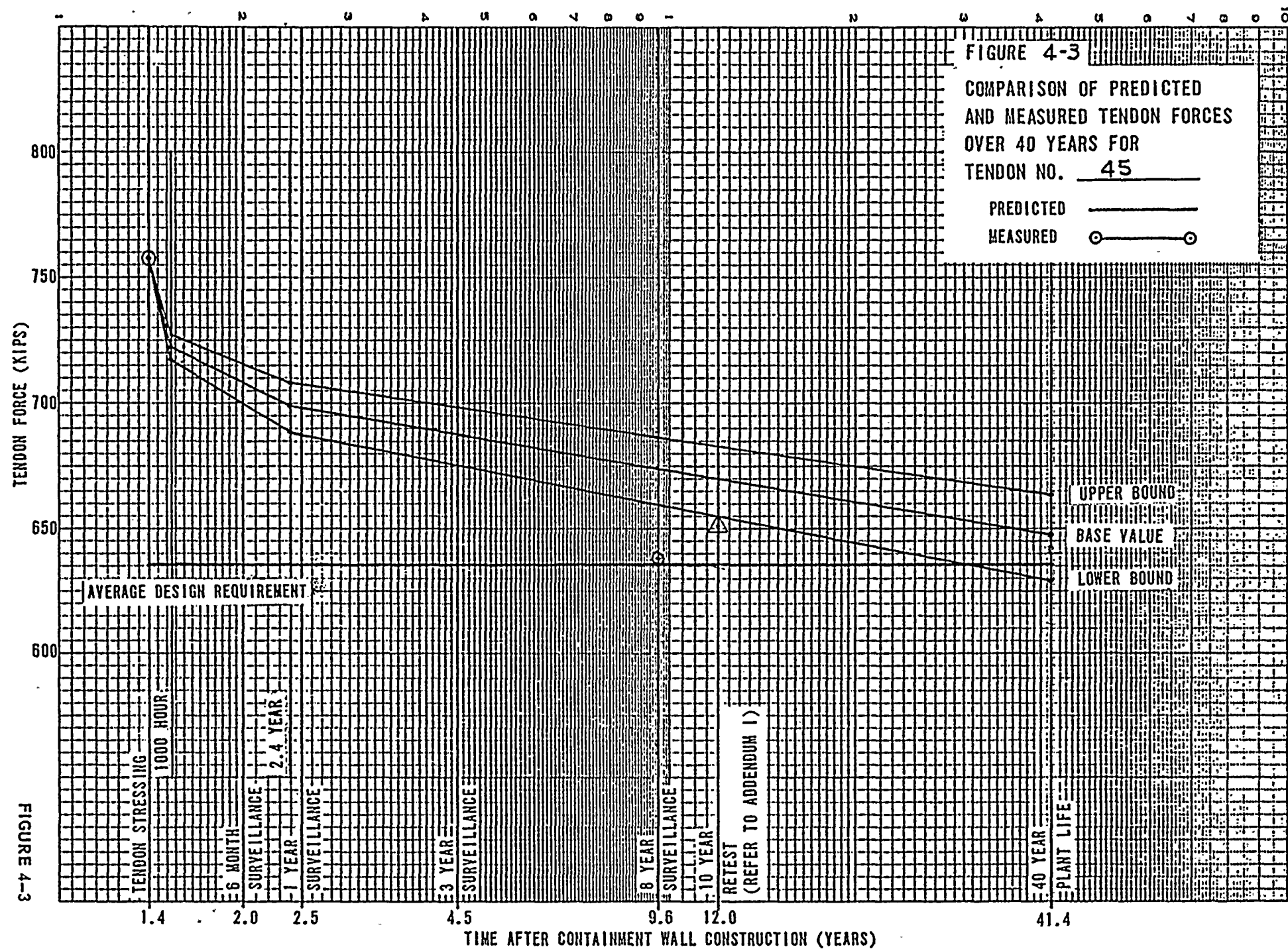
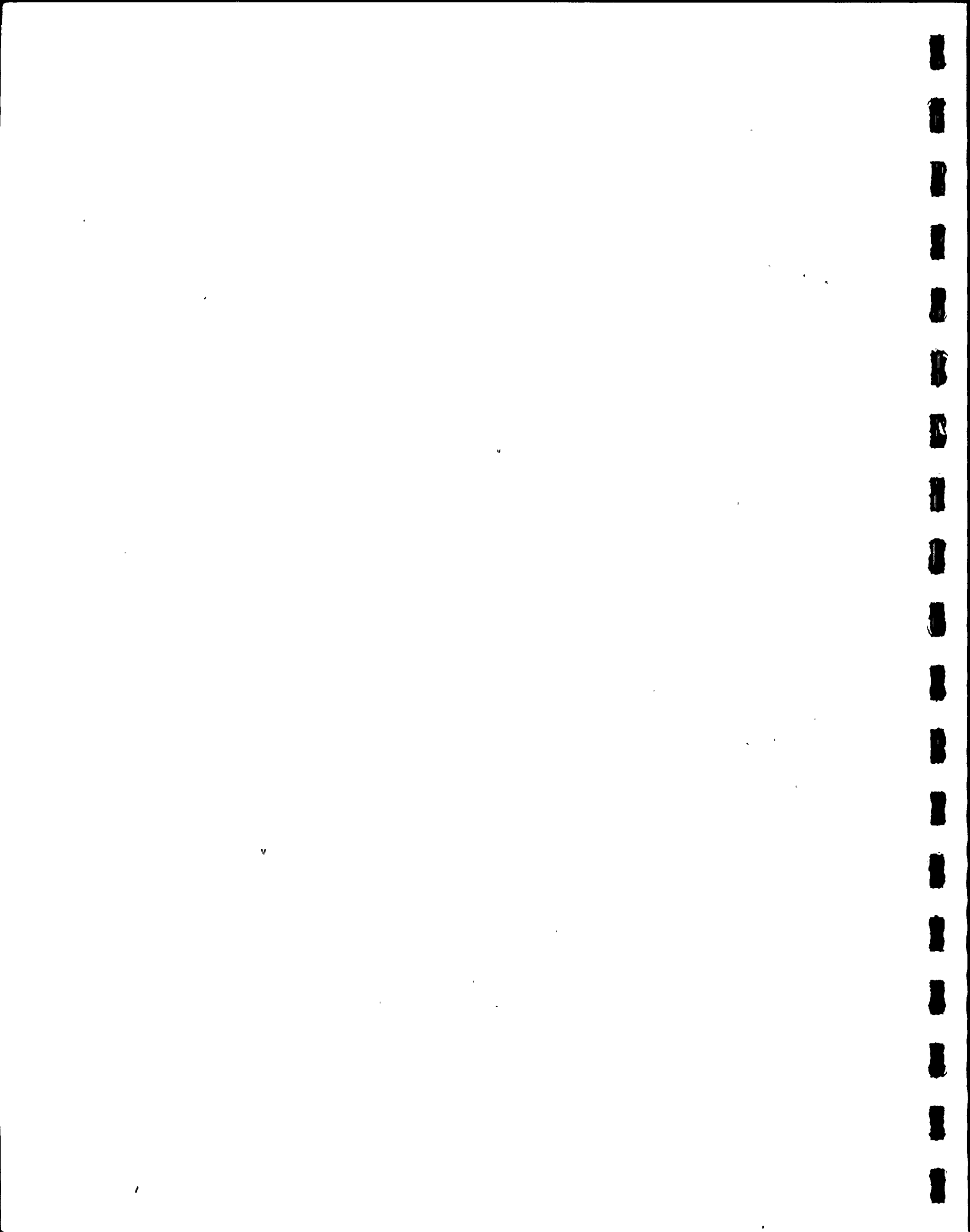
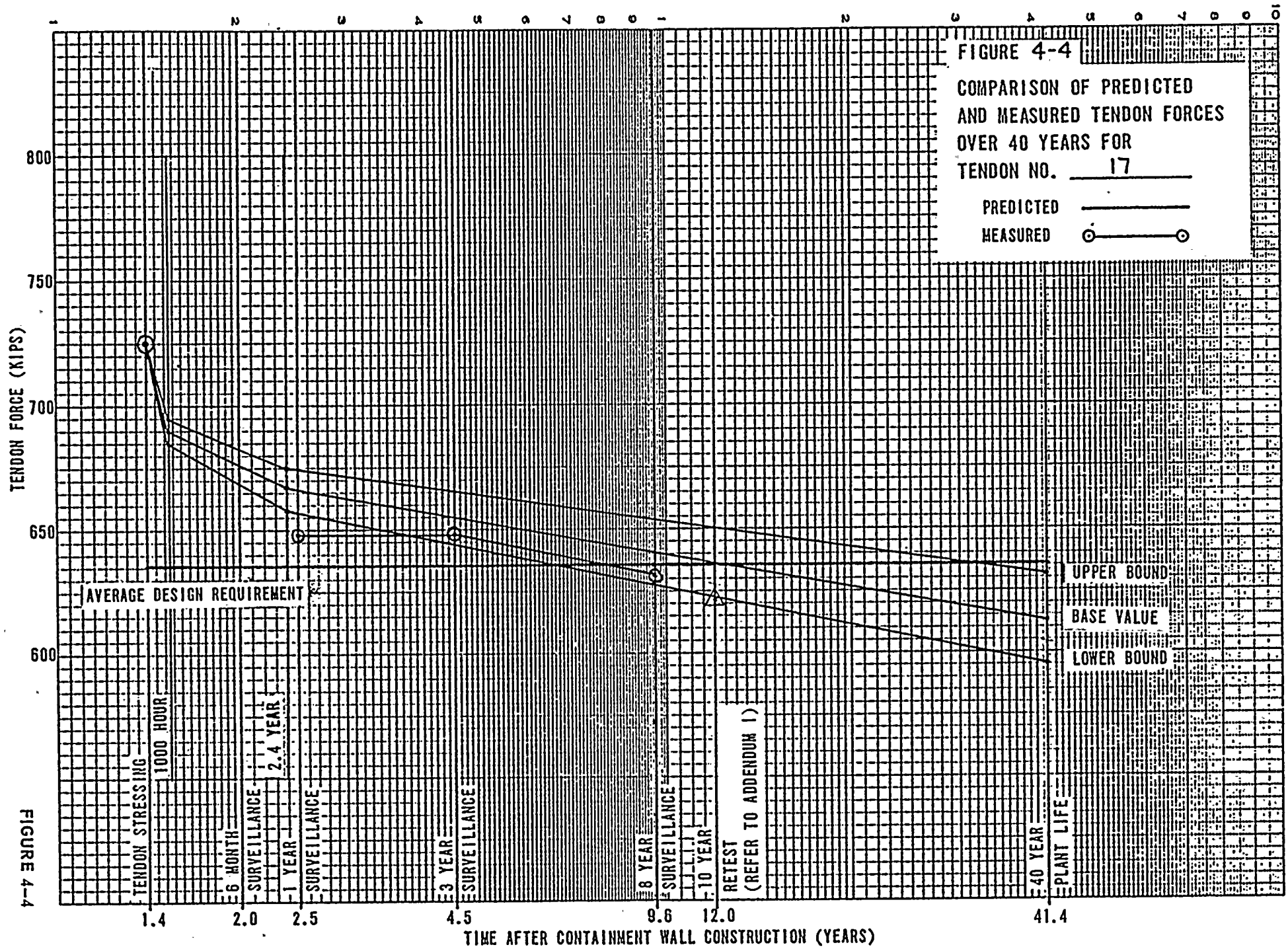




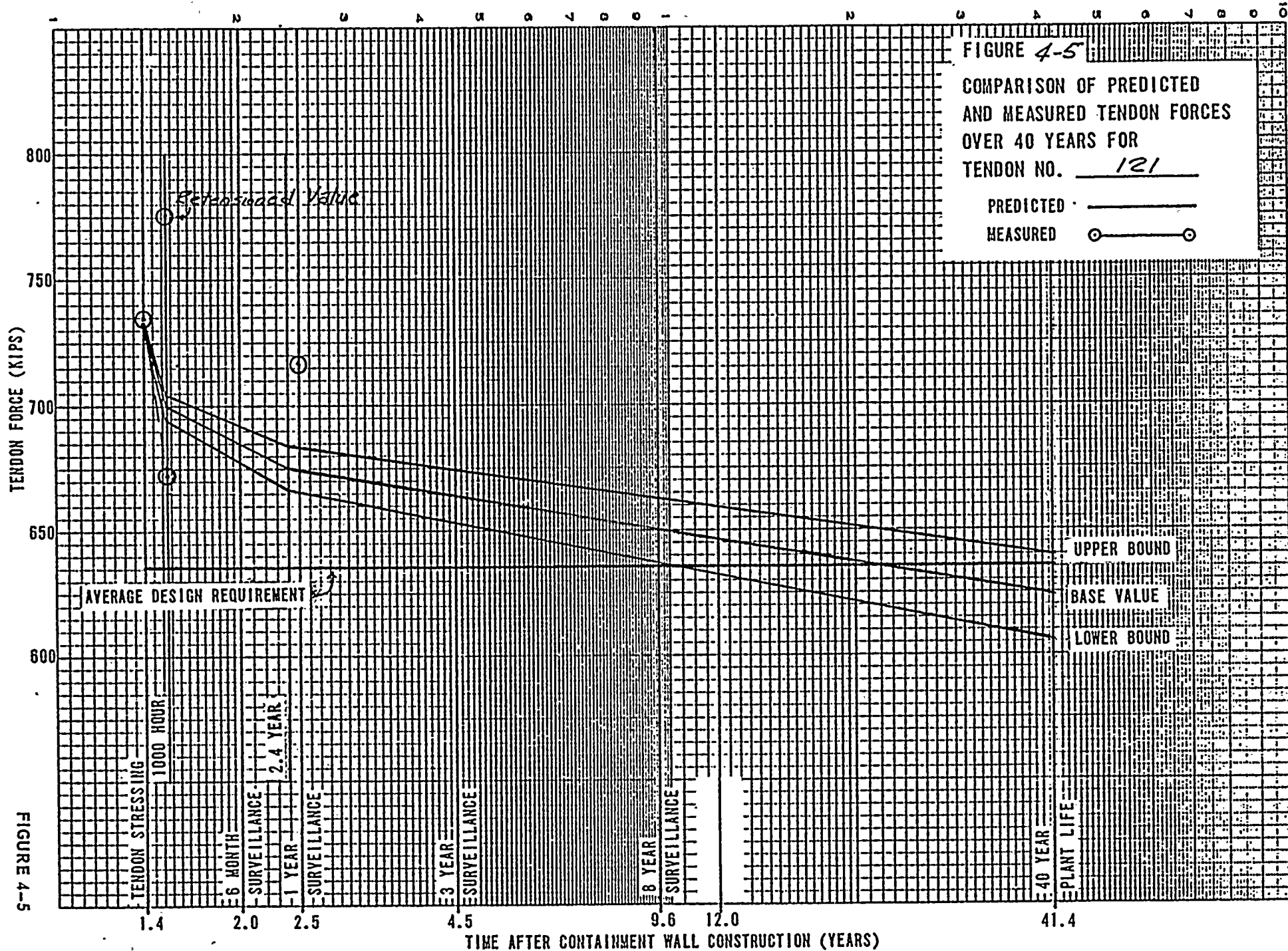
FIGURE 4-3



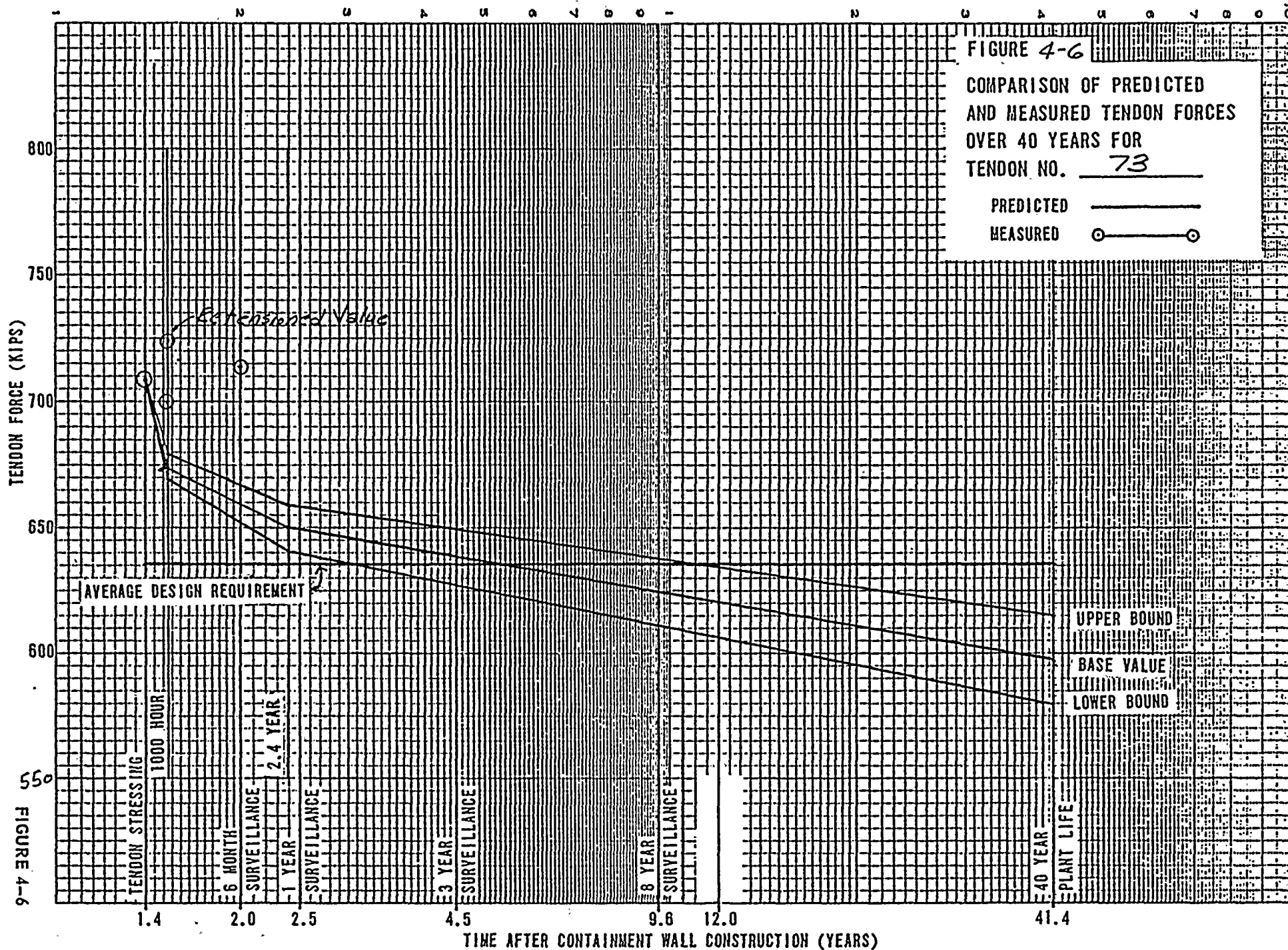




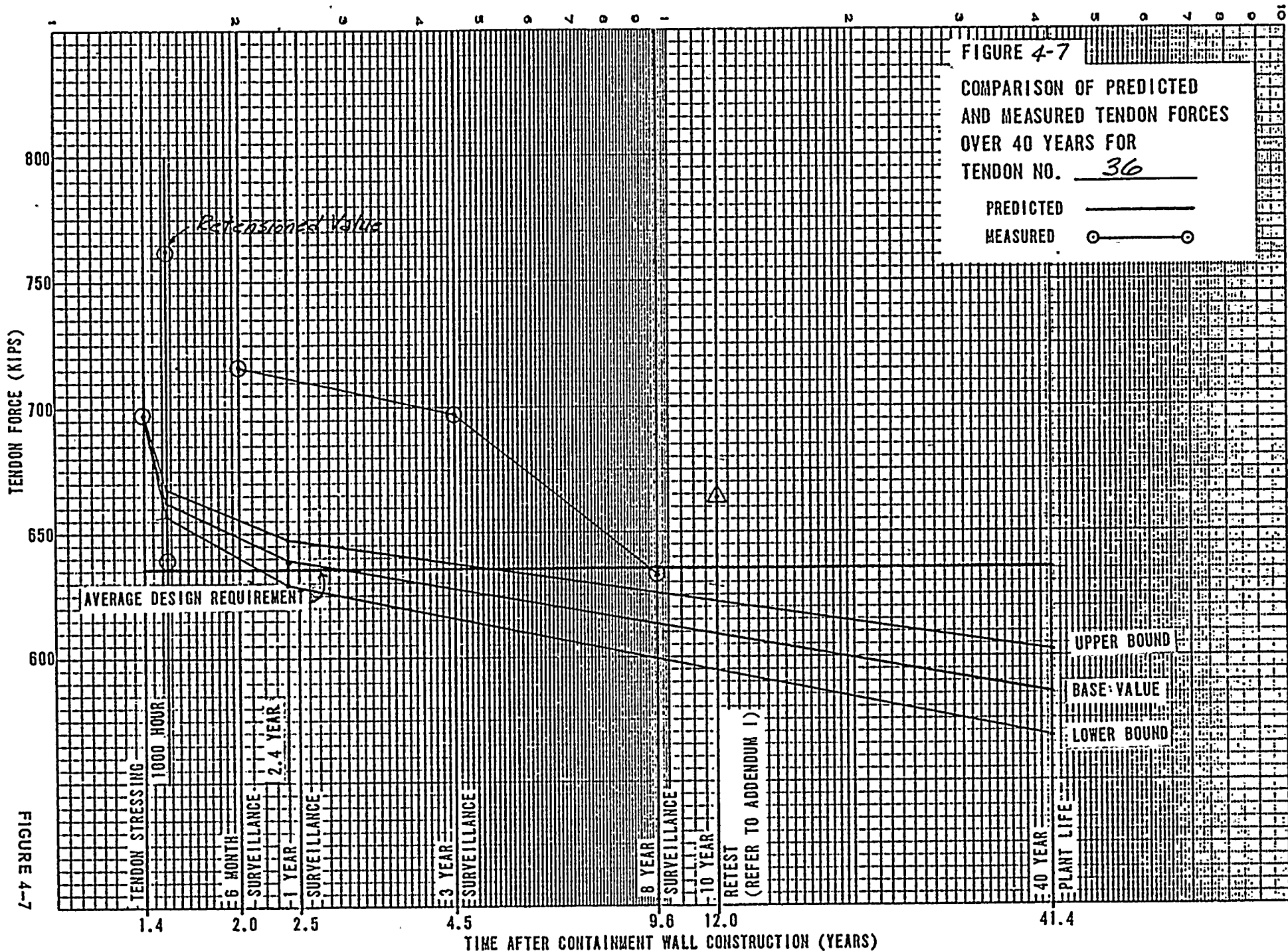








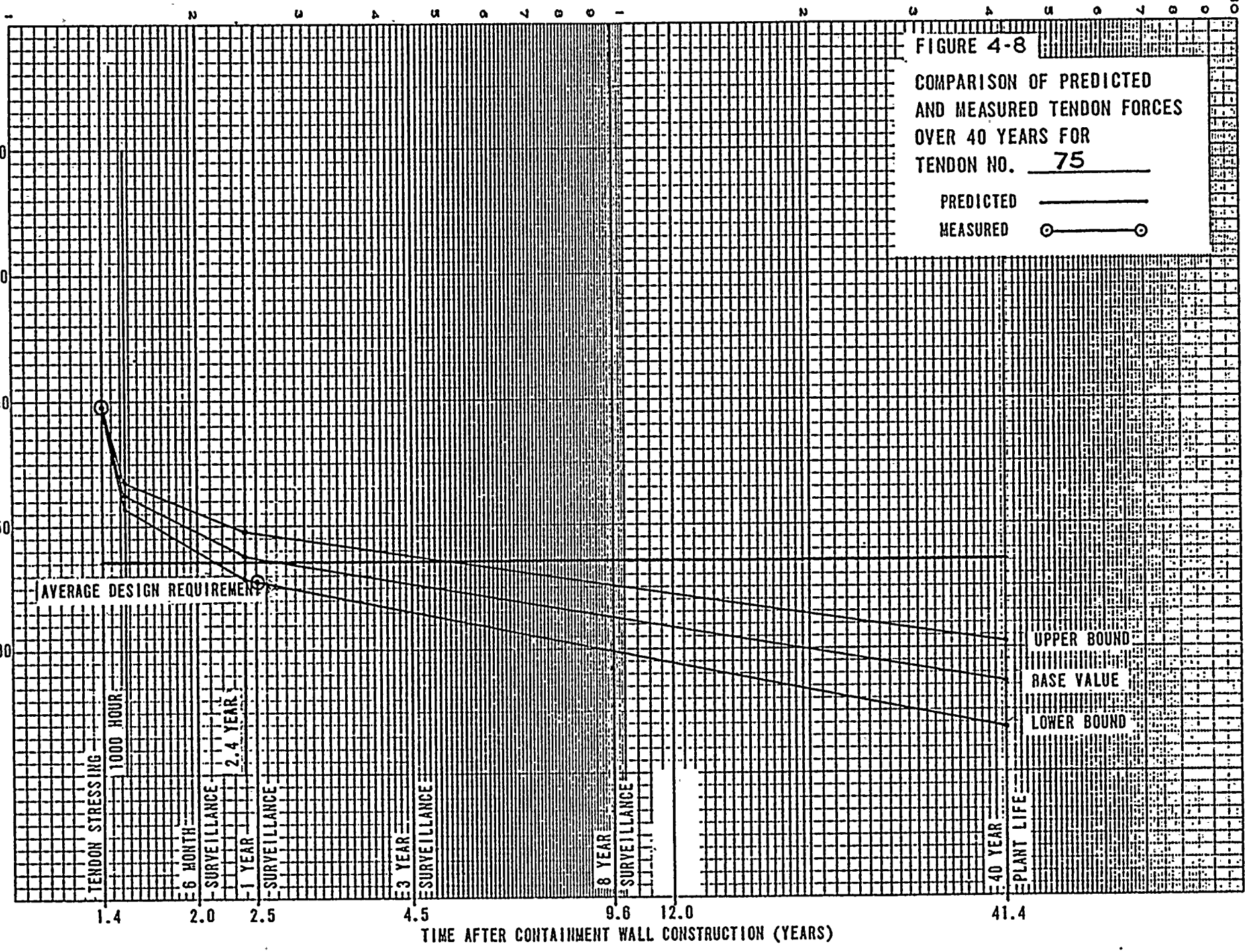






TENDON FORCE (KIPS)

FIGURE 4-8



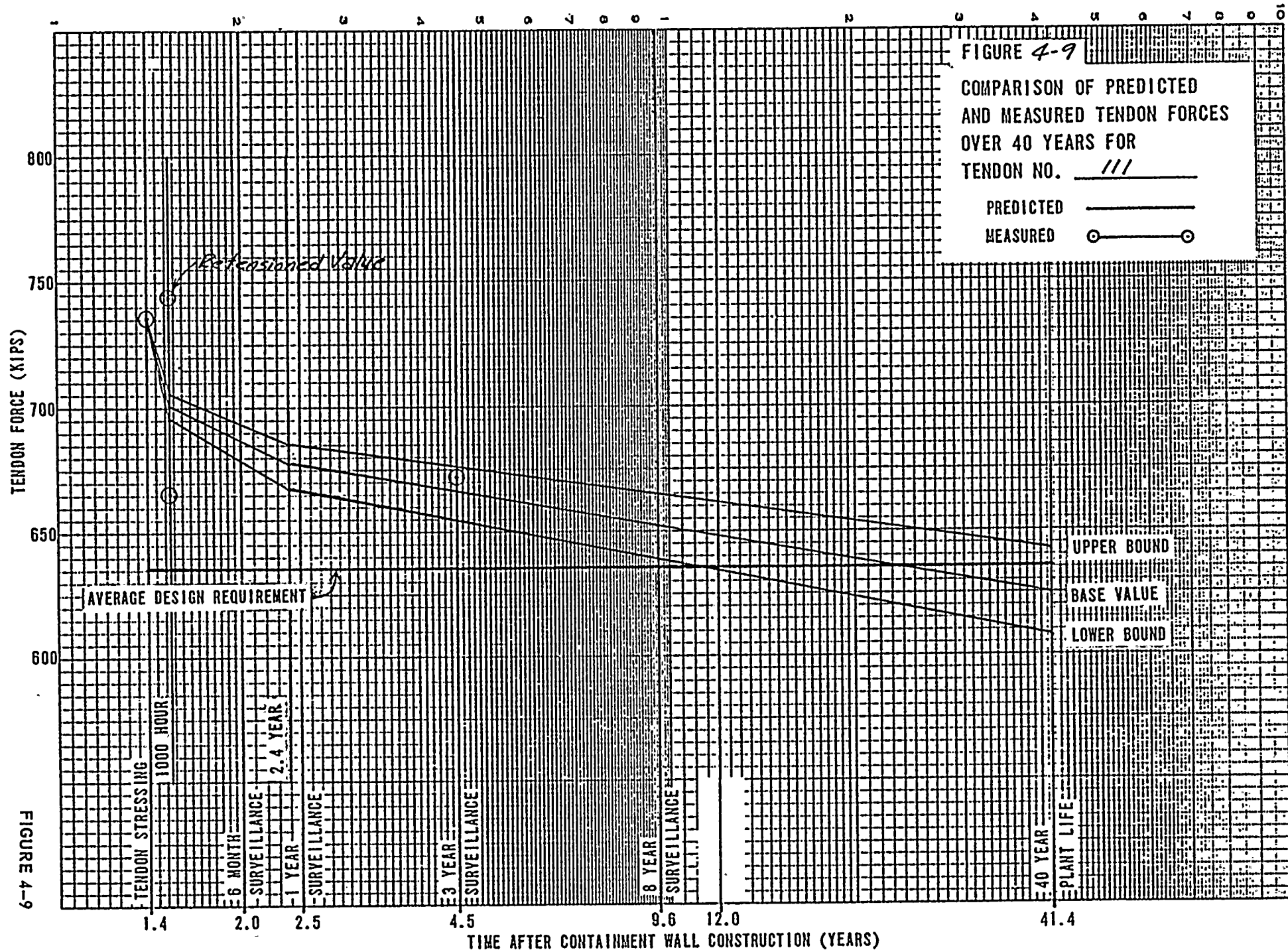
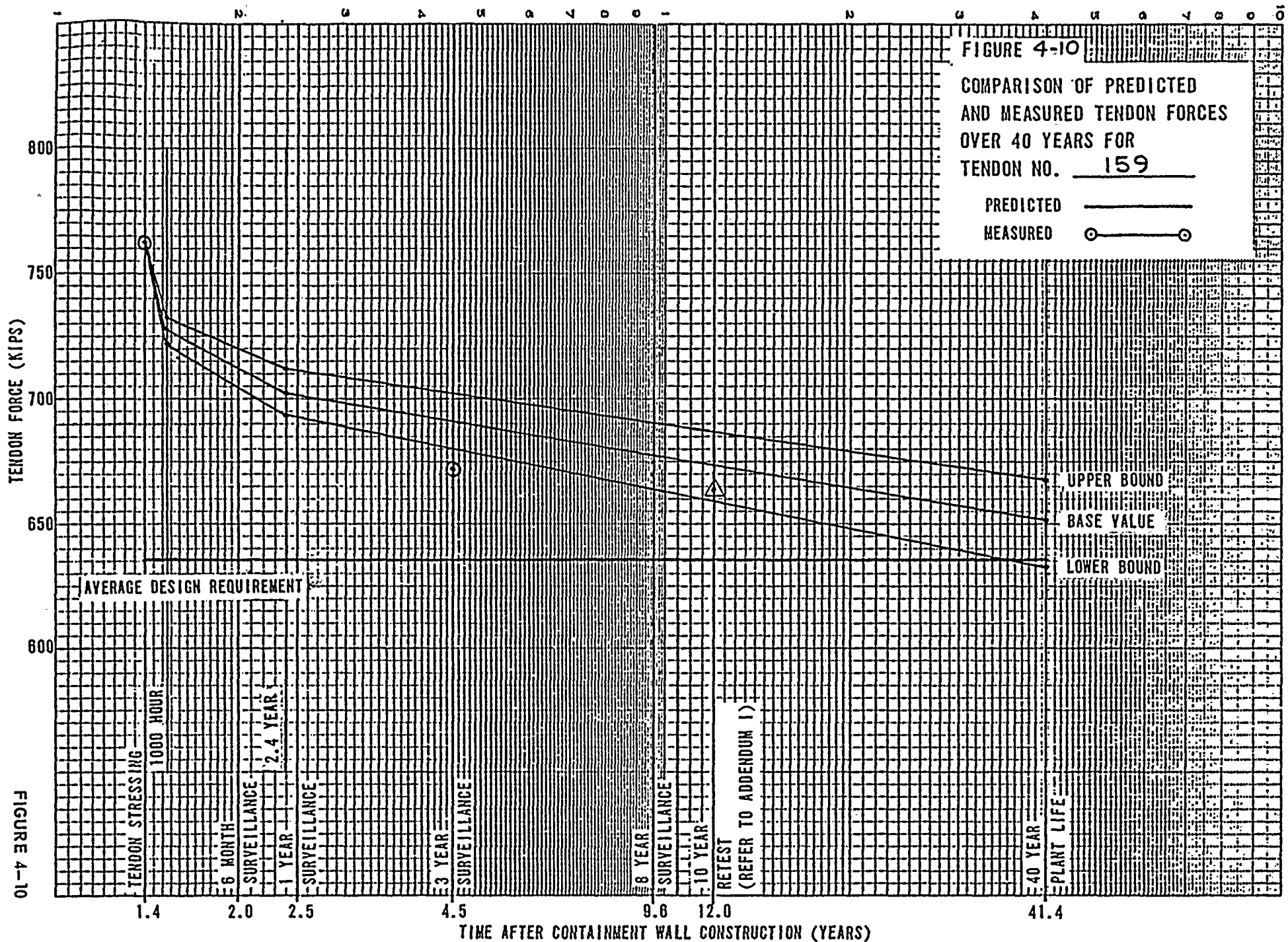


FIGURE 4-9







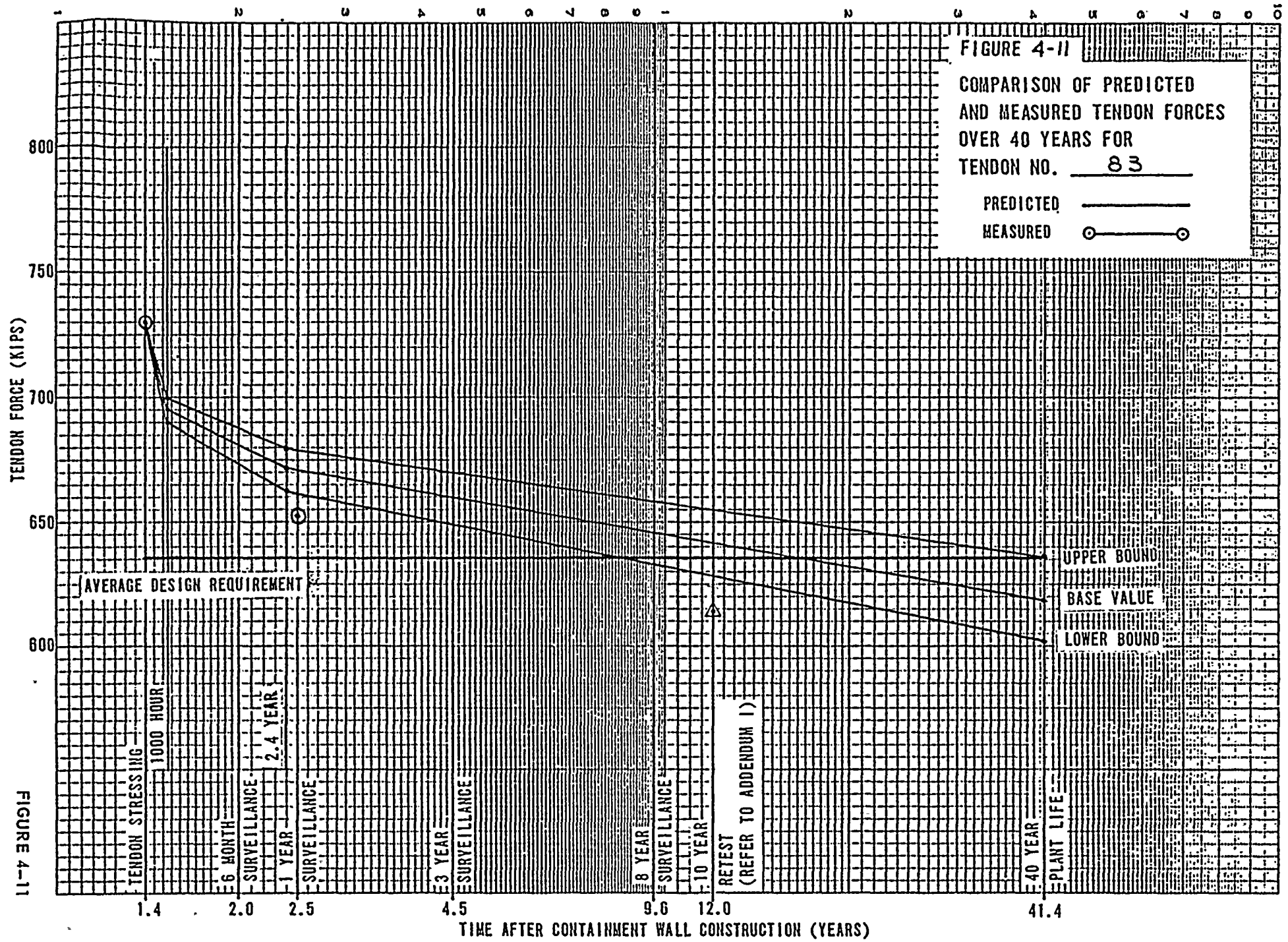
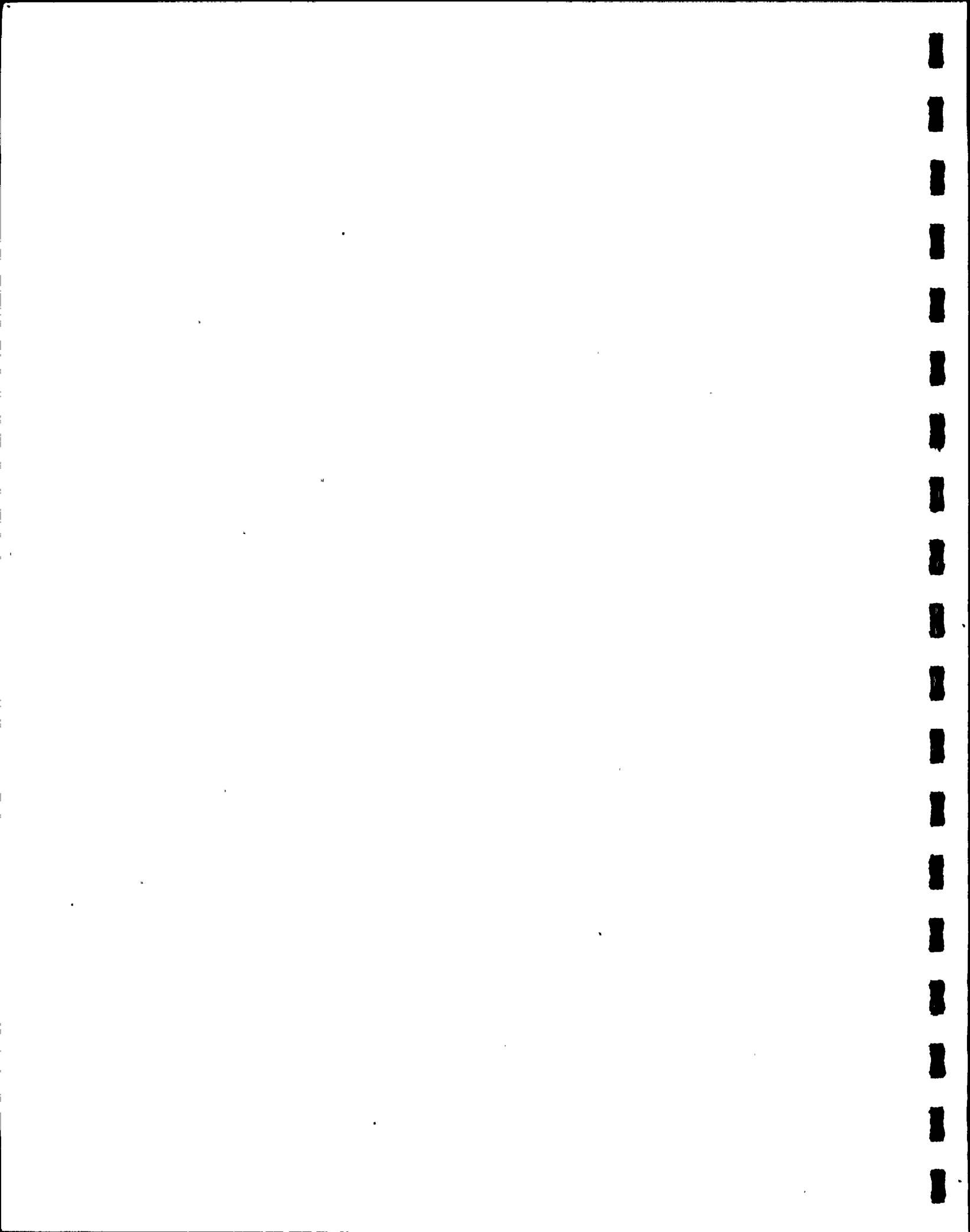
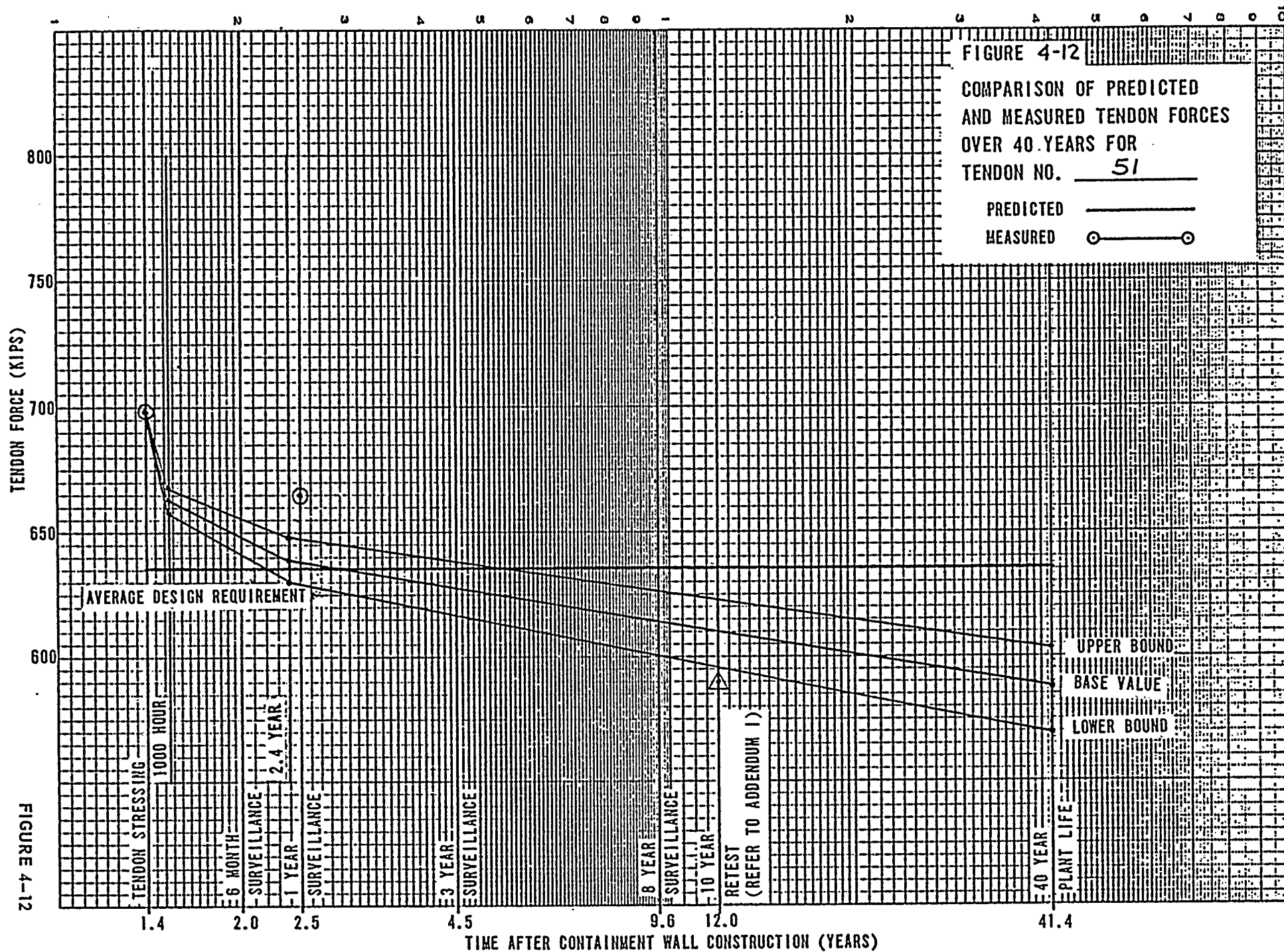


FIGURE 4-11





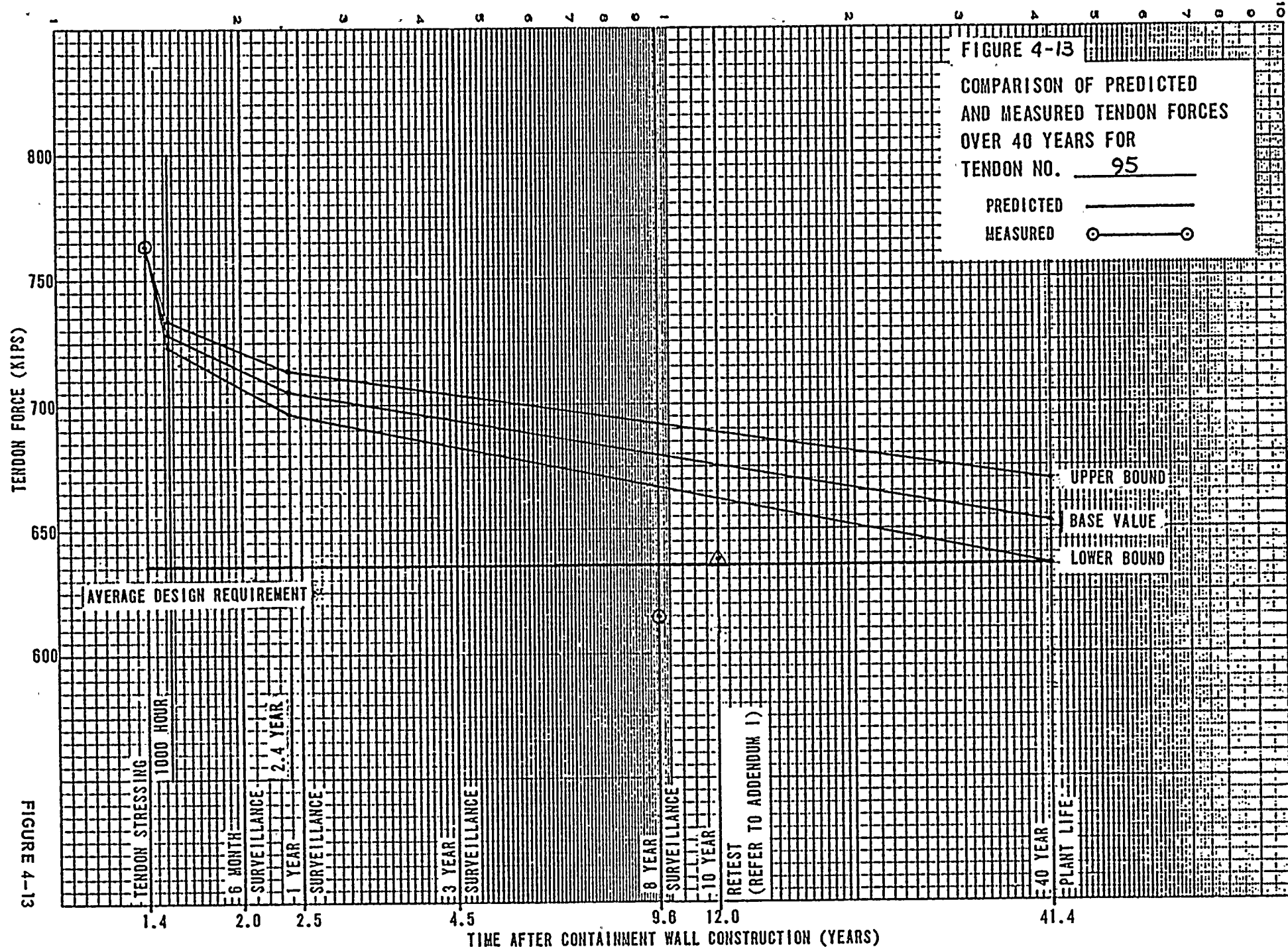
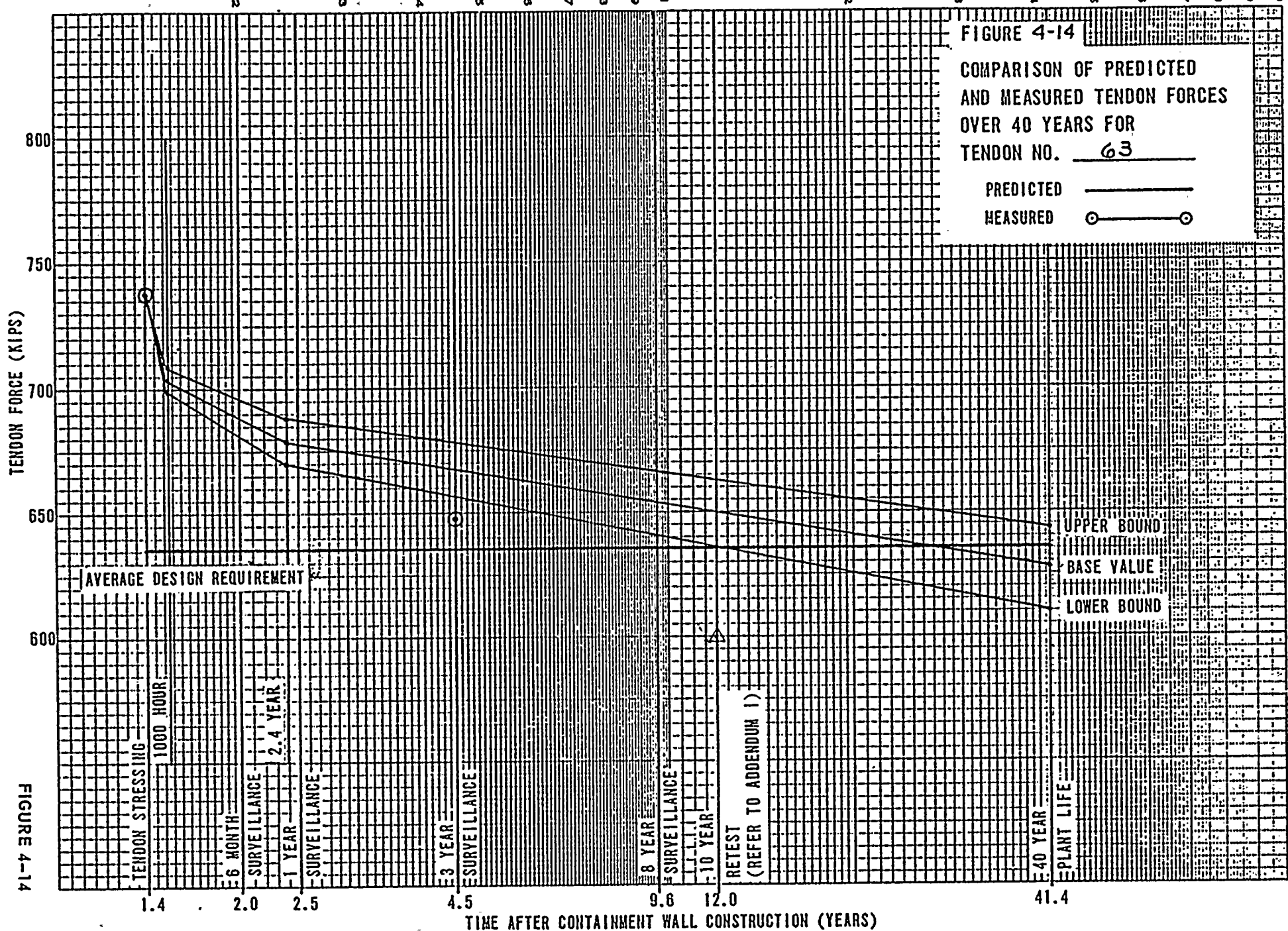
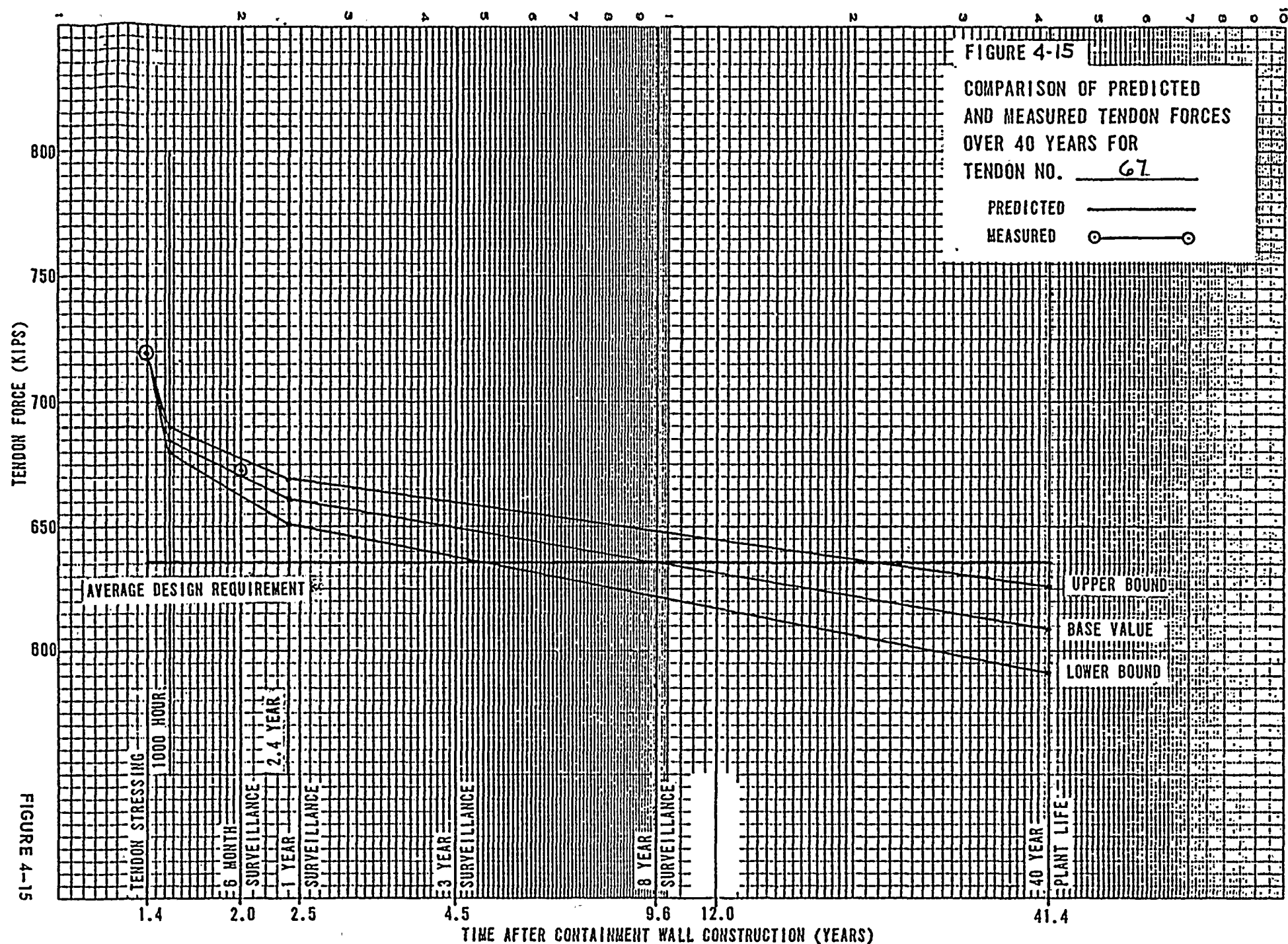


FIGURE 4-13

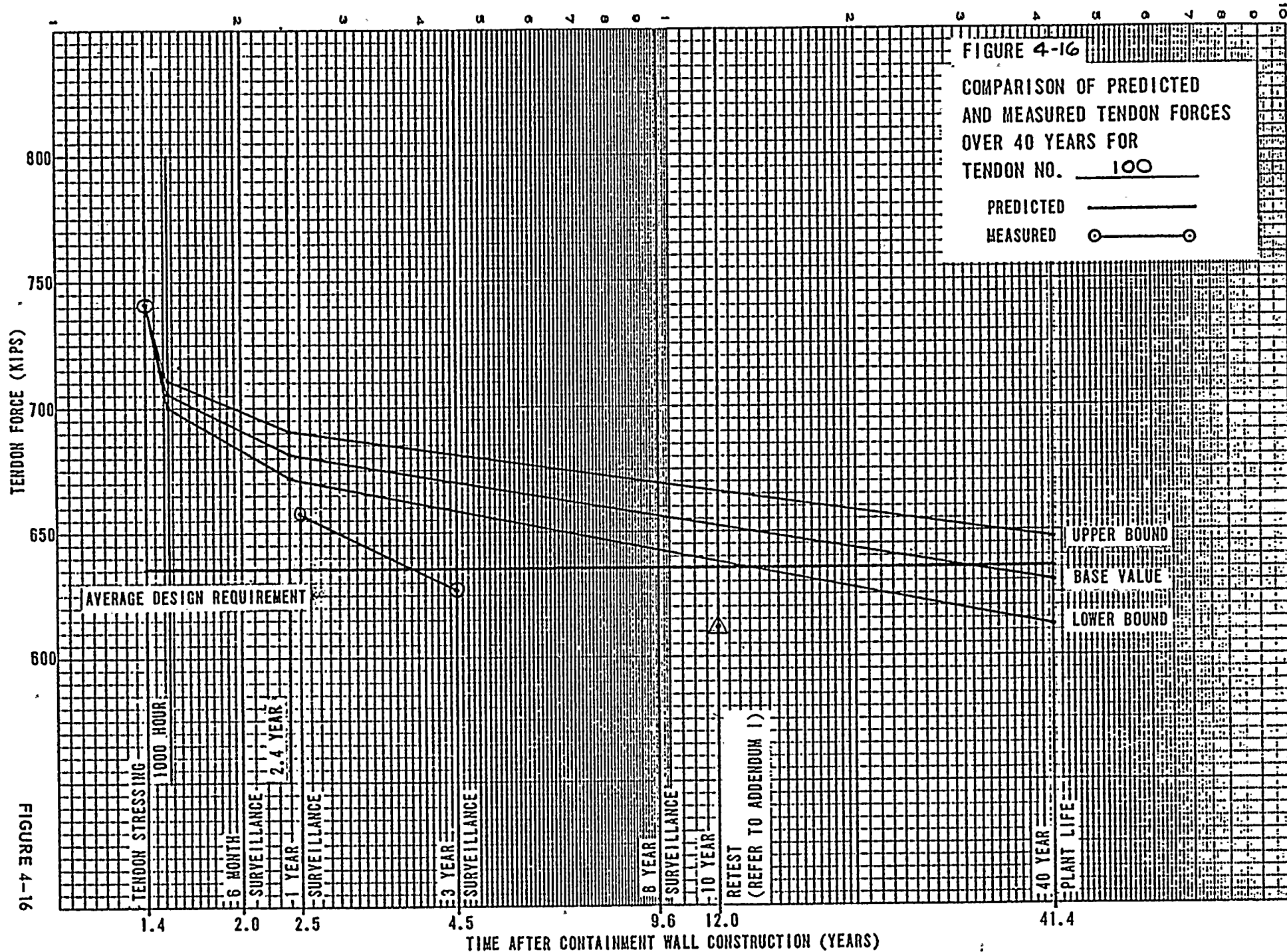












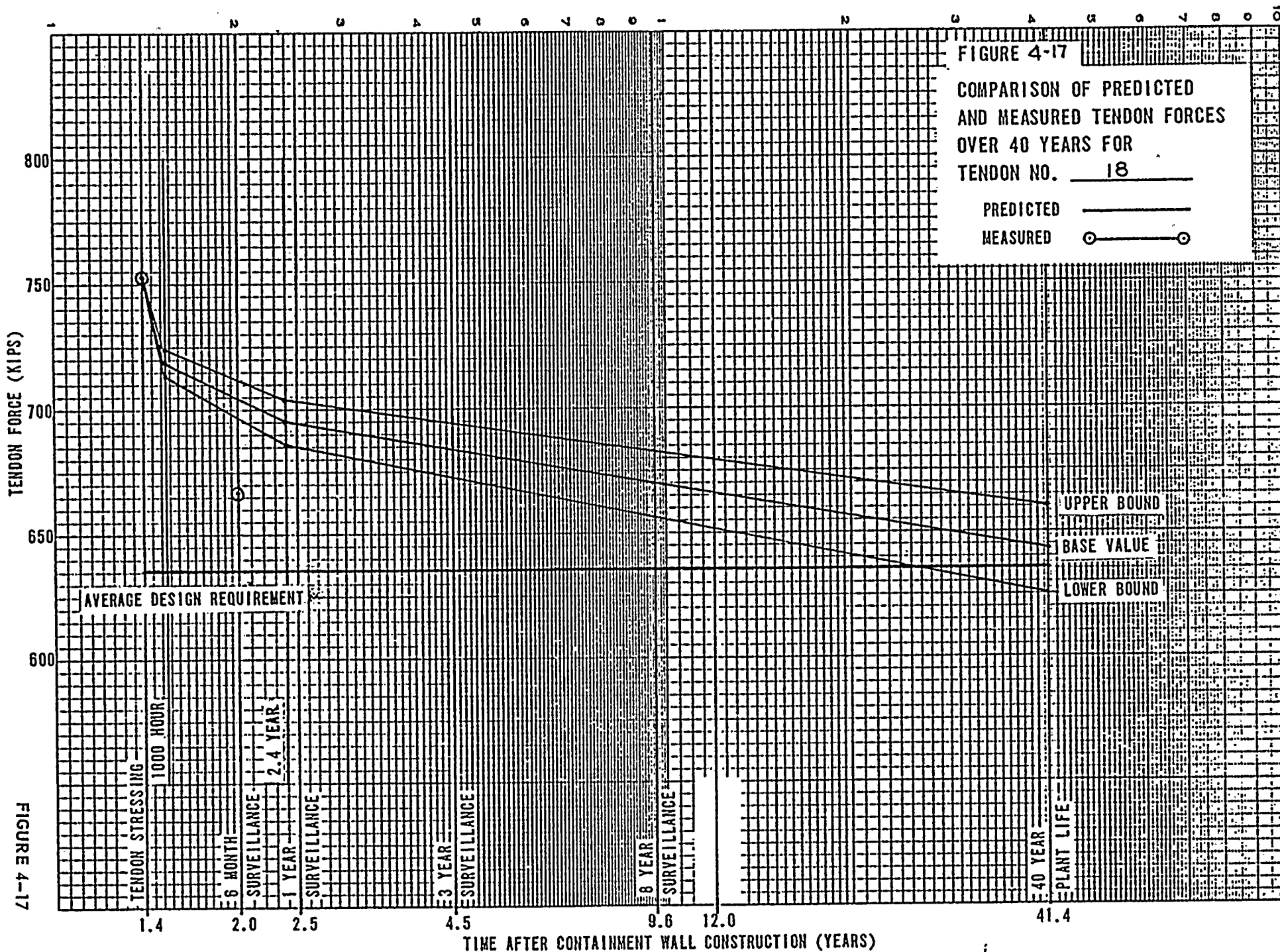


FIGURE 4-17

COMPARISON OF PREDICTED
AND MEASURED TENDON FORCES
OVER 40 YEARS FOR
TENDON NO. 142

MEASURED



AVERAGE DESIGN REQUIREMENT

UPPER BOUND

BASE VALUE

LOWER BOUND

1000 HOURS

三

2.4 YEAR

ILLANCE-

R ~~_____~~

ILLANCE

8

CHALLENGE

BAR

RETEST

TEST
(REFER TO ADDENDUM I)

40 YEAR

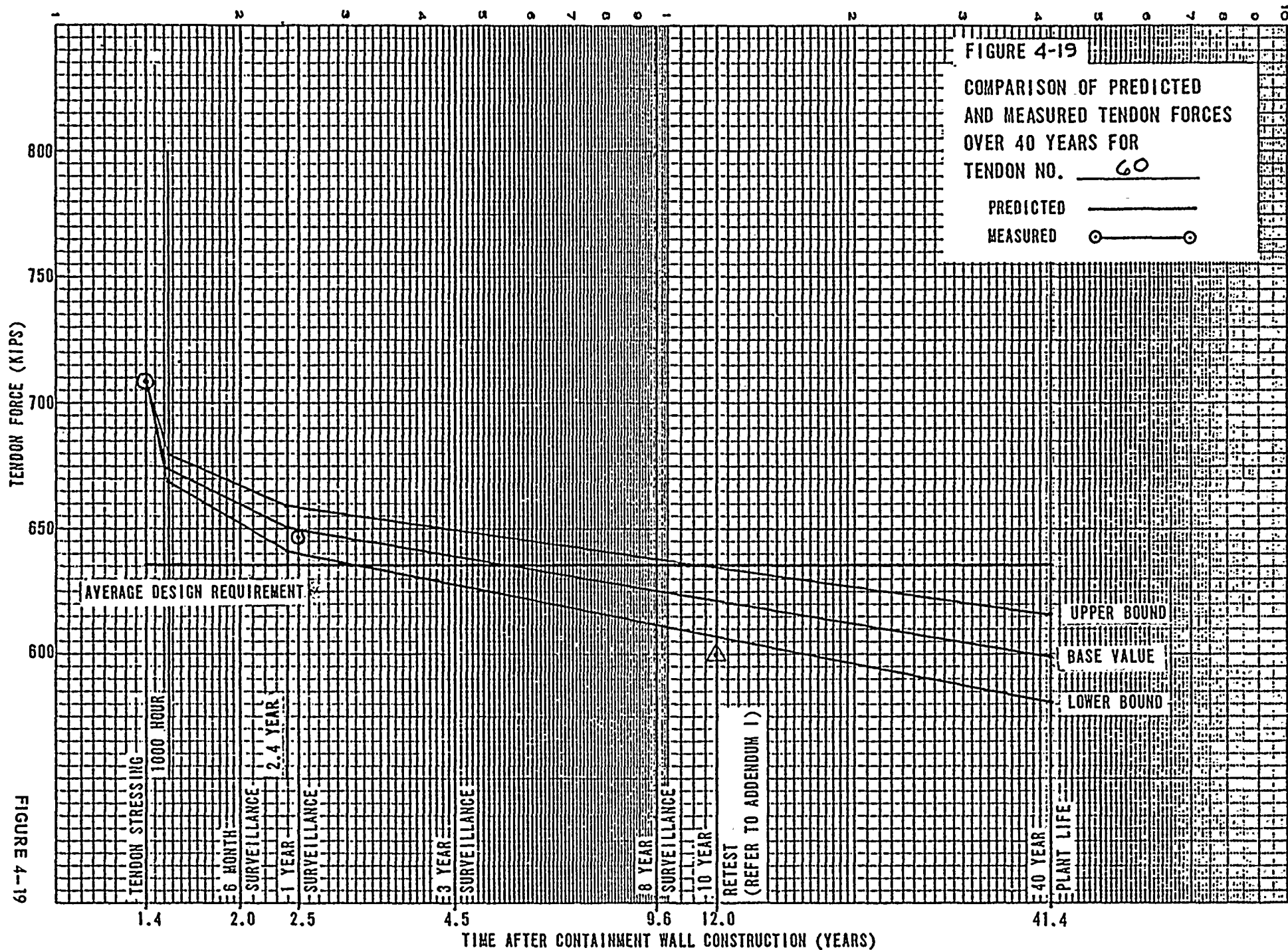
PLANT LIFE:

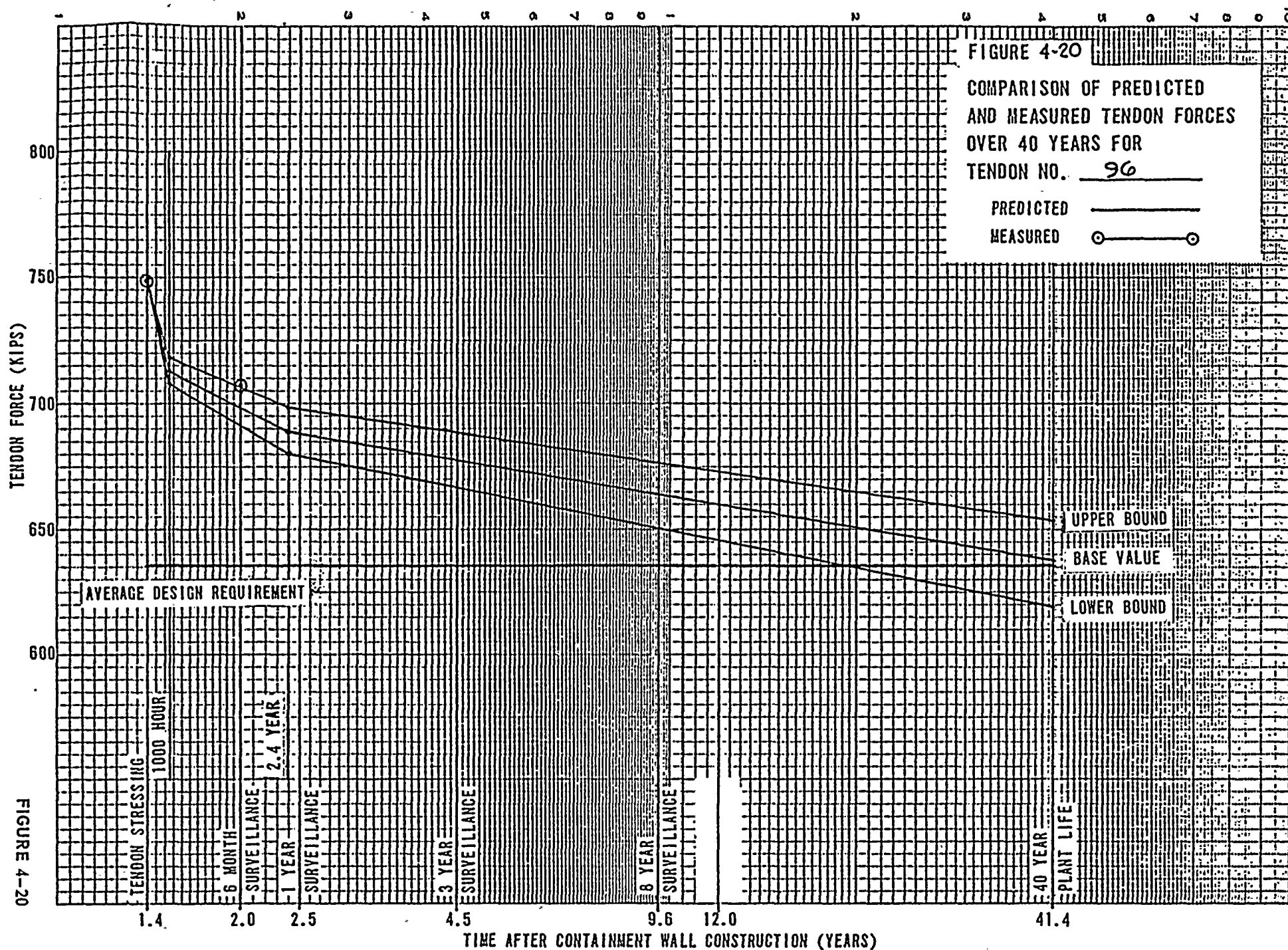
TENDON FORCE (KIPS)

FIGURE 4-18

TIME AFTER CONTAINMENT WALL CONSTRUCTION (YEARS)







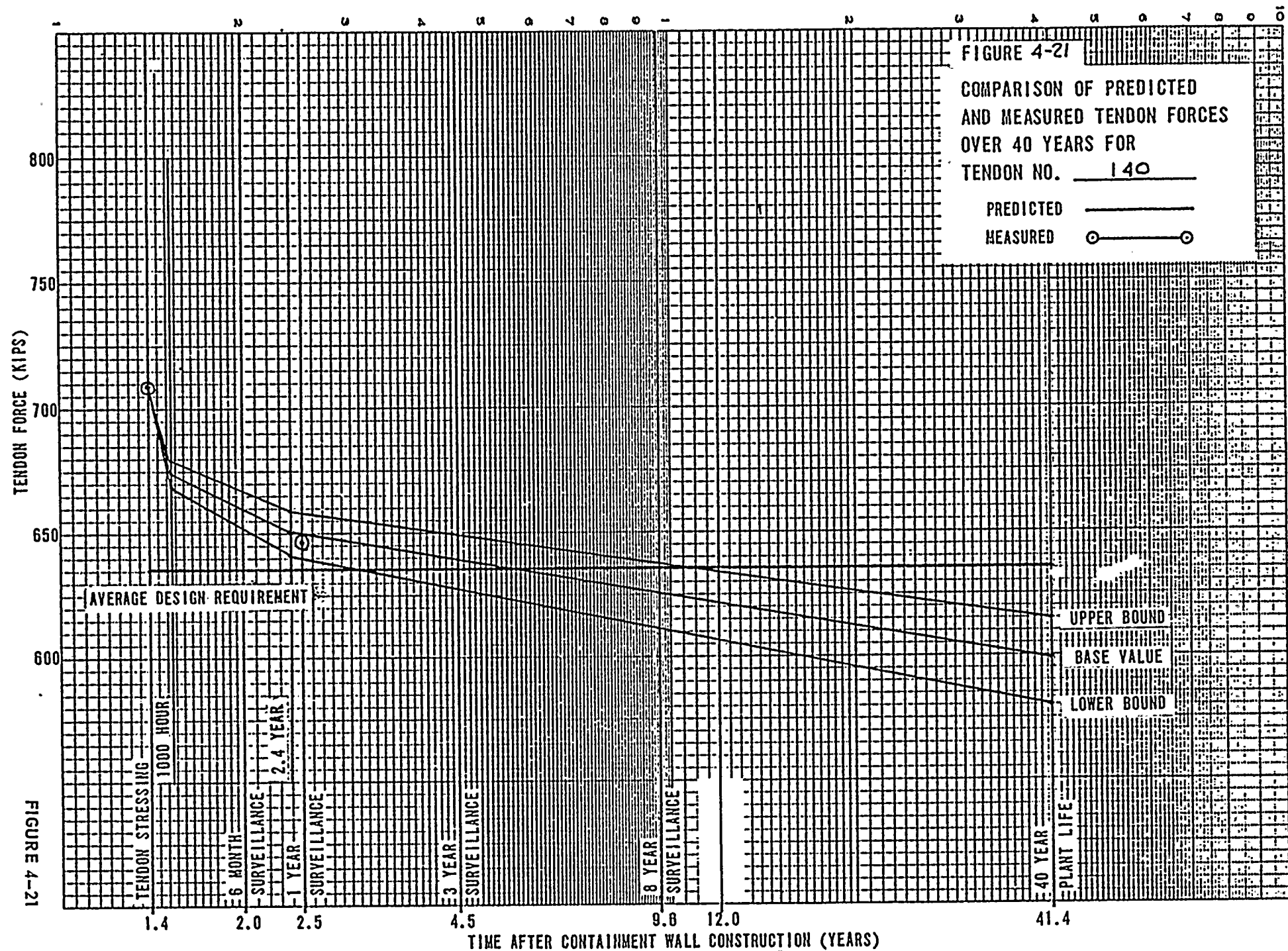




FIGURE 4-22

COMPARISON OF PREDICTED
AND MEASURED TENDON FORCES
OVER 40 YEARS FOR
TENDON NO. 58

PREDICTED —————
MEASURED ○ ————— ○

TENDON FORCE (KIPS)

800

750

700

650

600

AVERAGE DESIGN REQUIREMENT

UPPER BOUND
BASE VALUE
LOWER BOUND

TENDON STRESSING
1000 HOUR

6 MONTH
SURVEILLANCE

2.4 YEAR
SURVEILLANCE

1 YEAR
SURVEILLANCE

3 YEAR
SURVEILLANCE

8 YEAR
SURVEILLANCE

40 YEAR
PLANT LIFE

1.4

2.0

2.5

4.5

9.6

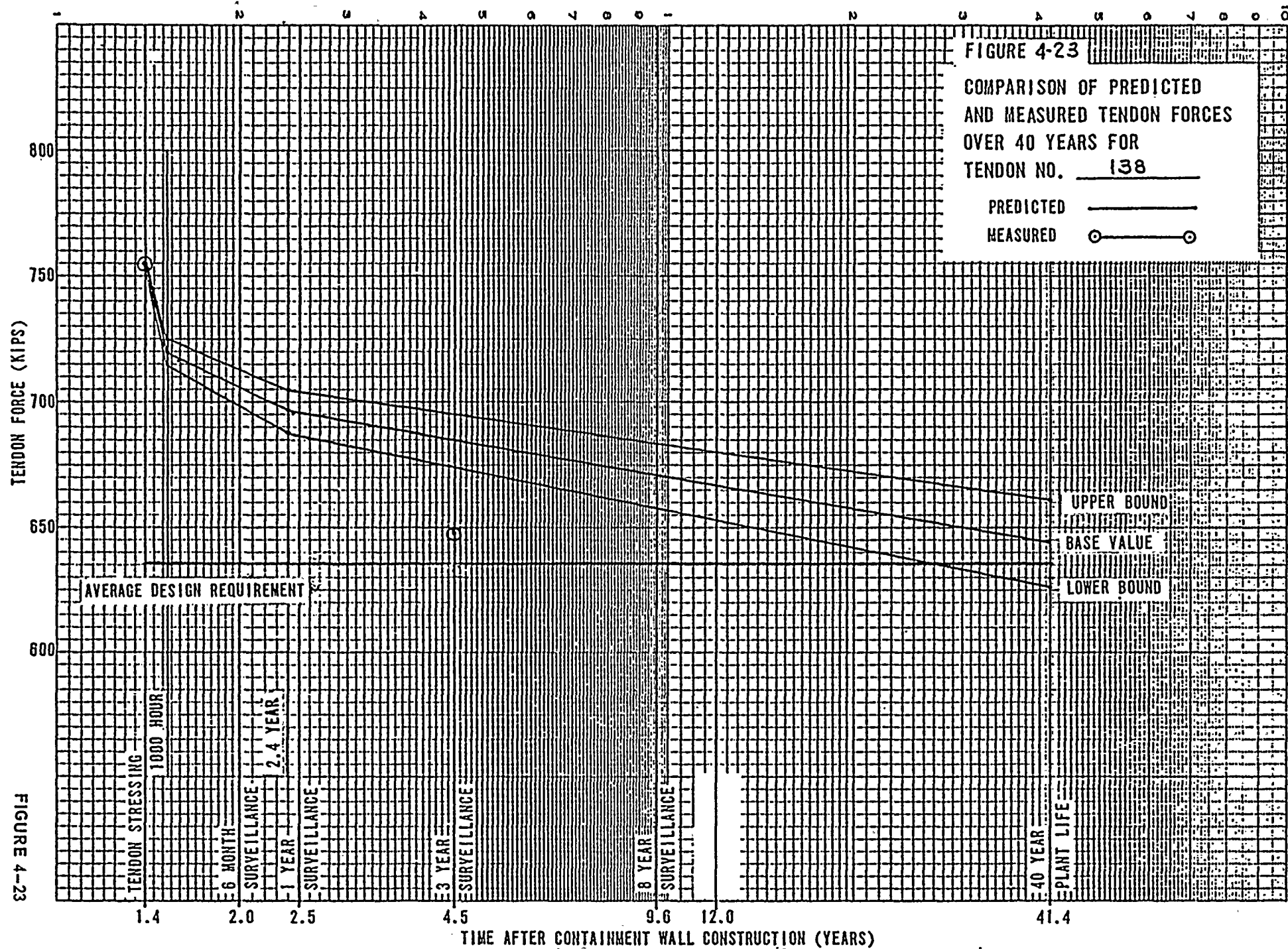
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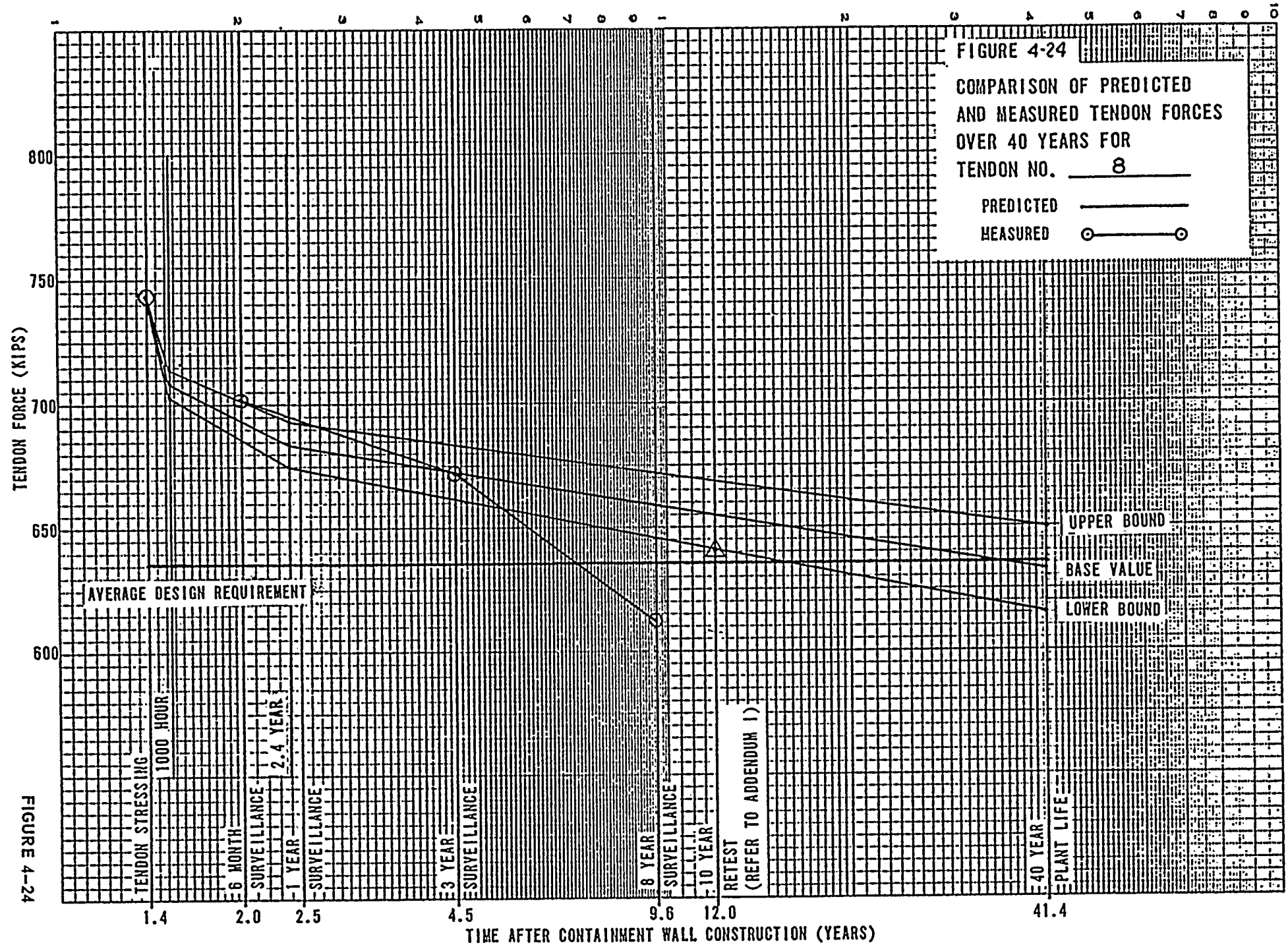
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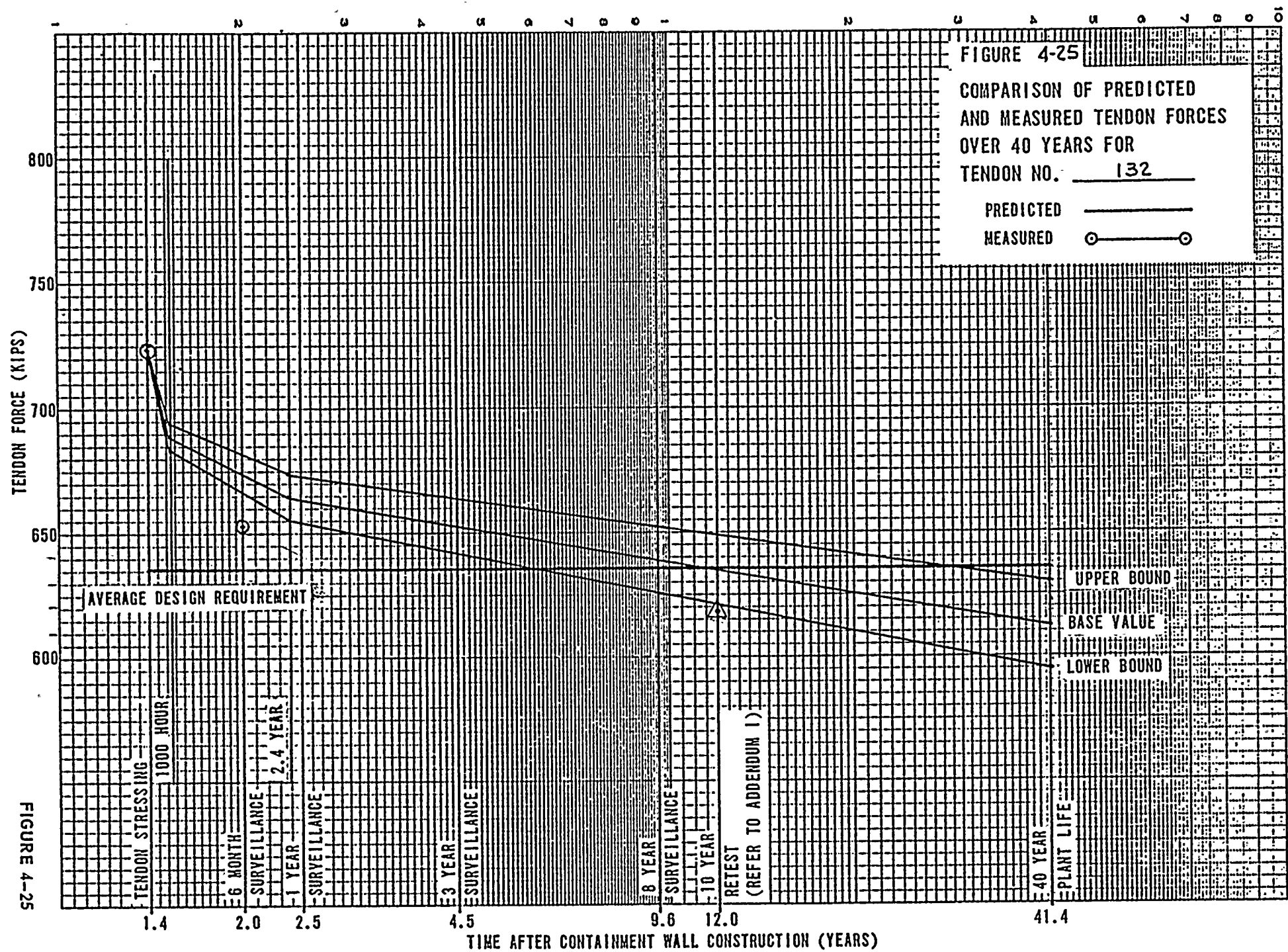
TIME AFTER CONTAINMENT WALL CONSTRUCTION (YEARS)

FIGURE 4-22











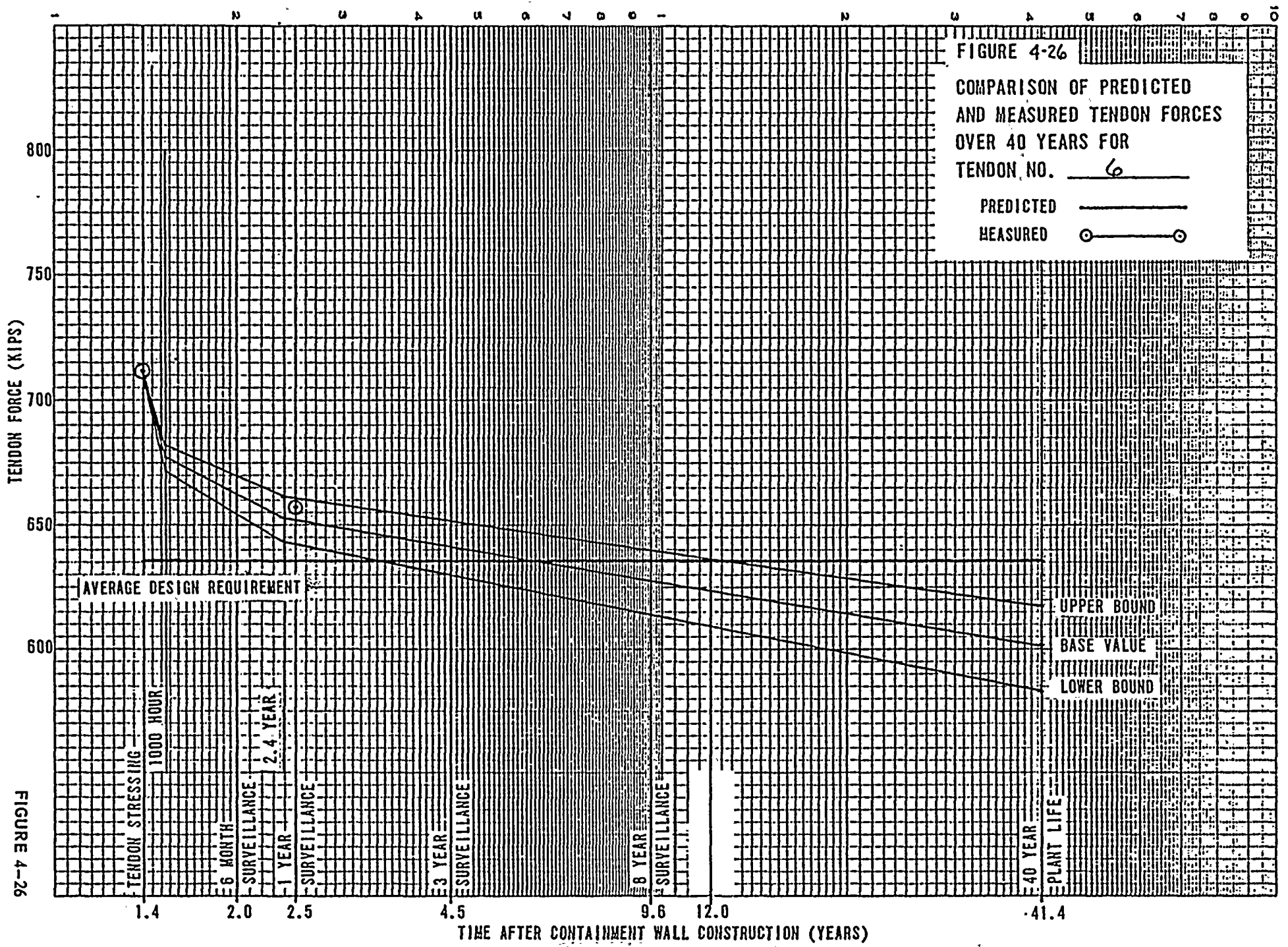


FIGURE 4-26



1

1

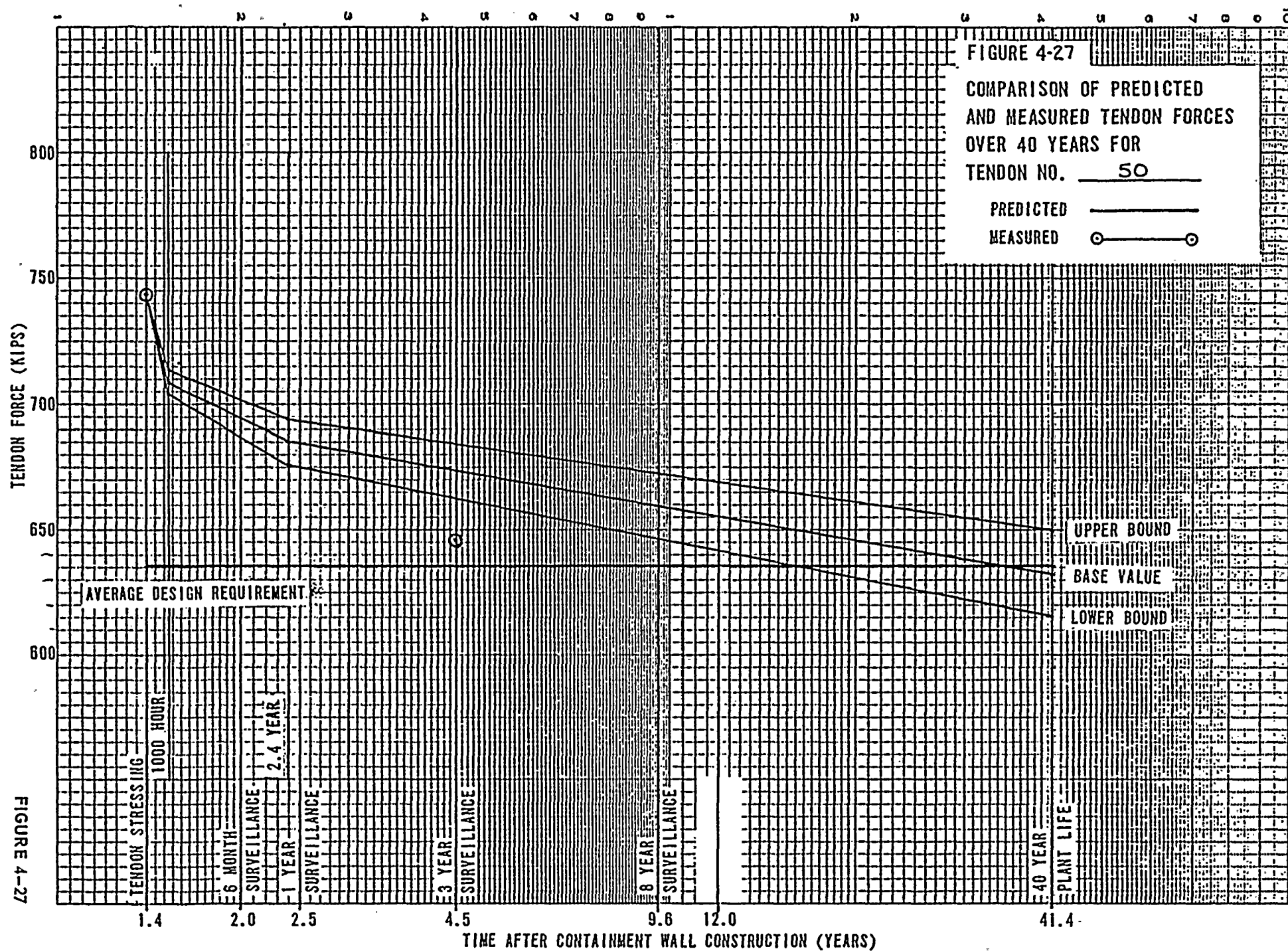


FIGURE 4-27

TIME AFTER CONTAINMENT WALL CONSTRUCTION (YEARS)



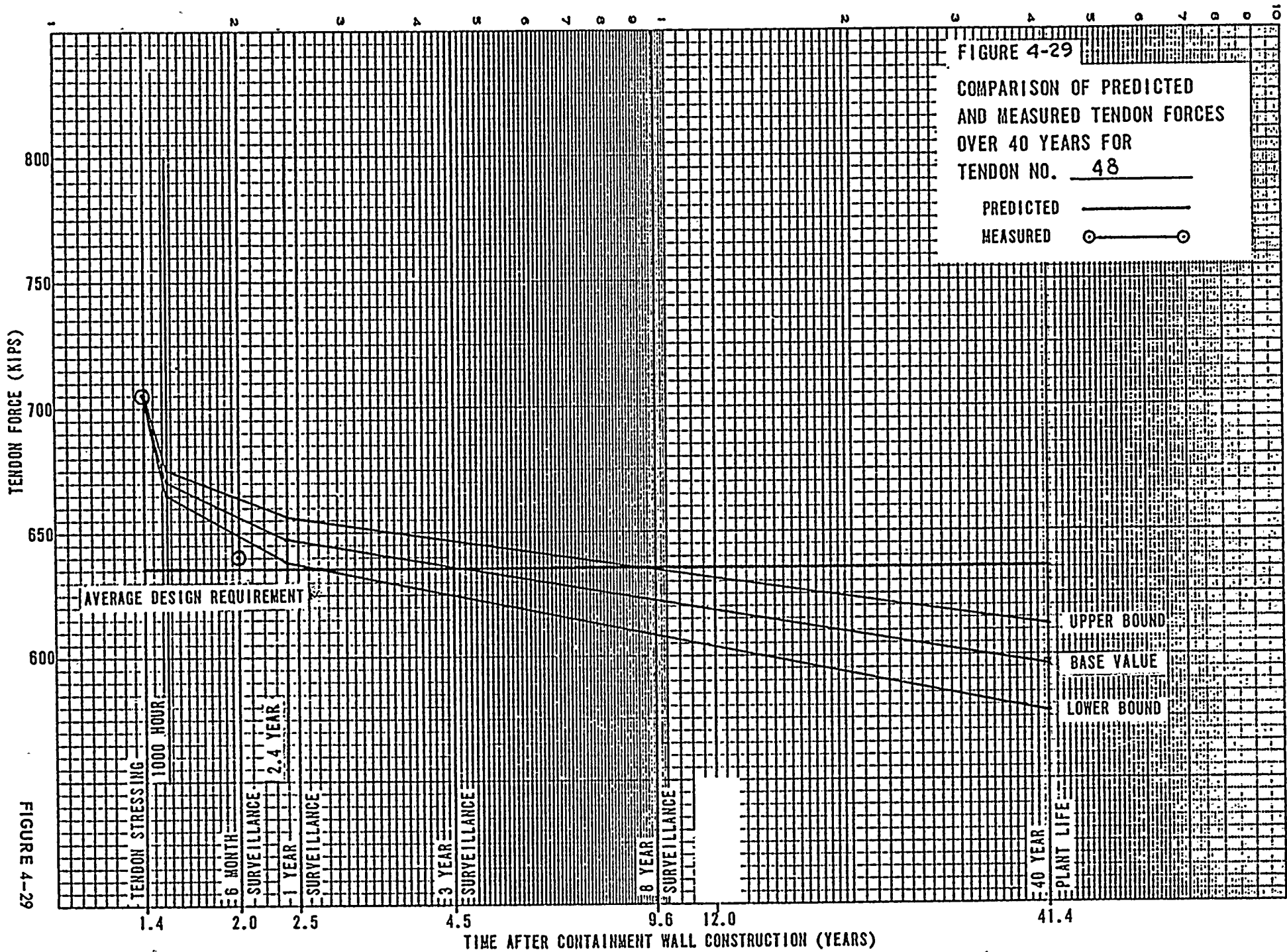
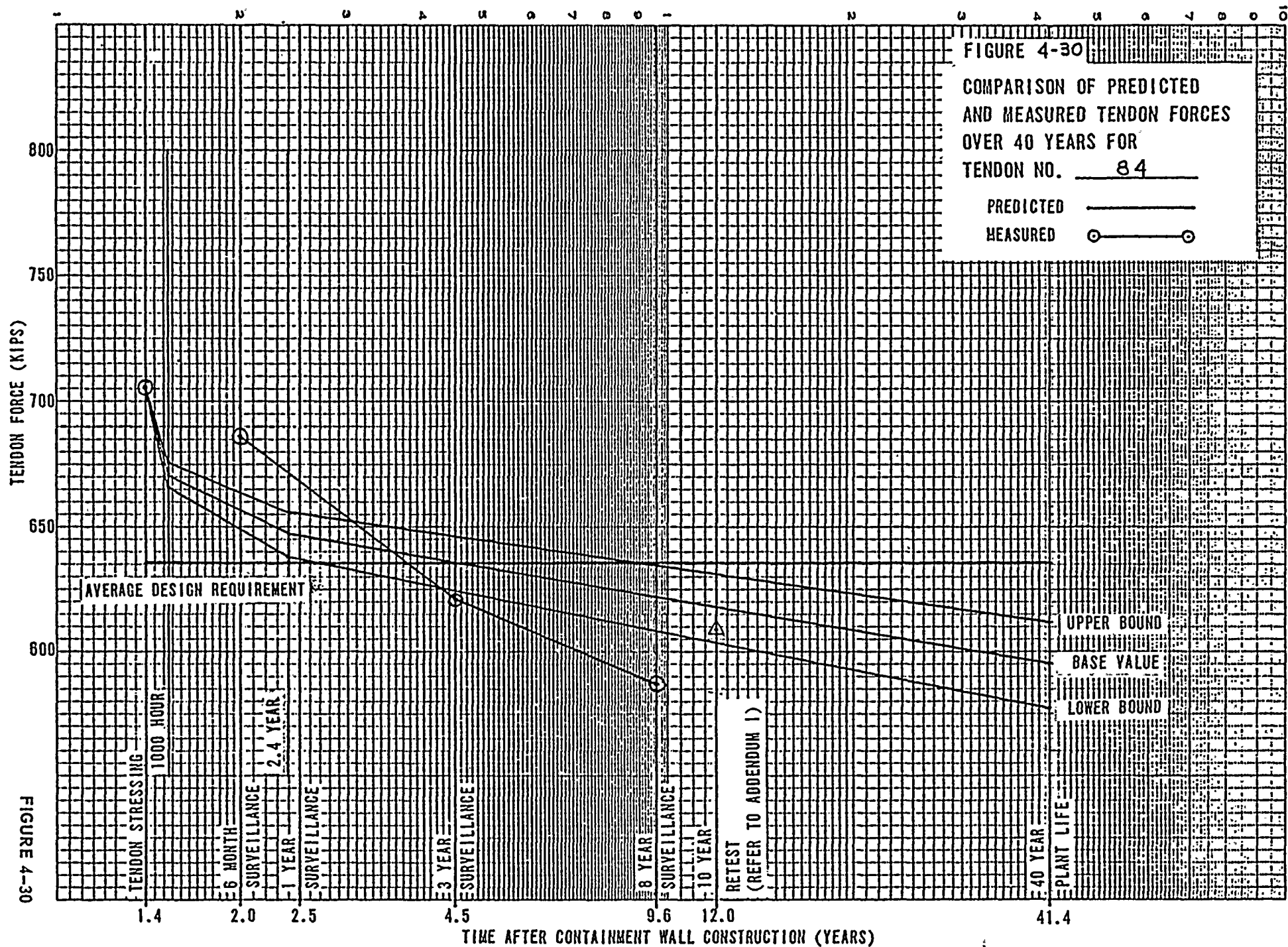
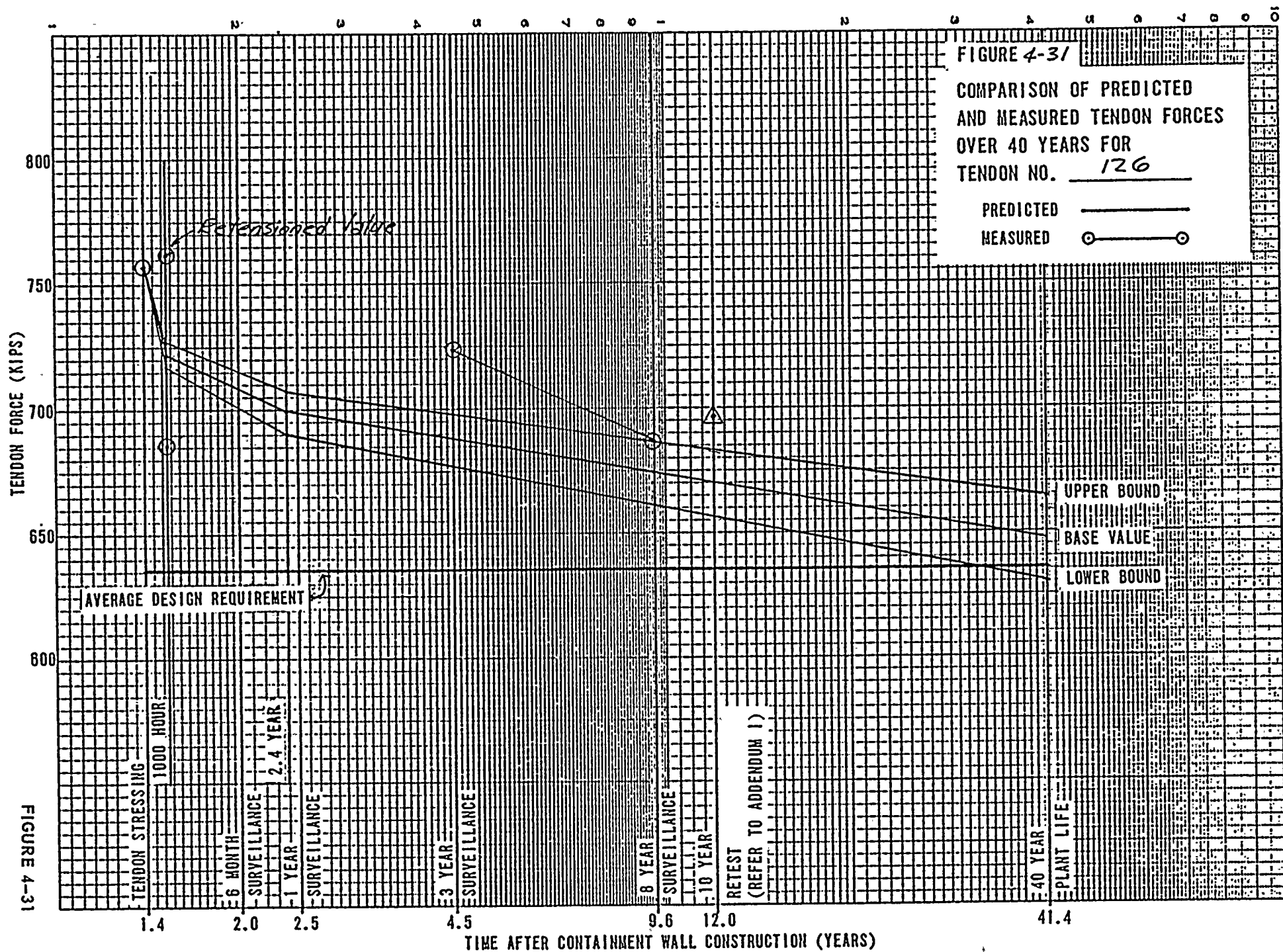


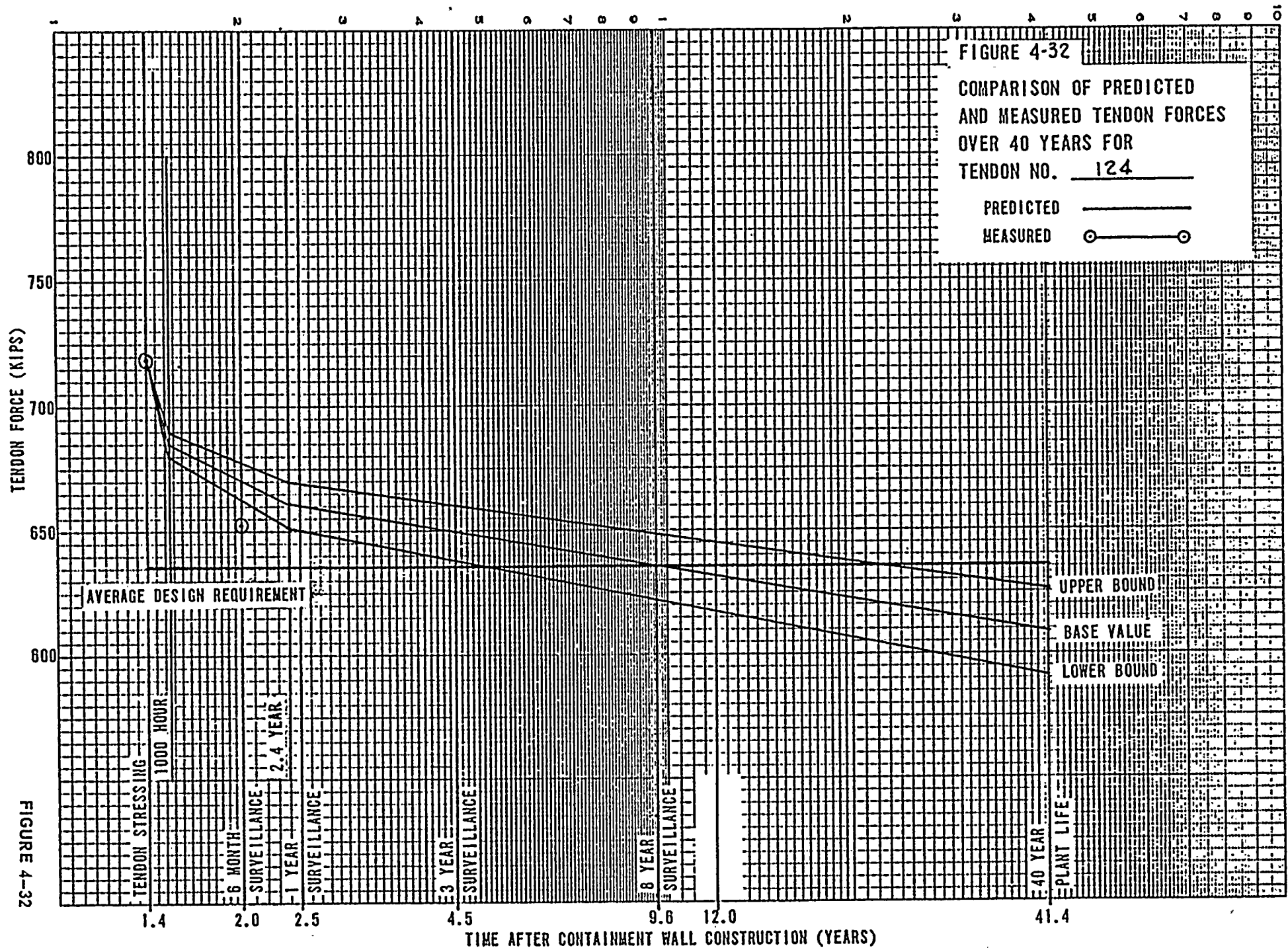
FIGURE 4-29



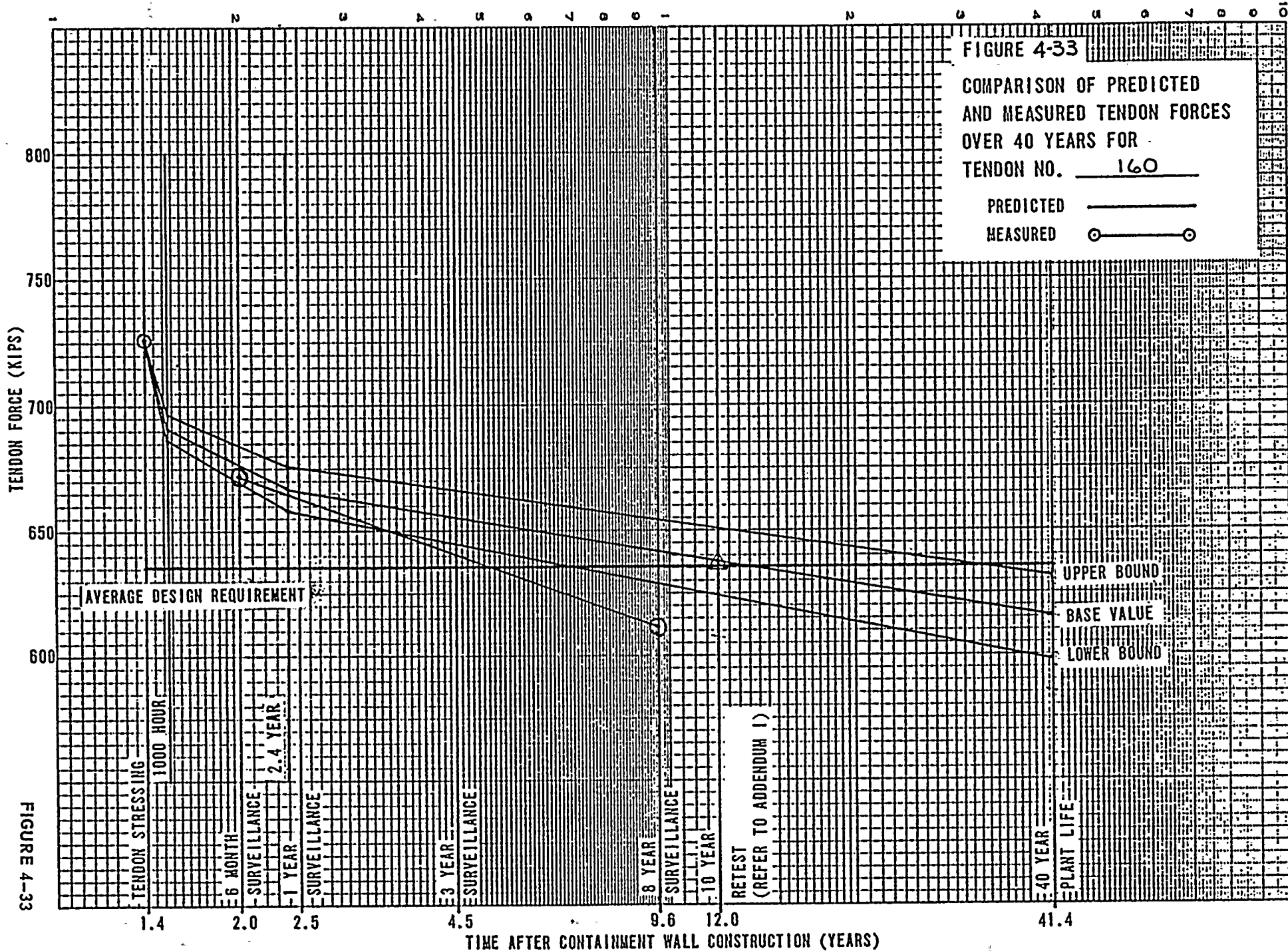




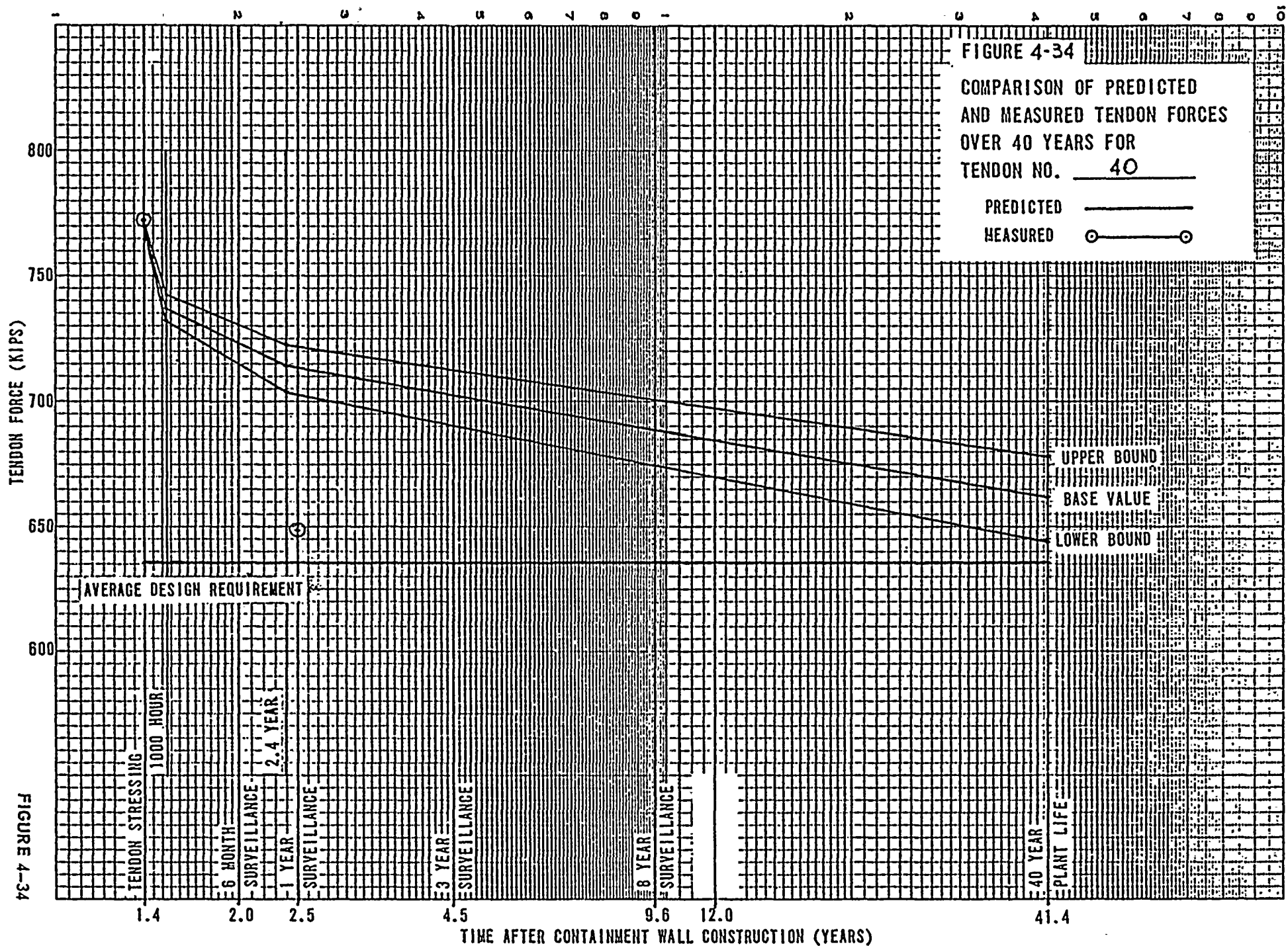














COMPARISON OF PREDICTED
AND MEASURED TENDON FORCES
OVER 40 YEARS FOR
TENDON NO. 76

MEASURED

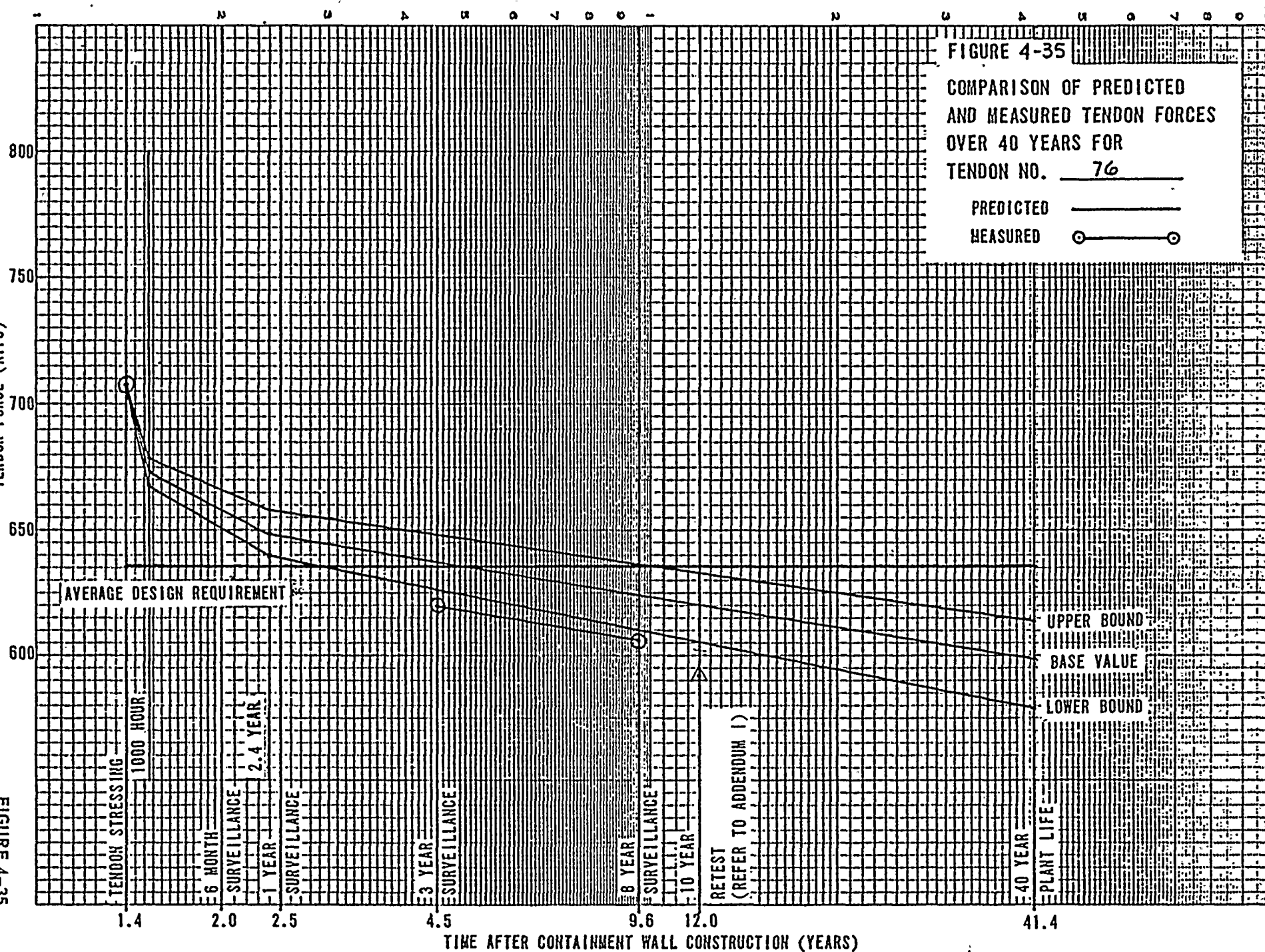
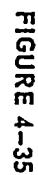




FIGURE 4-36

COMPARISON OF PREDICTED
AND MEASURED TENDON FORCES
OVER 40 YEARS FOR
TENDON NO. 36

PREDICTED

MEASURED

UPPER BOUND

BASE VALUE

LOWER BOUND

AVERAGE DESIGN REQUIREMENT

TENDON STRESSING
1000 HOUR

6 MONTH

SURVEILLANCE
2.4 YEAR

1 YEAR

SURVEILLANCE

3 YEAR

SURVEILLANCE

8 YEAR

SURVEILLANCE

40 YEAR

PLANT LIFE

TENDON FORCE (KIPS)

1.4

2.0

2.5

4.5

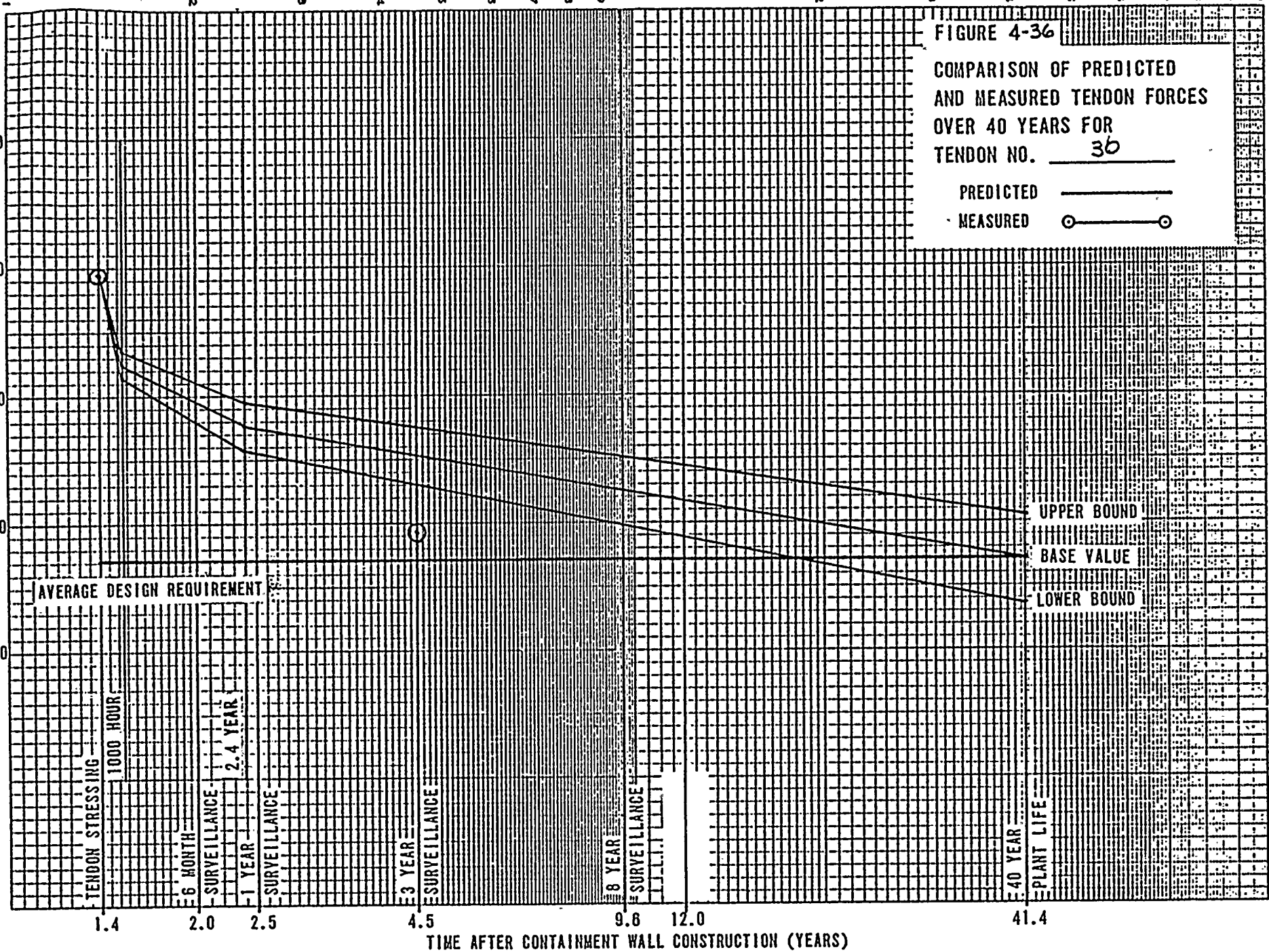
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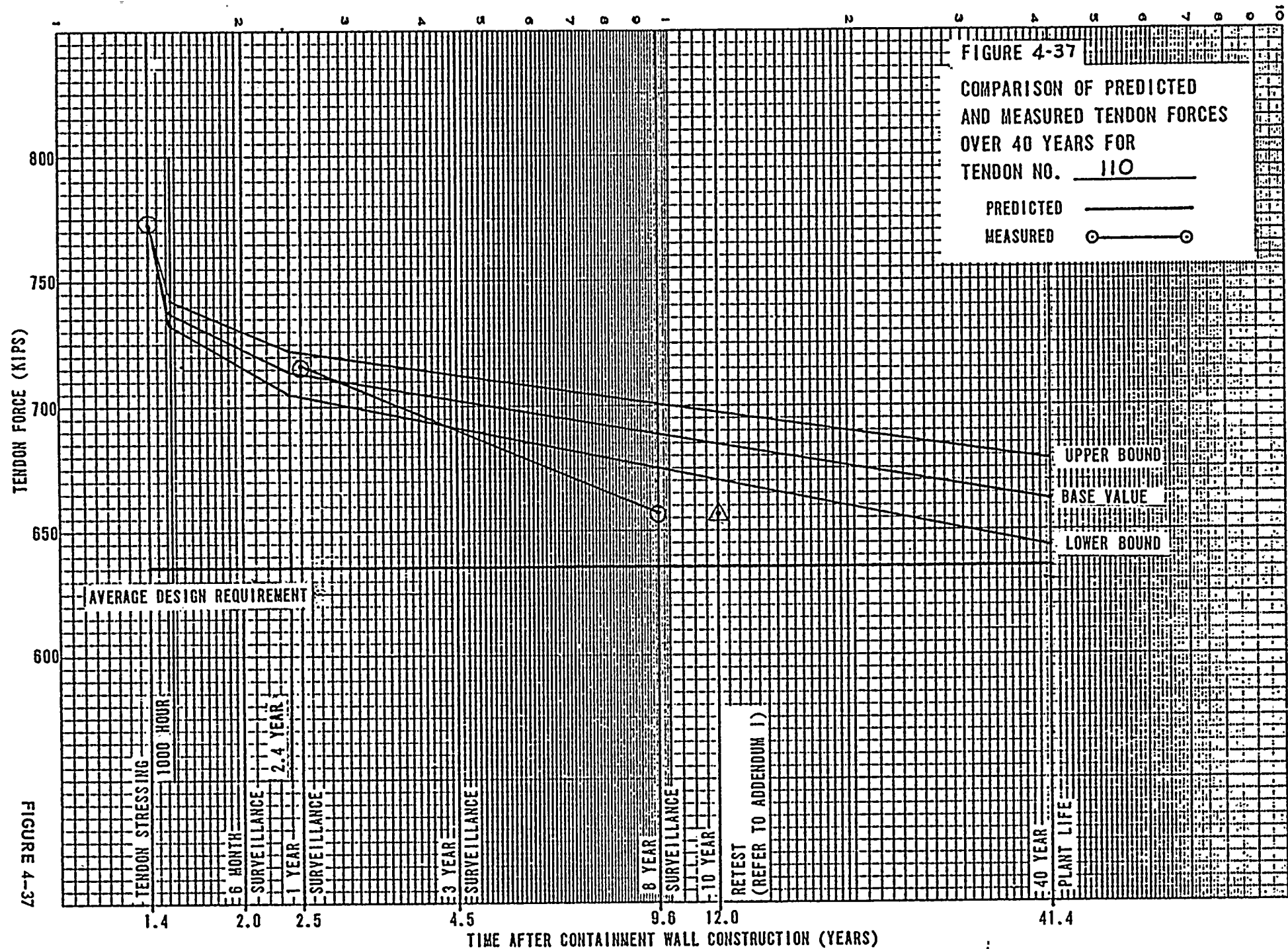
12.0

41.4

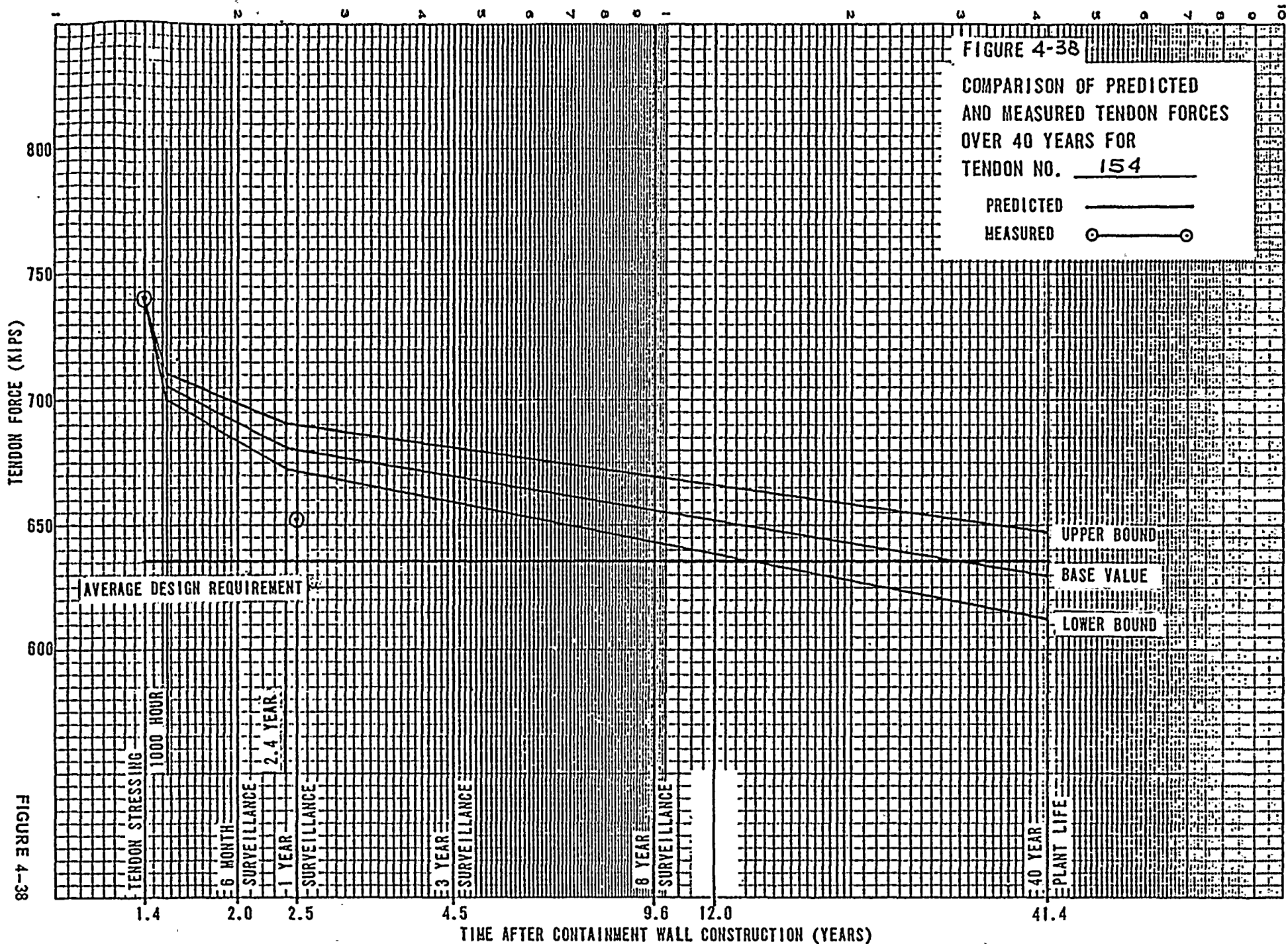
TIME AFTER CONTAINMENT WALL CONSTRUCTION (YEARS)

FIGURE 4-36

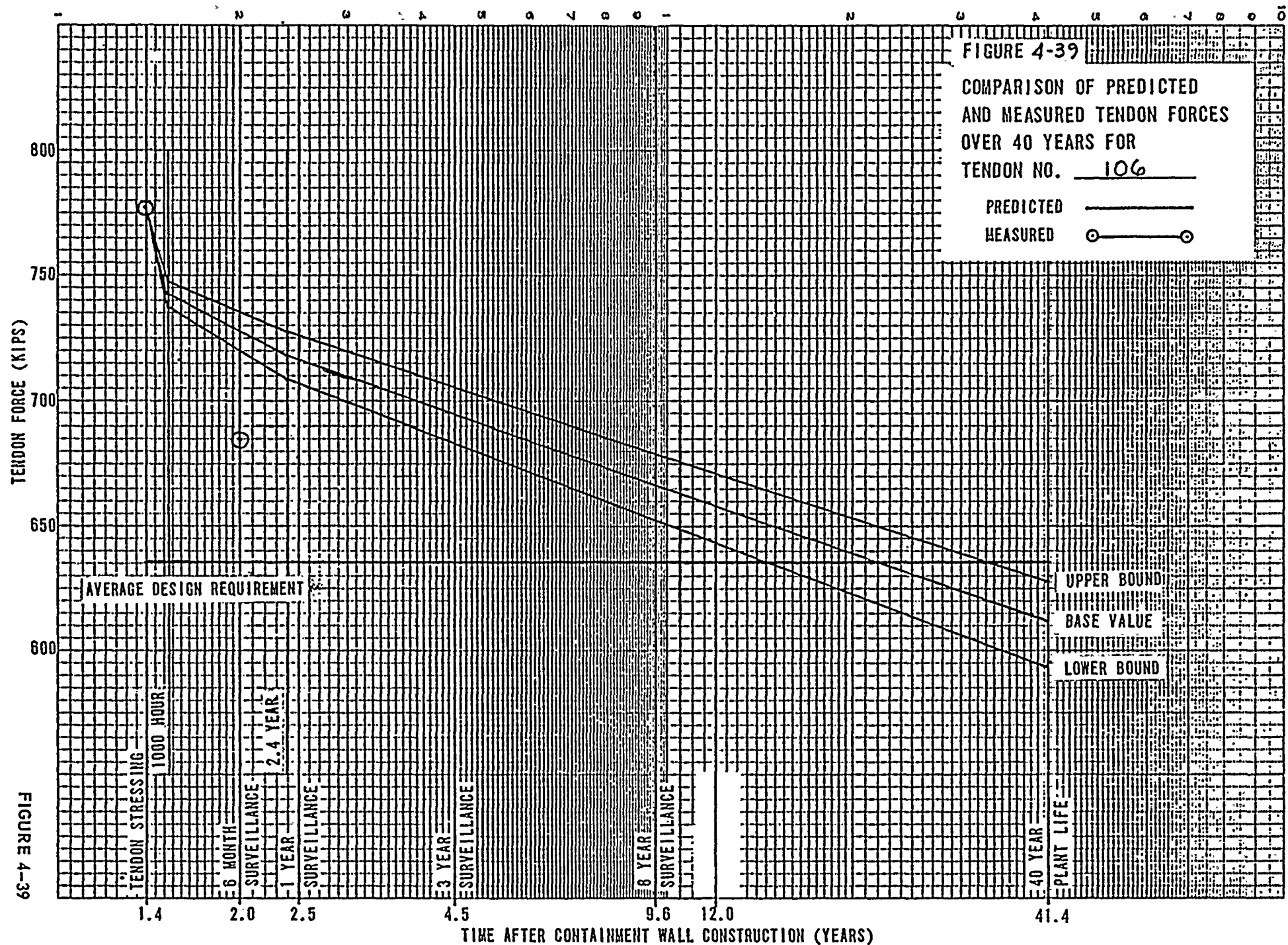




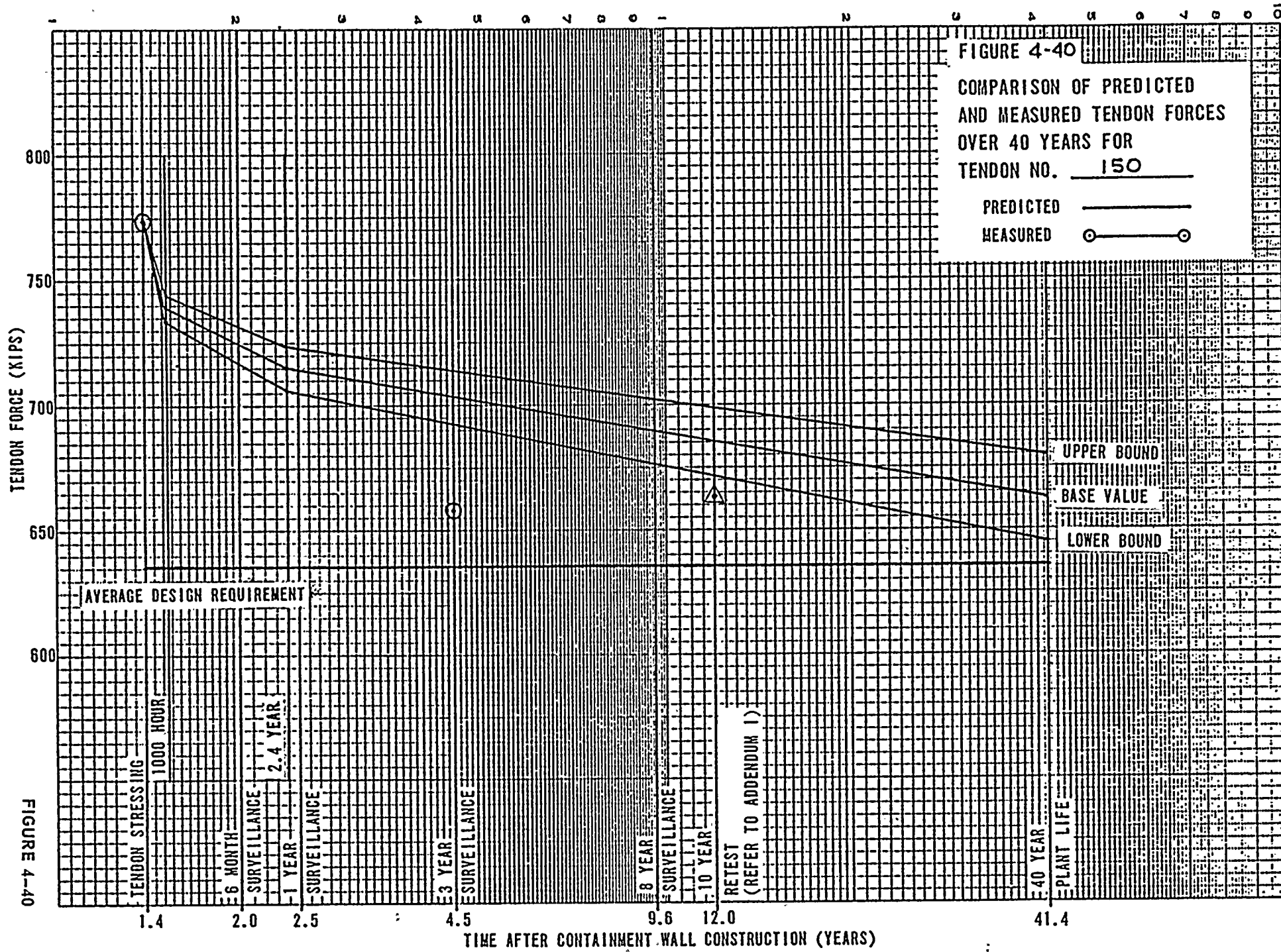




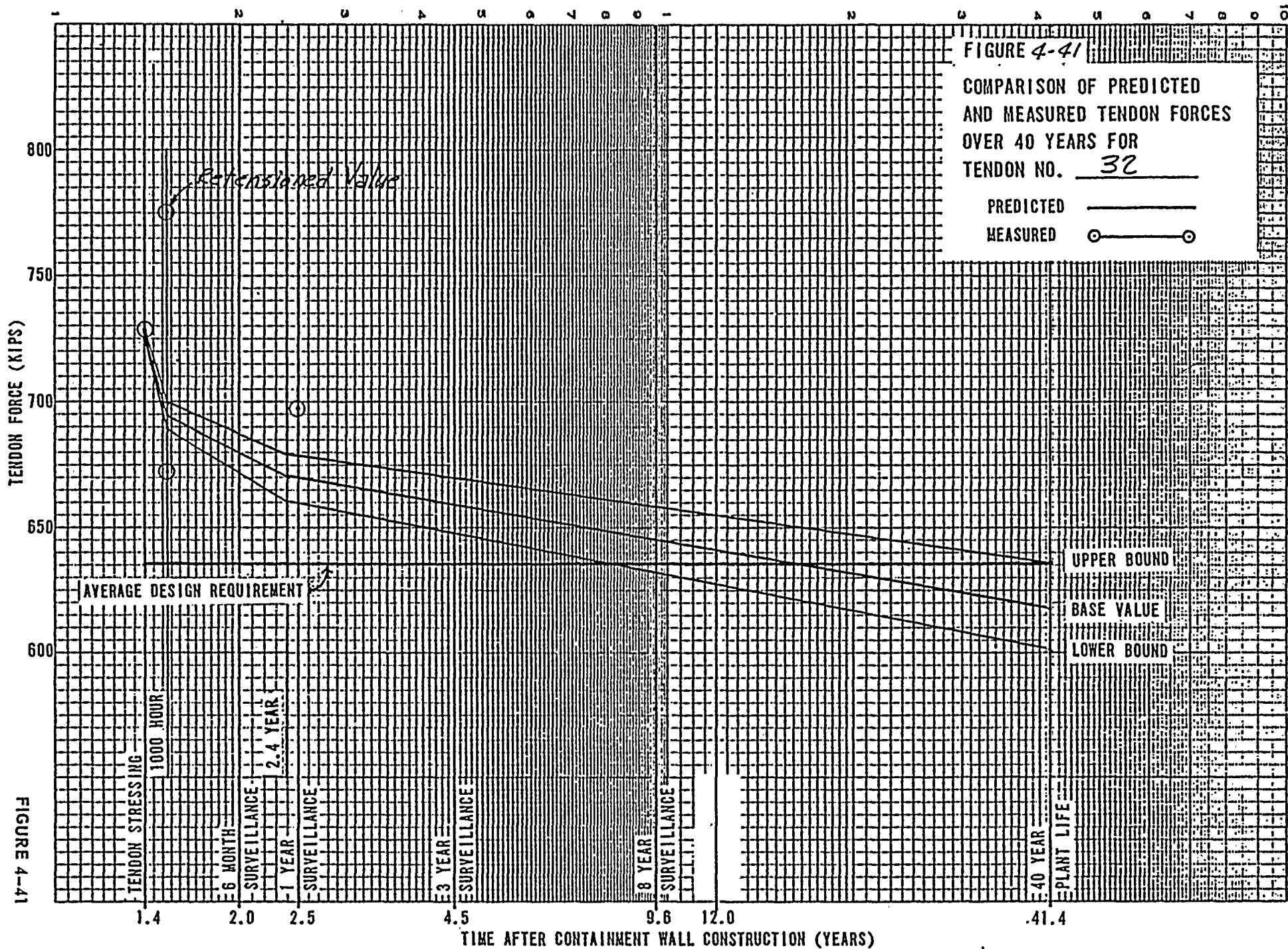


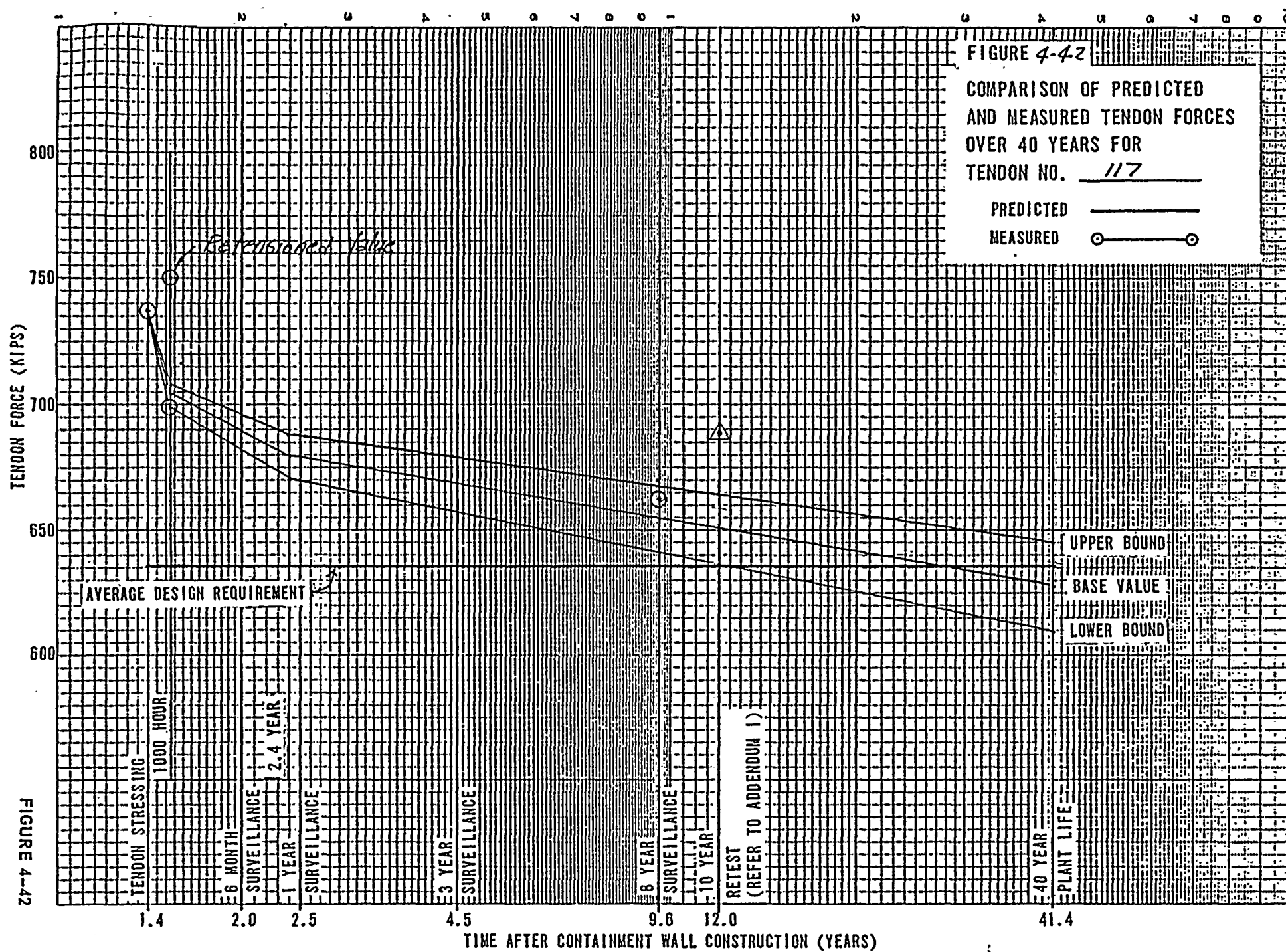














ATTACHMENTS

1. STATEMENT OF WORK AND MANHOUR ESTIMATE FOR THE CONTAINMENT INVESTIGATION
2. PERTINENT EXTRACTS FROM DOCUMENTS
3. MATERIALS REQUIREMENTS FROM ACI 318-63 AND ASME CODE
4. SUMMARY OF STRESSING OPERATIONS (1000 HOUR RETENSIONING)
5. RG&E INTERIM REPORT
6. ROCK ANCHOR LITERATURE SEARCH RESULTS

ATTACHMENT 1

STATEMENT OF WORK AND MANHOUR

ESTIMATE FOR THE CONTAINMENT INVESTIGATION





Gilbert/Commonwealth engineers and consultants

GILBERT ASSOCIATES, INC., P. O. Box 1498, Reading, PA 19603. Tel. 215 775-2600/Cable Gilasoc/Telex 836-431

September 5, 1979

Rochester Gas & Electric Corporation
89 East Avenue
Rochester, New York 14649

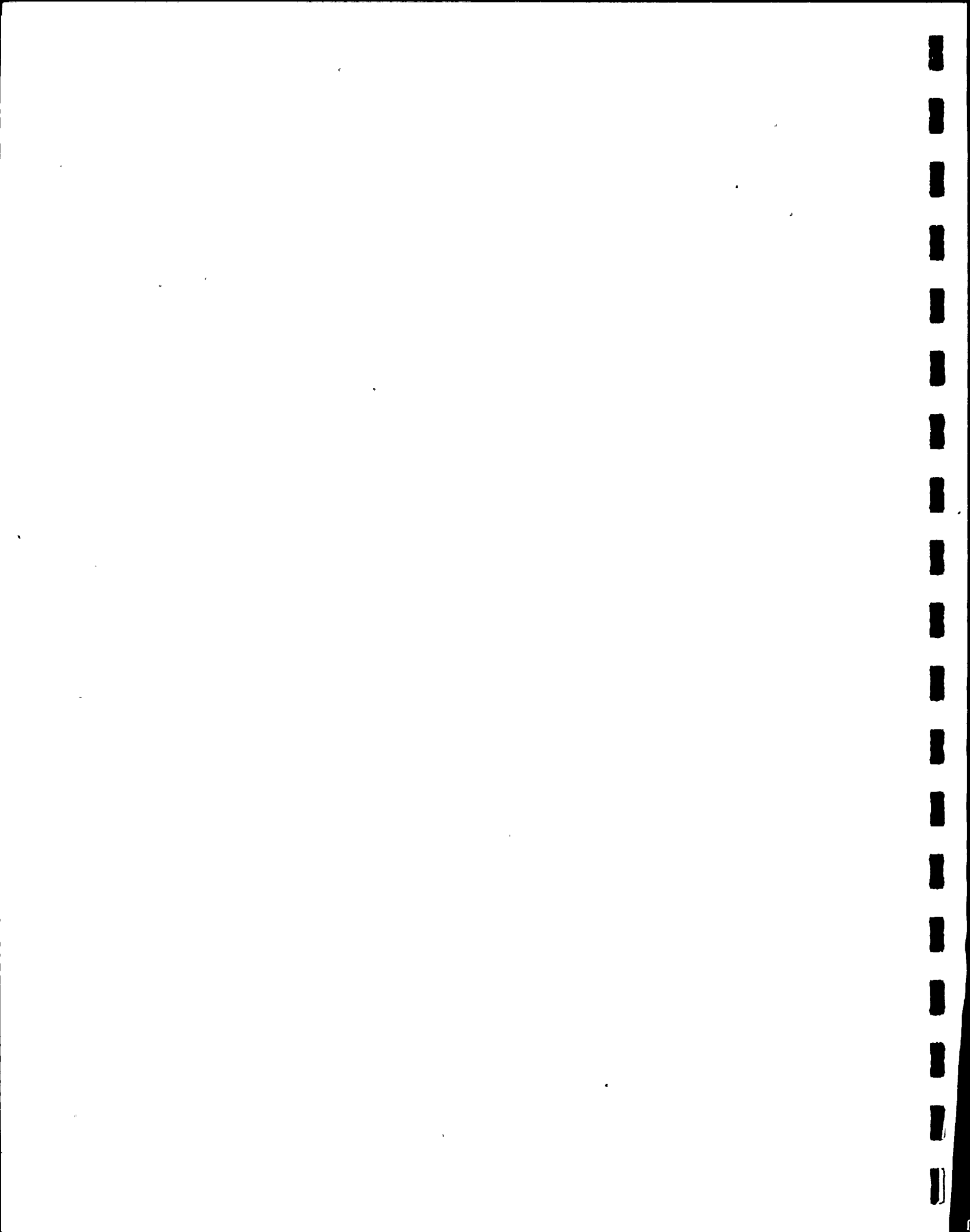
Atten: Mr. John E. Arthur
Chief Engineer

Re: Statement of Work and
Manhour Estimate for
the Containment Investigation

Dear Mr. Arthur:

The following is a statement of work and manhour estimate for the items concerning the containment investigation as discussed with Mr. Geotz at our meeting of August 31, 1979.

1. GAI will review the design calculations, FSAR and procurement specifications for Ginna to determine commitments and requirements for the prestressing system.
2. Responses will be made to the following questions:
 - a. Were tests conducted on the concrete to determine the amount of creep to be expected? What is current practice?
 - b. How were the results of creep tests used in the design? What is current practice?
 - c. Was there a chemical analysis of the ground-water at the site?
 - d. Was rock flow, creep or squeeze considered in the analysis for relaxation?
 - e. Were individual relaxation curves developed for specific tendon types? What is current practice?
 - f. Is there any assurance that broken wires can be detected by visual observation of the stressed tendon?



Rochester Gas & Electric Corp.
Atten: Mr. John E. Arthur
Chief Engineer

Page 2
September 5, 1979

- g. Is a minimum stress of 60% f_u required by the design?
 - h. Is the current tendon sampling method representative? How does it compare with other statistical methods?
 - i. Is there a tendency for relaxation to produce unbalanced forces around the perimeter of the containment?
 - j. Does there appear to be any relaxation relationship based on location or tendon type?
 - k. What relaxation assumptions were used for the rock?
 - l. Were temperature gradients across the walls and their variation around the perimeter taken into account in the design? Would temperature effects contribute to variations in tendon surveillance lift-off values?
3. The containment construction process, including concrete pour dates, tendon stressing and inspection sequence will be summarized and their theoretical effect on tendon losses established.
4. The "actual" tendon losses will be calculated from the ram pressure and ram areas reported in the lift-off tests. These losses will be compared to those committed to in the Project Technical Specifications, the design calculations and those which would be expected based on current NRC approved methods of predicting losses. Behavior at the next lift-off test will be predicted. Possible recommendations for future actions will be discussed pending results of the next lift-off tests.



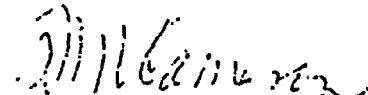
Rochester Gas & Electric Corp.
Atten: Mr. John E. Arthur
Chief Engineer

Page 3
September 5, 1979

5. GAI will retain Inryco (previously Inlad-Ryerson) to provide data on current and past stressing and lift-off procedures and to act as a consultant at the lift-off tests. GAI will also provide at least one observer for these tests.

This work will require approximately 300 manhours over the next three (3) weeks, including an interim recommendation as to possible modification of the anticipated lift-off tests by September 14, 1979. The estimate does not include travel costs, expenses, consulting costs for Inryco, the observer's at the lift-off tests or evaluation of those tests.

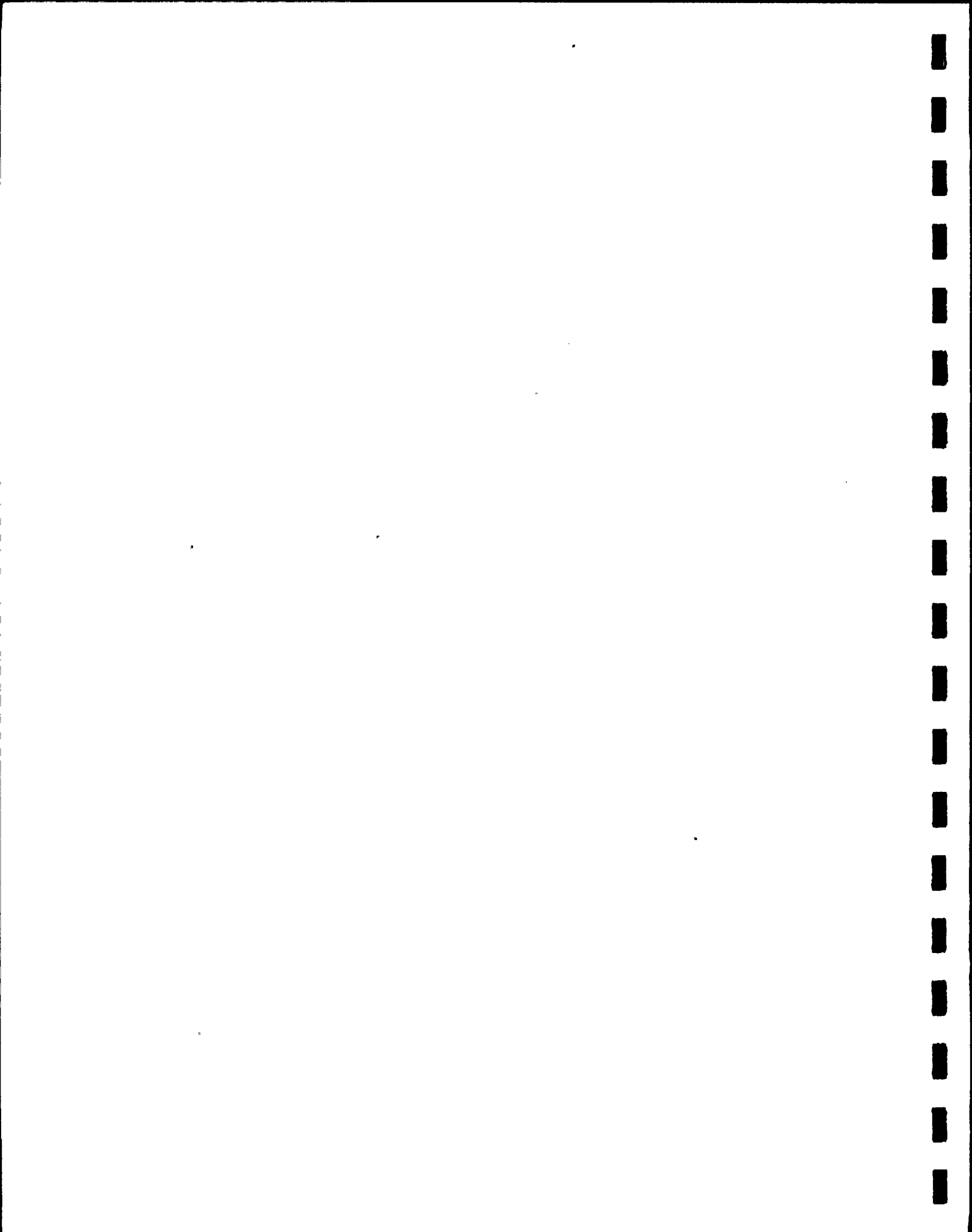
Very truly yours,



P. M. Cameron
Senior Project Engineer

PMC/skm

cc: R. E. Smith
J. N. Covey
J. Geotz
K. T. Momose
F. L. Moreadith
R. E. Pages



ATTACHMENT 2

PERTINENT EXTRACTS FROM DOCUMENTS

CONTENTS

1. CONCRETE TEST DATA
2. ELASTOMERIC PAD, TECHNICAL SPECIFICATION
3. INSTALLATION OF ROCK ANCHORS, TECHNICAL SPECIFICATION
4. SIDE WALL TENDON, TECHNICAL SPECIFICATION
5. STRUCTURAL CONCRETE, TECHNICAL SPECIFICATION
6. EXTRACTS FSAR
7. EXTRACTS GINNA TECHNICAL SPECIFICATION



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ESTABLISHED 1881

INSPECTING ENGINEERS AND CHEMISTS

850 POPLAR STREET

PITTSBURGH, PA. 15220

AREA CODE 412 • TELEPHONE 922-4000

PLEASE REPLY TO:
P. O. BOX 1646
PITTSBURGH, PA. 15230

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PG-16582

March 9, 1967

Mr. H. Lorenz
Chief Structural Engineer
Gilbert Associates, Incorporated
P. O. Box 1498
Reading, Pennsylvania 19603

Re: Ginna Station
Concrete for Containment Vessel

Dear Hans:

Thank you for your letter of March 7th regarding the shrinkage data. We are proceeding on the tests as outlined in my letter to Mr. Powell of Westinghouse, and I hope to send a progress report by March 10th.

Regarding the drying shrinkage of the concrete, I am attaching the report of the National Sand and Gravel Association Joint Research Laboratory Publication No. 8 "Effects of Aggregate Size on Properties of Concrete". The authors, Stanton Walker and Delmar Bloem, are renowned concrete experts, and I believe the report to be very reliable. Please note Page 289 regarding Series 173 length change data Table 4.

Regarding your first reference by A. M. Neville that air entrainment has been found to have no effect on shrinkage, I believe that this statement should be qualified to read "little" effect. This is shown in Table 4 and also shown in the shrinkage curve in the Bureau of Reclamation Handbook. However, I agree that a reduction of air from 6% to 4% would have little effect on shrinkage in our mix for the containment vessel.

- 1 -

Mr. H. Lorenz
Gilbert Assocs., Inc.

March 9, 1967

- 2 -

Regarding the reference ASTM STP 169-A, I believe the statements are basically correct but are only generalizations and cannot be quantitatively applied to our mixes. If you will examine the results in Table 4 I have referenced, there are only slight differences on shrinkage in any of the 4, 6, or 8 sack mixes. There is also slight differences in mixes of the various size aggregates. This has prompted the conclusion of the authors "For the range of maximum sizes from $3/4$ " to $2\frac{1}{2}$ ", differences in drying shrinkage were probably so small as to be of little practical significance in most concrete".

When you apply these results and conclusions to the containment vessel concrete, I can only conclude that the significant increase in strength afforded by reducing the size of aggregate justifies the new $3/4$ " aggregate size mix.

I believe you can also consider that all shrinkage studies are on unrestrained concrete (not-reinforced) and since the containment vessel is highly reinforced, the effect of shrinkage will be minimized. Also, ASTM STP 169-A indicates that there is less effect of shrinkage in mass concrete due to grain changes during curing.

I also believe that the recommended requirements of low mix temperature to reduce water requirements will enable lower shrinkage of our mix design. Basically the greatest influence on shrinkage is water content, and we should keep this at a minimum by maintaining the low temperature and low slump. Since there is a large amount of reinforcement, the $3/4$ " size mix will be easier to properly consolidate at a low slump.

Considering all of the factors and placing strength as the primary requirement, I believe that the $3/4$ " aggregate mix is the best one for this application.



Mr. H. Lorenz
Gilbert Associates, Inc.

March 9, 1967

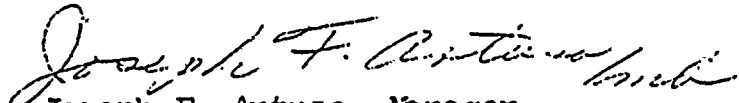
- 3 -

I shall have seven day compressive strengths on our mixes next week, but it will take an additional 3 weeks before we can have the preliminary shrinkage values. I shall submit them to you immediately as we progress.

Please let me know if we can be of any other service.

Very truly yours,

PITTSBURGH TESTING LABORATORY


Joseph F. Artuso, Manager,
Cement & Concrete Department

JFA/mb

Enclosures



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March 17, 1967

LABORATORY No. 653582

ORDER No. PJ-16532

CLIENT'S No. 54-H-60971-B

REPORT
REVISED REPORT (a)
REPORT OF AGGREGATE AND CONCRETE TESTS
FOR
WESTINGHOUSE ELECTRIC CORPORATION
ATOMIC POWER DIVISION
P. O. BOX 355
PITTSBURGH, PENNSYLVANIA 15230

- PROJECT : Ginna Station
Brookwood
Containment Vessel Concrete
- Tests Requested : Determination of Adequate 5000 P.S.I. Concrete
- A. 3 Point W/C Ratio Curves
with 1" Max. Aggregate (Combination N.Y. State
No. 1 and No. 2)
6, 7 and 8 Sk. Mixes - 4" Slump A.E.
 - B. 3 point W/C Ratio Curves
with 3/4" Max. Aggregate (N.Y. State No. 1)
6, 7 and 8 Sk. Mixes - 4" Slump A.E.
 - C. Aggregate Tests - Gradation, Specific Gravity
and Absorption
 - D. Void Spacing Factor of 7 Sk. 4% A.E. and
7 Sk. 3% A.E. of vibrated and non vibrated
concrete
 - E. Length Change (ASTM C-494 Method) of Mixes in
A and B
 - F. Complete Physicals and Chemicals of Portland
Cement
 - G. Further Making Properties of Fine Aggregate
compared with Standard Ottawa Sand

(a)-Revised report issued 4/19/67 to include 28 day results of above tests.



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CLIENT'S No. 54-H-60971-B

March 17, 1967

REPORT

LABORATORY No. 652582

ORDER No. PQ-15582

Concrete Supplier : Manitou Concrete Company

Materials furnished by : Concrete Supplier:

Portland Cement - Ischig Type II ASTM C-150

Fine Aggregate - Manitou Redman Deposit

Coarse Aggregate - Manitou Walworth Quarry

Plastiment by Sika Chemical Corporation

Dargx A.E.A. by Doway - Army



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LABORATORY No. 652582

March 17, 1967

ORDER No. PG-16582

CLIENT'S No. 54-H-60973-B

REPORT

SERIES A

LABORATORY BATCH-CONCRETE MIX PROPORTIONS AND RESULTS

MIX NO.	A-1	A-2	A-3
Cement (Lbs.)	56.4	65.8	75.2
Hard Aggregate (Lbs.)	132.0	125.0	116.0
Coarse Aggregate (Lbs.) (N.Y. No. 1)	180.0	180.0	180.0
Admixture	(a)	(b)	(c)
Slump (Inches)	4.0	4.0	4.0
Air Content (%)	4.0	4.2	4.2
Water (Gallons)	3.47	3.57	3.79
Unit Weight (Plastic Concrete)			
Lbs./Cu. Ft.	147.75	148.2	147.5
W/C Ratio Gals./Sk.	5.8	5.1	4.74
Yield (Cu. Ft.)	2.7	2.71	2.72
Compressive Strength P.S.I.			
7 Days	4600	4670	4780
	4530	4600	4880
	4420	4600	4850
Average	4510	4620	4830
10 Days (Client's Request)	4780	4920	5490
28 Days	5770	5870	6020
	5650	5840	5980
Average	5710	5850	6000

56 Days

Duo 5/2/67

(a) - 18 Lbs. ozs. Elastomers/cu. yd., 1-3/4 ozs. Duxon A.E.A./cu. yd.
 (b) - 21 ozs. Elastomers/cu. yd., 2 ozs. Duxon A.E.A./cu. yd.
 (c) - 24 ozs. Ltg. Elastomers/cu. yd., 3 ozs. Duxon A.E.A./cu. yd.



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LABORATORY No. 652582

March 17, 1967

ORDER No. 12-26582

CLIENT'S No. 54-H-60971-3

REPORT

CONCRETE MIX PROPORTIONS-ONE CUBIC YARD BASIS

MIX	<u>A-1</u>	<u>A-2</u>	<u>A-3</u>
	(27.0 C.F.)	(27.1 C.F.)	(27.2 C.F.)
Cement Lbs.	564	658	752
Fine Aggregate Lbs.	1320	1250	1160
Coarse Aggregate Lbs. N.Y. No. 1	1800	1800	1800
Admixture	(a)	(b)	(c)
W/C Ratio-Gals./Sk.	5.8	5.1	4.74
Total Water-Gals./Cu. Yd.	34.7	35.7	37.9
Maximum Slump	4.0	4.0	4.0
(a)-18 liq. ozs. Plastiment/cu. yd., 1-3/4 ozs. Durox A.E.A./cu. yd.			
(b)-22 ozs. Plastiment/cu. yd., 2 ozs. Durox A.E.A./cu. yd.			
(c)-25 ozs. liq. Plastiment/cu. yd., 3 ozs. Durox A.E.A./cu. yd.			

REMARKS: Above aggregate weights are in the saturated surface dry condition. Field adjustment must be made for aggregate surface water. Concrete should be checked in the field in truck load quantities and adjustment made for air content, yield and workability if necessary.

Recommend Maximum W/C tolerance maximum of plus 0.25 gals./sk.

SPECIAL REMARKS: Recommend continued use of Mix A-2 (7 Sk. with N.Y. No. 1 Aggregate). The 28 day compressive results of Mix A-2 corresponds to the ACI 214 overdesign which would conform to the ACI Building Code ultimate strength requirements for a coefficient of variation of about 12%. A greater allowance will be possible under 56 day evaluation of the Type II Class concrete.





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March 17, 1967

LABORATORY No.

652582

ORDER No. K-16582

CLIENT'S No. 54-H-50971-B

REPORT

SERIES B

LABORATORY BATCH-CONCRETE 127. PROPORTIONS AND RESULTS

TEST NO.	1-1	3-2	3-3
Cement (Lbs.)	56.4	65.6	75.2
Fine Aggregate (lbs.)	127.0	117.0	107.0
Coarse Aggregate (Lbs.) N.Y.No.1	76.0	76.0	75.0
N.Y.No.2	114.0	114.0	114.0
Admixture	(a)	(b)	(c)
Slump (Inches)	4.0	4.0	4.0
Air Content (%)	3.4	3.4	3.5
Water (Gallons)	3.44	3.54	3.77
Unit Weight (Plastic Concrete)			
Lbs./Cu. Ft.	149.0	149.5	148.0
W/C Ratio Gals./Sk.	5.7	5.05	4.71
Yield (Cu. Ft.)	2.7	2.7	2.72
Compressive Strength P.S.I.			
7 Days	3820	4230	4530
	3750	4390	4420
	4070	4460	4460
Average	3880	4380	4470
28 Days	5100	5420	5380
	5060	5650	5490
	5240	5840	5980
Average	5130	5630	5610

Due 5/3/67

CONCRETE MIX PROPORTIONS

ONE CUBIC YARD MIXTURE

	(27.00 C.F.)	(27.0 C.F.)	(27.2 C.F.)
Cement Lbs.	564	658	752
Fine Aggregate Lbs.	1270	1170	1070
Coarse Aggregate Lbs. N.Y.No. 1	760	760	760
N.Y.No.2	1140	1140	1140
Admixture	(a)	(b)	(c)
W/C Ratio-Gals./Sk.	5.7	5.05	4.71
Total Water-Gals./Cu. Ft.	34.4	35.4	37.7
Maximum Slump	4"	4"	4.0"
(a)-18 ozs. Lbs. Placeiment/cu. ft., 2-1/2 ozs. Barox/cu. yd.			
(b)-21 ozs. Lbs. Placeiment/cu. ft., 2 1/2 ozs. Barox/cu. yd.			
(c)-24 ozs. Lbs. Placeiment/cu. ft., 3 1/2 ozs. Barox/cu. yd.			

REMARKS: Above aggregate weights are in the saturated surface dry condition. Field adjustment must be made for aggregate surface water. Concrete should be checked in the field in truck load quantities and adjustment made for air content, yield and workability if necessary.

Recommended Minimum W/C tolerance maximum of plus 0.45 gals./sk.

REMARKS: The 7 day compressive strengths are not appreciably higher than Series A (3/4" gals.) but this is primarily due to the lower air content developed in Series B (2") agg. mixes.





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LABORATORY No. C52582

ORDER No. M-16582

CLIENT'S No. 54-H-60971-E

March 17, 1967

REPORT

C - AGGREGATE TESTS

FINE AGGREGATE - TEST RESULTS

GRADATION

<u>Sieve Size</u>	<u>Cumulative Percent Retained</u>	<u>Percent Passing</u>	<u>Specifications Percent Passing ASTM C-33</u>
3/8"	0	100	100
No. 4	3	97	95-100
No. 8	15	85	80-100
No. 16	31	69	50-85
No. 30	51	49	25-60
No. 50	79	21	10-30
No. 100	95	5	2-10
Fineness Modulus	2.74		2.54 to 2.94

Soundness (Percent)
(5 Cycles Sodium Sulfate)

Not Determined

10 Maximum

Clay Lumps (Percent)

None

1.0 Maximum

Material Finer than No. 200 Sieve
(Percent) - (concrete subjected to
Abrasion

3.5 %

3.0 Maximum

Coal and Lignite (Percent)

Trace

0.5 Max.
(Not Slag Sand)

Organics

No. 1

No. 3 Standard

Specific Gravity

2.65

Absorption (%)

1.30

REMARKS: * Suggest Supplier reduce this.





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CLIENT'S No.

54-H-60971-B

March 17, 1967

REPORT

LABORATORY No. 652582

ORDER No. FG-16582

C - AGGREGATE TESTS

COARSE AGGREGATE - TEST RESULTS N. Y. STATE NO. 1

GRADATION

Sieve Size	Cumulative Percent Retained	Percent Passing	Specifications Percent Passing ASTM C-33 No. 67 3/4" to No. 4
1"	0	100	100
3/4"	0	100	90-100
1/2"	4	96	- - -
3/8"	63	37	20-55
No. 4	96	4	0-10
No. 8	98	2	0-5
Clay Imps (Percent)		None	0.25 Maximum
Ft Particles (Percent)		Not Determined	5.0 Maximum
Finest Part than No. 200		Not Determined	1.0(1.5 Crushed Agg.) Maximum
Coal or Lignite (Percent)		None	1.0(Not Slag) Max.
Soundness (Sodium Sulfate) (%) (5 Cycles)		Not Determined	12 Maximum
Abrasion (L.A.) (Percent) 500 Rev.		Not Determined	50 Maximum
Specific Gravity		2.78	
Absorption		0.68%	

REMARKS: Test results conform to given specifications.





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March 17, 1957

LABORATORY No.

552582

ORDER No. PT-16582

CLIENT'S No.

54-H-60971-B

REPORT

C - AGGREGATE TESTS

COARSE AGGREGATE - TEST RESULTS N.Y. STATE NO. 2

GRADATION

<u>Sieve Size</u>	<u>Cumulative Percent Retained</u>	<u>Percent Passing</u>	<u>Specifications Percent Passing N.Y. State No. 2 1" to No. 4</u>
1 1/2"	0	100	100
1 1/8"	2	98	90-100
3/4"	24	76	- - -
1/2"	89	11	0-15
3/8"	96	4	
Specific Gravity		2.78	
Absorption		0.70%	

Sieve
SizeCombined Grading No. 1 & No. 2
40% No. 1 and 60% No. 2Specifications
ASPH C-39 10.57

1 1/2"	100	100
1 1/8"	99	95-100
3/4"	87	- - -
1/2"	47	25-60
3/8"	18	- - -
No. 4	5	0-10
No. 8	1	0-5

REMARKS: Test results conform to given specifications.



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March 17, 1967

LABORATORY No. 652582

ORDER No. FJ-16582

CLIENT'S No. 54-H-60971-B
(PRELIMINARY RESULTS)

REPORT

D - VOID SPACING FACTOR (ASTM C-457)

MIX B-2

7 Sk. - 4% Air (4.2 Plastic)

TEST RESULTS

	Air Content Determined on Hardened Concrete	Spacing Factor
--	--	-------------------

1. Non-Vibrated (Casually filled cylinder)

4.45%

0.0070"

2. Vibrated (Internal Vibrator)

2.97

0.0073"

MIX B-2 - Modified 7 Sk. 3% Air

MIX PROPORTIONS B-2 MODIFIED LABORATORY BATCH:

Cement, lbs. 65.8

Fine Agg. lbs. S.S.D. 125.0

Coarse Aggregate lbs. 180.0

S.S.D. (N.Y. No. 1)

Admixture 31 ozs. Liquid Plasticizer/cu. yd.

1 1/2 ozs. Daren/cu. yd.

Slump, in. 4.0

Air Content % (Plastic Concrete) 3.0

Water, Gals. 3.75

Unit Weight (Plastic)

lbs./cu. yd. 150.2

W/C Ratio Gals./sk. 5.35

TEST RESULTS

	Air Content Determined on Hardened Concrete	Spacing Factor
--	--	-------------------

3. Non-Vibrated

3.87%

0.0140"

4. Vibrated (Internal Vibrator)

2.9%

0.0124"

Recommended Maximum Spacing Factor - - - - - 0.0080"

REMARKS: Based on these preliminary results, a minimum of 4% air as determined by ASTM C-431 should be maintained for proper durability on structures that will be severely exposed to weathering. A higher air content and lower spacing factor is recommended for unusually severe exposure. Correlation of air content determined on plastic concrete and that determined by ASTM C-457 is considered good.





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March 17, 1967

LABORATORY No. 652582

ORDER No. PC-16582

CLIENT'S No. 54-II-60971-B

REPORT

E - LENGTH CHANGE

METHOD ASTM C-494 (14 Day Moist Storage)

SERIES A - TEST RESULTS

PERCENTAGE SHRINKAGE

MIX NO.	<u>A1 (6 Sk. 3/4")</u>			<u>A2 (7 Sk. 3/4")</u>			<u>A3 (8 Sk. 3/4")</u>		
AGE	Specimen			Specimen			Specimen		
	<u>a</u>	<u>b</u>	<u>c</u>	<u>a</u>	<u>b</u>	<u>c</u>	<u>a</u>	<u>b</u>	<u>c</u>
28 Days	0.028	0.028	0.027	0.028	0.027	0.027	0.027	0.025	0.025
	Ave. 0.028			0.027			0.026		
2 Months	Duo 5/8/67								
3 Months									
6 Months									
9 Months									
1 Year									





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March 27, 1967

LABORATORY No. 652582

CLIENT'S No. 54-K-60973-B

ORDER No. PQ-16582

REPORT

SERIES B - TEST RESULTS

PERCENTAGE SHRINKAGE

MIX NO.	<u>E2 (6 Slc. 1")</u>			<u>E2 (7 Slc. 1")</u>			<u>E3 (8 Slc. 1")</u>		
AGE	Specimen								
	<u>a</u>	<u>b</u>	<u>c</u>	<u>a</u>	<u>b</u>	<u>c</u>	<u>a</u>	<u>b</u>	<u>c</u>
28 Days	0.026	0.024	0.024	0.025	0.026	0.026	0.025	0.027	0.0225
	Avg. 0.025			0.026			0.026		
2 Months									
3 Months									
6 Months									
9 Months									
1 Year									

Dec 5/8/67

REMARKS: The length change test results confirm research conducted recently by the National Ready Mix Research Laboratory that within parameters to cement factor and aggregate size tested there are no significant differences in shrinkage.





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LABORATORY No. 652582

ORDER No. PG-16582

CLIENT'S No. 54-H-60371-B

March 17, 1967

REPORT

LEHIGH TYPE II

P - PORTLAND CEMENT TESTS

PHYSICAL ANALYSISTest MadeResult of TestASTM C150
Type II
Specification

Autoclave Soundness

/0.12%

/0.80% Maximum

Air Content

5.0%

12.0% Maximum

Setting Time (Vicat)

Initial

3 Hrs., 5 Mins.

45 Minutes Min.

Compressive Strength (P.S.I.)

3 Days

2350 Avg.

1000 Minimum

7 Days

3520 Avg.

1800 Minimum

28 Days

6075 Avg.

3500 Minimum

CHEMICAL ANALYSIS

Silicon Dioxide

21.60%

21.0 Min.

Aluminum Oxide

3.85%

6.0 Max.

Iron Oxide

4.35%

6.0 Max.

Magnesium Oxide

2.25%

5.0 Max.

Insoluble Residue

.25%

.75 Max.

Loss on Ignition

.95%

3.0 Max.

Sulfur Trioxide

2.46%

2.5 Max.

Tricalcium Aluminate

2.8%

8 Max.

Tricalcium Aluminate plus

57.8%

58 Max.

Tricalcium Silicate

Blaine Fineness

3112

2800 Min.

Total Alkali as Sodium Oxide

.53%

- - - -

The above sample complies with the specification requirements.





PITTSBURGH TESTING LABORATORY

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LABORATORY No. 653582

ORDER No. P-16562

CLIENT'S No. 54-H-60971-B

March 17, 1967

REPORT

MORTAR MAKING PROPERTIES OF FINE AGGREGATE

- A) - Compressive strength of mortars using Manitou Concrete Sand and
ASTM C190 standard sand.

<u>Test Made</u>	<u>RESULT OF TEST</u>	
	<u>Concrete Sand</u>	<u>ASTM C190 Standard Sand</u>
<u>Compressive Strength (PSI)</u>	<u>Manitou Concrete Sand</u>	<u>ASTM C190 Standard Sand</u>
3 Days	2690 Average	2250 Average
7 Days	3925 Average	3330 Average
28 Days	6735 Average	5850 Average

Test results show that the Manitou Sand has improved strength-
making properties.

Respectfully submitted,

PITTSBURGH TESTING LABORATORY

Joseph F. Artuso, Manager,
Cement & Concrete Department

JFA/nab

12-Westinghouse Electric Corp.

1-Mr. Hans Larson

1-Mr. W. Ferrell

1-Manitou Constr. Company

1-FEL, Rochester

1-FEL, Buffalo



ELASTOMERIC PAD
TECHNICAL SPECIFICATION

TECHNICAL SPECIFICATION
FOR
ELASTOMER BEARING PADS
FOR THE
BROOKWOOD PLANT UNIT NO. 1
OF THE
ROCHESTER GAS AND ELECTRIC CORPORATION
ROCHESTER, NEW YORK

1.0 GENERAL

The Supplier shall furnish a quantity of 320 elastomer bearing pads with materials and construction as hereinafter specified and of the following dimensions:

- a. Plan Area - 42 inches by 9 inches.
- b. Neoprene - Two layers of neoprene shall each be 0.84 inches thick.
- c. Steel Shims - An outer shim on one face only and one shim between the two neoprene layers shall each be a minimum of 16 gage.

2.0 NEOPRENE

2.1 The elastomer portion of the bearing pad shall be new, first run material, known as "Neoprene", with necessary plasticizers, extenders, deoxidizers, etc. required to ensure a product that will meet the specification requirements as hereinafter defined. Pads shall be cast under heat and pressure and shall be individually molded to the size and shape specified hereinbefore. Pads shall be furnished with only one lamination as detailed in Section 1.0. All molding and cutting shall be carefully performed to obtain pads that are flat, smooth, and free of mis-runs, mold flash or surface irregularities, and in which all cuts are smooth, straight and vertical and free of nicks or extraneous rubber.

2.2 Variations from specified dimensions shall not exceed the following:

Thickness	+ 1/16 inch
Width	+ 1/8 inch
Length	+ 1/8 inch

2.3 The bond between neoprene and metal shall be such that when tested for separation, failure shall occur in the neoprene and not between the neoprene and steel.

2.4 The neoprene shall have a nominal durometer hardness of 50. Physical requirements for the neoprene shall be as follows:

a. Original Physical Properties

- (1) Tear resistance*, ASTM D524 Die C,
pounds per inch of thickness, minimum

130.

*Test results for these properties for test samples prepared from finished pads shall be not more than 10 percent below the specified value.



ADDENDUM NO. 1
TO THE
TECHNICAL SPECIFICATION
FOR
ELASTOMER BEARING PADS
FOR THE
BROOKWOOD PLANT UNIT NO. 1
OF THE
ROCHESTER GAS AND ELECTRIC CORPORATION
ROCHESTER, NEW YORK
(1-25-67)

1.0 GENERAL

Change quantity from "320" to "324".
Change thickness of neoprene from "0.84 inches" to "11/16 inches".
Delete item c and insert the following:

"c. Steel Shims - An outer shim on each face with minimum thickness of 16 gage and one shim between the two neoprene layers with thickness as required to maintain location as specified herein-after. The middle shim shall be of constant thickness for all pads and shall be not less than 16 gage."

2.0 NEOPRENE

2.2 Delete "Thickness $\pm 1/16$ inch" and insert in lieu thereof the following:

"Total Pad Thickness $\pm 1/8$ inch
Deviation of middle shim from center of pad $1/8$ inch"

2.4 Change nominal durometer hardness of "50" to "55".

a. (2) Change "50 ± 5 " to "55 ± 3 ".

3.0 ADHESIVE

Insert the following at the end of this section:

"Prior to the application of the epoxy-resin the pads shall be cleaned with a solvent to remove any grease or oil and the steel shims cleaned by power tool or wire brush to remove any rust."



INSTALLATION OF ROCK ANCHORS

TECHNICAL SPECIFICATION

TECHNICAL SPECIFICATIONS
For
INSTALLATION OF ROCK ANCHORS
For The
ROBERT EMMETT GINNA NUCLEAR POWER STATION - UNIT NO. 1
Of The
ROCHESTER GAS AND ELECTRIC CORPORATION
Rochester, New York

1.0 SCOPE

These specifications cover the installation of rock anchors for the Containment Vessel for Robert Emmett Ginna Nuclear Power Station - Unit No. 1 of the Rochester Gas and Electric Company.

2.0 REFERENCE CODES AND SPECIFICATIONS

Except as specified hereinafter, all work shall be in accordance with "Standard Specifications for Structural Concrete for Buildings" ACI 301-66 and the PCI "Tentative Recommended Practices for Grouting Post-tensioned Prestressed Concrete".

3.0 MATERIALS FOR GROUT

3.1 Portland Cement

All cement shall be portland cement conforming to "Specifications for Portland Cement", ASTM C-150-64, Type II taken from one very recently manufactured batch. Air-entrained cement shall not be used.

3.2 Water

Mixing water shall be potable with a chloride content no greater than 100 ppm.

3.3 Admixtures

Proprietary admixtures containing a water-reducing retarder and expansion powder shall be used to obtain the expansion of the

grout as specified hereinafter and the properties required for proper placement. The expansive additive shall be either Intra-plast - C as manufactured by the Sika Chemical Company or Intrusion-Aid as manufactured by the Propakt Concrete Company and shall be proportioned and mixed in accordance with the manufacturer's instructions. The exact quantity of expansive additive to be used shall be determined by the preparation of trial batches using the same materials as are to be used in the work to obtain the properties specified hereinafter under Section 5.0 LABORATORY TESTS.

4.0 MIXING PROPORTIONS

The grout used for installing rock anchors shall be a neat cement grout. The proportions of grouting materials shall be based upon laboratory tests made on fresh and hardened grouts prior to their use in the field. The amount of mixing water employed shall be such as to produce a pumpable grout having the consistency of a thick cream or heavy paint. When permitted to stand until setting takes place, the grout should exhibit practically no bleeding or segregation and should expand not less than 6 nor more than 10 percent of its original volume. The water-cementing material ratio shall be as low as possible consistent with pumping requirements with a minimum water-cement ratio of 0.40 to 0.45 by weight.

5.0 LABORATORY TESTS

5.1 General

The proportions to be used for the grout will be tested by a Testing Laboratory to ensure compliance with the specified

properties of consistency and expansion. The Testing Laboratory shall be provided representative samples of the materials to perform these tests. Reimbursement for the Testing Laboratory's services will be by the Purchaser.

5.2 Plastic Volume Change

The expansion of a test specimen of grout of the proposed proportions shall be determined by measuring the change in volume of a grout column. The expansion of the sample shall be periodically observed. Depending upon the type of grout tested, the test may be discontinued at either 3 or 4 hours when it is evident that expansion has practically ceased. At the end of this period, the bleeding water, if any, shall be poured from the surface of the grout into a small graduated cylinder where its volume is observed.

The grout expansion measured as percent expansion based on original grout volume shall be 8 percent \pm 2 percent. Any bleeding water collected on the surface of the grout shall be measured and reported as percent of bleeding based on original volume of grout. In no event shall the grout exhibit bleeding in excess of 0.4 percent.

5.3 Compressive Strength

The compressive strength of the grout with the proportions to be used for the work shall be determined by the Testing Laboratory. The test molds used to determine compressive strength shall be provided with end plates and rods that will insure complete restraint of the grout after specimens are cast.

The standard method for testing two inch (2") cubes is found in ASTM C-109-64 "Standard Method of Test for Compressive Strength of Hydraulic Cement Mortars (using two inch (2") Cube Specimens).

6.0 MIXING

Care shall be taken to remove lumps, oversize material and foreign matter prior to introducing materials into the mixer. The temperature of the cement when placed into the mixer shall not exceed 140°F.

To produce a uniform grout mixture with a minimum of mixing time, a mixer that produces a shearing action in mixing shall be used. This may be accomplished by paddles, discs, or drums running at high speed in a vertical or horizontal position. Mixing time shall be approximately 1/2 minute per bag of cement. Due to the shearing action, a considerable amount of heat is generated and mixing at high speed shall be limited to about 2 minutes. Standard mortar and concrete mixers or hand mixing is not satisfactory. The grout mixture shall be screened with an 8-mesh strainer immediately after it leaves the mixer. Under no circumstances shall grout be retempered. The proportions of the grout shall be such as to have as low a water-cementing material ratio as is consistent with pumping requirements. These proportions are subject to approval by the Engineer.

7.0 PLACING GROUT

7.1 General

Immediately before grouting the water in the bottom of the hole shall be agitated and pumped out to remove any silt clay and fine rock debris. Pumping shall be terminated when the effluent is visibly

free of suspended material. If the rock surface in the hole is not damp, the hole shall be filled with water to thoroughly wet the rock surface. Each hole shall be inspected to determine if there is a flow of water into the hole sufficient to disrupt proper grouting. Any serious problem shall be called to the attention of the Engineer to determine if pressure grouting and re-drilling of the hole is required. Water shall be permitted to rise in the hole until a steady level is attained. The hole shall then be grouted as specified hereinafter. No more than two tendons shall be grouted under water until it has been ascertained by tensioning the initial anchors that the grout develops the required bond capacity. At the Contractors option the holes may be pumped out completely and, if the influx of water does not exceed one foot per hour, and grouted as a dry hole without waiting for tensioning the initial two anchors.

Standby water flushing equipment shall be provided with sufficient capacity to flush out a partially grouted hole if the grouting equipment breaks down during grouting. The placing of grout for each stage shall be a continuous operation. A list of all equipment and a description of the frequency and means for calibrating gages shall be submitted to the Purchaser for approval. No grout shall be placed during heavy rain or when there is heavy rain expected within six hours. The grout being placed is subject to inspection and sampling by the Testing Laboratory to ensure continued compliance with the requirements of Section 5.0 LABORATORY TESTS. Three mortar cubes shall be cast for first stage grout placed in each hole. These cubes shall be broken, two at seven days and

one immediately before the anchor is tensioned: The final cube shall not be tested more than 24 hours before tensioning the anchor in the hole represented by the cube. Since consistency tests are not being performed, the Testing Laboratory will ensure proportions are as approved by the Engineer.

The rock anchors shall be installed in a similar sequence to that specified for tensioning the tendons under Section 8.0 TENSIONING SEQUENCE to avoid an excessive time interval between installation and tensioning.

When the shipment of anchors is received at the jobsite, all tendons shall be examined for shipping damages and location of spacer. As the tendon is lowered into the hole, the tendon shall also be examined for any inconsistencies apparent in the arrangement of the wires.

After the anchor is inserted into the hole and prior to placing the first stage grout, the button heads shall be examined to insure wire lengths are as specified.

Prior to inserting each tendon, the depth of the hole shall be determined and the grout pipe extension protruding below the bottom anchor head cut to length so as to end at the lowest extremity no higher than two inches (2") above the bottom of the hole. The grout pipe shall have a 60° level.

7.2 First Stage Grout

The depth of each hole shall be determined and the volume of grout computed to provide the required embedment length. The grout shall be placed by gravity in a manner to displace any ground water in the hole upward without producing any significant dispersion of the grout. Special care shall be exercised to grout

approximately the first two feet at a slow speed to avoid excessive mixing of the grout with the water. Grout shall be placed to an elevation approximately one foot above the desired level.

Within thirty (30) minutes and after the grout feeder hose has been withdrawn, the pump shall be lowered to an elevation of 11'-0" below the top of the jacking plate and operated slowly until it is evident that no grout is being removed. A hydrostatic head of water shall be maintained constantly to within one foot (1') of the jacking plate by adding water through the hole in the anchor head. Two hours later the pumps shall be lowered to nine inches (9") above the previously pumped grout level (i.e., 10'-3" below the top of the jacking plate) and, while pumping, shall be lowered slowly until the grout level is detected. If grout level is more than six inches (6") below the original level, adjacent holes shall be inspected for grout. If grout level is within the specified tolerances, no further action is required pending tensioning except to maintain the hole full of water. Periodic checks shall be made to ensure that the hole remains filled with water. If the grout level is not within the specified tolerances, the tendon shall be removed and the hole flushed. The cause of abnormal grout rise or fall shall be determined before re-grouting.

7.3 Second Stage Grout

After tensioning the rock anchor and immediately before injecting second stage grout the hole shall be cleaned and filled with water as described herebefore. The second stage grout shall be injected as soon as practicable after the tendon is tensioned. Extreme care should be exercised to ensure that adjacent holes are not

prematurely filled with grout by using excessive grout pressures. The grout shall be injected by a positive displacement pump of the "progressing cavity" type. All hoses, valves, and fittings shall be water tight. Prior to grouting, the entire system (i.e. pump, hoses, valves and fittings) shall be pressure tested with water to ensure water tightness. Provision shall be made to properly vent the cavity when grout is injected. The grout shall be pumped continuously at a slow rate until the cavity is filled. The use of a pressure pot is not permissible. After grout appears at the vent opening, the grout hose shall be withdrawn as grouting continues (wasting excess grout) until it is clear that all entrapped air has been removed, and the duct is completely filled with grout of good quality. One pressure gauge shall be placed within three feet of the pump discharge and a second gauge within 15 feet of the rock anchor head.

After second stage grouting is completed the top anchor head shall be protected with a coating of NO-OX-ID "CM" or approved equal and with a metal cover.

8.0 TENSIONING SEQUENCE

Rock anchors shall not be tensioned until the concrete supporting the tendon hardware (baseplate, movable head and shims) has attained a minimum ultimate compressive strength of 4000 psi. When a tendon is located within 3'-0" of a construction joint in the ring girder, the minimum ultimate compressive strength of the abutting pour shall also be 4000 psi. No rock anchors shall not be tensioned until the grout specimens exhibit a minimum confined compressive strength of 4000 psi.

unless otherwise approved by the Engineer nor shall they be tensioned before the grout has cured a minimum of 10 days. The sequence for tensioning the anchors shall be as follows:

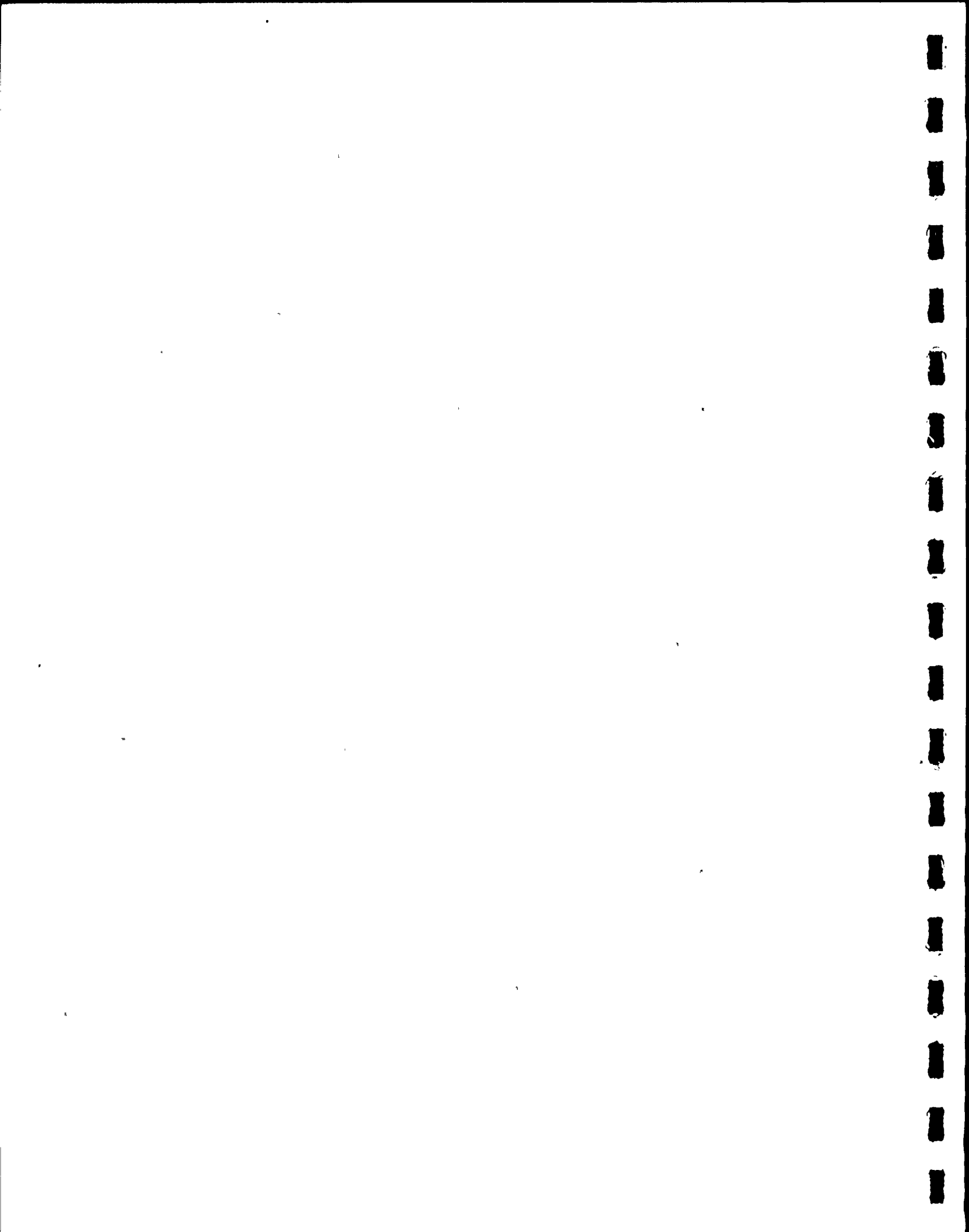
- a. Initially, tension every fourth anchor. There are no limitations on the sequence for tensioning these anchors.
- b. Secondly, tension the anchors located mid way between the tensioned anchors. There are no limitations on the sequence for tensioning these anchors.
- c. Thirdly, tension remaining anchors. Again, there are no limitations on the sequence for tensioning these anchors.

Elevations shall be obtained by the Constructor (Bechtel Corporation) on a minimum of 12 equally spaced locations on the ring girder immediately before and after the tensioning operation. This data shall be submitted to the Engineer.

One week after the tensioning of all anchors is completed, four equally spaced anchors shall be jacked to ascertain the magnitude of losses. This data shall be submitted to the Engineer.

Each rock anchor shall be jacked to eighty percent (80%) of the minimum guaranteed ultimate capacity of the wires. The jacking force shall then be reduced to seventy percent (70%) of ultimate capacity when finally anchored (shimmed) in place. The stress-strain curves for the production lots used shall be submitted to the Engineer along with the final gage reading and elongation for each stressed anchor. If the loss of prestress force due to failure of wires or buttonheads exceeds one half percent (0.5), the Engineer shall be immediately so advised. Based upon stress-strain curves for the wire used, the anticipated elongation shall be determined considering first, the effective length to the

elevation of the top of first stage grout and second, the effective length as the complete tendon length. The Engineer shall be advised if any slippage is discernable during the tensioning operation. Deviations from the specified sequence shall not be permitted without the prior approval of the Engineer. The Engineer shall be provided load and elongation readings at 1000 psi jack pressure, at eighty percent (80%) of the minimum guaranteed ultimate capacity of the wires and at the finally anchored position.



INSTALLATION AND TENSIONING SEQUENCE

I N S T A L L A T I O N			T E N S I O N		
28	121	69	18	54	132
31	118	71	10	58	128
34	114	73	92	62	124
38	111	75	113	66	121
42	108	77	147	70	118
46	104	79	2	74	114
50	100	81	6	78	111
54	96	83	14	82	108
58	93	85	22	86	104
62	90	88	26	89	100
66	87	91	24	95	96
70	84	92	20	98	93
74	80	94	16	102	90
78	76	97	12	106	87
82	72	99	8	110	84
86	68	101	4	116	80
89	64	103	160	119	76
95	60	105	3	122	72
98	56	107	5	126	68
102	52	109	7	130	64
106	48	112	9	134	60
110	44	113	11	138	56
116	40	115	13	141	52
119	36	117	15	144	48
122	32	120	17	150	44
126	29	123	19	153	40
130	27	125	21	156	36
134	33	127	23	158	32
138	35	129	28	1	29
141	37	131	25	154	27
144	39	133	31	152	33
150	41	135	34	148	35
153	43	137	38	145	37
156	45	140	42	142	39
158	47	143	46	139	41
154	49	146	50	136	43
152	51	147			
148	53	149			
145	55	151			
142	57	155			
139	59	157			
136	61	159			
132	63	160			
128	65				
124	67				



SIDE WALL TENDONS
TECHNICAL SPECIFICATION

2-14-67
Revised
(Rewritten)
5-15-67

TECHNICAL SPECIFICATION

SIDE WALL TENDONS
FOR THE
ROBERT EMMETT GRIFFIN NUCLEAR POWER PLANT
UNIT NO. 1
OF THE
ROCHESTER GAS AND ELECTRIC CORPORATION
ROCHESTER, NEW YORK

PRIME CONTRACTOR
WESTINGHOUSE ELECTRIC CORPORATION
ATOMIC POWER DIVISION

SP-5357
FEBRUARY 14, 1967

GILBERT ASSOCIATES, INC.
525 LANCASTER AVENUE
READING, PENNSYLVANIA



1.0 SCOPE OF WORK

The WORK to be performed under this Subcontract shall include the furnishing, fabricating, transporting, installing and tensioning of and applying a corrosion protective covering to the side wall tendons for the Containment Vessel for the Robert Emmett Ginna Nuclear Power Plant - Unit No. 1 of the Rochester Gas and Electric Corporation. The work does not include the furnishing and installation of the rigid tendon conduit, the rock anchor tendons and the covers over the top anchors.

2.0 DEFINITIONS

All parties referred to in this specification are defined as follows:

- a. Owner shall mean the Rochester Gas & Electric Corporation.
- b. Prime Contractor shall mean Westinghouse Electric Corporation, Atomic Power Division.
- c. Engineer shall mean Gilbert Associates, Inc., as Agent for the Prime Contractor.
- d. Constructor shall mean the Bechtel Corporation.
- e. Subcontractor shall mean the successful bidder for the systems specified herein.

3.0 SITE DATA

3.1 Location

The site for Robert Emmett Ginna Nuclear Power Plant - Unit No. 1 is located on the south shore of Lake Ontario, near Smoky Point, in Wayne County, approximately 18 miles northeast of Rochester, New York.

3.2 Transportation

The site will be graded and an access road provided to the work area. No rail facilities are available at the site. The nearest rail head is approximately four (4) miles from the site.

3.3 Tower Crane

A tower crane located immediately adjacent to the Containment Vessel will be available for installation of the tendons. The distance from base of tower at Elevation 235'-8" to the maximum height of hook is 219 feet. The crane has a maximum capacity of 22 tons with zero reach and 5.5 tons for maximum reach. The tower crane will be removed from the job site in January of 1968.

3.4 Access to Work Area

The Subcontractor will be provided all required and reasonable access to the work area so as not to impede his operations.



3.5 Services and Facilities

The availability of services and facilities for this Subcontractor will be detailed by the Constructor, Bechtel Corporation. Generally services and facilities are as follows:

Available Services

- a. Power will be available at existing locations at 480V in a capacity up to 60 amps. Power requirements in excess of this amount will require special consideration and a decision for each contract.
- b. Water will be available at existing locations.
- c. No compressed air will be available.
- d. No telephone service will be available except at the coin phone outside the Bechtel office.
- e. Clean up will be the responsibility of the Subcontractor and if it is not properly accomplished, the work will be performed and a backcharge written to cover the cost.
- f. The use of job site cranes will be offered when they are not in use on the base contract work. This service will be backcharged with a percentage added for overhead.

Work and Laydown Space

Adequate work and laydown space will be provided in areas not adjacent to the main buildings. Limited work and laydown space will be made available, depending on schedule and coordination of crafts, in the buildings and surrounding areas.

Office and Change Areas

No office space or craft change facilities will be provided. Portable toilet facilities will be available.

Job Coordination

Bechtel will coordinate the site work and direct the subcontractor to promote harmony and provide the overall best work sequence. The Bechtel inspecting engineer may request and shall be permitted to witness all work performed under this Contract to assure its continued quality. The Subcontractor's obligation is to provide workmanship within the requirements of the specifications and to make designated test to prove quality.

4.0 REFERENCE CODES AND SPECIFICATIONS

Except as noted hereinafter all work shall be in accordance with the Technical Specifications for Structural Concrete for the Robert Emmett Ginna Nuclear Power Plant - Unit No. 1, ACI 301-66 "Specifications for Structural Concrete for Buildings", and the current recommended practices of the Prestressed Concrete Institute.



5.0 ANCHORAGE SYSTEM

The anchorage system shall be the BBRV post-tensioning system as manufactured by Joseph T. Ryerson & Son, Inc. All anchorage components shall be capable of developing the ultimate tensile strength of the tendon. Test data and calculations establishing the adequacy of the anchorage hardware including the coupling and bushing shall be submitted to the Engineer for review before the work is performed. The bottom anchorage shall be capable of being inserted and drawn through a 6 inch nominal diameter conduit and the side wall tendon to rock anchor coupling made up within the space provided as shown on the Drawings. All anchorage hardware shall be 100% visually inspected to ensure no existence of surface flaws, notches and similar stress raisers.

6.0 MATERIAL

The wire shall be a high strength steel, bright, cold drawn and stress relieved, conforming to "Specifications for Uncoated Stress-Relieved Wire for Prestressed Concrete", ASTM-A421-65, Type BA. Physical and chemical test reports shall be submitted to the Constructor for each reel of wire prior to shipping materials to the field. The tendon fabricator shall cut coupons from each end of each reel, form buttonheads at the ends and test the specimens. These tests shall ensure that the wire ruptures before failure of the buttonhead and that the tensile strength of the wire meets the physical requirements of ASTM A421. Coupons, and the coils they represent, not meeting the aforementioned requirements shall be rejected. Records shall be maintained for each coupon test and for the tendons in which each coil of wire is used. The aforementioned records shall be submitted to the Constructor prior to shipment of fabricated components to the job site.

Anchorage components shall be fabricated from materials as specified on the manufacturer's parts drawings. Requirements for machining, tolerances and heat treating shall be as specified on the parts drawings. Physical and chemical test reports shall be submitted to the Constructor for each heat of material used for anchorage components. Test reports shall be submitted prior to shipping materials to the field.

All materials shall be protected from the weather and excess humidity so as to limit corrosion. Excessive corrosion of any one element may be cause for rejection of the complete shipping lot.

7.0 MANUFACTURING PROCEDURE

Buttonheads shall be cold upset to a nominal diameter of 3/8 inch. All buttonheads shall be visually inspected. A random check with a gauge for size verification shall be made of a minimum of 10% of the buttonheads. All wires in the tendon shall be cut to the same length by cutting all wires under the same conditions. All wires in a tendon under 50 feet long shall be cut to the same length within a tolerance of plus or minus 3/32 inch and in a tendon greater than 50 feet shall be within a tolerance of plus or minus 1/8 inch. Prior to fabrication, procedures shall be developed to ensure that tolerance on wire length have been met. These procedures shall be submitted to the Constructor for review

and approval. Procedures shall also be developed to ensure that replacement wires are cut to the same length as the replaced wires.

All fabricated tendons shall be properly identified, marked and checked to verify conformance with indicated quantity, sizes, lengths, fabrication details, and standards of surface acceptability. The marking shall be in the form of electro-etched markings on the anchor head and shall include identification from which coil of wire it was fabricated. Completed tendons shall be coiled or laid straight and fastened on shipping racks or dunnage so constructed that the tendon is protected from damage in transit and during storage at the job site. Tendon heads shall be protected by a close-fitting plastic sheath or similar protective shroud. The tendons shall be protected in shipment and storage from the weather so as to limit corrosion. Excessive corrosion of any one component may be cause for rejection of the complete shipping lot.

Dimensions of the buttonheads shall be as follows:

- a. Diameter shall be equal to or greater than 0.372 inches and equal to or less than 0.388 inches.
- b. The length shall be equal to or greater than 0.252 inches and equal to or less than 0.272 inches.
- c. A bearing surface shall exist on all sides of the lower portion of the head adjacent to its connection with the wire portion.

Limitations on splits (cracks) in buttonheads are as follows:

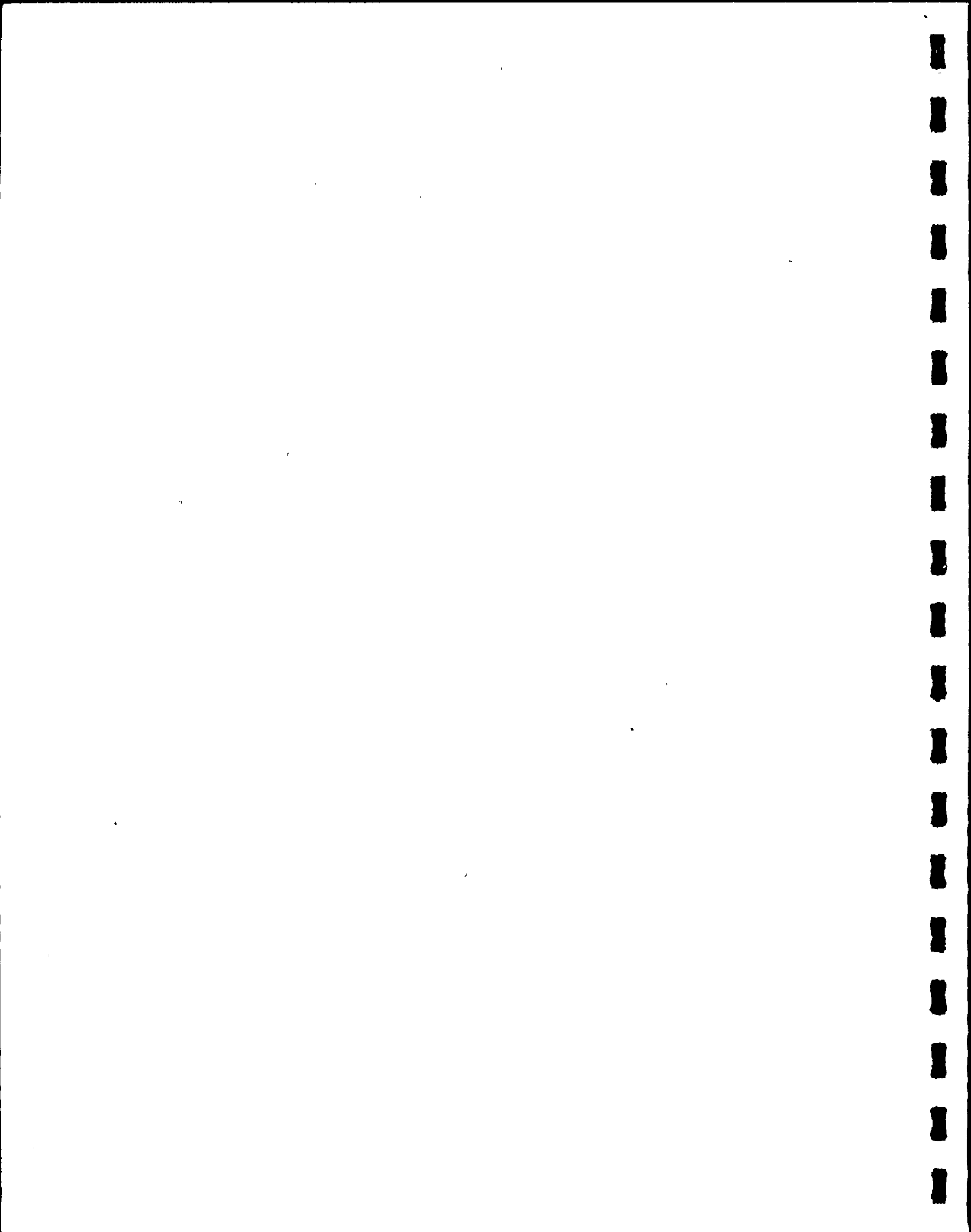
- a. Splits shall not be inclined more than 45° to the vertical axis of the wire.
- b. Sum of the widths of all splits shall be less than 0.06 inches with inclinations less than 20° to the vertical axis of the wire.
- c. No more than two splits shall occur in buttonheads which have splits inclined more than 20° but less than 45° to the vertical axis of the wire. In no event shall the two cracks occur in the same plane. The sum of the widths of all splits shall be less than 0.06 inches.

All materials and fabricated components shall be subject to inspection by the Owner, Prime Contractor, Constructor and Engineer.

The Constructor's inspection shall be permitted to reject any buttonheads which in his opinion, are not satisfactory.

8.0 TENDON INSTALLATION

Included with the Drawings are reference drawings which define the limitations on accessibility to the containment shell. The tendons shall be installed through a six inch nominal diameter rigid conduit after the conduit is embedded in the concrete through the conduit opening located at Elevation 343'-2". Procedures for installation of the tendons and making up of the couplings shall be submitted for review by the Constructor.



If the Subcontractor intends to verify the adequacy of wire length in the field, procedures shall be developed for the replacement of mis length wires in the field or in situ, if damaged or mis length (sic) wires occur as a result of installation. Prior to installing any tendons the conduit shall be purged with hot dry air to remove moisture accumulation.

9.0 CORROSION PROTECTION PRIOR TO SHIPMENT

The wire shall be protected prior to fabrication to ensure that the surface when inspected during fabrication is free from any imperfections other than a light oxide film. The tendon prior to shipment shall be protected with a coating of NO-OX-ID "490" as manufactured by the Dearborn Chemical Co. No alternate material shall be used without the prior approval of the Engineer. Tests shall be performed by the Dearborn Chemical Co. to ensure that there is no indication of chlorides, nitrates, sulfides or any other harmful impurities in the production lot used for this work. Certified test results on chloride, nitrate and sulfide determination shall be submitted to the Constructor.

10.0 PERMANENT CORROSION PROTECTION

10.1 General

The wall tendons shall be encased in a 6 inch pipe as shown on the Drawings. The pipe shall be pumped full of NO-OX-ID "CM", Casing Filler, nuclear grade as manufactured by the Dearborn Chemical Division of W.R. Grace and Co.

It is not the intent of these specifications to dictate exact procedures for filling tendon tubes nor the exact equipment required to accomplish the filling. The Subcontractor has the total responsibility to supply required equipment and materials and detailed procedures. The equipment specifications, any diagrams or sketches, and step by step detailed procedures, however, shall be submitted to the Constructor for his comments. These procedures shall include a method for removing a wax blockage in the conduit such as might occur during cold weather and a limiting temperature below which the wax should not be inserted. Superintendence for this operation shall be obtained from the Dearborn Chemical Division.

10.2 Equipment Requirements

The following suggested equipment and/or materials may be provided by the subcontractor to execute the required WORK.

1. Mobile melting kettle complete with propane burners, thermostatic control of temperature and rated at 500 gallons. Kettle shall be equipped with two burners each rated at 500,000 Btu per hour and supplied with 30 psi propane. Each burner shall be supplied from 100 pound propane cylinder complete with pressure regulators and gas hoses. The kettle shall be MATT 500 gallon tandem kettles (or equal) as manufactured by the Matt Coil-Less Burner Company of Chicago, Illinois.

2. Filling Pump and Motor - Blackmer Pump Type 1404-50-SRC or equal shall be provided. The pump shall be a 2" model with 1800 rpm to 400 rpm totally enclosed gear reducer, bronze internal bearings, with Teflon asbestos packing. The motor shall be a 5 hp, 1800 rpm, 3 phase, 60 cycle, 220/440 volt TEFC type. The pump and motor set shall be directly mounted on the mobile kettle so that total unit can be easily moved about.
3. Hose - The delivery and recirculating hose shall be supplied in sufficient lengths to practically keep the number of moves of equipment to a minimum. The hose shall be the equal of Pennsylvania Flexible Metallic Tubing Co., steel M-100 malleable short coupled, high pressure 2 inch diameter metal hose in twenty foot lengths complete with couplings, swivel female connected to male section with close nipple. It is estimated that a minimum of 300-350 feet of hose is required but required length will depend finally on number of moves and locations available for the kettle and pump unit.
4. Required valving including a 3 way valve for recirculation and quick redirection of flow. Permanent 2" shut-off valves at each supply point will be supplied and installed by others.

10.3 Suggested Procedure

1. Transfer NO-OX-ID from 55 gallon drums to melting kettle. Lifting of drums for material transfer can be done with an air or electrically operated chain hoist suspended from an A-Frame. If climatic conditions dictate, nominal preheating in the drums may be required and can be accomplished with electric tape or steam tracing.
2. Adjust melting kettle thermostat so that maximum temperature of material does not exceed 150°F \pm 5°F.
3. At or near the control temperature (150°F) melting kettle should be filled to no more than 3" from top. Initial charge should be 8 drums of NO-OX-ID. One additional drum may be added when temperature has reached 140°F.
4. After NO-OX-ID in melting kettle has reached 150°F \pm 5°F. begin recirculating as follows:
 - a. Close the permanently installed valve at the tendon entrance and set the three-way valve to deliver full pump discharge through the recirculating loop.
 - b. Start delivery pump with complete return to the melting kettle.
 - c. Continue full recirculation until molten NO-OX-ID in kettle has reached 150°F. At this point, delivery of NO-OX-ID to the tendon cavity may proceed.
5. Open the tendon entrance valve and reset the three-way valve to direct full pump discharge to the tendon cavity.



Time required for filling should be approximately 5 minutes and must include a positive means of communication between a person observing expansion reservoir at the top of the tendon and the valve operator at the bottom of the assembly.

6. When full signal is given reset the 3-way valve to direct the pump discharge back to the melting kettle by way of the by-pass loop. No additional material shall be added to the kettle while performing Steps 5 and 6.
7. Before disconnecting, close the permanently installed valve at the tendon entrance.
8. Three barrels of NO-OX-ID CM should now be added to the melting kettle.
9. Disconnecting and reassembly to a new tendon can proceed while melting kettle is recirculating and newly added material is heating.
10. When all is ready, then procedure may be repeated as described in steps 4c, 5, 6, 7, etc.

10.4 Material Properties

The NO-OX-ID CM Casing Filler, Nuclear Grade shall have the following physical properties:

<u>ITEM</u>	<u>RANGE</u>	<u>METHOD</u>
Specific Gravity	0.88 - 0.90	ASTM D-287
Weight Per Gallon	7.35 - 7.50 lbs	---
Pour Point	110 - 120°F	ASTM D-97
Flash Point (COC)	400°F., Minimum	ASTM D-92
Viscosity at 150°F.	130 - 145 SSU	ASTM D-88
Viscosity at 210°F.	60 - 75 SSU	ASTM D-88
Penetration (Cone) at 77°F.	328 - 367	ASTM D-937
Thermal Conductivity	0.12 BTU/Hr./Ft. ² /°F./Ft. Thickness (approx.)	---
Specific Heat (Heat Capacity)	0.51 BTU/lb./°F. (approx.)	---
Shrinkage Factor from 150°F. to 75°F.	3.5 - 4.5%	---

10.5 Quality Control Tests for Filler Material

The quality control determinations required on raw materials and the finished NO-OX-ID CM filler will include manufacturers raw material



inspection procedures plus additional tests for chloride, sulfide and nitrate content.

The standard tests which shall be performed shall be submitted to the Constructor.

The additional tests which shall be performed are as follows:

Chlorides: The initial screening test on both raw materials and finished product shall be the Beilstein Test. If a positive Beilstein indication is obtained, a confirming test shall be made on water extracts of the product, using standard titration or colorimetric procedures described in ASTM D-512-62T. A limit of 10 ppm chloride shall be set for either raw material or finished product.

Sulfides: The method shall be a water extraction followed by a total sulfide determination. To the extraction water shall be added zinc acetate to precipitate sulfides. Sulfides present shall then be measured in accordance with Paragraph 8. of ASTM D-1255. An alternate colorimetric procedure may also be used in which sulfides are volatilized from an acidified extraction solution, to create a colored spot on lead acetate paper. Spot intensity is measured to determine sulfide concentration. The extraction procedure shall be according to ASTM D-1255. A limit of 10 ppm sulfides shall be set for either raw material or finished product.

Nitrates: The method shall be a water extraction followed by chloroform extraction of the water extract, followed by colorimetric measurements, based on ASTM D-992-52. Either the Brucine or phenoldisulfonic acid procedures may be used. A limit of 10 mg per liter shall be set for either raw material or finished product.

10.6 Space Limitations for Equipment

The maximum width of kettle and pump set shall be 7 ft. but preferably less.

Twenty-four inch diameter manholes are provided along the periphery of the containment vessel and maximum lateral dimension between the manhole and furthest fill connection is estimated to be 100 ft. The Subcontractor shall supply sufficient hose to accommodate this distance plus additional distances considering location of kettle-pump set relative to the manhole and vertical distances.

The Subcontractor shall describe equipment he intends to provide for the execution of his WORK and submit drawings showing equipment and dimensional data in sufficient detail for the Constructor to judge their adequacy.

The Subcontractor shall provide adequate fire protection in the vicinity of the kettle and shall obtain approval from the Constructor of the adequacy of such protection prior to using the kettle.



10.7 Temperature Control

Depending on when actual operation is scheduled there may be a requirement for electric tape tracing of the pipe and/or hot dry air purging of the tendon tube to minimize cooling of filler below 135°F until completely filled.

The Subcontractor shall be responsible for showing how he can assure that filler temperature of material in tendon tube will not fall below 135°F until end of fill cycle.

11.0 TENSIONING

11.1 General

Tendons shall not be tensioned until all concrete for the complete containment shell, including the dome, has been placed and until the concrete below the top anchorage has attained a minimum ultimate compressive strength of 5000 psi and been in place a minimum of 28 days. The tensioning of tendons shall be done using a minimum of four jacks spaced evenly about the circumference of the vessel. Stressing positions shall be alternated to prevent concentrations of multiple stressed tendons adjacent to multiple unstressed tendons. This shall be accomplished initially by tensioning every 40th tendon, then every 20th tendon, then every 10th tendon, etc. The complete sequence and procedure for tensioning all tendons including the additional tendons at the two large openings shall be developed by the Subcontractor and is subject to approval by the Constructor.

Elevations will be obtained by the Constructor on a minimum of twelve equally spaced locations on the ledge at Elevation 343'-2" immediately before and after the tensioning operation. This data shall be submitted to the Engineer.

One week after the tensioning of all tendons is completed, four equally spaced tendons and shall be jacked to ascertain the magnitude of losses. This data shall be submitted to the Engineer.

Each tendon shall be jacked to eighty percent (80%) of the minimum guaranteed ultimate capacity of the wires. The jacking force shall then be reduced to seventy percent (70%) of ultimate capacity when locked off (shimmed in place). The stress-strain curves for the production lots used shall be submitted to the Engineer along with the final gage reading and elongation for each stressed tendon. If the loss of prestress force due to failure of wires or button heads exceeds one-half of one percent ($1/2\%$), the Engineer shall be immediately so advised.

11.2 Force and Strain Measurements

Force and strain measurements shall be made by measurement of elongation of the prestressing steel after taking up initial slack and comparing it with the force indicated by the jack-dynamometer or pressure gauge. The gauge shall indicate the pressure in the jack within plus or minus two percent. Force-jack pressure gauge or dynamometer combinations shall be

calibrated against known precise standards just before application of prestressing forces begins and all calibrations shall be so certified prior to use. Pressure gauges and jacks so calibrated shall always be used together. During stressing, records shall be made of elongations as well as pressures obtained. Jack-dynamometers or gauge combinations shall be checked against elongation of the tendons and the cause of any discrepancy exceeding plus or minus 5 percent of that predicted by calculations (using average load elongation curves) shall be corrected and if caused by differences in load-elongation from averages, shall be so documented. Calibration of the jack-dynamometer or pressure gauge combinations shall be maintained accurate within the above limits and if requested by the Purchaser, shall be recalibrated or newly calibrated combinations substituted during and at the end of the tensioning operations.

11.3 Load Cell Installation and Protection

Load cells shall be installed on a maximum of four tendons located at the movable head. These strain devices will be monitored during the tensioning operation and used during subsequent pressure testing. All load cells will be furnished by the Constructor.

11.4 Additional Corrosion Protection

Following the tensioning of all tendons and after the wax has cooled to ambient temperature, additional filler material (NO-OX-ID "CM" Casing Filler-Nuclear Grade) shall be inserted so as to completely fill all voids and cover the top anchors.

12.0 WIRE SURVEILLANCE SPECIMEN

A total of forty (40) tendons shall include an additional unstressed 1/4 inch diameter wire specimen obtained from a reel represented in the tendon. The specimen shall be a minimum of ten (10) feet in length and shall be located in the installed tendon between approximately Elevations 240 ft. and 250 ft. Means shall be provided to remove the specimen from the top side of the movable anchor head without having to remove the tendon. The wire ends shall be finished so as to ensure that no adjacent wires are damaged when the specimen is removed from the bundle. Details of the specimen shall be prepared and submitted to the Engineer for review and approval. A mock-up shall be prepared to demonstrate that the specimen can be readily removed.

13.0 DRAWINGS

The following drawings set forth the location and extent of the WORK to be performed under this Subcontract and are hereby expressly made a part of this specification:

<u>Drawing No.</u>	<u>Title</u>
D-421-007-IV	Reactor Containment Vessel Wall Reinforcement Sectional Plans & Details
D-421-031-I	Reactor Containment Vessel Exterior Wall Reinforcement Arrangement of Tendon Conduit & Reinforce- ment 0° - 90°
D-421-032-I	Reactor Containment Vessel Exterior Wall Reinforcement Arrangement of Tendon Conduit & Reinforce- ment 90° - 180°
D-421-033-I	Reactor Containment Vessel Exterior Wall Reinforcement Arrangement of Tendon Conduit & Reinforce- ment 180° - 270°
D-421-034-I	Reactor Containment Vessel Exterior Wall Reinforcement Arrangement of Tendon Conduit & Reinforce- ment 270° - 360°
D-421-035-0	Reactor Containment Vessel Exterior Wall Reinforcement Arrangement of Tendon Conduit & Equipment & Personnel Penetrations
D-521-047	Reactor Containment Vessel Miscellaneous Steel Reinforcing Ring for Equipment Access Opening



STRUCTURAL CONCRETE
TECHNICAL SPECIFICATION



1.0 SCOPE

These specifications cover all cast-in-place structural concrete for Brookwood Unit No. 1 of the Rochester Gas and Electric Company, the station site being located.

2.0 REFERENCE CODES

All concrete and concrete work shall be in accordance with the "Proposed Specifications for Structural Concrete for Buildings", ACI 301, copy of which is attached hereto, the latest edition of the State Building and Construction Code for the State of New York, and applicable standards of the Department of Public Works of the State of New York, all except as hereinafter revised or appended.

3.0 CONCRETE PROPORTIONS

3.1 DESIGN TYPE

All concrete structures except as noted hereinafter are designed on the basis of a working stress design. The shell of the containment vessel is designed on the basis of ultimate strength.

3.2 CONCRETE STRENGTH

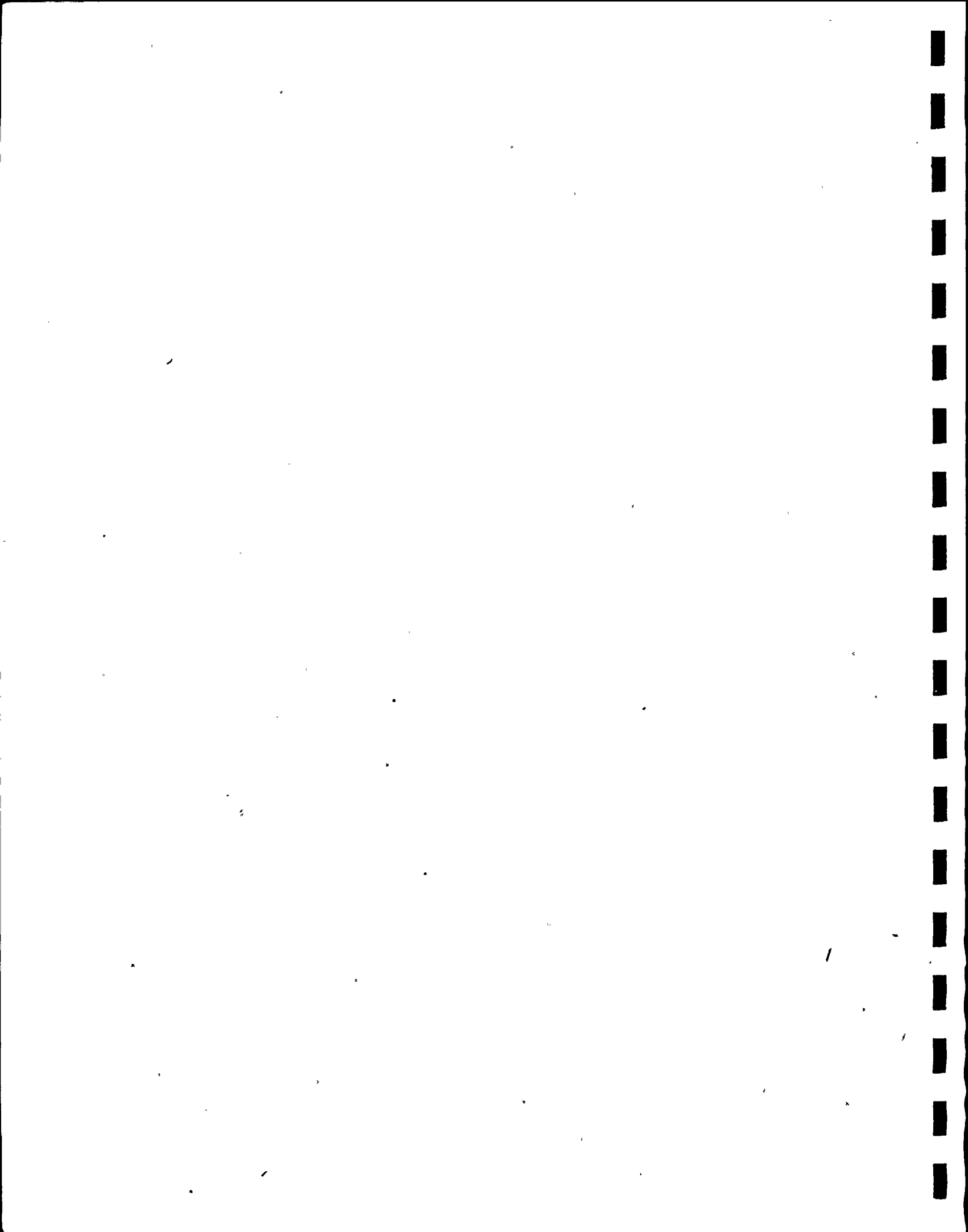
Except as noted hereinafter all structural concrete shall have a minimum ultimate compressive strength of 3000 psi in 28 days. Concrete fill shall have a minimum ultimate compressive strength in 28 days of 3000 psi or 1500 psi as designated on the Drawings. The structural concrete for the containment vessel shell including the ring girder, cylindrical walls and dome shall have a minimum ultimate compressive strength of 5000 psi in 28 days. The detailed requirements for high strength concrete in the containment vessel will be designated on the Drawings.

The determination of the water-cement ratio to attain the required strength shall be in accordance with Method 2, Section 308 (b) of Proposed ACI 301. The Testing Laboratory shall submit to the Engineer for approval the proportions proposed for use and shall also furnish the required test data as evidence that the proportions selected will produce concrete of the specified quality.

4.0 CEMENT

All cement shall be portland cement conforming to "Specification for Portland Cement", ASTM C150-64, Type II, except as otherwise specified hereinafter or on the Drawings or as specifically approved in writing by the Engineer. The cement shall be confined to a single brand with an established reputation for being uniform in character and shall be acceptable to the Engineer.

The Contractor shall store the cement in a dry place and in such a manner as to permit easy access for proper inspection and identification of each shipment. All cement stored at the mixing plant or construction site more than six months shall be resampled and tested before use.



5.0 AGGREGATES

5.1 FINE AGGREGATE

Fine aggregate shall conform to Proposed ACI 301 and to the State of New York, Department of Public Works Specification latest edition. Only natural sand shall be used. Samples of the proposed aggregate shall be submitted to the Testing Laboratory for such tests as the Engineer may require. The aggregate shall not be used unless approved by the Engineer in writing after the results of the tests have been ascertained. The source of the fine aggregate shall not be changed without the written approval of the Engineer.

5.2 COARSE AGGREGATE

Coarse aggregate shall conform to Proposed ACI 301 and to the State of New York Department of Public Works Specification latest edition. Samples of the proposed aggregate shall be submitted to the Testing Laboratory for such tests as the Engineer may require. The aggregate shall not be used unless approved by the Engineer in writing after the results of the test have been ascertained. The source of the coarse aggregate shall not be changed without the written approval of the Engineer.

The maximum size of aggregate shall not be larger than $1/3$ of the minimum dimension of the member nor larger than $3/4$ of the clear distance between reinforcing bars. The maximum size of aggregate where concrete is used for fire proofing of structural steel shall not be larger than $1/5$ the distance between form and steel member.

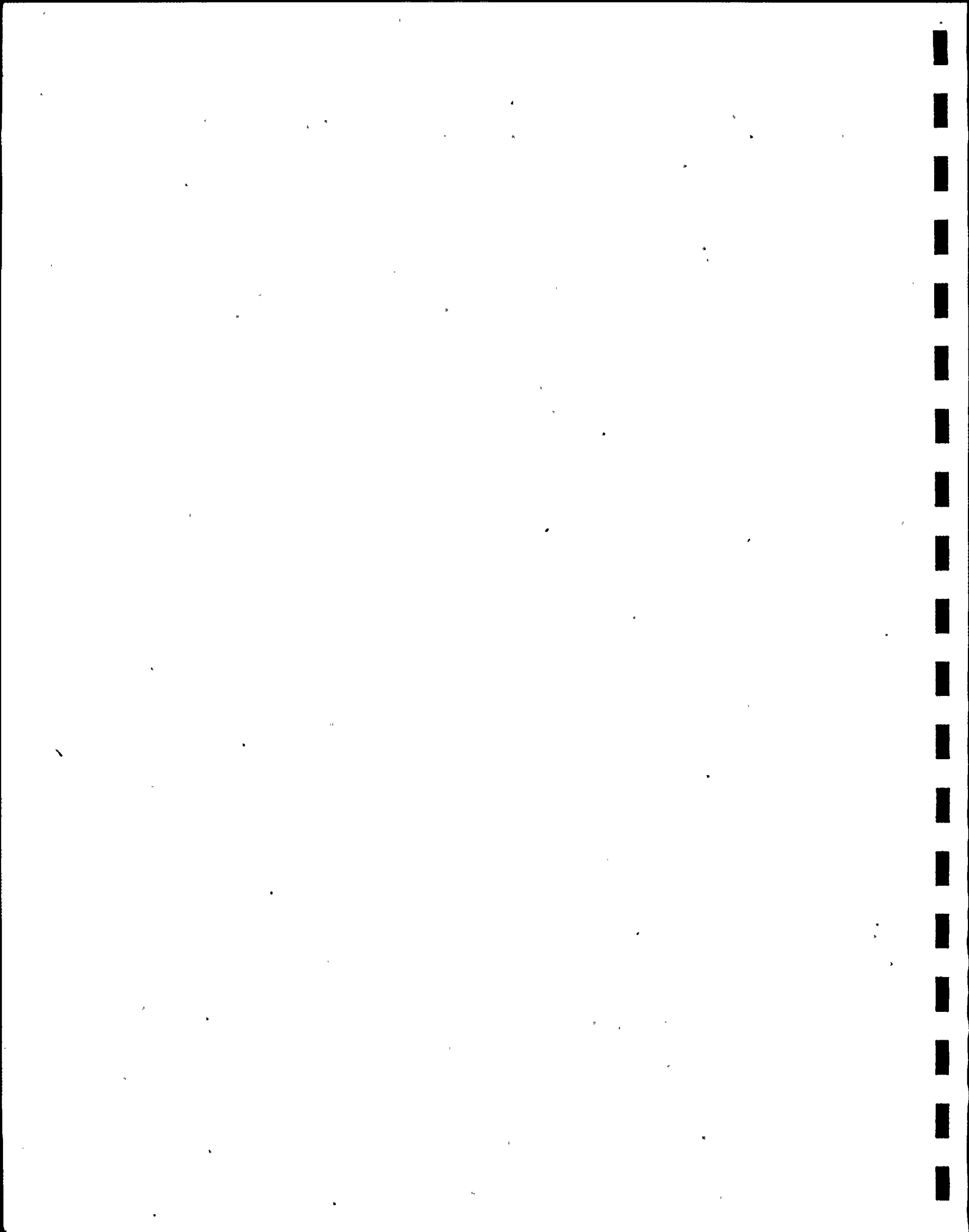
In addition to the above mentioned limitations the maximum size of coarse aggregate for the various portions of a structure shall not exceed the following:

<u>Portion of Structure</u>	<u>Maximum Size of Coarse Aggregate Based on Square Screen Openings</u>
Reinforced foundation walls, footings, piers, plinths, plain footings, caissons and substructure walls	1 1/2 inches
Supporting slabs, beams and reinforced walls	3/4 inch
Fire proofing of structural steel	3/8 inch stone or pea gravel
Pavement and slabs on fill	2 inches

6.0 ADMIXTURES

6.1 AIR ENTRAINING ADMIXTURE

All structural concrete shall be considered subject to potentially destructive exposure and shall contain entrained air in amounts conforming with Table 304 (b) of Proposed ACI 301. An air entraining admixture shall be used conforming to "Specifications for Air-Entraining Admixtures for Concrete", ASTM C260-63T.



6.2 WATER REDUCING DENSIFIER

A water reducing densifier shall be added to all structural concrete with a required ultimate compressive strength equal to or greater than 3000 psi at 28 days. The admixture shall be "Plastiment", a product of Sika Chemical Company. The quantity to be added, the controlling temperatures and the method of mixing shall conform to the manufacturer's recommendations for use of their product.

6.3 CALCIUM CHLORIDE

Admixtures containing calcium chloride shall not be used.

7.0 WATER-CEMENT RATIO

Maximum water-cement ratio for various strengths of concrete shall be as follows:

<u>Compressive Strength (psi at 28 days)</u>	<u>Gallons of Water/ Sack of Cement</u>
5000	5
3000	6

8.0 FORMWORK

8.1 GENERAL

All poured concrete shall be formed, including the sides of footing and other portions of structures below grade, except that rock cuts shall be used as forms for vertical surfaces as shown on the Drawings and/or as directed by the Engineer. Earth cuts shall not be used as forms for vertical surfaces.

8.2 MATERIAL

Forms shall be wood or metal that are of sufficient strength and rigidity, and have a surface suitable for the required finish. If wood is used to form concrete that will be exposed to view, it shall be made with 3/4 in. thick Douglas fir B/B "Plyform" as graded by D.F.P.A. Concrete that will be concealed from view may also be formed with 3/4 in. thick "Plyform", as above, or else shall be formed with seasoned wood boards of not less than 1 in. stock thickness. Boards shall be free from excessive warpage or other defects that would prevent tight joints or affect the true lines and surfaces of the concrete.

All forms lumber shall be new, but it may be reused in various parts of this construction as long as it remains in good condition.

Metal forms shall be straight and free from distortion that would be apparent in the poured concrete. The forms shall be accurately assembled and fitted so that joints will be straight and continuous and so that adjoining surfaces will be flush.

Forms shall be thoroughly cleaned after each use, and surfaces in contact with concrete shall be coated with form oil.

8.3 FORMWORK DESIGN

The design and engineering of the formwork shall be the responsibility

shall be carefully placed to completely fill the voids below the base plates.

15.2 PREPARATION OF SURFACES

Where exposed concrete surfaces are to be covered with grout, the Contractor shall prepare the surface of the concrete so that a good bond between concrete and grout can be obtained. The surfaces shall be scarified, roughened and all laitance removed.

16.0 PRESTRESSED CONCRETE

16.1 TENDON MATERIAL

Tendons shall consist of wire conforming to "Specifications for Uncoated Stress - Relieved wire for Prestressed Concrete", ASTM A421-65 and shall be the BBRV post-tensioning system as manufactured by Joseph T. Ryerson & Son, Inc. The steel tendons for prestressed concrete shall be fabricated with the following quality control procedures being observed:

- a. Physical and chemical test reports shall be submitted to the Engineer for each reel of wire.
- b. The tendon fabricator shall cut samples from each end of a reel, form buttonheads at the ends and test the specimens. These tests shall ensure that the wire ruptures before failure of the buttonhead and that the wire meets the physical requirements of ASTM A421.

High strength alloy steel bars shall conform to SPECIAL STRESSTEEL as manufactured by the Stressteel Corporation with a guaranteed minimum ultimate strength of 160,000 psi.

16.2 PROTECTION OF TENDONS

Rock anchors shall be grouted in two stages for their full length all as shown on the Drawings. The tendons in the cylindrical wall shall be of unbonded construction. The tendons used for unbonded construction shall be coated with grease and wrapped. The type of grease and wrapping shall be with specifications to be issued by the Engineer.

16.3 APPLICATION OF PRESTRESSING FORCE

The sequence for applying the prestressing force to all tendons shall be in accordance with a procedure to be detailed by the Engineer.

17.0 VAPOR BARRIER

The Contractor shall provide and install a vapor barrier under concrete slabs poured on grade as shown on the Drawings. The subgrade shall be level and well tamped before installing the vapor barrier. Where necessary, a layer of sand shall be applied to prevent any protrusions from rupturing the vapor barrier. Permanent Moistop as manufactured by the American Sisalkraft Company shall be used. The vapor barrier shall be installed in accordance with the manufacturer's printed instructions in the widest with all joints lapped no less than 6 inches.

18.0 PERIMETER INSULATION

The perimeter insulation to be placed vertically against the foundation walls and horizontally under slabs on grade shall be Styrofoam SB expanded polystyrene insulation board as manufactured by the Dow Chemical Company. The insulation board shall be one inch thick and shall be installed in accordance with the manufacturer's printed instructions.

19.0 QUALITY CONTROL

19.1 PRELIMINARY TESTS

The Westinghouse Atomic Power Division will obtain the services of a Testing Laboratory which will, prior to the Contractor commencing concrete work, make preliminary determinations of controlled mixes, using the materials proposed and consistencies suitable for the work, in order to determine the mix proportions necessary to produce concrete conforming to the type and strength requirements called for herein or on the Drawings. Aggregates shall be tested in accordance with the latest editions of the following ASTM Specifications: C29, C40, C127, C128 and C136. Compression tests shall conform to ASTM Specifications C39-64 and C192-65. The Contractor shall submit to the Testing Laboratory a sufficient time before concrete work will commence all concrete ingredients required by the Testing Laboratory for these preliminary tests.

The proportions for the concrete mixes will be determined by Method 2 of Section 308 of Proposed ACI 301 and as hereinbefore specified.

The Engineer shall have the right to make adjustments in concrete proportions if necessary to meet the requirements of these specifications.

In the event the Contractor furnished reliable test records of concrete made with materials from the same sources and of the same quality in connection with current work, then all or a part of the strength tests specified hereinbefore may be waived by the Engineer, subject however to any provisions to the contrary of building codes or ordinances of the governing authority.

19.2 FIELD TESTS

During concrete operations the Testing Laboratory will have an inspector at the batch plant who will certify the mixed proportions of each batch delivered to the site and sample and test periodically all concrete ingredients. Another inspector at the construction site will inspect reinforcing and form placements, take slump tests, make test cylinders, check air content and record weather conditions. Except as noted hereinafter, test cylinders will be molded, cured, capped and tested in accordance with Proposed ACI 301 except that one of the three cylinders will be tested at 3 days and the remaining two at 28 days. For the containment shell a set of four cylinders will be made for each 50 cubic yards or fraction thereof placed in any one day.

One cylinder shall be tested at 3 days, another cylinder at 7 days

and the remaining two cylinders at 28 days. Slump tests will be made at random with a minimum of one test for each 10 cubic yards of concrete placed, also slump tests will be made on the concrete batch used for test cylinders.

In the event that concrete is poured during freezing weather or that a freeze is expected during the curing period, an additional cylinder will be made for each set and be cured under the same conditions as the part of the structure which it represents.

19.3 TEST EVALUATION

The evaluation of test results will be in accordance with Chapter 17 of Proposed ACI 301. Sufficient tests will be conducted to provide an evaluation of concrete strength in accordance with this specification.

19.4 DEFICIENT CONCRETE

Whenever it appears that tests of the laboratory cured cylinders fail to meet the requirements set forth in this specification, the Engineer and for Testing Laboratory shall have the right, at the Contractor's expense, to:

- a. Order changes to the proportions of the mix to increase the strength.
- b. Require additional tests of specimens cured entirely under field conditions.
- c. Order changes to improve procedures for protecting and curing the concrete.
- d. Require additional tests in accordance with "Methods of Obtaining and Testing Drilled Cores and Sawed Beams of Concrete", ASTM C42-64.

If the aforementioned tests fail to prove that the questionable concrete is of the specified quality, the Contractor shall replace the concrete work as directed, all at the Contractor's expense.

ADDENDUM NO. 1
TO
TECHNICAL SPECIFICATIONS
FOR
STRUCTURAL CONCRETE
FOR THE
BROOKWOOD PLANT UNIT NO. 1
OF THE
ROCHESTER GAS AND ELECTRIC CORPORATION
ROCHESTER, NEW YORK

4.0 CEMENT

Second line after the words "Type II" insert "for moderate heat of hydration".

8.0 FORMWORK

8.1 GENERAL

At the end of this section add the following:

"All exposed edges shall be chamfered. The size of the chamfer strip shall be 3/4 inches unless otherwise noted on the Drawings."

10.0 JOINTS AND EMBEDDED ITEMS

10.1 CONSTRUCTION ITEMS

At the end of this section add the following:

"Construction joint surfaces shall be prepared for the placement of concrete thereon by cleaning thoroughly with wire brushes, water under pressure, or other means to remove all coatings, stains, debris or other foreign material."

10.4 ANCHOR BOLTS AND PIPE SLEEVES

At the end of this section add the following:

"Embedded items shall be checked for line and grade after concrete is placed."

11.0 MIXING CONCRETE

11.2 TRANSIT MIXING

At the end of this section add the following:

"As required by ASTM C94-65 all trucks shall be equipped with a revolution counter."



11.6 BATCH RECORD

At the end of this section add the following:

"As required by ASTM C94-65 the batch ticket shall also include the time loaded, amount of concrete and reading of revolution counter at first addition of water."

14.0 CURING AND PROTECTION

Curing methods detailed in ACI 301-66 shall be used except that a method other than using a curing compound shall be used for initial and final curing of concrete in the containment shell.

19.0 QUALITY CONTROL

19.1 PRELIMINARY TESTS

In the second paragraph change "Section 309" to "Section 308".

19.2 FIELD TESTS

At the end of the third paragraph add the following:

"This cylinder shall be tested at 28 days."

Add the following additional section:

"20.0 WATER

The chloride content of mixing water shall not exceed 100 ppm and turbidity shall not exceed 2000 ppm."



EXTRACTS FSAR

5.1.2 CONTAINMENT SYSTEM STRUCTURE DESIGN

5.1.2.1 General Description

The reactor containment structure is a reinforced concrete vertical right cylinder with a flat base and a hemispherical dome. A welded steel liner is attached to the inside face of the concrete shell to ensure a high degree of leak tightness. On the inside of the liner every weld seam has a leak test channel welded over it. The channels can be pressurized to design pressure for liner leak testing whenever the containment vessel is open. The thickness of the liner in the cylinder and dome is 3/8 in. and in the base is 1/4 in. The containment structure is 99 ft. high to the spring line of the dome and has an inside diameter of 105 ft. The containment vessel provides a free volume of 997,000 cu. ft. An elevation and details of the containment structure are shown on Figures 5.1.2-1 through 5.1.2-5.

The cylindrical reinforced concrete walls are 3 ft. 6 in. thick and the concrete hemispherical dome is 2 ft. 6 in. thick. These shell thicknesses are established to satisfy the requirements of the structural criteria as well as the shielding requirements. The concrete base slab is 2 ft. thick with an additional thickness of concrete fill of 2 ft. over the bottom liner plate. The containment cylinder is founded on rock (sandstone) by means of post-tensioned rock anchors which ensure that the rock then acts as an integral part of the containment structure. The hemispherical dome is reinforced concrete designed for all moments, axial loads and shears resulting from the loading conditions described here. The cylinder wall is prestressed vertically and reinforced circumferentially with mild steel deformed bars. The base is a reinforced concrete slab. The rock anchors are used for all vertical axial loads in the cylinder walls, and thereby avoid the transfer of an imposed shear to the base slab. The structural systems for the containment structure are summarized as follows:

1. Hemispherical Dome - mild steel reinforced concrete
2. Cylindrical Walls
 - a. Vertical Direction - prestressed concrete
 - b. Circumferential Direction - mild steel reinforced concrete
3. Rock Anchors - prestressed

The design ensures that the structure has an elastic response to all loads and that it strains within such limits that the integrity of the liner is not prejudiced. The liner participates with the shell as it reacts to these loads and is designed to ensure the vessel's vapor tightness.

The design of the structural elements are more fully described in Sections 5.1.2.3 and 5.1.2.4.

No drainage system was provided under the containment structure. The maximum ground water elevation in the vicinity of the containment structure is 252 ft. This compares with an elevation at the underside of the base slab of 231'-8". The contention that tensile stresses will produce cracks in the outside concrete face is questionable in that significant constraint is afforded by the irregular surface of the founding rock material. This rock has significant structural characteristics as described in Appendix 2B of the FSAR. Nevertheless whether cracks exist or not it would be imprudent to consider that the concrete is totally impermeable. For this reason the design of the liner, test channels, backup bars (structural tees), anchors on test channels (refer to Figure 5.1.2-31) and the concrete cover were based upon accommodating the full hydrostatic head of water. It is our judgment that a significant corrosion potential for embedded steel does not exist due to the close contact between the alkaline concrete and steel which provides a highly corrosive resistant environment for the liner.

The basement floor elevation of the Containment Vessel is 235'-8". The exterior of the cylinder walls will be covered from the edge of the ring girder to Elevation 253'-0" with a membrane water proofing. No water proofing was placed between the foundation material (rock) and the base slab. The liner and liner anchorage at the base of the vessel were designed



to withstand a theoretical pore pressure equal to the full hydrostatic head of water, 7.7 psi. The site is not subject to significant fluctuations in the ground water elevations. Consequently if the base liner is subject to the assumed water pressure, this pressure should remain essentially constant.

The net bouyant force due to the hydrostatic pressure acting on the containment base is transmitted by the base slab to the cylinder walls.

The side walls of the containment vessel are anchored to the foundation rock with prestressed rock anchors. The anchors place a pre-load between the foundation rock and a ring beam at the base of the side wall. The tendons in the side walls are coupled to the rock anchors and extend to a location 12 feet 6 inches above the spring line to provide accessibility to the upper anchorage and to permit tensioning following the completion of the dome concrete work.

The outer surface of the Containment Vessel can be inspected except to those limited areas where roofs, floors and walls of adjacent buildings preclude access.

A removable cover is placed over the top anchorage head for protection and to provide an expansion reservoir for the tendon protection system (NO-OX-ID CM). Refer to Figure 5.1.2-b for details of this enclosure.

The sequence for the construction of the shell of the containment vessel was as follows:

- a) Excavation was completed to the lines and grades shown on the construction drawings and the exposed rock examined by a soils engineer to ensure its competence.
- b) The concrete for the portion of the ring girder at the base of the cylindrical wall was placed. Sleeves and bearing plates for the rock anchors were embedded in this concrete pour.
- c) The holes for rock anchors were drilled through the embedded sleeves and into the rock. The anchor, which was completely fabricated in the shop, was inserted and the first stage grout

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placed. Following the required curing period the anchor was tensioned and the second stage grout inserted under pressure.

- d) The concrete for the base mat was placed with embedded bars for the back-up of liner welds. The outer concrete pour contains the tension bars (dowel at base of cylindrical walls). The base slabs for the sumps and pit were installed also with embedded bars for back-up of liner welds. The liner for the walls of the sumps was then erected and used an inner form for the placement of concrete.
- e) The liner was erected starting on the base and continuing to the knuckle, the cylindrical wall and the dome. All electrical and mechanical penetrations (i.e., sleeves for penetrations) were installed as liner erection progressed. Essentially all electrical and mechanical penetrations were shop assembled in the cylindrical wall plates. Provision was made to install the equipment access hatch and personnel air locks at a later stage of construction. Temporary openings were provided in the liner cylindrical wall for construction access requirements.

The closure procedure for temporary openings in the liner is similar to that for steel tank construction. Initially special reinforcement was provided around the periphery of the temporary openings. A sufficient width of plate extends beyond the limits of the concrete placement to preclude detrimental heat up of the concrete due to the welding of the closure plate (S). The welding procedures are identical to that used for all liner weld seams.

The preparation of construction joints and placement of concrete in temporary openings is as described in the Fifth Supplement to the FSAR.

- f) The tendon conduit was embedded in the second ring girder pour with provision made for installing the tendon and completing the coupling of rock anchor to sidewall tendon. The enclosure about the coupling was welded to the anchor plate and a window removed to permit making-up the coupling. An expansion bellows was provided where differential motion will occur at the level of the elastomer pads.

- g) The elastomer pads were installed and tendon conduit plus mild steel reinforcing placed. The mild steel reinforcing was temporarily supported from the tendon conduit and the stiffeners on the liner. Concrete placement at temporary openings was delayed and provision made to stagger reinforcement splices at these locations as well as elsewhere on the structure. For the cylindrical wall and dome the liner was used as an inner form.
- h) Where grade is adjacent to the structure, a retaining wall was erected to ensure no earth is placed against the cylindrical wall.
- i) The concrete cylindrical wall was completed and temporary openings closed after they no longer were required for construction. The reinforcement rings about the equipment access hatch and personnel air locks were installed. The reinforcement about the equipment access hatch was not placed until after major components were installed.

The cylinder walls were placed with horizontal joints spaced at approximately 11 ft. centers. Vertical joints were spaced at approximately 42.5 Ft. centers (i.e. the cylinder was divided into approximately eight equal pours). The final six lifts were poured with the spacing of vertical joints increased to approximately 57 Ft. (i.e. six approximately equal pours). Form ties consisting of 1/2 inch diameter threaded studs spaced at approximately two foot centers were welded to the liner (both plate and channel anchors) for attaching the liner to the outer form. The outer form was supported by cantilever construction from the lower pour. No attempt was made to stagger vertical or horizontal joints. A minimum delay of three days was maintained before placing new concrete against abutting pours.

Initial and final concrete curing are the wet method as specified in ACI 301-66.

The dome concrete (i.e. all concrete above the ledge at Elevation 343'-2") was placed as continuous rings with a chord width of



approximately 4.2 ft. the final pour (center "dollar" section) consisted of an approximately 8 ft. diameter section. All concrete was placed in one pour for the full thickness of the concrete shell. A galvanized expanded metal mesh located 1 inch inboard of the exposed face was used as an outer form on the greater sloped portion of the dome (i.e. up to an angle of approximately 55° from the spring line). Form ties in the form of 1/2" diameter studs were welded to the liner plate and attached to the cage of reinforcing bars. Again a minimum delay of three days was maintained before placing new concrete against the previous concrete ring.

- j) The tendons were installed in the embedded conduits and the sidewall tendon and the rock anchor coupled. Then the remaining concrete pour in the ring girders was completed and the wax inserted into the conduit.

- k) The concrete dome was completed and the sidewall tendons tensioned.
- l) Following the tensioning of the tendons the equipment access hatch with inset personnel air lock plus the second personnel air lock were installed. The vessel was then ready for structural and leakage testing.

The principal dome reinforcement is continuous except for the anchorage at Elev. 366'-8" which is provided in the form of a mechanical connection to a continuous circumferential plate. Additional steel to control spalling on the outer face of the shell is provided in the form of welded wire fabric. At the dome to cylinder discontinuity additional reinforcement is provided on both faces with 180° hooks with total anchorage provided to satisfy the requirements of ACI 318-63. Details are shown on Figure 5.1.2-5.

In the anchorage zone of the prestressing steel the major steel provided to withstand bursting forces consists of continuous spirals. Radial reinforcement is provided with 180° hooks around the vertical flexural steel for anchorage. Vertical (meridional) reinforcement used for flexural and temperature resistance is lap spliced in accordance with ACI 318-63 requirements on the basis of splice requirements at points of maximum tensile stress. Details are also shown on Figure 5.1.2-5.

All principal circumferential reinforcement is continuous except at the small penetrations where mechanical anchors are provided as shown typically on Figure 5.1.2-4 and for a limited number of bars at the large openings as described in the Third Supplement. Vertical (meridional) reinforcement is lap spliced (except for special large size bars which are Cadweld spliced) in accordance with ACI 318-63 requirements on the basis of splices at points of maximum tensile stress. At the base of the wall all vertical (meridional) reinforcement is provided with 90° hooks for anchorage. Details are shown on Figure 5.1.2-4.

5.1.2.2 Design Leakage Rate

The containment structure is designed to contain radioactive material which might be released within the containment following a loss-of-coolant accident by passing the initial integrated leak rate test criterion of a maximum leak rate of 0.1% by weight of the free volume of air per day at design pressure.



5.1.2.3 Mechanical Design Bases

General

The containment vessel is a steel lined concrete shell designed ensure that it responds elastically to all loads and strains within such limits that the integrity of the liner is not prejudiced. The liner will be anchored so as to ensure composite action with the concrete shell.

Design Loads

The following loads have been considered in the structural design:

- 1) Internal Pressure
- 2) Test Pressure - 69 psig
- 3) Live Loads - Roof loads plus pipe reactions
- 4) External Pressure - 2.5 psig
- 5) Wind Load
- 6) Internal Temperature
 - (a) Accident
 - (b) Operating - 120°F
- 7) Seismic Ground Accelerations
- 8) Dead Loads
- 9) Prestressing Loads

The thermal loads on the containment vessel and their variation with time are developed on the basis of the transients shown in Figures 14.3.4-2 and 14.3.4-3, Section 14.3.4. The seismic loads were evaluated as outlined in Section 5.1.2.5.

The wind and snow loads used for the design of structures were those specified in State Building Code for the State of New York. The wind loads given in this code are as follows:

<u>Height Above Ground (lb)</u>	<u>Pressure Load (psf)</u>
0-15	12
16-25	15
26-40	18
41-60	21
61-100	24
101-200	28

The snow load specified in the code for the plant location is 40 psf for a flat roof. This value was used also in the design of the containment.

Design Stress Criteria

The design is based upon limiting load factors which are used as the ratio by which accident, earthquake, and wind loads are multiplied for design purposes to ensure that the load deformation behavior of the structure is one of elastic, low strain response. The loads utilized to determine the required limiting capacity of any structural element on the containment vessel are computed as follows:

- a. $C = 0.95 D + 1.5 P + 1.0 T$
- b. $C = 0.95 D + 1.25 P + 1.0 T' + 1.25 E$
- c. $C = 0.95 D + 1.0 P + 1.0 \underline{T} + 1.0 E'$

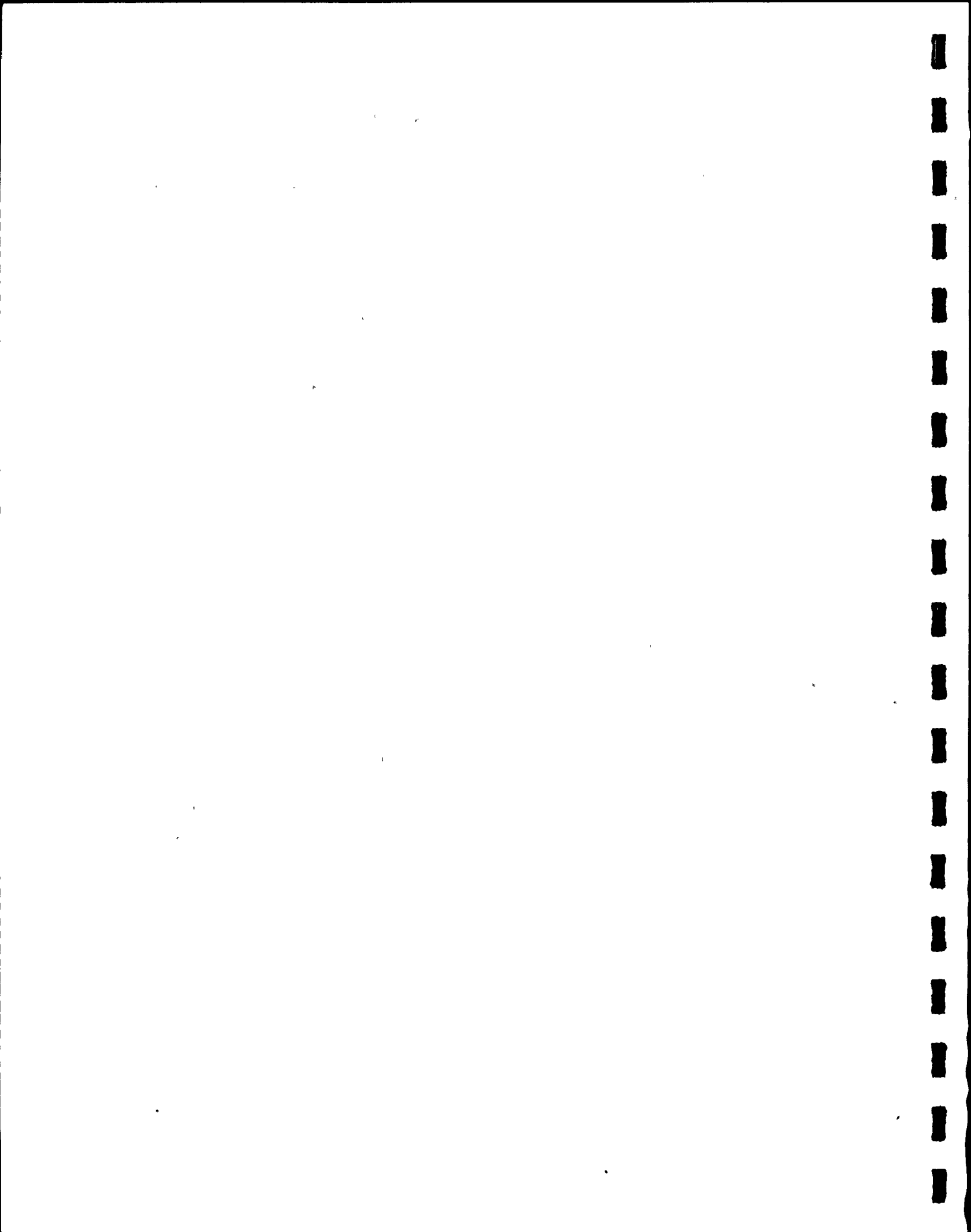
Symbols used in the above equations are defined as follows:

- C: Required load capacity of section
- D: Dead Load of structure
- P: Accident pressure load - 60 psig
- T: Thermal loads based upon temperature transient associated with 1.5 times accident pressure
- T': Thermal loads based upon temperature transient associated with 1.25 accident pressure
- T: Thermal loads based upon temperature transient associated with accident pressure.
- E: Seismic load based on 0.08g ground acceleration
- E': Seismic load based on 0.20g ground acceleration

If the required resisting capacity on any structural component resulting from the wind load on any portion of the structure exceeds that resulting from the design earthquake, the wind load "W" will be used in lieu of "E" in the second equation. The factor of 1.05 times dead load will be used should it control in determining the required load capacity. All structural components will be designed to have a capacity, as defined hereafter, required by the most severe loading combination.

The load factors used in these equations make provision for safety of the containment structure in the same manner as does the ultimate strength design procedure in ACI 318. Because of the refinement of the analysis and the restrictions on construction procedures, the load factors in this design primarily provide for a safety margin on the load assumptions. The load factors utilized in this criteria are based upon the load factor concept employed in Part IV-B, "Structural Analysis and Proportioning of Members - Ultimate Strength Design" of ACI 318-63. The load factor of 0.95 applied to Dead Load represents the accuracy of dead load calculations (i.e., $\pm 5\%$) considering the greater severity of reduced dead loads for tension members. The load factor applied to accident pressure loads is consistent with that suggested by Waters and Barrett^(1,2) as the limit of low strain behavior on prestressed concrete pressure vessels for nuclear reactors. This factor is also consistent with the proposed set of "French Regulations Concerning Concrete Reactor Pressure Vessels" wherein it is stated that: "The design pressure shall not exceed $2/5$ of the pressure calculated to bring about destruction of the structure by rupture of the cables." The load factor considering a tendon stress of $0.60 f_u$ at factored load would therefore equal $0.6 \div 2/5$ or 1.5. The load factor of 1.25 applied to the design earthquake load is consistent with that utilized in ACI 318 Part IV-B, Chapter 15.

The design includes the consideration of both primary and secondary stresses. When a structure experiences only elastic strains there is only a minimal relief of restraints causing secondary stresses. If a structure experiences increased strains beyond the elastic range, the restraints at any point will cease to be as significant due to local yielding in these regions and, if increased loads were applied



until collapse of the structure was imminent, all restraints would be effectively removed and only membrane forces (primary stresses) should be experienced unless premature shear failure were to occur. The design limit for the containment structure has been conservatively established to ensure elastic, low strain behavior at design loads thereby requiring design consideration of all secondary stress effects.

The maximum expected values of "T" at any section are based upon the following conditions:

- a. The maximum operating temperature inside the vessel is 120°F and the minimum ambient temperature outside the vessel is -10°F.
- b. The maximum temperature of the inner surface of the liner (inner face of insulation where the liner is insulated) will be that temperature associated with the factored load, 1.5 times accident pressure. This temperature is approximately 312°F. The design of the shell where the liner is insulated is based upon a maximum temperature rise of 10°F in the liner coincident with maximum pressure.
- c. The maximum operating temperature at the basement floor elevation is 120°F and five feet below the floor elevation is 50°F. The upper two feet of the basement slab were designed for a transient thermal gradient equal to 30°F. Thermal expansion of the basement slab approximately balances drying shrinkage.

The steady state operating thermal transient considered in the design for winter conditions (external ambient temperature equals -10°F) is shown on Figure 5.1.2-34. The steady operating thermal transient for summer conditions was not developed in detail in that it was concluded that such a condition would not affect the reinforcement requirements in that a lesser gradient would exist.



The transient thermal gradients through the containment shell for the insulated liner due to the design basis accident (factored loads) was assumed for purposes of analysis to be the super-position of a liner increase of 10°F. onto the operating thermal gradient described above. This is conservative as compared to the expected results described in Appendix 5B of the FSAR. The maximum concrete fiber temperature where the liner is uninsulated (dome) is 220°F in the region immediately in contact with the liner. The calculated shell elongation due to the pressure load exceeds the concrete fiber elongations due to the thermal load indicating that no restraint of concrete thermal growth occurs.

Load Capacity

Reinforced Concrete

The value of Young's modulus (E_c) for uncracked concrete was assumed to be 4.1×10^6 psi calculated on the basis of the equation in Table 1002(a) ACI 318-63. E_c and Poisson's ratio (ν_c) for cracked concrete were assumed to be zero. This latter assumption is considered to be substantiated by test data^(17,18,19) for reinforcement experiencing stresses in excess of the 20,000 to 30,000 psi. (Refer to Third Supplement to the FSAR regarding similar assumptions regarding the analysis of large openings.)

This structure is prestressed vertically only and the liner is insulated in the prestressed portion. The liner stresses (meridional direction) were calculated to be 4500 psi compression based upon a prestress force of 0.70 fs. The concrete strain due to creep and shrinkage was established as being 320×10^{-6} in./in. This increases the liner stress to 14,100 psi compression at the end of plant life.



Concrete reinforcement is intermediate grade billet steel conforming to ASTM A15-64 and A408-62T with a guaranteed minimum yield strength of 40,000 psi.

The design limit for tension members (i.e. the capacity required for the factored loads) is based upon the yield stress of the reinforcing steel. No mild steel reinforcement will experience average strains beyond the yield point at the factored load. The load capacity so determined is reduced by a capacity reduction factor " ϕ " which provides for the possibility that small adverse variations in material strengths, workmanship, dimensions and control, while individually within required tolerances and the limits of good practice, occasionally may combine to result in any under-capacity. The coefficient " ϕ " is 0.95 for tension, 0.90 for flexure and 0.85 for diagonal tension, bond and anchorage. The coefficient " ϕ " of 0.95 for tension members compares with a coefficient of 0.90 utilized in ACI 318 for ultimate strength design of flexured members. However, in a tension member, unlike the case of a flexural member, only the variation of steel strength and not concrete strength is of concern. Also, the effect of reinforcement misplacement is not as critical as it is for a flexural member. Therefore, the capacity reduction factor of 0.95 is considered to be conservative.

The two equations developed previously for the loss-of-coolant accident and the loss-of-coolant accident combined with the design earthquake could be written as follows for the mild steel reinforced sections:



1. $C = 0.95 \text{ Y.P.} = .95D + 1.0 T + 1.5P$
2. $C = 0.95 \text{ Y.P.} = .95D + 1.25P + 1.0 T' + 1.25E$

To compare these equations with a working strength design the following equations are developed:

1. $f = \frac{(D + P + T) 0.95}{(.95D + 1.5P + 1.0 T)} = 63\%$
2. $f = \frac{(D + P + T + E) 0.95}{(.95 D + 1.25P + 1.0 T' + 1.25E)} = 74\%$

The new symbol in the above equations is defined as follows:

f: Ratio of the working stress to yield stress

Prestressed Concrete

The design for the Containment Vessel provides for prestressing the concrete in the cylinder walls in the longitudinal (vertical) direction with a sufficient compressive force to ensure that upon application of the design load combinations there will be no tensile stresses in the concrete due to membrane forces. In addition to the membrane stresses there are also flexural and shear stresses which result from discontinuity effects. On the basis of the design criteria the concrete stresses and the stresses on the mild steel reinforcing upon application of the combined loads will then be produced by combined flexure and shear and/or compression. The structural elements are then acting in a manner similar to those tested as a basis for Chapter 17 - Shear and Diagonal Tension - Ultimate Strength Design of ACI 318-63 and there is a basis for designing shear reinforcement.

The steel tendons for prestressing consist of high tensile, bright, cold drawn and stress-relieved steel wires conforming to ASTM A 421-59 T. Type BA, "Specifications for Uncoated Stress-Relieved Wire for Prestressed Concrete" with a minimum ultimate tensile stress of 240,000 psi.

The prestressed concrete is assumed to develop no tensile capacity in a direction normal to a horizontal plane. The design load capacity of tension elements is based upon a resultant condition of zero concrete stress due to the maximum combination of primary and secondary membrane forces. Any nominal secondary tensile stresses due to bending will be assumed to cause partial cracking. Mild steel reinforcing will be provided to control this cracking by limiting crack width, spacing and depth. The load capacity so determined will be reduced by a capacity reduction factor " ϕ " which will be conservatively established as 0.95 which compares with a coefficient of 0.90 utilized in ACI 318 for ultimate design of flexural members. In a prestressed tension member only variations in the field-applied tensioning loads are of any concern. Tendon location and concrete strength variations are not critical as they are for flexural members.

Generally, if no tension stresses can be developed in the concrete, prestressed concrete tension members have a relatively low reserve strength above the point of zero stress. If cracking is initiated as the very low tensile stresses are developed in the concrete, all additional loads will be carried by the steel alone. Since the prestressing steel has a relatively small area of cross-section, the strain at any section increases markedly after cracking begins. For this reason this design is conservatively based upon no complete cracking of any prestressed wall section.

Tensile stresses in the concrete resulting from diagonal tension will be permitted. The nominal shear stresses as a measure of this diagonal tension will be less than the maximum value stipulated in Chapter 17 of ACI 318.

The steel tendons are stressed during the post-tensioning operation to a maximum of 80 percent of ultimate strength and locked-off for an initial stress of 70 percent of the ultimate strength. The maximum effective prestress is determined taking into consideration allowances for the following losses which are deduced from the transfer prestress:

- a) Elastic shortening of concrete.
- b) Creeping of concrete.
- c) Shrinkage of concrete.
- d) Relaxation of steel stress.
- e) Frictional loss due to intended or unintended curvature of the tendons.

In no event does the effective prestress exceed 60 percent of the ultimate strength of the prestressing steel or 80 percent of the nominal yield point stress of the prestressing steel, whichever is smaller. The design of all prestressed concrete elements for shear, bond and other design considerations is in accordance with Chapter 26 - "Prestressed Concrete" of ACI 318-63.

The prestressing force applied in the field is determined by measuring tendon elongation and also by checking jack pressure on a calibrated gage or by the use of an accurately calibrated dynamometer. The cause of any discrepancy which exceeds 5 percent is ascertained and corrected. Elongation requirements are taken from load-elongation curves for the steel used.

With the exception of the large openings (refer to the Third Supplement to the PSAR) reinforcing bars are not draped around openings. Consequently the minimum radius is the radius of the cylinder. The reinforcement about small openings is shown typically on Figure 5.1.2.4. The horizontal reinforcement is concentrated near the hole to accommodate stress concentrations. The tendons are draped only if required for clearances. The magnitude of prestress under construction and operating conditions is well within accepted limits based on ACI-318 requirements. The initial average membrane stress is 640 psi. Even a stress concentration factor of 3 results in acceptable stresses. The requirements for anchoring reinforcing bars are discussed in Section 5.1.2.5, Anchorage Stresses.

Liner

The liner is carbon steel plate conforming to ASTM A442-60T Grade 60 with a minimum yield point of 32,000 psi. The liner plate thickness is one-quarter inch for the base and three-eighths inch for the cylinder and dome. Liner welds in general were made from both sides of the plate hence backup strips were not used. In the base where the liner was welded to structural tees, the tees were continuous at all plate intersections.

The load capacity is based upon the yield stress of the liner as reduced by the capacity reduction factor " ϕ " previously described. Sufficient anchorage is provided to ensure elastic stability of the liner. Anchorages are in the form of stagger welded channels on the cylinder and studs on the dome.

Insulation is provided for the side walls to a point 15'-0" above the spring line so as to limit the maximum liner temperature due to the loss-of-coolant accident and thereby avoid excessive compressive stresses in the steel plate.

All weld seams in the liner plate are covered with a test channel to permit testing of leak tightness. Except for the equipment access hatch, as described hereafter, all penetrations provide a double barrier against leakage and can be pressurized to permit testing of leak tightness. The equipment access hatch contains weld seams with no test channels.

The liner plate on the base of the vessel is welded to backup bars. These bars are continuous as shown on Figure 5.1.2-31.

Rock

The containment vessel is founded on rock (sandstone) for which the soils consultant recommended an allowable bearing pressure of 35 tons per square foot. The maximum bearing pressure occurs under the ring girder where the maximum bearing pressure was limited to 30 TSF. This

bearing pressure occurs under operating conditions and is reduced under incident conditions. The soils consultant also recommended a limit on the lateral resistance of the rock of 25,000 psf. The maximum lateral pressure, occurring at the ring girder under the combination of operating and incident loads (Load Combination (a)) is 24,000 psf. A detailed description of subsurface conditions is found in Section 2.8 Geology.

5.1.2.4 Seismic Design Classification

The site seismology is described in Section 2.9. The specific stress criteria for the containment vessel are presented in Section 5.1.2.3.

The classifications of all components, systems and structures of the Ginna Nuclear Station for purposes of seismic design are given below:

Definition of Seismic Design Classes

All equipment and structures are classified as Class I, Class II or Class III as recommended in:

- a) TID-7024, "Nuclear Reactors and Earthquakes," August, 1963, and
- b) G. W. Housner, "Design of Nuclear Power Reactors Against Earthquakes," Proceedings of the Second World Conference on Earthquake Engineering, Vol. I, Japan 1960, pages 133, 134 and 137.

Class I

Those structures and components including instruments and controls whose failure might cause or increase the severity of a loss-of-coolant accident or result in an uncontrolled release of excessive amounts of radioactivity. Also, those structures and components vital to safe shutdown and isolation of the reactor.

Class II

Those structures and components which are important to reactor operation but not essential to safe shutdown and isolation of the reactor and whose failure could not result in the release of substantial amounts of radioactivity.

Class III

Those structures and components which are not related to reactor operation or containment.

All components, systems and structures classified as Class I are designed in accordance with the following criteria:

1. Primary steady state stresses, when combined with the seismic stress resulting from the response to a ground acceleration of 0.08g acting in the vertical and horizontal planes simultaneously, are maintained within the allowable working stress limits accepted as good practice and, where applicable, set forth in the appropriate design standards, e.g., ASME Boiler and Pressure Vessel Code, USAS B31.1 Code for Pressure Piping, ACI 318 Building Code Requirements for Reinforced Concrete, and AISC Specifications for the Design and Erection of Structural Steel for Buildings.
2. Primary steady state stresses when combined with the seismic stress resulting from the response to a ground acceleration of 0.20g acting in the vertical and horizontal planes simultaneously, are limited so that the function of the component, system or structure shall not be impaired as to prevent a safe and orderly shutdown of the plant.

All Class II components are designed on the basis of a static analysis for a ground acceleration of 0.08g acting in the vertical and horizontal directions simultaneously. For this plant, there are no class II structures.

The structural design of all Class III structures meets the requirements of the applicable building code which is the "State Building Construction Code," State of New York, 1961. This code does not reference the Uniform Building Code.

Table 5.1.2-1 gives the damping factors used in the design of components and structures.

The design of Class I structures and components utilizes the "response spectrum" approach in the analysis of the dynamic loads imparted by the earthquake. The analysis is based upon the response spectra shown on Figures 5.1.2-7 and 5.1.2-8.

Seismic accelerations are computed as outlined in the TID-7024⁽²¹⁾ and Portland Cement Publication.⁽²²⁾

The following method of analysis is applied to Class I structures and components, including instrumentation.

1. The natural periods of vibration of the structure or component is determined.
2. The response acceleration of the component to the seismic motion is taken from the response spectrum curve at the appropriate period.
3. Stresses and deflections resulting from the combined influence of normal loads and the seismic load due to the 0.08_g earthquake are calculated and checked against the limits imposed by the design standard.
4. Stresses and deflections resulting from the combined influence of normal loads and the seismic loads due to the 0.2_g earthquake are calculated and checked to verify that deflections do not cause loss of function and that stresses do not produce rupture.

The Class I piping systems are analyzed by a lumped mass approach. The number of masses lumped between any two supports is based upon the spacing interval and increases with the length of the spacing interval. Every

mass is given an acceleration equal to the maximum response from the response curve with .5% of critical damping, i.e. .8_g for .2_g ground acceleration. Each piping system with its supports is modeled as a three dimensional frame and the loads given by the mass times the acceleration are applied at each lumped mass along three directions, two horizontal and one vertical, separately. The moments and torque for each of the three loading directions are then obtained by stiffness analysis. The stresses are calculated at critical points in the piping and its supports for each loading direction. The stresses in the piping are found by using the USAS B31.1 formula

$$S = \frac{M_x^2 + M_y^2 + M_z^2 + T^2}{Z}^{1/2}$$

where:

S = stress

M_x, M_y, M_z = moments about the two horizontal directions and the vertical direction

T = torque

Z = section modulus

At each point the stresses obtained for the two horizontal loadings are conservatively combined by the square root of the sum of the squares. This value is then conservatively combined with the stress obtained for the vertical loading by direct addition. The analyses show that the stresses in the piping and its supports are within the limits specified in Table 1 of Appendix 4A.

The containment vessel is a Class I structure. It was analyzed as a single lumped mass cantilever beam system to determine its natural frequency. For the containment structure the damping factor as a per cent of critical damping is assumed to be 2.0%. The resultant load developed from the maximum horizontal response is distributed in a triangular

manner with the base of the triangle at the top of the structure. The stress criteria for the containment vessel and all reinforced concrete members in tension are as described in Section 5.1.2.3, based on the response to a ground motion on 0.08g acting in the vertical and horizontal planes simultaneously. Design of the containment vessel is checked to ensure that the combined stresses resulting from gravity, incident and seismic loadings based on the response to a ground motion of 0.20g acting in the vertical and horizontal planes simultaneously are within the stress limits described herebefore in Section 5.1.2.3.

The natural period of the first harmonic was determined using an analysis consisting (for horizontal motion) of a cantilever fixed at the base with the mass lumped at the centroid of the structure. Bending stiffness was established based on a Young's modulus of 4.1×10^6 psi and shear stiffness was established based on a shear modulus of 1.8×10^6 psi. No rotation of the foundation material was considered. The natural period of the first harmonic is calculated to be 0.22 seconds for horizontal motion and 0.07 seconds for vertical motion.

The resultant base shear was established on the basis of the maximum response acceleration (0.46g) for the maximum hypothetical design earthquake considering 2% of critical damping. The resultant load was conservatively assumed to be distributed in the form of an inverted triangle extending the full height of the structure.

The resulting maximum meridional forces are as shown in Figure 5.1.2-8A.

As a check on the initial seismic design of the containment it was reanalyzed using normal mode theory with a number of lumped masses. A check was also made on the containment considering the rock foundation as an elastic media with rotation and translation of the containment

considered. This flexible foundation modeling of the containment changed the total shear and overturning moment on the structure by less than 5 percent as compared to the rigid foundation model. The base shear for the modal analysis on a rigid foundation resulted in a equivalent containment response acceleration of .26g as compared to the .46g used in design. Comparable results were obtained with respect to overturning moments. As a result of the somewhat more rigorous modal analysis the containment design can be shown to be highly conservative. A detailed description of the modal analysis follows:

- (a) The containment structure is modelled as a cantilever consisting of lumped masses connected by weightless springs. This model is shown in Figure 5.1.2-8B.
- (b) The normal modes are calculated using computer program SAND. This program is a modified version of a program developed by the Jet Propulsion Laboratory for the dynamic analysis of lumped mass systems. Shear deformations and rotational inertia are included in program SAND.
- (c) The input required for program SAND consists of the modal coordinates, member properties and material properties. These are shown in Figures 5.1.2-8B and 5.1.2-8C. The masses are calculated by the program using a density of 160 lb/cu. ft. This is representative of heavily reinforced concrete.
- (d) The response in each mode is read off the response curves determined for the site as given in the FSAR. The deflections, accelerations and member forces are computed in each mode and are then summed on a root-mean-square basis. This computation is executed by program SPECTA.
- (e) The natural frequencies and response are summarized in Figure 5.1.2-8D. The mode shapes are plotted in Figure 5.1.2-8E and the shear forces and bending moments in Figure 5.1.2-8F.
- (f) The effect of ground motion was investigated by considering the rock as an elastic medium with coefficients similar to concrete:
 $E = 3.0 \times 10^6 \text{ psi}$
 $r = 0.2$

The fundamental frequency was reduced from 6.95 cps to 6.28 cycles/sec. The alterations to the deflections, accelerations, shear forces and moments was insignificant, being less than five percent.

5.1.2.5 Detailed Design

Stress Analysis

The analysis of the containment structure for operating plus incident load is based upon shell theory analogy.

The containment structure is analyzed for seismic load as a cantilevered beam with all mass assumed concentrated at the center of gravity. Both shear as well as bending stiffness are considered in determining the fundamental frequency. The total horizontal inertial load is determined from the response curves given in Figures 5.1.2-7 and 5.1.2-8 for two percent damping for the fundamental frequency of the cantilevered beam. This total horizontal load is distributed over the height of the containment structure in the form of an inverted triangle to determine the inertial overturning moment. The vertical seismic component is assumed unamplified due to the high axial stiffness of the containment.

Stresses induced by the horizontal and vertical components of seismic motion are combined algebraically. The seismic shear distribution assumed is that given for a hollow thin-walled cylinder with shear flow perpendicular to the containment radius and the maximum shear flow equal to twice the average value.

The results of this analysis for the loading combinations of:

1. Operating plus incident load
 2. Operating plus incident plus design earthquake loads, and
 3. Operating plus incident plus maximum potential earthquake
- are depicted in Figures 5.1.2-9 through 5.1.2-11.

The displacement resulting from the seismic excitation will produce a base shear which is transferred via the base mat to the side walls of the structure by the radial reinforcement. During an incident these bars should be in tension. As the lateral load (i.e., earthquake shear) is imposed, these bars will react similar to a wheel with prestressed

spokes with a load applied to the hub and the rim restrained from moving. In this design these members are assumed to have no shear resistance. The load transfer from the radial bars, which established longitudinal shear stresses in the wall, will occur by means of varying circumferential membrane forces in the lower portion of the wall.

The side wall at loads resulting from the factored pressure (1.5P) will be uncracked in a horizontal plane due to membrane prestress forces. The only cracking that occurs will be partial cracking due to secondary flexure. The depth of these cracks will be limited by the use of mild steel reinforcement. At the design pressure there will then be sufficient uncracked section of concrete to limit radial shear stresses to less than the maximum allowable value stipulated in ACI 318-63. Details of the radial shear analysis are provided more fully hereafter.

The amount of prestressing force provided in the meridional direction of the cylinder is determined to ensure no resultant tensile stress due to the factored load combinations described in Section 5.1.2.3. Consequently radial cracking is predicted to be only a result of flexure which is similar to the basis for the derivation of concrete shear capacity and shear reinforcement requirements stipulated in ACI 318-63 for flexural members. The derivation of shear reinforcement requirements at the base to cylinder discontinuity is described in Section 5.1.2.5.

The capacity to resist membrane shears is affected by the concrete cracking. Refer to the Third and Fifth Supplements to the FSAR for the consideration of membrane shears in the vicinity of the large openings.

For the cylindrical portion of the vessel resistance to the vertical shears resulting from the earthquake loading will be developed in the circumferential reinforcement by dowel action⁽⁵⁾. The resulting principal stress in the reinforcement will not exceed $0.95 \times$ yield stress as provided in the design criteria. This design further ensures no failure of the adjacent concrete in bearing. Details of the longitudinal shear analysis are provided more fully hereafter.

In the dome all membrane and shear stresses resulting from the earthquake loading will be developed in mild steel reinforcing.

The loading on the concrete shell of the containment following an accident must be transmitted to it through the liner. The liner attempts to expand under the combined influence of the temperature and pressure. Since the containment structure may be classed as a thin shell, (the diameter to thickness ratio is 30), it is considered that it would have been valid to treat the temperature rise in the liner as an equivalent pressure increase.

Nevertheless the analysis as actually performed considered an equivalent liner force occurring at the location of the liner. Such equivalent liner forces were established based upon no thermal strain relief at points where concrete is uncracked. The liner temperature increase was assumed to be 10°F due to accident conditions where the liner is insulated. Based upon no relief of thermal strains with uncracked concrete this effect of this temperature rise was converted to an axial force plus a moment about the centroid of this section. As a design conservatism, the elastic expansion of the concrete shell under pressure and temperature loads has not been used to reduce the temperature induced stresses.

Rock Anchors

The basic criterion for the determination of anchor length is that the pull of the anchor is resisted only by the submerged weight of rock and that the rock offers no tensile strength. This criterion further assumes that the rock breaks out at an angle of 45° to the bond development length of the tendon. This criterion also allowed for any additional loads on the rock imposed from the inside of the containment vessel. The hold-down capability of the rock in the rock anchor design has taken into consideration the circular geometry of the vessel.

The design of the rock anchors is based upon the simplified assumption that the rock breaks out at an angle of 45° to the axis of the tendon with the apex of the angle at mid-height of the first stage grout. This implies that the rock failure mode is one of diagonal tension. This assumption of a half-angle of 45° for rock is not unique as is evident by the following references:

1. The Raising and Strengthening of Steenbrar Dam, By S. S. Morris and W. S. Garrett, Proceedings, I.C.E., Vol. 1., Pt. 1, No. 1, p. 23; Discussion, Vol. 1, Pt. 1, No. 4, 1956, p. 399.
2. Stress Analysis and Special Problems of Prestressed Dams, O. C. Zienkiewica and R. W. Gerstner, Journal of the Power Division, ASCE, January, 1961.
3. 1300 - Ton Capacity Prestressed Anchors Stabilize Dam, A. Eberhardt and J. A. Veltrop, Journal of the Prestressed Concrete Institute, Vol. 10, No. 4, August, 1965.

Further verification of the conservative nature of this assumption was demonstrated by the rock anchor tests described in Section 5.6.1.1.

The sockets for the rock anchors are percussion drilled into the rock through steel pipe sleeves which are welded into the underside of the bearing plates for the rock anchors and extended through the ring girder. The sockets in the rock plus the pipe sleeves are filled with a neat cement grout in two stages after the rock anchors are installed. Protective steel covers, as shown on Figure 5.1.2-1, are welded to the bearing plates for the rock anchors to enclose the sidewall tendon to rock anchor couplings. The tendon conduit extending above this enclosure is 6 inch diameter schedule 40 pipe with threaded couplings. This tendon conduit is threaded into half coupling welded to the top of the protective steel cover. In order to permit the required conduit movement, stainless steel bellows are provided. The tendon conduit, including the protective steel cover, is bulk filled with the corrosion protection system described in Section 5.1.2.3. This filler material is injected through a connection in the protective steel cover. The exterior surface of the containment structure will be waterproofed from the edge of the ring girder to Elevation 253'-0" to provide corrosion protection.

Prior to installing any rock anchors, a test was performed by grouting a rock anchor in a water filled, clear, six inch diameter tube. This rock anchor contained 90-1/4 inch diameter wires with the grout tube and bottom hardware all identical to the proposed for the permanent installation. This test demonstrated that the grout did flow so as to completely encase the tendon. However, it also indicated that the use of bleeder holes near the bottom of the group pipe, as well as the group pipe terminating above the bottom of the hole, tended to produce an unacceptable dispersion of the grout. This condition was remedied by deleting the bleeder holes and extending the grout pipe with the addition of a bevel to the bottom of the hole. No tests could be made on the completeness of grouting of permanent rock anchors. However, procedures used for grouting did comply with those found to be satisfactory in the previously described test.

The side wall tendons are coupled directly to the rock anchors. When lift-off readings are made on the side wall tendons, this will also provide a measure of the prestress force at the fixed end (i.e. upper anchor head for the rock anchors). However, as in any bonded tendon, it is not possible to measure the prestress in the full rock anchor tendon.

These criteria are identical with those used for dams in the USA and Europe.^(6,7) Confirming information was also obtained from The Cementation Company Limited of Great Britain, a specialty firm whose activity in recent years has been devoted, in large measure, to the prestressing of both existing and new dams, especially in South Africa and Australia.

Large capacity, post-tensioned anchors designed on this basis have previously been used in a number of dams in Europe, Africa, Australia and this country to provide stability for the structures. One of the early applications was the anchoring of the Cheurfas Dam in France 1935. Similarly, prestressed rock anchors have been used for tie backs on retaining walls on a permanent as well as temporary basis and for suspension bridge anchorages. Recent major structures for which prestressed rock anchors were used are listed in Table 5.1.2-2. A list of recent major applications of BBRV ninety - 1/4 inch diameter wire prestressed rock anchor assemblies is given below.



Wanapum Dam, Washington	- Rock anchors and trunnion anchors
Mayfield Dam, Washington	- Rock anchors for penstock slope stabilization
Boundary Dam, California	- Rock anchors for rock stabilization
John Hollis Bankhead Dam, Alabama	- Rock anchors for dam stabilization
Ice Harbor Dam, Washington	- Rock anchors
Mangla Dam, West Pakistan	- Trunnion girder anchorage, main spillway

The design is based upon the use of the BBRV system developed originally in Switzerland and used extensively for rock anchor applications.

Laboratory tests on core representative of rock in the approximate area and depth of the rock anchor installation indicate a bulk specific gravity of the rock of 2.54. Since the rock participating with the rock anchors is below the ground water table the submerged weight of rock of 96 pcf $(2.54 - 1.0) \times 62.45$ is used in determining the hold-down capability.

The bond development length (first stage grout) for the ninety - 1/4 diameter wire tendon is computed as follows:

For $0.60 f_u = 635$ kips

$$P = \frac{80/60 \times 635000}{\pi \times 6 \times 170 \times 12} = 22.0 \text{ ft.}$$

Each rock anchor is initially tensioned to 80% of ultimate strength and the jacking force is then reduced at lock-off to 70% of ultimate. The bond stress assumed between rock and grout is 170 psi. This value was determined to be conservative as demonstrated during the test performed on reduced scale rock anchors as reported here-in and also as reported by the Swiss Federal Laboratory for the Testing of Material (Reference VSL Prestressed Rock and Aluvium Anchors, Losiner & Co. SA dated March 1965) and as documented in Grolversuchemit Spannankern an Talsperran der Asterreichen Bunderbahnen und die Anwendung der Vorspannbouweise auf den Talsperrenban, Von A. Ruttner, Wien, Austrian Engineering Journal 1964. Test data



obtained for the John Hollis Bankhead Dam, Warrior River, Alabama, also confirm the conservatism of a bond development length developed on the basis of the average bond stress of 170 psi between grout and rock.

The diameter of the drilled hole for each rock anchor is 6 inches. The assumed breakout angle of 45° to the vertical is most conservative as demonstrated during the reduced scale rock anchor test, and in Reference 8.

Weight of rock in kips per ft. circumference = $0.096d^2$

Internal Pressure in kips per ft. circumference

=

$$\frac{0.072 \text{ pd } (2r - d)}{r}$$

The depth $d = 26.5$ ft., was established based on preliminary design. No surcharge beyond the internal pressure of the containment vessel was considered to be effective in determining the rock anchors hold-down capability. Therefore, for varying internal pressures the rock hold-down capacity uniform around the circumference of the vessel, is as follows:

<u>Internal Pressure (psig)</u>	<u>Rock Hold-down Capacity (kips per ft. circumference)</u>
0	67.4
60	240.4
69	266.4
75	283.7
90	327.0

For the combination of operating plus incident loads (i.e. Load Combination (a) in Section 5.1.2.3), the uplift per foot circumference is constant at 259.0 kips per ft., less than the assumed rock anchor capacity of 327.0 kips per ft. Therefore, the factor of safety on pull-out against the factored load is 1.26. For the structural proof test, uplift per foot circumference is constant at 182.0 kips per ft., less than the rock anchor capacity of 266.4 kips per ft. for a factor of safety of 1.47.

For the combination of operating plus incident plus design earthquake loads (i.e. Load Combination (b)), the maximum uplift per foot circumference is 274.1 kips per ft. and the minimum is 150.5 kips per ft. This considers horizontal and vertical components of ground motion occurring simultaneously and their effects added algebraically. Due to the group action of anchors, the overcapacity of the rock against lateral loads can be represented by the factor of safety against overturning. This factor, using the rock hold-down capacity based on the pressure load of 75 psig, is 2.38.

For the combination of operating plus incident plus maximum potential earthquake loads (i.e. Load Combination (c)), the maximum uplift per foot circumference is 289.2 kips per ft. and the minimum is 25.4 kips per ft. The factor of safety against overturning again using the same consideration is 1.96.

Consideration was also given for seismic loading without internal pressure. For the 0.1g ground motion (vertical and horizontal components considered to occur simultaneously and the effects added algebraically) there is no uplift. Minimum downward component is 0.9 kips per ft. The factor of safety against overturning is 4.62. For the 0.2g ground motion (vertical and horizontal components considered to occur simultaneously and the effects added algebraically) the maximum uplift is 69.2 kips per ft. The factor of safety against overturning is 2.31.

The tendons are anchored into the rock socket with an expanding grout. The grout contained an additive designed to reduce the water requirement of the cement, have a slightly expanding action and retard the initial set. The expansion based upon original grout volume is $8\% \pm 2\%$. This expansion is accomplished by the reaction of aluminum powder with the alkalies of the cements. This reaction results in liberation of hydrogen gas in the form of small bubbles which have an expanding effect. Tests have verified that the molecular form of the hydrogen in the alkaline medium will not adversely affect the steel.

The top (movable) anchor head for the rock anchor is coupled to the bottom (fixed) anchor head of the side wall tendon as shown in the fully engaged position on the attached Figure 5.1.2-12. Dimensions and material will be as shown thereon. The bushing provides for coupling the smaller diameter fixed head to the larger movable (i.e., tensioning) head). The coupling has right-hand threads on each end.

During construction, after the rock anchors were tensioned, the coupling was set in place on the top head of the rock anchor. When the sidewall tendon was inserted in the conduit, the coupling was threaded onto the bottom head of the sidewall tendon to the end of thread. The coupling was then turned down onto the top head of the rock anchor resulting in all threads on both anchor heads being fully engaged as shown on the sketch. The design of the tendon hardware ensures that the hardware remains elastic up to the ultimate capacity of the wires. Therefore, at the effective prestress force of 60% of the ultimate strength of the tendon, average strains in the coupling are designed to be no greater than 60% of the yield strain of the coupling material. Details of the anchorage hardware are shown on Figures 5.1.2-13 through 5.1.2-18.

Tendons

General Design

The design for the Containment Vessel provides for prestressing the concrete in the cylinder walls in the longitudinal (vertical) direction with a sufficient compressive force to ensure that upon application of the design load combinations there will be no tensile stresses in the concrete due to membrane forces. In addition to the membrane stresses there are also flexural and shear stresses which result from discontinuity effects. On the basis of the design criteria the concrete stresses and the stresses on the mild steel reinforcing upon application of the combined loads will then be produced by combined flexure and shear and/or compression. The structural elements are then acting in a manner similar to those tested as a basis for Chapter 17 - Shear and Diagonal Tension - Ultimate Strength Design of ACI 318-63 and there is a basis for designing shear reinforcing.

The design also provides for anchoring the cylindrical walls to rock with anchors which will be post-tensioned tendons anchored into grouted sockets in the rock. The anchors are designed to resist all membrane stresses in the cylindrical wall. A sufficient physical separation is provided between wall and base slab to ensure the transfer of no vertical reaction to the base slab.

In order to produce minimum practical base restraint and to most effectively use the rock anchors (i.e. no moment applied to the ring girder), the design provides for the development of a hinge at the cylinder to base transition using an elastomer pad. The elastomer pad permits a predictable rotation of the hinge with the only restraint to rotation being a minimal resistance due to compression on the pad. The elastomer (neoprene) pad was selected for the hinge because of its predictability of behavior, maintenance-free properties, and ability to withstand environmental condition far more severe than that associated with this design. Detailed background data on the use of neoprene bearing pads is included hereinafter.



Under the dead load of the vessel and the application of the prestress force, the elastomer pad will compress vertically approximately 0.08 inches. Upon being subjected to the most severe loading combination the tendon elongates and the pad reverts back to essentially its original thickness (i.e. prestress force equals or is slightly greater than membrane forces due to this loading combination). This elongation must extend over a sufficient length of tendon to ensure no yielding of the steel. In an effort to minimize the increase in wire stresses under load, the tendon is unbonded for the entire length from coupling between rock anchor and anchorage of tendon at the top of the side wall.

A large amount of vertical reinforcement is provided near the outer surface of the wall at the lower elevations. This steel is provided to resist bending moments which occur in the wall due to the base restraint. Mild steel reinforcement provided for flexure is shown in Figures 5.1.2-4 and 5.1.2-5. Since the wall has a steel liner on the inside, the minimum mild steel reinforcement required for crack control has been provided on the outside only, in the amount of 0.19% of the concrete cross sectional area. The pre-stressing tendon is positioned at the center of the wall section, thus causing the participation of the prestress force to be minimal in resisting bending moments. The design requires all bending or shear stresses to be resisted by mild steel reinforcement, thus making the design quite conventional in the region of bending and shear.

Due to the initial tendon force ($0.6 f'_s$) the maximum average concrete membrane (meridional) stress is 640 psi compression and the liner (meridional) stress is 4500 psi compression. Considering a concrete creep and shrinkage of 320×10^{-6} in/in., the final average concrete membrane (meridional) stress is 550 psi compression and the liner (meridional) stress is 14,100 psi compression. This implies that a linear temperature gradient of 39°F. through the concrete shell (i.e. a temperature on the liner side 39°F. below the exterior fiber temperature) would result in a zero stress on the inner fiber. This situation is not considered credible. During shutdown the refueling and purge system which has no cooling coils, could not reduce the interior temperature below the external ambient temperature. The fan coolers could possibly reduce the internal ambient temperature but not to the extent required to exceed the foregoing gradient. Therefore a reversal of stresses is not possible and no concern exists regarding crack control

on the inner face. As noted above, a minimum mild steel reinforcement has been provided on the outside face in the amount of 0.19% of the concrete cross sectional area. This amount exceeds the frequently used minimum amount of steel for crack control of 0.15%. It should be further noted that this structure has liner insulation (except for a region of the dome) and will consequently not be subject to rapid temperature changes due to fluctuations in the interior ambient temperature.

The sole purpose of prestress is to balance vertical tensile membrane forces in the wall thus allowing confidence in the use of the provisions of ACI-318 section 1701 and 1702 for shear reinforcement design. Therefore, the prestressing requirements would be those of a tension member rather than of a bending member.

All side wall tendons, can be removed or retensioned. Two tendons are permanently accessible for either operation, while the remainder can be reached by removing concrete at approximately elevation 228 feet (See Figure 5.1.2-2) to obtain access to the coupling enclosure. Any tendon can be uncoupled from the rock anchor for removal by opening a window in the coupling enclosure.

The two permanently accessible tendons are located on the south side of the Containment Vessel, and have the coupling enclosure exposed in the Auxiliary Building Sump (Figure 5.1.2-2). A bolted door on the coupling enclosure permits removal and inspection of the tendons without removing concrete.

It is correct to assert that a failure of an unbonded tendon or tendons in the upper portion of the wall would result in a loss of prestress in the section of the wall subjected to bending and shear; however extensive failures of this type would cause a tensile failure in the wall, thus making a secondary shear failure at the base of little consequence.

Seismic Considerations

In evaluating the relative safety of a tendon for a prestressed concrete structure subject to seismic loads consideration was given to the stresses in the tendon (the ninety 1/4-inch diameter wires) and to the tendon anchorage.

The design for Ginna is based upon a dynamic analysis using a basic ground acceleration of 0.2g. The design further does not consider the ultimate strength and plastic deformation of the structure but considers only an elastic response with damping selected on the basis of such a response.

Other considerations which are generally recommended for seismic design and are incorporated in the design are (1) to provide a symmetrical structure thereby avoiding the torsional effect produced by structure rigidity and (2) include sufficient rattle space between the containment shell and adjacent structures, including the structures within the Containment Vessel, to avoid any possible physical interaction as the structures deflect independently under the seismic load.

By using unbonded tendons high local strains or elongations can be distributed over the length of the tendons. Another problem is the control of cracking in the concrete. In Prestressed Concrete Pressure Vessels for Nuclear Reactors - T. C. Waters and N. T. Barrett state that an adequate amount of bonded reinforcement or the bonding of a portion of the prestressing tendons will ensure that cracking of the concrete is uniformly distributed and that concentrations of large local tensile strains at particular points will be avoided. In the Ginna design where cracking might occur due to flexure produced by discontinuities, bonded mild steel reinforcement is used to control crack spacing and width. Where flexural stresses are minimal, bonded mild steel reinforcement is used to control crack spacing and width. Where flexural stresses are minimal, bonded mild steel reinforcement is also provided to control the spacing and width of cracks thereby serving to increase the ultimate capacity of the structure.



The rupture of one tendon when unbonded normally might on a multi-span structure, endanger the portion of structure adjoining the failed part of the structure by the total loss of prestress force. The design is for a pressure vessel with the prestress force applied to a tension element. Although dynamic loads should not produce wire failures, this design is such that it can accommodate a limited loss of prestress force without jeopardizing the integrity of the vessel.

The design criteria provide for a capacity reduction factor of .95 which allows for the possibility that small adverse variations in material strengths, workmanship, dimensions, control and degree of supervision, while individually within required tolerances and the limits of good practice, occasionally may combine to result in undercapacity. This factor is reasonably consistent with the factor of 0.90 for flexural members so stipulated in ACI, 318-63 taking into consideration the greater concern for concrete strength variations and for positioning a tendon in a flexural member. Normal practice as exemplified in ACI 301-66 relative to broken wires is that "The total loss of prestress due to unreplaced broken tendons in a Containment are that no more than .5% of the total number of wires will break during tensioning. Records from previous applications of the BBRV System indicate that only about 0.03 per cent of the wires will break during the tensioning operation. Even should broken wires equal 0.5 per cent of the total there remains a sufficient margin in the capacity reduction factor of 0.95 to ensure that sufficient prestress remains to develop resistance to design static and dynamic loads. To date there has been no breakage of wires during tensioning of tendons for the vessel. At the time of this report this is based upon the experience with rock anchors only. Considering the above capacity reduction factor, it is possible to have a symmetrical failure of up to 5 percent of tendons individual wires and meet the design criteria for the factored loads.

A study was performed to determine the effect of the total loss of three adjacent 90 wire tendons. This study indicates that the loss of three adjacent tendons will not jeopardize the capability of the containment structure to withstand the design accident loading condition.

To our knowledge there is no record of the failure of production BBRV anchorage components due to low temperature brittle fracture. There have been no temperature limitations on tensioning tendons in the past. As stated in Section 5.1.1.1 of the FSAR we consider that "The containment is not susceptible to a low temperature brittle fracture". This conclusion is consistent with information provided in the First Supplement to the PSAR.

For a flexural member there may be merit in localizing a wire failure in that the loss of prestress force might not extend over a region where maximum flexural capacity is required. This would be especially true for a failure at or near an anchorage. However, the design provides for prestressing tension, not flexural, members and there is no similar advantage in localizing the failure of a tendon in a tension member.

The behavior of the anchorage hardware is of prime importance when the element is subjected to reversal of loading produced by the dynamic loads from an earthquake. The anchorage system for this design, the BBRV (buttonhead) system, was especially chosen because of its positive anchorage and excellent properties when subjected to cyclic loadings. Drawings of the anchorage hardware are included in Figures 5.1.2-13 through 5.1.2-18. The BBRV system used parallel wires with cold formed buttonheads at the ends which bear upon a perforated steel anchor head thus providing a positive mechanical means for transferring the prestress force. The buttonheads on the wire are formed by cold upsetting to a nominal diameter of 3/8 inch on the 1/4 inch diameter wire. Professor Fritz Leonhardt in Prestressed Concrete Design and Construction reports that "Extensive tests show that this BBRV 'buttonhead' provides a reliable anchorage, even under dynamic loading conditions, if an anchor of softer steel (ST 52 to ST 90), provided with an appropriate bore (opening for wire) is employed." The anchor heads for the design are fabricated from C1141 steel which is a softer steel than the wire heads approximately equivalent to ST 70 covered under the German Specification DIN 17 100.

Fatigue tests were conducted by the Swiss Federal Testing Station (EMPA) in 1960 on individual 7mm. wires with upset heads and on tendons consisting of 18-7 mm. wires each. The anchorage heads for the tendons were for 22-wire units but the number of wires was limited by the capacity of the testing apparatus. The tests on individual wires indicate that 7mm. wire

with upset heads is capable of sustaining two million stress application cycles with an upper limit of about 1301 kg/sq mm (180 ksi) when the lower limit is 95 kg/sq. mm (135 ksi). Several tests were conducted on the 18-wire tendons. The results of the one test with stress limits most similar to that used for design of prestressed concrete are summarized as follows:

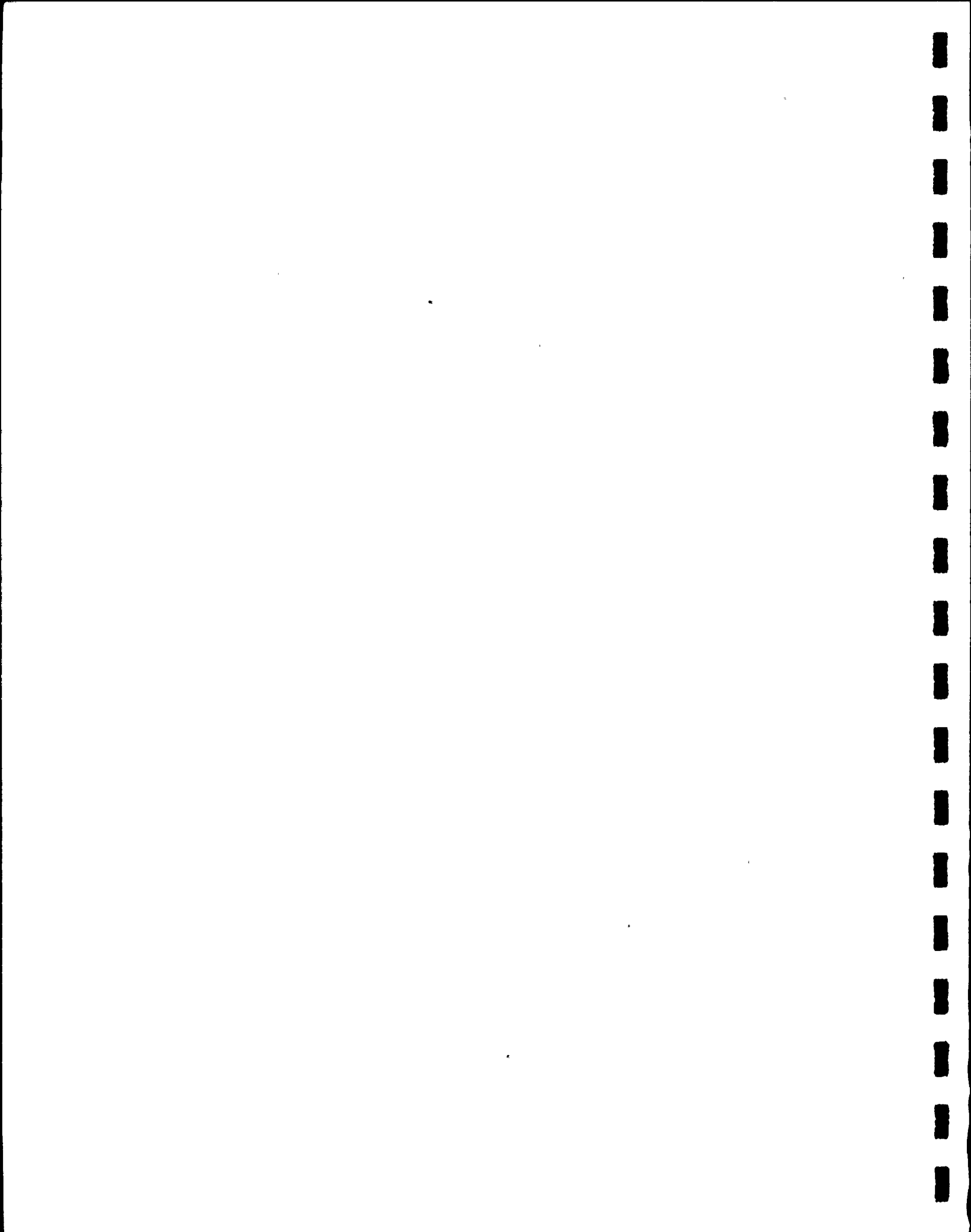
With a lower limit of 95 kg/sq mm (135 ksi), the tendon withstood over 2,040,000 stress application cycles to an upper limit of 111 kg/sq. mm (158 ksi) without any of the wires fracturing. Only after the upper limit was raised to 113 kg/sq. mm (160 ksi) did one of the wires break after an additional 113,000 stress application cycles. The rate of stress applications was 350 cycles per minute.

Cutting tolerance for the test tendon was plus or minus 0.5 mm. The ratio of tolerance to total wire length for the test tendon is 1/2377 which compares with 1/3210 for the rock anchors and 1/4800 for the side wall tendons. The ultimate strength of the wire being tested was 160 kg/sq. mm (225 ksi).

Therefore it can be concluded that dynamic loads, considering especially pulsating loads resulting from an earthquake, do not jeopardize the button-head anchorage.

The tendon bearing plates are 18 1/2 inch in diameter with a 5 1/2 inch center hole. Considering uniform bearing the concrete bearing pressure due to the initial tendon force (742^{kips}) is 3040 psi. This compares with an allowable stress (ACI 318-63 Equation 26-1) of 3720 psi. The maximum splitting force⁽¹³⁾ due to the initial tendon force considering no concrete tension is 58.0^{kips} based upon tension extending from 6 inches to 30 inches below the bearing plate. The required reinforcing is 1.45 square inches per foot compared with the furnished 5/8 inch diameter spiral at 2 inch pitch with an area of 1.86 square inches per foot. The calculated spalling force⁽¹⁶⁾ is 22.2^{kips/tendon} for which #7 reinforcing bars were provided at 12 3/4 inch centers.

Bond development of the spiral reinforcement is not considered relevant. The reinforcement for spalling is anchored in excess of ACI 318-63 requirements. Experience indicates that long term loadings will not degrade the integrity of the anchorage zone.



A Seismic Committee was established by the Prestressed Concrete Institute to develop guidelines for the design of prestressed concrete structures for seismic loads. This report,⁽⁹⁾ PCI provides detailed guidelines for the design of prestressed concrete structures for seismic loads. These guidelines apply to bonded and unbonded tendons. The Ginna design has been reviewed in light of this report and has been found to comply with all guidelines.

Stressing Procedure

Stressing of tendons is accomplished by hydraulic jacks and pumping units which are equipped with dial gages not less than 6 inches in diameter which indicates the pressure in the system within plus or minus two per cent. The stressing procedure is as follows:

- a. Stress by pumping until the required overstressing force is reached with backup provided by direct measurement of differential displacement of tendon head and bearing plate made to the nearest 1/32 of an inch.
- b. Insert shims, filling the space as completely as possible.
- c. Reduce pressure to seat the anchor head on the shims.
- d. Take "lift-off" reading and record.
- e. Adjust shims as necessary

The pattern and sequence of post tensioning was established by the designer so as to provide basically for initially tensioning every 40th tendon of the total 160 tendons and then in a systematic manner to tension the tendons approximately midway between previously tensioned tendons. This approach minimizes the loss due to elastic shortening. The anticipated elongation of the side wall tendons during the stressing operation is approximately 8 inches.



The philosophy behind this sequence of post tensioning is:

1. To provide in each stage of stressing an essential symmetric loading on the Containment Vessel cylindrical wall and neoprene pad at the base.
2. The prestress load will be applied as far as practical symmetrically with respect to the two large access openings.
3. The curved tendons around the large access openings will be retensioned after 1000 hours in order to counteract the time dependent losses due to shrinkage, creep and steel relaxation. The retensioning is required in order to fulfill minimum prestress requirements up to the "end" of plant life which is 40 years.

A. Instrumentation and Control Racks in Control and Relay Room

All instruments and control racks have been attached to the floor to withstand earthquake loading as described in the FSAR Section 5.1.2.4. Supports for components vital to safe shutdown of the reactor are defined as Class I, and are designed accordingly.

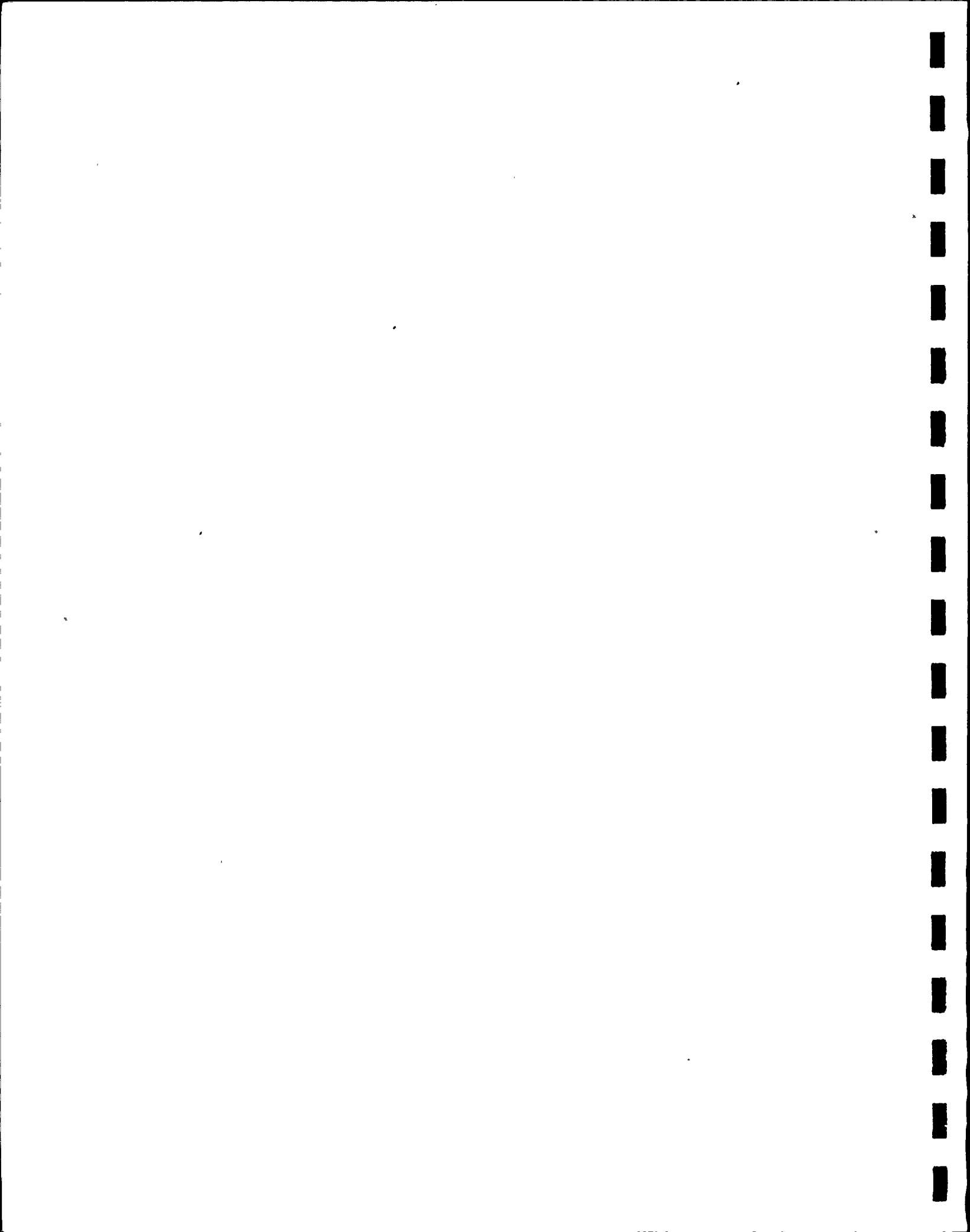
B. Seismic Design - Class I Structures

For Class I structures, modal parameters were determined considering each structure as a lumped mass system. The natural periods of vibration were determined and the response spectrum curve at each appropriate period.

The stresses and deflections resulting from the combined influence of normal loads and seismic load due to the 0.08g earthquake were calculated and checked against the limits imposed by the design as

described in Section 5.1.2.3 of the FSAR. In addition, the design of the Class I containment structure was checked to ensure that the combined stresses resulting from gravity, incident and seismic loadings based on the response to a ground motion of 0.20g acting in the horizontal and two-thirds of this value in the vertical planes simultaneously are within the stress limits described in Section 5.1.2.3 of the FSAR and that deflections do not cause loss of function.

The highest tendon stresses occur during the jacking operation which, in effect, pretests the tendon including all hardware prior to the application of a pressure load. The effective prestress considering all losses (i.e., 60% of ultimate stress) is 144,000 psi. Upon subjecting a tendon to the most severe loading combination which is design accident plus maximum earthquake, the tendon stress increases by 4.6% i.e., 6,600 psi.



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 which was 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. As shown hereafter tendons are protected to ensure no loss of wires due to corrosion. The most effective means of allaying concern about loss of prestress force due to corrosion is not by working to lower unit stresses but to develop a sufficiently conservative method for protecting each tendon.

The tendon temperature never significantly exceeds that resulting from plant operation and high ambient temperatures external to the vessel. The average daily temperature of the tendon will, therefore, never exceed approximately 90°F.

The prestressing sequence for the rock anchors is generally as follows:

- a) . Initially, every fourth anchor is tensioned. Horizontal spacing of anchors as shown on Figure 5.1.2-2 is 2' - 1 9/16".
- b) Secondly, every second tendon not included in (a) is tensioned.
- c) Finally, all remaining anchors are tensioned.

The tensioning of side wall tendons is done using a minimum of four jacks spaced generally about the circumference of the structure. Stressing positions are alternated to prevent concentrations of multiple stressed tendons adjacent to multiple unstressed tendons. This is accomplished

by tensioning tendons in a sequence wherein the tensioned tendon is approximately equidistant between previously tensioned tendons. The four jacks are used so that the resultant of the prestress force remains approximately symmetrical around the circumference of the structure.

Corrosion Protection

A steel conduit (six inch diameter Schedule 40 pipe) is embedded in the side wall concrete to permit insertion of the prestressing steel tendon and in addition provide electrical shielding against stray ground currents. The conduit is specially designed where it passes through the elastomer pads so as not to jeopardize the action of the hinge by using a bellows-type expansion joint.

The 6" Ø threaded pipe is screwed on to a 6" Ø half coupling. This connection meets the criteria specified in the standard code for Power Piping, USAS B31.1.0 - 1967, and as such provides a leak-proof joint.

The wire is protected prior to fabrication to ensure that the surface is free from any imperfections other than a light oxide film. The tendon prior to shipment is protected with a coating of NO-OX-ID "490" as manufactured by the Dearborn Chemical Division of W. R. Grace & Company. The NO-OX-ID "490" provides a light coating satisfactory for temporary protection. Following insertion of the tendons in the conduit, the conduit is filled with NO-OX-ID "CM" so as to provide bulk filling of the void in the conduit. An expansion reservoir is provided at the top anchorage as shown on Figure 5.1.2-6. Access to this reservoir is provided as shown on Figure 5.1.2-19. The tendon conduit is filled by pumping the NO-OX-ID "CM" in at the level of the tendon coupling and venting from the top anchorage.

The water table is approximately 16 feet above the bottom of the tendons. The tendon and its conduit are approximately 110 feet high. This leaves a hydraulic head of filler material of 94 feet which is equivalent to about 36 psi above the highest point of the water table and would thus ensure no water seepage into the conduit.

This is an under-estimation of the pressure required to displace the filler material in that it is based upon the material having the viscosity of water with no friction loss and a specific gravity of 0.9. During the actual placement of the filler material, a pressure of 42 to 45 psi was required to pump the material after it was agitated.

The radial tension bars, as shown on Figure 5.1.2-2, are protected against corrosion as follows:

- a) In the cylinder wall the bars are coated with grease. (The grease insures there is no bond development).
- b) In the base slab, the bars are inserted in a pipe sleeve for a length of 2' - 10". The annular space between bar and pipe sleeve is filled with the corrosion protection system described in Section 5.1.2.3 for the side wall tendons.

- c) The remaining length of the bars in the base slab are in intimate contact with the concrete.

The button-heads at the rock anchor heads are encased in grout to provide continuity of environment along the full length of the wire. The moveable (top) anchor heads for the side wall tendons are protected by covering the head with the NO-OX-ID "CM" made to prevent rain water from entering the conduit by the expansion reservoir. The top anchor heads can be inspected for corrosion by unbolting the cover on the expansion reservoir shown on Figure 5.1.2-6 and removing the wax covering the heads. The wax can also be sampled by this method.

NO-OX-ID "CM" Casing Filler is composed essentially of a selected paraffin-base refined mineral oil, blended with a microcrystalline petroleum-derived base (petrolatum) of definite melting point and penetration range. Additives consisting of lanolin, and sodium petroleum sulphonates are incorporated as water-displacing surface-active agents and corrosion inhibitors. The proportion of oil to microcrystalline wax in the formulation is adjusted to give a pour or gelling point within the range of 110 to 120°F. The oil and wax are highly refined long-chain saturated paraffinic petroleum derivatives, resistant to oxidation and chemical or physical degradation, within the temperature ranges to which they will be exposed in this service. The lanolin is a polar substance which enhances inhibitor performance and wetting of the metal surface by the microwax blend. The petroleum sulphonate is a surface-active, water displacing corrosion inhibitor of long tested merit. See Table 5.1.2-3.

Quality Control Tests

Quality control determinations on required raw materials and on the finished NO-OX-ID "CM" protective coating include those tests already being done in the standard raw material inspection procedures, plus additional controls requested on chloride, sulfide and nitrate content. The latter tests include the following:

Chlorides

The initial screening test on both raw materials and finished product is the sensitive Beilstein Test. This is a flame determination using an oxidized copper carrier. A green or blue-green color appears in the flame if chlorides (halides) are present. This test detects as little as 1/2 ppm halide. If a positive Beilstein indication is obtained, a confirming test is made on water extracts of the product, using standard titration or colorimetric procedures, described in ASTM D-512-62T. (Note: A positive Beilstein test may be obtained when halides are not present, because of interferences from traces of pyridines, thiourea, thiocyanate, etc. This is the reason for a confirming titration on water extracts, following a positive Beilstein indication).

Sulfides

The method used is a water extraction followed by a total sulfide determination. To the extraction water is added zinc acetate to precipitate sulfides. Sulfides present are then measured in accordance with Paragraph 8. of ASTM D-1255. This method detects as little as 0.1 ppm sulfide. An alternate colorimetric procedure also is available in which sulfides are volatilized from an acidified extraction solution, to create a colored spot on zinc acetate paper. Spot intensity is measured to determine sulfide. The extraction procedure is described in ASTM D-1255.

Nitrates

The method proposed is a water extraction followed by colorimetric measurement, based on ASTM D-992-52. Either the Brucine or phenoldisulfonic acid procedures are used. Either will detect as little as 0.01 mg./liter nitrate.

Cathodic Protection

All of the tendons are connected to the liner of the Containment Vessel and then to the copper grounding system. Also electrically connected to the grounding system is the mild steel reinforcement below the high ground water level. Permanent and stable potential reference cells are installed at significant locations to measure the corrosion potential.

At the time of containment vessel construction, Durichlor anodes were installed around the perimeter of the vessel. Protective current can be applied from these anodes and regulated as needed to maintain a protective potential if cathodic protection is found necessary by measurements from the reference cells.

In addition, sacrificial steel cable has been installed along side all bare copper cables. Also four potential bridge pipe test stations were installed on the rock anchor potentials, the magnitude of the earth potential current gradient caused by current flow into or out of the rock anchors and to provide a basis for regulating any applied current from the anodes.

Attention has been given to certain European knowledge on the use of grease protection for prestressing wires. Included in this investigation were problems of stress corrosion which have occurred in Europe, mainly in France due to the use of oil tempered, high tensile wires. This material is susceptible to hydrogen cracking and failures have occurred due to this phenomena. A British study of the problem indicates that cold drawn perlitic wire of the type to be employed in the Ginna vessel is not susceptible to this type of failure.⁽¹⁰⁾

Absence of Backfill Around Containmentment

The Ginna design provides for no backfill against the containment wall. The excavation around the major portion of the vessel is graded to insure slope stability of the in-place material under all conditions, and at a limited portion of circumference where grade level is maintained adjacent to the vessel, there exists a retaining wall spaced 2'-6' clear of the vessel wall designed specifically to resist all earth pressure due to backfill. No provision is made to prevent ground water from penetrating the void created between the retaining wall or earth and the vessel wall. Provision is made to cover the opening between the retaining wall and the vessel wall with a concrete slab which will ensure that the void is not filled with debris. Where the exterior walls of the Containment Vessel are exposed to ground water, the walls from the edge of the ring girder up to Elevation 235'-0" are waterproofed with a bitumastic membrane system reinforced with glass fibers. In addition prior to the application of the membrane courses the angle at the intersection of wall and ring girder is further reinforced with glass fabric.

Hinge Design

Tension Bars

A hinge is developed at the base of the cylinder wall by supporting the wall vertically on a series of elastomer bearing pads and anchoring the wall horizontally into the base mat with radially positioned, high strength steel bars. The bars are 1-3/8" in diameter with a minimum ultimate tensile strength of 145,000 psi and a yield strength of 130,000 psi and are spaced approximately 1 foot 1 inch on centers at the centerline of sidewall. The bars are unbonded over a pre-determined length to provide for an elongation of the bar under load consistent with that required for the rotation of the wall with the elastomer pad acting as a hinge. The only rotational restraint on the base of the wall is that produced by the resistance of the elastomer pads to deformation. Actual tension bar stresses resulting from the factored loads follow:



<u>Loading Combination</u>	<u>Bar Force, kips</u>	<u>Bar stress, ksi</u>	<u>% Yield Strength</u>
a	130	87.5	67
b	149	100.0	77
c	170	114.3	88

A computer program is not used for this analysis. The effect of the base to cylinder discontinuity is based upon equations developed for the analogy of a semi-infinite beam on an elastic foundation^(1,2), in which the spring constant for the circumferential bars and liner is taken as the foundation modulus. As such, the hoop stiffness is generated independently of the concrete. The elastic modulus of the uncracked concrete is assumed to be equal to 4.1×10^6 psi ($E = \omega^{1.5} 33\sqrt{f'_c}$ from ACI 318-63). The assumption on a single elastic modulus which is considered to be a reasonable upper limit is conservative in that it results in the highest discontinuity stresses.

Except for its participation in anchoring the radial tension bars at the base of the cylinder, the base slab is not an integral part of the containment shell for this design. The loads on this slab, which is more properly described as a cap on the rock, are those from the interior structures.

A simple check is made, based on an assumed 45° bearing distribution, to ensure that rock bearing pressures does not exceed the limits listed in Appendix 2B.

The means for transferring the radial reaction at the base of the cylinder into the foundation rock is shown on Section 1-1 of Figure 5.1.2-2. The base reaction is transferred from the radial dowels into the ring girder and thickened portion of the base slab and thence, as a lateral load, on the rock outboard of the ring girder. The concrete for the ring is placed

1. S. Timoshenko, Strength of Materials - Part II, Third Edition, D. Van Nostrand Company Inc. 1956.
2. M. Hetenyi, Beams on Elastic Foundations, The University of Michigan Press, 1961.

directly against the rock. Thus the load is transferred to the rock on the interface from elevation 231'-8" to elevation 224'-8". The maximum allowed lateral pressure is 25,000 pounds per square foot as stipulated in Appendix 2B. Where no lateral rock support is available at the auxiliary building sump, a special beam and struts are required to span this area as shown in Sections 2-2 and 3-3 of Figure 5.1.2-2.

The details of the expansion joint in the tendon conduit at the hinge are shown on Figure 5.2.1-32. This is a stainless steel bellows as conventionally used on process piping for expansion joints.

Liner Knuckle

The liner design at the hinge provides for a base to cylinder transition in the form of a knuckle with a 10 inch radius. This detail provides sufficient flexibility as the sidewall moves with respect to the base during the tensioning of the sidewall tendons and under the application of the design loads.

The stresses in the liner base to sidewall transition knuckle have been determined for the following cases. The analysis was based on the method described in reference 11.

1. Under the application of the prestress force plus the dead weight of the vessel, the sidewall moves vertically downward with respect to the base 0.08". The maximum bending stress in the knuckle due to this motion is 25 ksi.
2. Under loading combination (a), the sidewall moves vertically upward 0.08" with respect to the base, and radially outward 0.08". The maximum bending stress in the knuckle following the movement is 10 ksi and membrane stress is 1.2 ksi. This loading combination represents the most severe loading on the knuckle.



The calculated stresses for the tension bars at the base of the cylinder listed in Section 5.1.2.5 of the FSAR were based upon the assumption that the stiffness of the base is a function only of the tension bars. A study was made to validate this assumption. It was found that the liner knuckle offers negligible restraint of radial motions but does offer very significant restraint of lateral (horizontal earthquake) motions.

The dimensions of the liner knuckle are shown on Figure 5.1.2-19A. The method of solution involved the use of a shell computer program based on the solution described in Reference 23 wherein stresses were determined on the basis of a lateral translation of Point A (Refer to Figure 5.1.2-19A). It was conservatively assumed that the support lines for the knuckle remain circular. For the lateral motion the calculated spring constant of the knuckle is 785,000 k/in. Based upon a maximum earthquake shear force at the base of the cylinder of 12,080 k it is determined that the maximum shear stress in the knuckle is 16.4 ksi. Bending stresses are small.

Elastomer Bearing Pads

Each bearing pad is a flat pad 1.628 inches thick, made of two layers of 55 durometer hardness neoprene between three steel shims. The outer shims are 16 gauge and the middle shim is 10 gauge carbon steel.

The pads are placed between the cylinder walls and the ring beam. Because of the ability of neoprene to deform, it will provide an effective medium of load transfer. By conforming to surface irregularities uniform bearing will be provided. No lubrication or cleaning will be necessary for the bearing. The pad dimensions will be 9" x 42" and two pads will be placed between each pair of prestressing tendons.

Each pair of pads will carry a maximum load of 371 tons resulting in a bearing pressure of 980 psi. This pressure will be reduced to 840 psi after prestress losses occur. Both pressures are well within allowable values. A pad under load should not exceed a vertical deflection greater than 15% of the thickness. The steel shims being used reduce the calculated strain to 5.2 as further verified by the tests reported in Section 5.6.1.1. The creep of neoprene pads is dependent on the hardness of the neoprene which was the reason for using low hardness (55 durometer) pads. Creep as verified by tests is estimated to be 13% of initial deflection.

On most of the circumference of the Containment Vessel the elastomer pads are accessible or could be made accessible by removing insulation to view from one side.

Specifications for the elastomer pads are summarized in Section 5.1.2.6.

Neoprene pads have been in use since 1932 so that current practice is based on over 30 years of experience and research. These pads were first used in France in the late 1940's as the load transfer bearings between piers and beams. In the United States and Canada development has more or less paralleled the use of pre-cast, prestressed concrete beams because of the problem of seating such beams. By 1957 concrete bridges had been built with neoprene bearings in Texas, New Hampshire, Rhode Island and Ontario. At the present time thousands of bridges and buildings throughout the world have been build using neoprene bearing pads.

The neoprene pads will have a local effect on seismic shears at the base. This effect however is comparable to Saint-Venant effects which are present locally at any discontinuity. The seismic design of containment for shear and moment loads as a cantilever beam is not affected by the neoprene pads since the cylindrical shell is tied to the base by means of the vertical prestressing.

The effect of vertical cracking of the containment shell under pressure loading will tend to reduce the stiffness of the containment which in turn for the modal analysis discussed in Section 5.1.2.4 will increase the period and response of the structure. However this same cracking will tend also to increase structural damping and thereby reduce the structural response. Considering the large design margin contained in the actual seismic design of the containment as compared to that dictated by the more rigorous modal analysis presented in Section 5.1.2.4, the local perturbations caused by use of neoprene pads are not sufficient to affect design adequacy.

The seismic design of large openings is described in the Third Supplement to FSAR.

A typical properties specification for bridge bearing pads (the hardness Shore A 50 approximately applying to the pads to be used for the Containment Vessel) is given by the American Association of State Highway Officials as follows:

Original Physical Properties

Hardness Shore A	50 \pm 5	60 \pm 5	70 \pm 5
Tensile minimum psi	2500	2500	2500
Elongation at break minimum % (ASTM D-412)	400	350	300
Ozone, 1 ppm in air by volume, 20% strain 100 \pm 2°F 100 hr.	No cracks	No cracks	No cracks
Compression set 22 hr. at 158°F maximum %	25	25	25

Oven Aged 70 hr. @ 212°F

Hardness pts. change maximum	0 to \pm 15	0 to \pm 15	0 to \pm 15
Tensile % change maximum	\pm 15	\pm 15	\pm 15
Elongation % change maximum	\pm 40	\pm 40	\pm 40

Low Temperature Stiffness at -40°F

Young Modulus maximum psi	10,000	10,000	10,000
Tear. Die C lbs/ in minimum	225	225	225

Among the most notable bridges built with neoprene bearings are: The Pensacola Bay Bridge, Florida; all the bridges of the Van Wyck Expressway built to serve the New York World's Fair; more than 300 bridges on the Autostrada System in Italy, the two-mile Champlain Bridge across the St. Lawrence River at Montreal; the Viaduc, an elevated expressway in Brussels, Belgium; the 17-mile long bridge-tunnel system crossing the mouth of the Chesapeake Bay. The Connecticut River Bridge at East Haddam, Connecticut, was repaired with neoprene bearings replacing worn out roller bearings under the two fixed spans, one of them 326'



long. Some prominent buildings built with neoprene pads are: The Marine Plaza in Milwaukee, Wisconsin; the Connecticut Milk Producers Association Building in Newington, Connecticut; Le Centre International Rogier in Brussels, Belgium and Spain's new bullring in Jain near Granada. In the Marine Plaza building the pads were used to effect load transfer; in the Milk Producers building the pads effected load transfer and allowed for the isolation of expansion and contraction between 100 ft. prestressed roof T's and concrete columns. Le Centre International isolated thermal movement in the 26-story apartment tower and 8-story garage and theater wing by using the pads; forty-eight neoprene pads support the roof of the Jain bullring.

In all of the foregoing examples of both bridges and buildings, neoprene pads were selected firstly because of their long lasting maintenance-free qualities in the face of the corrosive effects of oil, grease, dirt, ozone and other substances, all of which have been corroborated by laboratory tests; secondly, because neoprene is elastic, resilient and does not lose its shape even under severe pressure and is able to give and take with thermal movements, rotations and horizontal forces without any moving parts, thus forming an excellent load transfer material.

A simulated service test was employed to collect data on the physical reactions of a rubber component under projected end use conditions so that the entire assembly could be properly designed and engineered. The assemblies involved were highway bridges and the rubber components were elastomeric bearing pads for bridge beams. The project was undertaken by Charles A. Maguire & Associates Engineers, Providence and Boston, in cooperation with the State of Rhode Island Department of Public Works, Division of Roads and Bridges. Maguire & Associates is a Consultant engineer firm, well known in the construction field for the past 25 years for its progressive standards.



The test apparatus, following common engineering practice, was a mock-up of a typical bridge beam-pier interface. It consisted, essentially, of a 500,000 psi Southwark-Emery tension compression machine, a 40,000 psi hydraulic jack, and various recording gauges. Test samples of neoprene - 6 inches by 12 inches and of varying thickness - molded or in bonded layers, and furnished by five firms, were used as bearing pads.

Compressive loads, simulating actual beam weight plus traffic load, were applied by the Southwark-Emery machine while shear loads, simulating expansion and contraction effects, were applied by the hydraulic jack. As a rule two bearing pads were tested simultaneously, a steel or concrete shear plate being placed between them and this assembly set between two concrete (beam and pier) compression members. The jack pressed horizontally against the shear plate to provide the shear loads.

The required compressive and shear loads on the neoprene bearing pads were applied directly, alternately, and in combination for varying periods. Test phases included:

1. Vertical deflection under compression
2. Horizontal and vertical deflection in combined compression and shear
3. Shape effects
4. Time and creep relationships
5. Fatigue
6. Hardness factor comparisons

It was found that all neoprene bearing types tested bore close stress-strain relationships. Within the limits of the test procedures, it was assumed that neoprene-based bearing products meeting current specifications have similar physical properties. Based on the results



of the Maguire & Associates testing adequate design criteria were established for use of neoprene bearing in bridge construction. Details were reported in a brochure prepared by Charles A. Maguire & Associates in March 1958.

An example of the resistance of neoprene to corrosion and chemical change is its use in jacketed cables which show no signs of cracking or crazing despite continuous exposure to the weather; moreover, stressed samples of neoprene jacketed cable exposed over 30 years have retained more than 80% of their original properties. The Bell Telephone Co. has installed neoprene in about 8,000,000,000 feet of covered line wire and service drop wire in the last 20 years, all of which are still tough and flexible; it is also used in machinery mounts, auto parts, seals, gaskets, conveyor and power belts, all giving dependable service after 20 to 25 years service.

Concrete

Radial Shear

The maximum value of radial shear is 253 psi and this occurs 3 ft. above the higheststressed radial tension bar under the combination of operating incident and maximum credible earthquake loads (Load Combination (c)). The critical section for shear is taken 3 ft. above the radial tension bar level to conform with the requirements of A.C.I. 318, Section 1701. The ultimate shear capacity of the reinforced wall without shear reinforcement as defined in A.C.I. 318 1701 is 126 psi. Shear reinforcement is required and is provided according to the requirements of Section 1702 as No. 7 bars at 11" centers. Thus, under the conditions of 60 psi internal pressure and 0.2g simultaneous earthquake (load combination c), the shear capacity of the vessel wall is sufficient to resist the maximum shear stress which occurs at only one position on the circumference.



EXTRACTS GINNA
TECHNICAL SPECIFICATION



Basis:

The reactor coolant system conditions of cold shutdown assure that no steam will be formed and hence there would be no pressure buildup in the containment if the reactor coolant system ruptures.

The shutdown margins are selected based on the type of activities that are being carried out. The (2000 ppm) boron concentration provides shutdown margin which precludes criticality under any circumstances. When the reactor head is not to be removed, a cold shutdown margin of $1\% \Delta k/k$ precludes criticality in any occurrence.

Regarding internal pressure limitations, the containment design pressure of 60 psig would not be exceeded if the internal pressure before a major loss-of-coolant accident were as much as 6 psig.⁽¹⁾ The containment is designed to withstand an internal vacuum of 2.5 psig.⁽²⁾ The 2.0 psig vacuum is specified as an operating limit to avoid any difficulties with motor cooling.

References:

- (1) FSAR - Section 14.3.5
- (2) FSAR - Section 5.5

- c. Visual inspection shall be made for excessive leakage from components of the system. Any significant leakage shall be measured by collection and weighing or by an equivalent method.

4.4.3.2 Acceptance Criterion

The maximum allowable leakage from the recirculation heat removal systems components (which includes valve stems, flanges and pump seals) shall not exceed two gallons per hour.

4.4.3.3 Correction Action

- a. Repairs shall be made as required to maintain leakage within the acceptance criterion of 4.4.3.2.
- b. If repairs are not completed within 24 hours, the reactor shall be shut down and depressurized until repairs are effected and the acceptance criterion of 4.4.3.2 is satisfied.

4.4.3.4 Test Frequency

Tests of the recirculation heat removal system shall be conducted at intervals not to exceed 12 months.

4.4.4 Tendon Stress Surveillance

4.4.4.1 Inspection for Broken Wires

- a. Fourteen specific tendons, equally spaced around the



containment shall be inspected periodically for the presence of broken wires.

- b. The inspection intervals, measured from the date of the initial structural test, shall be as follows:

6 months

1 year

3 years

8 years and 5 years intervals thereafter.

- c. The acceptance criteria for the inspection are that no more than a total of 38 wires (in 14 tendons) are broken and that not more than 6 broken wires exist in any one tendon. If more than 38 broken wires are found, all tendons shall be inspected. If inspection reveals more than 5% of the total wires broken, the reactor shall be shut down and depressurized.
- d. If more than 20 wires (in 14 tendons) have been broken since the last inspection, all tendons shall be inspected. If inspection reveals more than 5% of the total wires broken, the reactor shall be shut down and depressurized.
- e. If as many as 6 broken wires are found in any one tendon, four immediately adjacent tendons (two on each side of



the tendon containing 6 broken wires) shall be inspected. The acceptance criterion then shall be no more than 4 broken wires in any of the additional 4 tendons. If this criterion is not satisfied, all of the tendons shall be inspected and if more than 5% of the total wires are broken, the reactor shall be shut down and depressurized.

4.4.4.2 Pre-Stress Confirmation Test

- a. Lift-off tests shall be performed on the 14 tendons identified in 4.4.4.1a above, at the intervals specified in 4.4.4.1b. If the average stress in the 14 tendons checked is less than 144,000 psi (60% of ultimate stress), all tendons shall be checked for stress and retensioned, if necessary, to a stress of 144,000 psi.
- b. Before reseating a tendon, additional stress (6%) shall be imposed to verify the ability of the tendon to sustain the added stress applied during accident conditions.

Basis

(1)

The containment is designed for an accident pressure of 60 psig. While the reactor is operating, the internal environment of the containment will be air at approximately atmospheric pressure and a maximum temperature of about 120°F. With these initial conditions, the temperature of the steam-air mixture at the peak accident pressure of 60 psig is calculated to be 286°F.

Prior to initial operation, the containment was strength tested at 69 psig and then was leak tested. The acceptance criterion for this pre-operational leakage rate test was established as 0.1% per 24 hours at 60 psig. This leakage rate was believed consistent with the construction of the containment,⁽²⁾ which is equipped with independent leak-testable penetrations and contains channels over all containment liner welds, which were independently leak tested during construction.

Safety analyses have been performed on the basis of a leakage rate of 0.20% per 24 hours at 60 psig. With this leakage rate and with minimum containment engineered safeguards operating (i. e., either 2 filter units and no spray, or 1 filter unit and 1 spray, or no filter units and 2 sprays) the public exposure would be well below 10 CFR 100 values in the event of the design basis accident.⁽³⁾

Performance of the integrated leakage rate test provides an over-all assessment of potential leakage from the containment in case of an accident that would pressurize the interior of the containment. In order to provide a realistic appraisal of the integrity of the containment under accident conditions, the test is to be performed without preliminary leak detection surveys or leak repairs, and containment isolation valves are to be closed in the normal manner. The test pressure of 35 psig for the integrated leakage rate test is sufficiently high to provide an accurate measurement of the leakage rate and it duplicates the preoperational leakage rate test at 35 psig.

The Specification also allows for possible deterioration of the leakage rate between tests, by requiring that the total measured leakage rate be only 75% of the maximum allowable leakage rate.

The duration and methods for the integrated leakage rate test established by ANSI N45.4-1972 provide a minimum level of accuracy and allow for daily cyclic variation in temperature and thermal radiation. The frequency of the integrated leakage rate test is keyed to the refueling schedule for the reactor, because these tests can best be performed during refueling shutdowns. Refueling shutdowns are scheduled at approximately one year intervals.

The specified frequency of integrated leakage rate tests is based on three major considerations. First is the low probability of leaks in the liner, because of (a) the use of weld channels to test the leaktightness of the welds during erection, (b) conformance of the complete containment to a 0.1% per day leak rate at 60 psig during preoperational testing, and (c) absence of any significant stresses in the liner during reactor operation. Second is the more frequent testing, at the full accident pressure, of those portions of the containment envelope that are most likely to develop leaks during reactor operation (penetrations and isolation valves) and the low value (0.60 La) of the total leakage that is specified as acceptable from penetrations and isolation valves. Third is the tendon stress surveillance program, which provides assurance that an important part of the structural integrity of the containment is maintained.



The basis for specification of a total leakage of 0.60 La from penetrations and isolation valves is that only a portion of the allowable integrated leakage rate should be from those sources in order to provide assurance that the integrated leakage rate would remain within the specified limits during the intervals between integrated leakage rate tests. Because most leakage during an integrated leak rate test occurs through penetrations and isolation valves, and because for most penetrations and isolation valves a smaller leakage rate would result from an integrated leak test than from a local test, adequate assurance of maintaining the integrated leakage rate within the specified limits is provided. The limiting leakage rates from the Recirculation Heat Removal Systems are judgment values based primarily on assuring that the components could operate without mechanical failure for a period on the order of 200 days after a design basis accident. The test

pressure, 350 psig, achieved either by normal system operation or by hydrostatic testing, gives an adequate margin over the highest pressure within the system after a design basis accident. Similarly, the hydrostatic test pressure for the containment sump return lines and the reactor coolant drain tank piping connections to the residual heat removal system of 100 psig gives an adequate margin over the (4) highest pressure within the lines after a design basis accident.

A recirculation system leakage of 2 gal. /hr will limit offsite exposure due to leakage to insignificant levels relative to those calculated for leakage directly from the containment in the design basis accident.

The dose calculated as a result of this leakage is 7.7 mr for a 2-hr (5) exposure at the site boundary.

In case of failure to meet the acceptance criteria for leakage from the residual heat removal system or the penetrations, it may be possible to effect repairs within a short time. If so, it is considered unnecessary and unjustified to shut down the reactor. The times allowed for repairs are consistent with the times developed in Specification 3.3.

The tendon surveillance program is based on assuring that, on the average, the load-carrying capability of the tendons is maintained at approximately 95% design.

This is consistent with the design criteria for the tendons, which allow for uniform capacity reduction of 0.95 and which contemplate that a small fraction of the individual wires 0.03--0.5% may break during
(6)
tensioning.

Periodic visual inspection is the method to be used to determine loss of load-carrying capability because of wire breakage. Since the tendon is under a stress of approximately 144,000 psi, should a wire break, the button head will rise above the top anchor head where it can be readily observed. Assuming that 38 broken wires are observed in 14 tendons (90 wires per tendon), which corresponds to a mean breakage of 3% (97% design load-carrying capability), it can be stated with 95% confidence that the fraction of broken wires in the total containment is between 2.1 and 4.0%. This is based on reliability tables developed
(7)
by North American Aviation, for statistical situations that can be represented by a Poisson distribution. A condition for fitting a Poisson distribution is that the possibility of wire breakage is constant and small. The specification relating to as many as 6 broken wires in one tendon (6.6%) provides that the assumption of a constant probability of occurrence is not significantly violated. The design load can be carried
(8)
even if three adjacent tendons fail completely. The specification has the purpose of alerting against possible deterioration at any time in the plant operating lifetime.

The pre-stress confirmation test provides a direct measure of the load-carrying capability of the tendon.

If the surveillance program indicates by extensive wire breakage or tendon stress relation that the pre-stressing tendons are not behaving as expected, the situation will be evaluated immediately. The specified acceptance criteria are such as to alert attention to the situation well before the tendon load-carrying capability would deteriorate to a point that failure during a design basis accident might be possible. Thus the cause of the incipient deterioration could be evaluated and corrective action studied without need to shut down the reactor. The containment is provided with two readily removable tendons that might be useful to such a study. In addition, there are 40 tendons, each containing a removable wire which will be used to monitor for possible corrosion effects.

References:

- (1) FSAR Section 5.1.2.3
- (2) FSAR Section 5.1.2
- (3) FSAR Section 14.3.5
- (4) FSAR Table 6.2-8
- (5) FSAR Section 6.2.3
- (6) FSAR Page 5.1.2-28
- (7) North-American-Rockwell Report 550-x-32, Autonetics Reliability Handbook, February 1963.
- (8) FSAR Page 5.1.2-28

ATTACHMENT 3

MATERIALS REQUIREMENTS

CONTENTS

1. MATERIALS REQUIREMENTS FROM ACI 318-63
2. MATERIALS REQUIREMENTS FROM ASME CODE



MATERIALS REQUIREMENTS

FROM ACI 318-63

1403—Proportioning

(a) Cement factors shall not be greatly in excess of those necessary to attain the required compressive strength, durability, and other specified properties.

1404—Placing

(a) The maximum slump of the concrete shall be 2 in.

(b) The maximum placing temperature of the concrete when deposited shall be 70 F, and shall be lower if so required by the project specifications.

(c) Concrete shall be placed in lifts approximately 18 in. thick. Vibrator heads shall extend into the previously placed layer.

1405—Curing and protection

The following requirements apply in addition to those of Chapter 12 (Curing and Protection).

(a) The minimum curing period shall be 2 weeks.

(b) When the surrounding air temperature falls below 32 F the surface of the concrete shall be protected against freezing but steam or other curing methods that will add heat to the concrete shall not be used.

(c) The forms and exposed concrete shall be kept continuously wet for at least the first 48 hr after placing, and whenever the surrounding air temperature is above 90 F during the final curing period.

(d) At the conclusion of the specified curing period, means shall be provided, if necessary, to insure that the temperature of the air immediately adjacent to the concrete shall not fall more than 30 F in 24 hr.

CHAPTER 15—PRESTRESSED CONCRETE**1501—General**

(a) Job-cast, post-tensioned, linear prestressed, flexural structural members shall conform to the special provisions of this chapter in addition to all applicable provisions of other chapters in these specifications.

1502—Materials**(a) *Admixtures in concrete***

1. Approved admixtures not containing chlorides, fluorides, or nitrates, may be used.

(b) *Grout*

1. Grout shall consist of a mixture of cement and water unless the gross inside area of the duct exceeds five times the tendon area, in which case fine sand may be added.



2. Fly ash conforming to tentative "Specifications for Fly Ash for Use as an Admixture in Portland Cement Concrete" (ASTM C 350), and pozzolanic material meeting "Specifications for Raw or Calcined Natural Pozzolans for Use as Admixtures in Portland Cement Concrete" (ASTM C 402) may be used. Fly ash or pozzolans shall not exceed 30 lb per sack of cement.

3. When permitted or required by the project specifications, aluminum powder of the proper fineness and quantity or other approved materials may be added to obtain a maximum of 10 percent expansion of the grout when measured unconfined.

4. Proprietary admixtures not containing chlorides, fluorides, or nitrates may be used but not until approved tests or performance records indicate that they will have no harmful effects on the tendons, accessories, or grout.

5. The fine aggregate shall conform to "Specifications for Aggregates for Masonry Grout" (ASTM C 404), Size 2, except that all material shall pass the No. 16 sieve.

6. Proportions of materials shall be based on results of tests made on the grout before grouting is begun. The water content shall be the minimum necessary for proper placement and shall not exceed 5.5 gal. per sack of cement (0.487 absolute ratio by weight). The minimum 7-day compressive strength of 2 in. cubes molded, cured, and tested in accordance with "Test for Compressive Strength of Cement Mortars" (ASTM C 109) shall be 2500 psi.

(c) *Tendons*

1. Tendons shall be of the type required by the project specifications and shall conform to the appropriate specifications as indicated:

a. Wire—one of the following:

- (1) "Specifications for Uncoated Stress-Relieved Wire for Prestressed Concrete" (ASTM A 421)
- (2) "Specifications for Uncoated Seven-Wire Stress-Relieved Strand for Prestressed Concrete" (ASTM A 416)

b. Bars—High-strength alloy steel bars shall be proof stressed to the minimum yield stress. After proof stressing, the bars shall meet the following minimum requirements:

- (1) Class I bars
 - Ultimate strength 145,000 psi
 - Yield stress (0.2 percent offset) 130,000 psi
 - Elongation at rupture in 20 diameters 4 percent
 - Reduction of area at rupture 15 percent
- (2) Class II bars
 - Ultimate strength 160,000 psi
 - Yield stress (0.2 percent offset) 140,000 psi
 - Elongation at rupture in 20 diameters 4 percent
 - Reduction of area at rupture 20 percent



2. The stress-strain curve of the tendons shall be submitted, preferably from the production lot from which the project material was taken. A typical curve may be submitted when this results in a more accurate curve provided that the basis for establishing the curve is approved. For materials not produced under an ASTM standard, the guaranteed breaking strength, yield strength, elongation, composition, and other pertinent data shall be submitted. Certified mill test reports shall be submitted when requested.

3. The amount of slip normally expected in seating anchorage devices, the friction wobble coefficient, and friction curvature coefficient expected for the tendons and duct-forming material shall be submitted. If requested, acceptable test data substantiating the expected coefficients and anchorage slip shall be submitted.

4. Oil-tempered wires shall not be used.

5. Tendons shall be clean and free of excessive rust, scale, and pitting. A light oxide coating is permissible. Unbonded steel shall be permanently protected from corrosion.

6. Tendons shall not be subjected to excessive temperatures, welding sparks, or ground currents. To insure that this requirement is met, burning and welding operations shall not be conducted in the vicinity of tendons without prior approval. Superfluous extension of tendons beyond anchorages may be removed by rapid oxy-acetylene burning, unless such procedures are contrary to the recommendations of the manufacturer of the steel.

(d) *Ducts*

1. Duct-forming material shall be strong enough to retain its shape and resist damage during construction. It shall prevent the entrance of cement paste and water from the concrete. Material left in place shall not cause electrolytic action nor deteriorate. When grouting is required, duct-forming material shall be strong enough to transfer tendon stresses.

2. When grouting is required, the inside diameter of the duct shall be at least $\frac{1}{4}$ in. larger than the bar or strand tendons, or in the case of parallel lay wire cable tendons, the inside area of the duct shall be at least 60 percent larger than the area of the tendon.

3. When grouting is required, the duct shall have grout holes at each end and at all high points except where tendon curvature is small and the tendon is relatively level such as in continuous slabs. Drain holes shall be provided at all low points.

(e) *Anchorage and splices*

1. Anchorages and splices shall be capable of developing the ultimate strength of the tendons without excessive deformation. When requested, satisfactory test data confirming the adequacy of

the proposed device shall be submitted by the contractor. Anchor plates shall be of sufficient size to keep bearing pressures within the stress allowed by "Building Code Requirements for Reinforced Concrete" (ACI 318) for the specified concrete strength at stressing. Splices shall be placed in approved areas and shall be enclosed in housings long enough to permit the necessary movements. Splices shall not be used at points of curvature.

2. Friction type anchorages shall not be used in ungrouted construction unless permitted or approved.

1503—Formwork

(a) Formwork shall not restrain elastic shortening, deflections, or cambers resulting from application of the prestressing force.

(b) Form supports shall not be removed until sufficient prestressing has been applied to support the dead load, formwork, and anticipated construction loads.

(c) When tendons are supported on the forms by draping, the forms shall be sufficiently rigid to prevent displacement of the tendons beyond the tolerances of Section 1504.

1504—Placement and protection of tendons and accessories

(a) Tendons and ducts, or duct-forming devices for use in bonded construction shall be free of excessive rust, grease, oil, paint, and other foreign matter. A light coat of rust is permissible, provided loose rust has been removed and the surface of the steel is not pitted.

(b) Materials for use in unbonded construction shall be clean and undamaged and shall be permanently protected as specified in the project specifications.

(c) End anchorages which will be permanently protected with concrete shall be kept free of loose rust, grease, oil, paint, and other foreign matter.

(d) Ducts, tendons, and anchorages shall be firmly supported to prevent displacement during subsequent operations. They shall be placed with a tolerance of $\pm \frac{1}{8}$ in. in concrete dimensions of 8 in. or less, $\pm \frac{1}{4}$ in. in concrete dimensions over 8 in. but not over 2 ft, and $\pm \frac{1}{2}$ in. in concrete dimensions over 2 ft. These tolerances apply separately to both vertical and horizontal dimensions and might be different for both directions.

(e) Grout fittings and ducts for bonded construction shall be adequately protected from collapse and other damage. Prior to placing concrete, the ducts and grout fittings shall be examined for holes. All such holes shall be repaired.



(f) The bearing surface between anchorages and concrete shall be perpendicular to and concentric with the tendons and the line of action of the prestressing force.

1505—Application of prestressing force

(a) The steel shall be stressed in the sequence, at the concrete strength, and at the construction stage indicated by the project specifications.

(b) The prestressing force shall be determined (1) by measuring tendon elongation and also (2) either by checking jack pressure on a recently calibrated gage or by use of a recently calibrated dynamometer. Any discrepancy which exceeds 5 percent shall be corrected. Elongation requirements shall be based on load-elongation curves for the steel used. The contractor shall keep a record of gage pressures and dynamometer readings, and measured elongations for each tendon and such records shall be submitted.

(c) The total loss of prestress due to unreplaced broken tendons shall not exceed 2 percent of the total prestress.

1506—Grouting

(a) A dependable high-pressure water supply shall be provided before grouting is begun. Ducts shall be cleaned of dirt and other foreign substances by thoroughly flushing with water immediately prior to grouting. Unless tendons are adequately protected from corrosion, grouting operations shall be completed not more than 28 days after concrete is placed around the ducts or after tendons are placed inside the ducts, whichever is later.

(b) Grout shall be mixed in a high-speed mechanical mixer and passed through a strainer into pumping equipment which has provision for recirculation. Pumping of grout shall begin as soon as possible after mixing and may be continued as long as the grout retains the proper consistency. Grout which has set shall not be retempered but shall be discarded.

(c) Grout shall be injected into all voids between prestressing tendons and duct and anchorage fittings. Flow shall continue until grout of the consistency equivalent to that injected, flows without the presence of air bubbles from all vent openings. Vent openings shall be closed progressively in the direction of flow. After all vent openings are closed, the grouting pressure shall be raised to at least 50 psi and the injection hole plugged.

(d) In the event of a blockage or an interruption of grouting, all grout shall be removed from the duct by flushing with water.



(e) Trapped water in ungrouted ducts shall be protected from freezing. The concrete around grouted tendons shall be maintained at a temperature of 45 F or higher for at least 3 days after grouting.

1507—Shop drawings

(a) Shop drawings for prestressed concrete shall meet the following special provisions in addition to applicable provisions of Section 103:

1. Shop drawings shall indicate the location of tendons throughout their length.

2. Size, details, location, materials, and stress grade (where applicable), and installation of all tendons and accessories shall be indicated. Jack clearances, procedures, stressing sequence, initial tensioning forces, gage pressures, and tendon elongation shall be noted.

3. Information required in Sections 1502 (c) 2 and 3 shall be submitted with the shop drawings.

CHAPTER 16—TESTING

1601—General

(a) Routine testing of materials, of proposed mix designs, and of resulting concrete for compliance with technical requirements of the specifications shall be the duty of the testing agency designated in the project specifications and will be performed without expense to the contractor. The contractor shall be at liberty to engage at his expense a separate testing agency for his own information and guidance.

(b) Testing of field cured test cylinders or testing required because of changes in materials or proportions of the mix requested by the contractor, as well as any extra testing of concrete or materials occasioned by their failure to meet specification requirements, shall be at the contractor's expense.

1602—Testing services

(a) The designated testing agency shall:

1. Test the contractor's proposed materials for compliance with the specifications.

2. Review and check test the contractor's proposed mix design.

3. Secure production samples of materials at plants or stockpiles during the course of the work and test for compliance with the specifications.

4. Conduct strength tests of the concrete in accordance with the following procedures:

- a. Secure composite samples in accordance with "Method of Sampling Fresh Concrete" (ASTM C 172). Each strength test shall be obtained from a different batch of concrete on a



CHAPTER 26 — PRESTRESSED CONCRETE

2600—Notation

- $a = A_s f_{su} / 0.85 f'_c b$
 $A_b =$ bearing area of anchor plate of post-tensioning steel
 $A_b' =$ maximum area of the portion of the anchorage surface that is geometrically similar to and concentric with the area of the anchor plate of the post-tensioning steel
 $A_s =$ area of prestressed tendons
 $A_{sf} =$ area of reinforcement to develop compressive strength of overhanging flanges in flanged members
 $A_{sr} =$ area of tendon required to develop the web
 $A_s' =$ area of unprestressed reinforcement
 $A_v =$ area of web reinforcement placed perpendicular to the axis of the member
 $b =$ width of compression face of flexural member
 $b' =$ minimum width of web of a flanged member
 $d =$ distance from extreme compression fiber to centroid of the prestressing force
 $f'_c =$ compressive strength of concrete (see Section 301)
 $f_{ci}' =$ compressive strength of concrete at time of initial prestress
 $f_{cp} =$ permissible compressive concrete stress on bearing area under anchor plate of post-tensioning steel
 $f_d =$ stress due to dead load, at the extreme fiber of a section at which tension stresses are caused by applied loads
 $f_{pc} =$ compressive stress in the concrete, after all prestress losses have occurred, at the centroid of the cross section resisting the applied loads, or at the junction of the web and flange when the centroid lies in the flange. (In a composite member f_{pc} will be the resultant compressive stress at the centroid of the composite section, or at the junction of the web and flange when the centroid lies within the flange, due to both prestress and to the bending moments resisted by the precast member acting alone.)
 $f_{pe} =$ compressive stress in concrete due to prestress only, after all losses, at the extreme fiber of a section at which tension stresses are caused by applied loads
 $f_s' =$ ultimate strength of prestressing steel
 $f_{se} =$ effective steel prestress after losses
 $f_{su} =$ calculated stress in prestressing steel at ultimate load
 $f_{sy} =$ nominal yield strength of prestressing steel
 $f_v' =$ strength of unprestressed reinforcement (see Section 301)
 $F_{sp} =$ ratio of splitting tensile strength to the square root of compressive strength (see Section 505)
 $h =$ total depth of member



I	=	Moment of inertia of section resisting externally applied loads*
K	=	wobble friction coefficient per foot of prestressing steel
L	=	length of prestressing steel element from jacking end to any point x
M	=	bending moment due to externally applied loads*
M_{cr}	=	net flexural cracking moment
M_u	=	ultimate resisting moment
p	=	A_s/bd ; ratio of prestressing steel
p'	=	A_s'/bd ; ratio of unprestressed steel
q	=	$p f_{su}/f_c'$
s	=	longitudinal spacing of web reinforcement
T_o	=	steel force at jacking end
T_x	=	steel force at any point x
t	=	average thickness of the compression flange of a flanged member
V	=	shear due to externally applied loads*
V_c	=	shear carried by concrete
V_{ci}	=	shear at diagonal cracking due to all loads, when such cracking is the result of combined shear and moment
V_{cr}	=	shear force at diagonal cracking due to all loads, when such cracking is the result of excessive principal tension stresses in the web
V_d	=	shear due to dead load
V_p	=	vertical component of the effective prestress force at the section considered
V_u	=	shear due to specified ultimate load
y	=	distance from the centroidal axis of the section resisting the applied loads to the extreme fiber in tension
α	=	total angular change of prestressing steel profile in radians from jacking end to any point x
e	=	base of Napierian logarithms
μ	=	curvature friction coefficient
ϕ	=	capacity reduction factor (see Section 1504)

2601—Definitions

(a) The following terms are defined for use in this chapter:

Anchorage — The means by which the prestress force is permanently delivered to the concrete.

Bonded tendons — Tendons which are bonded to the concrete either directly or through grouting. Unbonded tendons are free to move relative to the surrounding concrete.

Effective prestress — The stress remaining in the tendons after all losses have occurred, excluding the effects of dead load and superimposed loads.

*The term "externally applied loads" shall be taken to mean the external ultimate loads acting on the member, excepting those applied to the member by the prestressing tendons.

Friction:

Curvature friction — Friction resulting from bends or curves in the specified cable profile.

Wobble friction — Friction caused by the unintended deviation of the prestressing steel from its specified profile.

Jacking force — The temporary force exerted by the device which introduces the tension into the tendons.

Nominal yield strength — The yield strength specified by appropriate ASTM specification or as indicated by Section 405(f).

Post-tensioning — A method of prestressing in which the tendons are tensioned after the concrete has hardened.

Pretensioning — A method of prestressing in which the tendons are tensioned before the concrete is placed.

Tendon — A tensioned steel element used to impart prestress to the concrete.

Transfer — The operation of transferring the tendon force to the concrete.

2602—Scope

(a) Provisions in this chapter apply to flexural members prestressed with high-strength steel. Pavements, pipes, and circular tanks are not included.

(b) For prestressed concrete designs or constructions in conflict with, or not encompassed by the provisions of this chapter, see Section 104.

(c) All provisions of this code not specifically excluded and not in conflict with the provisions of this chapter are to be considered applicable to prestressed concrete.

(d) The following provisions shall not apply to prestressed concrete: Sections 906, 911, 913, Chapters 13 and 14, Section 1508, Chapter 18, Section 2001(a), Chapter 21, and Section 2504(b).

2603—General considerations

(a) Stresses and ultimate strength shall be investigated at service conditions and at all load stages that may be critical during the life of the structure from the time prestress is first applied.

(b) Stress concentrations due to the prestressing or other causes shall be taken into account in the design.

(c) The effects on the adjoining structure of elastic and plastic deformations, deflections, changes in length, and rotations caused by the prestressing shall be provided for. When the effect is additive to temperature and shrinkage effects, they shall be considered simultaneously.

(d) The possibility of buckling of a member between points of contact between concrete and prestressing steel and of buckling of thin webs and flanges shall be considered.

2604—Basic assumptions

- (a) The following assumptions shall be made for purposes of design:
1. Strains vary linearly with depth through the entire load range.
 2. At cracked sections, the ability of the concrete to resist tension is neglected.
 3. In calculations of section properties prior to bonding of tendons, areas of the open ducts shall be deducted. The transformed area of bonded tendons may be included in pretensioned members and in post-tensioned members after grouting.
 4. Modulus of elasticity of concrete shall be assumed as prescribed in Section 1102.
 5. The modulus of elasticity of prestressing steel shall be determined by tests or supplied by the manufacturer.

2605—Allowable stresses in concrete

- (a) Temporary stresses immediately after transfer, before losses due to creep and shrinkage, shall not exceed the following:

1. Compression $0.60 f_{ci}'$
2. Tension stresses in members without auxiliary reinforcement (unprestressed or prestressed) in the tension zone. $3\sqrt{f_{ci}'}$

Where the calculated tension stress exceeds this value, reinforcement shall be provided to resist the total tension force in the concrete computed on the assumption of an uncracked section.

- (b) Stresses at design loads, after allowance for all prestress losses, shall not exceed the following:

1. Compression $0.45 f_c'$
2. Tension in the precompressed tension zone:
Members, not exposed to freezing temperatures nor to a corrosive environment, which contain bonded prestressed or unprestressed reinforcement located so as to control cracking $6\sqrt{f_c'}$
All other members 0

These values may be exceeded when not detrimental to proper structural behavior as provided in Section 104.

- (c) The bearing stress on the concrete created by the anchorage in post-tensioned concrete with adequate reinforcement in the end regions shall not exceed:

$$f_{cp} = 0.6 f_{ci}' \sqrt{A_b' / A_b} \dots \dots \dots (26-1)$$

but not greater than f_{ci}'

2606—Allowable stresses in steel

- (a) Temporary stresses

1. Due to temporary jacking force $0.80 f_s'$



but not greater than the maximum value recommended by the manufacturer of the steel or of the anchorages.

2. Pretensioning tendons immediately after transfer, or posttensioning tendons immediately after anchoring.... $0.70 f'_s$

(b) *Effective prestress* $0.60 f'_s$
or $0.80 f_{sy}$
whichever is smaller

2607—Loss of prestress

(a) To determine the effective prestress, allowance for the following sources of loss of prestress shall be considered.

1. Slip at anchorage
2. Elastic shortening of concrete
3. Creep of concrete
4. Shrinkage of concrete
5. Relaxation of steel stress
6. Frictional loss due to intended or unintended curvature in the tendons

(b) Friction losses in post-tensioned steel shall be based on experimentally determined wobble and curvature coefficients,* and shall be verified during stressing operations. The values of coefficients assumed for design, and the acceptable ranges of jacking forces and steel elongations shall be shown on the plans. These friction losses shall be calculated:

$$T_o = T_s e^{(KL + \mu\alpha)} \dots \dots \dots (26-2)$$

When $(KL + \mu\alpha)$ is not greater than 0.3, Eq. (26-3) may be used.

$$T_o = T_s (1 + KL + \mu\alpha) \dots \dots \dots (26-3)$$

(c) When prestress in a member may be reduced through its connection with adjoining elements, such reduction shall be allowed for in the design.

*Values of K (per lineal foot) and μ vary appreciably with duct material and method of construction. For metal sheathing the following table may be used as a guide.

Type of steel	Usual range of observed values		Suggested design values	
	K	μ	K	μ
Wire cables	0.0005-0.0030	0.15-0.35	0.0015	0.25
High strength bars	0.0001-0.0006	0.08-0.30	0.0003	0.20
Galvanized strand	0.0005-0.0020	0.15-0.30	0.0015	0.25



2608—Ultimate flexural strength

(a) The required ultimate load on a member, determined in accordance with Part IV-B shall not exceed the ultimate flexural strength computed by:

1. Rectangular sections, or flanged sections in which the neutral axis lies within the flange:*

$$M_u = \phi \left[A_s f_{su} d (1 - 0.59 q) \right] = \phi \left[A_s f_{su} \left(d - \frac{a}{2} \right) \right] \dots (26-4)$$

2. Flanged sections in which the neutral axis falls outside the flange:†

$$M_u = \phi \left[A_{sr} f_{su} d \left(1 - \frac{0.59 A_{sr} f_{su}}{b' d f_c'} \right) + 0.85 f_c' (b - b') t (d - 0.5t) \right] \dots (26-5)$$

where

$$A_{sr} = A_s - A_{sf}$$

and

$$A_{sf} = 0.85 f_c' (b - b') t / f_{su}$$

3. Where information for the determination of f_{su} is not available, and provided that f_{se} is not less than $0.5 f_c'$, the following approximate values shall be used:

Bonded members

$$f_{su} = f_s' (1 - 0.5 p f_s' / f_c') \dots (26-6)$$

Unbonded members

$$f_{su} = f_{se} + 15,000 \text{ psi} \dots (26-7)$$

4. Nonprestressed reinforcement, in combination with prestressed steel, may be considered to contribute to the tension force in a member at ultimate moment an amount equal to its area times its yield point, provided

$$\frac{p f_{su}}{f_c'} + \frac{p' f_y}{f_c'} \text{ does not exceed } 0.3$$

2609—Limitations on steel percentage

(a) Except as provided in (b), the ratio of prestressing steel used for calculations of M_u shall be such that

$$p f_{su} / f_c' \text{ is not more than } 0.30$$

For flanged sections, p shall be taken as the steel ratio of only that portion of the total tension steel area which is required to develop the compressive strength of the web alone.

*Usually where the flange thickness is more than $1.4 d p f_{su} / f_c'$.

†Usually where the flange thickness is less than $1.4 d p f_{su} / f_c'$.



MATERIALS REQUIREMENTS

FROM ASME CODE

ARTICLE CC-2000

MATERIAL

CC-2100 GENERAL REQUIREMENTS FOR MATERIAL

CC-2110 SCOPE

CC-2111 Terms

(a) The term *material* as used in this Subsection applies to those items produced to the requirements of:

- (1) an SA¹ or SFA² Specification of Section II; or
- (2) any other material specification permitted by this Subsection.

(b) The term *Material Manufacturer* is defined as a Manufacturer who produces the materials to the requirements of the material specification and any additional or special tests designated by this Article except as provided in CC-2122. This includes, for example, concrete, admixtures, cements, and welding materials. Additionally, the Material Manufacturer shall be responsible for packaging and shipping the materials in the manner prescribed in the Construction Specification.

(c) The terms *pressure-retaining* and *load-bearing* materials apply to items such as concrete, reinforcing material, prestressing material, liners, attachments, and other embedments.

¹Material produced under an ASTM designation may be accepted as complying with the corresponding ASME specification provided the ASME specification is designated as being identical with the ASTM specification for the grade, class, or type, and provided that the material is confirmed as complying with the ASTM specification by a Certified Material Test Report (CMTR) or Certificate of Compliance, in accordance with the requirements of CC-2130.

²Welding or brazing material produced to an AWS designation may be accepted in lieu of the corresponding ASME specification provided that the latter specification is indicated to be identical with the AWS specification, and provided that the material is confirmed as complying with the AWS specification by a Certified Material Test Report (CMTR) in accordance with the requirements of CC-2130.

CC-2112 Special Rules

Material for parts and appurtenances designated to meet the requirements for Class MC and which are not backed by concrete for load-carrying purposes shall meet the requirements of NE-2000. Subsection CA shall be used in lieu of the requirements of Subsection NA. Material for attachments to parts meeting the requirements of Division I shall meet the requirements of this Article.

CC-2120 PRESSURE-RETAINING AND LOAD-BEARING MATERIALS

CC-2121 Permitted Material Specifications

(a) Concrete material shall conform to the requirements of CC-2200. Steel material for pressure-retaining and load-bearing purposes shall conform to the requirements of one of the specifications included in this Article or in Table 1-1.2 and Table 1-2.2 of Appendix I, and to all the special requirements of this Article which apply to the product form in which the material is used.

(b) Material other than those described in (a) above shall not be used until the requirements of Appendix IV have been met.

(c) The requirements of this Article do not apply to items not associated with the pressure-retaining or load-bearing function of a component, such as forms, tie wires, chairs, supports, form ties, grease and grout fittings, and retaining caps, or to seals, packing, and gaskets.

CC-2122 Special Requirements

The special requirements stipulated in this Article shall apply in lieu of the requirements of the material specifications wherever these special requirements conflict with the material specification requirements. Where the special requirements include an ex-

amination, test, or treatment which is also required by the material specification, the examination, test, or treatment need be performed only once. All required examinations, tests, and treatments shall be performed as specified for each product in this Article and may be performed by either the Material Manufacturer, Fabricator, or Constructor as provided in CC-4121.

CC-2123 Size Ranges

CC-2123.1 Concrete Material. Concrete material shall be furnished in the sizes required by the Construction Specification.

CC-2123.2 Metallic Material. Metallic material outside the limits of size or thickness given in any specification allowed by this Division may be used if the material is in compliance with the other requirements of the specification and no size limitation is given in the rules for construction. In those specifications in which chemical composition and mechanical properties are indicated to vary with size or thickness, any material outside the specification range shall be required to conform to the composition and mechanical properties shown for the nearest specified range.

CC-2130 CERTIFICATION OF MATERIAL

CC-2131 Introduction

The Material Manufacturer shall provide a Certified Material Test Report (CMTR) or a Certificate of Compliance for all material in accordance with the following subparagraphs.

CC-2131.1 Certified Material Test Reports. Certified Material Test Reports (CMTR) shall be provided for concrete material, reinforcing system material, prestressing system material, liner material, other load-bearing and pressure-retaining material, and welding and brazing material, and shall include the following:

(a) certified reports of the actual results of all required chemical analyses, physical tests, mechanical tests, examinations (including radiographic film), repairs, and heat treatments (including times and temperatures) performed on the material

(b) a statement listing any chemical analyses, tests, examinations, and heat treatment required by the material specification, which were not performed

(c) a statement giving the manner in which the material is identified, including specific marking

CC-2131.2 Certificate of Compliance. A Material Manufacturer's Certificate of Compliance with the material specification, grade, class, and heat treatment condition, as applicable, may be provided in lieu of a Certified Material Test Report for metallic material $\frac{3}{4}$ in. nominal pipe size and less (pipe, tube, flanges, and fittings), and bolting 1 in. nominal diameter and less. The Certificate of Compliance shall contain a description of the marking of the material.

CC-2132 Certification by Fabricator or Constructor

The Fabricator or Constructor shall provide a Certified Material Test Report for all operations performed by him or his subcontractors. The Fabricator or Constructor shall certify that the contents of his report are correct and accurate and that all operations performed by him or his subcontractors are in compliance with the requirements of the material specification and this Subsection. Alternatively, the Fabricator or Constructor shall provide a Certified Material Test Report for those operations he performed and a Certified Material Test Report from each of his subcontractors for operations they performed. Material identification, including any marking code, shall be described in the Certified Material Test Report.

CC-2140 DETERIORATION OF MATERIAL AND COATINGS DURING SERVICE

Material provided to meet the service conditions specified in the Design Specification shall be evaluated for their adequacy to withstand the service conditions, including deteriorating factors which may destroy the minimum capabilities needed to satisfy the requirements of this Subsection.

CC-2150 HEAT TREATMENT TO ENHANCE MECHANICAL PROPERTIES OF METALLIC MATERIAL

CC-2151 Notch-Toughness and Hardness Properties

Carbon and low-alloy steels may be heat treated by quenching and tempering to enhance notch-toughness or hardness properties. For carbon steels, postweld heat treatment of the material at a temperature of not less than 1100 F may be considered to

be the tempering phase of the heat treatment when postweld heat treatment is a requirement of CC-4500.

CC-2152 Stress Relaxation Properties

Wire, strand, or bars for prestressing systems may be heat treated, as in stress-relieving, under load to enhance stress-relaxation properties. Quenching and tempering treatments to produce specific mechanical properties are not permitted.

CC-2160 DIMENSIONAL STANDARDS

Dimensions of standard metallic items shall comply with the standards and specifications listed in Table CC-2160-1 unless otherwise specified in the Construction Specification or designated in the Design Drawings.

CC-2200 CONCRETE AND CONCRETE MATERIAL

CC-2210 INTRODUCTION

(a) This Subarticle establishes the requirements for concrete materials in their individual and mixed conditions for use in conventional concrete (for example, using stationary or transit mixer), concrete with preplaced aggregate, and grout.

(b) The requirements of this Subarticle are meant as material acceptance requirements for concrete and its constituents. Those tests and additional re-

quirements which are performed on concrete and concrete material during storage, construction, and curing are described in CC-5200.

CC-2211 General Requirements

(a) Consideration shall be given to minimizing the temperature rise in concrete due to heat of hydration, and to proper strength development with respect to formwork removal, construction stresses, and application of prestress.

(b) The concrete and its materials shall be sufficiently investigated to ensure acceptable creep and other properties under the environmental conditions and the long term requirements described in the Construction Specification.

CC-2220 MATERIAL FOR CONCRETE

CC-2221 Cement

CC-2221.1 Material Requirements. Cement shall conform to the requirements of ASTM C 150, "Specification for Portland Cement," Type II, Type IV, or Type V having moderate heat of hydration. In areas where the structure may come into contact with soils having more than 0.20% water-soluble sulphate, or ground water with a sulphate concentration exceeding 1000 ppm, Type V only shall be used for concrete in contact with the ground unless other suitable means are used to prevent sulphate attack and concrete deterioration.

TABLE CC-2160-1
DIMENSIONAL STANDARDS¹

Standard	Designation
Pipe and Tubes	
Wrought Steel and Wrought-Iron Pipe	ANSI B36.10-1959
Stainless Steel Pipe	ANSI B36.19-1965
Fittings, Valves Flanges, and Gaskets	
Steel Pipe Flanges and Flanged Fittings	ANSI B16.5-1968
Wrought Steel Butt welding Fittings	ANSI B16.9-1964
Wrought Steel Butt welding Short Radius Elbows and Returns	ANSI B16.28-1964
Steel Butt welding Fittings (26 in. and larger)	MSS SP-48-1969
Standard Finishes for Contact Faces of Pipe Flanges and Connecting-End Flanges of Valves and Fittings	MSS SP-6-1963
Spot Facing For Bronze, Iron, and Steel Flanges	MSS SP-9-1964
Standard Marking System for Valves, Fittings, Flanges and Unions	MSS SP-25-1964
Stainless Steel Butt welding Fittings	MSS SP-43-1959

NOTE:

(1) Standards incorporated in this Division by reference. The names of the sponsoring organizations are shown in the Reference Materials portion of this Division.

CC-2222 Aggregates

CC-2222.1 Material Requirements. Aggregates shall conform to the requirements of ASTM C 33, "Specification for Concrete Aggregates," with the following additional requirements:

(a) Gradations 357 or 467 shall not be furnished as one graded aggregate but shall be obtained by combining at least two separate gradation sizes.

(b) Coarse and fine aggregate for normal weight concrete, placed by the preplaced aggregate method, shall meet the gradation requirements presented in ACI 304, "Recommended Practice for Measuring, Mixing, Transporting, and Placing Concrete."

(c) Heavy aggregates required for concrete weighing more than 160 lb per cu ft shall conform to ASTM C 637, "Specification for Aggregates for Radiation Shielding Concrete."

(d) Fine aggregate for general purpose grout shall meet the requirements of ASTM C 33 except that the gradation may be adjusted to comply with the size of the opening and the method of placement.

(e) The potential reactivity of the aggregate shall be established by the methods described in the Appendix to ASTM C 33.

(f) The water-soluble chloride content of the aggregate shall be established by the methods described in ASTM D 1411, "Water-Soluble Chlorides Present as Admixes in Graded Aggregate Road Mixes."

(g) A petrographic examination in accordance with ASTM C 295 shall be performed.

CC-2222.2 Aggregate Shapes and Sizes

(a) Aggregate shapes shall be rounded or cubical and shall contain less than 15%, by weight, flat and elongated particles as determined by CRD-C 119, "Method of Test for Flat and Elongated Particles in Coarse Aggregate."

NOTE: A flat particle is defined as one having a ratio of width to thickness greater than three; an elongated particle is defined as one having a ratio of length to width greater than three.

(b) The maximum size of coarse aggregate shall not be larger than one-fifth of the narrowest dimension of the finished wall or slab, nor larger than three-fourths of the minimum clear spacing between reinforcing steel and embedments.

CC-2223 Mixing Water

CC-2223.1 General Requirements. Water for mixing shall be clean with a total solids content of not more than 2000 ppm as measured by American Public Health Association's "Standard Method for De-

termination of Total Solids." The mixing water, including that contained as free water in aggregates, shall contain not more than 250 ppm of chlorides as Cl^- as determined by ASTM D 512, "Chloride Ion in Industrial Water and Industrial Waste Water." Chloride ions contained in the aggregates shall be used in calculating the total chloride ion content of the mixing water. The chloride ion contributed by the aggregates shall be determined in accordance with CC-2222.1(f).

CC-2223.2 Additional Requirements. A comparison of the proposed mixing water properties shall be made with distilled water by performing the following tests:

(a) Soundness, in accordance with ASTM C 151, "Autoclave Expansion of Portland Cement"—The results obtained for the proposed mixing water shall not be more than +0.10 above those obtained for distilled water.

(b) Time of Setting, in accordance with ASTM C 191, "Time of Setting Hydraulic Cement by Vicat Needle"—The results obtained for the proposed mixing water shall be within ± 10 min for initial setting time and ± 1 hr for final setting time of those obtained for distilled water.

(c) Compressive Strength, in accordance with ASTM C 109, "Compressive Strength of Hydraulic Cement Mortars (Using 2 in. Cube Specimens)"—The results obtained for the proposed mixing water shall not be lower by more than 10% of those obtained for distilled water.

CC-2224 Admixtures

CC-2224.1 General Requirements for the Use of W75 Admixtures. When admixtures are to be used, the Construction Specification shall specify the type, quantity, and additional limitations. Admixtures containing more than 1% chloride ions shall not be used for prestressed containments.

CC-2224.2 Detailed Requirements

CC-2224.2.1 Air Entraining Admixtures. Air entraining admixtures shall conform to the requirements of ASTM C 260, "Air Entraining Admixtures for Concrete."

CC-2224.2.2 Fly Ash and Pozzolanic Material. Fly ash and pozzolanic material shall conform to the requirements of ASTM C 618, "Fly Ash and Raw or Calcined Natural Pozzolans for Use in Portland Cement Concrete."

CC-2224.2.3 Chemical Admixtures. Chemical admixtures shall conform to the requirements of

ASTM C 494, "Chemical Admixtures for Concrete," except that grout fluidifier for use with preplaced aggregate concrete shall be in accordance with CRD-C 566, "Specification for Grout Fluidifier."

CC-2230 CONCRETE MIX DESIGN

CC-2231 Concrete Properties

CC-2231.1 General Requirements. The properties of concrete which influence the design of the containment shall be established in the Construction Specification. Concrete *types* will normally be established on the basis of strength and workability limits.

CC-2231.2 Specified Properties

(a) The concrete properties given in Table CC-2231.2-1 shall be defined in the Construction Specification and shall be measured prior to construction in accordance with the respective specification.

(b) The Construction Specification shall specify the age and temperature at which the properties listed in Table CC-2231.2-1 shall be obtained and any environmental or design conditions which apply. If a particular property listed in Table CC-2231.2-1 is not required, the Construction Specification shall so state. Likewise, if properties other than those listed in Table CC-2231.2-1 are required, they shall be defined in the Construction Specification.

CC-2231.3 Additional Property Requirements

CC-2231.3.1 Mechanical Properties. Where the Construction Specification includes maximum allowable creep and shrinkage limits (rate and total), tests shall be performed at least for the limiting combination of stress-strength ratios, ages of loading and temperature levels, and cycles stated in the

TABLE CC-2231.2-1
CONCRETE PROPERTIES

Property	Specification
Slump	ASTM C 143
Compressive strength	ASTM C 39
Flexural strength	ASTM C 78
Splitting tensile strength	ASTM C 496
Static modulus of elasticity	ASTM C 469
Poisson's ratio	ASTM C 469
Coefficient of thermal conductivity	CRD-C 44
Coefficient of thermal expansion	CRD-C 39
Creep of concrete in compression	ASTM C 512
Shrinkage coefficient (length change of cement-mortar and concrete)	ASTM C 157
Density (specific gravity)	ASTM C 642
Aggregates for radiation-shielding concrete	ASTM C 637

Construction Specification. The creep test procedure shall be based on ASTM C 512, "Method of Test for Creep of Concrete in Compression," with modifications for specimen curing and sealing, and for testing temperatures as described in the Construction Specification.

CC-2231.3.2 Physical Properties. Where the Construction Specification defines an allowable range of secant moduli of elasticity, Poisson's ratios, and loading rates, concrete specimens shall be tested at those loading rates and the experimental values determined. The testing procedures and acceptance standards shall be based on ASTM C 469, "Method of Test for Static Young's Modulus of Elasticity and Poisson's Ratio in Compression of Cylindrical Concrete Specimens," with suitable modifications.

CC-2231.3.3 Thermal Properties. When required by the Construction Specification, the effect of thermal cycles on expansion and other concrete properties shall be determined by the test standards described therein.

CC-2232 Selection of Concrete Mix Proportions

CC-2232.1 Introduction. Proportions of material for concrete shall be established on the basis of laboratory trial batches to provide the following:

(a) conformance with the concrete strength requirements

(b) adequate workability and proper consistency to permit the concrete to be worked readily into the forms and around reinforcement under the conditions of placement to be employed, without excessive segregation or bleeding

(c) resistance to freezing and thawing and other aggressive actions, where required

CC-2232.2 Strength Tests

(a) The proportions, including water-cement ratio, shall be established on the basis of laboratory trial batches in accordance with ACI 211.1, "Recommended Practice for Selecting Proportions for Normal and Heavyweight Concrete." The proportions shall be selected to produce an average strength at the designated test age exceeding f'_c by at least 1200 psi when both air content and slump are the maximum permitted by the specifications.

(b) Using the methods of ACI 214, "Recommended Practice for Evaluation of Compression Test Results of Field Concrete," the amount by which the average strength must exceed f'_c may be reduced to an appropriate level below 1200 psi after



sufficient test data becomes available from the job to indicate that, at the lower average strength, the probable frequency of test more than 500 psi below f_c' will not exceed 1 in 100 and that the probable frequency of an average of three consecutive tests below f_c' will not exceed 1 in 100.

(c) The strength tests shall be made in accordance with ASTM C 39, "Method of Test for Compressive Strength of Molded Concrete Cylinders," on specimens prepared in accordance with ASTM C 192, "Method of Making and Curing Test Specimens in the Laboratory." A curve shall be established showing the relationship between water-cement ratio and compressive strength. The curve shall be based on at least three points representing batches which produce strengths above and below that required. Each point shall represent the average of at least three specimens tested at 28 days or other age as specified in the Construction Specification.

(d) The maximum permissible water-cement ratio for the concrete to be used in the structure shall be that shown by the curve to produce the average strength given in (a) or (b) above, unless a lower water-cement ratio is required by the durability requirements.

CC-2232.3 Durability

CC-2232.3.1 Concrete Subject to Freezing Temperatures. Concrete that, after curing, will be subject to freezing temperatures while wet shall contain entrained air within the limits of Table CC-2232.3-1 and the water-cement ratio shall not exceed 0.53 by weight.

CC-2232.3.2 Watertight Concrete. Concrete that is intended to be watertight shall have a maximum water-cement ratio of 0.48 for exposure to fresh water and 0.44 for exposure to sea water.

CC-2232.3.3 Sulphate-Resisting Concrete. Concrete that will be exposed to injurious concentrations of sulphate-containing solutions shall conform to maximum water-cement ratio of 0.44 and be made with sulphate-resisting cement as given CC-2221.1.

W75 CC-2232.4 Preplaced Aggregate Concrete. Preplaced aggregate concrete materials and proportions shall conform to the requirements of ACI 304, "Recommended Practice for Measuring, Mixing, Transporting, and Placing Concrete."

W75 CC-2232.5 Heavy Weight Aggregate Concrete
(a) The proportions of heavy weight aggregate concrete shall be made in accordance with ACI 211.1, "Recommended Practice for Selecting Proportions for Normal and Heavyweight Concrete."

TABLE CC-2232.3-1
CONCRETE AIR CONTENT FOR VARIOUS SIZES
OF COARSE AGGREGATES

Nominal maximum size of coarse aggregate, in.	Total air content (vol), %
3/8	6 to 10
1/2	5 to 9
3/4	4 to 8
1	3.5 to 6.5
1-1/2	3 to 6
2	2.5 to 5.5
3	1.5 to 4.5

(b) The requirements for compressive strength, durability, and specific properties given in CC-2231.2 shall be as required by the Construction Specification. The aggregates shall conform to ASTM C 637, "Aggregates for Radiation Shielding Concrete."

CC-2240 GROUT

CC-2241 Material-for Cement Grout for
General Grouting Purposes

CC-2241.1 Cement. Cement for grout shall conform W75 to ASTM C 150, Types I, II, III, IV, or V.

CC-2241.2 Aggregates. Aggregates shall meet the requirements of ASTM C 33 except that the gradation may be adjusted in the Construction Specification to comply with the size of the opening and the method of placement.

CC-2241.3 Water. Mixing water shall conform to the requirements of CC-2223.

CC-2241.4 Admixtures. Admixtures shall conform to the requirements of CC-2224.

CC-2241.5 Other Material. Other material, such as aluminum powder of the proper fineness and quality, may be added to the grout mix provided adequate tests are made to demonstrate the suitability of the mix for the purpose intended.

CC-2242 Design of Cement Grout for General
Grouting Purposes

CC-2242.1 General Requirements

(a) Requirements described herein do not apply to grout which is used with grouted prestressing systems.

(b) The proportions of materials for general purpose grout shall be based upon trial mixes using the same type and brand of cement, fine aggregate, and admixtures as will be used for construction. Insofar as is practical, the same type of mixer shall be used in preparing trial mixes as will be used for construction.



CC-2242.2 Compressive Strength. The compressive strength of the grout shall be established by 2 in. cubes, modeled, cured, and tested in accordance with ASTM C 109, "Standard Test for Compressive Strength of Cement Mortars," except that, if expansive grout is used, the tests shall be performed in accordance with CRD-C 589.

CC-2242.3 Fly Ash and Pozzolans. When fly ash or pozzolans are used as admixtures, the quantity shall be such that the requirements for strength, workability, and durability are met.

CC-2242.4 Water. The water content shall be the minimum quantity necessary for the proper placement by the grouting method employed, dry pack or fluid grout.

CC-2243 Cement Grout for Grouted Tendon Systems

CC-2243.1 General Requirements. Grout for grouted tendon systems shall generally consist of a mixture of cement, water, and an expanding additive, unless the gross inside area of the tendon tube exceeds five times the tendon area, in which case fine aggregate fillers may be used provided the design of the mix is such that it can penetrate all spaces and obtain the expected expansion and sedimentation characteristics (bleeding) for both horizontal and vertical tendons, as required.

CC-2243.2 Physical Properties of the Grout

(a) The water-cement ratio should be as low as possible consistent with adequate workability. The fluidity of the grout shall be measured in accordance with CRD-C 79.

(b) The efflux time at the pump discharge shall not be less than 11 sec at zero quiescent time and shall not increase less than 3 sec nor more than 8 sec at 20 min of quiescent time.

NOTE: Quiescent time is defined as the amount of time that a sample of grout remains undisturbed (quiescent) in the flow cone and is expressed in minutes. Efflux time is the amount of time that a sample of grout requires to run out of the flow cone after the plug is pulled, expressed in seconds, and should be measured by a stopwatch.

(c) The bleeding of the grout at 65 F should not exceed 2% of the volume 3 hr after mixing or a maximum of 4%. In addition, the separated water must be absorbed after 24 hr. Bleeding should be measured in a metal or glass cylinder with an internal diameter of approximately 4 in. and with a height of grout of approximately 4 in. During the test, the container should be covered to prevent evaporation.

(d) The design of the mix shall be such as to result in a minimum compressive strength of the grout of 2500 psi after 28 days when tested in accordance with ASTM C 109.

CC-2243.3 Chemical Requirements. The chloride and nitrate content of the grout constituents shall be controlled so that the following limits for freshly mixed grout are not exceeded:

(a) chlorides as Cl^- —300 ppm maximum

(b) nitrates as NO_3^- —100 ppm maximum

CC-2250 MARKING AND IDENTIFICATION OF CONCRETE MATERIAL

CC-2251 Cement

Conveyances of bulk cement shall be sealed and tagged before leaving the place of manufacture, showing lot number, controlling specification, date of manufacture, and type. If bag cement is used, each shipment shall be tagged with the same identification as for bulk cement. All tags and markings shall be maintained with the material at site storage.

CC-2252 Aggregate

Aggregate shall be identified to show size, source, and controlling specification. The identification shall remain with the aggregate during production, transit, and concrete plant storage.

CC-2253 Admixtures

All containers of admixtures shall be clearly marked, showing storage requirements and controlling specification.

CC-2300 MATERIAL FOR REINFORCING SYSTEMS

CC-2310 INTRODUCTION

(a) The material to be used for reinforcing systems for containments shall conform to ASTM A 615 Grades 40 and 60 and the special requirements described in CC-2330.

(b) The material to be used for bar to bar splice sleeves in reinforcing bars shall conform to ASTM A 513 or A 519.

(c) The material to be used for reinforcing bar splice sleeves attached to liner plates or structural steel shapes shall be carbon steel conforming to ASTM A 513 or A 519 Grades 1008 through 1030.



CC-2320 MATERIAL IDENTIFICATION

Reinforcing systems material shall be tagged or marked to ensure traceability to the Certified Material Test Report during production and while in transit and storage.

CC-2330 SPECIAL MATERIAL TESTING**CC-2331 Tensile Tests**

CC-2331.1 Number of Tests Required. One full-diameter tensile test bar from each bar size shall be tested for each 50 tons or fraction thereof of reinforcing bars produced from each heat of steel. The tensile test procedures shall be in accordance with SA-370.

CC-2331.2 Acceptance Standards. The acceptance standards shall be in conformance with the tensile requirements of Table 2 of ASTM A 615. If a test specimen fails to meet the tensile requirements of ASTM A 615, two additional specimens from the same heat and of the same bar size shall be tested. If either of the two additional specimens fails to meet the tensile requirements, the material represented by the tests shall not be accepted.

CC-2332 Bend Tests

CC-2332.1 Number of Tests Required and Test Procedures

(a) Bend tests on bar sizes No. 3 through No. 11 shall be performed in accordance with ASTM A 615.

(b) If it is intended that bars of sizes No. 14 and No. 18 will be bent during construction, bend testing of bars from such heats of material shall be conducted at the rate of one test bar for each bar size from each heat. The bend tests shall be conducted at a temperature of at least 60 F. Full-section test bars shall be bent in the unmachined condition over a ten-bar-diameter pin through 90 deg.

CC-2332.2 Acceptance Standards

(a) The acceptance standards for bar sizes No. 3 through No. 11 shall be in conformance with the requirements of Section 8.1 of ASTM A 615.

(b) For bar sizes No. 14 and No. 18, the acceptance standards shall be an absence of cracks on the outside of the bend portion. Material from heats meeting this criterion may be bent during construction.

CC-2333 Chemical Analysis

(a) A ladle analysis of each heat of reinforcing bar shall be made and reported to the purchaser.

(b) The ladle analysis of reinforcing bar heats intended for arc-welding shall be as follows:

carbon: 0.30% maximum

manganese: 1.50% maximum

sulphur: 0.05% maximum

phosphorus: 0.05% maximum

An analysis for the following residual elements shall also be performed and reported: copper, nickel, chromium, molybdenum, and vanadium. The carbon equivalent of such bars shall comply with the 0.55 maximum as computed in CC-4334.6.

(c) Check analysis tolerances for carbon, manganese, phosphorus, and sulphur shall be in accordance with ASTM A 29.

CC-2400 MATERIAL FOR PRESTRESSING SYSTEMS**CC-2410 INTRODUCTION**

This Subarticle establishes the requirements for the material to be used for bonded and unbonded containment prestressing systems.

CC-2420 PRESTRESSING STEEL**CC-2421 Permitted Material**

Prestressing wire, strand, and bar materials are limited to those listed in Appendix I, Table I-1.2. The materials shall conform to their respective material specifications and to the additional requirements described in the following subparagraphs.

CC-2422 Test Specimen Sizes

All mechanical tests on prestressing wire, strand, and bar shall be performed on full-diameter test pieces.

CC-2423 Tensile Tests

Material produced to an ASTM specification shall be sampled and tested as required by that specification. Material not produced to an ASTM specification shall be sampled and tested at the rate of one test for every 20 tons or fraction thereof produced from each heat of steel. The tensile strength, yield strength, elongation, and other pertinent data shall be reported on the Certified Material Test Report.



CC-2424 Stress Relaxation Properties

The stress relaxation properties of the wire, strand, or bar, in accordance with ASTM E 328, "Standard Recommended Practice for Stress Relaxation Tests for Materials and Structures," shall be provided by the Material Manufacturer. Reports relating thereto shall be based upon tests performed on material previously manufactured to the same ASTM or other applicable specification, and produced in the same plant utilizing the same procedures that will be employed to produce the wire, strand, or bar for the production tendons.

CC-2424.1 Data To Be Furnished. The following data shall be furnished:

- (a) detailed test method
- (b) initial stress
- (c) final stress
- (d) test time
- (e) temperature limits
- (f) mathematical tools used to interpret test results
- (g) percentage stress relaxation properties for design life

CC-2424.2 Number of Tests. A minimum of three relaxation tests of 1,000 hr duration shall be performed and reported to document adequately that the relaxation losses are in compliance with the Construction Specification.

**CC-2430 ANCHORAGE COMPONENTS
(Including Bearing Plates)****CC-2431 Permitted Material**

Specific materials for anchorage components including bearing plates for prestressing systems are not given in this Code. The tendon Manufacturer is allowed to use any material he feels is compatible with his tendon system, taking into account the loading conditions that the tendons will undergo during testing and in service. It is a requirement, however, that the tendon Manufacturer performs the tests prescribed in CC-2460 and accordingly documents his anchorage component materials. The following subparagraphs describe the basic material tests the tendon Manufacturer must perform on the material he selects and uses for production tendon anchorage components.

CC-2432 Bearing Plates

Materials for bearing plates shall conform to their respective material specifications. The dimensions,

finish, alignment, and tolerances of the bearing plates shall be within the limits set forth in the Construction Specification.

CC-2433 Anchorhead Assemblies and Wedge Blocks

CC-2433.1 General. Materials for anchorhead assemblies and wedge blocks shall conform to their respective material specifications and the hardness test requirements of CC-2433.2. The dimensions, finish, alignment, and tolerances of the anchorhead assemblies and wedge blocks shall be within the limits set forth in the Construction Specification.

CC-2433.2 Hardness Tests

CC-2433.2.1 Scope and Extent of Testing. Rockwell or Brinell hardness tests shall be conducted on 10% of the parts from each lot. If the material is heat treated, the measurements need be made only after heat treatment. The 10% sample shall be taken at random.

CC-2433.2.2 Test Procedures. Each test shall consist of taking three measurements on the outer surface of the parts. The test procedures shall conform to ASTM E 18 for Rockwell hardness testing and to ASTM E 10 for Brinell hardness testing.

CC-2433.2.3 Acceptance Standards. The results of all measurements shall be within the limits set forth in the construction procedures.

CC-2433.2.4 Retests. If the hardness requirement is not met by any single part, then all parts from the lot shall be tested and only those parts meeting the requirement shall be used.

CC-2434 Wedges and Anchor Nuts

CC-2434.1 General. Materials for wedges and anchor nuts shall conform to their respective material specifications and the hardness test requirements of CC-2434.2. The dimensions, finish, alignment, and tolerances of the wedges and anchor nuts shall be within the limits set forth in the Construction Specification.

CC-2434.2 Hardness Tests

CC-2434.2.1 Scope and Extent of Testing. Core hardness, surface hardness, and, in the case of wedges, case depth tests shall be conducted on 5% of each heat treatment lot after heat treatment. The 5% sample shall be taken at random.



CC-2434.2.2 Test Procedures. The test procedures shall be the same as stated in CC-2433.2.2.

CC-2434.2.3 Acceptance Standards. The results of all measurements shall be within the limits set down in the construction procedures. Lots not meeting the requirements shall be rejected.

CC-2440 NONLOAD-CARRYING AND ACCESSORY MATERIALS

CC-2441 Tendon Ducts, Trumpets, and Transition Cones

Tendon ducts, trumpets, and transition cones shall have the following properties:

(a) Material shall be ferrous and shall not cause harmful electrolytic action.

(b) Ducts shall be strong enough to retain their shape and resist irreparable damage during containment construction.

(c) Joints in the ducts shall prevent intrusion of cement paste from the concrete.

(d) When used for single wire, strand, or bar tendons, the inside diameter of ducts shall be at least $\frac{1}{4}$ in. larger than the nominal diameter of the tendon.

(e) When used for multiple wire, strand, or bar tendons, the inside cross-sectional area of ducts shall be at least twice the nominal net area of the enclosed tendon.

(f) Ducts shall have openings at both ends for filler injection.

(g) Ducts shall be capable of withstanding an internal pressure greater than the initial pumping pressure when adequately backed by concrete.

CC-2442 Corrosion Prevention Material

CC-2442.1 Introduction. This subparagraph describes the requirements for material to be used for the temporary and permanent corrosion prevention of prestressing systems.

CC-2442.2 Temporary Coatings

CC-2442.2.1 Bonded Systems. Temporary corrosion prevention for bonded tendons shall be provided by environmental control. If any temporary corrosion preventive coating is used, it shall allow the portland cement grout to bond uniformly to all parts of the tendon steel. The temporary coating shall be specified in the Construction Specification.

CC-2442.2.2 Unbonded Systems. A temporary corrosion prevention coating shall be applied to all unbonded tendons during or after fabrication. The

coating material shall be compatible with the permanent corrosion preventing coating described in CC-2442.3.2. The temporary coating shall be specified in the Construction Specification. It shall be made to be easily removable in the field with the use of nonchlorinated petroleum solvents for the installation of field-attached anchorages.

CC-2442.3 Permanent Coatings

CC-2442.3.1 Bonded Systems. The permanent corrosion prevention material for bonded tendons shall be portland cement grout in accordance with CC-2243.

CC-2442.3.2 Unbonded Systems

(a) *Permissible Types of Coatings.* The permanent corrosion prevention coating applied to tendons shall be a petrolatum or microcrystalline wax base material containing additives to enhance the corrosion inhibiting and wetting properties as well as to form a chemical bond with the tendon steel.

(b) *Properties of the Coating.* The coating Manufacturer shall provide test data verifying that the following properties are met for the service life and temperature for evaluation and acceptance by the Designer:

(1) freedom from cracking and brittleness

(2) continuous self-healing film over the coated surfaces

(3) chemical and physical stability

(4) nonreactivity with the surrounding and adjacent materials such as concrete, tendons, and ducts

(5) moisture displacing characteristics

(c) *Chemical Analysis of the Coating.* Each batch of coating material shall be analyzed for the presence of water-soluble chlorides, nitrates, and sulphides. The analysis shall conform to the limits shown in Table CC-2442-1. The method for obtaining the water extraction shall be approved by the Designer.

CC-2450 PERFORMANCE REQUIREMENTS

CC-2451 Anchorages and Couplings

Anchorages and couplings shall be designed to assure that the mode of failure of the tendon system will be a rupture of the prestressing steel. Each individual element of an anchorage or coupling shall be capable of developing at least 110% of the specified minimum ultimate tensile strength of the corresponding prestressing steel element of elements being anchored or coupled. This requirement shall



TABLE CC-2442-1
ANALYSIS LIMITS OF PERMANENT
COATING MATERIAL

Compound	Max. Quantity, ppm	Analysis Method
Water-soluble chlorides, Cl^-	10	ASTM D 512 (Limit of accuracy, 0.5 ppm)
Water-soluble nitrates, NO_3^-	10	ASTM D 992 (Limit of accuracy, 0.5 ppm)
Water-soluble sulphides, S^-	10	APHA-Test Methods Sulphides in Water (Limit of accuracy, 1.0 ppm)

apply to the most adverse combination of tolerances set forth in the Construction Specification.

CC-2452 Tendon Assemblies

CC-2452.1 Bonded Tendons

CC-2452.1.1 Anchorages. A full-capacity straight tendon, complete with anchorages, shall develop an ultimate strength equal to at least 95% of the minimum specified ultimate tensile strength of the prestressing steel in the unbonded condition without exceeding the anticipated set of the anchorage elements. In a bonded condition, the anchorage shall perform in compliance with CC-2442.2.1. An adequate bond length shall be provided for in the design of the containment.

CC-2452.1.2 Couplings. Tendons with couplings shall conform with the requirements of CC-2452.2.2.

CC-2452.2 Unbonded Tendons

CC-2452.2.1 Anchorages. A full-capacity straight tendon, complete with anchorages, shall develop an ultimate strength equal to at least 100% of the minimum specified ultimate tensile strength of the prestressing steel without exceeding the anticipated set of the anchorage elements. The total elongation under ultimate load of the tendon shall not be less than 2% measured in a minimum gauge length of 100 in.

CC-2452.2.2 Couplings. Tendons with couplings shall conform with the requirements of CC-2452.2.1. Couplings shall be enclosed in housings long enough to permit the necessary movements during tensioning.

CC-2460 PERFORMANCE TESTS

CC-2461 General Requirements

W75

A series of performance tests shall be conducted to qualify the tendon system for use in the concrete containment prestressing system. The required tests are designed to demonstrate that the combination of materials for the tendon system is adequate and to assess the overall strength and integrity of the tendon system.

CC-2462 Material To Be Used for Performance Tests

The material to be used for the performance test tendons shall be that which the tendon Manufacturer proposes using for production tendons. All of the actual materials used and the necessary dimensions shall be documented on a form the same as or similar to that shown in Fig. CC-2462-1.

CC-2463 Type and Number of Performance Tests

CC-2463.1 Static Tensile Test. One static tensile test shall be conducted to destruction so that the following information may be obtained:

- (a) yield strength (or proof stress)
- (b) ultimate tensile strength
- (c) elongation (over 100 in. minimum gauge length)
- (d) number of failed wires or strands

The results shall comply with the requirements of CC-2462.

CC-2463.2 High Cycle Dynamic Tensile Test. One high cycle dynamic tensile test shall be conducted so that the tendon shall withstand, without failure, 500,000 cycles of stress variation from 60% to 66% of the minimum specified ultimate tensile strength of the tendon. One loading cycle is defined as an increase from the lower load to the higher load and return.

CC-2463.3 Low Cycle Dynamic Tensile Test. One low cycle dynamic tensile test shall be conducted so that the tendon shall withstand, without failure, 50 cycles of stress variation from 40% to 80% of the minimum specified ultimate tensile strength of the tendon. One cycle is defined as an increase from the lower load to the higher load and return.

NOTE: It is considered satisfactory to use the same test specimen for performing the tests prescribed in both CC-2463.2 and CC-2463.3.



Specification No. _____	Date _____
Prestressing System Type _____ (wire, strand, or bar)	Single or Multiple Element _____
	If Multiple, No. of Elements _____
Prestressing Element Material _____	Nominal Size _____
Bearing Plate Material _____	Thickness _____ Diameter _____
Wedge Material _____ (Attach diagram showing shape and size)	Hardness _____
Buttonhead Shape and Size (Attach diagram)	
Swaged Fitting Material _____ (Attach diagram showing shape and size)	Hardness _____
Screwed Fitting Material _____ (Attach diagram showing shape and size)	Hardness _____

FIG. CC-2462-1 MANUFACTURER'S RECORD OF TENDON PERFORMANCE QUALIFICATION TESTS

CC-2464 Size of Performance Test Specimens

CC-2464.1 Length. The test specimens shall be long enough so that measurements may be taken on a 100 in. minimum gauge length.

CC-2464.2 Number of Prestressing Elements³

CC-2464.2.1 Single Element Tendons. A single element specimen shall be used for both static and dynamic tensile tests.

CC-2464.2.2 Multiple Element Tendons

(a) **Static Tensile Tests.** All static tensile tests shall be performed on specimens having the full anchorage capacity of the proposed tendons.

(b) **Dynamic Tensile Tests.** All dynamic tensile tests shall be performed on specimens having at least 10% of the full-size prestressing steel area of the proposed production tendons.

CC-2465 Test Results

The measurements and results obtained from the tests shall be documented on a form the same as or similar to that shown in Fig. CC-2465-1.

CC-2466 Essential Variables

CC-2466.1 General Requirements. The performance tests must be completely reconducted when any of the applicable changes listed below are made to the tendon system material. Changes other than those listed below may be made in the tendon system material without the necessity for repeating the performance tests.

³The term *prestressing element* is defined as an individual wire, strand, or bar, whether in a multiple or single wire, strand, or bar system.

CC-2466.2 Essential Variables in Prestressing Elements. Essential variables in prestressing element materials shall be as stipulated in (a) through (e) below:

(a) a change in the prestressing element material from one ASTM or ASME specification to another

(b) a change in the tensile grade of the prestressing element material within the same ASTM or ASME specification

(c) a change in the heat treatment condition of the prestressing element material

(d) a change in the prestressing element diameter

(e) in multiple element systems, an increase of more than 10% in the number of elements in the tendon

CC-2466.3 Essential Variables in Anchorage Items. Essential variables in anchorage items shall be as stipulated in (a) through (f) below:

(a) for buttonhead anchorage systems, a change in the shape or dimensions of the buttonhead

(b) for wedge anchorage systems, a change in the shape, size, or dimensions of the wedge

(c) for threaded nut anchorage systems, a change in the nut or thread size

(d) for swaged systems, a change in the shape or size of the swaged fitting

(e) a change in the anchor block material from one type to another

(f) a change in the bearing plate material from one P-Number to another P-Number from Section IX or any other material

CC-2470 MARKING AND IDENTIFICATION OF PRESTRESSING MATERIAL

Prestressing system material shall be marked or tagged in such a manner as to ensure traceability to

ATTACHMENT 4

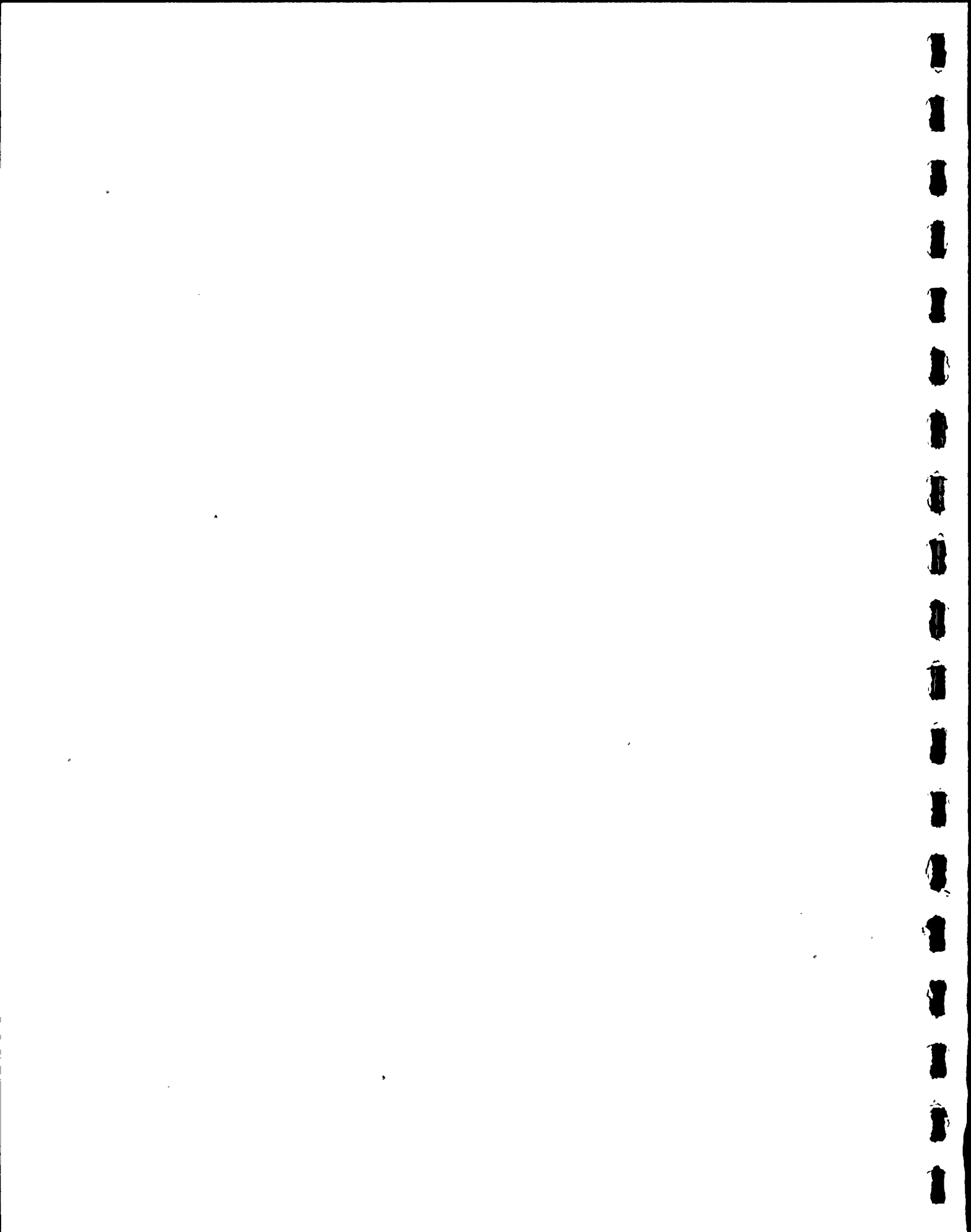
SUMMARY OF STRESSING OPERATIONS (1000 HOUR RETENSIONING)

SUMMARY OF STRESSING OPERATION

Hole # % Tendon Mark	ORIGINAL TENSIONING			1,000. HOUR RETENSION				
	Gauge @ Transfer 1	Gauge @ Overstress 2	Elongation Over a 1,000 PSI 3	Gauge @ Lift-off From Shims 4	% Difference Column 1 & 4 5	Gauge @ Overstress 6	Elongation Over a 1,000 PSI 7	Gauge @ Transfer 8
111 A2	5,800	6,400	7-3/8"	5,150	11.206%	6,200	7-7/16"	5,750
122 A2	5,700	6,400	7-9/16"	5,300	7.017%	6,400	7-7/16"	6,000
119 EN	5,700	6,500	6-5/16"	5,600	1.754%	6,400	6-5/16"	5,700
114 EN	5,500	6,300	6-7/8"	5,250	4.545%	6,350	6-7/8"	5,850
112 A1	5,650	6,400	7-1/8"	5,350	5.309%	6,250	7-11/16"	5,800
121 A1	5,800	6,400	7-1/8"	5,200	10.344%	6,500	7-5/16"	6,000
120 DM	5,600	6,300	7"	5,200	7.142%	6,400	6-15/16"	5,900
113 DM	5,600	6,250	7"	5,100	8.928%	6,300	7-1/4"	5,700
115 FIRI	5,650	6,300	6-3/4"	5,300	6.194%	6,500	6-5/16"	5,850
118 FIRI	5,550	6,300	6-13/16"	5,300	4.504%	6,500	6-5/16"	5,900
116 FR	5,700	6,400	6-5/8"	5,100	10.526%	6,500	6-9/16"	5,850
117 FR	5,720	6,500	6-7/16"	5,400	5.594%	6,500	5-15/16"	5,800
32 A3	5,650	6,250	7-5/8"	5,200	7.964%	6,400	7-15/16"	6,000
39 A3	5,800	6,250	7-5/8"	5,200	10.344%	6,300	8-1/8"	6,000
34 HITI	5,650	6,500	7"	5,200	7.964%	6,500	7-1/4"	5,700
37 HITI	5,500	6,400	7-1/8"	5,000	9.09%	6,500	7-11/16"	5,900

SUMMARY OF STRESSING OPERATION

Hole # % Tendon Mark	ORIGINAL TENSIONING			1,000 HOUR RETENSION				
	Gauge @ Transfer 1	Gauge @ Overstress 2	Elongation Over a 1,000 PSI 3	Gauge @ Lift-off From Shims 4	% Difference Column 1 & 4 5	Gauge @ Overstress 6	Elongation Over a 1,000 PSI 7	Gauge @ Transfer 8
33 35 GS	6,000	6,450	7-1/16"	5,400	10%	6,500	7-13/16"	5,950
38 GS	5,800	6,400	7-7/16"	5,200	10.344%	6,450	7-13/16"	5,900
35 HT	5,750	6,500	6-13/16"	5,200	9.565%	6,500	7"	5,800
36 HT	5,500	6,400	6-7/8"	4,950	10%	6,500	6-15/16"	5,900
126 C2	-	-	-	5,300	-	6,500	6-3/16"	5,900
56 C2	5,600	6,560	7-1/8"	4,850	13.392%	6,500	7-1/16"	6,000
73 B	-	-	-	5,400	-	6,500	6-7/8"	5,600



ATTACHMENT 5

RG&E INTERIM REPORT





Gilbert/Commonwealth engineers and consultants

GILBERT ASSOCIATES, INC., P. O. Box 1498, Reading, PA 19603/Tel. 215 775-2600/Cable Gilasoc/Telex 836-431

September 20, 1979

Rochester Gas & Electric Corporation
89 East Avenue
Rochester, New York 14649

Attention: Mr. A. G. Goetz

Subject: Interim Report and Recommendations
For Tendon Lift Off Test

Dear Mr. Goetz:

Based on our evaluation of the lift off tests for the prestressing tendons at Ginna, we have the following suggestions for the lift off tests scheduled for October 8:

- (1) Lift off the 14 tendons tested at the last test using both the procedure used for that test and any revised procedure which may be recommended by GAI and Inryco prior to the test.
- (2) In addition, perform lift off tests for tendons 60, 63, 150, and 159 to achieve an improved test sample relative to location, original stressing sequence, and access to rock anchor stressing washers.
- (3) If possible, record the movement of the rock anchor stressing washer, if any, for 150 and 159 during lift off. Such observation would be at the observation ports at elevation 227'-8". Prior to performing the lift off tests on these two tendons, record the existing location of the rock anchor stressing washers relative to the bearing plates and the shims. Record the number and size of shims. Record the gap, if any, between the washers and shims.
- (4) Lift off tests on tendons 51, 83, 100, and 132 would also improve the test sample distribution but are not absolutely necessary.
- (5) Detension 84 and 150 (after lift off test) completely to check for broken wires. Tendon 150 will indicate effect of detensioning on rock anchors, 84 checks tendon with lowest force based on previous lift off tests. Record the detensioning displacement at the top ends of each tendon. In addition, for tendon 150, record the displacement of the rock anchor stressing washer, if any.

On the basis of the data we have received, it is our opinion that the average force level in the containment tendons may presently be below the level required by the Technical Specification and that all tendons should be retensioned at the first available opportunity. Although a few of the values appear to be low due to recording and procedural errors, it does not appear that there will be



Rochester Gas & Electric Corporation
Attention: Mr. A. G. Goetz
September 20, 1979
Page 2

sufficient improvement in the overall average to delay retensioning for an extended period of time. The retensioning would require approximately 1 3/4" to 2" additional shims for each tendon.

The rate that the tension losses are occurring appears to currently indicate a performance that is normal. However, there appears to have been an initial loss for which we have no exact explanation. This loss appears to have occurred after the original tensioning, and before the 1000 hour retensioning. It has, however, in all cases been followed by what we believe to be normal loss behavior.

To verify that the structure will respond as predicted after the retensioning, we would suggest that 6 months and 1 year after retensioning, 14 tendons be tested for lift off. If these tests indicate a return to a normal tension loss pattern a normal lift off test sequence could be utilized thereafter.

As we have previously stated, the cause of the initial losses is not clear. We will include in our report of this phase of the work a summary of our current evaluations, the backup data to support our conclusions, and an estimate of the time and costs necessary to investigate the several potential sources of the initial prestress loss. This report will be available by October 1, 1979.

Very truly yours,



P. M. Cameron
Senior Project Manager

PMC:bmk

cc: J. E. Arthur
R. E. Smith
J. N. Covey
K. T. Momose
F. L. Moreadith
R. E. Pages
F. X. Rehill



ATTACHMENT 6

ROCK ANCHOR LITERATURE SEARCH RESULTS

ID NO.- EI790751274 951274

UPLIFT CAPACITY OF ROCK ANCHOR GROUPS.

Ismael, N. F.; Radhakrishna, H. S.; Klym, T. W.

Ont Hydro, Toronto

IEEE Power Eng Soc, Winter Meet, Prepr, New York, NY, Feb 4-9 1979 Publ for the IEEE Power Eng Soc, by IEEE, New York, NY, 1979 Pap F.79 272-6, 7 p

The uplift capacity of rock anchor groups was examined by full-scale field tests in different rock formations. The results are analyzed and the different modes of failure are discussed in view of the test results with emphasis on the rock pull-up failure mechanism. Important design parameters are summarized for the convenient use of the engineer in the design of transmission tower foundations anchored in bedrock. 12 refs.

DESCRIPTORS: (*ELECTRIC LINES, *Towers), (FOUNDATIONS, Anchorages),

IDENTIFIERS: ROCK ANCHORS, ANCHORAGE UPLIFT CAPACITY

CARD ALERT: 706, 408, 405

ID NO.- EI790533584 933584

RECENT APPLICATIONS OF PRESTRESSED ROCK & SOIL ANCHORS.

Drew, George E.

VSL Corp, Los Gatos, Calif

Proc Annu Eng Geol Soils Eng Symp 16th, Boise, Idaho, Apr 5-7 1978. Publ by Idaho Transp Dep, Div of Highw, Boise, 1978 p 23-35 CODEN: EGSSBT

The developing requirement to design economical foundation and retaining structures to allow the utilization of sites heretofore considered unacceptable, has forced engineers and owners to look for new and more efficient design and construction methods. In addition, a need to strengthen and refurbish existing facilities has forced the soils engineer to develop special techniques to allow the repair program to be conducted without disabling the structure's function. This paper presents, with illustrations, recent applications of prestressed rock and soil anchors to new foundation and retaining structures, as well as to existing structures.

DESCRIPTORS: (*FOUNDATIONS, *Anchorages), ROCK, SOILS, RETAINING WALLS,

IDENTIFIERS: PRESTRESSED ANCHORS, ROCK ANCHORS, SOIL ANCHORS

CARD ALERT: 405, 483

ID NO.- EI790428166 928166

ROCK ANCHOR SYSTEM FOR SECURING FAILING WALL

Nicholson, Peter J.

Nicholson Anchorage Co, Bridgeville, Pa

ASCE Transp Eng J v 105 n 2 Mar 1979 p 199-209 CODEN: TPEJAN

A description of the design, installation, and testing of 132 permanent rock anchors used by the Pennsylvania Department

of Transportation to save a failing concrete retaining wall. Design parameters of the three level rock tieback system are examined along with installation techniques and special corrosion protection used to make the system permanent. Grouting techniques and grout mixes are examined. Recommendations are made against the use of admixtures in the grout. Load testing and load cell monitoring of the anchors show some initial movements but achieve stability in a short period of time. 3 refs.

DESCRIPTORS: (*RETAINING WALLS, *Design), (FOUNDATIONS, Anchorages), GROUTING,

IDENTIFIERS: ROCK ANCHORS

CARD ALERT: 405

ID NO.- EI790105182 905182

DETERMINATION OF ROCK SQUEEZE POTENTIAL FOR UNDERGROUND POWER PROJECTS.

Lee, C. F.; Klym, T. W.

Ont Hydro, Toronto

Eng Geol v 12 n 2 Jul 1978 p 181-192 CODEN: EGGOAO

Post-excavation inward movements in the order of 7 cm have been recorded in the turbine pits of some of the hydroelectric power plants in the Niagara area, often resulting in an excessive build-up of loads on the supporting structures and hence considerable difficulties in maintaining operation. It is suggested that the time rate of Sleft double quote\$ swell \$right double quote\$ measured on fresh cores be used as an index in assessing the liability of underground structures to this adverse effect of rock squeeze. The experimental aspects of determining such an index are discussed in this paper, along with their application to the design of underground structures in Sleft double quote\$ squeezing \$right double quote\$ rock. 10 refs.

DESCRIPTORS: *ROCK MECHANICS, (TUNNELS AND TUNNELING, Stresses), HYDROELECTRIC POWER PLANTS,

IDENTIFIERS: ROCK SQUEEZE POTENTIAL

CARD ALERT: 502, 483, 611, 401



ID NO.- EI790105175 905175-

REGIONAL DISTRIBUTION OF IN SITU HORIZONTAL STRESSES IN ROCKS OF SOUTHERN ONTARIO.

Lo, K. Y.

Univ of West Ont, London

Can Geotech J v 15 n 3 Aug 1978 p 371-381 CODEN: CGJOAH

Some structures built in different rock formations in Southern Ontario have been subjected to various degrees of distress. These case histories include heaves of quarry bottoms, buckling of concrete lining of canal floors, cracking of concrete lining of tunnels at the springline, and long term movement of the walls of unsupported excavations. Inference from these case histories, together with direct measurements of in situ stresses, indicate that high horizontal stresses exist in the Silurian and ordovician rocks. The magnitude of the maximum stress in the horizontal plane varies from 6-14 MPa depending on the depth and rock type. Excavations in rock relieve the in situ stresses. The stress relief serves as an initiating mechanism for time-dependent deformation to occur leading to the process loosely termed as 'rock squeeze'. It appears, therefore, that due consideration must be given to this prevalent phenomenon for the design of underground structures in rock in this region. Refs.

DESCRIPTORS: (*ROCK, -Stresses), GEOLOGY,

IDENTIFIERS: ROCK SQUEEZE

CARD ALERT: 483, 421, 481

ID NO.- EI780960916 868916

RETAINING WALLS: TAKING IT FROM THE TOP.

Hunt, Roy E.; da Costa Nunes, Antonio J.

Tecnosolo, Rio de Janeiro, Braz

Civ Eng (New York) v 48 n 5 May 1978 p 73-75 CODEN: CIEGAG

Fully anchored curtain walls have been used increasingly to retain slopes in Brazil where topographic, climatic, and geologic conditions are extremely favorable to landsliding and mass wasting. The wall consists of a thin slab of reinforced concrete tied-back into the hillside with earth or rock anchors. Anchored curtain walls provide high retention capacity and can be anchored at as many points as desirable. The method has been used successfully to construct walls over 25-m high. In Brazil, these walls are constructed from the top downward, so the slope can be cut and continuously retained. For foundation excavations, shoring and bracing are not required and the excavation remains free of obstructions.

DESCRIPTORS: *RETAINING WALLS, (FOUNDATIONS, Anchorages), SOIL MECHANICS,

IDENTIFIERS: SLOPE STABILIZATION

CARD ALERT: 405, 406, 483, 931

Scott, James J.

Univ of Mo-Rolla

Proc Symp Rock Mech 18th; Keystone, Colo, Jun 22-24 1977. Publ by Colo Sch of Mines Press, Golden, 1977 v 1 p 3A2. 1-3A2. 7 CODEN: PSRMA6

Friction Rock Stabilizers, trade name Split Sets, have been under research and development by the author and the Ingersoll-Rand Company for the past 4 1/2 years. This patented rock anchor system has been extensively tested in both hard rock and soft rock and has been reported upon by the author in other publications. This paper deals with results obtained from field testing under soft rock conditions in uranium mines of the western United States. 6 refs.

DESCRIPTORS: (*URANIUM MINES AND MINING, *Roof Control), ROCK MECHANICS,

IDENTIFIERS: FRICTION ROCK STABILIZERS, SPLIT SETS

CARD ALERT: 504, 502, 483

ID NO.- EI780750572 850572

APPLICATIONS OF PRESTRESSING TECHNIQUES TO HYDROELECTRIC PROJECTS.

Drew, George E.

VSL Corp, Los Gatos, Calif

Proc Symp Rock Mech 18th; Keystone, Colo, Jun 22-24 1977. Publ by Colo Sch of Mines Press, Golden, 1977 v 1 p 3A3. 1-3A3. 7 CODEN: PSRMA6

Engineers and scientists involved in the field of rock mechanics have a special appreciation for prestressing techniques, since it is through these techniques that the properties of the rock can be used to develop economical solutions to problems which are difficult, and extremely costly, to solve by other methods. This paper discusses a number of prestressing techniques recently applied to hydroelectric facilities. The following major techniques are covered: (1) Repair and stabilization of existing dams; (2) Power house and dam abutment rock anchoring techniques; and (3) Long-term monitoring of rock anchors.

DESCRIPTORS: (*HYDROELECTRIC POWER PLANTS, *Concrete Construction), ROCK MECHANICS, (DAMS, CONCRETE, Stability)

CARD ALERT: 483, 611, 405, 441

ID NO.- EI780754510 854510

FRICTION ROCK STABILIZERS IN URANIUM MINING.



ID NO.- EI780646078 846078
USE OF ROCK ANCHORS AT TARBELA.

Anon
Int Water Power Dam Constr v 30 n 2 Feb 1978 p 44-47.
CODEN: IWPCDM

The repair work to the floor of stilling basin number three at Tarbela, Pakistan involved the use of almost 600 prestressed rock anchors, fabricated and assembled on site. The way in which the work was organized, and the ways in which the anchors were installed are described.

DESCRIPTORS: (*STILLING BASINS, *Maintenance), (CONCRETE CONSTRUCTION, Anchorages),
IDENTIFIERS: ROCK ANCHORS
CARD ALERT: 405, 441

ID NO.- EI780041082 841662
TIME-DEPENDENT RESPONSE OF A DEEP RIGID ANCHOR IN A VISCOELASTIC MEDIUM.

Selvadurai, A. P. S.
Carleton Univ, Ottawa, Ont
Int J Rock Mech Min Sci Geomech Abstr v 15 n 1 Feb 1978 p 11-19 .CODEN: IRMGBG

The currently available techniques for the analysis and design of both ground and rock anchors largely concentrate upon the estimation of the ultimate bearing capacity of shallow and deep anchors, taking into account their individual or group action. Relatively little consideration has been given to the examination of time-dependent effects in the anchor problem. Such time-dependent effects are of fundamental importance to the determination of the anchor efficiency under long term conditions and in calculation of creep deformations of anchorages under sustained loads. This paper is primarily concerned with the analytical treatment of a deep-anchor region embedded in a soil or rock medium exhibiting creep properties. The analysis is restricted to the class of soil and rock materials whose behavior can be adequately described by a linear viscoelasticity theory. The linear theory of viscoelasticity, like its elastic counterpart, provides only an approximate theoretical basis for the analysis of creep phenomena in soil and rock media. The loss of load in an anchor is influenced by the geometric shape of the anchor region and the shear relaxation behavior of the viscoelastic material. 43 refs.

DESCRIPTORS: (*FOUNDATIONS, *Anchorages), (ROCK, Elasticity) ROCK MECHANICS, SOIL MECHANICS, (MATHEMATICAL TECHNIQUES, Boundary Value Problems), (GEOPHYSICS, Rock Properties),
IDENTIFIERS: ROCK ANCHORS, TIME DEPENDENCE
CARD ALERT: 405, 408, 421, 483, 505

ID NO.- EI780425478 825478
UPLIFT CAPACITY OF FOOTINGS AND ANCHORS FOR TRANSMISSION TOWERS.

Adams, J. I.; Radhakrishna, H. S.
Ont Hydro Res Q v 29 n 1 First Quarter 1977 p 7-20 CODEN: OHRQAW

Research and development by Ontario Hydro as regards the uplift capacity of footings and anchors is reviewed. The work centered largely on augered concrete footings, and on grouted soil anchors, helix anchors and rock anchors. Approximate theories developed from full-scale tests to evaluate the ultimate uplift capacity of the foundation units are described. The uplift capacity of footings and anchors in clay was shown to be much lower under sustained loading than under short-term loading, depending on depth. Grouped anchors in clay showed only a slight decrease in efficiency whereas for rock anchors at fairly shallow depth the loss in efficiency was high. The modes of failure of single and grouped anchors grouted in soft and medium-strength rocks are discussed. 18 refs.

DESCRIPTORS: (*FOUNDATIONS, *Soil Structure Interaction), (DAMS, Uplift Pressure), CLAY, TOWERS,
IDENTIFIERS: FOOTINGS, ANCHORS, TRANSMISSION TOWERS
CARD ALERT: 405, 441, 414, 408

ID NO.- EI780318027 818027
ROCK ANCHORAGE TECHNOLOGY.
Zajic, Josef
Natl Build Geol Corp, Prague, Czech
Batim Int Build Res Pract v 5 n 5 Sep-Oct 1977 p 306-313
CODEN: BINTDO

The main types of anchor, installation techniques and anti-corrosion precautions are described. 12 refs. In English and French.

DESCRIPTORS: (*FOUNDATIONS, *Anchorages), CORROSION PROTECTION,
IDENTIFIERS: ROCK ANCHORAGES
CARD ALERT: 405, 539



AUXILIARY DESCRIPTOR CODES: 140000

GA432000468 0509568
Corrosion protection and lateral displacement
characteristics of rock anchors
FINES, H; SLATER, W; SAGE, R
CANMET Rep (Ottawa)
1978
77/56 28P
CREPO1
LANGUAGE: ENGLISH
DESCRIPTORS: IMPROVEMENT OF ROCK CONDITIONS; OPENCAST-MINING
MINING GEOMECHANICS
DESCRIPTOR CODES: 365300; 346400; 346800

GA428000363 0504455
Determination of rock squeeze potential for underground
power projects
LEE, CF; KLYM, TW
CONF. DATES: 25 AUGUST TO 27 AUGUST 1976 NO: 76-0503
Eng Geol (Amsterdam)
1978
12/2 P181-192
EGEOA1
LANGUAGE: CONFERENCE PROCEEDINGS
DESCRIPTORS: ROCK ENGINEERING PROBLEMS; UNDERGROUND
STRUCTURES
DESCRIPTOR CODES: 365200; 360500

GA377000417 0458967
Application of prestressed rock anchoring techniques to
hydroelectric projects
DREW, GE
18 US Symp Rock Mech (Keystone) CONF. DATES: 22 JUNE TO 24
JUNE NO: 77-0056
LANGUAGE: CONFERENCE PAPER TITLE
DESCRIPTORS: IMPROVEMENT OF ROCK CONDITIONS; HYDROELECTRIC
POWER
DESCRIPTOR CODES: 365300; 395400

GA366004594 0451353
Rock anchors at loch Kishorn
TURNER, MJ
Ground Eng (Brentwood)
1977
10/3 P37-41
GENGII
LANGUAGE: ENGLISH
DESCRIPTORS: FOUNDATION ENGINEERING
AUXILIARY DESCRIPTORS: SCOTLAND
DESCRIPTOR CODES: 360200

GA360004671 0445865
Surface rock creep on sandstone slopes in the Northern
Territory of Australia
WILLIAMS, MAJ
Aust Geogr (Sydney)
1974
12/5 P419-424
AGEOSQ
LANGUAGE: ENGLISH
DESCRIPTORS: TALUS CREEP; SANDSTONES & QUARTZITES; VALLEY &
SLOPE PROFILES
AUXILIARY DESCRIPTORS: NORTHERN TERRITORY
DESCRIPTOR CODES: 749300; 553300; 743400
AUXILIARY DESCRIPTOR CODES: 581000

GA354002602 0439718
A study of rock creep under laboratory conditions
MATVEYEV, BV; KARTASHOV, JM
Proc 3 Congr ISRM (Denver) CONF. DATES: 01 SEPTEMBER TO 07
SEPTE NO: 74-0002
1974
P342-347
LANGUAGE: CONFERENCE PROCEEDINGS
DESCRIPTORS: DEFORMATION; EXPERIMENTAL ROCK MECHANICS
DESCRIPTOR CODES: 731100; 365600

GA316003475 0402205
ROCK ANCHORS - STATE OF THE ART - PART 3 STRESSING AND
TESTING
LITTLEJOHN, GS; BRUCE, DA
Ground Eng (Brentwood)
1976
9/2 P20-29
GENGII
LANGUAGE: ENGLISH
DESCRIPTORS: EXPERIMENTAL ROCK MECHANICS; DEFORMATION;
IMPROVEMENT OF ROCK CONDITIONS
DESCRIPTOR CODES: 365600; 731100; 365300



GA31G002211 0400960

Rock anchors - state of the art, part 3 Stressing and testing

LITTLEJOHN, GS; BRUCE, DA

Ground Eng (Brentwood)

1976

9/3 P55-60

GENGII

LANGUAGE: ENGLISH

DESCRIPTORS: FOUNDATION ENGINEERING

DESCRIPTOR CODES: 360200

GA312003798 0397731

Rock squeeze at Thorold Tunnel (Ontario)

BOWEN, CFP; HEWSON, FI; MACDONALD, DH; TANNER, RG

Can. Geotech. J (Ottawa)

1976

13/2 P111-126

CGJOTI

LANGUAGE: ENGLISH

DESCRIPTORS: ROCK ENGINEERING PROBLEMS; IMPROVEMENT OF ROCK

CONDITIONS; UNDERGROUND STRUCTURES

AUXILIARY DESCRIPTORS: ONTARIO

DESCRIPTOR CODES: 365200; 365300; 360500

AUXILIARY DESCRIPTOR CODES: 745300

GA312001528 0395490

Rock anchors - state-of-the-art - part 3 Stressing and testing

LITTLEJOHN, GS; BRUCE, DA

Ground Eng (Brentwood)

1976

9/4 P33-44

GENGII

LANGUAGE: ENGLISH

DESCRIPTORS: IMPROVEMENT OF ROCK CONDITIONS; EXPERIMENTAL

ROCK MECHANICS

DESCRIPTOR CODES: 365300; 365600



ADDENDUM I
TENDON RETEST

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ADDENDUM I
TENDON RETEST

1.0 INTRODUCTION

On the basis of a recommendation contained in Attachment 5 of the report, a retest was performed on the 14 tendons last surveyed at eight years after initial tensioning, plus 8 added tendons. The purpose of the retest was to:

- 1) Evaluate the accuracy of the 8 year data and confirm predicted results.
- 2) Check the current condition of the tendon wires.
- 3) Check the condition of the rock anchors and confirm their response to changes in tendon force.

2.0 EVALUATIONS

2.1 LIFT-OFF TEST EVALUATION

The results of the retests are shown in Table I-1. A representative of Inryco, who had performed the lift-off test for the initial tensioning and six month surveillance, determined the sonic lift-off point (column 1). Since this method is no longer considered accurate, due to its reliance on individual perceptions, a pair of 1/32 inch shims were also used to determine the lift-off force (column 2). Appendix I-A of this addendum contains the procedure used. The results obtained using the shim removal method are considered the more accurate of the two methods used. The values obtained from this test have been added to Figures 4-1 through 4-42 of the report.



The tendons were also overstressed to approximately 6% above the force required to produce lift-off, and the tendon elongations recorded. The results are presented in Table I-2. The elongations ranged from 3/16 inch to 11/26 inch, and the corresponding tendon force increases ranged from 8.9 to 51 kips. The average tendon force increase was 33.7 kips, and the average elongation was 8/16 inch. The predicted elongation for a straight tendon under the 33.7 kip force is 6/16 inch. Therefore, the tendon elongations appear to be reasonable, and no abnormal tendon behavior is indicated. Differences in elongations (for approximately the same tendon force increase) are attributed to (1) inaccuracy in determining the point of lift-off, which could be significant for the relatively small tendon force increment, and (2) variations in the friction at each tendon.

The retest indicated higher values than the 8 year tests and general agreement with the slopes of the predicted curves. Where several previous tests are available, Figures 4-4, 4-7, 4-24, the retest value appears to be in line with other earlier tests. Of the 22 tendons, only tendons 51 and 63, Figures 4-12 and 4-14, show greater slopes than predicted. The results would indicate that many of the 8 year test values were in error.

The average force in the tested tendons was found to be 634 kips. The predicted average force was 631 kips. The average force based on the technical specification requirement that the tendon stress be at least 144,000 psi is 636 kips. The technical specification also allows for 5% broken wires. If no wires are broken and the allowance were applied to the stress, the minimum required force per tendon would be 604 kips.



2.2

CONDITION OF TENDON WIRES

In order to evaluate the condition of the wires, the following was done:

- 1) The top end of all surveyed tendons was examined for signs of broken tendon wires or corrosion.
- 2) Surveillance wires from three tendons, 51, 76, and 150, were extracted and examined for their full length for corrosion or other signs of deterioration.
- 3) One tendon, No. 150, was completely detensioned and both ends examined.

No signs of either corrosion or damaged wires were found in any case. Experience with vertical tendons on other applications indicates that vertical tendon wires seldom, if ever, break. Even in horizontal applications, where breaks sometime occur, the break normally happens during tensioning or other major changes in force.

2.3

ROCK ANCHOR EVALUATION

The condition and the response of the rock anchor connection to changes in tendon force were observed at tendon 150. This tendon is provided with an access panel to the connection between the anchor and the wall tendon. The access panel was removed as was the tendon grease. The connection was observed to be clean and bright, with no signs of corrosion or water in the chamber. During detensioning and retensioning, no observable movements occurred. On the basis of these observations, it is concluded that the rock anchors are not adversely affecting the performance of the tendons.

CONCLUSIONS

The Technical Specification states that, "The specified acceptance criteria are such as to alert attention to the situation well before the tendon load-carrying capabilities would deteriorate to a point that failure during a design basis accident might be possible. Thus the cause of the incipient deterioration could be evaluated and corrective action studied without need to shut down the reactor." It provides two acceptance criteria, that less than 5% of the total wires be broken and that the stress level in the tendons be above 144,000 psi.

Experience with vertical tendons indicates that tendon wires seldom, if ever, break. It also indicates that when tendon wires break it is usually during tensioning. Visual examination of the tendons coupled with past experience would lead to a high degree of certainty that no broken wires exist, although a 5% allowance for broken tendons is made in the design.

An evaluation of the data from this test and all previous surveillances indicates that although the actual losses exceed those predicted using current techniques (NRC Draft RG 1.35.1), the current rate of loss is in reasonable agreement. The current deficiency in tendon stress of .3% is predicted to increase to .6% in the next year.

Based on:

- 1) Good agreement between predicted and actual rate of loss,
- 2) Lack of any evidence of wire breakage or corrosion,
- 3) The 5% allowance for broken wires in the design, and
- 4) The very minor extent of the variation from the technical specification, currently and in the near future,



it is concluded that the situation does not represent any danger to the public at this time and will not in the near future. It is recommended that either the tendons be retensioned or that further analysis be performed to evaluate the structures' response at lower prestress levels.



TABLE I-1 10 YEAR RETEST LIFT-OFF FORCES

Column--		(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
Tendon Number	Figure Number	Ram Pressure at Lift Off (psi)		Lift-Off Force (kips) Based on Shim Method ^d	Predicted Lift-Off Force (kips)		Actual minus Predicted Lift-Off Force (kips) Col.(3)-Col.(5)	X Difference in Actual minus Predicted Loss Col.(6)/Col.(5)	Lock-Off Force minus Elastic Shortening Loss (kips) - at t=1.4 yrs. from Figs.4	Actual Long-Time Tendon Force Loss (kips) Col.(8)-Col.(3)	X Long-Time Loss Col.(9)/Col.(8)
		Acoustical Method ^a	Shim Method ^a		Table 3 of Report	Base Value at t=12 yrs. from Figs.4					
95	4-13	4950	5000	638	611	675	-37	-5.48	763	125	16.4
84	4-30	4750	4775	609	584	618	- 9	-1.46	706	97	13.7
83	4-11	4775	4810	614	624	641	-27	-4.21	730	116	15.9
76	4-35	4600	4650	593	602	620	-27	-4.35	707	114	16.1
63	4-14	4675	4700	600	630	650	-50	-7.69	738	138	18.7
60	4-19	4700	4700	600	621	621	-21	-3.38	708	108	15.3
53	4-1	4900	4950	632	621	681	-49	-7.20	769	137	17.8
45	4-3	5050	5100	651	634	670	-19	-2.84	758	107	14.1
36(R) ^b	4-7	5050	5200 ^c	664	679	— ^e	— ^e	— ^e	763	99	13.0
17	4-4	4850	4875	622	627	636	-14	-2.20	725	103	14.2
8	4-24	5000	5020	641	655	655	-14	-2.14	743	102	13.7
160	4-33	4975	5000	638	607	638	0	0	726	88	12.1
159	4-10	5175	5200	664	650	674	-10	-1.48	763	99	13.0
150	4-40	5180	5200	664	640	685	-21	-3.07	773	109	14.1
51	4-12	4600	4625	590	636	610	-20	-3.28	698	108	15.5
142	4-18	4800	4810	614	607	628	-14	-2.23	715	101	13.4
133	4-2	5000	5000	638	626	643	- 5	-0.80	731	93	12.7
126(R) ^b	4-31	5400	5450	696	683	— ^e	— ^e	— ^e	763	67	8.8
132	4-25	4810	4850	619	614	635	-16	-2.52	723	104	14.4
117(R) ^b	4-42	5200	5400	689	658	— ^e	— ^e	— ^e	750	61	8.1
110	4-37	5125	5150	657	653	685	-28	-4.09	772	115	14.9
100	4-16	4750	4800	612	610	652	-40	-6.13	741	129	17.4
Ave.				634	631		-22	-3.40			14.2

NOTES TO TABLE I-1:

- a. Refer to Appendix I-A for these two methods of determining the lift-off.
b. (R) denotes tendons which were retensioned at 1000 hrs. after initial stressing.
c. Lift-off pressure is suspect due to bent 1/32" shims.
d. Ram area = 127.6 in².
e. Predicted tendon forces are not applicable for a retensioned tendon.

TABLE I-2 10 YEAR RETEST - TENDON ELONGATIONS

Column →		(1)	(2)	(3)	(4)
Tendon Number	Figure Number	Ram Pressure at Lift Off (psi)	Ram Pressure at 6% Overstress ^b (psi)	Increase in Tendon Force ^a (kips)	Measured Tendon Elongation (inches)
95	4-13	5000	5250	31.9	9/16
84	4-30	4775	5050	35.1	11/16
83	4-11	4810	5060	31.9	8/16
76	4-35	4650	4880	29.4	8/16
63	4-14	4700	5100	51.0	11/16
60	4-19	4700	4980	35.7	9/16
53	4-1	4950	5190	30.6	7/16
45	4-3	5100	5350	31.9	7/16
36(R)	4-7	5200	5350	19.4	3/16
17	4-4	4875	5150	35.1	5/16
8	4-24	5020	5300	35.7	8/16
160	4-33	5000	5275	35.1	11/16
159	4-10	5200	5485	36.4	5/16
150	4-40	5200	5490	37.0	9/16
51	4-12	4625	4875	31.9	8/16
142	4-18	4810	5190	48.5	10/16
133	4-2	5000	5325	41.5	10/16
126(R)	4-31	5450	5725	35.1	8/16
132	4-25	4850	5100	31.9	3/16
117(R)	4-42	5400	5520	15.3	4/16
110	4-37	5150	5220	8.9	5/16
100	4-16	4800	5200	51.0	10/16
Ave.				33.7	8/16
Range				8.9 to 51	3/16 to 11/16

^a Tendon Force = [Col. (2) - Col. (1)] × (Ram Area); Ram Area = 127.6 in².

^b The tendons were intentionally stressed approximately 6% above the force required to produce lift off.

APPENDIX I-A
PERIODIC TEST PROCEDURE
CONTAINMENT TENDON STRESSING

1.0 PURPOSE

- 1.1 To provide the steps necessary for performing the Containment Vessel (C.V.) Tendon Lift-Off Test.

2.0 TEST REQUIREMENTS

- 2.1 Twenty-two (22) specific tendons around the containment shall be inspected.

NOTE: Results and Test Department will designate the specific tendons to be tested.

- 2.2 Inspection will include a visual inspection for broken wires and signs of corrosion.

- 2.3 The lift-off force for each of the tendons, at the top of the tendon, shall be determined.

- 2.4 Two tendons shall be detensioned completely.

- 2.5 The connection to the rock anchor for one tendon shall be inspected for corrosion and other deleterious conditions. Prior to completely detensioning this tendon, its initial position relative to a fixed datum shall be determined. During detensioning this position shall be monitored and its final position indicated.

- 2.6 After lift-off, a force to achieve an additional 6% stress shall be applied to verify the ability of the tendon to sustain the added stress applied during accident conditions.

3.0 REFERENCES

3.1 Technical Specifications' Section 4.4.4

3.2 FSAR - Volume 2, Sections 5.1.2.5 and 5.1.2.9 and 5.6.2.2

3.3 Inland-Ryerson Tendon Stressing Report

4.0 INITIAL CONDITIONS

4.1 Plant may be in any phase of operation. _____

4.2 Pressure gauge has been calibrated. _____

4.3 Hydraulic pump and ram are functional and ready for
operation. _____

4.4 All tendons to be stressed have been inspected for
broken wires. _____

5.0 PRECAUTIONS

5.1 Observe all RG&E safety rules and regulations. _____

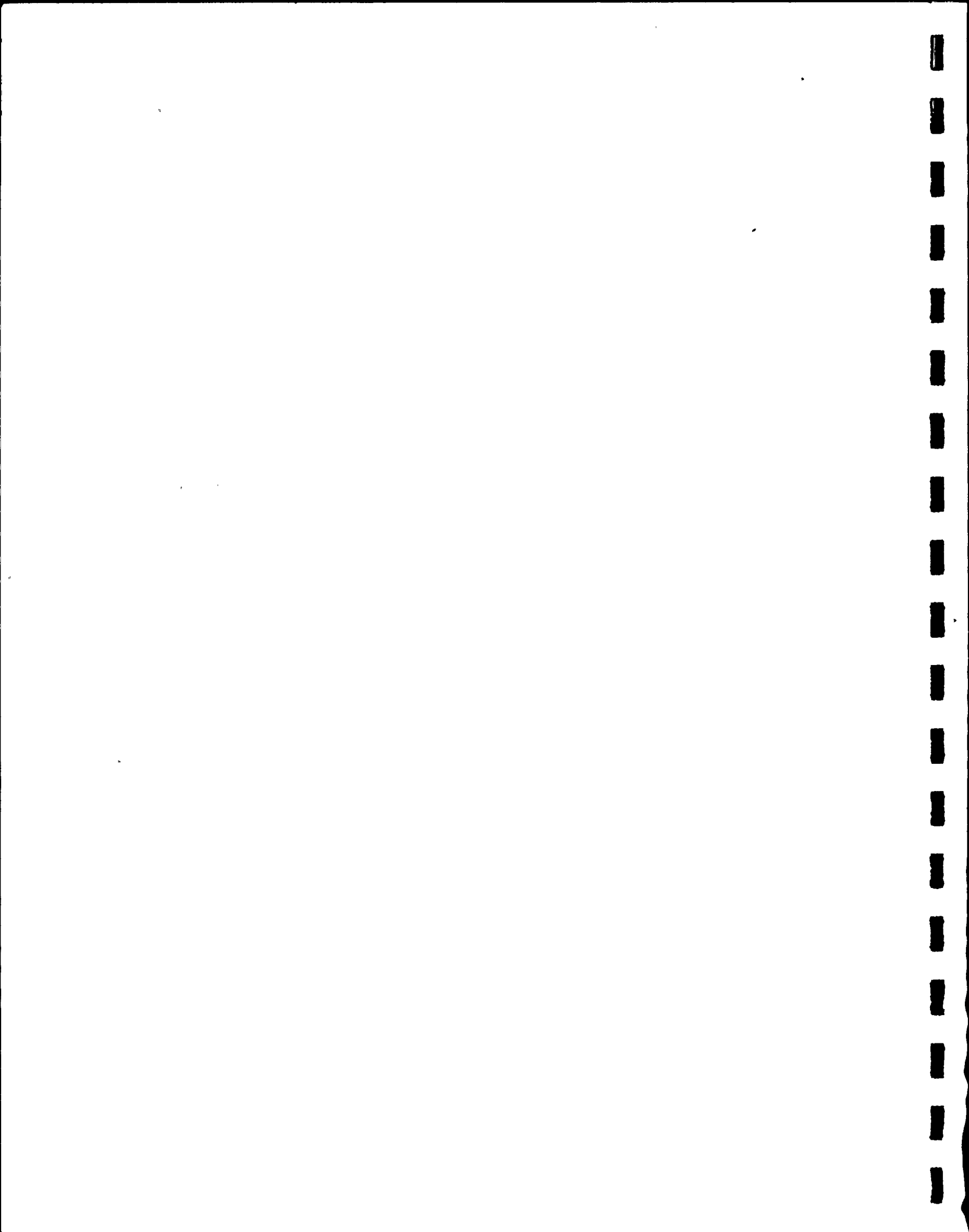
5.2 Do not exceed 6560 psi gauge pressure for jack and tendons. _____

5.3 Whenever hydraulic pump is operating, the reservoir vent
valve must be open. _____

5.4 Do not extend ram more than 8 inches. _____

6.0 INSTRUCTIONS

6.1 Transport assembled hydraulic jack and pump to position
on top of containment wall. _____



6.1.1 Place jack assembly on tendon base plate. _____

6.1.2 Center jack chair carefully on the base plate. _____

6.1.3 Carefully thread stressing adaptor onto tendon anchor head. _____

NOTE: Leave a minimum of one thread and maximum of three threads on tendon anchor exposed below the lower edge of the stressing adaptor.

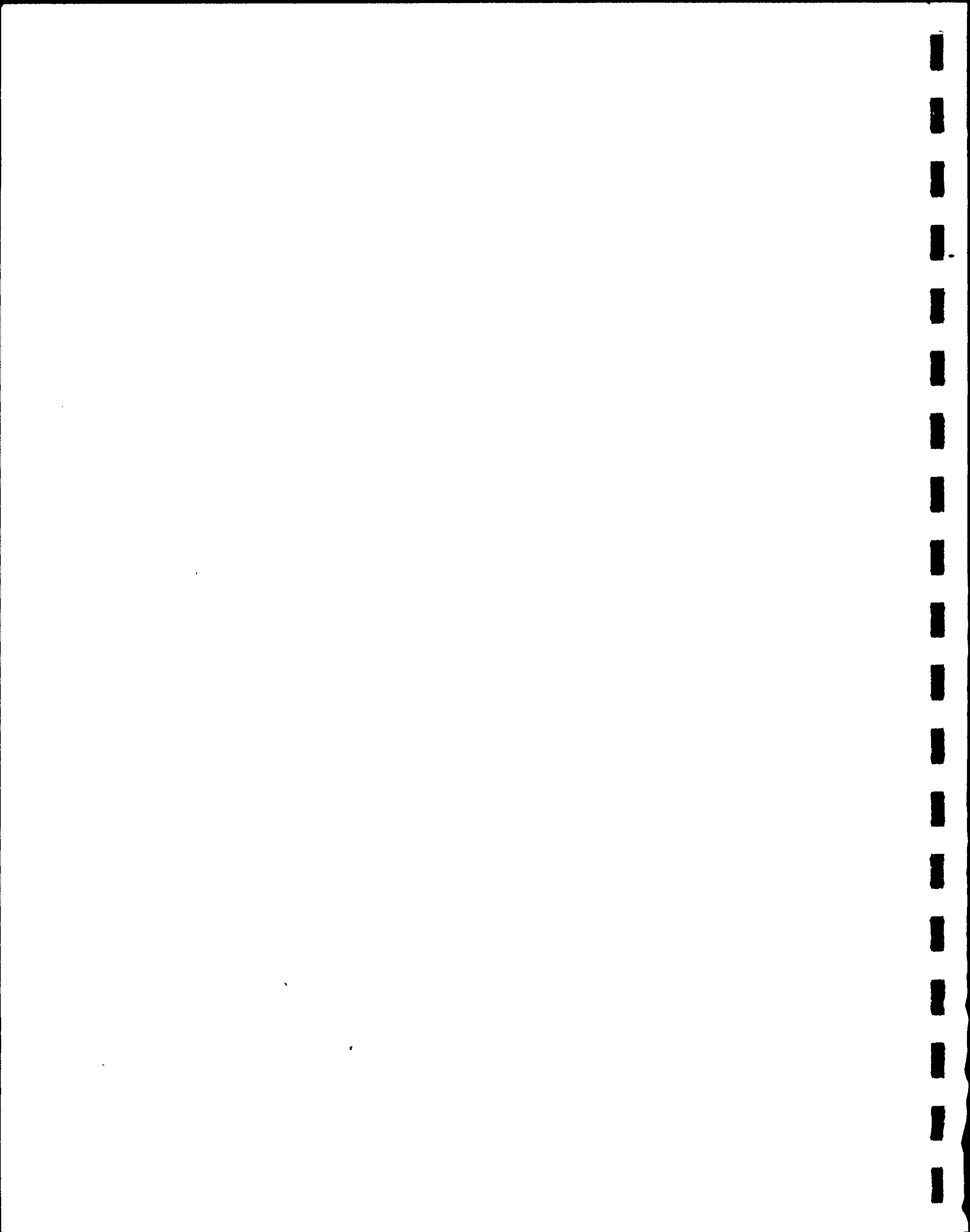
6.1.4 Tighten and center compression ring and jack rod nut at top (ram) end of jack. _____

6.2 Before attaching hoses to jack for the first tendon, check pump and hoses by performing the following:

- 1) Set valves to pump position. _____
- 2) Start pump at the same time depress the ball valve in quick coupler at the end of the hose. _____
- 3) Pump slowly to fill hose with oil until it comes out of the hose free of bubbles. _____
- 4) Release ball valve and start pumping again. _____
Continue pumping; hose will become stiff; gauge pressure will rise rapidly and then hold constant.

NOTE: If pressure will not go up to 8,500 psi or if it goes over 8,500 psi, the automatic cut off valve must be adjusted.

- 5) Reverse hoses and repeat Steps 1) through 4). _____



- 6.3 Reduce pressure and connect hoses to jack. _____
- 6.4 Start pump and increase pressure to 1,000 psi. _____
- 6.5 Record ram position in inch in column "A" on data sheet. _____
- 6.6 Increase pressure up to 2,900 psi and hold. _____
- 6.6.1 Inspect for leakage of hydraulic fluid and note if leakage is excessive. _____
- NOTE: This is approximately 2,000 psi below the required minimum stress of 144 ksi.
- 6.7 Tap on the installed shims below the anchor head with a piece of metal and slowly increase pressure. _____
- 6.7.1 Record in Column B of data sheet, the pressure gauge reading when a change in the sound of the tap occurs. _____
- 6.8 Stress tendon an additional 6% over recorded lift-off pressure. _____
- 6.8.1 Record computed 6% greater stressing value in column C of data sheet. _____
- 6.8.2 Increase pressure to computed 6% stressing pressure and record ram position in column D of data sheet. _____
- 6.8.3 Place a .035 inch thick shims on the shim pack and reduce ram pressure to the level of 6.6. _____
- 6.8.4 Slowly increase pressure until the added shims can be withdrawn from shim stack. Record in column E. _____

- 6.9 Reseat tendon on shims, remove stressing equipment and
 move on to next tendon. _____
- 6.10 Repeat steps 6.1 through 6.9 until all tendons have been
 tested. _____
- 6.11 For tendon 150 to be fully detensioned, all shims shall
 be removed after step 6.8.4. _____
- 6.11.1 Initial position of head shall be measured from a
 datum point. Position of lower head shall also be
 recorded. _____
- 6.11.2 Ram pressure shall be slowly reduced with both head
 positions recorded at a minimum of 1000 psi increments
 to zero. _____
- 6.11.3 Ram pressure shall be slowly increased to original
 pressure and shims replaced. _____
- 6.11.4 Reseat tendon on shims, remove stressing equipment and
 move on to next tendon. _____

