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Office of Nuclear Reactor Regulation
ZIEMANN,D.L. Procedures & Test Review Branch

SUBJECT: Submits addl info re tendon insp & lift-off verification,in response to NRC 800326 ltr.Also forwards revised pages & tables for GAI-2074, "Evaluation of Prestressed Tendon Forces."

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LEON D. WHITE, JR.
VICE PRESIDENT

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May 13, 1980

Director of Nuclear Regulation
Attention: Mr. Dennis L. Ziemann, Chief
Operating Reactors Branch #2
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555

Subject: Tendon Inspection and
Lift-Off Verification
Ginna Nuclear Power Plant
Docket No. 50-244

Dear Mr. Ziemann:

In response to your letter dated March 26, 1980, we have prepared the following information. Also, revised sheets dated April 11, 1980 to be incorporated into the report "Evaluation of Prestressed Tendon Forces - Robert E. Ginna Nuclear Power Station" are included as Attachment 1 to this letter.

Response to Question No. 1:

Regarding the statement made in the first sentence, shrinkage is not considered to be an important factor contributing to loss of prestress. As seen from Table 2 of the report, shrinkage accounts for only 2.8 kips (2.1%) of the total 40 yr. loss of 130.5 kips.

Shrinkage tests were conducted on various trial concrete mix designs. The tests were in accordance with ASTM C-494, "Chemical Admixtures for Concrete." The results were used for comparing the shrinkage of various mix designs. The shrinkage values, which were in the neighborhood of 250×10^{-6} in./in. at 28 days, were based on 3 in. square by 11¼ in. long specimens which were stored in air at 73°F and 50% relative humidity. The shrinkage strains which these specimens experienced are not applicable to those occurring in the 3 ft. 6 in. massive containment walls. These walls have a volume-to-exposed surface ratio of 42 in., versus 0.7 in. for the test specimens. The information given in References (4) and (6) of the report indicates that 100×10^{-6} in./in. is a conservatively large 40 yr. shrinkage strain for V/S = 42 in.

A001 S/1

8005210500

Project documents identify the concrete mix design used for the containment wall as:

Cement (Type II)		658	lbs. (7 sacks)
Fine Aggregate		1310	lbs.
Coarse Aggregate	(N.Y.#1)	1290	lbs.
	(N.Y.#2)	440	lbs.
Water		35.7	gals. (298 lbs.)
Admixtures			
Darex		4.5	oz. per yard
Plastiment		21	oz. per yard

The water-cement ratio of the mix is 0.45 (by weight). Neither the Proposed Regulatory Guide 1.35.1 nor its referenced document, Reference (4), explicitly relates shrinkage to water-cement ratio. The shrinkage test specimens of Reference (4) contained 5.44 sacks of Type III cement per cubic yard, and some sealed specimens were included. Although Reference (4) does not state the water content for the specimens, it would be expected that this concrete had a water-cement ratio equal to or greater than that of the Ginna containment concrete which used a 0.45 water-cement ratio.

As stated in the report, the Mean Daily Relative Humidity for the Ginna site is in the 70% to 80% range. This humdity is within the 40% to 80% range of Proposed Regulatory Guide 1.35.1 for which 100×10^{-6} in./in. is specified.

Therefore, it is concluded that the shrinkage value of Reference (4) as applied to the Ginna containment concrete is conservative.

Response to Question No. 2:

Two basic types of shrinkage occur in concrete: drying shrinkage and autogeneous shrinkage. Due to the massive nature of the 42" thick containment wall, most of the vertical shrinkage of the wall that is significant enough to cause a loss of prestress force, occurs over its interior and is due to autogeneous shrinkage. It is generally accepted that autogeneous shrinkage is assumed to start as soon as the hydration process starts. Thus, to simulate this condition, "one hour after concrete placement" was used to start the shrinkage strain curve in Figure 3 of the report.

As shown in Figure 2 of the report, the concrete placement in the containment wall started in April 1967 and was completed in June 1968. Use of the average placement date for the shrinkage

strain calculation assumes that the lower half of the wall was constructed prior to this average date and the upper half was constructed after this average date. Thus, at any point in time, concrete in the lower half of the wall would experience shrinkage strains greater than that predicted based on the average date, and concrete in the upper half would experience shrinkage strains less than that predicted based on the average date. The total shrinkage shortening of the wall is the sum of the shrinkage strains for each concrete lift times the lift height; however, the shrinkage strain is different for each lift. Use of an average placement date for the concrete to establish an average shrinkage for all the lifts is a reasonable and sufficiently accurate approach.

Response to Question No. 3:

The creep curve of Reference (6) of the report is included as Attachment 2. It was used to establish the specific creep values defined in the report. Independent creep tests were not required for Ginna and were, therefore, not conducted.

The concrete mix design used for the containment wall is given in the response to Question 1. For the Ginna mix, the volume concentration of cement paste in the concrete is 0.34, and the water-cement ratio by weight is 0.45. For Reference (6) these values were 0.30 and 0.50, respectively. Because of the similarity of the two mixes, the specific creep data of Reference (6) is considered applicable to the Ginna concrete.

Response to Question No. 4:

The design basis for the Ginna containment was ACI 318-63. It does not contain a requirement to provide relaxation tests for the tendon wires. Data used was provided by the tendon supplier based on typical wires produced at that time.

As stated in the report, the prestressing wire was specified to have a nominal 40-year stress relaxation of 12%. Figure 3A of the report is based on stress-relaxation curves from Armco Steel Corporation (See Attachment 3), the supplier of the wire for Ginna. The curves for Specimens 1 and 2 were provided by Armco as being representative of the relaxation grade of wire provided for Ginna, but the specimens were not taken from Ginna production wire. The curve in Figure 3A of the report was constructed by multiplying the relaxation curve for Specimen 2 by the factor (12/11.5). Note that the curve for Specimen 2 includes data from 1000-hour relaxation tests.

Response to Question No. 5:

In the original design, a required minimum average tendon force of 636 kips was calculated. This force resulted from the controlling load combination:

$$0.95\ D + F + 1.5P + T_a$$

where

D = Dead load
F = Vertical prestress
P = Accident pressure of 60 psig
T_a = Accident temperature, including operating temperature

Response to Question No. 6:

The principle cause of the loss of tendon prestress is the relaxation of the steel. The results of the ten year retest indicate that what appeared to be an increased rate of loss was in fact discrepancies in the acoustic lift-off readings for the 8 year surveillance. During the ten year retest, a visual observation of the effect of complete detensioning on the connection between the rock anchor and the wall tendon was made. No observable movement occurred (see page I-3 of Addendum 1 of the report). The effect of detensioning one tendon, of a total of 160, on the elastomeric pad was not addressed during the ten year retest since the change would have been too small to provide reliable data.

Response to Question No. 7:

The use of 1/32 inch shims rather than the acoustical method to determine lift-off was recommended by Inryco, Inc., who supplied and stressed the tendons and conducted the 10 year retest of the tendons. A lack of reproducibility of lift-off readings on some projects, as demonstrated by the differences between the 8 and 10 year Ginna tests, prompted a change to this procedure. Note from Table I-1 of the report, columns (1) and (2), that there is not a significant difference in the lift-off pressures using the two methods. The acoustic method of determining lift-off is highly dependent on the individual conducting the test while the ability to remove a 1/32 inch shim from the shim stack can clearly be demonstrated.

Response to Question No. 8:

An inspection of the concrete around the tendon anchorages has not been required during tendon surveillance. Hence, documentation, visual inspection or photographs are not available for past surveillances. The concrete surrounding the rock anchor is inaccessible and has not been inspected.

Question 9 is in several parts which will be responded to as individual questions.

Response to Question No. 9a:

Pages 5.1.2-20, 5.1.2-20a, and 5.1.2-21 of the Ginna FSAR (See Attachment 2, part 6 of the report) give the design bases for the rock anchors and contain the basic references used to justify the criteria.

Response to Question No. 9b:

Page 5.1.2-23 of the Ginna FSAR (See Attachment 2, part 6 of the report) indicates the capacities and safety factors for the original design.

Response to Question No. 9c:

RG&E records indicate that the typical rock anchor length is 33'-6". Individual records for each tendon are available if desired.

Response to Question No. 9d:

Pages 5.6.1-4 through 5.6.1-5 of the Ginna FSAR (See Attachment 4) describe the scaled down tests performed to develop rock anchor criteria.

Response to Question No. 9e:

Page 5.1.2-24 of the Ginna FSAR (See Attachment 2, part 6 of the report)describes the technique used to install the tendon/rock anchor coupling device. Attachment 5 is an assembly drawing of the tendon to rock coupling, and the manufacturer's drawings of the individual parts.

Response to Question No. 9f:

The design assumption to utilize the weight of the rock (See response to question 9a) as the maximum force for the anchor takes into account all adverse effects of group action since the capacity of the anchor was based on .8 UTS or 847 kips which is a larger force. Since the rock anchors were stressed to .8 UTS during initial installation, new loads to be applied during the re-tensioning program should have no adverse effects.

If there are any further questions regarding this information, please contact us.

Very truly yours,

L.D. White, Jr.

L.D. White, Jr.

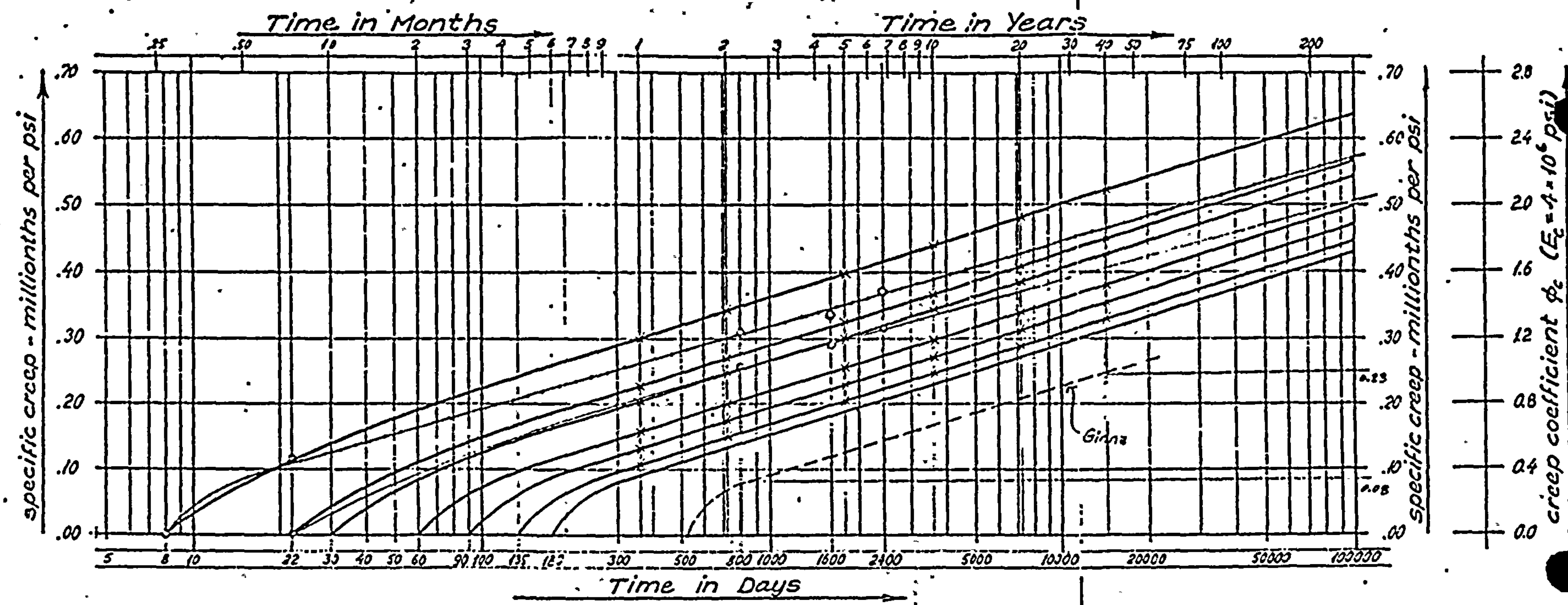
LDW:rb
Attachments

xc: Mr. Boyce H. Grier, Director
Office of Inspection and
Enforcement Region I
U.S. Nuclear Regulatory Commission
w/attachments

Attachment 1

Attachment 2

Basic Creep for 5000 psi Concrete



—o———o— Total creep @ cgs ...., Port Hueneme Tests[1]; age @ loading: 8 days;

—o———o— Differential creep, top fibre, —"— —"— —"—[1], —"— @ —"— : 22 —"—;

———— Estimated basic creep[2] for loading ages: 8 & 22 days and 1, 2, 3, 4½ & 5 months.

[1] Technical Report R212, U.S. Naval Civil Engineering Laboratory, see also Appendix B.

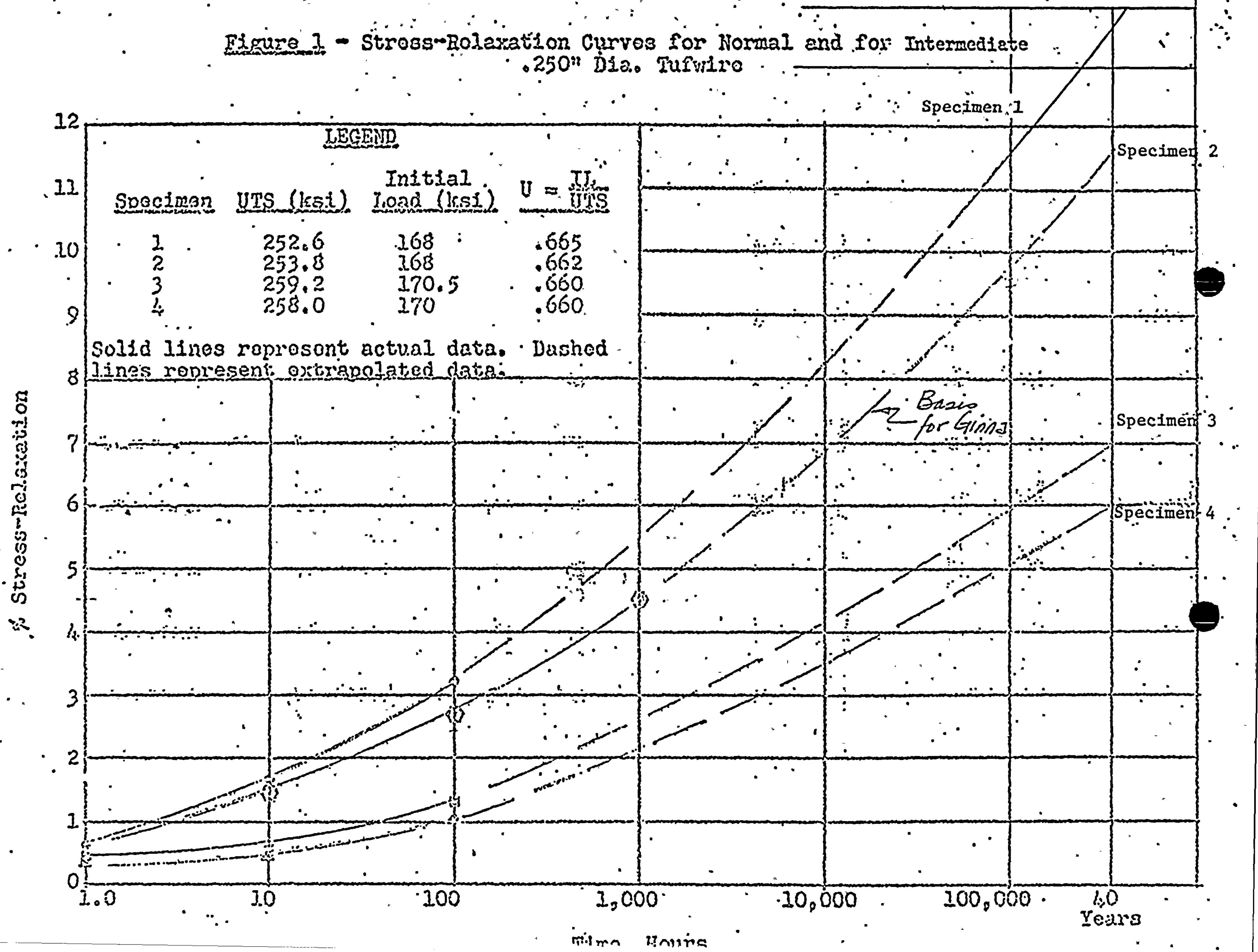
[2] Formula for basic creep: $\frac{\varepsilon_{creep}}{\sigma} \cdot 10^6 = \frac{0.183(t_0 \ldots)}{g(t_0)}(1 - e^{-(t-t_0)/30}) + 0.6 \ln(\frac{t}{t_0})$, ref. equation [5], Appendix A.

SCHUPACK & ASSOCIATES
CONSULTING ENGINEERS - STAMFORD, CONN.
Job: Oyster Creek Unit 2
By: C.S. Date: May 10, 1968

APPENDIX C

Attachment 3

Figure 1 - Stress-Relaxation Curves for Normal and for Intermediate .250" Dia. Tufwire



Type II cement, modified for low heat of hydration, is used to minimize shrinkage.

"Grab" samples are taken periodically at the batch plant, upon delivery of cement. Each sample is tested by the Testing Laboratory for conformance to ASTM C 150, and the results are also compared with the certificate supply with each delivery of cement.

Elastometer Bearing Pads

Tests are performed on elastomer specimens to ensure compliance with requirements for (1) original physical properties including tear resistance, hardness, tensile strength and ultimate elongation, (2) change in physical properties due to overaging, (3) extreme temperature characteristics, (4) ozone cracking resistance, (5) oil swell and (6) shear modulus. In addition, two full size pads are tested, one for creep and one for ultimate load. Specimen No. 1 is initially placed under essentially a constant compressive load of 1000 psi (the design pressure) for four days to measure creep. This pad is then loaded up to 2000 kips (5.3 times design load) when the test was terminated without failure. Specimen No. 2 was similarly loaded up to 2000 kips without failure. The rebound of the pads after the 2000 kip load was removed is essentially complete. A summary of the test results is shown in Figures 5.6.1-3 and 5.6.1-4.

Rock Anchor Tests

Three scaled down test rock anchors were installed to demonstrate first the hold-down capacity of the rock and second the capacity of the bond between rock and grout.

Two tests were made on rock anchor "A" which was installed at the center of the proposed containment vessel. The first test, called test A-1 was to determine rock hold-down capacity. The set-up for test A-1 is illustrated in Figure 5.6.1-5. The beam support piers were located beyond the assumed influence circle of rock having a diameter of 23 feet 6 inches. An independent frame was erected to obtain deflection measurements on the concrete pier at the anchor. This placed all supports for lifting as well as measuring devices outside the influence circle of rock. Dial gauges were used to measure the movement of the concrete pier and the anchor head. The test load was applied with a 150 ton jack mounted on the beams spanning the test anchor. Measurements of the jacking force were made with a dynamometer, calibrated immediately before the test. The second test on rock anchor "A" (Test A-2) and the tests on rock anchors "B" and "C", also installed near the center of the proposed containment vessel, were made to demonstrate bond capacity. The set-up for test A-2 and for rock anchors "B" and "C" was an arrangement whereby the jack was supported directly by the concrete pier adjacent to the test anchor.

Rock anchor "A" consists of twenty-eight 1/4 inch diameter wires grouted for a length of 4 feet 5-1/2 inches in a 3-1/2 inch diameter hole. All test rock anchors were oversized so that the test load of 100 kips would develop only about 30% of the ultimate capacity of tendon wires while developing a bond stress of 170 psi which is the design stress for the containment rock anchors. This permitted testing bond stresses well in excess of design (170 psi) without exceeding ultimate wire stresses.

The test procedure for test A-1 was as follows:

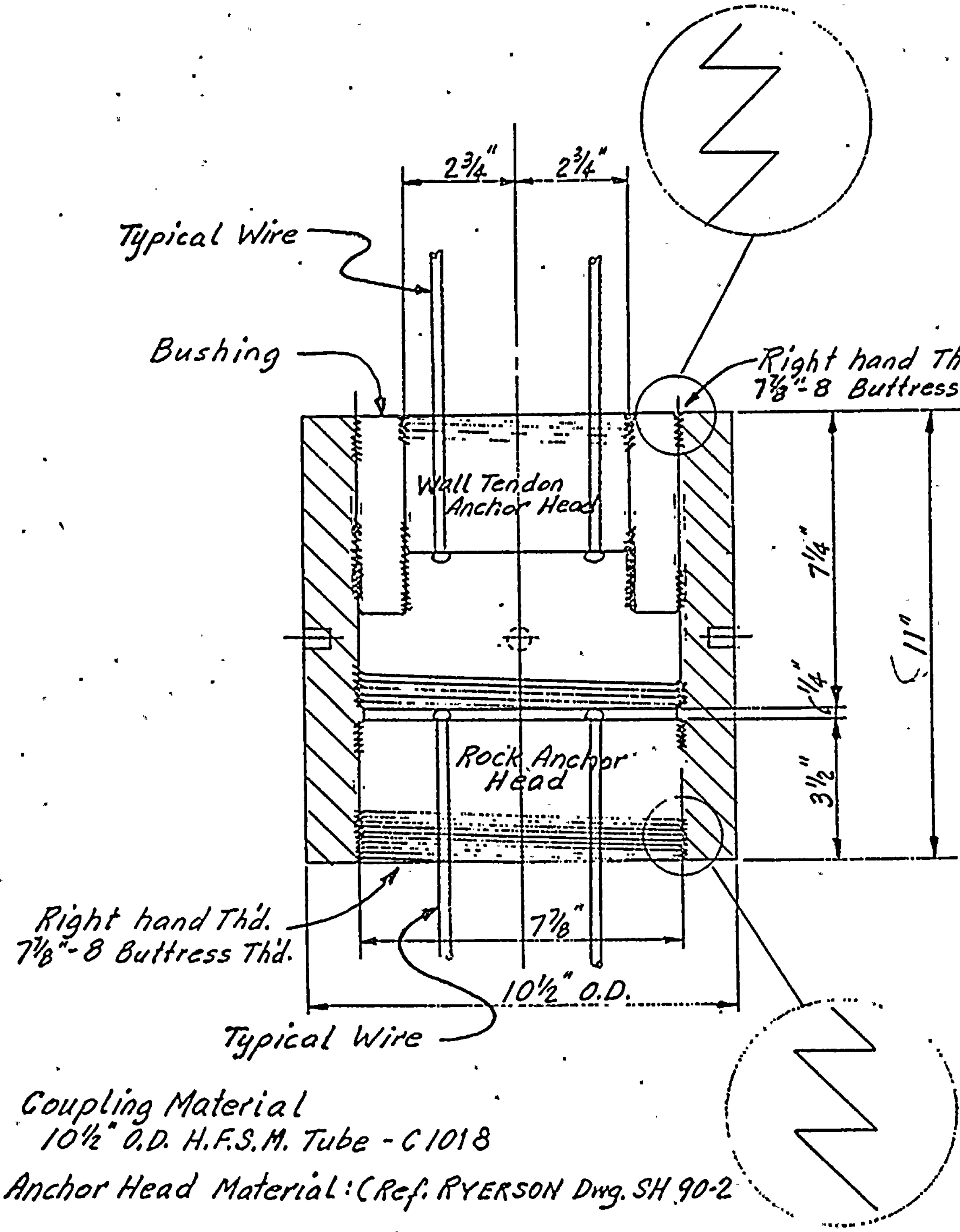
The anchor was loaded in 20,000 pound increments to 100,000 pounds. The load was maintained at each increment for 15 minutes prior to taking measurements for elongation of the tendon and clevations of the concrete pedestal and adjacent rock surface. Because the anchor head appeared from visual observation to not have lifted off at the 100,000 pound load, the load was increased to 110,000 pounds at which point lift off was apparent. Subsequent review of measurements on the movement of the anchor head indicate that actual lift off occurred between 80,000 pounds and 100,000 pounds as would be expected.

In test A-2, "B" and "C", tendon was jacked from the concrete pier immediately adjacent to the tendon.

Table 5.6.1-1 lists measurements taken during test A-1. figures 5.6.1-6, 5.6.1-7 and 5.6.1-8 show plots of load vs. elongation deflection for all tests.

The application of a test load of 110 kips to rock anchor "A" (as indicated by the results of test A-1 shown on Figure 5.6.1-6) is equivalent to 137.5% of the calculated hold-down capacity assumption used in the design is valid. The plot of load vs. elongation deflection for rock anchor "A" tests A-2 (see Figure 5.6.1-6) and "B" and "C" (see Figures 5.6.1-7 and 5.6.1-8) indicate a factor of safety against slippage by the grout and rock of at least 2.0 (200 kip load vs. 100 kip design load) for rock anchor "B". If slippage occurred within the grout the factor of safety against failure is even greater. The plot of load vs. elongation for rock anchor "A" shows an apparent discontinuity which is indicated by a dashed line on Figure 5.6.1-6. This represents settlement of the concrete pier adjacent to the rock anchor when the load was transferred from the lifting frame used in test A-1 to the lock nut which bears on the concrete pier.

Attachment 5

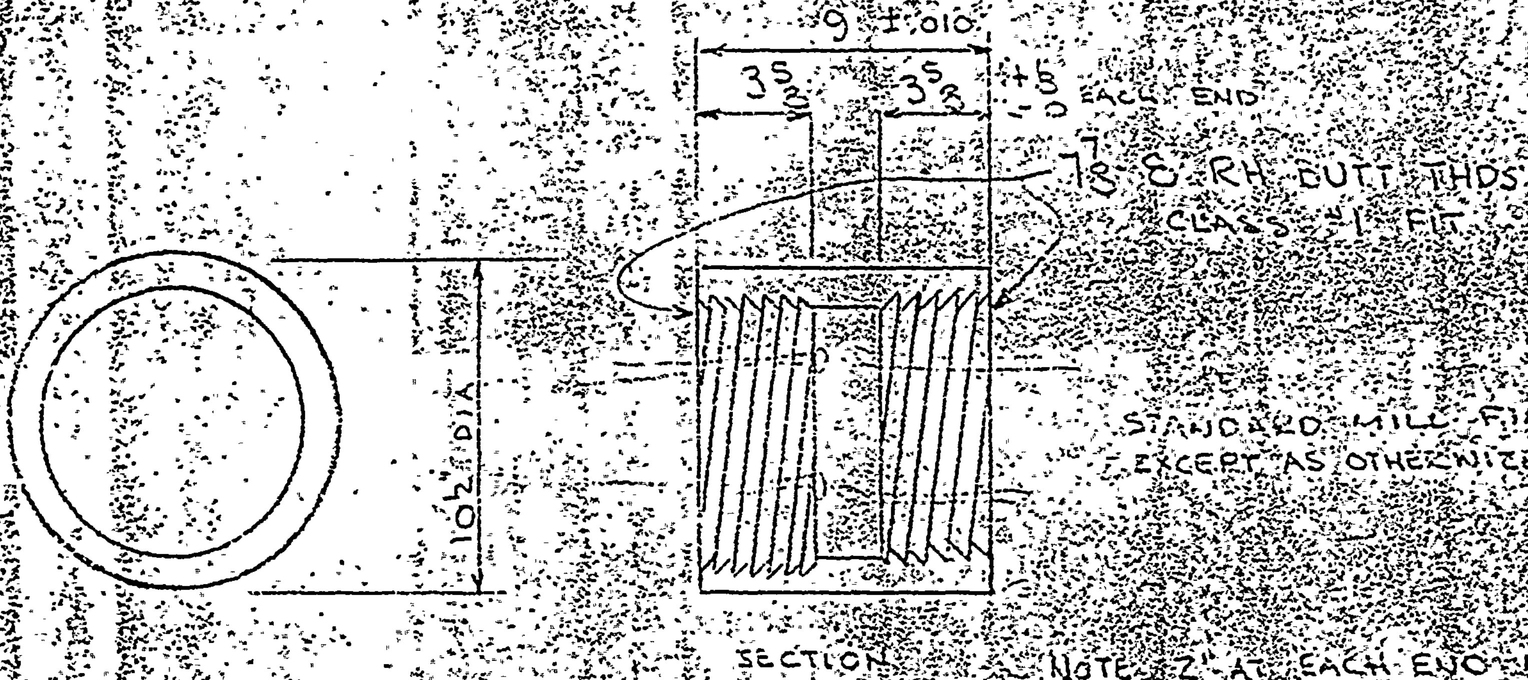


Coupling Material
10 1/2" O.D. H.F.S.M. Tube - C 1018

Anchor Head Material: (Ref. RYERSON Dwg. SH 90-2

Jacking Force 847^K
Effective Prestress Force 635^K

TENDON TO ROCK COUPLING
FIG. 5.1.2-12



STANDARD MILL FINISH EXCEPT AS OTHERWISE NOTED

NOTE: 2" AT EACH END MAY BE ROUGH TURNED TO 10 7/16 ±.010 AS AN AID TO CENTERING FOR THREADING.

MATERIAL
10 1/2 O.D. HFSM TUBE
1/2 WALL 9 LONG
C1018

TOLERANCE FOR TUBE STOCK IS STANDARD COMMERCIAL TOLERANCE
WALL THICKNESS ±7.5%
O.D. ±.045" ; I.D. ±.063"
MAX CAMBER .060" IN ANY 3 FT
OUT OF ROUND WITHIN DIA TOL.

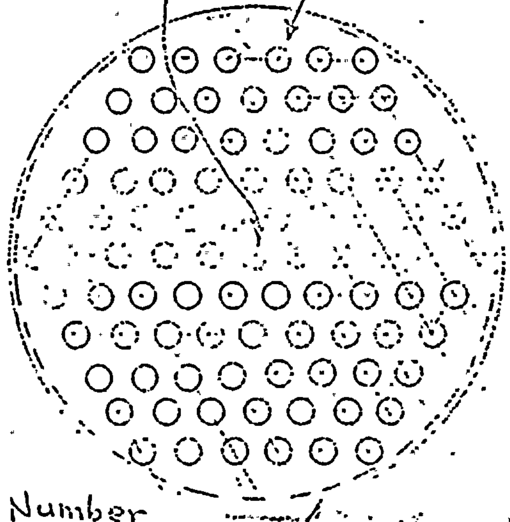
COUPLER FOR 90 WIRE 7/8 C.R. THREADED ANCHOR HEADS	DRAWN BY CLYDE. / DATE 7/21/66
	REV 11/1/66
RYERSON — JOSEPH T. RYERSON & SON, INC. — METALOGICS — CENTRAL REINFORCING PRODUCTS — POST-TENSIONING SERVICE	21T341891
	DRAWING NO. 90X10

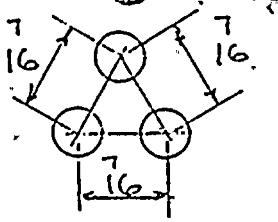
To be inspected per machining specification of 7/27/67.

REC'D MPG
NOV 3 1967
GILBE... ... ... INC.
4155

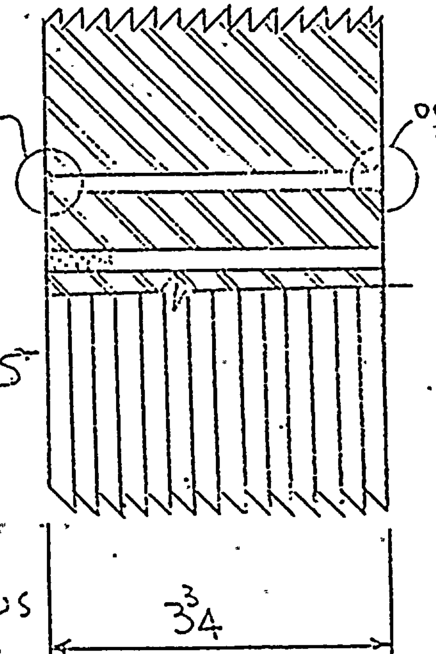
90 WIRE HOLES .266 $^{+.004}_{-.001}$ DIA DRILL THROUGH; SPACE AS SHOWN AND NOTED

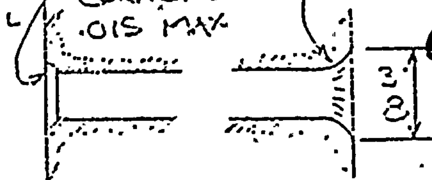
TAP CENTER HOLE 3/8-16 3/4" DEEP AS SHOWN DRILL HOLE THROUGH





TYPICAL HOLE SPACING





DETAIL #1 DETAIL #2

TYPICAL WIRE HOLE

Button Head 5" Bearing Surface

stamp Heat Number code on button head bearing surface

5 1/8-8 RH BUTTRESS THREADS CLASS #1 FIT

PARTIAL SECTION

OFFICE COPY

90 WIRE ANCHOR HEAD

NOTICE

MATERIAL
C1141 5 1/4 DIA H.R. ROUND
Heat Treat - Oil Quench & Temper to a Rockwell C29 to C33 @ B.H. Bearing Surface

MADE BY	DATE		
CLYDE	10/20/67	RYERSON — JOSEPH T. RYERSON & SON, INC. — METALOGICS	CUSTOMER Bechtel
90W2-A			21T159

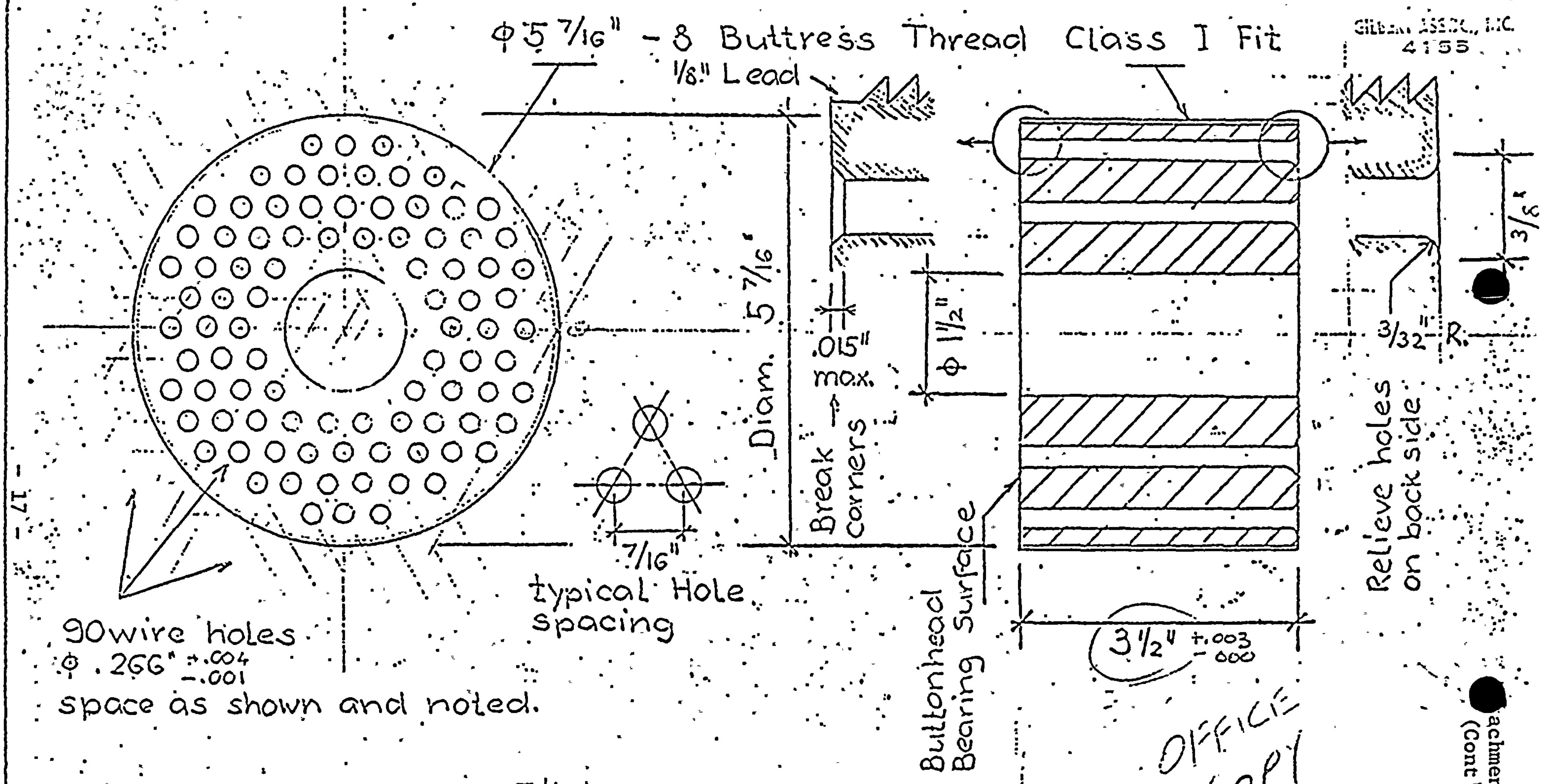
FORM 760.12-2 APR. 63

formerly 90x6 (6-1-66)

Attachment #5 (Cont'd).

To be inspected per machining specifications of 7/27/67
stamp Heat Number code on button head bearing surface.

REC'D-MPG.
NOV 3 1967



Material : H.R C1141 Φ 5 1/2" BAR

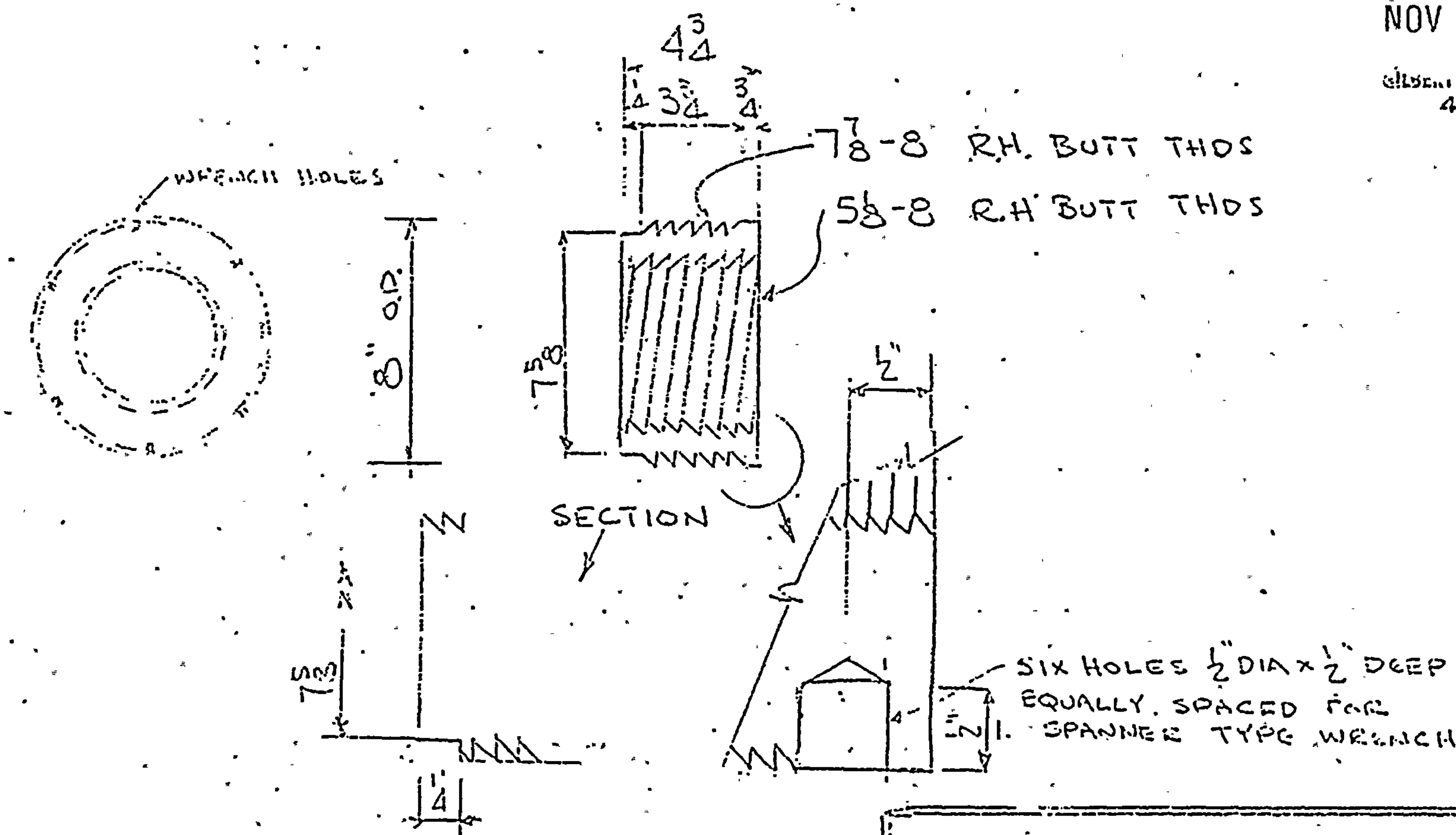
Hacksaw cut to 3 9/16" Length
Heat treat, oil quench and temper
to Rockwell C29 to C33 at B.H. Bearing Surface. NER Rev. 10/20/67

Attachment #5 (Cont'd)

90 wire Anchor Head (for Shop Buttonheading End)	DRAWN BY A.Ws / DATE 3/20/67
RYERSON METALOGICS — JOSEPH T. RYERSON & SON, INC. — CENTRAL REINFORCING PRODUCTS — POST-TENSIONING SERVICE	Bechtel
	21T159
	DRAWING NO. [illegible]

To be inspected per machining specification of [illegible]

Stamp Heat Number Code on Surface



MATERIAL
8" O.D. H.R. C1141 x $4\,7/8\,^{+1/16}_{-0}$

ADAPTOR FOR 90 WIRE 5 5/8 O.D. THD ANCHOR HEAD TO 7 7/8 I.D THD COUPLER	DRAWN BY CLYDE	DATE 10/20/67

RYERSON METALOGICS
JOSEPH T. RYERSON & SON, INC.
CENTRAL REINFORCING PRODUCTS
POST-TENSIONING SERVICE

Bechtel

21T159

DRAWING NO. 90W15-A

formerly 90-X11

Attachment #5 (Cont'd)