ROCK ANCHOR TEST PROGRAM REPORT
FOR
NORTH ANNA UNITS 3&4

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VIRGINIA ELECTRIC AND
POWER COMPANY

AND RPT 7902270277

STONE & WEBSTER ENGINEERING CORPORATION
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BACKGROUND

1.1 INTRODUCTION

On July 23, 1975, Virginia Electric and Power Company (VEPCO) notified Nuclear Regulatory Commission (NRC) Region II Inspection and Enforcement personnel that certain North Anna Units 3 and 4 Class I structures would experience unacceptable translation and rotation during a Design Basis Earthquake (DBE). VEPCO considered this circumstance to be reportable under the provisions of 10CFR50.55(e). An interim report was sent to the NRC on August 22, 1975 (Serial No. 660) which stated that a complete report would be forthcoming by December 23, 1975. The interim report listed three alternatives under consideration to ameliorate the situation. Due to delays in the rock anchor testing program, a final report was not sent until March 30, 1976 (serial no. 956). The final report describes in detail the methods to be used to accommodate rotation and translation motions of the various structures. The selection of the rock anchor alternative is described together with the objectives to be obtained from a rock anchor test program.

A typographical error in the nominal strand diameter in the final report was corrected in an April 27, 1976 letter (serial no. 956A).

On January 11, 1977, members of the NRC Regulatory Staff contacted VEPCO concerning certain aspects of rotation and translation motions under DBE conditions for North Anna Units 1 and 2 and Units 3 and 4. Specifically, the NRC requested documentation of the design differences in Units 1 and 2 and Units 3 and 4 which make rock anchors necessary on one plant and not on the other. Consequently, a report in response to this request was submitted to the NRC on February 18, 1977, serial no. 051.

This report documents all aspects of the Rock Anchor Test Program performed at the North Anna Nuclear Power Station, Units 3 and 4, between December 1975 and June 1976. It is submitted to the NRC for information and documentation. The test program was performed by the Nicholson Anchorage Company under the supervision of Stone & Webster Engineering Corporation and in accordance with the requirements given in Specification NAS-30163, "Posttensioned Rock Anchor Proof Test," which is included as Appendix A of this report.

The report is divided into four sections. Section 1 presents the background to the Rock Anchor Test Program. Section 2 covers the requirements and construction of the test program, including anchor material, testing, and instrumentation, and summarizes the Rock Anchor Test Report by the Nicholson Anchorage Company, included as Appendix B of this report. A detailed presentation and evaluation of the data obtained from the test program is presented in Section 3. Recommendations for the design, the construction requirements, and the in-service inspection program for the permanent anchors are presented in Section 4.

1.2 PURPOSE OF ROCK ANCHORS

The containment auxiliary structures (the main steam quench spray and safeguards area buildings), due to their geometry and lack of lateral restraint (3 in rattle space) adjacent to the reactor containments (see Figures 1-1 to 1-4) would experience unbalanced lateral forces and overturning moments during static and seismic loading conditions, thereby resulting in translational and rotational movements of these structures. Concrete reinforced shear keys have been added to the foundation mat to transfer the unbalanced lateral loads to a continuous reinforced concrete ring girder around the containment foundation. Permanent rock anchors, housed in separate galleries, have been included to prevent rotational motion.

Figures 1-1 to 1-4 show the details of the above structural modifications. Figures 1-1 and 1-2 are plan views of the containment auxiliary structures. Figures 1-3 and 1-4 show sections through the Unit 3 main steam valve house and the Unit 3 safeguards building, respectively. The galleries housing the permanent rock anchors will provide access to the anchors for the purpose of monitoring the performance of the anchors throughout the plant's lifetime.

1.3 OBJECTIVES OF THE TEST PROGRAM

State-of-the-art practice for designing permanent anchors includes the confirmation of design parameters with a field test program. A comprehensive test program was developed for this reason and also to evaluate the effect that installation procedures have on the design capacity of the anchor system. Specifically, the objectives of the test program were as follows:

1. To determine the effects of anchor type, grout type, grout mix design, and anchor length on anchor capacity and, therefore, to determine the anchor components most suited to the permanent anchors
2. To verify design assumptions regarding grout-rock and grout-steel bond strengths and document the ability of the grouted anchorage to transfer the load from the tendon to the surrounding rock
3. To provide a rational basis for estimating tendon elongation, prestress loss, rock and foundation mat deformations, and grout column movement
4. To determine the best procedure for installing and stressing the production anchors

1.4 TEST PROGRAM DESIGN

The state-of-the-art paper by Littlejohn and Bruce(1) provided the basis for the test anchor design, in particular, the basis for determining the total and bonded lengths of the tendons. The stressing procedures were based upon the guidelines given by Schnabel(2) and upon the anticipated loading conditions for the permanent anchors. Material selection for the test anchor components was based upon the guidelines given in subsection CC 2400, "Materials for Prestressing Steel," of ACI 359-74,(3) the Prestressed Concrete Institute,(4) and Regulatory Guide 1.103 provided by the U.S. Nuclear Regulatory Commission.(5) A similar test program performed by the TVA at the Bellefonte Nuclear Power Plant, Units 1 and 2(6) was also reviewed. A detailed description of the anchor components is given in section 2 of this report. The terms used to describe the anchor components are defined in Specification NAS-30163 (Appendix A).

1.5 TEST PROGRAM LOCATION

The test anchors were located as close as possible to the planned rock anchor galleries and, therefore, to be in rock conditions similar to those anticipated for the permanent anchors. Test anchor locations are shown in Figure 1-5. The first four anchors were located just north of the Unit 4 containment auxiliary structures, and the remaining three anchors within the area planned for the auxiliary building.

The geologic conditions at each test anchor borehole were verified by recovering NX size core samples. The results of the core sampling are discussed in detail in the following subsection.

1.6 SITE GEOLOGY

The North Anna site lies on the northwest flank of a northeast plunging anticlinal structure. The major stratigraphic units in ascending order (south to north) are a biotite hornblende gneiss, a biotite granite gneiss, a granite gneiss, and a schistose biotite-hornblende gneiss. The banding and foliation displayed in the rocks strike northeast and dip 45 deg to 60 deg to the northwest. The general geology of the site is shown in Figure 1-6. The site geology is presented in detail in geologic reports by Stone & Webster(7),(8) and Dames & Moore.(9)

The stratigraphic units are cut by four major joint sets. The most predominant joint set consists of foliation plane joints. They have an average orientation of N67E, 47NW and are commonly clay filled. The average orientations of the three remaining sets are N67E, 30SE (cross-foliation joints); N15E, 72NW; and N01E, 90. Laboratory shear tests were conducted on natural joint surfaces. The average friction angle for the foliation plane joints is approximately 22 deg. For the remaining joint sets, the average friction angle is approximately 34 deg.

The granite gneiss unit is cut by several faulted chlorite zones which are oriented parallel to the foliation plane joints. The surface out-croppings of these zones are shown in Figure 1-6. References 7, 8, and 9 provide the results of detailed geologic mapping in the containment and adjacent areas.

NX size core samples were taken to document rock conditions at each test anchor location. Logs for these borings (RATH 1 through 7) are included in Appendix C. The cores recovered a gray, fresh to slightly weathered granite gneiss with close to moderately close joint spacing. The cores also intersected occasional pegmatites and quartz veins. Core recovery averaged nearly 100 percent and the Rock Quality Designation (RQD) ranged from 70 to 100 percent, with most values exceeding 85 percent.

SECTION 2

DESIGN AND CONSTRUCTION REQUIREMENTS FOR THE TEST PROGRAM

2.1 INTRODUCTION

This section presents the design and construction requirements of the test program as given in Specification NAS-30163 (Appendix A). The intent of this section is to summarize the Rock Anchor Test Report prepared by the Nicholson Anchorage Company (Appendix B). The test program was conducted using foundation conditions, materials, equipment, instrumentation, installation procedures, and stressing sequences similar to those envisioned for the permanent anchors. These items are presented in this section and are discussed in more detail in Appendix B.

2.2 ANCHOR MATERIALS

Multistrand and multiwire tendon systems were selected for the test program due to their wide application in the nuclear industry for posttensioned reactor containment vessels and as foundation anchors to stabilize dams, retaining structures, and transmission towers. The multistrand system, supplied by the VSL Corporation, consisted of 13 seven-wire strands with an 0.5 in nominal strand diameter. The multiwire system, supplied by the Prescon Corporation, consisted of 46 wires, each being 0.25 in diameter. The ultimate capacities of the strand and wire tendons are 537 kips and 543 kips, respectively. The working capacity and lock-off loads for the permanent anchors were anticipated to be 300 kips and 350 kips, respectively. A detailed description of both tendon systems and the stressing equipment are given in Appendix B. A typical multiwire and multistrand anchor is shown in Figures 2-1 and 2-2, respectively.

Certified Material Test Reports (CMTRs) were required for all tendon components. Selected material qualifications, which include actual results of chemical analyses and physical and mechanical properties, are included in Appendix B.

2.3 GROUT TYPES AND MIX DESIGN

Two grout types were used in the test program, Master Builder's 814 Cable Grout (MB 814) and a Type II Portland cement with an expansive admixture, hereafter referred to as Type II+N. MB 814 is a blend of selected cements, fluidifiers, and thickening agents and is designed to be a high strength, nonbleed grout providing good corrosion protection for highly stressed steel cables. MB 814 was used to grout Anchors 1 through 4 with 2.0 gal of water per 55 lb bag of grout and Anchor 5 with 2.4 gal per bag. The Type II+N cement grout had a water cement ratio of 0.44 by weight and contained an expansive admixture, "Intraplast N" as supplied by Sika Chemical Corporation. The expansive agent was incorporated into the mix at a rate of 0.5 percent by weight of cement. The Type II+N grout was used for Anchors 6 and 7.

2.4 INSTRUMENTATION

A comprehensive array of instrumentation was included in the test program to provide the data necessary to evaluate anchor performance. The instruments fell into three broad categories: anchor instrumentation, rock instrumentation, and permeability test instrumentation.

The anchor instrumentation consisted of (1) electronic load cells and hydraulic jack gages to monitor the loads on the tendons, (2) dial gage indicators to measure tendon elongation, and (3) grout extensometers to measure grout column movement. The extensometers were placed in the grout column as shown in Figure 2-3.

Rock instrumentation consisted of mechanical extensometers, surface markers, and grouted rebars. Double-position mechanical extensometers were placed between Anchors 1 and 2, and Anchors 3 and 4. Surface markers and rebars embedded in rock were used to monitor ground surface deformations. The locations of all rock instrumentation are shown in Figures 1-5 and 2-3. Rock instrumentation was installed in Anchors 1 through 4 only. The results obtained during the first stage of the test program provided sufficient information; therefore, this instrumentation was deleted in Anchors 5, 6, and 7.

Borehole permeability test instrumentation consisted of the Weksler pressure gages and the Rockwell flow meters.

A detailed description of all instrumentation is given in Appendix B.

2.5 REQUIREMENTS OF TEST PROGRAM

2.5.1 Test Anchors

The original scope of the test program included two multistrand and two multiwire anchors. One anchor of each pair was 45 ft long and the other 55 ft long. All four anchors were grouted with MB 814 grout. The failure of the multistrand anchors at stress levels lower than the guaranteed ultimate tensile strength of the strands and the questionable performance of the grout mix necessitated extending the test program. Three multiwire anchors were added; the first anchor was 45 ft long and the other two were 55 ft long. The 45 ft long anchor was grouted with the MB 814 grout, as were the first four, except that a higher water cement ratio was used. The last two anchors were grouted with the Type II+N cement grout.

The behavior of each anchor was determined from tendon elongation and grout column movement data obtained during stressing. These data are presented and discussed in Section 3 of this report.

2.5.2 Grout Tests

Specification NAS-30163 in Appendix A provides the specific grout testing requirements. The results of these tests are presented and discussed in Section 3.

Prior to the start of the test program, the strength, time of setting, expansion, bleeding, and flow characteristics of the grouts were determined by the McCallum Testing Laboratories, Inc. Time-strength relationships of both grouts were determined with mortar cubes by the Stone & Webster Field Quality Control Laboratory in accordance with ASTM C109. (See Appendix A.) The time-strength relationships were used to determine when anchor stressing could begin.

A separate 45 ft deep borehole (Grout Test Hole 1) was drilled and filled with MB 814 grout for the purpose of obtaining core samples to determine time-strength relationships of the grout subjected to the in-situ conditions. Grout Test Holes 2 and 3, approximately 25 ft deep, were drilled; one was filled with Type II+N grout and the other with the MB 814 grout. Core samples of the grout-rock interface were taken to visually determine the rock-grout bond characteristics. Petrographic analyses of thin-sections, made from the core samples, were also conducted to better evaluate the bonding characteristics of the grouts. The results of the petrographic analyses are discussed in Section 3 and a detailed report is included in Appendix D.

In addition, two grout encapsulation tests and one grout column test were performed. The encapsulation tests were performed to evaluate the effectiveness of the tremie grouting procedures to encapsulate the tendons. The grout column test was performed on the Type II+N grout to evaluate the effect of the expansive agent on the strength and bleeding characteristics of the grout mix. The results of the above tests are presented and discussed in Section 3.

2.5.3 Measurement of Rock Movements

The effect of anchor stressing on the in-situ rock was determined for the first four anchors in the following manner:

1. Double position mechanical extensometers were installed between each pair of anchors to determine the amount of rock movement between the top of rock and the top of the bonded length of the tendon.

2. Surface markers were installed on each concrete pad to determine displacement of the pad.

3. Rebar dowels were grouted several feet into rock at distances of approximately 10, 30, and 50 ft from each pair of anchors to determine surface rock movements.

2.5.4 Rock Quality Parameters

As indicated earlier, core samples were obtained from the anchor and the grout test boreholes to determine percent of recovery and the RQD. Falling head and water pressure tests were performed before and after preliminary grouting to determine the permeability of the rock and the effectiveness of the preliminary grouting prior to anchor installation. Direct shear tests were performed by the Stone & Webster Geotechnical Laboratory to determine the shear strength of the major joint sets.

2.5.5 Miscellaneous Tests

Specification NAS-30163 required that certain quality requirements be met for lake and groundwater used in the grout mix and the lubricants for the tendons. The results of these tests are discussed in Section 3.

2.6 ANCHOR CONSTRUCTION

Each test anchor was stressed against the reaction of a reinforced concrete pad as shown in Figures 2-1 and 2-2. The concrete pads were 4 ft thick, the anticipated design thickness of the gallery mats. Each pad contained an 8 in dia steel pipe sleeve to simulate the permanent rock anchor mat penetrations. A detailed description of the anchor construction procedures is given in Appendix B. A brief summary of the procedures is given below:

1. Anchor holes were drilled to obtain a continuous rock core sample for the full depth, except for holes 6 and 7 where only the bonded length was cored. The holes were then reamed out to 6.5 in with a down-hole buttonhead bit. All anchor holes were overdrilled 18 in to allow drilling debris to settle below the bottom of the anchor. The holes were flushed using compressed air and then water.

2. Vertical alignment of each hole was measured as described in Appendix B. Deviations from the vertical were well within the maximum allowable deviation of three degrees.

3. Following completion of drilling, falling head and water pressure tests were performed in anchor holes 1, 2, and 3 to measure rock permeability.

4. All anchor holes were preliminary grouted with Type II Portland cement mixed with 6.5 gal of water per bag of cement to reduce the permeability of the rock surrounding the hole. Holes 5, 6, and 7 were pressure grouted for added assurance that the holes were watertight. However, the pressure-grouted holes did not have any appreciable increase in grout take, a further indication of the high rock quality.

5. The holes were then reamed to their original 6.5 in dia and water pressure tested. If the leakage rate from the borehole during the pressure test exceeded 0.001 gal per in dia, per ft of depth per min for 10 min, the specification required that the borehole be regrouted and retested until satisfactory results were achieved. The permeability requirement was met for all boreholes after one application of preliminary grout. The boreholes were flushed, first with compressed air and then with water, just prior to tendon installation.

6. Tendon installation began with inspection of the tendon system at the warehouse and again at the laydown area just prior to insertion in the borehole. Inspection items consisted of tendon condition, the temporary corrosion protection packaging, the PVC sheathing, unbonded corrosion protection, and steel cleanliness over the bonded length. Grout extensometers were attached to the anchor after inspection. The positions of the extensometers depended on the tendon length (see Table 2-1). The tendons were then carefully lowered into the borehole and were supported on the bearing plate (with shim stacks) to ensure a vertical tendon. Immediately after inserting the tendons, grout was placed, until the hole was filled. Samples were taken from each batch of grout for compressive strength testing. The grout was allowed to set for approximately 30 min and then the unbonded section of the grout column was flushed. Flushing was performed using a 1.0 in dia, side discharge metal pipe.

7. After the grout reached a minimum compressive strength of 3,000 psi, the anchors were stressed. The stressing procedures are discussed below.

2.7 ANCHOR STRESSING PROCEDURE

The stressing procedure specified in NAS-30163 (Appendix A) included four phases. Figure 2-4 shows the sequence of each phase. Each phase was designed to test a specific anchor component and simulate actual forces that could be placed on the permanent anchors during the plant's lifetime. For each phase, all increases and decreases in load were in increments of 50 kips, and instruments were monitored at each increment. Lift-offs were performed after lock-offs to determine the changes in prestress level in the anchor system.

Phase I, or the proof test, consisted of loading from 0 to 400 kips in 50 kip increments and locking off at 350 kips for a minimum of 24 hr. This phase simulated the stressing sequence anticipated for the permanent anchors. Hence, the behavior of the permanent anchors during installation, in particular, tendon elongation and grout column movements, can be estimated from the Phase I results. Phase II, or the performance test, involved tendon relaxation from 350 kips to 50 kips and then cyclic loading as shown in Figure 2-4 with lock-off at 350 kips for a minimum of 24 hr. This phase was designed to determine whether significant debonding at the grout-steel and the grout-rock interfaces occurred.

Phase III, or the cyclic load test, involved 10 cycles of loading from 350 kips to 400 kips and locking off at 350 kips for a minimum of seven days. This phase was designed to simulate cyclic loading resulting from potential seismically induced motion and lift-off checks during in-service inspection.

Phase IV, or the ultimate test, involved stressing from 350 kips to 500 kips as shown in Figure 2-4. This load was then held for 10 min and the test was considered complete. This phase was designed to test the ultimate strengths of the anchors and to verify the load range at which the tendon system began to yield. Upon completion of this phase, the anchor stressing system was dismantled.

Several changes were made in the stressing procedure during the course of the test program. A fifth phase was added to the test for Anchor 3 to determine the "weakest link" in the wire tendon system and to Anchor 4 to check failure and the type of failure of the strand tendon system. Phase III was increased for Anchors 5, 6, and 7 to 24 cycles of the 50 kip cyclic load.

The 24 hr lock-offs at the end of Phases I and II were performed to evaluate the ability of the anchor system to hold the 350 kip load. The subsequent phases of the test program were allowed to take place only if the prestress loss was less than 25 kips. If the prestress loss exceeded 25 kips, the load was restored to 350 kips and another 24 hr lock-off period was required. A second lock-off was required for Anchor 2.

2.8 QUALITY ASSURANCE REQUIREMENTS

To ensure compliance with the terms of the specification, the work performed by the Nicholson Anchorage Company was supervised by the Stone & Webster Field Quality Control Inspector and the Geotechnical Representative. Their specific duties are presented in detail in the Quality Assurance section of Appendix A.

SECTION 1

GENERAL INFORMATION AND REQUIREMENTS

The contents of Section 1 are as follows:

SCOPE

The Contractor shall furnish all supervision, labor, materials, tools, and equipment necessary to install and proof test four (4) posttensioned rock anchors in accordance with the procedures specified herein.

PROJECT SIZE, LOCATION, AND SITE CONDITIONS

North Anna Units 3 and 4 are each nominal 950-Mw, nuclear fueled, steam turbine generator units. The reactors are pressurized water type.

North Anna Power Station is located near Mineral, Virginia, in Louisa County, on the south bank of Lake Anna. The site is approximately 40-miles NNW of Richmond, Virginia, and 70-miles SW of Washington, D.C.

The site is graded to approximately El.-271-ft with access from a state road and also from a railroad siding serviced by the Chesapeake and Ohio Railroad.

DEFINITIONS

For the definition of the terms Contractor, Engineers, and Purchaser, see the General Conditions.

Various other terms used herein are defined, or mean, as follows:

Bidder	-	A company submitting a proposal to fulfill the requirements of this specification.
Approved	-	This word when applied by the Engineers to the Contractor's drawings or documents means that the Engineers do not request any change on the drawing or document on the basis that the Contractor retains the entire responibility for complete conformance with all of the specification's requirements.
Approved As Revised	-	These words when applied by the Engineers to the Contractor's drawings or documents mean that the Engineers believe the changes delineated thereon are necessary to be in conformance with the specification or with existing conditions. On the basis that the Contractor shall retain the entire responsibility for the material, the Contractor shall either: a. Incorporate the suggested changes into his drawing or document and resubmit to the Engineers, or b. Inform the Engineers that the suggested changes cannot be made without prejudice to the Contractor's responsibility under warranty.

SCHEDULE AND PROGRESS

The proof test program shall begin in the Unit-3 Quench Spray Area. Continuity shall be maintained throughout the test program. The work shall be scheduled such that as soon as one activity (e.g., drilling) is completed in the Unit-4 Quench Spray Area, the same activity shall be started in the Unit-3 Main Steam Valve House

immediately thereafter and at the direction of the 4.21
Engineers.

The Contractor shall submit with his proposal a 4.24
construction schedule which shall include realistic starting
dates, elapsed times, and completion dates to meet the 4.25
Contract Schedule.

As soon as the contract is placed, it shall be the 4.27
responsibility of the Contractor to promptly arrange for a 4.28
meeting with the Engineers' Senior Construction Site
Representative to discuss in detail the planning for yard 4.29
layout, and sequence and timing of his work, in order to 4.30
provide for the most expeditious and economical
construction.

The Contractor shall be alert to the presence and 5.3
requirements of other contractors and equipment on the job 5.4
during certain phases of the erection work and shall
cooperate with the other contractors as required for the 5.5
most expeditious and economical completion of the project as
a whole. 5.6

The Contractor will be required to adhere to the 5.8
Contract Schedule. To ensure this goal, the Contractor 5.9
shall maintain sufficient equipment and a labor force of
sufficient size and competence to complete all work on 5.10
schedule. If the Contractor fails to maintain the Contract 5.11
Schedule for reasons not the fault of the Purchaser, the 5.12
Engineers, or labor disputes, the Purchaser will hold the
Contractor responsible for returning to Contract Schedule, 5.13
with any additional expense to be borne by the Contractor.
No change in the Contract Schedule shall be made unless 5.14
approved, in writing, by the Engineers' Senior Construction 5.15
Site Representative by issuance and acceptance of a
Memorandum of Change to the contract.

FURNISHED BY THE CONTRACTOR 5.19

The Contractor shall: 5.22

1) Furnish all necessary materials, tools, equipment, 5.24
facilities, and supplies to satisfactorily deliver, 5.27
assemble, install, and stress the four proof test
anchors, as specified herein.

2) Furnish the grout mixer and pump, as specified in 5.29
"Accessories for Grouting" 5.30

3) Furnish wooden boxes to store rock cores. 6.2

4) Furnish 6-1/2-in. ID pipe sleeve to be placed at 6.4
anchor location in the concrete mat. 6.5

5) Furnish, design and install tell-tale markers and 6.7
extensometers. 6.8

SUBMITTAL OF TECHNICAL DATA 6.11

The Contractor shall: 6.14

1) Submit with bid for the Engineers' approval the 6.16
manufacturer and type of tendon, anchorages, grout 6.17
pipes, all lubricants for the tendon components and
drilling equipment, free end extension of 6.18
prestressing steel, pullout test equipment and
accessories, and instrumentation for all required 6.19
measurements.

2) Submit to the Engineers for approval, at least two 6.21
weeks prior to the start of the test program, the
name of the testing firm and personnel, the 6.22
technique and equipment, and certificates of
calibration for the hydraulic jacks, pressure 6.23
gages, load cells, long range dial indicators, 6.24
flowmeters, packers, and instrumentation for
measuring the alignment of the boreholes and the
height of the grout column inside the borehole. 6.25

3) Submit with bid for the Engineers' approval the 6.28
methods and equipment proposed to:

   a. Measure the height of grout column during all 6.30
   grouting operations, as specified herein; 7.1

   b. Insert the tendon assemblies into the borehole 7.3
   and maintain the tendons at the center of the 7.4
   borehole without movement during the grouting
   procedure.

   c. Remove the grout pipes during or immediately 7.6
   after the completion of the grouting 7.7
   procedures.

4) Submit to the Engineers, at least two weeks prior to 7.10
the start of the test program, an analysis 7.11
indicating the anticipated elongations of the rock
anchor system during each increment of the load-
unloading cycles specified under "Stressing 7.12
Procedure."

5) Submit to the Engineers for approval, at least two 7.15
weeks prior to the start of the test program, the
name and experience record of the manufacturer's 7.16
prestressing specialist.

6) Provide the Engineers, at least two weeks prior to 7.18
the start of the test program, the certification of 7.19
Material Test Reports for the complete tendon and
anchorage components and accessories, as specified 7.20
in "MATERIALS."

7) Submit to the Engineers, no later than two weeks 7.22
after the completion of the test program, 12 copies 7.23
of a report summarizing the procedures and results
of the entire test program.

FURNISHED BY THE PURCHASER 7.26

The Purchaser will: 7.30

1) Furnish rights of access to all property on which 8.2
work is to be done prior to the commencement of 8.3
work on such property.

2) Furnish the results of previous subsurface 8.6
investigations and compression tests on the grout 8.7
mix.

3) Furnish the survey work required to locate and 8.9
determine ground surface elevations for the 8.10
boreholes.

4) Furnish all the grout ingredients, as specified 8.12
under "Grout." 8.13

5) Furnish Lake Anna water for washing the boreholes. 8.16

6) Furnish crane for moving equipment (Note: Contractor 8.19
remains responsible for providing labor, rigging,
and handling.

7) Drill holes for extensometers and surface markers. 8.22

REFERENCE TO STANDARD SPECIFICATIONS 8.26

The letters ASTM, ACI, PCI and CRD used herein, 8.29
refer to the American Society for Testing and Materials, the 9.2
American Concrete Institute, the Prestressed Concrete
Institute, and the U.S. Army Corps of Engineers,
respectively. It is intended that the revisions to standard 9.3
specifications in effect on the date the specification is
issued for bids shall apply, unless otherwise specified. 9.4

APPLICABLE DOCUMENTS 9.8

Various codes (and addenda thereto), standards, or 9.11
other documents that are mentioned by short-form name
elsewhere in this specification are fully identified below. 9.12

To the extent that these documents apply, as stated herein, the version of the document that is applicable shall be as itemized below. It is recognized that a later version of some of the dated documents may become mandatory under regulations that have jurisdiction. If this develops, the newer version of each document that changes will be specifically identified by means of an addendum to this specification; if the changed requirements in the newer document have a demonstrable effect on the cost to the Contractor of doing the work, an adjustment will be made in the Contract price.

If there is or seems to be a conflict between this specification and a referenced document, the matter shall be referred to the Engineers, who will clarify the matter in writing.

The various documents mentioned herein are as follows:

Short Name As Used Herein	Date of Issue	Complete Identification of the Document and of the Sponsor Organization
ACI 306	*	Recommended Practice for Cold Weather Grouting
ACI 359-74	*	ASME Boiler and Pressure Vessel Code
ASTM-A-416	*	Standard Specification for Uncoated Seven Wire Stress Relieved Strand For Prestressed Concrete
ASTM-A-421	*	Standard Specification for Uncoated Stress-Relieved Wire for Prestressed Concrete
ASTM-A-325	*	Standard Specification for High Strength Bolts for Structural Steel Joints Including Suitable Nuts and Plain Hardened Washers
ASTM-A-36	*	Standard Specification for Structural Steel
ASTM-D-2938	*	Test for Unconfined Compressive Strength of Rock Core Specimens

Short Name As Used Herein	Date of Issue	Complete Identification of the Document and of the Sponsor Organization
ASTM-C-109	*	Test for Compressive Strength of Hydraulic Cement Mortars (Using 2 in. Cube Specimens)
ASTM-E-328	*	Standard Recommended Practice for Stress-Relaxation Tests for Materials and Structures.
ASTM-E-18	*	Standard Methods of Test for Rockwell Hardness and Rockwell Superficial Hardness of Metallic Materials
ASTM-E-10	*	Standard Method of Test for Brinell Hardness of Metallic Materials.
CRD-C79	*	Method of Test for Flow of Grout Mixtures (Flow-Cone Method).
ASTM-C-494	*	Standard Specification for Chemical Admixtures for Concrete
ASTM-D-512	*	Standard Methods of Test for Chloride Ion in Water and Waste Water
ASTM-C-150	*	Standard Specification for Portland Cement
ASTM-C-151	*	Standard Method of Test for Autoclave Expansion of Portland Cement
ASTM-C-191	*	Standard Method of Test for Time of Setting of Hydraulic Cement by Vicant Needle.
ASTM-D-992	*	Standard Method of Test for Nitrate Ion in Water

*The issue (including addenda) in effect on date of Invitation to Bid.

TEMPORARY CONSTRUCTION UTILITIES 13.22

Electrical power and drinking water will be 13.25
provided to the Contractor at no cost, to the extent as 13.26
determined by the Engineers. All other utilities shall be 13.28
furnished by the Contractor. Lake Anna water to be used 13.29
with the grout mix and for washing boreholes shall be
provided by the Purchaser.

BORING LOGS AND SUBSURFACE DATA 14.3

A subsurface investigation of the site has been 14.6
performed. Boring and test pit logs, field and laboratory 14.7
test results, soil samples and rock cores, results of
refraction seismic surveys and the Engineers' 14.8
interpretations of these data are available for examination
at the Engineers' site office.

The above subsurface information shows the 14.11
conditions encountered at the particular location and time
they were obtained. Bidders and other users of the above 14.13
documents are cautioned that some conditions may change
between the time of a subsurface investigation and the 14.14
period during the execution of this contract, particularly
groundwater conditions. Conditions at the boring locations 14.15
may not reflect conditions prevailing elsewhere.

The Engineers have based their design on standard 14.17
procedures derived from interpretations and conclusions
drawn from the data contained in these reports. It shall be 14.19
the responsibility of the Bidder to: review the documents,
drawings, and reports and any addenda thereto; to 14.20
familiarize himself with the basis for the Engineers'
design; and to satisfy himself that the information is
accurate and acceptable for his bid and work. It is 14.22
recommended that the Bidder familiarize himself with the
site also. The Bidder shall assume all responsibility for 14.23
his conclusions based on the data furnished pertaining to
and affecting the method used in the performance of his work 14.24
and the selection of equipment required.

If during the course of the work, the Contractor 14.27
encounters conditions that differ from those indicated by 14.28
the subsurface investigation, he shall immediately notify
the Engineers in writing of the changed conditions. Failure 15.1
by the Contractor to make such a notification within 7-days
or in time to permit the Engineers to document the changed 15.2
conditions, whichever is less, shall preclude subsequent
claims from being made thereon. 15.3

If the Engineers concur that subsurface conditions 15.5
differ from those anticipated, immediate negotiations will
be undertaken to arrive at a change in contract price for 15.6

the additional or reduced work resulting from the unanticipated conditions. Changes in contract price brought about by such negotiations shall become effective from the date of written notification.

SUPPLEMENTAL DOCUMENTS

Documents that are equally important as the specification in delineating the requirements and conditions are as follows:

- The Contract
- General Conditions
- Supplementary Conditions of Contract
- Correspondence, Invoicing, and Shipping Instructions
- Contractor's Licensing Ordinance
- Project Insurances
- Federal Register - Vol. 36 No. 75 - April 17, 1971

COMMUNICATIONS

All letters by the Contractor to the Engineers shall consist of the original and four copies.

All communications pertaining to technical matters sent by the Contractor shall be addressed as follows:

Mr. W. M. Sweetser
Senior Construction Site Representative
Stone & Webster Engineering Corporation
Box 38
Mineral, Virginia 23117

Distribution for test reports, schedules, layout drawings, or written procedures shall be as follows:

Original and nine copies to:

Chief of Procurement
Procurement Quality Control Division
Stone & Webster Engineering Corporation
P. O. Box 2325
Boston, Massachusetts 02107

One copy to:

Superintendent of Field Quality Control
Stone & Webster Engineering Corporation
P. O. Box 38
Mineral, Virginia 23117

Two copies to:

Mr. W. M. Sweetser
Senior Construction Site Representative
Stone & Webster Engineering Corporation
P. O. Box 33
Mineral, Virginia 23117

One copy to:

E. H. McCallig
Project Engineer
Stone & Webster Engineering Corporation
P. O. Box 2325
Boston, Massachusetts 02107

Samples and specimens required by the Virginia Electric and Power Company and/or the Engineers shall be sent to the following address:

Superintendent of Field Quality Control
Stone & Webster Engineering Corporation
P. O. Box 38
Mineral, Virginia 23117

Contractor's drawings shall be sent to the Engineers at the job office for approval and shall be addressed as follows:

Mr. W. M. Sweetser
Senior Construction Site Representative
Stone & Webster Engineering Corporation
P. O. Box 38
Mineral, Virginia 23117

CONTRACTOR'S DRAWINGS

Where Contractor's drawings are required by the Engineers, the following shall apply:

The Contractor shall submit three good quality direct reading black and white contact prints and one reproducible made from Contractor's original drawing to the Engineers for approval with such promptness as to cause no delay in the Contractor's work. Submission of drawings by the Contractor and their return by the Engineers shall be in accordance with the schedule indicated. It is mandatory that these drawings be checked before submitting them to the Engineers.

The Engineers will return to the Contractor, as promptly as possible, one print of each of the Contractor's drawings with any necessary corrections and/or revisions. The disposition of each will be clearly marked.

Any specification or drawings heretofore or hereafter prepared by the Contractor in connection with the work shall be supplementary to the Engineers' specifications and drawings, and in case of discrepancy between the two, the Engineers' specifications and drawings shall take precedence, unless and to the extent that the Engineers shall otherwise specifically, in writing, direct.

The Engineers' approval shall not relieve the Contractor from responsibility for deviation from the Engineers' drawings or specifications unless he has, in writing, called the Engineers' attention to such deviations at the time of submission of drawings. The Engineers' approval shall be construed to apply to, and only to, general arrangement and shall not relieve the Contractor from entire responsibility for correctness of details and dimensions. Any work done in advance of the receipt of approved drawings shall be done entirely at the Contractor's risk and expense.

The Engineers reserve the right to reproduce any and all drawings or prints considered necessary for construction and/or Purchaser's purposes received from the Contractor on this contract despite any notice to the contrary prohibiting the same appearing on the print.

The number 12180/12181 and contract number shall be shown on each drawing.

RESULTS OF PROOF TEST PROGRAM

The Contractor shall provide the Engineers with twelve (12) copies of a complete and detailed report of the test program. The report shall include for each anchor, but not necessarily be limited to, mill certifications of all anchor materials used, logs of boreholes, records of the rock permeability tests, records of grouting procedure, plots of tendon elongation versus load, plots of load versus time elapsed, and comments on any difficulties or other pertinent data as encountered in the rock structure during drilling operations and stressing of anchors.

SUBCONTRACTORS

For the Contractor's responsibilities regarding subcontractors, see the General Conditions. Subcontractors, if any, shall be submitted with the Contractor's quotation and they shall be subject to approval by the Engineers.

To the extent that they apply, the Contractor shall impose on each of his contractors the complete requirements of this specification. He shall be completely responsible that the subcontractors are completely aware of all these requirements and that they abide thereby.

EXCEPTIONS

If the Bidder takes any exceptions to this specification, the exceptions shall be itemized and described in detail and included as an integral part of the proposal. However, no exception will be binding unless included as an addendum to this specification.

All exceptions must be specifically stated as such in Bidder's proposal.

The bidder shall be considered as meeting the requirements of this specification and his quoted price evaluated as such unless Bidder's exceptions are itemized.

SECTION 2

INSTALLATION OF TEST ROCK ANCHORS

The contents of Section 2 are as follows:

GENERAL

The posttensioned rock anchors shall consist of high strength steel tendons fitted with a stressing anchorage at one end and a grouted section to permit transfer of the stressing force to the peripheral rock at the other end. Certification of the tendons shall be provided by the Contractor as set forth in Section-4, "Quality Assurance Program," of this specification.

DEFINITION OF ANCHOR COMPONENTS AND STRESSING PROCEDURE

Terms to describe the components of the posttensioned rock anchors and the stressing procedure are defined as follows:

Posttensioning

Posttensioning shall refer to a method of prestressing in which the tendon is stressed to the required tension level after the grout has reached the required compressive strength.

Tendon

Tendon shall refer to the complete rock anchor assembly consisting of anchorage, prestressing steel, PVC pipe, smooth PVC sheath, grout pipes and miscellaneous accessories.

Anchorage

Anchorage shall refer to the means by which the stressing force is permanently transmitted from the prestressing steel to the concrete and consists of the anchorage head, bearing plates, wedges, shims, and buttonheads.

Prestressing Steel

Prestressing steel shall refer to the element of a posttensioning tendon which is elongated (unbonded length) and anchored (bonded length) to provide the necessary permanent prestressing force.

Bonded Length

Bonded length shall refer to the grouted portion of the tendon that transmits the prestressing force to the peripheral rock.

Unbonded Length 6.16

Unbonded length shall refer to the part of the 6.18
tendon that is free to elongate during the stressing
procedure.

Sheathing 6.22

Sheathing shall refer to the PVC material around 6.24
the prestressing steel in the unbonded length of the tendon 6.26
to provide corrosion protection and preclude bond between
the prestressing steel and the surrounding grout. The void 6.27
between the steel and pipe shall be filled with grease to
provide permanent corrosion protection.

Coating 6.30

Coating shall refer to the material used to protect 7.2
against corrosion and/or lubricate the prestressing steel.

Preliminary Grout 7.5

Preliminary grout shall refer to grouting the 7.7
borehole prior to inserting the tendon if necessary to
improve the permeability characteristics of the peripheral 7.8
rock.

First Stage Grout 7.11

First stage grout shall refer to the grouting of 7.13
the bonded length.

Second Stage Grout 7.16

Second stage grout shall refer to the grouting of 7.18
the annulus between the unbonded length of the tendon and 7.19
the peripheral rock.

Consolidation Grout 7.22

Consolidation grout shall refer to grouting after 7.24
the completion of either the first stage grout and/or second 7.25
stage grout to recover any grout level losses due to seepage 7.27
of the grout into the peripheral rock.

Lock-off 7.30

Lock-off shall refer to the transferral of the 8.2
stressing force from the hydraulic jack to the anchorage at 8.3
the completion of the stressing procedure and as specified 8.4
in the "Stressing Procedure."

Lift-off (Check)

Lift-off shall refer to checking the prestress force in the posttensioned anchor at any specified time with a hydraulic jack and a load cell.

Quiescent Time

Quiescent time shall refer to the amount of time that a sample of grout remains undistrubed in the flow cone.

Efflux Time

Efflux time shall refer to the amount of time that a sample of grout requires to run out of the flow cone after the plug is pulled.

ROCK TENDON TYPE

Tendons shall consist of prestressing steel, anchorage assembly, sheathing or PVC pipe, grout pipes, spacers, lubricants, and other miscellaneous accessories. The prestressing steel shall consist of both the 7-wire multistrand and the 1/4-in. diam wire multiwire systems. The tendons shall be as manufactured by the VSL Corporation, the Stress Steel Corporation, the PIC Corporation, the Prescon Corporation, the Inland-Ryerson Construction Products Company, or equal, as approved by the Boston Engineers.

LOCATION OF TEST PROGRAM

The test program shall be performed in the Unit-4 Quench Spray Area and Main Steam Valve House. The location of the test anchors is shown in sketch 12180-GSK-18. The anchors shall be located in the field by the Engineers.

EXISTING CONDITION AT TEST LOCATION

A temporary mat of 4.0-ft thick reinforced concrete will be placed prior to the proof test program. The Purchaser will provide the necessary blockouts through the mat to permit drilling and rock anchor installation.

MATERIALS

General

The Contractor shall provide Certified Material Test Reports (CMTR) for all tendon and anchorage components and accessories.

The CMTR shall include the following:

1. Certified reports of the actual results of all required chemical analyses, physical tests, and mechanical tests performed on the material.

2. A statement listing any chemical analyses, and tests required by the material specification which were not performed.

3. A statement giving the manner in which the material is identified, including specific marking.

Prestressing Steel

The prestressing steel used in the tendons shall consist of the 13-strand 7-wire steel conforming to ASTM Designation A416 and the 46-wire multiwire steel conforming to ASTM-A-421. The prestressing steel may be heat treated to enhance the stress-relaxation properties. Quenching and tempering treatments to produce specific mechanical properties are not permitted.

The physical properties of the strand are as follows:

Nominal strand diam	0.50 in
Nominal minimum ultimate tensile stress	270,000 psi
Nominal minimum breaking strength of strand	41,300 lb
Nominal minimum load (at 1.0 percent extension)	35,000 lb

The physical properties of the wire are as follows:

Nominal wire diam	0.25 in.
Nominal minimum ultimate tensile stress	240,000 psi
Nominal minimum breaking strength of wire	11,800 lb
Nominal minimum load (at 1.0 percent extension)	9,425 lb

Tendon Anchorage

Anchorage for the tendons shall consist of anchor heads, bearing plates, wedges, shims, buttonheads, and other miscellaneous items. Design of anchorage bearing plate shall be in accordance with the provisions of ACI-359-74. The concrete test pad shall be approximatley 5.5-ft-x-5.5-ft in plan, and 4-ft thick, for each anchor.

The Contractor shall submit with the bid for the Engineers' approval drawings showing the details of the anchor assemblies including the necessary data and procedure for proper installation, testing, and capping. These drawings shall be approved by the tendon assembly manufacturer prior to submittal to the Engineers.

Grout 12.7

The first and second stage grout and the 12.9
preliminary and consolidation grout, if necessary, shall be
made with a Type-II, low alkali Portland cement, 0.5-percent 12.10
by weight of an expansive agent, and a 0.44-water-cement 12.12
ratio by weight. The above agent shall be Interplast "N", 12.13
manufactured by Sika Chemical Corp., or similar material
recommended by the manufacturer of the anchor assemblies and 12.14
to be approved by the Engineers. This grout mix shall be 12.16
used for each bag of grout.

The grout mix is designed to reach a compressive 12.18
strength of at least 3,000-psi in 12-days. Preliminary 12.20
grout tests to verify the compressive strength-time
requirements have been performed and are available to the 12.21
Contractor for review. The Purchaser shall furnish all the 12.22
ingredients for the grout mix. The Contractor shall furnish 12.23
the equipment and shall mix these ingredients, as specified
herein, to meet the above requirements. Compression tests 12.25
of the grout shall be performed by the Engineers to verify
the above requirements prior to stressing the tendons.

Grout Pipes 12.27

The grout pipes shall be made of a material that 12.29
will not be significantly affected by temperature and with 12.30
sufficient strength to retain its shape and resist
irreparable damage during installation and stressing of the
tendon. The Contractor shall ensure that the grout pipes 13.1
will be free of kinks and other structural defects prior to 13.2
and after installation into the borehole. Care must be 13.4
taken during the placement of the grout pipes to avoid
interference with the prestressing steel. The grout pipes 13.5
shall be sized to fit between prestressing steel and the
annular space around the tendon and to handle the grout mix 13.6
without clogging.

Lubricant 13.9

Temporary corrosion protection for the tendon and 13.11
anchorage components shall be a high quality, low friction 13.12
grease, to be approved by the Engineers. The temporary 13.14
coating shall allow the grout to bond uniformly to all parts
of the tendon steel. The coating shall be made to be easily 13.15
removable in the field with the use of nonchlorinated
pertroleum solvents for degreasing the bonded area portion 13.16
of the tendon and for the installation of field-attached
anchorages. The coating material shall be compatible with 13.17
the permanent corrosion coating. Alternately, the 13.18
Contractor may select to encase the prestressing steel with 13.19

a PVC material to provide corrosion protection during transportation and site storage.

The permanent corrosion coating in the unbonded portion and the free end extension of the tendon 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.

The lubricants used for temporary and permanent corrosion protection shall meet the requirements under "Performance Tests on Materials."

Performance Tests on Materials

Prestressing Steel

All mechanical tests on the prestressing steel shall be performed on full-diam test pieces.

The prestressing steel shall be in accordance with ASTM Designation A-416 and ASTM-A-421 for the strand wire types, respectively. The tensile strength, yield strength, elongation, and other pertinent data shall be reported in the CMTR.

The stress relaxation properties of the prestressing steel shall be in accordance with ASTM-E-328. A minimum of three relaxation tests of 1,000-hr duration shall be performed. The following data shall be reported in the CMTR:

1) Detailed test method
2) Initial stress
3) Final stress
4) Test time
5) Temperature limits
6) Mathematical tools used to interpret test results
7) Percentage stress relaxation properties for design life

In lieu of performing the stress relaxation tests specifically for the prestressing steel required in this specification, it will be acceptable to furnish test results performed on materials previously manufactured to the above requirements and produced in the same plant utilizing the same procedures that will be employed to produce the prestressing steel required for this proof test program.

Additional tests required on the prestressing steel 15.5
are the following:

1) One static tensile test 15.9
2) One high cycle dynamic tensile test 15.10
3) One low cycle dynamic tensile test 15.11

The above test requirements shall be as indicated 15.15
in Sections-CC-2462 to CC-2466.3 of the ACI Standard-359-74, 15.16
"Code for Concrete Reactor Vessels and Containments." The 15.17
above test results shall be reported in the CMTR.

Loss of prestress due to frictional losses 15.19
resulting from intended or unintended curvature in the
tendons shall be based upon experimentally determined wobble 15.20
and curvature coefficients. These coefficients shall be 15.21
reported in the CMTR and shall be verified during the
stressing operation.

Tendon Anchorage 15.24

The tendon manufacturer shall select the materials 15.26
for the anchorage components to be compatible with the 15.27
tendon system. The anchorage assembly shall have suitable 15.29
physical properties to fully develop the minimum guaranteed
ultimate strength of the tendon assembly and shall be 15.30
fabricated of steel which will not exceed the anticipated
set of the anchorage components when tested to 95-percent of 16.1
the ultimate strength of the tendon assembly.

Materials for anchorhead assemblies, wedge blocks, 16.3
and wedges shall conform to the hardness test requirements 16.4
of Sections-CC-2433.1 to CC-2434.2.3 of ACI-Standard-359-74.
The test procedures shall conform to ASTM E-18 for Rockwell 16.5
hardness testing and to ASTM-E-10 for Brinell hardness
testing.

Anchoring assemblies utilizing wedges for anchor 16.7
lock-off shall consist of wedges capable of sustaining the 16.8
tensile loads during the test program to ensure against 16.9
unseating of the wedges after lock-off. Anchoring 16.10
assemblies utilizing buttonheads for anchor lock-off shall 16.11
consist of buttonheads capable of sustaining the tensile
loads to ensure against rupture of the buttonheads. 16.12

The CMTR for the anchorage materials shall include 16.14
the following:

1. Results of tests to ensure the compatability between 16.17
the tendon and anchorage systems throughout all 16.18
loading conditions.

2. Results of hardness tests on anchorhead assemblies, 16.22
wedge blocks, and wedges. 16.23

3. Results of tensile tests on the wedges to ensure the 16.26
permanent transfer of the prestress loads to the 16.27
wedges after lock-off.

4. Results of tensile tests on buttonheads to ensure 16.29
transfer of the prestress loads without rupture. 16.30

Grout 17.3

Cement for the grout mix shall conform to the 17.5
requirements of ASTM-C-150.

The fluidity of the grout shall be measured in 17.8
accordance with CRD-C79. The efflux time at the pump 17.9
discharge shall not be less than 11-sec at zero quiescent
time and shall not increase less than 3-sec nor more than 17.10
8-sec at 20-min of quiescent time. These time measurements 17.11
shall be made in the field with a stopwatch.

Bleeding of the grout mix at 65-F shall not exceed 17.13
2-percent of the volume three-hours after mixing or a 17.14
maximum of 4-percent. In addition, the separated water must 17.15
be absorbed after 24-hr. Bleeding shall be measured in a 17.16
metal or glass cylinder, approximately 4-in. I.D., with a 17.17
height of grout of approximately 4-in. During the test, the 17.18
container shall be covered to prevent evaporation.

Compressive strength tests of the grout mix shall 17.20
be performed in accordance with ASTM-C-109 and ASTM-D-2938, 17.21
as specified under "Grout Tests."

The chloride and nitrate contents of the grout mix 17.23
shall not exceed the following limits:

1. Chlorides as Cl- -300 ppm (parts per million) 17.27

2. Nitrates as NO- -100 ppm 17.29

The above required tests on the grout ingredients 18.3
and grout mix shall be performed by the Engineers or by an 18.4
approved laboratory at the direction of the Engineers.

The expansive agent used in the grout mix must be 18.6
free of chlorides. The Engineers will require submittal 18.7
from the Manufacturer of chemical analyses results. The 18.9
expansive agent shall conform to the requirements of
ASTM-C-494.

Mixing Water

Mixing water shall be clean with a total solids content of not more than 2,000-ppm as measured by the American Public Health Association's "Standard Method for Determination of Total Solids." The mixing water shall contain not more than 250-ppm of chlorides as Cl- as determined by ASTM-D-512. In addition, a comparison of the proposed mixing water properties shall be made with distilled water in accordance with ASTM-C151, ASTM-C-191, and ASTM-C-109, as specified in Section-CC-2223.2 of ACI Standard-359-74.

The above required tests shall be performed by the Engineers or by an approved laboratory at the direction of the Engineers.

Lubricants

The lubricants used to provide temporary and permanent corrosion protection for the tendon and anchorage components shall be analyzed for the presence of water-soluble chlorides, nitrates and sulphides. The lubricants shall not exceed the following chemical contents in accordance with the listed methods of testing:

Compound	Max. Quantity (ppm)	Method
Chlorides, Cl^-	10	ASTM D 512 (limit of accuracy: 0.5 ppm)
Nitrates, NO $^-$	10	ASTM D 992 (limit of accuracy: 0.5 ppm)
Sulphides, $S^=$	10	APHA - Test Methods, Sulphides in Water (limit of accuracy: 1.0 ppm)

The coating manufacturer shall provide for evaluation and acceptance by the Engineers' test data verifying that the following properties are met:

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 20.18
materials such as concrete, tendons, and ducts, and 20.19

5. Moisture displacing characteristics. 20.21

Marking and Identification of Samples 20.24

All materials used in the tendon and anchorage 20.26
assemblies shall be marked or tagged in such a manner to 20.27
ensure traceability to the CMTR during production and while 20.29
in transit and storage. If the original identification 20.30
markings are cut out or the material is divided into two or
more pieces, the markings shall be accurately transferred to 21.1
the pieces prior to cutting, or a coded marking or other
means of control shall be used to ensure identification of 21.2
each piece of material during the subsequent assembly. A 21.3
tabulation shall be made in the CMTR for the materials used
in an assembly or group of assemblies.

Upon completion of fabrication into a whole or 21.5
partial tendon, the tendon shall be identified with a tendon 21.6
number. The materials in the tendon shall be recorded so 21.7
that they can be traced to the tests which have determined
their quality. 21.8

Conveyances of bulk cement shall be sealed and 21.10
tagged before leaving the place of manufacture, showing lot
number, controlling specification, date of manufacture, and 21.11
type. If bag cement is used, each shipment shall be tagged 21.12
with the same identification as for bulk cement. All tags 21.13
and marking shall be maintained with the material at site
storage.

All containers of the expansive agent shall be 21.15
clearly marked, showing storage requirements and controlling 21.16
specification.

TENDON LENGTH 21.20

Total length shall be measured from end to end of 21.23
the tendon. Each tendon type shall be provided with 45 and 21.24
55-ft long tendons.

The bonded length shall be 20-ft and 30-ft for the 21.27
45-ft and 55-ft tendons, respectively. The unbonded length 21.29
shall be 25-ft for all tendons.

The free end extension of the production tendons 22.1
shall not exceed 14-in. after stressing measured along the
tendon axis, from the face of the bearing plate. The 22.3
Contractor shall submit with the bid for the Engineers'
approval drawings, design criteria for the free end 22.4

extension of each tendon and accessories to perform the 22.5
required stressing. The 14-in. extension requirement of the 22.6
test tendons shall be waived upon submittal of the above 22.7
data and approval by the Lead Geotechnical Engineer.

CORROSION PROTECTION 22.11

Delivery and Storage of Anchor Assembly 22.14

The tendon and anchorage assemblies shall be 22.16
delivered to the site according to the recommendations of 22.17
the manufacturer. Care shall be taken during delivery to 22.19
preclude the development of corrosion and/or structural 22.20
damage to all components of the anchor assembly. Components 22.21
of the anchor assembly not intended to be used immediately
upon arrival to the site shall be stored in an area that 22.22
will inhibit corrosion development. This storage area shall 22.23
be approved by the Engineers. Temporary corrosion 22.24
protection shall be provided by the manufacturer in
accordance with the requirements under "Lubricant."

Each tendon and anchorage assembly shall be 22.26
inspected on two separate occasions: upon arrival at the
site and prior to installation to ascertain that the wires 22.27
are free from nicks, scoring, corrosion development, or 22.28
other damage, and that the jacking and anchor heads are in
satisfactory condition. If in the opinion of the Engineers 22.29
there are any defects or damage, the assembly or any part 22.30
thereof shall be returned to the manufacturer for repair or
replacement, at the direction of the Engineers.

Tendons 23.2

The permanent corrosion protection for the bonded 23.4
length of the tendon shall be grout. The annular space 23.5
between the tendon assembly and rock shall be grouted by
pumping through a grout pipe extending to a point not more 23.7
than 6-in. above the bottom of the hole. Pumping shall 23.8
continue until sufficient grout has been placed to ensure
filling the hole to the required depth. Grout shall be 23.9
placed in such a manner as to fill the entire space around
and between the prestressing steel inside the borehole. All 23.10
grouting shall be performed as specified under "Grouting."

The unbonded length of the tendons shall be 23.12
protected with a coating meeting the requirements for
permanent corrosion protection under "Lubricants," and shall 23.13
be encased in a smooth PVC sheathing. The sheathing shall 23.14
be wrapped tightly around the tendon to preclude the
entrapment of deleterious materials within the steel 23.15
strands. Alternately, the tendon Manufacturer may select to 23.16
encase the unbonded length with a smooth self-supporting PVC 23.17

pipe. The void between the prestressing steel and the pipe shall be filled with a grease injected under pressure to fully encapsulate the steel. The grease shall meet the requirements of permanent corrosion protection under "Lubricants." The PVC pipe shall be sealed at both ends to preclude the entrapment of deterious materials within the pipe and to ensure that the grease is maintained inside the pipe. The grout for the bonded length shall extend a minimum of 6-in. above the unbonded length. A seal shall be placed directly below the PVC pipe or sheath to prevent the migration of grout into the pipe or sheath. The diam of the seal shall be at least 0.5-in. smaller than the borehole diam to allow water to be displaced vertically during grouting.

Grout Encapsulation Test

A field test shall be performed to measure the level of grout encapsulation of the prestressing steel. Two complete 45-ft long anchor assemblies, including both the wire and strand type, prestressing steel, PVC pipe, grout pipes and other accessories, shall be placed inside two separate 6-in. I.D. self-supporting PVC pipes. The pipe shall be oriented vertically. After inserting the anchor assembly inside each pipe, the entire length shall be grouted as specified under "Grouting." After 12-days, and at the direction of the Engineers, the PVC pipe shall be sheared and removed. The level of grout encapsulation shall be determined and documented by the Engineers.

Surface Anchorage

All anchorage components at the rock surface shall be protected with a coating meeting the requirements of permanent corrosion protection under "Lubricants" and to be approved by the Engineers prior to delivery to the site.

EQUIPMENT

General

All equipment used for handling and placing the rock anchors shall not damage or deteriorate the tendon and anchorage, assemblies, and shall be approved by the Engineers.

Accessories for Grouting

Grout equipment shall be capable of continuous mechanical mixing that will produce uniform and thoroughly mixed grout, free of lumps and undispersed cement. Grout equipment shall prevent introduction of oil, air, or other

foreign substances into the grout. No loss of water from 25.8
the grout due to poor seals, connections or other causes
shall be permitted. A screen with 0.125-in. maximum clear 25.9
opening shall be used to sieve the grout before it is 25.10
introduced into the grout pump.

The grout mixer shall include a high-speed mixing 25.12
drum and low-speed agitating drum arranged such that the 25.13
agitating drum will hold approximately two complete mixes.
New mixes shall be added as the stored material is being 25.14
pumped into the borehole, thus assuring a continuous pumping
operation. A Moyno pump shall be used for placing the 25.15
grout. Once grouting is started, it shall be continuous 25.16
until completed for that hole. The grout equipment shall be 25.17
capable of continuously grouting each grout stage in less
than 20-min.

The grout mixer or pump shall be equipped with a 25.19
metering device to measure the quantity of grout used. The 25.20
Contractor shall maintain a record of the grout quantities
used in each borehole.

Accessories for Stressing and Testing Anchors 25.23

The Contractor shall at all times provide and 25.25
maintain in good working conditions a set of pullout test 25.26
equipment approved by the Engineers. The set shall consist 25.28
of: 1)--a suitable hydraulic jack with a capacity of not
less than 600-kips; 2)--a center bore in the ram for 25.29
installation of the jack concentrically over the 25.30
longitudinal axis of the tendons; 3)--means of attaching the
jack to the tendons; 4)--a hydraulic pump; 5)--load cells, 26.1
hydraulic pressure gages and long range dial indicators; and
6)--all other necessary accessories. The Contractor shall 26.2
also have available at the site duplicate accessories to
ensure a continuous stressing and testing operation for each
anchor in the event that one or all the required accessories 26.3
fail during the stressing and the testing operation. As a 26.4
minimum, the Contractor is required to maintain at the site
an additional jack and additional load cells, pressure gages
and long range dial indicators all of which are in good 26.5
working condition and meet the above requirements and all
others specified herein.

The jacks shall be capable of stressing equally and 26.7
simultaneously all stressed elements of the rock anchor. 26.8
Each jack shall be equipped with a pressure gage for 26.9
determining the jacking load. The gage shall have a reading 26.10
dial at least 6-in. in diam and capable of being read to
500-lb. Load cells, accurate to ±1.0-percent of the 26.12
stressing load, shall be used in conjunction with the 26.13

pressure gages on the jacks. The output from the load cells 26.14
shall govern the test stressing procedure.

The Contractor shall provide long range dial 26.16
indicators reading to an accuracy of a 100th of an inch to 26.17
measure the tendon elongations during the stressing
procedure. The long range dial indicators shall be placed 26.18
in front of the tendon and shall be oriented parallel to the
tendon axis.

The hydraulic jacks, pressure gages, load cells, 26.20
and long range dial indicators shall be calibrated to verify 26.21
the accuracy specified above. The Contractor shall provide 26.22
the Engineers, at least two weeks prior to the start of the 26.23
test program, the following items:

1) Calibration charts for the hydraulic jacks, 26.25
pressure gages, load cells and long range dial 26.26
indicators;
2) The name and qualifications of the testing 26.28
laboratory and/or personnel performing the 26.29
calibration; and
3) The technique and equipment used for calibration. 27.1

Accessories for the Rock Permeability Tests 27.4

The Contractor shall ensure that a complete set of 27.6
pressure testing equipment is available in good working 27.7
order at such times as the tests are required. All pumps, 27.10
packers, gages, flow meters, hoses, and related equipment 27.11
required for the pressure tests shall be kept available and 27.12
properly maintained so that this equipment will be ready for
testing at such times as the tests are required. The 27.14
Contractor shall not be compensated for time lost due to
equipment failure.

Pumps shall be of the positive displacement type 27.16
and shall have a capacity of not less than 25-gal per min 27.17
(gpm) at a discharge pressure of 100-lb per sq-in. (psi). 27.18

Packers and associated equipment shall be provided 27.21
so that either single or double packer tests can be per- 27.22
formed in boreholes of the sizes that may be drilled. 27.23
Packers shall be pneumatically expanded and shall be of 27.24
sufficient length, in the opinion of the Engineers, to
ensure a positive seal in the rock boring. Hydraulically 27.26
expandable packers may be used only if it can be 27.27
demonstrated that they can be satisfactorily deflated to 27.28
allow for easy withdrawal from the borehole. The packers 27.29
shall be field tested in a length of casing, as directed by
the Lead Geotechnical Engineer or his appointed
representative (Geotechnical Representative) and shall be 27.30

capable of withstanding a water pressure of 25-psi without leakage for a period of 5-min.

Flow meters shall read in gallons and shall be calibrated immediately prior to use on this project. The Contractor shall either furnish a certification of factory or laboratory calibration or shall calibrate the meters in the field as directed by the Geotechnical Representative. Field calibration checks shall be made before the tests and at regular intervals throughout the pressure testing work as directed by the Geotechnical Representative. The Contractor shall provide a container with a capacity of at least 20-gal, and approved by the Geotechnical Representative, for making these calibration checks. Pressure gages shall read in pounds per square inch and shall be calibrated immediately prior to use on this project. The Contractor shall furnish the Geotechnical Representative with a certification of calibration of the pressure gages before the start of any pressure testing. Flow meters and pressure gages shall be capable of registering over the full delivery range of the pumps.

INSTALLATION OF TEST ROCK ANCHORS

General

The type and length of tendons and anchorage to be used shall be as shown on the drawings and as specified herein.

Borehole Alignment

The orientation of the boreholes shall be vertical. The Contractor shall submit with the bid a detailed description of his proposed methods for maintaining the specified alignment and methods to measure and document the alignment of the holes. The tolerance for the borehole orientation shall be ±3 degrees. Instrumentation and methods proposed for measuring and documenting the alignment of the boreholes will be subject to the approval of the Geotechnical Representative.

If a borehole is rejected by the Geotechnical Representative due to failure of the Contractor to maintain the required alignment, or due to any fault or negligence on the part of the Contractor, the hole shall be backfilled with grout, at the Contractor's expense, and no payment will be made for drilling the rejected borehole. If a borehole is rejected due to circumstances beyond the Contractor's control, the borehole shall be filled with grout and payment shall be made in accordance to the applicable items under "Measurement and Payment."

Drilling Technique 29.18

Boreholes shall be drilled using a rotary type 29.20
drill with diamond bits. Holes shall be 6-in. in diam. The 29.24
entire length of the boreholes shall be cored first with an
NX-size split inner core barrel, as manufactured by 29.25
Christensen Diamond Products, or equal, as approved by the
Geotechnical Representative. Core runs shall not exceed 29.26
5-ft without the Geotechnical Representative's prior
approval. After the core samples have been obtained, the 29.27
boreholes shall be reamed to the full 6-in. diam.
Alternately, the Contractor can use conventional drilling 29.28
techniques to obtain full size core samples from the 6-in.
diam boring. Boreholes shall be drilled at least 18-in. to 29.29
no more than 24-in. deeper than required for the 29.30
installation of the anchor assemblies as directed by the
Geotechnical Representative. Any additional drilling beyond 30.1
the limits set by the Geotechnical Representative will be at
the Contractor's expense.

The rock core samples shall be placed in wooden 30.3
boxes, furnished by the Contractor, after the necessary 30.4
photographs and measurements are completed by the
Geotechnical Representative. The core boxes shall be stored 30.5
on site at the direction of the Geotechnical Representative.
Box design must be approved by the Geotechnical 30.6
Representative.

The use of "rod dope" or grease, or other 30.8
lubricants on the drill rods, tools, or in the hole will not
be permitted. If one is necessary, the Contractor shall use 30.9
a lubricant composed of bio-degradable organic polymers to 30.10
preclude a slippery surface buildup around the interior rock
walls of the borehole. The type and contents of the 30.11
lubricant must be approved by the Geotechnical
Representative, and verification of the above requirements 30.12
shall be required by the Engineers. The Contractor shall 30.13
obtain water samples from the borehole after drilling and
washing is completed. The Engineers shall determine if oil 30.15
or grease is contained in the water samples and if so, the 30.16
Contractor shall wash the hole until all the oil or grease 30.17
is removed. The method of washing shall be as approved by 30.18
the Geotechnical Representative.

The Contractor, in selecting the equipment and 30.20
methods of drilling for the rock anchor test program, shall 30.21
carefully consider the following:--1)--the limited concrete
cover around the borehole; and 2)--permanent rock anchors to 30.22
be installed later adjacent to the four proof test anchors.
Therefore, the Contractor shall select drilling equipment 30.23
and methods which will preclude cracking, spalling, or 30.24
possible failure of the concrete and possible damage to the

surrounding rock. The Bidders shall submit with bid the 30.25
type of drilling equipment and the methods of drilling.

Washing and Cleaning Anchor Holes 30.28

The 6-in. diam boreholes shall be thoroughly washed 30.30
and cleaned before anchor assemblies are installed. Drill 31.3
cuttings and slurry shall be removed by applying water to
the bottom of the hole and shall continue until the effluent 31.4
is clear of mud and chips. Washing shall be done 31.5
immediately after completion of drilling, and the hole shall
be suitably capped and protected to prevent entrance of 31.6
foreign material until anchor installation.

The drilled hole shall be checked by lowering a 31.8
full size bit to verify that it is clear to its full
required depth prior to insertion of the anchor assembly. 31.9
If the hole is found to be blocked, such debris or material 31.10
shall be removed by washing or by using a bailer. If these 31.12
procedures are not effective, the hole shall be redrilled to
remove the blockage and rechecked as described above.

Rock Permeability Tests 31.16

Falling Head Test 31.18

After the 6-in. borehole is drilled, all drilling 31.20
rods and bits shall be immediately removed from the
borehole. The borehole shall be completely filled with 31.22
water and the drop in water level shall be measured by the 31.23
Geotechnical Representative with an electrical water level 31.24
indicator. The frequency of readings shall be determined by 31.25
the Geotechnical Representative at the site. However, the 31.26
time allotted for these readings shall not exceed 30-min.

Water Pressure Test 31.28

After the completion of the falling head 31.30
permeability test, the borehole shall be completely filled
with water and the entire length of the borehole shall be 32.1
pressure tested in the following manner:

1. The section of the borehole between the packers or 32.3
below the packer shall be tested at a pressure of 32.4
25-psi.

2. The pressure shall be held essentially constant at 32.7
the required test pressure by varying the pumping
rate.

3. The total water take in gallons for a 10 min 32.9
interval shall be measured at the flow meter. 32.10

4. In the event that the maximum pressure cannot be developed due to a water take exceeding the capacity of the pump, the maximum test pressure shall be taken as that which can be maintained constant for the 10 min interval.

5. During the test, the Contractor will record the pressures and water takes and will also note any particular events such as erratic pressure or flow readings, water leaks around the packer, surface water leaks, rock uplift, and any other anomalous events.

Pressure Test Records

Water pressure test data recorded by the Contractor will be presented as two graphs on the attached Pressure Test Record (Form-T-12---4/70).

Preliminary Grouting

If the leakage rate from the borehole during the water pressure test exceeds 0.001 gallons per inch diam per foot of depth per minute for a 10 minute period, the borehole shall be preliminary grouted, redrilled and retested. Should the water pressure test exceed the above criteria, the entire process shall be repeated until the above criteria is satisfied. In addition, preliminary grouting shall be required if the leakage rate into the borehole during the falling head permeability test exceeds 0.5-gpm.

Tendon Installation Procedure

The tendon and anchorage assemblies shall be installed in the boreholes in accordance with the details and dimensions shown on the drawings and in accordance with the anchor manufacturer's recommendations. The Contractor shall select such equipment to suspend and install the tendons to prevent damage to the tendons and any part thereof. The Contractor shall provide for the Engineers' approval, at least two weeks prior to the start of the proof test program, the method and equipment proposed for installing the tendons.

The tendon and anchorage assemblies shall be free of dirt, loose rust, grease, or other deleterious substances when installed. The drilled holes shall be inspected to insure they are cleaned prior to insertion of the tendons and protected from entrance of dirt or other foreign matter until both the first and second stages of grout have been placed. Tendons shall be installed after the borehole has

been cleaned to the satisfaction of the Geotechnical
Representative.

The tendons shall be securely fastened inside the 33.27
borehole to prevent any movement during the grouting 33.28
procedure. Appropriate spacers shall be placed on 10-ft 33.29
centers, to center the tendons and ensure full grout
encapsulation. The edges of the spacers shall be smooth to 33.30
prevent damage to the strands and the grout pipes. Care 34.1
shall be taken during placement of the tendons into the
borehole to prevent the strands from interfering with the 34.2
grout pipes.

Grouting 34.5

Prior to commencing grouting, grout vent networks shall be 34.7
checked with compressed air or water to ensure that they are 34.9
clear. All grout piping shall be clean and free of 34.10
deleterious materials that would interfere with the grouting 34.11
procedure. Piping shall be thoroughly flushed and blown out 34.12
prior to grouting. The grout mixture shall be mixed for a 34.13
minimum time of 3-min by means of a high-speed mechanical 34.14
agitator. The grout shall be used as soon as possible after 34.15
thoroughly mixing all ingredients, but not later than 34.16
45-min. after the addition of water to the cement. Mixing 34.17
of grout shall be in accordance with the PCI "Recommended
Practice for Grouting of Posttensioned Concrete," Vol.-17, 34.18
No.-6, November-December, 1972. Under applicable 34.19
conditions, grouting shall be done in accordance with
ACI-306, "Recommended Practice for Cold Weather Grouting." 34.20
Mixing should be of such duration as to obtain a uniform 34.21
thoroughly blended grout, without excessive temperature
increase or loss of expansive properties of the admixture. 34.22
The grout shall be continuously agitated until it is pumped. 34.23

The bonded length shall be grouted first 34.25
immediately after insertion of tendons into the borehole. 34.26
The bonded length shall vary, as shown on the drawings and 34.27
as specified herein. Grout shall be injected from the 34.28
bottom of the borehole up to 6-in. above the start of the
unbonded length. The grout pipe used for the first stage 34.30
grouting operation shall be removed as grouting proceeds 35.1
upward or immediately after grouting is completed. If the 35.2
Contractor selects the former technique, the Contractor
shall ensure that the grout pipe is embedded in the grout at 35.3
least 12-in. at all times to preclude entrapment of air in
the grout. The Contractor shall ensure that grout is forced 35.4
into the drill hole to fully encapsulate the prestressing 35.5
steel and fill completely the annular space around the
tendons. A method to determine the height of the grout 35.6
during the grouting operation must be established by the 35.7
Contractor and approved by the Geotechnical Representative.

After the first stage grouting is completed, a barrier, such 35.8
as a large concrete pipe shall be placed around the surface
extension of the anchor to prevent disturbance of the tendon 35.9
prior to stressing. Alternately, the Contractor can grout 35.10
the bonded length through a tremie tube from the bottom
until clean grout overtops the hole at the surface. After 35.12
allowing a period of time to elapse to enable an initial set
of the cement to take place, the grout above the bonded 35.13
length shall be washed out using a flush pipe to be removed
after washing is completed.

The annular space around the unbonded length of the 35.15
anchor shall be grouted immediately after the stressing 35.16
procedure, as specified herein, is completed and at the
direction of the Engineers. Prior to commencing the second 35.17
stage grouting, the Contractor shall check all grout vent
networks and grout pipes to ensure that they are clean and 35.18
free of deleterious materials. The Contractor shall ensure 35.19
that grout is forced into the drill hole to fill completely
the annular space around the tendons up to and including the 35.20
surface anchorage, and that all air is vented out of the 35.21
drill hole. The grouting operation will be considered 35.22
successful when grout is returned full flow without air 35.23
bubbles through the vent pipe.

The grout level, after the first and second stage 35.25
grouting are completed, shall be checked every two hr to 35.26
measure grout consolidation. If the grout level is more 35.27
than 2-in. below the required level after each two-hr check, 35.28
additional grout shall be added to raise the grout to the 35.29
required level.

For both stages, the grout shall be vibrated by 36.2
attaching an electric vibrator to the grout pipe to ensure
full grout coverage around the tendon. Consolidation grout, 36.4
if required as described above, need not be vibrated.

<u>Testing Alternate Grouting Procedure</u> 36.7

The alternate grouting procedure for the bonded 36.9
length, consisting of initially tremie grouting the entire
borehole and then washing the upper portion, shall be tested 36.10
in the borehole selected for the in-hole grout tests 36.11
discussed below. The borehole shall be completely tremie 36.13
grouted. The upper grout shall be washed by methods 36.14
discussed under "Grouting". Accurate measurement techniques 36.15
to determine the top of the bonded length shall be
established during this test. Subsequent to washing down to 36.16
the bonded length, the in-hole grout test, below, shall be
performed.

Grout Tests 36.19

Two types of grout tests shall be made by the 36.21
Engineers to determine the compressive strength of the grout 36.22
before stressing the anchors:

1. Unconfined compression tests of grout cores, as 36.25
specified for rock cores in ASTM-D2938 , and 36.26

2. Cube tests, as specified in ASTM-C109 36.28

The bottom 25-ft of the drilled borehole for one of 36.30
the 45-ft anchor shall be filled with grout. The grout 37.2
shall be allowed 3-days to set. Then, the Contractor shall 37.3
obtain 5-ft cores, in accordance with the equipment and
methods specified in "Drilling-Technique," after 3, 7, 12, 37.4
and possibly 14-days after the grout was placed.
Representative sections at the top and bottom of each 5-ft 37.5
core shall be tested in accordance with ASTM-D2938.
Alternately, a separate borehole could be drilled for the 37.6
above tests at the discretion of the Engineers. In addition 37.7
to the above tests, samples of the grout used in the
borehole shall be tested in accordance with ASTM-C109 to
obtain the 3, 7, 12, 14, and 28-day compressive strengths. 37.8

The Engineers' Inspector shall obtain eight samples 37.10
per batch of grout to be used for the bonded length. These 37.12
samples shall be tested by the Engineers in accordance with
the procedures specified in ASTM-C109 to obtain the 3, 7, 37.13
12, 14, and 28 day compressive strengths. Stressing of the 37.14
anchors shall begin after the required compressive strength
of the grout has been achieved and at the direction of the 37.15
Engineers.

Instrumentation 37.18

In addition to the long-range dial indicators to 37.20
monitor tendon elongation, the Contractor shall provide and 37.21
install instruments to monitor movements of the rock at
depth and of the grout in the bonded length. The 37.23
instruments shall consist of tell-tale markers,
extensometers, and surface markers.

The tell-tale markers shall consist of a 0.25-in. 37.25
diam wire encased in a suitable PVC tube and encapsulated 37.26
with a high quality, low friction lubricant, to be approved
by the Engineers. The position of the wire tip in the 37.27
primary grout shall be as indicated in 12180-GSK-19. The 37.28
wire shall be fastened at several locations to the unbonded
length of the tendon. The design of the protrusion of the 37.29
wire above the bearing plate shall be as approved by the
tendon manufacturer.

Extensometers shall be the single- and/or double-position mechanical type and shall be located as shown in 12180-GSK-19. Two, single-position extensometers shall be installed to a depth of 25-ft. In the event that the tendon manufacturer cannot incorporate the tell-tale markers into the present tendon design, then, four, double-position extensometers shall be installed as shown in 12180-GSK-19. The Engineers shall drill the holes required to install the extensometers.

Surface markers, consisting of steel bars grouted into rock, shall be installed by the Engineers, as shown in 12180-GSK-19.

Elevation readings of these instruments shall be made by the Engineers. Readings shall be taken subsequent to each load increment and/or decrement.

Stressing Procedure

Stressing the anchors shall begin after the grout in the bonded length has reached the required compressive strength, as determined by the Engineers, but no sooner than 12-days after the first stage grouting has been completed. The anchors shall be stressed with a hydraulic jack, as specified herein. The stressing process shall be conducted to allow accurate measurements of tendon elongation, stressing load (jack gage pressure and load cell reading) and elapsed time for each increment of load. These measurements shall be made with high precision calibrated instruments, as specified in "Accessories for Stressing and Testing Anchors." All loading and unloading shall be done in 50-kip increments. Increments of load and/or unload shall be applied when the measured elongations of the tendons have stabilized under the previous increment of load and/or unload and at the direction of the Geotechnical Representative.

The stressing procedure for all tendons (the stressing schedule is shown in 12180-GSK-17):

1. Load tendon from zero kips to a maximum of 400-k.

2. Unload tendon from 400-k to 350-k and lock off tendon at this load.

3. Check load after 24-hr. If prestress loss is less than 25-k, restore load to 350-k, and proceed with step-6. If loss exceeds 25-k, restore load to 350-k and hold for a further 24-hr.

4. Repeat step 3. If loss of prestress load again 39.15
exceeds 25k, then the anchor shall be rechecked at 39.16
24-hr intervals at the direction of the Geotechnical Representative to assess whether the loss of prestress is likely to continue.

5. If the anchor shows a continuous loss of prestress, 39.18
the tendon shall be allowed to relax and will be 39.19
monitored continuously, at the direction of the Geotechnical Representative until a constant load is obtained.

6. If, in step 3 or 4 for both anchors, the prestress 39.21
loss is less than 25k, proceed as follows: 39.22

   a. Unload tendons from 350 k to 50 k. 39.24

   b. Load tendons from 50 k to 100 k. 39.26

   c. Unload tendons from 100 k to 50 k. 39.28

   d. Load tendons from 50-k to 150-k. 39.30

   e. In a similar manner, alternately load and 40.2
   unload from 50-kips to 200, 250, and 300-kips 40.3
   and back to 50-k.

   f. Load tendons from 50-k to 400-k. 40.5

   g. Unload tendons from 400 k to 350 k. 40.7

   h. Proceed with step 3 and 4, or 5 40.9

   i. If steps 3 and/or 4 are satisfied proceed with 40.11
   step 7. 40.12

7. After the above prestress loss criteria is 40.14
satisfied, proceed as follows: 40.15

   a. Perform 10-cycles of loading from 350-k to 40.17
   400-k and back to 350-k, and lock-off. 40.18

   b. Perform lift-off seven-days after above lock-off. 40.21

   c. Load incrementally from 350-k to 500-k and hold 40.24
   this load for 10-min.

   d. Reduce load to zero. 40.26

The Contractors shall provide the Engineers, at 40.28
least two weeks prior to the start of the test program, an 40.29

analysis indicating the anticipated elogations of the tendons during each increment of the above load-unloading cycles. If the observed elongations deviate from this analysis at any cycle by more than 10-percent during the stressing procedure, an investigation shall be undertaken to determine the cause of the deviation and the effect it may have on the final anchor design.

Safety precautions shall be taken to prevent workers from standing behind the jacks or near the hydraulic pressure lines during the stressing procedure.

Measurements

The elongations of and the prestress loads applied to the anchor assembly shall be monitored by the equipment and methods under "Accessories for Stressing and Testing Anchors" and "Stressing Procedure."

Free Ends of Tendons

The free end extension of the tendons after stressing shall be less than 14 in. from the bearing plate, measured along the tendon axis. The free end shall be protected against damage or corrosion and shall be approved by the Engineers prior to implementation.

Manufacturer's Specialist

The Contractor, at his expense, shall arrange to have a qualified manufacturer's specialist, skilled in tensioning work, at the site during the prestressing procedure to instruct, train and advise the Contractor's personnel in the proper execution of the work. Two weeks prior to the start of the test program, the Contractor shall furnish to the Engineers the experience record of the manufacturer's specialist for approval.

MEASUREMENT AND PAYMENT

Mobilization and Demobilization

A lump sum will be paid for mobilization and demobilization in accordance to the Bidding Schedule. The contract lump sum shall be full compensation for moving in all materials, tools, equipment, facilities, and supplies necessary for performance of the specified work and removal of the same from the site at the completion of the work. No separate payment will be made for relocating or moving any or all of the above accessories within the project site after moving equipment to the site. No separate payment will be made for mobilization and demobilization of

equipment used for extra work not specified in this contract 42.19
provided that payment for mobilization and demobilization of
the specific equipment has already been included under this 42.20
item and that the specific equipment has not in fact been
demobilized and removed from the site.

Rock Drilling and Sampling 42.23

The unit of measurement for rock drilling and 42.25
sampling the 45 and 55-ft boreholes shall be the linear
foot. An additional 45-ft borehole for the in-hole grout 42.26
tests may be required by the Engineers. The unit price 42.28
shall be full compensation for drilling and recovering NX-
size core samples with the split inner barrel and subsequent 42.29
reaming of boreholes to full 6-in. diam. A separate unit 42.30
price shall be submitted for drilling and recovering full
core samples from the 6-in. diam boring with conventional 43.1
drilling equipment.

Test Grout in Borehole 43.4

The unit of measurement for the test grout shall be 43.6
the cubic foot of grout pumped into the borehole. The unit 43.9
price shall be full compensation for grouting the bottom
25-ft of one borehole.

Drilling and Sampling Test Grout 43.11

The unit of measurement for drilling and sampling 43.13
the test grout in the borehole shall be the linear foot.
The unit price shall be full compensation for drilling and 43.14
recovering NX-size grout cores with the split inner barrel. 43.15
The length of each core run shall be 5-ft. Core recoveries 43.16
shall be made after 3, 7, 12 and possibly 14-days after
grouting. 43.17

Rock Permeability Tests 43.19

The unit of measurement for the rock permeability 43.21
tests shall be the hr. The unit price shall be full 43.22
compensation for performing the falling head test and the 43.23
water pressure test at the direction of the Geotechnical
Representative. Payment shall start at the beginning of 43.24
first reading and terminate at end of last reading.

Preliminary Grouting of Boreholes 43.27

The unit of measurement for grouting boreholes 43.29
which exceed the seepage requirements under "Rock 43.30
Permeability Tests" shall be the cubic foot. The unit price 44.2
shall be full compensation for grouting the entire borehole
at the direction of the Geotechnical Representative.

Drilling Grouted Boreholes

The unit of measurement for drilling grouted boreholes shall be the linear foot. The unit price shall be full compensation for fully drilling a grouted borehole.

Tendons and Installation

The unit of measurement for the tendons and installation thereof shall be the linear foot. The unit price shall be full compensation for satisfactory completion of the following:

1. Furnishing the tendon components to the site,
2. Assemblage of the tendon components at the site,
3. Cleaning and washing borehole before insertion of tendons,
4. Insertion of the full tendon assembly into the borehole,
5. Grouting of the total tendon length, and
6. Stressing of the tendons as specified herein.

Consolidation Grout

The unit of measurement for grouting boreholes which exceed the grout consolidation requirements specified herein shall be the cubic foot of grout. The unit price shall be full compensation for regrouting to the required levels after the completion of the first and second stage grouting.

Standby Time

Equipment

The unit of measurement for equipment stand-by time shall be the day. The unit price shall be full compensation for delays during the period starting after the completion of the first stage grouting to the start of the stressing procedure and as directed by the Engineers.

Crew and Equipment

The unit of measurement for crew and equipment stand-by time shall be the day. The unit price shall be full compensation for delays at any time during the installation and testing of the anchors where the

anticipated delay would not warrant removal of the labor
forces from the site and as directed by the Engineers. 45.25

Grout Encapsulation Test 45.28

A lump sum will be paid for the grout encapsulation 45.30
test. The contract lump sum shall be full compensation for: 46.1

1. Furnishing two complete 45-ft long tendon assembly 46.4
   along with a 45-ft long 6-in. I.D. PVC pipe to the 46.5
   site,
2. Grouting the entire length, and 46.7
3. Extracting the entire tendon assembly from the pipe. 46.10

Instrumentation 46.13

A lump sum will be paid for the design, 46.15
procurement, and installation of the required tell-tale 46.16
markers and extensometers.

BIDDING SCHEDULE

Item No.	Description	Unit of Measurement	Estimated Quantity	Unit Price	Estimated Total Price
1.0	Mobilization and Demobilization	L.S.	1		
2.0	Rock Drilling & Sampling				
2.1	Split inner barrel	L.F.	100		
2.2	Conventional	L.F.	100		
3.0	Test Grout in Borehole	C.F.	5		
4.0	Drilling and Sampling Test Grout	L.F.	25		
5.0	Rock Permeability Tests	Hr.	8		
6.0	Preliminary Grouting of Boreholes	C.F.	--		
7.0	Drilling Grouted Boreholes	L.F.	--		
8.0	Tendons and Installation				
8.1	Multistrand	L.F.	100		
8.2	Multiwire	L.F.	100		
9.0	Consolidation Grout	C.F.	--		
10.0	Standby Time				
10.1	Equipment	Day	--		
10.2	Crew and Equipment	Day	--		

Item No.	Description	Unit of Measurement	Estimated Quantity	Unit Price	Estimated Total Price	
11.0	Grout Encapsulation Test					2.11 2.12
11.1	Multistrand	L.S.	1			2.14
11.2	Multiwire	L.S.	1			2.16
12.0	Instrumentation	L.S.	1			2.19

SECTION 3

DRAWINGS AND SKETCHES

The contents of Section 3 are as follows:

ENGINEERS' DRAWINGS AND SKETCHES

The extent and location of work to be performed shall be as shown on the following Engineers' Drawings:

Drawing No.	Title
12180-SS-39	Typical Rock Anchor
12180-GSK-18	Stressing Schedule of Test Program in the Unit 4 Containment Auxiliary Structures
12180-GSK-19	Instrumentation for Rock Anchor Test Program
12180-GSK-20	Location of Rock Anchor Test Program in the Unit 4 Containment Auxiliary Structures
Form T-12-4/70	Pressure Test Record

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

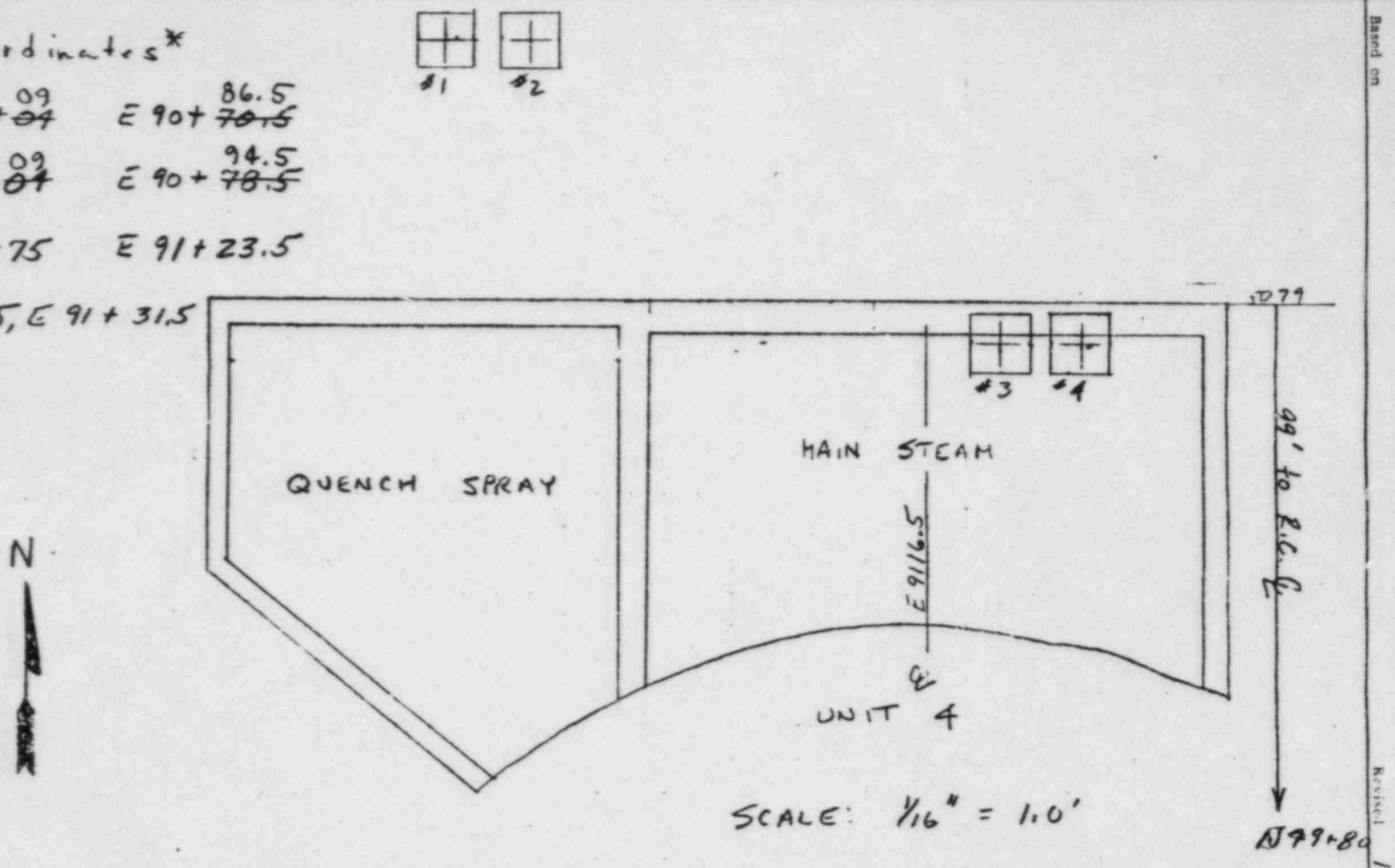
▲ 5010.10

Page No. ____
Preliminary ____
Item ____

Client VEPCO
Subject LOCATION OF ROCK ANCHOR TEST PROGRAM IN UNIT 4
Location NA 3 & 4 Est. No. J.O. No. 12180/81
Date 10/28/75 By
Checked By
Revised 11/17/75 By
Based on

Coordinates*

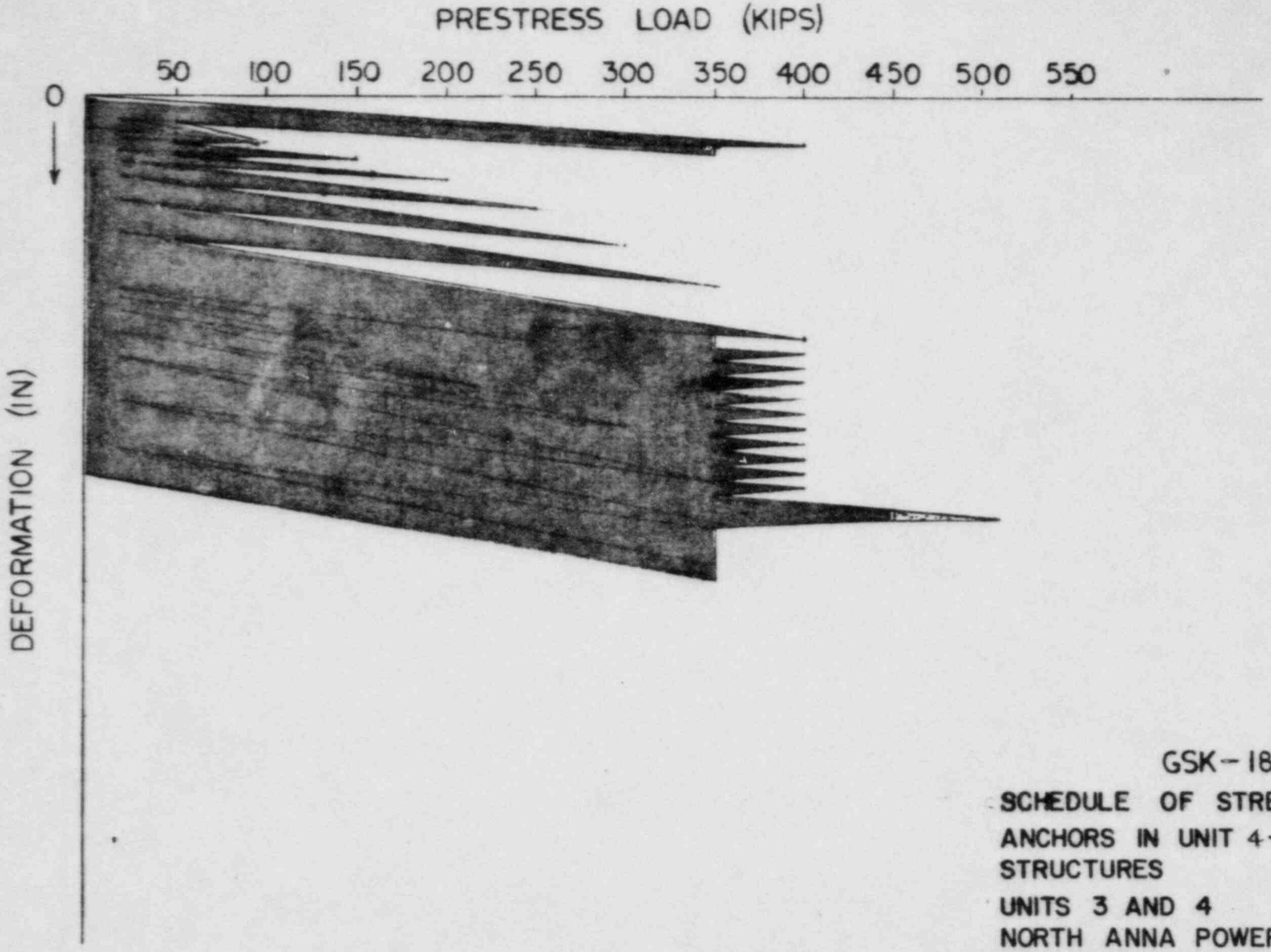
#1	N51+~~04~~ 09	E 90+~~70.5~~ 86.5
#2	N57+~~04~~ 09	E 90+~~78.5~~ 94.5
#3	N50+75	E 91+23.5
#4	N50+75,	E 91+31.5



SCALE: 1/16" = 1.0'

LOCATION OF TEST ROCK ANCHORS IN UNIT 4

* Revised to correspond with JSK-271



GSK-18

SCHEDULE OF STRESSING ROCK ANCHORS IN UNIT 4 - AUXILIARY STRUCTURES

UNITS 3 AND 4

NORTH ANNA POWER STATION

VIRGINIA ELECTRIC & POWER COMPANY

SECTION 3

ANCHOR TEST RESULTS AND DISCUSSION

3.1 GENERAL

The test program consisted of seven anchors. Anchors 1, 3, 5, 6, and 7 were multiwire systems. Anchors 2 and 4 were multistrand systems. Anchor 4 was the first anchor stressed, with Anchors 1, 2, 3, 5, 6, and 7 following sequentially. Stressing loads and tendon elongations were determined with load cells and long range dial indicators, respectively. Movements of the grout column were measured with telltale extensometers. The first five anchors were fitted with two extensometers. A third extensometer was added to the last two anchors.

The results of the test program are presented in three separate stages to better describe the chronological development of the test program. Each stage is evaluated separately and includes the logic for continuing the test program.

3.2 SUMMARY OF TEST PROGRAM RESULTS

The most significant results of the test program are summarized below:

1. The MB 814 grout performed poorly, particularly with respect to its ability to bond with the in-situ rock, as indicated by significant grout column movements in Anchors 1 through 5. Also, visual inspection of cores of the rock-grout interfaces recovered in Grout Test Holes 1 and 2 indicated that the grout had not bonded to the rock. On the other hand, the Type II+N grout performed significantly better than MB 814 as indicated by very small movements in the grout columns and by visual inspection of the rock-grout interfaces from cores recovered from Grout Test Hole 3.

2. The multiwire anchors were stressed up to 96 percent of the guaranteed ultimate tensile strength (GUTS) of the tendon steel during Phase IV stressing with no wire failures. Wires in the multistrand anchors failed at 82 percent of GUTS. The apparent primary cause of the multistrand failure is due to center wire slippage, whereby the outer six wires of each strand carried the total tendon load.

3. The method for lock-off for the multiwire system was simpler and less destructive to the tendon wires than that for the multistrand system.

4. The in-situ rock adjacent to the anchors did not undergo any significant movement during anchor stressing.

5. The variation in the total and bonded lengths selected for the test program did not provide significant differences in the test results.

Based upon the above results, the multiwire system and the Type II+N grout were selected for the permanent anchors. Further evaluation of the test program resulted in the removal of the expansive admixture from the grout mix for the permanent anchors.

3.3 TEST PROGRAM RESULTS - STAGE I

3.3.1 General

Stage I of the test program included two multistrand and two multiwire anchor systems, all grouted with MB 814 grout mixed with 2.0 gallons of water per 55 lb bag. Rock movements during anchor stressing were determined with mechanical extensometers, surface markers, and grouted rebars. This stage also included a grout hole test to determine "in-situ" grout strength, a grout encapsulation test to evaluate the effectiveness of the grouting techniques, grout compressive strength tests and water quality tests. The results of Stage I are presented and discussed separately below.

3.3.2 Anchor 1 (Multiwire System)

Anchor 1 is a 55 ft long multiwire system. Refer to Table 2-1 for the physical characteristics of the complete anchor system and the location of the grout extensometers. The results of the four stressing phases are shown in Figure 3-1, a plot of tendon elongation vs stressing load. This plot is typical of the elongation-load response of the other multiwire anchors where progressive grout-steel debonding occurred as the stress loads increased to 500 kips.

Figure 3-2 presents the elongation vs load results for Phase I stressing. This plot shows the characteristic parabolic-shaped curve, for both the loading and unloading cycles. This is typical for all the multiwire tendons. The tendon elongations correspond to the elongations for a fully bonded system* until a load of 100 kips was reached. With increasing load, the tendon wires in the bonded length progressively debonded from the grout column until, at 400 kips, the recorded elongation most nearly correspond to the predicted elongation for a fully unbonded system.* After unloading back to 50 kips, the tendon showed approximately 1.5 in of permanent elongation.

When the wires are completely debonded, the full anchor load is transferred to the baseplate and the grout column is placed in compression. The latter is considered desirable since grout is better able to transfer load in compression than in tension. Progressive grout-steel debonding may be

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*A fully bonded system implies that the tendon elongation occur within the unbonded length of the tendon (upper 25 ft). A fully unbonded system implies that the tendon elongation occurs throughout the entire length of the tendon.

attributed to the smoothness of the tendon wires and the Poisson's Ratio effect upon the tendon steel. The latter results in a decrease in the wire diameter as it elongates, thereby creating an annular space between the wire and the grout column. During unloading, the wire returns to its original diameter (below plastic yield levels) with friction eventually developing between tendon steel and grout. When the grout column is in compression, dilation in the grout column can occur, thereby reducing the annular space between the wires and the grout column. Therefore, contact between the wires and the grout may begin at a load level higher than that at which debonding occurred. This will result in a portion of the tendon stress being retained within the grout column, thus explaining the residual strain or permanent elongation of 1.5 in of the tendon after unloading to the 50 kip level.

Figure 3-3 presents the grout column movements for the Phase I stressing sequence. The grout column movements increased gradually up to a load of about 200 kips after which only minor movements occurred. The recorded grout column movements at the 400 kip load level were 0.6 and 0.85 in for the upper and lower extensometers, respectively. The higher movement of the lower extensometer seems unrealistic since this behavior would indicate that the lower grout column had moved up more than the upper grout column. During unloading the extensometers showed no movement, as indicated by the vertical shape of the plot, until the load dropped below 200 kips. From 200 kips to 50 kips, the grout column moved downward linearly. The permanent set of the grout column at 50 kips indicates that the grout column at the extensometer locations moved down below its initial starting position (by 0.38 and 0.08 in for the upper and lower extensometers, respectively). These results are unrealistic and most likely indicate either inaccuracy of the grout extensometer system or entanglement of extensometer wires with the tendon wires, with the initial slack in the wires being recovered during stressing. Field observations during anchor installation indicated that entanglement of the extensometer and tendon wires was possible.

Based upon the evaluation of the Phase I extensometer data the following observations can be made:

1. Up to a load of 200 kips, the anchor load is transferred to the upper portion of the grout column and then to the borehole rock with significant upward movement of the grout column.

2. As progressive debonding at the grout-steel interface occurs with increasing loads, stresses are transferred to lower sections of the grout column.

3. Complete debonding of the grout-steel interface in the upper portions of the grout column takes place as the load increases beyond 200 kips as indicated by the lack of upward grout movement during the last 200 kips.

4. The movement of the grout column during unloading follows closely the movement that occurred during loading. The upper portion of the grout column does not undergo any downward movement during unloading from 400 kips to 200 kips due to the Poisson's Ratio effect on the steel.

The elongation vs load curves for Phases II and III (Figure 3-1) fall in narrow bands that are approximately parallel to the theoretical fully bonded line.

During the seven-day lock-off between Phases III and IV, the anchor load increased 4.9 kips above the lock-off level. This slight increase is probably due to either changes in temperature or release of some of the residual stresses within the grout column.

The results of the Phase IV stressing are shown in Figure 3-1. The wires behaved elastically up to 460 kips and then plastically to the final load of 500 kips. These results are consistent with elongation tests on the steel as given in the CMTRs in Appendix B. The loading sequence in Phase IV was above 300 kips, hence the behavior of the grout column (Figure 3-4) indicates complete grout-steel debonding in the upper portion of the grout column. Grout column movements did not increase appreciably during Phase II, III, and IV stressing. The residual elongations of the tendon at the end of Phase IV are the result of the sum of the movements due to plastic yielding of the steel, short term creep of the steel during testing, and residual stress retained by the steel within the grout column.

The behavior of the rock mass during Phase I is shown in Figure 3-5. Note that rock movements, as monitored with mechanical extensometers and optical survey of surface markers and grouted rebars, are not appreciable.

3.3.3 Anchor 2 (Multistrand System)

Anchor 2 is a 55 ft long multistrand system. Figure 3-6 presents the elongation vs load results for the four stressing phases. The extent of steel-grout debonding was less than in Anchor 1. Initial plastic yielding of the tendon seems to have occurred between 350 kips and 400 kips, much earlier than expected, with significant yielding occurring beyond 400 kips. A permanent set of about 4.4 in remained after the Phase IV stressing was completed.

The results of the Phase I stressing are shown in Figure 3-7. A significant departure from the theoretical fully bonded line occurred at a load of 50 kips; thereafter, the elongation-load is essentially a straight line up to a load of 350 kips. The excessive elongation at 50 kips is most likely due to slack in the unbonded length of the tendon wires or to slack in the entire length due to the geometrical arrangement of the strand wires. This slack was recovered quickly during this first increment of loading.

The recorded elongation vs load curve for Phase I stressing lies between the theoretical fully bonded and unbonded lines. However, the equal movements of both grout extensometers, shown in Figure 3-8, suggest that the entire grout column moved uniformly, and if these grout column movements are deducted from the total tendon elongation, then the resultant net elongations of the steel wires would fall very close to the theoretical fully bonded line. Therefore, it may be inferred that minimal grout-steel debonding occurred. Steel-grout bond characteristics for a multistrand system are significantly better than for a multiwire system due to the following reasons:

1. The geometric arrangement of the strands in the bonded section (refer to Figure 2-2) ensures an excellent mechanical interlock between steel and grout, irrespective of actual grout-steel bond.
2. The non-vertical configuration of the strand wires does not allow for any significant contraction in wire diameter, that would lead to steel-grout debonding (Poisson's Ratio effect).

The behavior of the grout column during Phase I stressing, as indicated by Figure 3-8, is significantly different from that recorded in multiwire Anchor 1. This is due to the better grout-steel bonding discussed above. The plot of grout column movement vs load is essentially linear, indicating that little or no debonding had occurred between the grout column and the tendon steel. However, the magnitude of grout column movement, approximately 2.0 in at 400 kips, indicates poor bond between the grout and rock.

The results of the Phase II and III stressing (Figure 3-6) fall within narrow bands with the same slope as the Phase I stressing, except above 350 kips where elongations due to plastic yield occurred. During the Phase II stressing sequence, one wire in a seven-wire strand was heard to break as the load was being increased. Examination of the tendon at that time revealed no indication of wire breakage. However, after completing the test, further inspection of the tendon showed that one wire had broken by coming in contact with the rough edge of the center hole of the bearing plate.

The load in Anchor 2 during the seven-day hold dropped 44 kips. This large loss was apparently due to deformations of the horseshoe destressing shim. This was discovered after completion of the hold and liftoff cycle.

Grout column movements during the Phase II and III stressing are shown in Figure 3-9. These plots are essentially linear, again indicating good bond between the steel and grout column. However, the magnitude of grout column movement indicates poor grout-rock bond.

Phase IV loading of the anchor from 350 kips to 500 kips was attempted but excessive elongations occurred at the 400 kip and 450 kip levels, indicating yield in the steel (refer to Figure 3-6). At a load of 471 kips, wires in the strands were heard to snap, and jacking was immediately discontinued. The permanent tendon set was about 4.4 in. Grout column movements during Phase IV were constant at about 2.0 in (see Figure 3-9).

The results of the rock measurements during all the stressing phases are shown in Figure 3-10. Note that the rock movements were very small.

Examination of the tendon strands after the test equipment had been dismantled showed that, with one exception, the wires had broken level with the bottom of the grip wedges. The exception was the one wire that had come into contact with the edge of the bearing plate. The significance of this observation is discussed with the results of Anchor 4.

3.3.4 Anchor 3 (Multiwire System)

Anchor 3 is a 45 ft long multiwire system. The elongation vs load data for all stressing phases are shown in Figure 3-11. The results indicate a performance very similar to that of Anchor 1. The same progressive debonding of the wires was noted with almost complete debonding at 400 kips. This can best be seen in Figure 3-12 which shows the data for the Phase I stressing. The Phase II and III stressing results (see Figure 3-11), as in Anchor 1, plot approximately parallel to the theoretical fully bonded line, thus indicating the presence of retained residual stressing in the grout column during unloading. During the seven-day hold between Phases III and IV loading, a 4.5 kip increase in load was measured, as occurred with Anchor 1.

Two differences were noted between the results of Anchors 3 and 1. At a load of 450 kips, the elongation of Anchor 3 was in excess of the theoretical fully debonded line (refer to Figure 3-11). The excess movement was about 0.25 in which corresponds to the grout column movements shown in Figure 3-13. Plastic yield of the steel could not be the cause of the excess movement since mill test reports indicate 460 kips to be the lowest load at which yield could be expected and no wire failures were found after stressing. Therefore, it is assumed that the baseplate as well as the entire grout column moved uniformly up the borehole. The other difference between performance of the two multiwire anchors (Anchors 3 and 1) is the smaller grout column movements recorded in Anchor 3. This result is either due to better rock-grout bond or a faster transfer of anchor load to the baseplate leading to dilation of the grout column (Poisson's Ratio effect in compression) and, earlier development of mechanical friction between the grout column and the rock. The erratic readings of the grout extensometers during Phase IV stressing (see Figure 3-14) are probably due to entanglement of the grout extensometer wires with the tendon wires and are therefore not considered to be reliable.

The permanent set of the anchor was recorded to be approximately 1.1 in and was due to yielding in the wires at loads above 460 kips and to the unrecovered grout column movements.

Rock movements during the stressing of Anchor 3 are shown in Figure 3-15. The rock extensometers, the surface markers, and grouted rebars measured negligible rock movements.

Upon completion of the Phase IV loading, an attempt was made to load the anchor to failure (Phase V) to determine the "weakest link" in the anchor system. The elongation vs load for this loading sequence, shown in Figure 3-16, indicates that the anchor behaved as a fully unbonded system with load being transferred to the baseplate. The tendon wires did not fail but yielded plastically. Loading was discontinued at 522 kips.

3.3.5 Anchor 4 (Multistrand System)

Anchor 4 is a 45 ft long multistrand system. This was the first anchor tested, and the stressing was performed in extremely cold weather. The low temperatures did not affect the load-elongation data obtained but did cause brittle failure of the tack welds holding the pulley section of the grout extensometer. The extensometers were rezeroed and subsequent data was adjusted to account for the initial deflections. The results of the four stressing phases are shown in Figure 3-17.

The elongation vs load plot for Phase I stressing is shown in Figure 3-18. The tendon elongation at the initial load increment of 50 kips plots beyond the theoretical fully unbonded line. As in Anchor 2, this relatively large elongation is due to initial slack in the tendon wires which was recovered immediately after initial loading. The shape of the elongation vs load plot above 50 kips is nonlinear, indicating progressive steel-grout debonding.

Grout column movements during Phase I stressing are shown in Figure 3-19. The shape of the grout column movement vs load for the upper extensometer is similar to that of Anchor 1, and indicates progressive debonding. The movement of the upper extensometer reaches a maximum of 1.0 in. The movements recorded by the lower extensometer are much smaller, approximately 0.25 in and more linear. The curves for both extensometers become nearly vertical above 250 kips, indicating that at this load level the tendon load is transferred below the lower extensometer. The higher movement recorded by the upper extensometer indicates poor rock-grout bond. The difference in the recorded movements of the two extensometers indicates that the grout column cracked at some point between the two extensometers, with the upper portion acting independently of the lower section.

The results of cyclic loading during the Phase II and Phase III stressing sequences are shown in Figure 3-17. These results plot in narrow bands and parallel the curve for the Phase I stressing. The tendon elongation at load levels above 350 kips plotted above those for a fully unbonded tendon due to either plastic yield in the steel or grout column movements. At 400 kips, the deformation in excess of the fully unbonded tendon is approximately 0.25 in which corresponds to the lower grout extensometer movement.

The lift-off check at the end of the long-term hold period, which for this anchor was 12 days, showed a loss of 1.8 kips. This loss is negligible and is within the accuracy range of the load cells.

As the Phase IV stressing sequence was being carried out, wires in the strands started to snap at a load between 435 kips and 445 kips with the tendon rapidly elongating. Consequently, the load was reduced to zero and the equipment dismantled to permit inspection. It was found that in eight of the thirteen strands one wire per strand had snapped level with the bottom of the grip wedges. All the broken wires were on the same side of each strand, and the eight strands were all on one side of the tendon. The location of the failed strands indicates that the tendon loading from the hydraulic jack was not symmetrically distributed for the 13 strands. At this point in the test program, the tendon failure at such low load levels (approximately 82 percent of the manufacturer's guaranteed ultimate tensile strength) was attributed to:

1. "Teeth marks" on the wires left by the wedges, resulting in localized reduction of the cross-sectional area of the wire

2. Resetting the wedges during stressing phases, resulting in slight overlaps in "teeth marks" (i.e., the wedges were not necessarily reset on the same "teeth marks") thus, leaving the area where the wedges gripped the wires in a macerated condition

To further investigate the cause of these failures, an additional phase, Phase V, was added to Anchor 4. For this phase the wedges were set at a lower level than for the prior phases. The elongation vs load plot for Phase V is shown in Figure 3-20. Wires in the strands began to break at approximately 470 kips, slightly higher than during the Phase IV stressing but, nonetheless, significantly lower (87.5 percent) than the guaranteed ultimate tensile strength of the strands. This phase was stopped at this point and the stressing equipment was removed. Inspection of the tendon showed that wires in all the strands had failed and that the wires had broken level with the bottom of the grip wedges. The center wire had been broken level with the other six wires in two of the strands and was still intact in the other eleven strands.

These observations indicate that the "early" failures are due to a combination of the following:

1. "Teeth marks" from the grip wedges leading to localized stress concentrations

2. Slippage of the center wire relative to the six exterior wires in each strand, with little or no load transfer to the center wire.

The reduction of load capacity due to the presence of the "teeth marks" is probably fairly low but, nonetheless, it is important to note that all outside wire failures occurred at the bottom of the grip wedges, with the failure beginning at the first indentation. Resetting of wedges may lead to slight overlap of the "teeth marks," and failure at lower stress (435 kips to 445 kips for Anchor 2) than when the wedges were not reset (470 kips for Anchor 4).

Most likely, slippage between the center wire and the exterior six wires of each strand is the principal reason for the reduction in anchor capacity. Each strand is comprised of six wires wound helically around a straight center wire. Load is transferred from the exterior wires to the center wire by friction and the clamping force of the grip wedges. In normal posttensioning systems, the strands are held with barrels and wedges at both ends, but in foundation anchors the strands are held only with barrels and wedges at one end. Apparently this results in some looseness of the center wires of each strand with the center wire carrying less load than the outer wires. Center wire slippage for all strands would mean a drop in ultimate strength to approximately 85 percent of the guaranteed ultimate tensile strength. This compares well with the observed ultimate anchor capacities of Anchors 2 and 4. For a six wire strand, plastic yield would commence at 390 kips. Figures 3-6 and 3-17 show that plastic yielding occurs between 350 kips and 400 kips for Anchors 2 and 4, respectively.

The movement of the grout column during the last four phases of stressing was less than 0.10 in and is shown in Figure 3-21. In comparison with the initial large displacements that occurred in the upper extensometer during Phase I stressing, these small displacements indicate an increase in grout-rock friction caused by the grout column "jamming" in the borehole, possibly due to dilation of the column with increasing tendon stress.

Figure 3-22 shows the recorded movements of the rock extensometers and the surface markers. The latter recorded negligible rock movements.

3.3.6 Grout Test Hole No. 1

The purpose of Grout Test Hole 1 was to determine the in-situ time-strength relationships of the anchor grout via unconfined compression tests of grout core samples and to compare these results to those obtained from mortar cubes.

The diameter and depth of this test hole were approximately 6.0 in and 46 ft, respectively. The drilling and preparation of this borehole were performed in the same manner and sequence as the anchor boreholes. After reaming the preliminary grout, MB 814 grout (with 2.0 gallons of water per 55 lb bag) was tremie grouted to the top of the borehole. The upper 20 ft of the grout was flushed as in the anchor boreholes. The bottom 25 ft of the borehole was cored at 5 ft intervals at 3, 8, 13, 18, and 23 days after the borehole was grouted. The compressive strength vs time plots for both cores and mortar cubes are shown in Figure 3-23. The compressive strengths from the core samples are significantly higher for samples tested within the first 12 days after the borehole was grouted. The strength differences between the two sample methods decreases with time.

During coring of the grout, it was found that the hole was inclined at a slight angle from the vertical. Consequently, the lower run of core intersected the borehole wall, thus providing a section of the grout-rock interface for inspection. It was found that the grout had not bonded to the rock. The significance of this observation became more evident as the test program continued.

3.3.7 Grout Encapsulation Test

A grout encapsulation test was performed to evaluate the ability of the MB 814 grout and the effectiveness of the tremie grouting procedures in encapsulating the multistrand anchors. The multiwire anchors were not tested because their simple configuration was not thought to present an encapsulation problem.

A 25 ft long tendon was placed in a 45 ft long, 6 in dia PVC pipe. The pipe was lifted and hung vertically from the edge of the Unit 4 reactor containment excavation. The PVC pipe was tremie grouted with the MB 814 grout. After eight days, the pipe was lifted from the scaffolding and laid horizontally. The grout column was cut into several sections. The cut end of each section showed excellent encapsulation of the tendon strands with complete filling of the interstitial spaces between wires.

3.3.8 Grout Compressive Strength

Mortar cube samples of the MB 814 grout were taken during the grouting of the anchor test holes, grout encapsulation test, and grout test hole. All samples reached unconfined compressive strengths in excess of 3,000 psi within seven days and in general, the compressive strengths exceeded 6,000 psi after 28 days. Figure 3-23 shows compressive strength vs time for the grout test hole. Cube strength results for the MB 814 grout used in Anchors 1 through 5, and the grout encapsulation tests are shown in Figure 3-24.

3.3.9 Water Tests

Lake Anna water was used for washing the grout test and anchor boreholes and for mixing the grout ingredients. It is anticipated that Lake Anna water will be used to perform the same function for the permanent anchors. Chemical analyses of Lake Anna water and groundwater were performed to determine the presence of any substances that would be deleterious to either the grout or the tendons. Groundwater temperatures with respect to depth and time were determined from several boreholes to better document the in-situ environment in the grouted anchors (Table 3-2).

The results of the chemical analyses, shown in Table 3-1, indicate that lake and groundwater are neutral and contain relatively low concentrations of sulfides, sulfates, nitrates, and chlorides. Also, the concentration of these ions and the pH value are essentially constant over the time period shown (1975 to 1978). Groundwater temperatures with respect to depth and time, as shown in Table 3-2 indicate that groundwater temperatures are essentially constant with respect to depth, time, and site location. Recorded temperatures range from 53°F to 58.5°F with an overall average of 55°F. Based upon these data, it was determined that both lake and groundwater will not have an adverse effect on either the grout or the tendons.

Tests were also conducted on borehole wash water to determine the effectiveness of the borehole washing operation in removing oil or grease introduced into the borehole during the drilling process. The tests showed oil concentrations of less than 5 parts per million and indicate that the borehole washing procedures were effective.

3.3.10 Evaluation of Stage I Test Results

Stage I of the test program provided many significant results. Based upon an evaluation of these results the following conclusions can be drawn:

1. Failure of the multistrand tendons at approximately 82 percent of the guaranteed ultimate tensile strength (GUTS) was due primarily to slippage of the center wire in each strand. In addition, local weakening of the tendon can occur at the grip wedges, especially if the wedges are reset. This presents a potential problem for in-service inspection lift-offs. Failures of multistrand tendons at loads significantly less than GUTS have been documented elsewhere. During an inservice inspection of the posttensioning system at the Rancho Seco Nuclear Station(10) in California, a multistrand anchor failed at 82 percent of GUTS. Also, during the test program for the rock embedded portion of the posttensioning system at the Bellefonte Nuclear Station(11) in Alabama, one multi-strand anchor failed at 82 percent of GUTS. Based on these results the multistrand system was eliminated from further consideration for the production anchors.

2. The multiwire system was tested to a maximum of 522 kips (96 percent of GUTS) without failure of the tendon wires. Lock-off was achieved with shims placed between the anchor head and the bearing plate and lift-offs were performed satisfactorily.

3. The MB 814 grout displayed unsatisfactory grout-rock bonding characteristics as indicated by the large grout column movements. Core recovery of the grout interface in Grout Test Hole 1 also showed that the grout had not bonded to the rock.

4. Tendon elongation results for Phase I stressing (similar to stressing sequence planned for the permanent anchors) were not significantly different for the multistrand vs the multiwire tendons, indicating that the results were insensitive to the total and bonded lengths selected for the test program. These results also show that the multiwire tendons experienced almost complete debonding in the bonded length, significantly more than for the multistrand tendons. However, this is not a problem since the base plate at the bottom of the multiwire places the grout in compression. The "permanent set" at the end of Phase IV stressing is primarily due to plastic yield of the tendons.

5. Rock movements during all stressing phases were negligible.

6. Lift-offs after seven day lock-offs showed that greater changes in prestress level occurred in the multistrand tendons than in the multiwire tendons.

7. The grout encapsulation test indicated that the selected grouting techniques resulted in excellent encapsulation of the multistrand tendon.

8. Compressive strengths of the MB 814 grout, as determined from mortar cubes and grout cores taken from Grout Test Hole 1, were significantly above the minimum requirements. The strengths determined from the grout cores were significantly higher than those determined from mortar cube tests.

9. The array of instrumentation, with minor exceptions, performed well.

10. Water quality tests performed on lake and groundwater indicate that very low concentrations of nitrates, sulfates, chlorides and sulfides exist.

11. The Phase V stressing of Anchor 3 indicated that the "weak link" in the steel-grout-rock system is the tendon steel. Since the strength properties of the steel are easily quantifiable, these results make possible the determination of the lowest factor of safety in the anchor system.

3.4 TEST PROGRAM RESULTS - STAGE II

3.4.1 General

Based upon the results of Stage I, the test program was continued with a 45 ft long multiwire tendon (Anchor 5) and MB 814 grout mixed with 2.4 gallons of water per 55 lb bag of prepackaged grout. It was anticipated that the higher water-cement ratio would reduce the apparent high level of thixotropy in the lower water content (2.0 gallons of water per bag) grout mix used with the first four anchors and would hence result in higher rock-grout bond. Two grout test holes approximately 25 ft deep, were added to the test program. Grout Test Hole 2 was filled with MB 814 grout (and 2.4 gallons of water per bag) and Grout Test Hole 3 was filled with Type II+N. Both grouted boreholes were then cored to recover the rock-grout interface to visually examine the bonding characteristics of the two grouts.

The rock instrumentation system was not continued in subsequent stages of the test program since the results obtained during Stage I clearly indicated that rock movements would be negligible.

3.4.2 Anchor 5 (Multiwire System)

Anchor 5 is a 45 ft multiwire anchor. Refer to Table 2-1 for the physical characteristics of the complete tendon system. The results of the four stressing phases are shown in the tendon elongation vs load plot in Figure 3-25. The results show good grout-steel bond up to 200 kips, as indicated by the load-extension line plotting along the theoretical fully bonded line. Thereafter, progressive debonding occurred with almost complete debonding at a load of 400 K (Figure 3-26). The behavior of Anchor 5 during Phase I stressing is similar to that of Anchor 3, the other 45 ft multiwire tendon (see Figure 3-27). The results of Phases II and III stressing plot in narrow bands roughly parallel to the fully bonded line, as shown in Figure 3-25. During the hold test, which lasted 24 days, the anchor lost 10 kips of load. During Phase IV stressing, the anchor behaved elastically (Figure 3-25). After unloading, 1.1 in of permanent deformation remained.

The lower grout column extensometer exhibited little movement during all stressing phases (Figure 3-28), and behaved in a manner similar to both Anchor 3 extensometers (Figure 3-29). However, the upper extensometer displayed an anomalous behavior. During Phase I stressing, small movements occurred up to 150 kips. At this point, a loud crack was heard and the extensometer wire jumped 0.125 in (Figure 3-30). Thereafter, the movements increased, and at the end of Phase I, the extensometer had moved 1.65 in. During the remaining three phases, the upper extensometer continued to exhibit large displacements (Figure 3-28). This behavior may have been due to cracking of the grout column at the 150 K load, followed by movement of the upper grout column as a single unit with the tendon wires being separated from the lower grout column Further evaluation of the possible grout column separation, made after the completion of the test program, indicated that the presence of the flat disc-shaped spacers in the bonded length may have divided the grout column into separate sections. The diameter of the spacers was approximately 1.0 in less than the borehole, and they were placed every 10 ft along the bonded length. The grout extensometers were located at the midsection of the top two grout sections.

3.4.3 Grout Test Holes No. 2 and 3

These two grout test holes were approximately 6.0 in dia and were drilled to a depth of about 25 ft. Grout Test Hole 2 was filled with MB 814 grout mixed at 2.4 gallons of water per 55 lb bag of cement. Grout test Hole 3 was tremie grouted with Type II+N grout mixed at 5.0 gal of water per 94 lb bag of cement. Once the grout reached the minimum required strength of 3,000 psi, as determined by mortar cube tests, each hole was cored to recover the grout-rock interface. Tables 3-3 and 3-4 provide logs of cores recovered from Grout Test Holes 2 and 3, respectively. Visual examination of the cores indicated that adequate bonding had occurred in approximately 39 percent of the recovered cores of the MB 814 grout-rock interface. Cores from Grout Test Hole 3 showed that adequate bonding occurred in approximately 100 percent of the recovered grout-rock cores for the Type II grout.

A petrographic analysis was performed on four thin sections of the grout-rock interfaces obtained from the grout test hole cores. Two thin sections were made from each test hole. The results of the petrographic analysis are discussed in detail in Appendix D. Briefly, the analysis revealed that during curing the MB 814 grout has a tendency to crack radially and along the contact with the borehole rock. The cracks are filled with a silicate mineral formed by a chemical reaction within the grout itself or between the grout and groundwater. It is believed that the presence of the cracking along the grout-rock interface and the mineral filling therein contributed to the poor grout-rock bonding of the MB 814 grout. Examination of the Type II cement grout-rock interface found significantly less cracking at the grout-rock interface, with little evidence of a chemical reaction and no mineral filling in the cracks.

3.4.4 Evaluation of Stage II Results

Results from Grout Test Hole No. 2, the thin section analysis, and the cracking of the grout column of Anchor 5 clearly indicated that the MB 814 grout did not provide adequate rock-grout bond. Based upon these results it was decided to abandon the use of MB 814 grout and to extend the program to include two anchors grouted with Type II+N grout.

3.5 TEST PROGRAM RESULTS - STAGE III

3.5.1 General

Stage III of the test program consisted of two 55 ft long multiwire tendons grouted with Type II+N grout, a grout column test to determine the effects of the expansive admixture on the grout strength, and a grout encapsulation test to visually determine the encapsulation of the tendon with this grout mix. Anchors 6 and 7 differed from the other multiwire tendons in two ways. First, the tendons were outfitted with three grout extensometers to obtain more information relative to the behavior of the grout column. Second, the tendons were shipped with 6 in dia keeper plates instead of 5 1/4 in. The effect of this change was considered minimal.

3.5.2 Anchor 6 (Multiwire System)

The results from all four stressing phases are summarized in a tendon elongation vs load plot shown in Figure 3-32. The stressing results indicate a performance similar to the other 55 ft multiwire tendons, Anchors 1 and 7 as shown in Figure 3-33. During Phase I, progressive debonding commenced at 100 K and continued until complete debonding occurred at 400 K (Figure 3-34). During Phasec II and III, the results plot in narrow bands, roughly parallel to the theoretical fully unbonded line rather than the theoretical fully bonded line as was observed in Anchors 1, 3, and 5. This departure from previous observations is probably a result of less friction developing between the grout and steel.

Anchor 6 experienced elongations greater than a theoretical fully unbonded system during Phases II and III (Figure 3-32). Anchor 3 experienced similar results. This behavior cannot be attributed to yield of the steel, since the mill test report shows that plastic yield should not begin until a load of 460 K is imposed and no wire failures were noted after stressing. The excess movement could be due to movement in the entire grout column.

During a 10-day holding period, the stress in the anchor increased 10 K which is probably due to rearrangement of residual stresses within the grout column.

Phase IV stressing to 500 K was accomplished with some yielding of the steel. After complete unloading, 1.90 in of permanent deformation remained.

Grout extensometer movements are plotted in Figures 3-35 and 3-36. The upper two extensometers displayed very small movements. The middle extensometer moved about 0.05 in, and the upper extensometer moved 0.15 in. In general, the movements occurred during loading to 200 K, with no additional movements observed thereafter. These small movements indicate good grout-rock bond. Considering the small movements of the upper extensometers, the larger movements of the lowest extensometer appear to be anomalous, and are most likely due to the extensometer detector wire being entangled with the tendon wires.

3.5.3 Anchor 7 (Multiwire System)

Anchor 7 is a 55 ft multiwire tendon. The physical characteristics of the complete tendon system are shown in Table 2-1. During construction of this anchor, two problems arose that had bearing on the results of the test. During insertion of the tendon into the borehole, it was noted that the detector wire from the highest extensometer had become tangled with the tendon wires. The other problem concerned the contractor's inability to properly thread the puller bar into the anchor head. Visual inspection of the threads in the anchor head revealed some irregularities in the threads. However, sufficient thread was engaged to allow stressing of the anchor.

Phases I and II stressings were successfully completed. The load-extension curves for Phase I stressing is shown in Figure 3-37. Figure 3-33 shows similar results of the Phase I stressing for Anchors 1, 6, and 7, all 55 ft long multiwire tendons. Debonding began at 100 K, continuing until almost complete debonding occurred at 400 K.

Plots of the grout column extensometer movement vs load is shown in Figure 3-38. The two lower extensometers moved less than 0.03 in, thus indicating a good grout-rock bond. The upper extensometer showed movements of 0.80 in at the 300 K load with no additional movements thereafter. These movements are most likely due to the detector wire being tangled with the tendon wires.

At the beginning of Phase III stressing, the threads in the anchor head failed, resulting in a sudden destressing of the tendon with damage to the stressing test equipment, the anchor head, and tendon wires. Prior to the resumption of testing, every wire was individually stressed to 76 percent of GUTS to determine whether any wire had been damaged. The results of the above stressing sequence for selected wires, shown in Figure 3-39, indicated that none of the wires had been damaged. The damaged anchor head was replaced with the head from Anchor 5. Due to an error in rebuttoning the wires, only 45 of the 46 wires were engaged. The refitting of a new anchor head reduced the length of the anchor by approximately 6 in. Refer to Appendix B for a more detailed discussion of the anchor damage and the subsequent repairs.

All four stressing phases were then performed on the repaired Anchor 7. The plot of tendon elongation vs load is shown in Figure 3-40. During Phases II and III, the results plot in narrow bands, which are parallel to the theoretical fully bonded line, and indicate a retention of residual stress in the grout column. Phase IV jacking was discontinued at 459 K as excessive elongation occurred due to plastic yielding of the steel. After removal of the load, 0.85 in of deformation remained due to plastic yield of the steel and the residual stress on the anchor.

The extensometer movements are plotted in Figures 3-41 and 3-42. The movements are relatively small for all three grout extensometers. These small movements indicate much better bonding of the grout to the rock. Note that the movements recorded by the upper extensometer are significantly less than during the stressing phases prior to the thread damage. These results confirm the earlier observations that these movements were anamolous and were due to the detector wire being entangled during the anchor installation.

3.5.4 Grout Column Test

The grout column test was performed to determine the effect the expansive agent would have on the grout strength and to visually observe the expansion process. For the test, a 4 in dia, 42 ft long pipe was used. The pipe was suspended from the scaffolding in the reactor containment excavation and tremied with a grout mix consisting of Portland Type II cement mixed at 5 gal of water per bag. The expansive agent Intraplast "N," was added to the grout at a rate of 0.49 lb per bag.

After placing the grout, the pressure gages were monitored continuously until the grout column set and any reaction or expansion due to the admixture had ceased. A table of pressure vs time with observational remarks is given in Figure 3-43.

Fluid pressures in the PVC pipe were measured with two gages at approximately 6.5 ft and 2.5 ft above the bottom of the pipe. The purpose of the gages was to measure any pressure changes during the expansion process. The initial recorded pressures are lower than expected for a static head from a vertical column of water and fluid grout. It is possible that the water leaks observed at the gage locations may be responsible for the lower recorded gage pressures. However, assuming that the difference between actual and recorded gage pressures remained constant during the entire test (leaks at the gage locations continued until the grout set), then the changes in recorded pressures during the expansion process would be considered accurate.

The gage pressures increased when the water in the pipe was replaced with the tremie grout. For the next two to three hours, during which time the expansion process was well underway, the recorded pressures dropped slightly. Thereafter, the gage pressures decreased slowly as the grout began to set and dropped to zero approximately 24 hours after tremie grouting, at which time the grout had completely set.

Bleed water and grout consolidation began to occur within 15 min after tremie grouting. Initial expansion in the grout column was evident approximately one hour after tremie grouting at which time the grout column began to rise. Overtopping of the pipe began shortly thereafter. Foaming, indicating gaseous release from the expansive agent, became evident approximately four hours after tremie grouting and continued for another two to three hours. Inspection of the pipe on the following day revealed that 1.5 ft of bleed water was left at the top of the pipe and that the top of the grout was extremely soft.

The slow drop in gage pressures and overtopping of the pipe by the grout column during expansion clearly indicate that expansion in a vertical pipe, confined at one end, does not occur radially but rather vertically. This results in a lower density grout which, as discussed below is especially weak at the top of the grout column.

After the grout had set, the pipe was lifted off the scaffolding and cut into sections for the purpose of obtaining cylindrical samples for grout strength testing. The results are plotted in Figure 3-44 as a function of time and depth. Also plotted on this figure are the compressive strengths from mortar cubes made from the same grout used to fill the pipe. The cylinder strengths are much lower than the cube strengths, especially at shallow depths. It was also noted that the top section of the grout column was extremely soft and moldable even three weeks after grout placement in the pipe. Based on these results, it appears that the presence of the expansive admixture results in lower compressive strengths of the grout. In particular, the very low strengths at the top of the grout column suggest that this grout may have a higher water content due to bleed water being forced up the pipe by the vertically rising gases released by the expansive agent.

3.3.5 Grout Encapsulation Test

A grout encapsulation test was performed using the same procedures as in the earlier test (during Stage I) except that a Type II Portland cement grout was used. Visual inspection of cut sections showed that the tendon was well centered in the pipe and that the grout provided excellent encapsulation. Several observations made in Appendix B indicate that this grout had bonded better to the tendon than the MB 814 grout.

3.5.6 Evaluation of Stage III Test Results

The test program was completed with the stressing of Anchor 7. The stressing results of Anchors 6 and 7 were satisfactory and the small movements of the grout column extensometers indicated a significant improvement in grout-rock bond with the Type II grout (Figures 3-45). This conclusion is substantiated by the grout test hole results obtained during Stage II of the test program. The repair and retesting of Anchor 7 provides a convincing demonstration that, even after the unexpected rapid destressing and subsequent damage to the anchor, that the same anchor could be restored, and restressed with satisfactory results.

SECTION 4

RECOMMENDATIONS FOR PERMANENT ANCHORS

4.1 GENERAL

The results of the test program provide the basis for selecting the anchor system, the grout mix, and the best installation methods for the permanent anchors, as well as determining the anticipated behavior of the anchors during stressing. Subsequent to the test program, several modifications were made to the permanent anchors to accommodate structural design changes.

4.2 RECOMMENDATION BASED ON THE TEST PROGRAM

4.2.1 Anchor System

The multiwire system was selected over the multistrand system for the permanent anchors due to the following reasons:

1. The lock-off and lift-off procedure for the multiwire system is superior to that for the multistrand system and provides the least chance of anchor damage for the lift-offs required during in-service inspection.

2. The multiwire system was stressed to 96 percent of the guaranteed ultimate tensile strength (GUTS) of the tendon while the multistrand system could only be stressed to approximately 82 percent of GUTS, at which point the tendon behaved plastically and some wires failed. The wire failures at load levels less than the ultimate strength of the steel are due primarily to center-wire slippage whereby the tendon load is carried solely by the outer six wires.

3. The repair and retesting of Test Anchor 7 provides a convincing demonstration that, even after the unexpected rapid destressing and subsequent damage to the anchor, the same anchor could be restored and restressed with satisfactory results.

4.2.2 Grout Type

The two types of grout used during the test program were MB 814 grout and a Type II Portland cement grout with an expansive admixture, Interplast N (Type II+N grout). Despite high unconfined strength, the MB 814 grout bonded poorly to the rock. The inability of the MB 814 grout to form a good bond is demonstrated by the excessive grout column movements in Anchors 1 through 5. The grout extensometer movements for Anchors 4 and 5 (Figures 3-21 and 3-28) suggest that the grout column had cracked somewhere between the two extensometers. Also, examination of NX cores of the grout-rock interface recovered from Grout Test Holes 1 and 2 indicates that MB 814 grout had not bonded satisfactorily to the rock. On the other hand, Anchors 6 and 7, which were grouted with Type II+N experienced small grout column movements as shown in Figure 3-45. In addition, NX cores recovered from Grout Test Hole 3 showed the Type II+N grout to be well bonded to the rock.

Based on these results, it was determined that Type II+N grout should be used for the permanent anchors. However, a further review of the grout mix design for the permanent anchors raised concerns about the effects of the Intraplast "N" on the grout. The results of the investigation which included a literature review, discussions with Nicholson Anchorage Company, and a further evaluation of the Grout Column Test indicated that the expansive agent should be eliminated for the following reasons:

1. Expansive agents are added to grout mixes to provide radial expansion of the grout in confined areas (such as in cable ducts of posttensioning systems) and to safeguard against shrinkage of the grout. In foundation anchors, confinement is not available at the top of the borehole; therefore, grout expansion occurs in the direction of least resistance, i.e., vertically, not radially (see results of Grout Column Test). Also, at North Anna, the permanent rock anchors will be below the construction and postconstruction groundwater levels; hence, the potential for grout shrinkage is not present. Therefore, the benefits of the expansive agent are minimal.

2. The expansive agent may have deleterious effects on the grout. The release of gas from Intraplast "N" does not necessarily begin immediately upon mixing of the grout. Tests performed by Stressteel (^{13}) indicate that the start and duration of "expansion" varies significantly, depending upon the chemistry of the cement. The expansion process may begin and/or continue for some time after the initial set of the grout, resulting in high tensile stresses and possible cracking of the grout column. Also, studies (^{14}) have shown that the addition of expansive agents above 2.0 percent by weight of cement will result in a very weak grout.

The vertical expansion of the grout due to Intraplast "N" results in a higher porosity, lower density material with localized pockets of bleed water. This will result in a lower quality grout, particularly in its ability to provide adequate corrosion protection.

4.2.3 Installation and Stressing Procedures

The techniques used during the test program for borehole drilling and installing, grouting, and stressing the anchors worked well and will be employed for the permanent anchors. In addition, the following improvements will be made:

1. Preliminary grouting will be required for all anchor holes as an added layer of corrosion protection. Water testing will be done after preliminary grouting.

2. The stressing sequence used in the test program will not be necessary for the permanent anchors. The stressing sequence for the permanent anchors will include a load cycle, a 10 min hold test, and an unload-reload cycle. Lift-offs to determine any changes in stress levels will be made approximately 24 hr after lock-off and just prior to the completion of permanent anchor installation.

3. The entire length of the tendon will be grouted in a single stage. Single-stage grouting provides better corrosion protection to the tendon by eliminating the primary-secondary grout interface. This interface may be a potential zone of weaker and more porous grout due to the presence of bleed water. Also, the presence of the grout column in the unbonded length will provide additional confinement to the lower grout column and eliminate any concerns about cracking or movement of the grout column during anchor stressing. Single stage grouting is made possible by the presence of the unbonded length which allows the anchor to be stressed and to have the tendon load transferred to the bonded length.

4. During stressing, the wires elongate and are no longer protected by the PVC tubing in the unbonded length, just below the anchor head. Therefore, immediately after stressing, the void between the grout column, shims, and anchor head will be filled with a corrosion-inhibiting grease. Additional corrosion protection of the wires and top anchorage will be provided by filling the cover box with a corrosion-inhibiting grease after stressing.

5. The flat disk-shaped spacers used with the multiwire tendons in the test program will be replaced with spider-shaped spacers. The latter will provide less impedance to the tremie grouting process and will result in a more continuous grout column. Thus, any concern about grout column separation by the disk-shaped spacers will be eliminated.

6. A separate wire, buttonheaded at the top only, will be included in 28 of the permanent anchors to monitor corrosion development in the steel as well as any chemical changes that may occur in the lubricant. This wire will be encapsulated inside a PVC tube filled with a corrosion-inhibiting grease.

7. Nineteen permanent anchors will be fitted with permanent load cells to monitor changes in prestress levels after installation of the anchors and during in-service inspection.

4.2.4 Behavior During Stressing

Stressing of the permanent anchors will be closely monitored. Stress-strain relationships will be developed, using results from the test program, to evaluate the behavior of each tendon during stressing.

4.3 ANCHOR DESIGN MODIFICATIONS

4.3.1 Anchor Capacity

The anchors used for the test program had a working capacity of about 300 kips. Since the completion of the test program, pipe break loads and increased structural inertial forces have been accommodated by increasing the design capacity of the anchors to 575 kips. To achieve a higher capacity, the borehole diameter and the bonded length of the permanent anchors have been increased so as to maintain the grout-rock shear stress within the range established during the test program. In addition, the number of wires has increased from 46 to 90. To mobilize a larger mass of rock to resist the higher anchor loads, the anchor length will be increased to an average of 65 ft and the anchor lengths will be staggered with the deepest anchors located the farthest from the containment buildings.

4.3.2 Anchor Head Design

The TVA's Bellefonte Nuclear Power Station recently experienced failure of 16 anchor heads due to stress corrosion cracking. The causes of the anchor head failures are discussed in detail in a TVA report, entitled "Final Report - Rock Anchor Tendon Stressing Head Failures."(15) To prevent a similar problem from occurring at North Anna, the following requirements have been placed on the top anchorage:

1. The head shall be fabricated from steel with a tensile strength of less than 170,000 psi. This strength value is considered the dividing line between material not susceptible and susceptible to stress corrosion cracking(16)

2. Flatness tolerances will be specified for the shims and bearing plates. The maximum vertical deviation from a horizontal surface along any section of the shims and bearing plates will not exceed 0.025 and 0.090 in, respectively.

3. A minimum contact width of 2.0 in between shims and anchor head will be required.

4. The head will be protected at all times with a corrosion-inhibiting grease.

4.4 IN-SERVICE INSPECTION OF TENDONS

The NRC does not have a staff position for in-service inspection of posttensioned foundation anchors and will review programs for such inspection on a case-by-case basis. However, the NRC regulatory guides 1.35 and 1.90,(17);(18) governing in-service inspection of grouted tendons in posttensioned concrete containment structures have been reviewed and the program at North Anna will incorporate the applicable sections of these guidelines. Specifically, the program at North Anna will be designed to permit visual inspection of the anchorage assembly hardware (shims, bearing plate, anchor head, and the buttonheads) of all anchors, to permit monitoring changes in the prestress level and the effectiveness of the corrosion protection system of selected anchors, and to permit lift-off tests. A technical specification will be developed to provide the requirements and procedures for performing in-service inspection.

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TABLE 2-1*
PHYSICAL CHARACTERISTICS OF TEST ANCHORS

ANCHOR NO.	1	2	3	4	5	6	7
Tendon Type	Wire**	Strand+	Wire	Strand	Wire	Wire	Wire
Overall Length	55'0"	55'0"++	45'0"	45'0"++	45'0"	55'0"	55'0"
Bonded Length	30'1"	29'6"	20'2"	19'6"	20'2"	29'9"	30'-0"
Unbonded Length	24'11"	25'6"	24'10"	25'6"	24'10"	25'3"	25'0"
Borehole Length	57'0"	57'0"	47'0"	47'0"	56'0"	57'0"	57'0"
Distance Bearing Plate-Grout Extensometer Anchor Points	1@30'0" 1@40'0"	1@30'0" 2@40'0"	1@28'0" 1@36'0"	1@28'0" 1@36'0"	1@28'0" 1@36'0"	1@30'0" 1@40'0" 1@50'9"	1@30'0" 1@40'0" 1@50'0"
Grout Type	MB814	MB814	MB814	MB814	MB814	Type II+N	Type II+N
Water/Cement Ratio	2 Gal/ 55# Bag	2 Gal/ 55# Bag	2 Gal/ 55# Bag	2 Gal 55# Bag	2.4 Gal/ 55# Bag	5 Gal/ 94# Bag	5 Gal/ 94# Bag

*Modified after Table I of Appendix B

**Prescon 46, 0.25" Ø wires of 240 KSI steel with 542.8 K GUTS

+VSL 13, 0.5" Ø strands of 270 KSI steel with 537 K GUTS

++Does not include a stressing tail of approximately 5 ft.

TABLE 3-1

WATER CHEMISTRY

	LAKE ANNA WATER			GROUNDWATER
	3-31-75	5-4-76	10-22-78	5-8-74
pH	7.1	7.0	7.5	7.4
Sulfate (ppm)	8.5	6.0	1.0	N/A*
Sulfide (ppm)	N/A	N/A	0.05	0.0
Chlorides (ppm)	3.1	2.1	4.0	2.10
Nitrates (ppm)	N/A	34.0	0.4	N/A

* Not available

TABLE 3-2

Groundwater Temperatures

Hole No.	Date Taken	Air Temp	Groundwater Level*	Temperatures at Depths Below Groundwater Table					
				0'	10'	20'	30'	40'	50'
R.A.T.H. #1	12-9-75	40°	20'	54					
	12-23-75	30	8'	53	54	55	56	56	56
	12-29-75	33°	8'	53	54.5	55.5	56.5	56	56.5
	1-5-76	30°	6'	53	54	55	56	56	56.5
R.A.T.H. #3	12-23-75	30°	19'	53	53.5	54	55.5		
	12-29-75	33°	19'	53	53.5	54	55.5		
	1-5-76	30°	17'	53	53	54	55.5		
G.T.H. #1	12-18-75	33°	3'	53	54	57	58.5	57.5	57
	12-19-75	29°	5'	53	55.5	57	57	56	
	12-22-75	31°	7'	53	55.5	57	57	56	
	12-23-75	30°	5'	53	55.5	57	56.5	56	
	12-29-75	33°	5'	53	55.5	57	56.5	56	
	1-5-76	30°	3'	53	54	56	57	57	

R.A.T.H. = Rock Anchor Test Hole
G.T.H. = Grout Test Hole
* Below Surface

TABLE 3-3

LOG OF GROUT TEST HOLE NO. 2

DESCRIPTION OF CORE	LENGTH	TOTAL LENGTH
Concrete	7'-10"	7'-10"
Grout/Rock Interface Bonded	0'-11"	8'-9"
Grout/Rock Interface Unbonded	0'- 2"	8'-11"
Grout/Rock Interface Bonded	4'-2"	13'-1"
Grout/Rock Interface Unbonded	1'-1"	14'-2"
Grout/Rock Interface Bonded	0'-6"	14'-8"
Grout/Rock Interface Unbonded	7'-4"	22'-0"

NOTES:

1. Grout test hole no. 2 filled with MB814 with 2.4 gal/55# bag
2. This table modified after Figure 60 of Appendix B

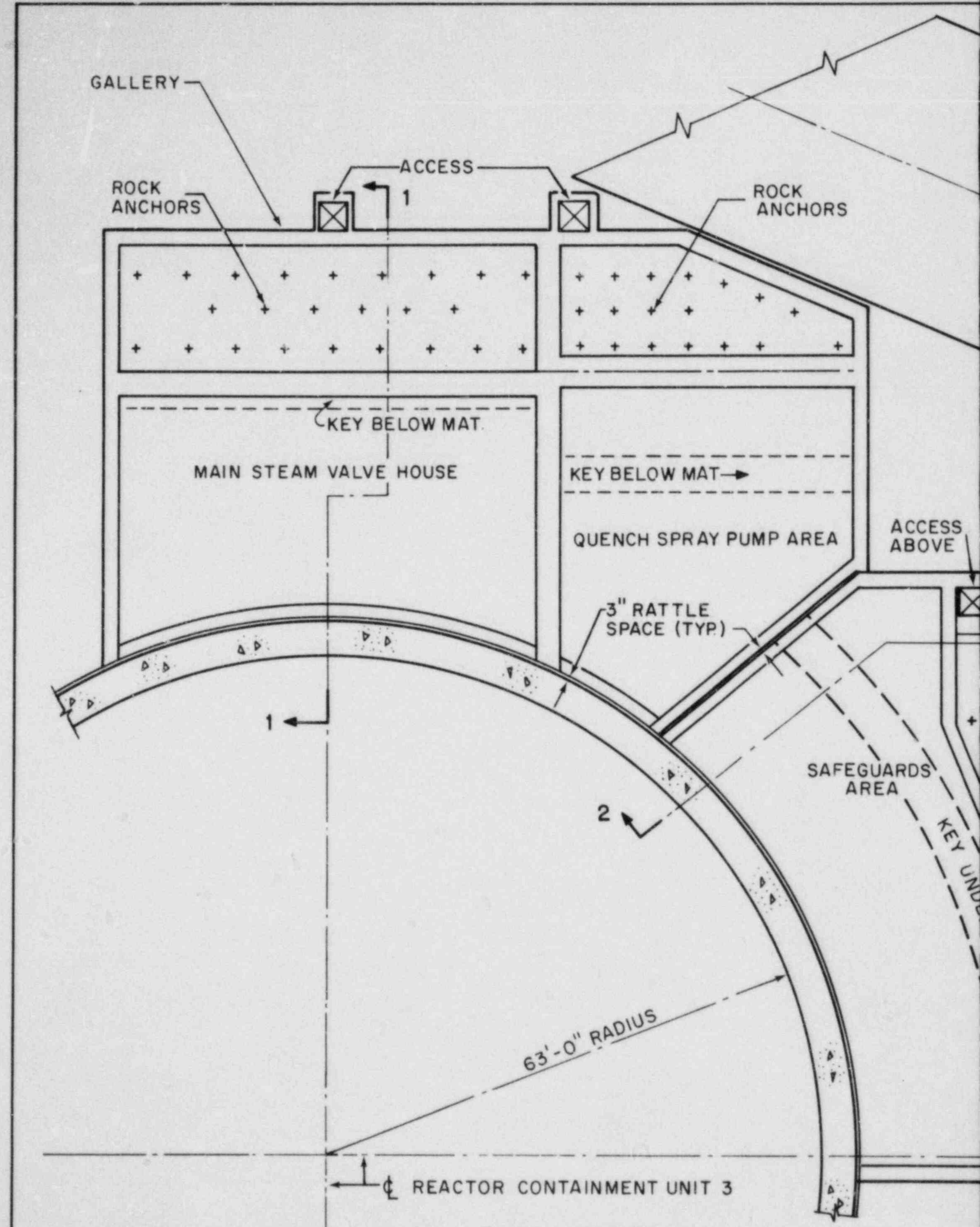
TABLE 3-4

LOG OF GROUT TEST HOLE NO. 3

DESCRIPTION OF CORE		LENGTH	TOTAL LENGTH
Run 1	Concrete	1'-0"	1'-0"
	Grout	13'-0"	14'-0"
	Grout/Rock Interface Bonded	4'-0"	18'-0"
	Grout	0'-4"	18'-4"
Run 2	Concrete	1'-0"	1'-0"
	Grout	17'-0"	18'-0"
Run 3	Concrete	1'-0"	1'-0"
	Grout	12'-6"	13'-6"
Run 4	Concrete	1'-0"	1'-0"
	Granite	4'-0"	5'-0"
	Grout/Rock Interface Bonded	4'-6"	9'-6"

NOTES:

1. Grout test hole no. 3 filled with Portland Type II grout with 5.0 gal/bag.
2. This table modified after Figure 61 of Appendix B


GALLERY
ACCESS
1
ROCK ANCHORS
ROCK ANCHORS
KEY BELOW MAT
MAIN STEAM VALVE HOUSE
KEY BELOW MAT
QUENCH SPRAY PUMP AREA
ACCESS ABOVE
3" RATTLE SPACE (TYP.)
1
SAFEGUARDS AREA
2
KEY UND
63'-0" RADIUS
℄ REACTOR CONTAINMENT UNIT 3

PLAN - UNIT 3

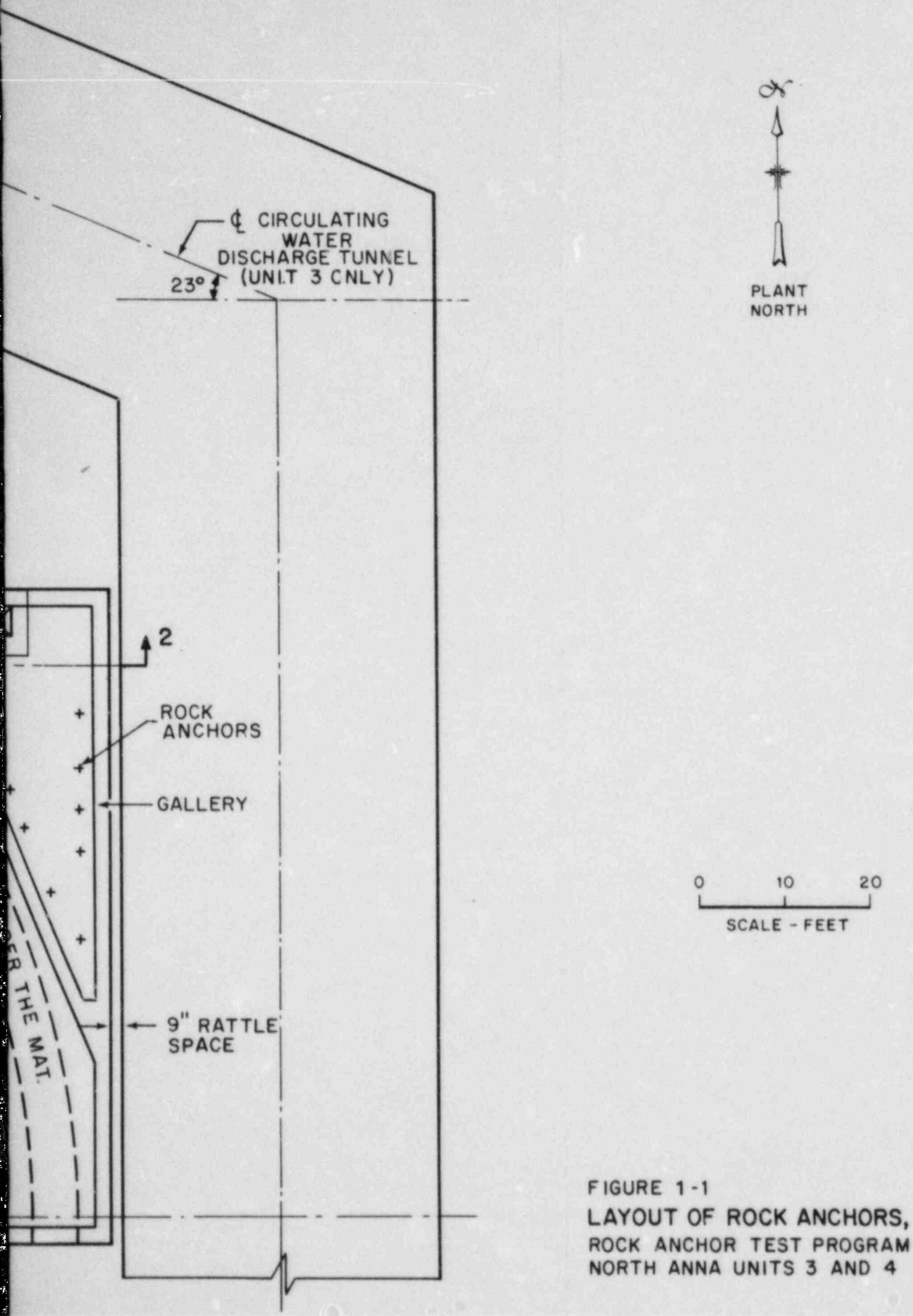


FIGURE 1-1

LAYOUT OF ROCK ANCHORS, UNIT 3

ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4

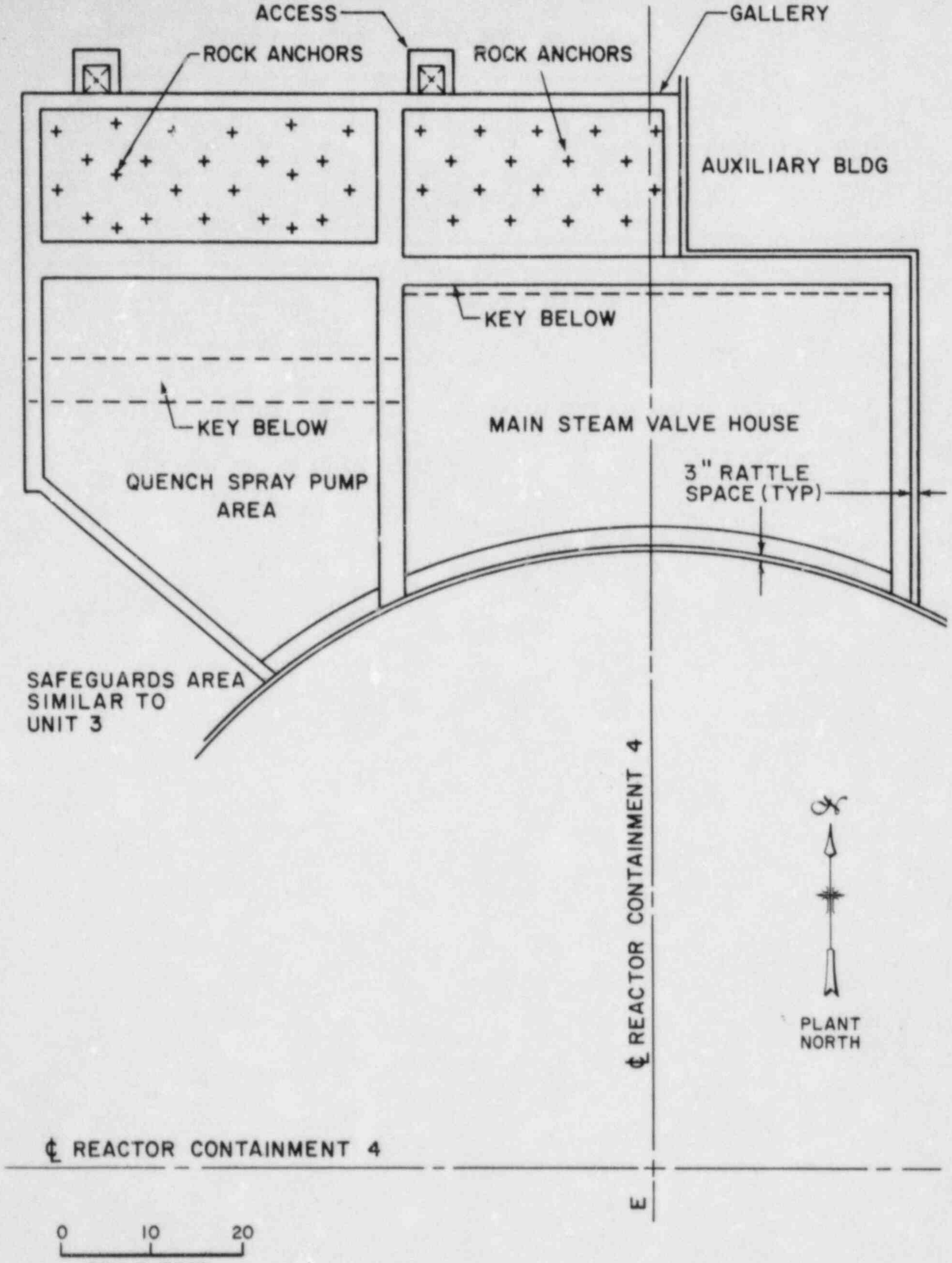
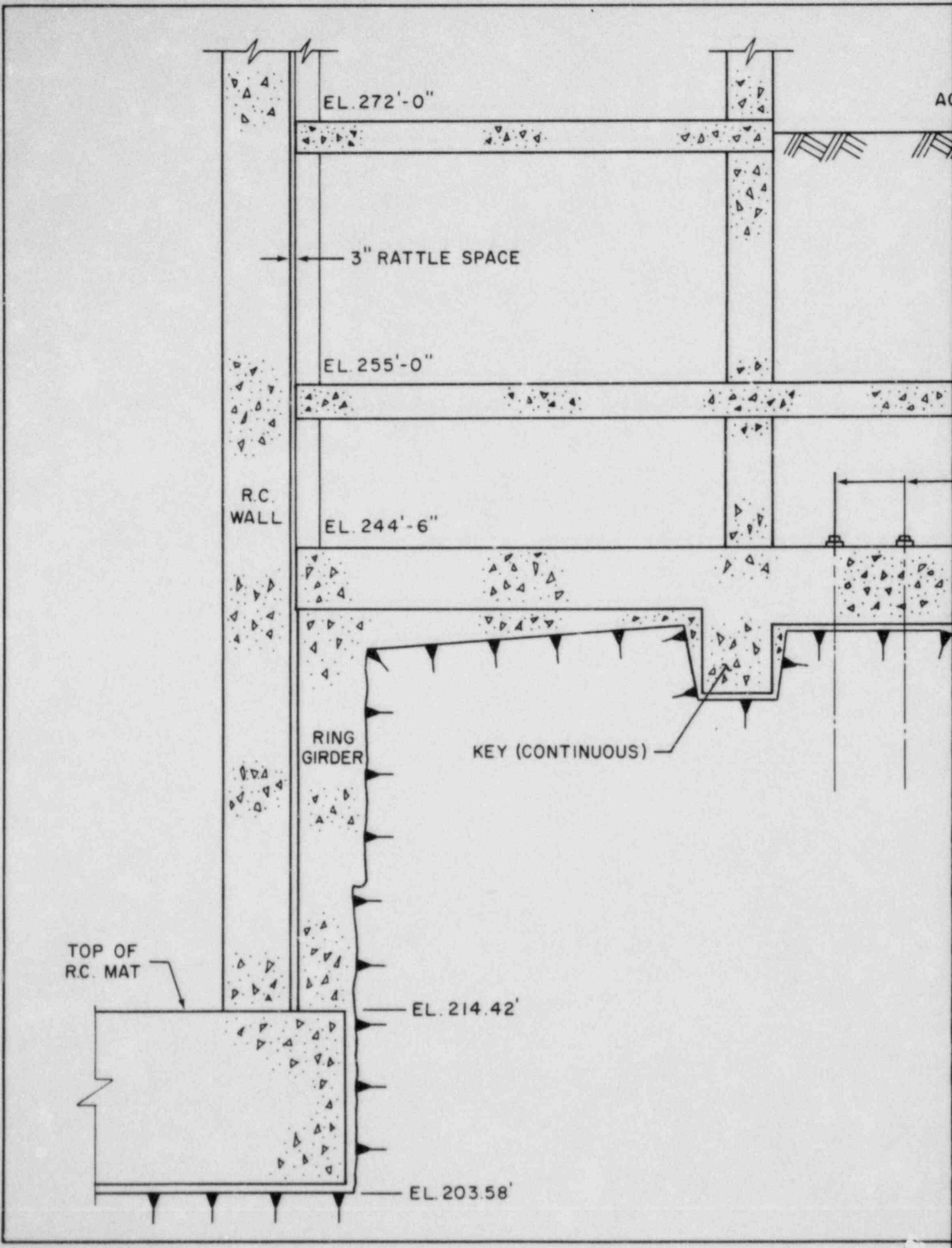
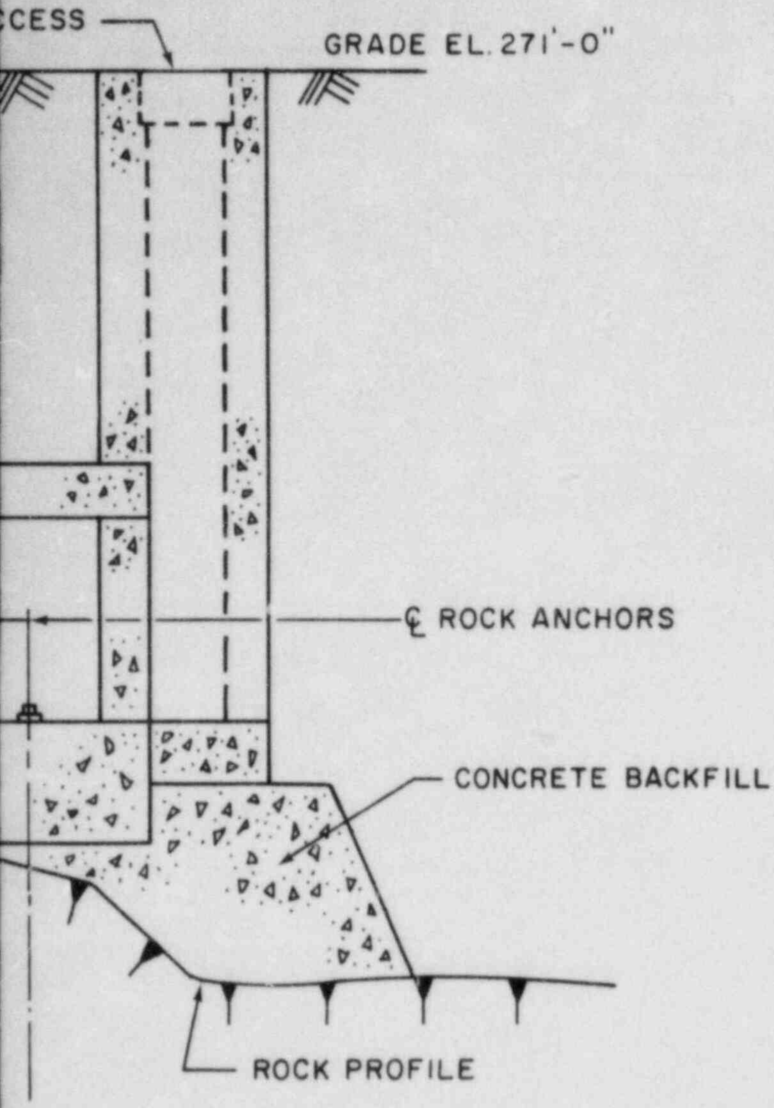


FIGURE 1-2

LAYOUT OF ROCK ANCHORS, UNIT 4

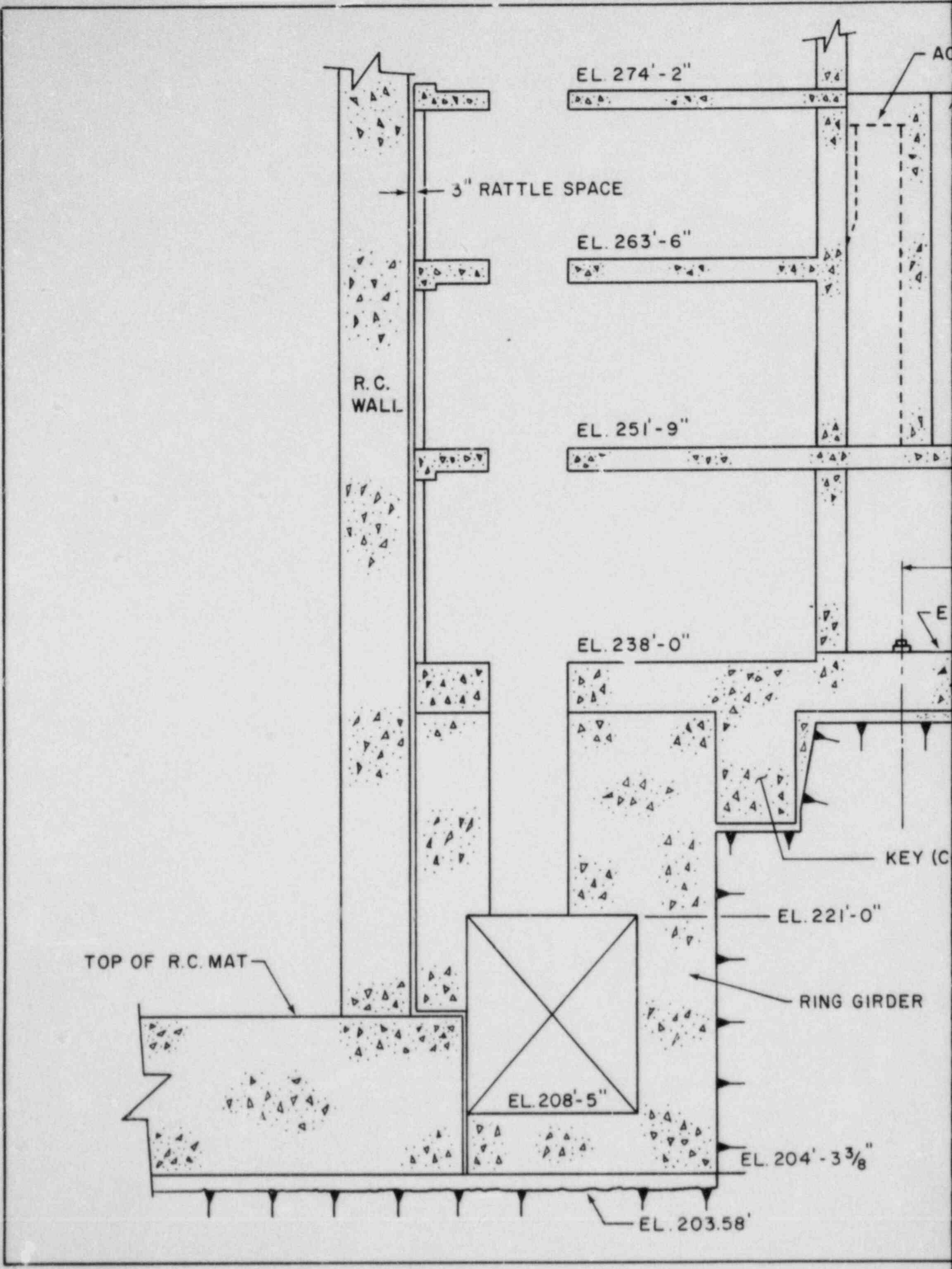
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4

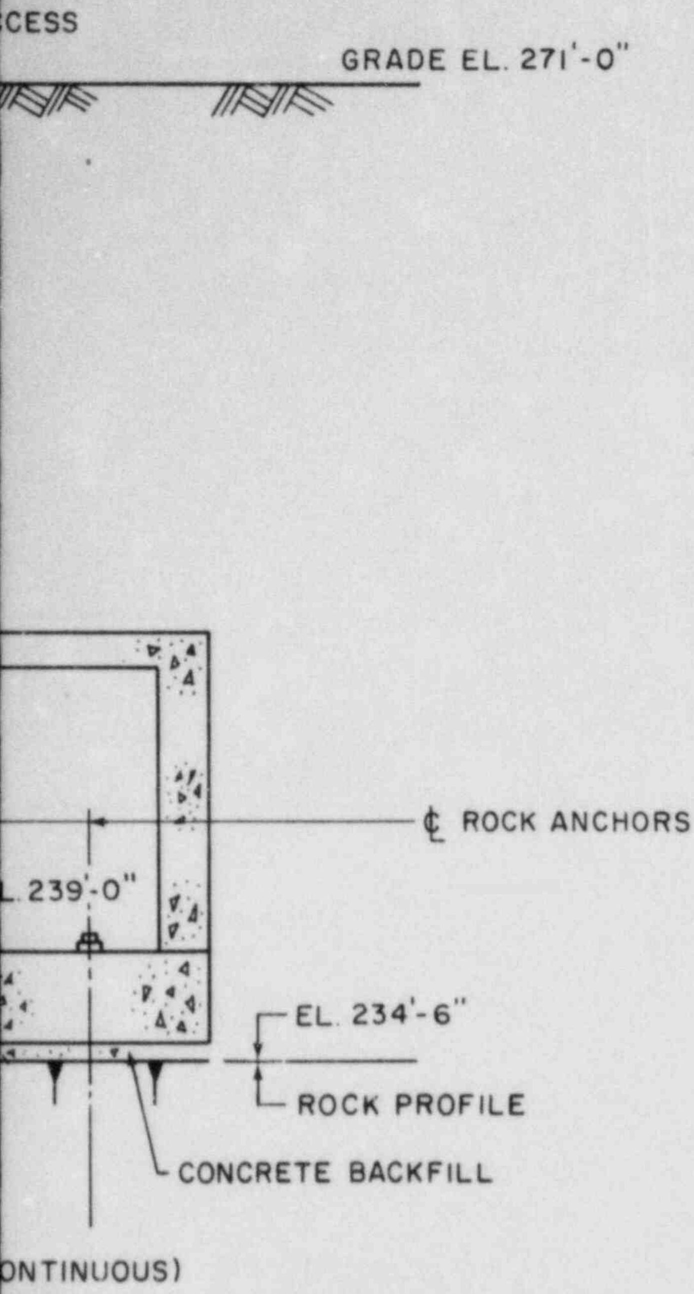

EL. 272'-0"
AC
3" RATTLE SPACE
EL. 255'-0"
R.C. WALL
EL. 244'-6"
RING GIRDER
KEY (CONTINUOUS)
TOP OF R.C. MAT
EL. 214.42'
EL. 203.58'



NOTE
SEE FIGURE 1-1 FOR SECTION LOCATION

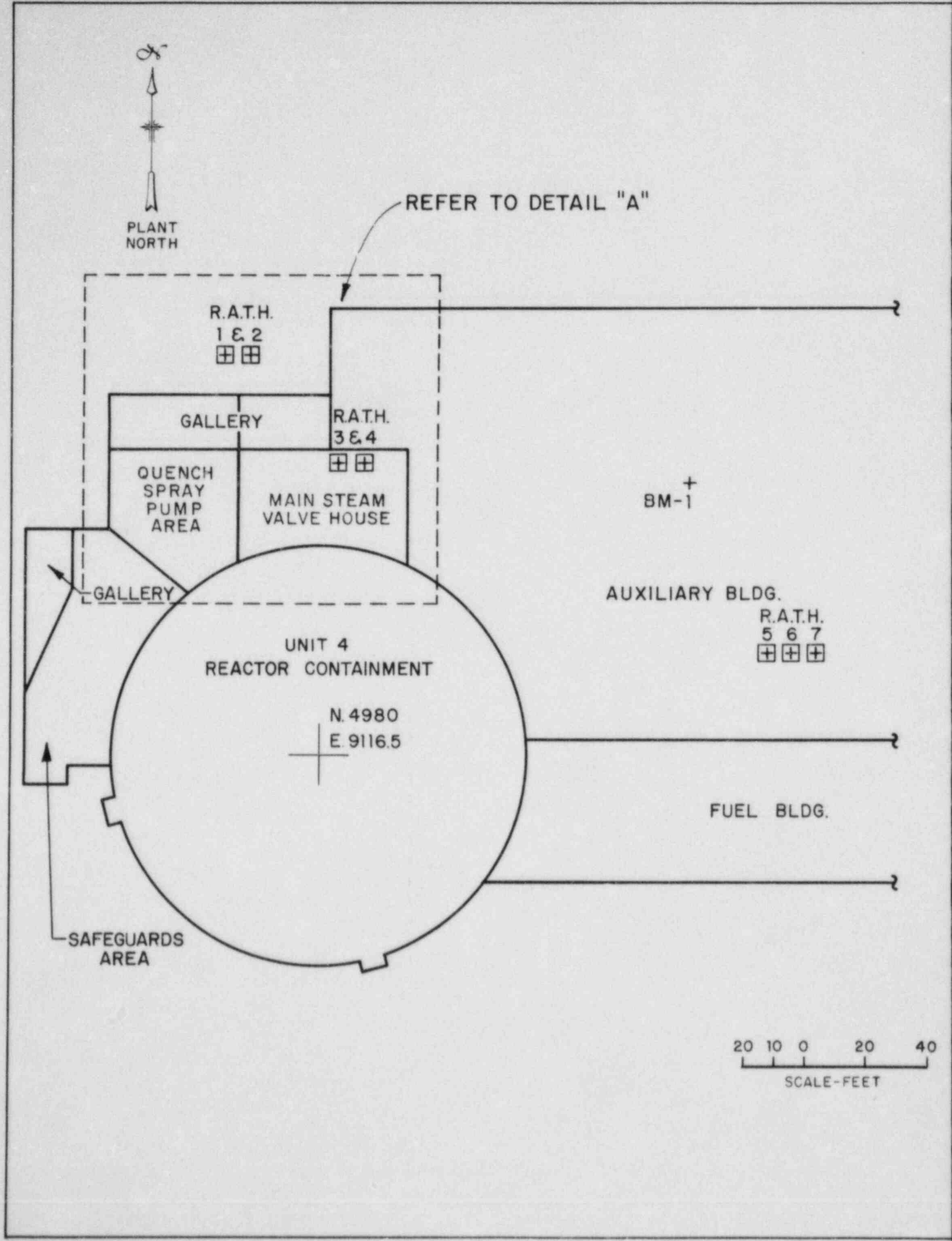
FIGURE 1-3
SECTION 1-1
MAIN STEAM VALVE HOUSE, UNIT 3
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4


AC
EL. 274'-2"
3" RATTLE SPACE
EL. 263'-6"
R.C. WALL
EL. 251'-9"
E
EL. 238'-0"
KEY (C
EL. 221'-0"
TOP OF R.C. MAT
RING GIRDER
EL. 208'-5"
EL. 204'-3 3/8"
EL. 203.58'

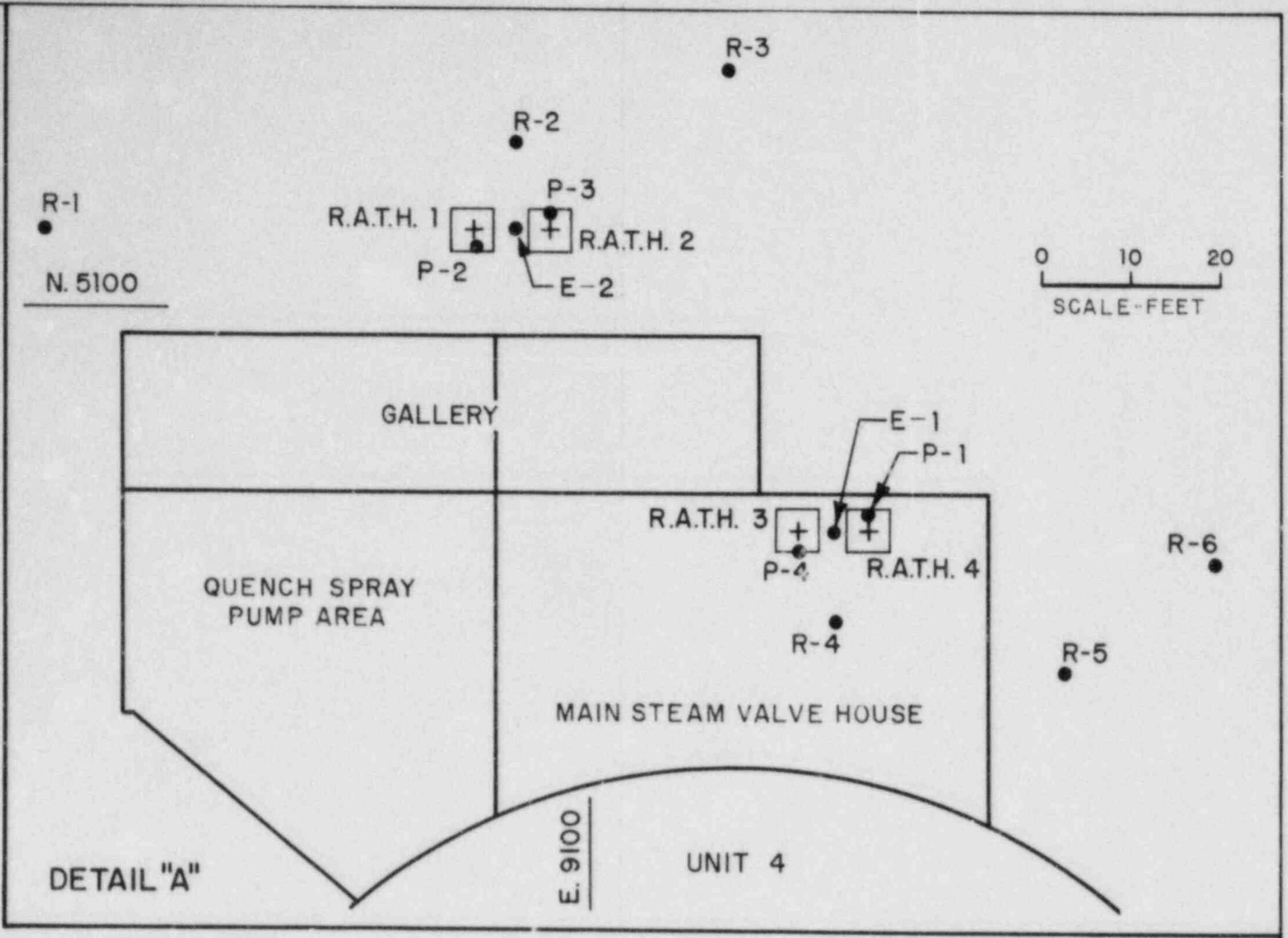


NOTE:
SEE FIGURE 1-1 FOR SECTION LOCATION

FIGURE 1-4
SECTION 2-2
SAFEGUARDS BUILDING, UNIT 3
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



PLANT NORTH
REFER TO DETAIL "A"
R.A.T.H. 1 & 2
GALLERY
R.A.T.H. 3 & 4
QUENCH SPRAY PUMP AREA
MAIN STEAM VALVE HOUSE
BM-1
GALLERY
AUXILIARY BLDG.
R.A.T.H. 5 6 7
UNIT 4 REACTOR CONTAINMENT
N. 4980
E. 9116.5
FUEL BLDG.
SAFEGUARDS AREA
20 10 0 20 40
SCALE-FEET



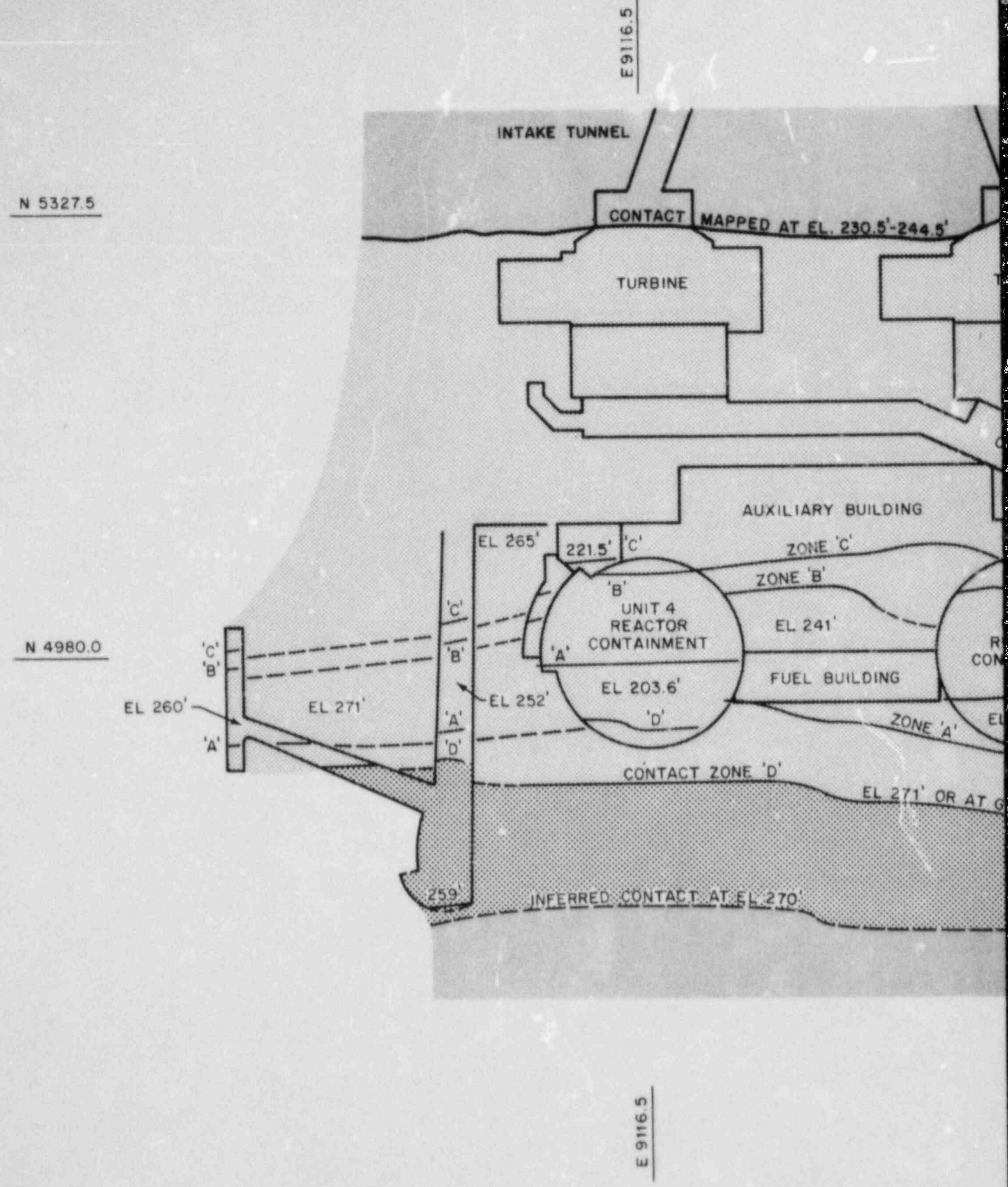
COORDINATES

R.A.T.H. 1	N.5109	R - 1	N.5109	E - 1	N.5075
	E.9086.5		E.9037.5		E.9122.5
R.A.T.H. 2	N.5109	R - 2	N.5118.9	E - 2	N.5109
	E.9094.5		E.9091.3		E.9090.5
R.A.T.H. 3	N.5075	R - 3	N.5127.2	P - 1	N.5077
	E.9123.5		E.9114.4		E.9131.5
R.A.T.H. 4	N.5075	R - 4	N.5064.9	P - 2	N.5107
	E.9131.5		E.9126.9		E.9086.5
R.A.T.H. 5	N.5013	R - 5	N.5059.56	P - 3	N.5111
	E.9250		E.9152.9		E.9094.5
R.A.T.H. 6	N.5013	R - 6	N.5071.7	P - 4	N.5073
	E.9258		E.9170.5		E.9123.5
R.A.T.H. 7	N.5013	BM - 1	N.5069.5		
	E.9266		E.9238.11		

NOTE: R-1 COORDINATE APPROXIMATE

LEGEND
R = REBAR
BM= BENCH MARK
E = EXTENSOMETER
P = TEST PAD POINT

FIGURE 1-5
ANCHOR AND SURVEY MONUMENT LOCATIONS
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4


E 9116.5
N 5327.5
INTAKE TUNNEL
CONTACT MAPPED AT EL. 230.5'-244.5'
TURBINE
AUXILIARY BUILDING
EL 265'
221.5'
'C'
ZONE 'C'
ZONE 'B'
'B'
UNIT 4
REACTOR
CONTAINMENT
EL 241'
'C'
'B'
N 4980.0
'C'
'B'
'A'
FUEL BUILDING
EL 203.6'
EL 252'
EL 260'
EL 271'
'A'
'D'
'D'
ZONE 'A'
'A'
CONTACT ZONE 'D'
EL 271' OR AT G
259'
INFERRED CONTACT AT EL 270'
E 9116.5

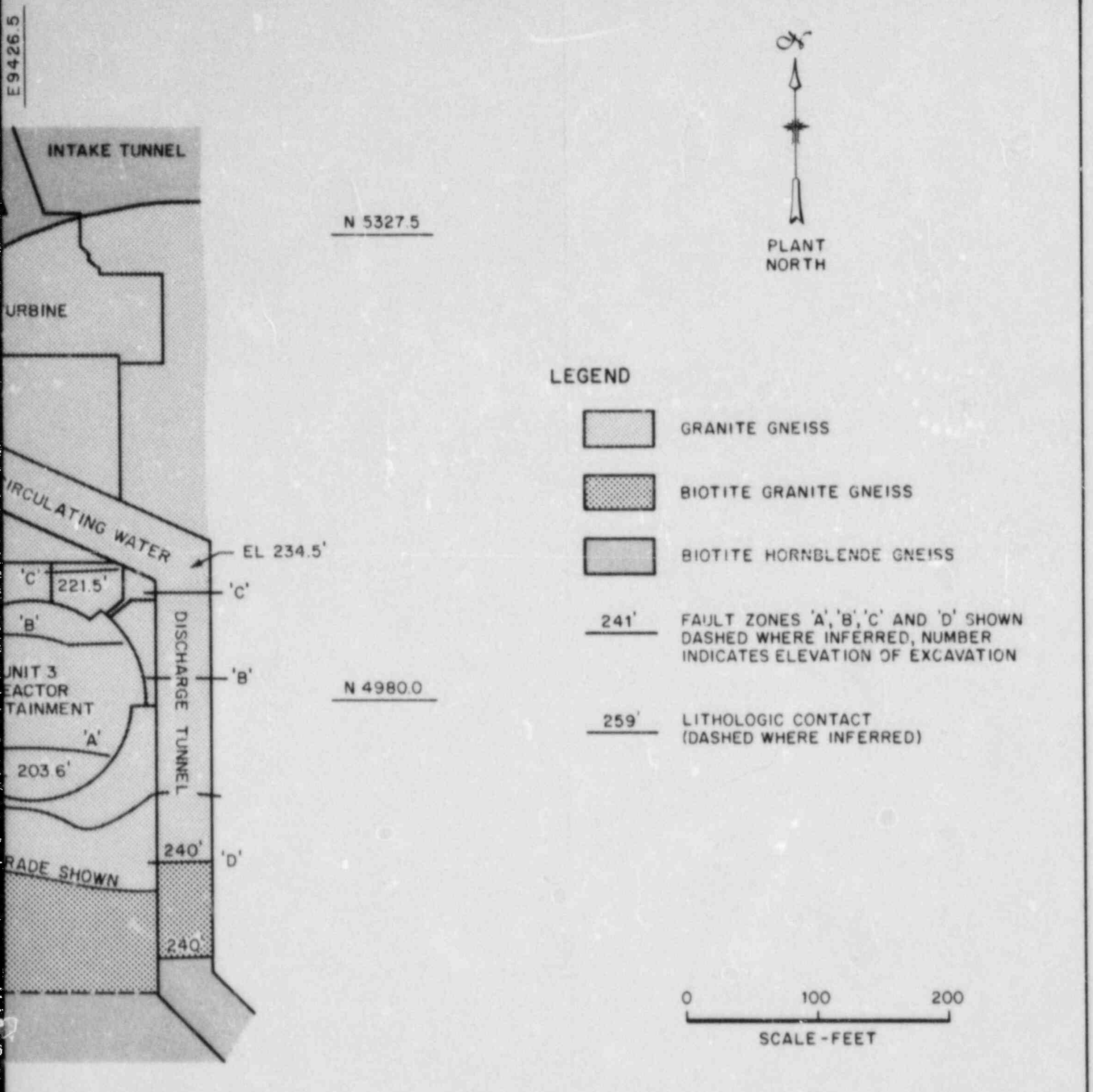


FIGURE 1-6
SITE GEOLOGIC MAP
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4

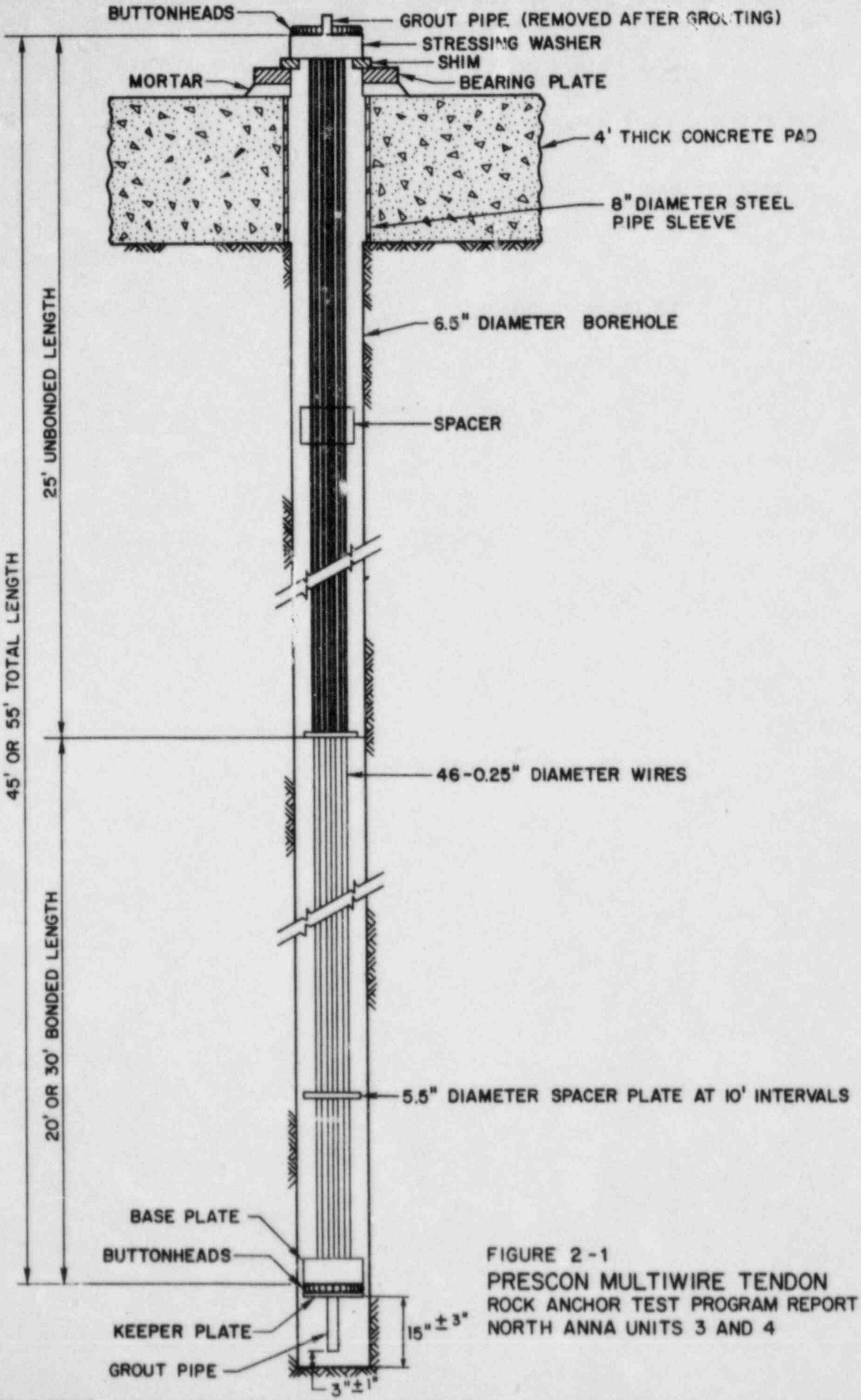


FIGURE 2-1
PRESCON MULTIWIRE TENDON
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4

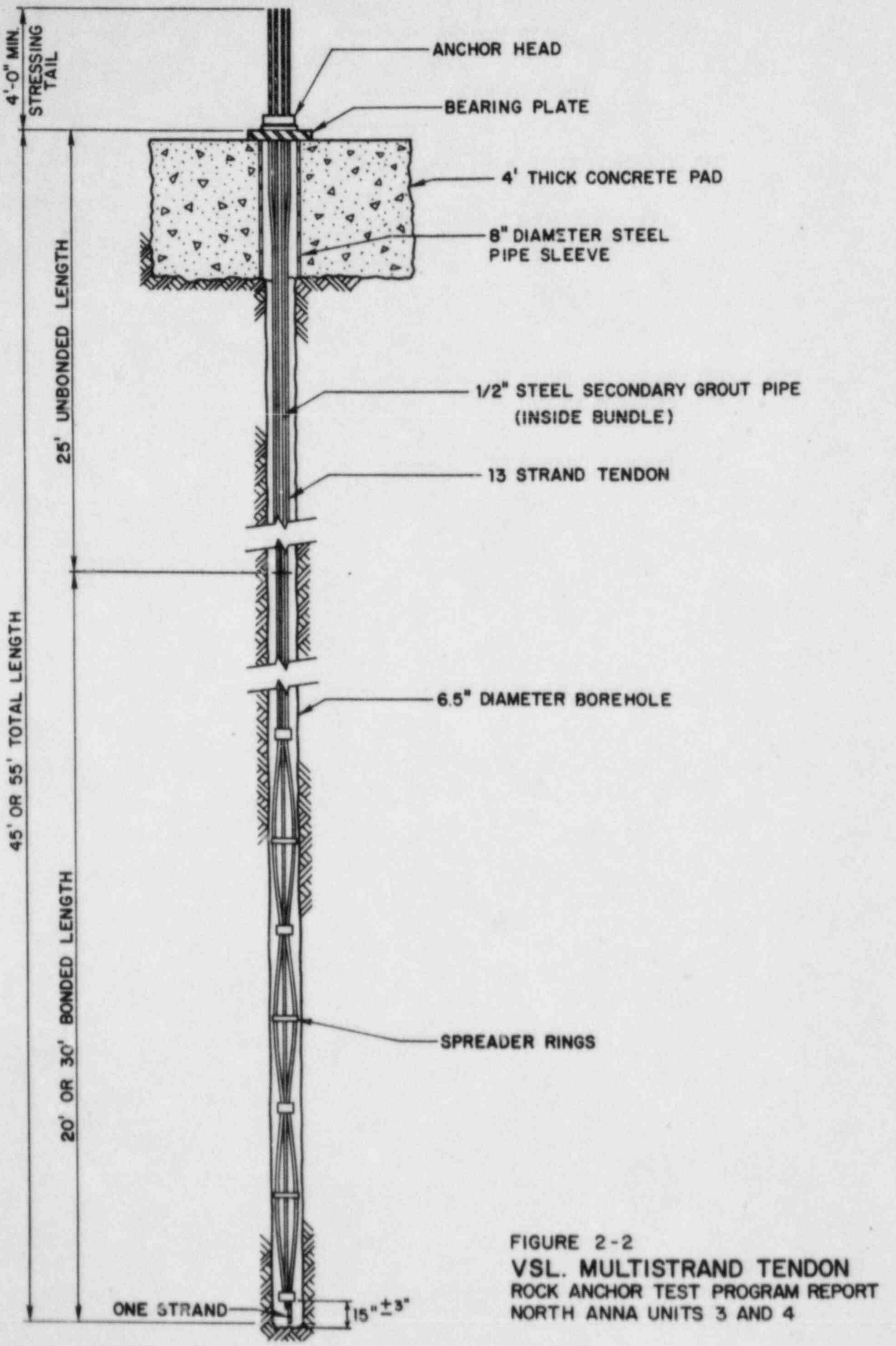


FIGURE 2-2

VSL. MULTISTRAND TENDON

ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4

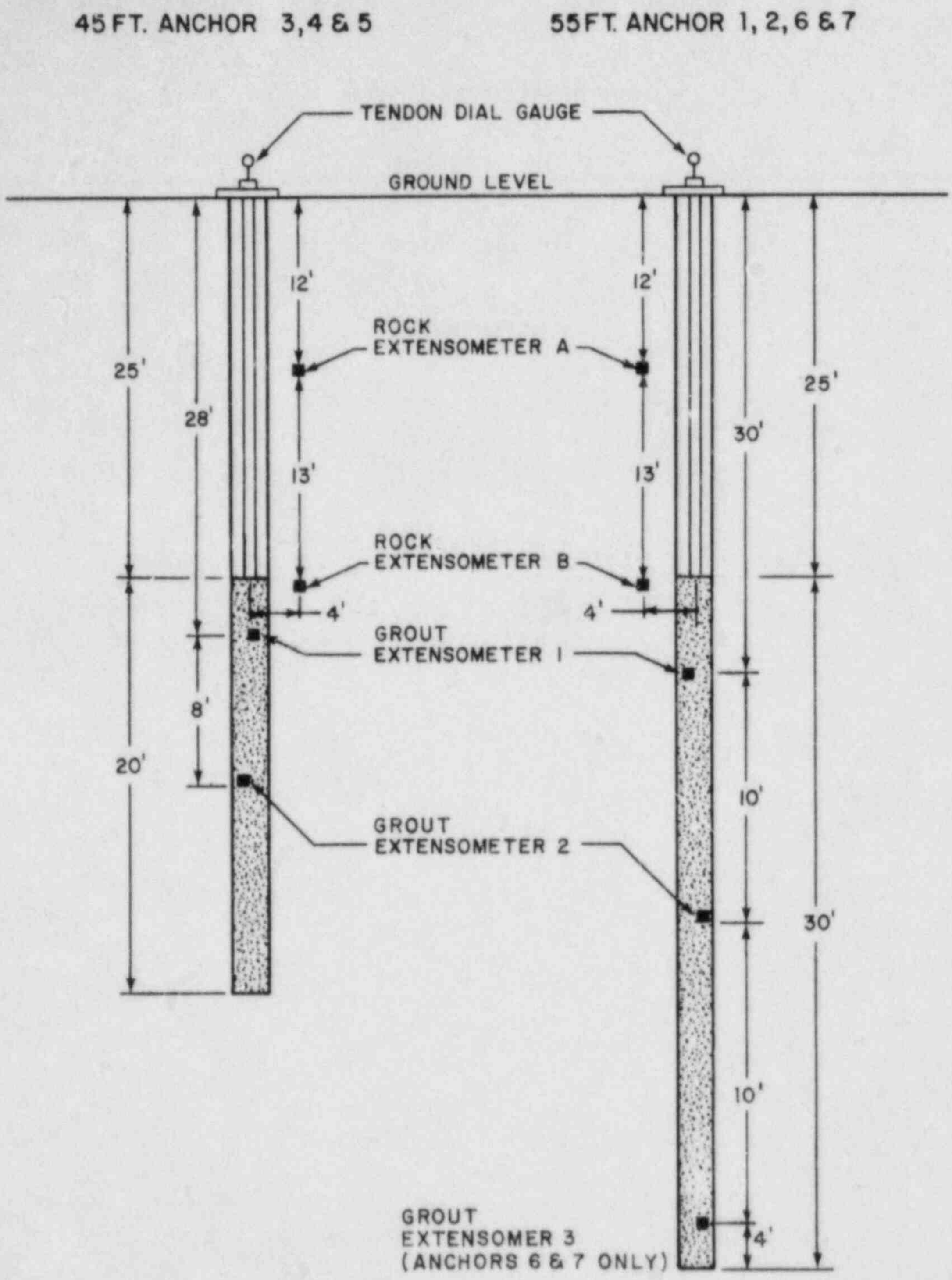


FIGURE 2-3

INSTRUMENTATION LAYOUT

ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4

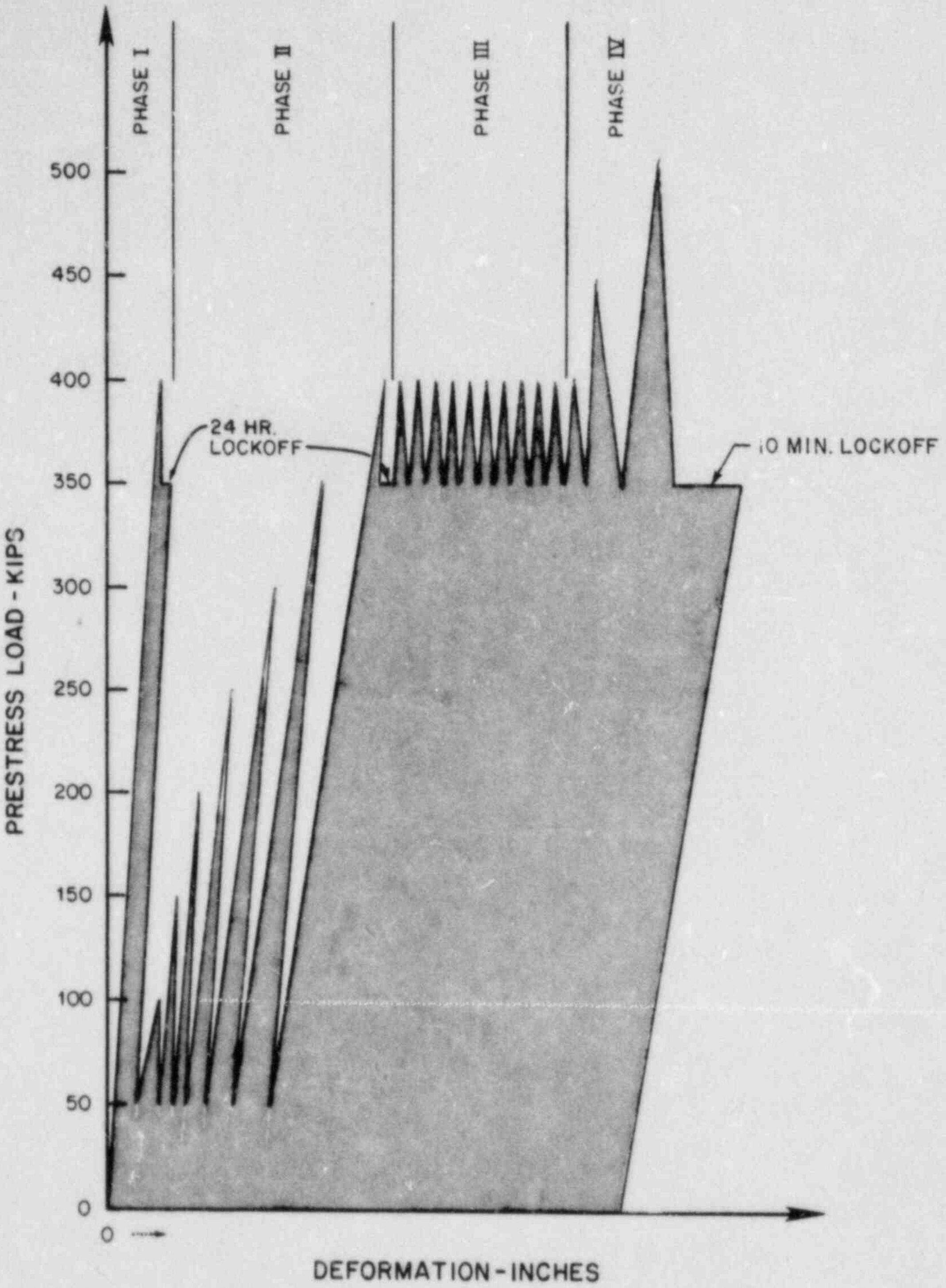
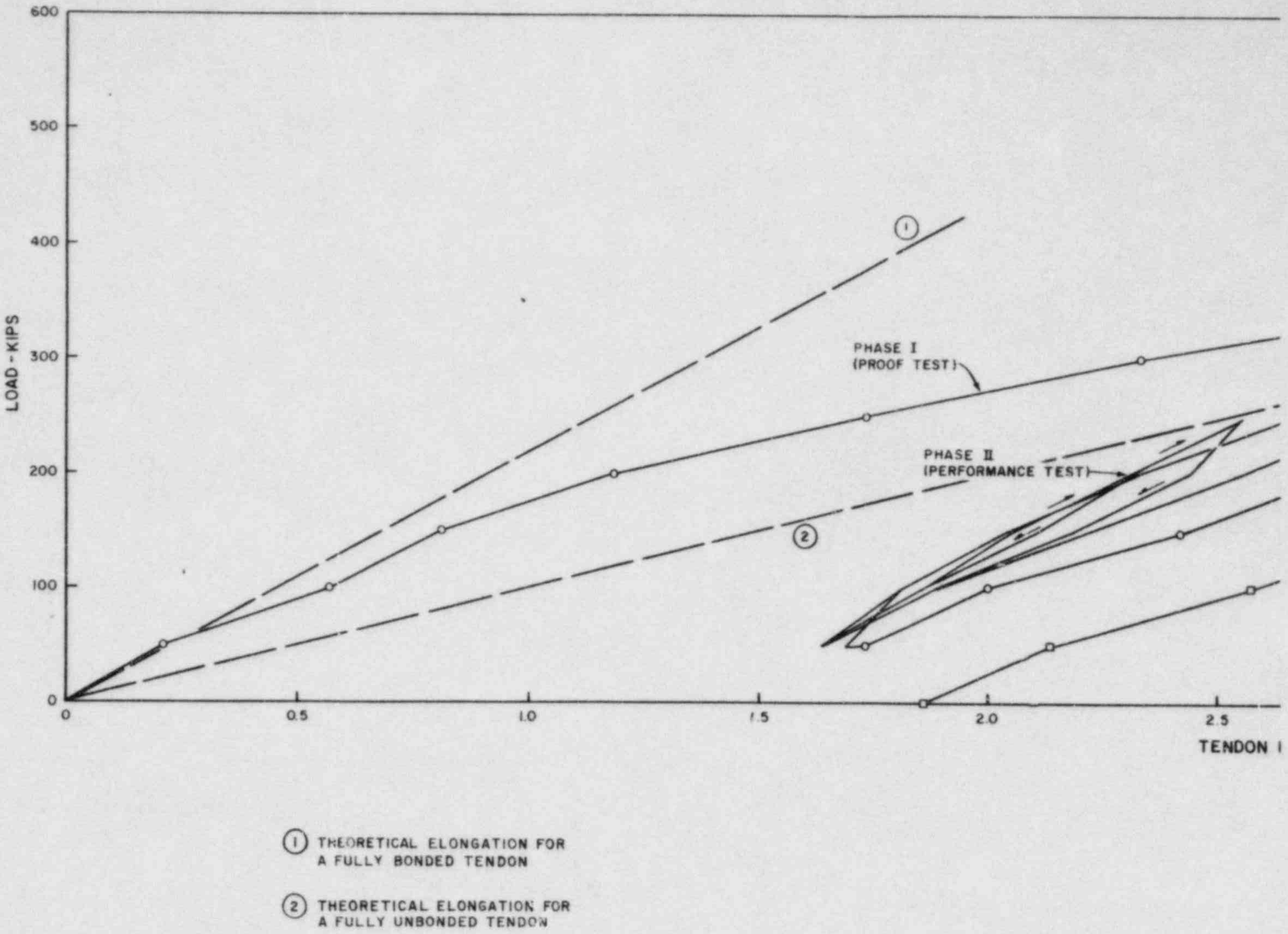


FIGURE 2-4
STRESSING SEQUENCE FOR TEST ANCHORS
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4


LOAD - KIPS
600
500
400
300
200
100
0
0
0.5
1.0
1.5
2.0
2.5
TENDON I
①
②
PHASE I
(PROOF TEST)
PHASE II
(PERFORMANCE TEST)
① THEORETICAL ELONGATION FOR A FULLY BONDED TENDON
② THEORETICAL ELONGATION FOR A FULLY UNBONDED TENDON

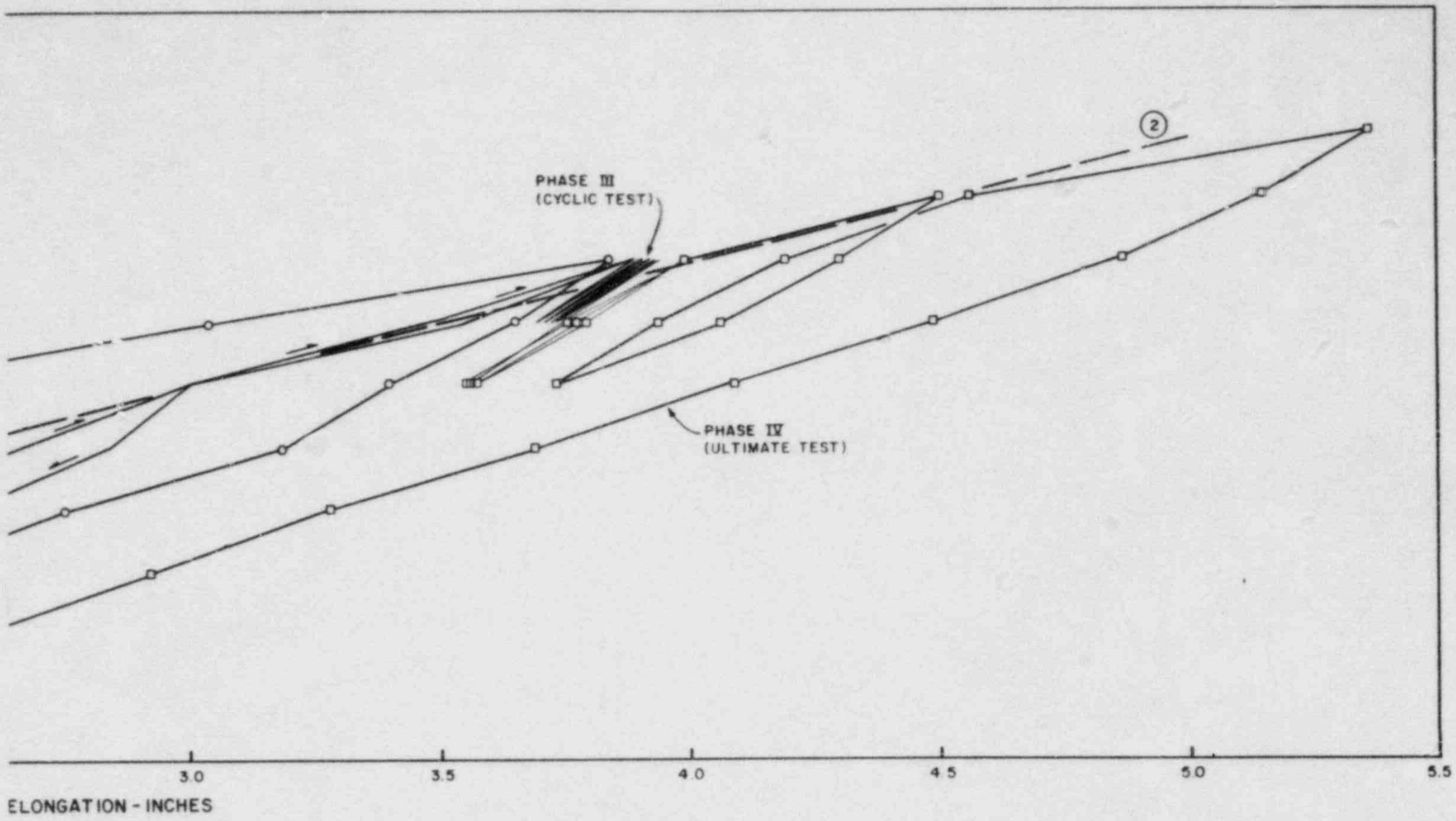
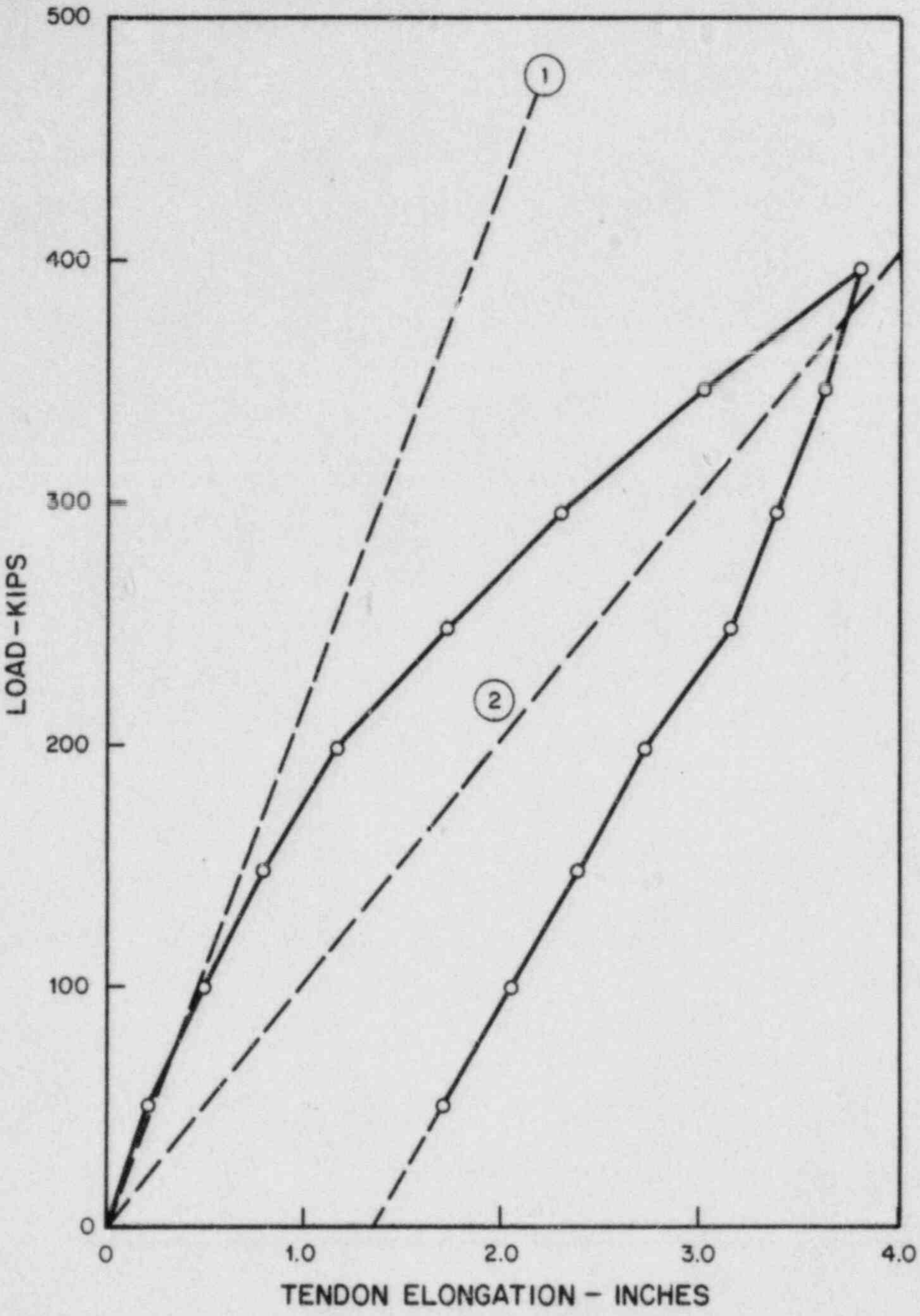
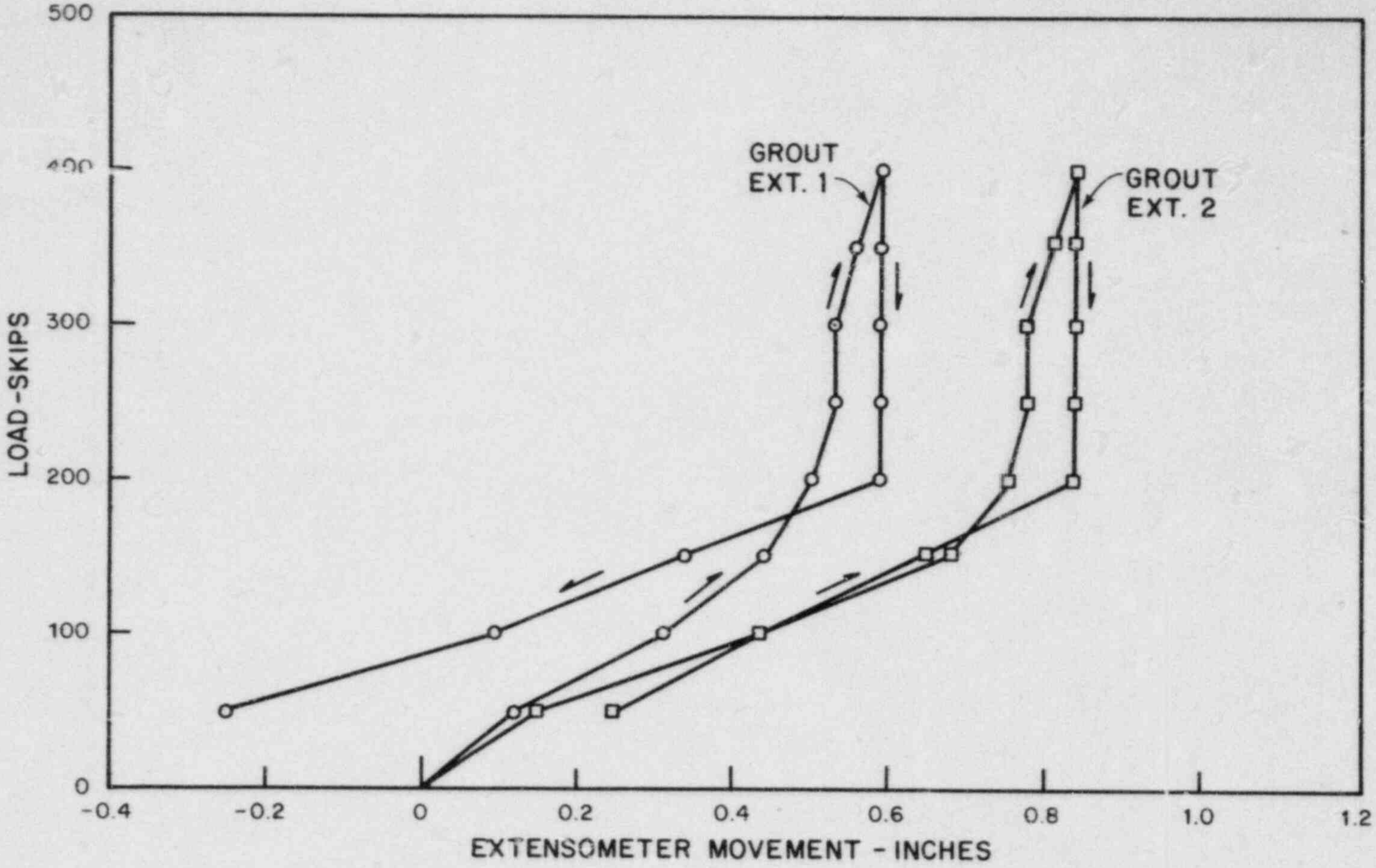


FIGURE 3-1
TENDON ELONGATION
PHASES I - IV STRESSING
TEST ANCHOR 1, 55 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



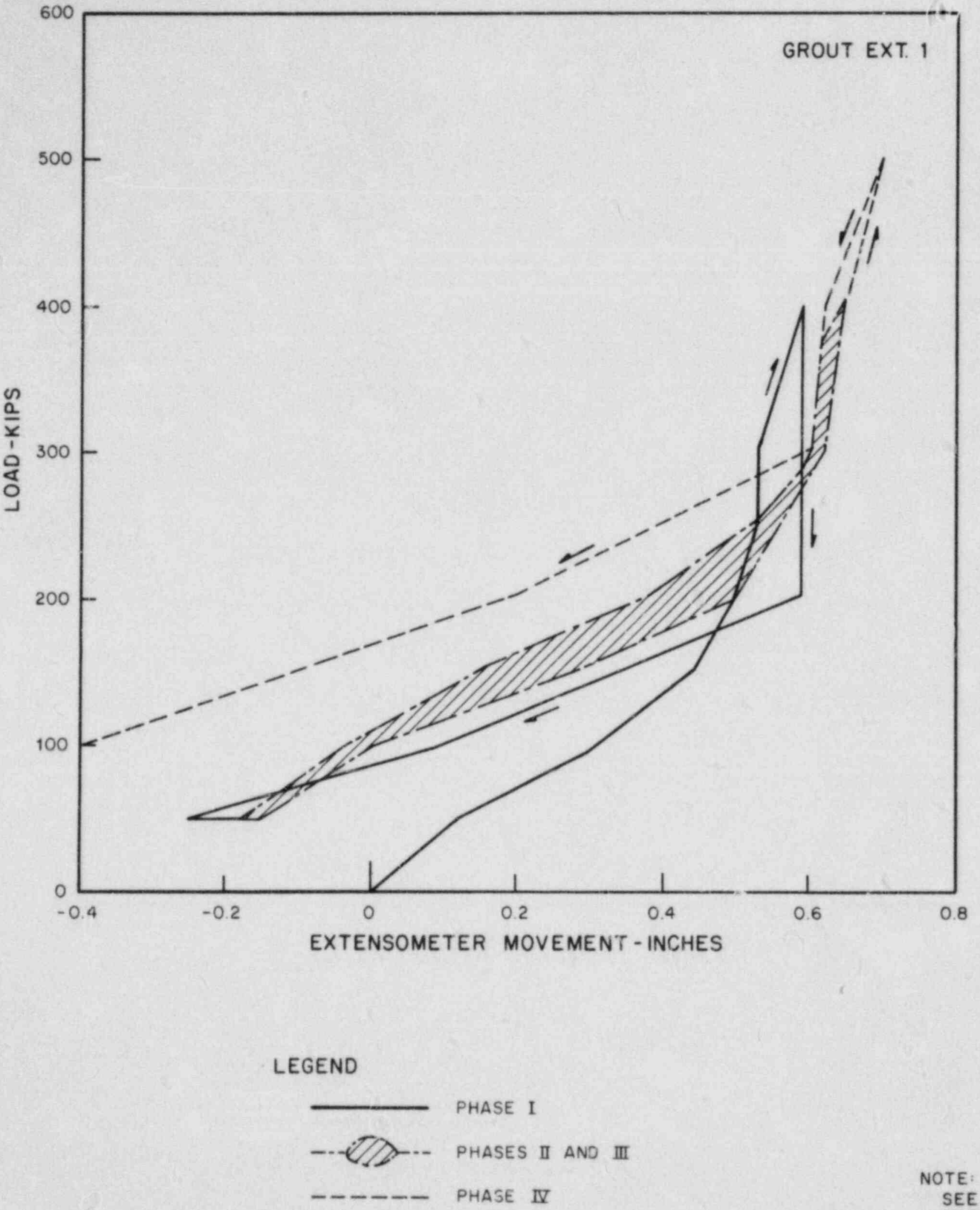
SEE NOTE ON FIGURE 3-1

FIGURE 3-2
TENDON ELONGATION
PHASE I STRESSING
TEST ANCHOR 1, 55 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4

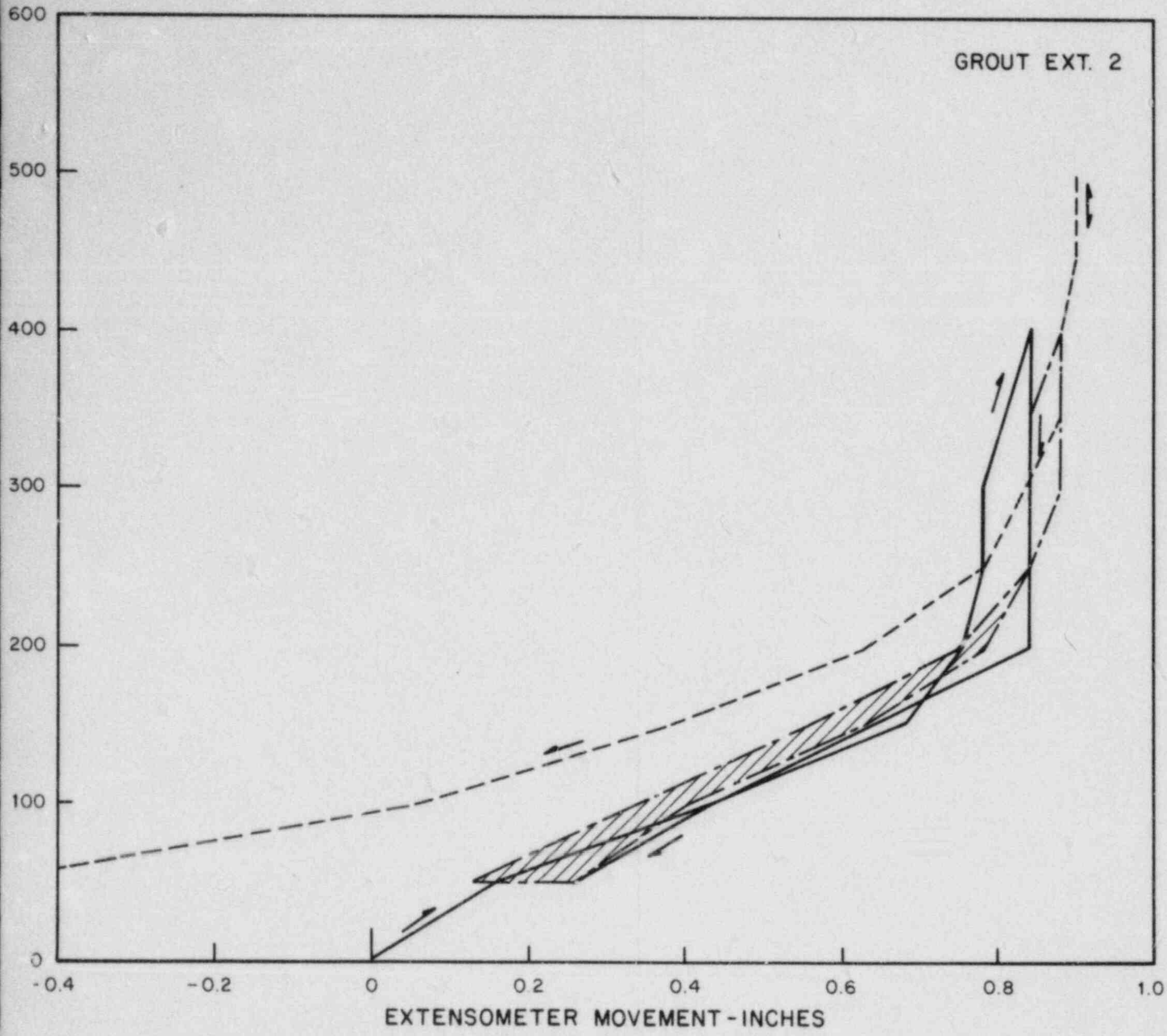


NOTE:
SEE FIGURE 2-3 FOR
EXTENSOMETER LOCATIONS.

FIGURE 3-3
EXTENSOMETER MOVEMENT
PHASE I STRESSING
TEST ANCHOR 1, 55 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



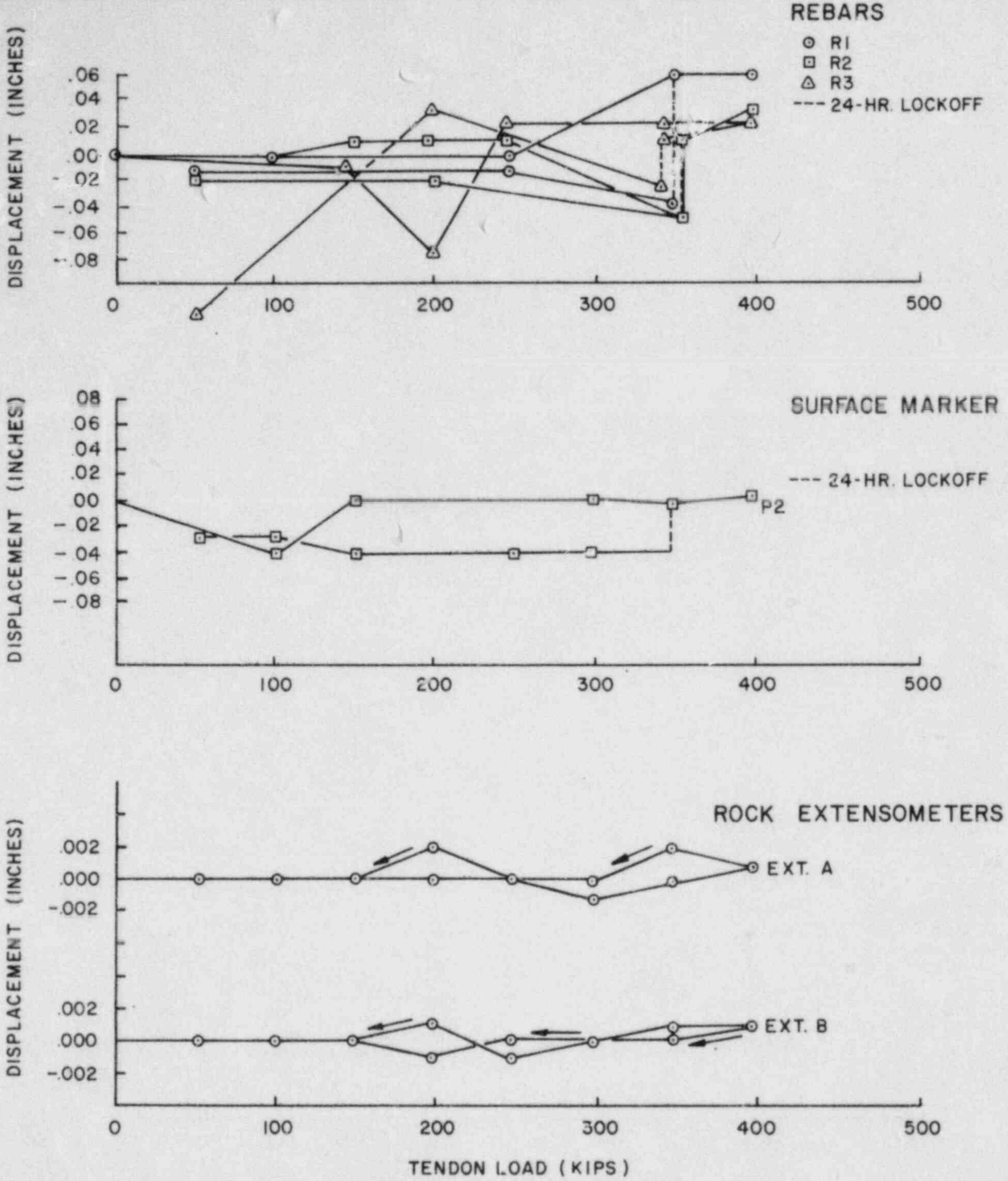
NOTE:
SEE FIGURE
EXTENSOM



2-3 FOR
TER LOCATIONS.

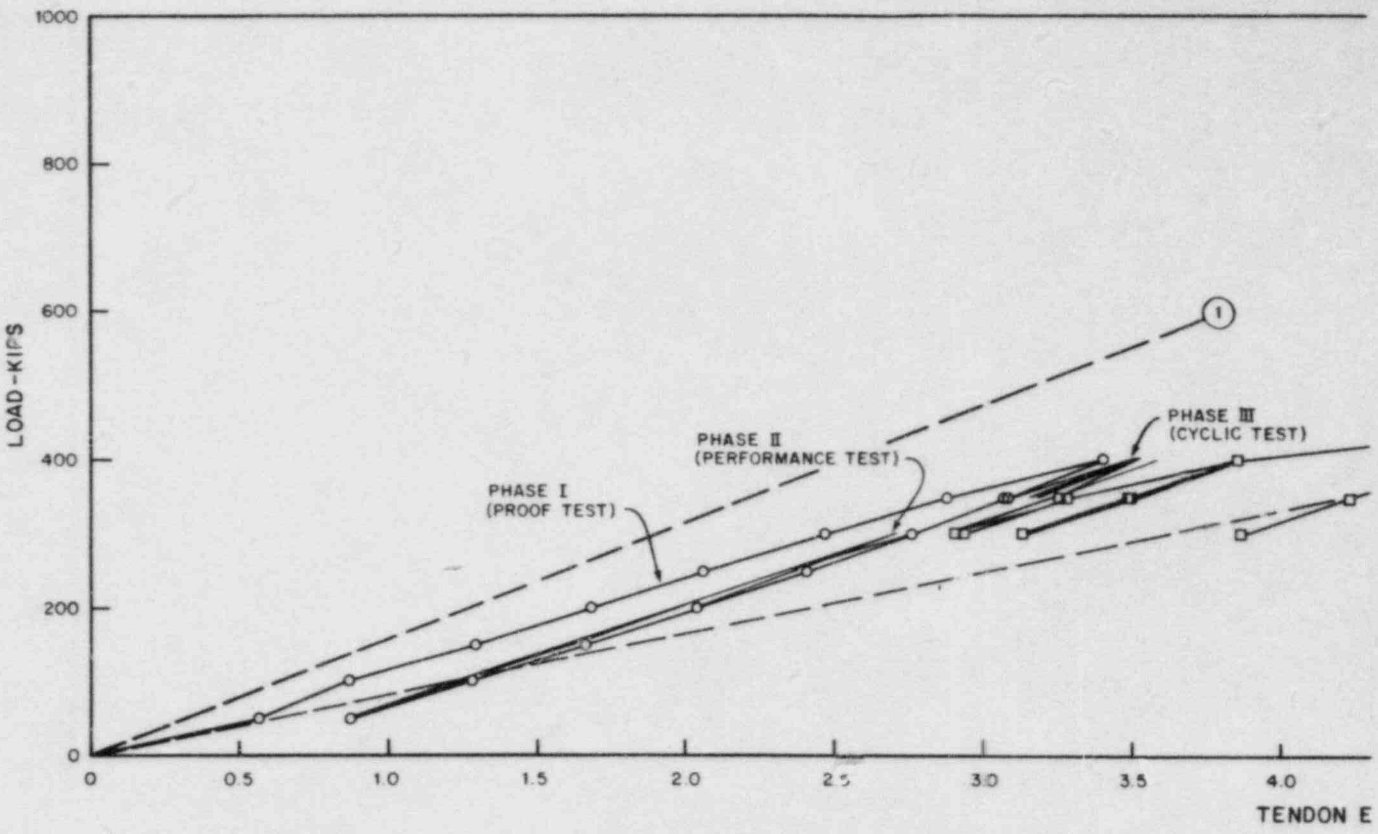
FIGURE 3-4

EXTENSOMETER MOVEMENT
PHASES I-IV STRESSING
TEST ANCHOR 1, 55 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



NOTE:
SEE FIGURES 1-5 AND 2-3 FOR LOCATION OF INSTRUMENTATION.

FIGURE 3-5
ROCK MOVEMENTS
PHASE I STRESSING
TEST ANCHOR 1,55FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



SEE NOTE ON FIGURE 3-1.

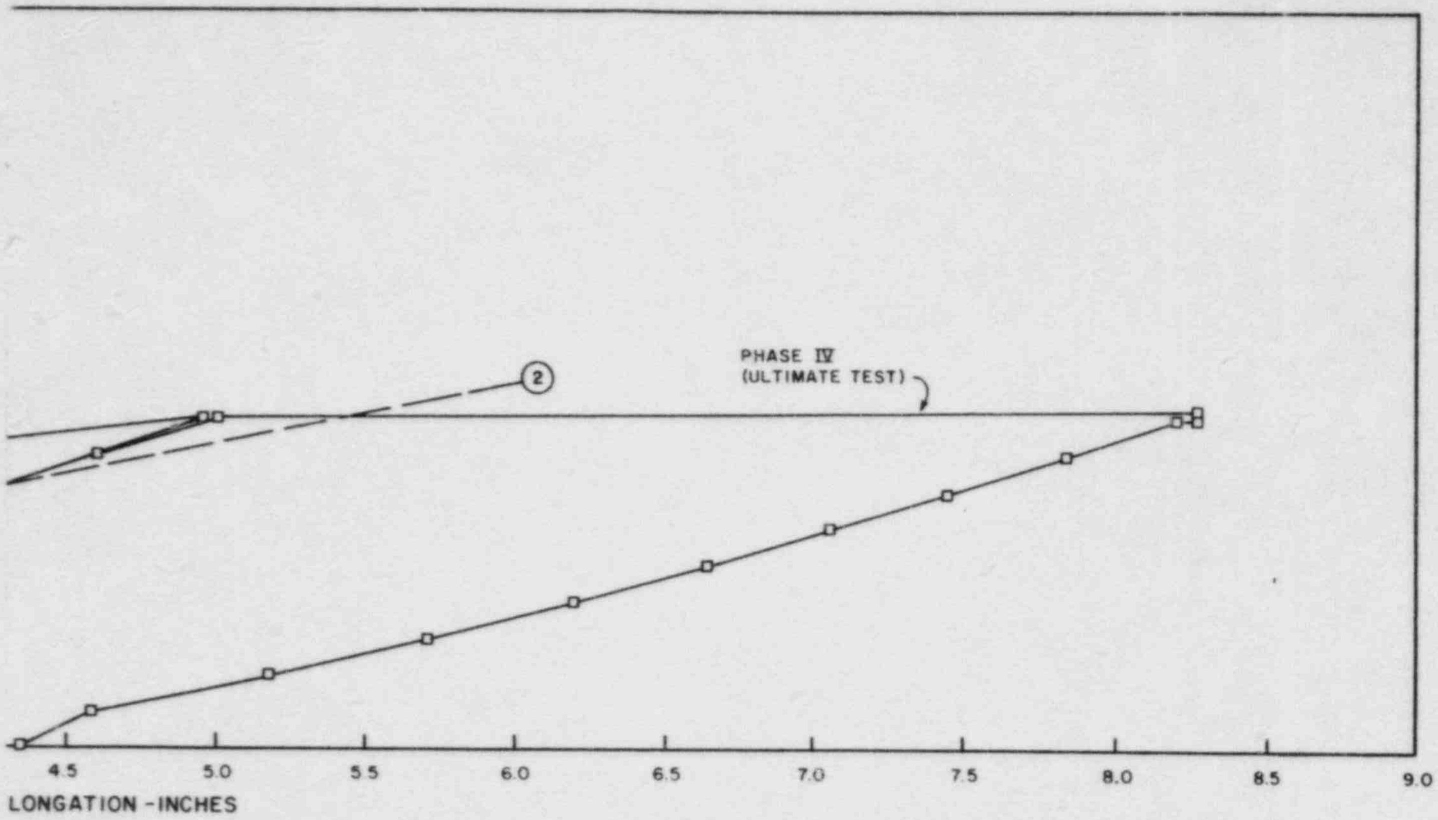
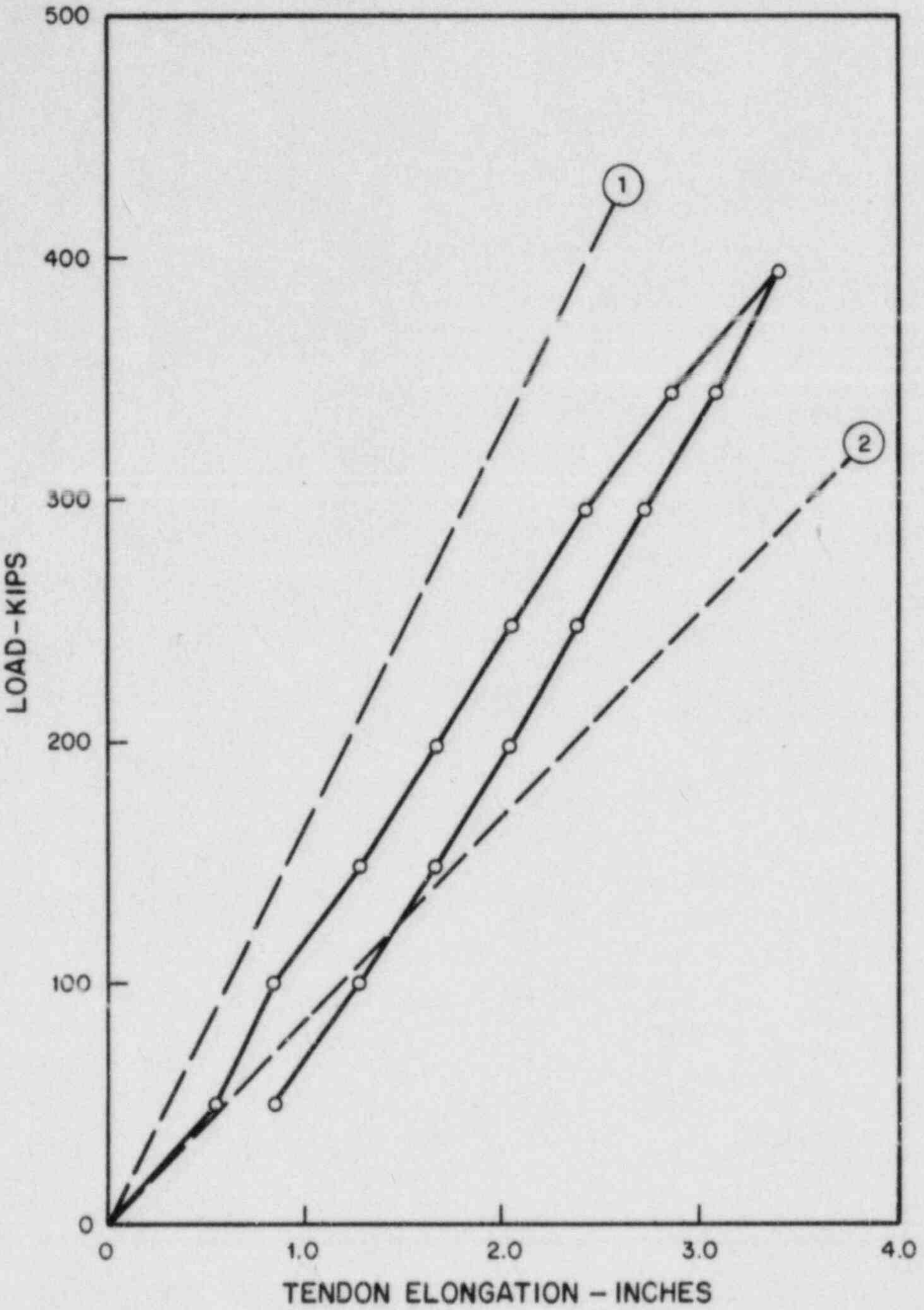
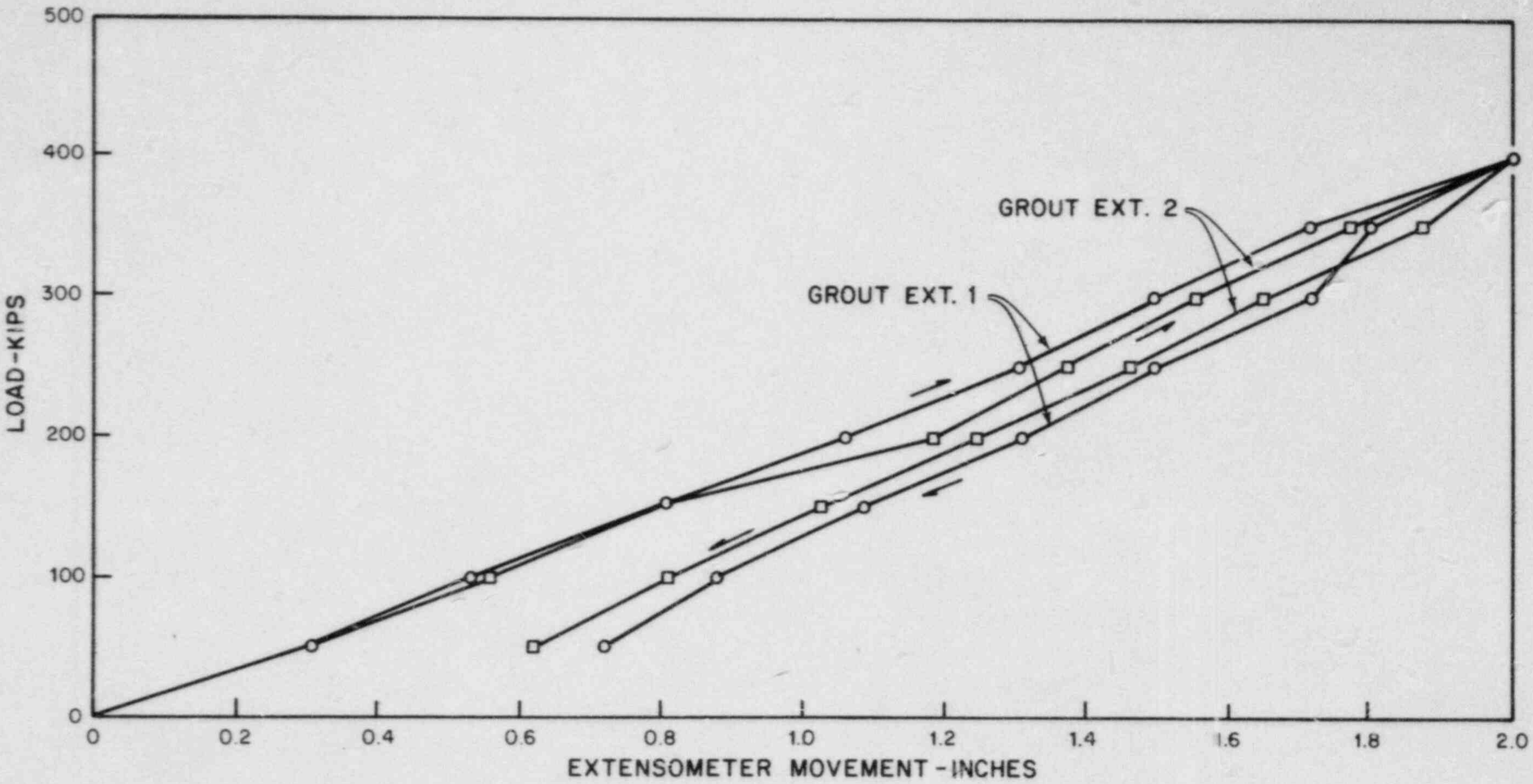


FIGURE 3-6
TENDON ELONGATION
PHASES I-IV STRESSING
TEST ANCHOR 2, 55 FT. MULTISTRAND
ROCK ANCHOR TEST PROGRAM TEST
NORTH ANNA UNITS 3 AND 4



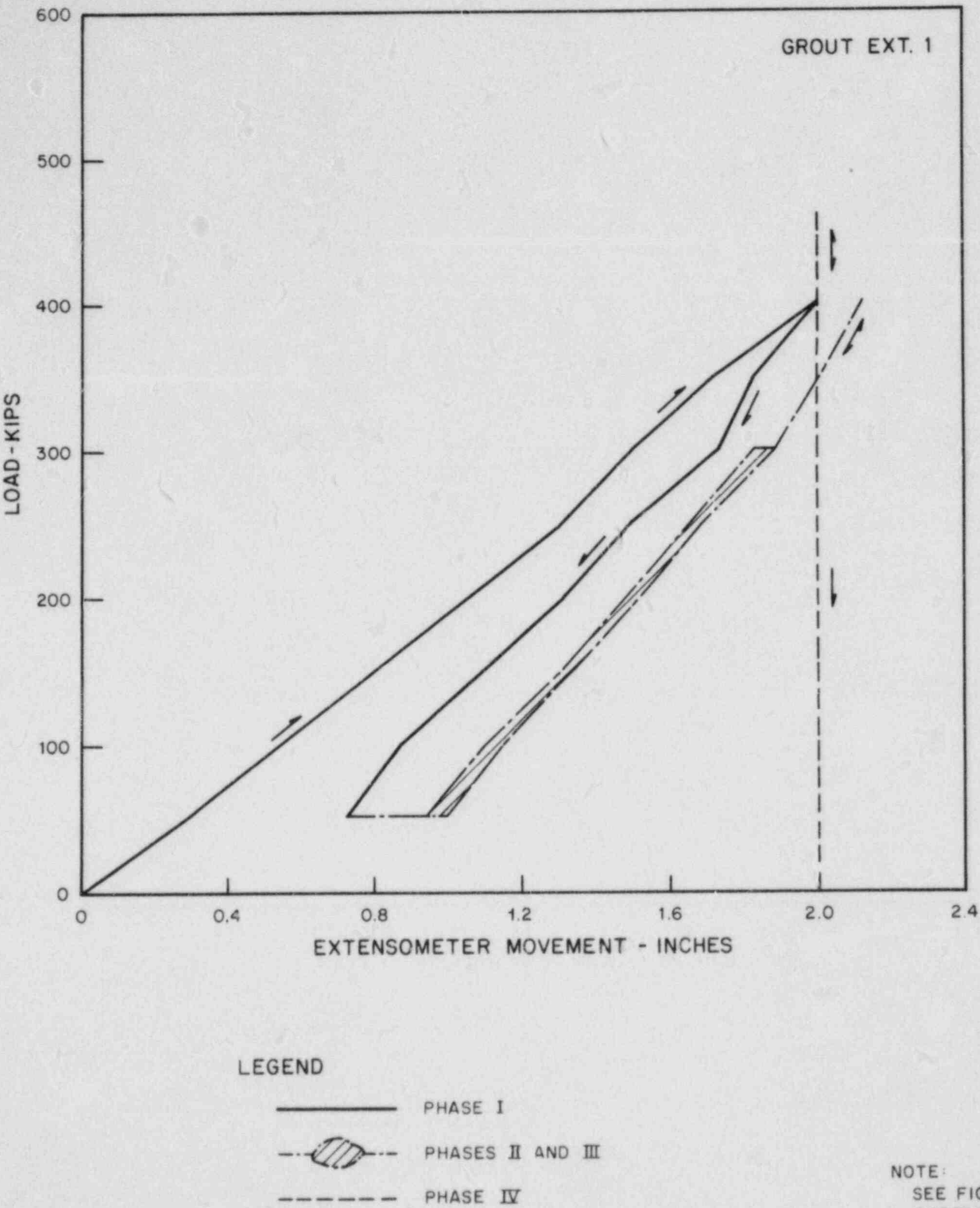
SEE NOTE ON FIGURE 3-1.

FIGURE 3-7
TENDON ELONGATION
PHASE I STRESSING
TEST ANCHOR 2, 55 FT. MULTISTRAND
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4

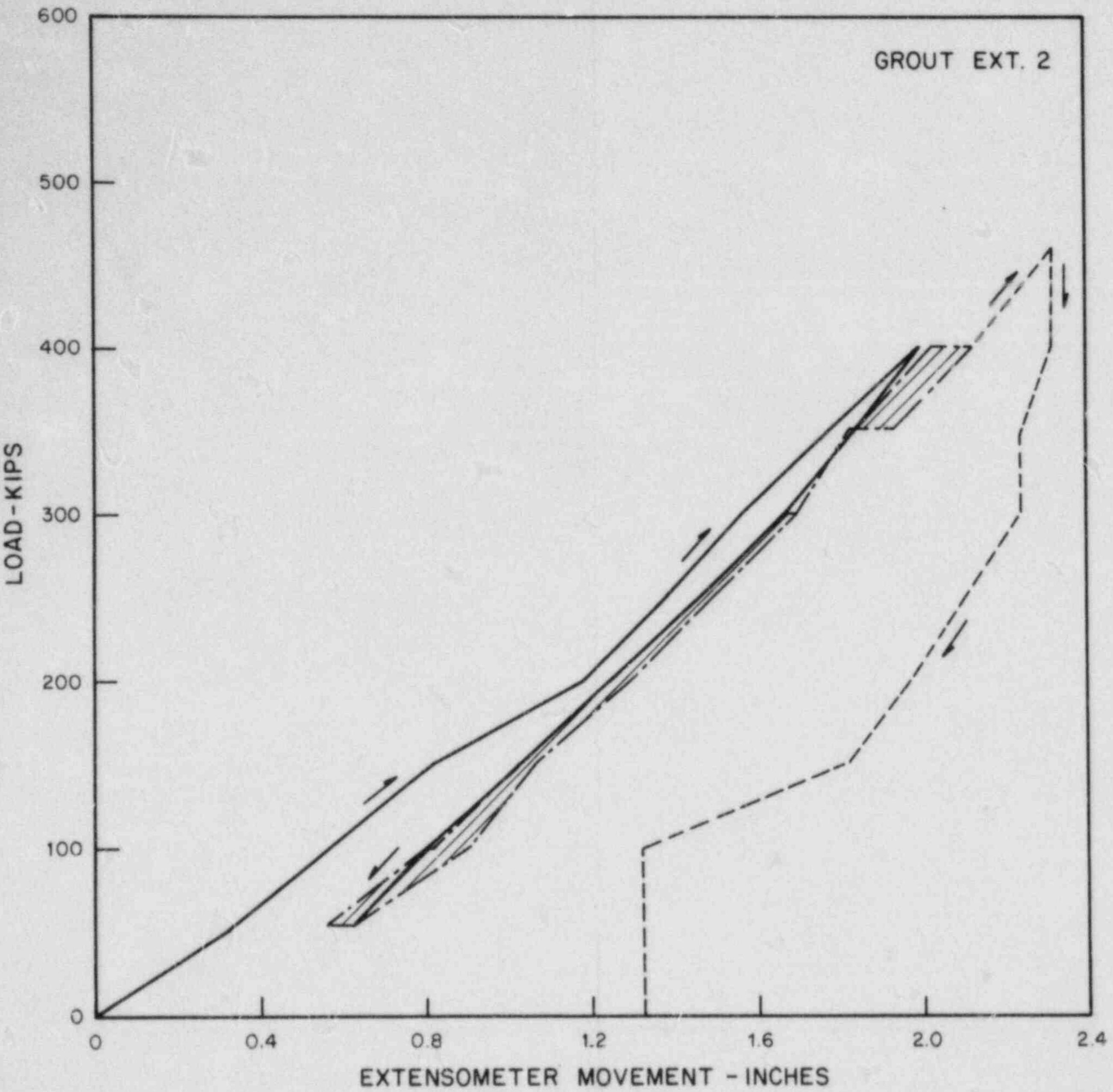


NOTE:
SEE FIGURE 2-3 FOR
EXTENSOMETER LOCATIONS.

FIGURE 3-8
EXTENSOMETER MOVEMENT
PHASE I STRESSING
TEST ANCHOR 2, 55 FT. MULTISTRAND
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4

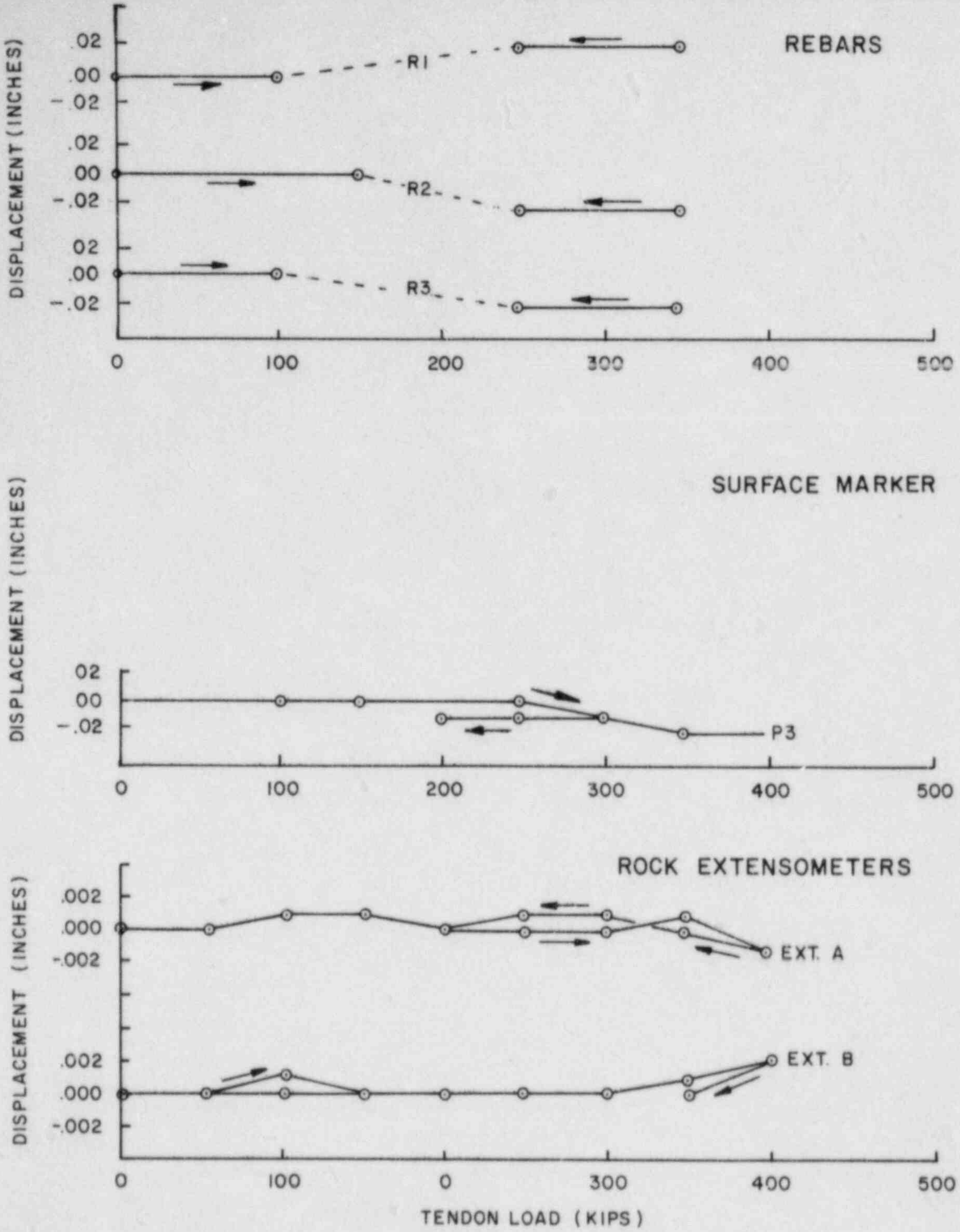


NOTE:
SEE FIGURE
EXTENSOM



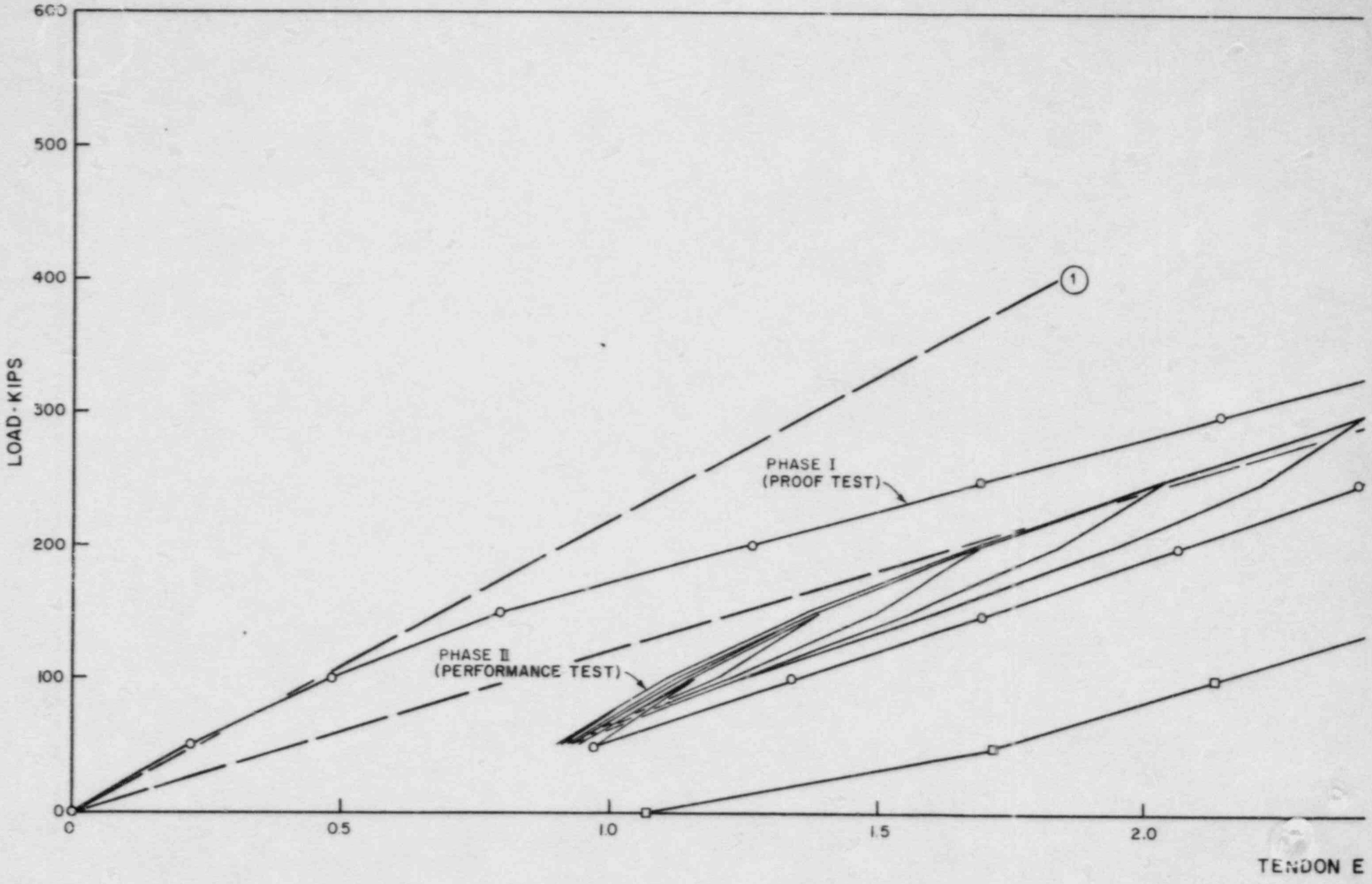
2-3 FOR
TER LOCATIONS.

FIGURE 3-9
EXTENSOMETER MOVEMENT
PHASES I-IV STRESSING
TEST ANCHOR 2, 55 FT. MULTISTRAND
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



NOTE:
SEE FIGURES 1-5 AND 2-3 FOR LOCATION OF INSTRUMENTATION.

FIGURE 3-10
ROCK MOVEMENTS
PHASE I STRESSING
TEST ANCHOR 2, 55 FT. MULTISTRAND
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4


600
500
400
300
200
100
0
LOAD-KIPS
0
0.5
1.0
1.5
2.0
TENDON E
①
PHASE I
(PROOF TEST)
PHASE II
(PERFORMANCE TEST)

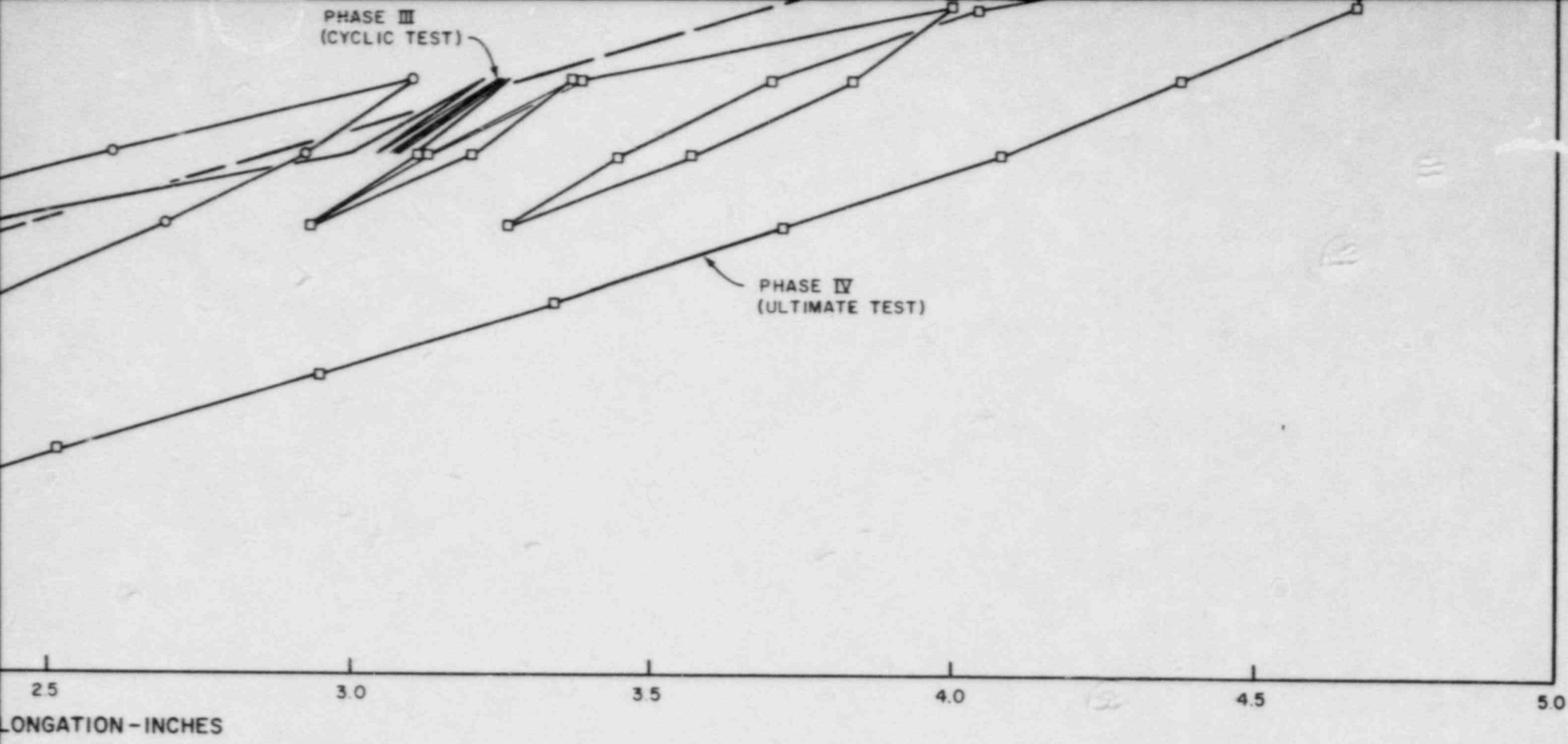
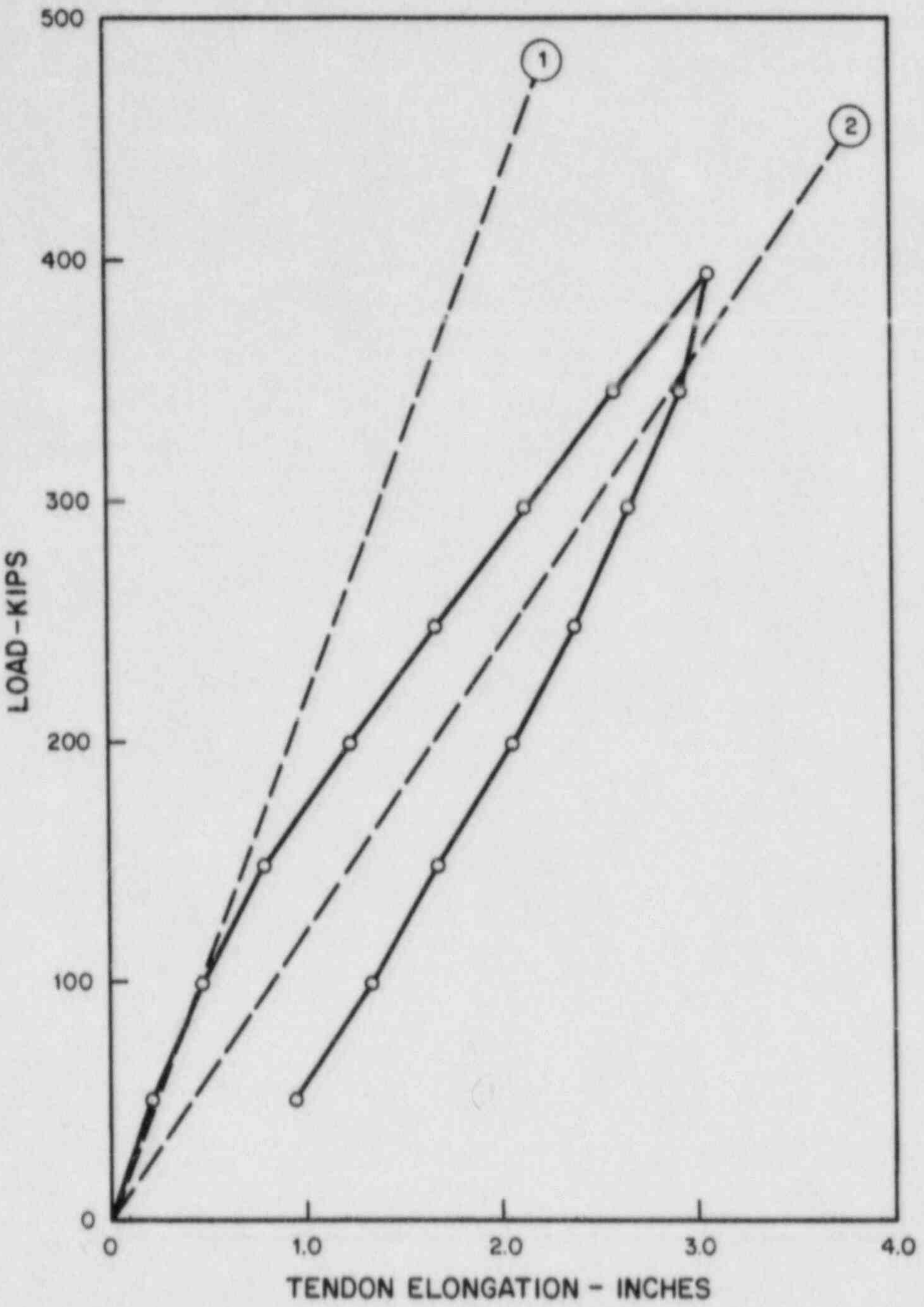
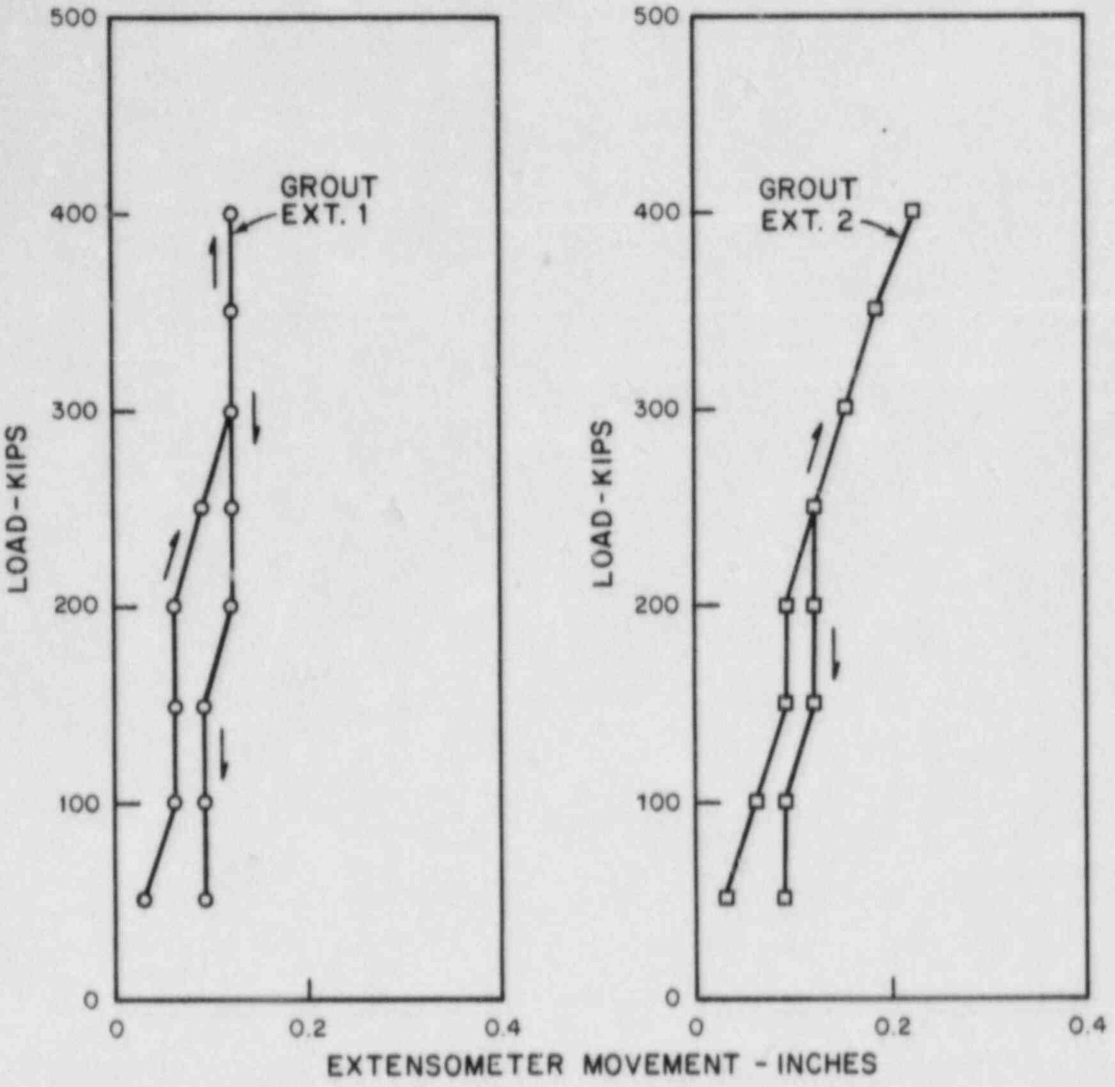


FIGURE 3-11
TENDON ELONGATION
PHASES I - IV - STRESSING
TEST ANCHOR 3, 45 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



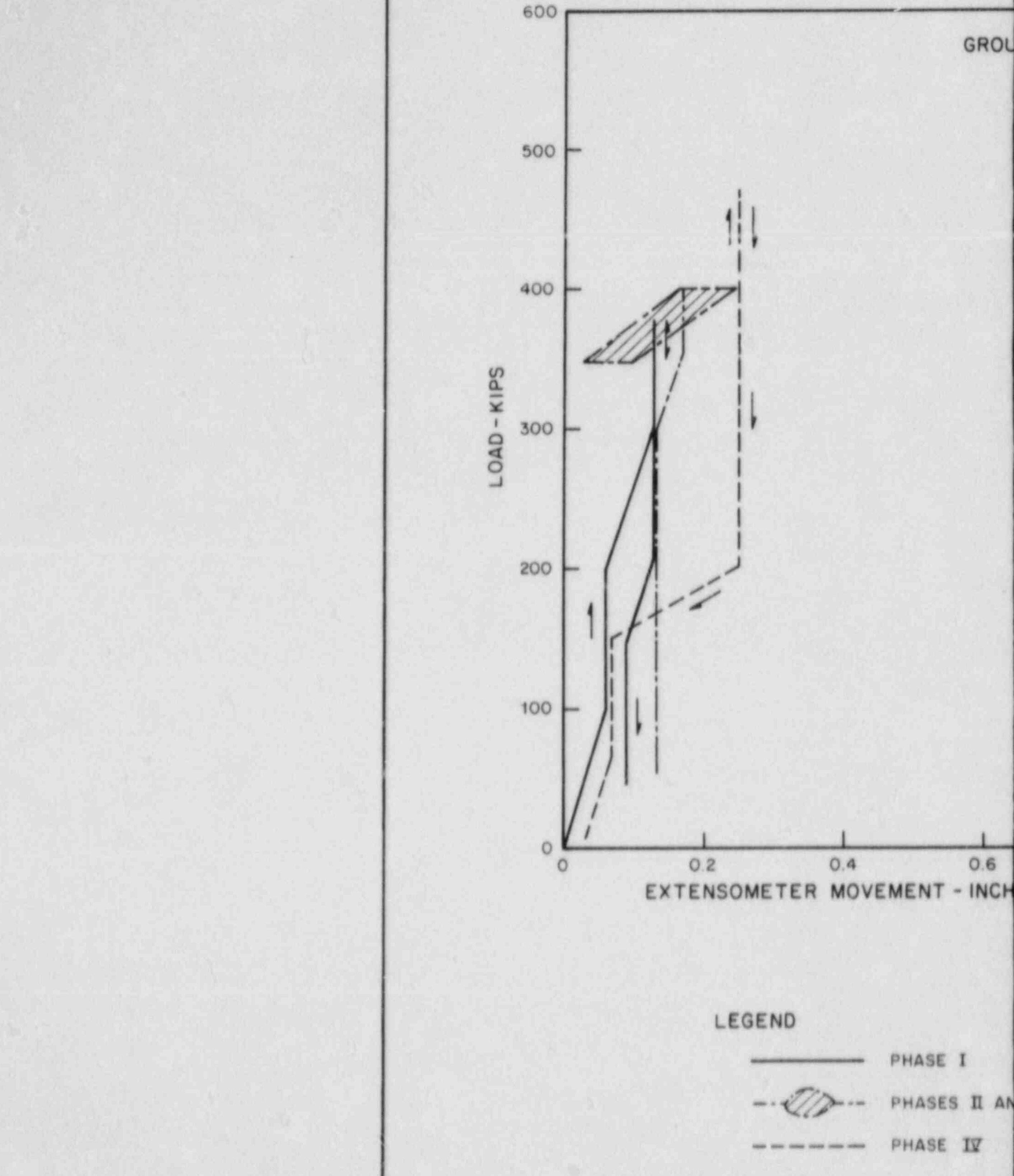
NOTE:
SEE NOTE ON FIGURE 3-1

FIGURE 3-12
TENDON ELONGATION
PHASE I STRESSING
TEST ANCHOR 3, 45 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4

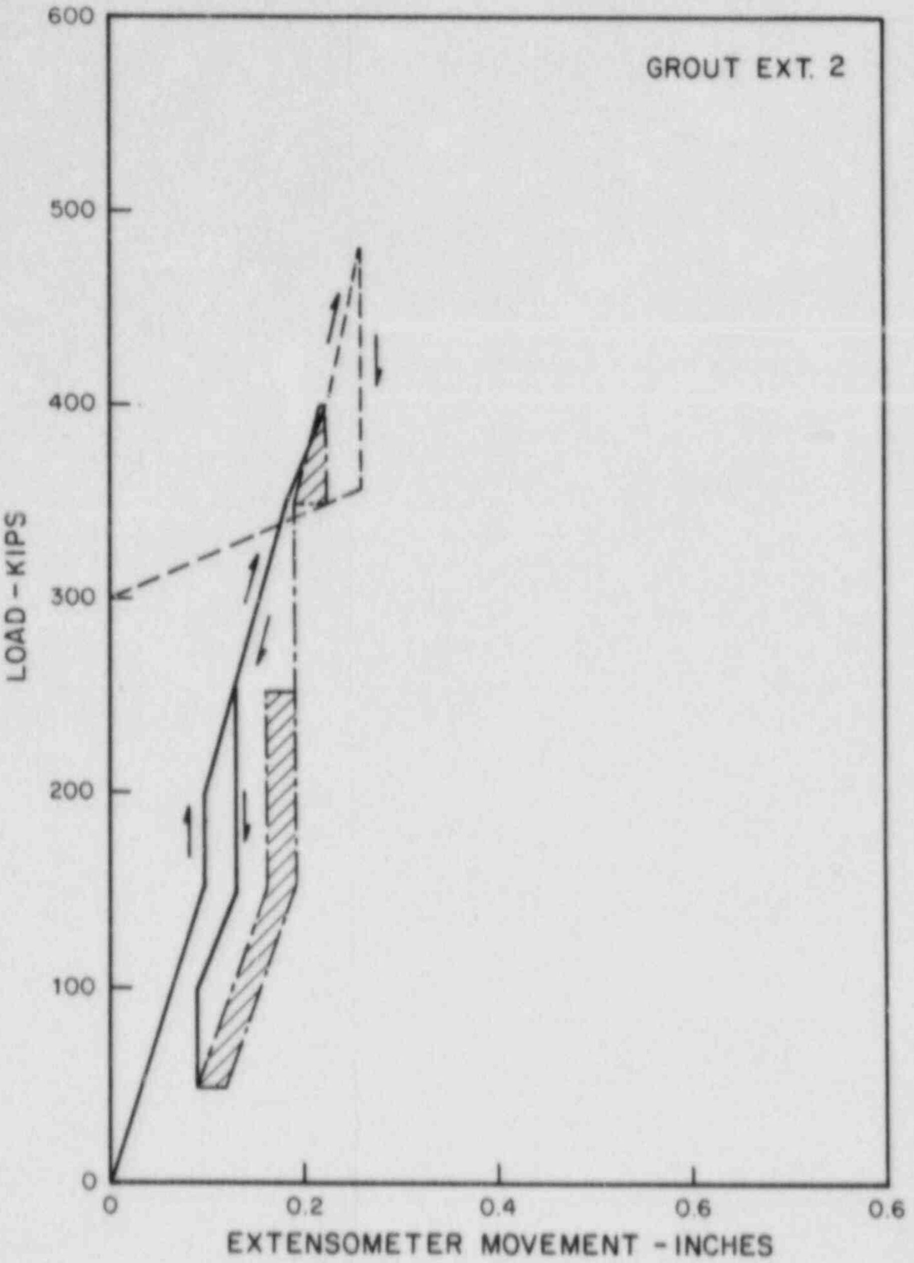


NOTE:
SEE FIGURE 2-3 FOR
EXTENSOMETER LOCATIONS

FIGURE 3-13
EXTENSOMETER MOVEMENT
PHASE I STRESSING
TEST ANCHOR 3, 45 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



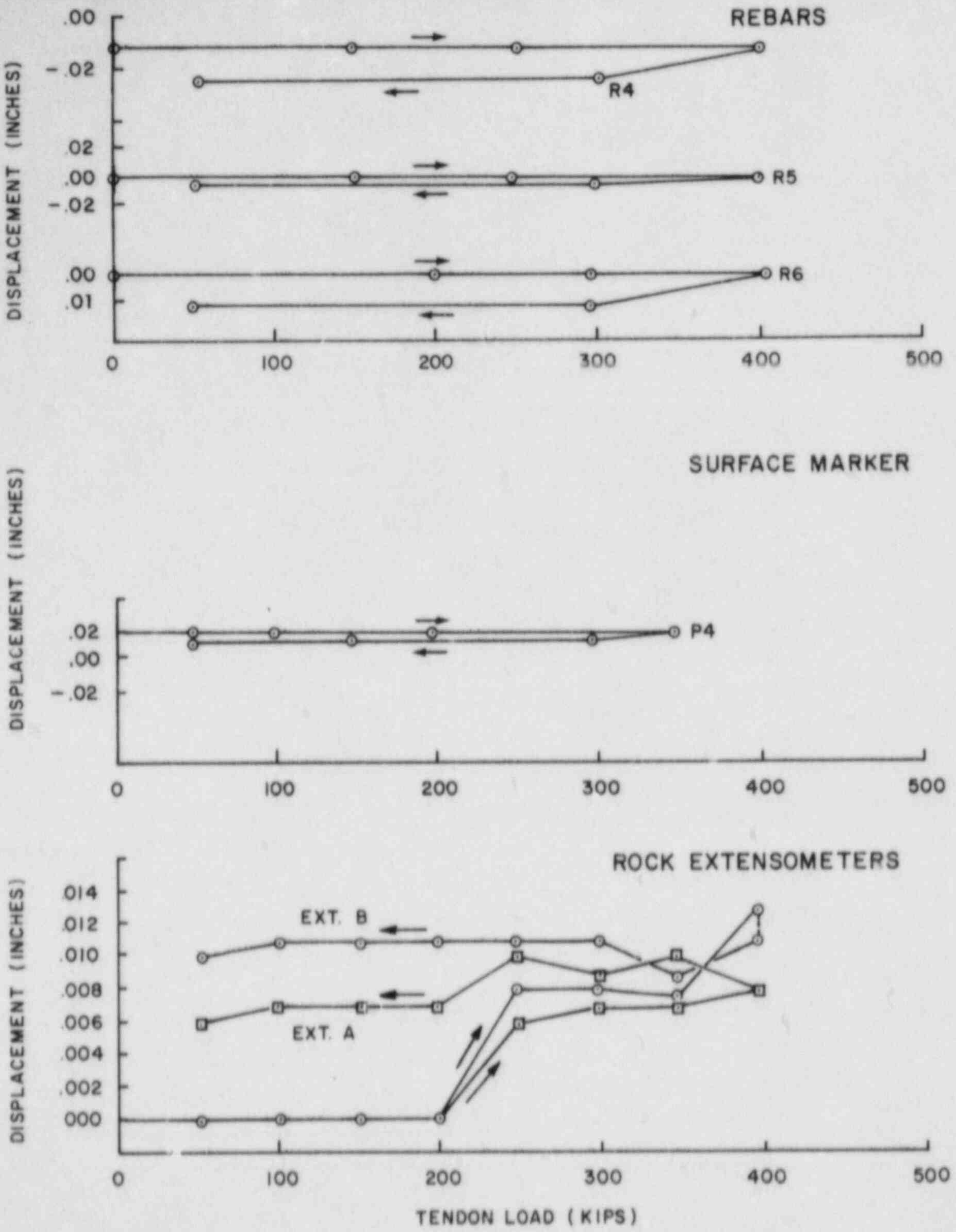
GROU
600
500
400
300
200
100
0
LOAD - KIPS
0
0.2
0.4
0.6
EXTENSOMETER MOVEMENT - INCH
LEGEND
PHASE I
PHASES II AN
PHASE IV



D III

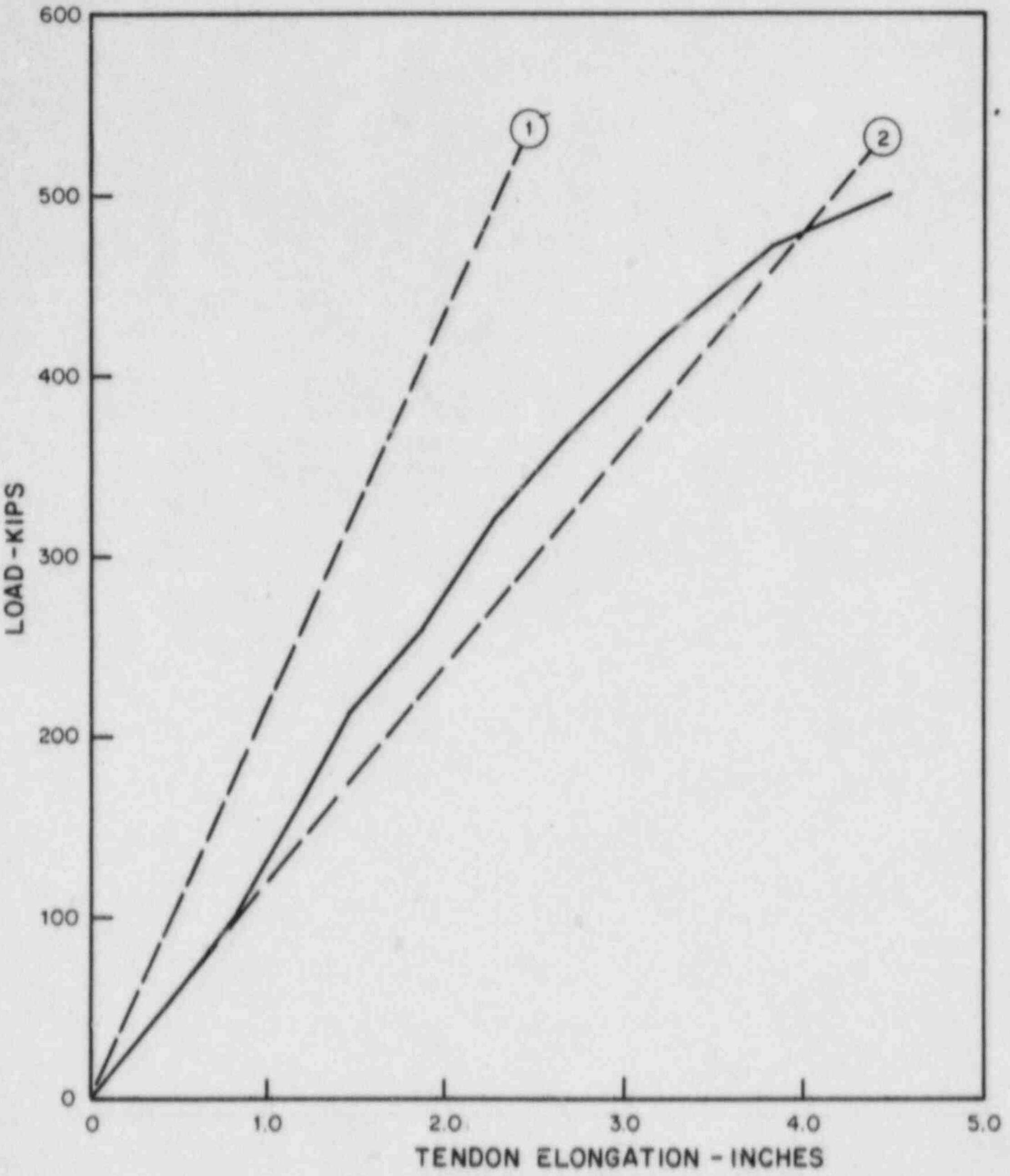
NOTE:
SEE FIGURE 2-3 FOR
EXTENSOMETER LOCATIONS

FIGURE 3-14
EXTENSOMETER MOVEMENT
PHASES I - IV STRESSING
TEST ANCHOR 3, 45 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



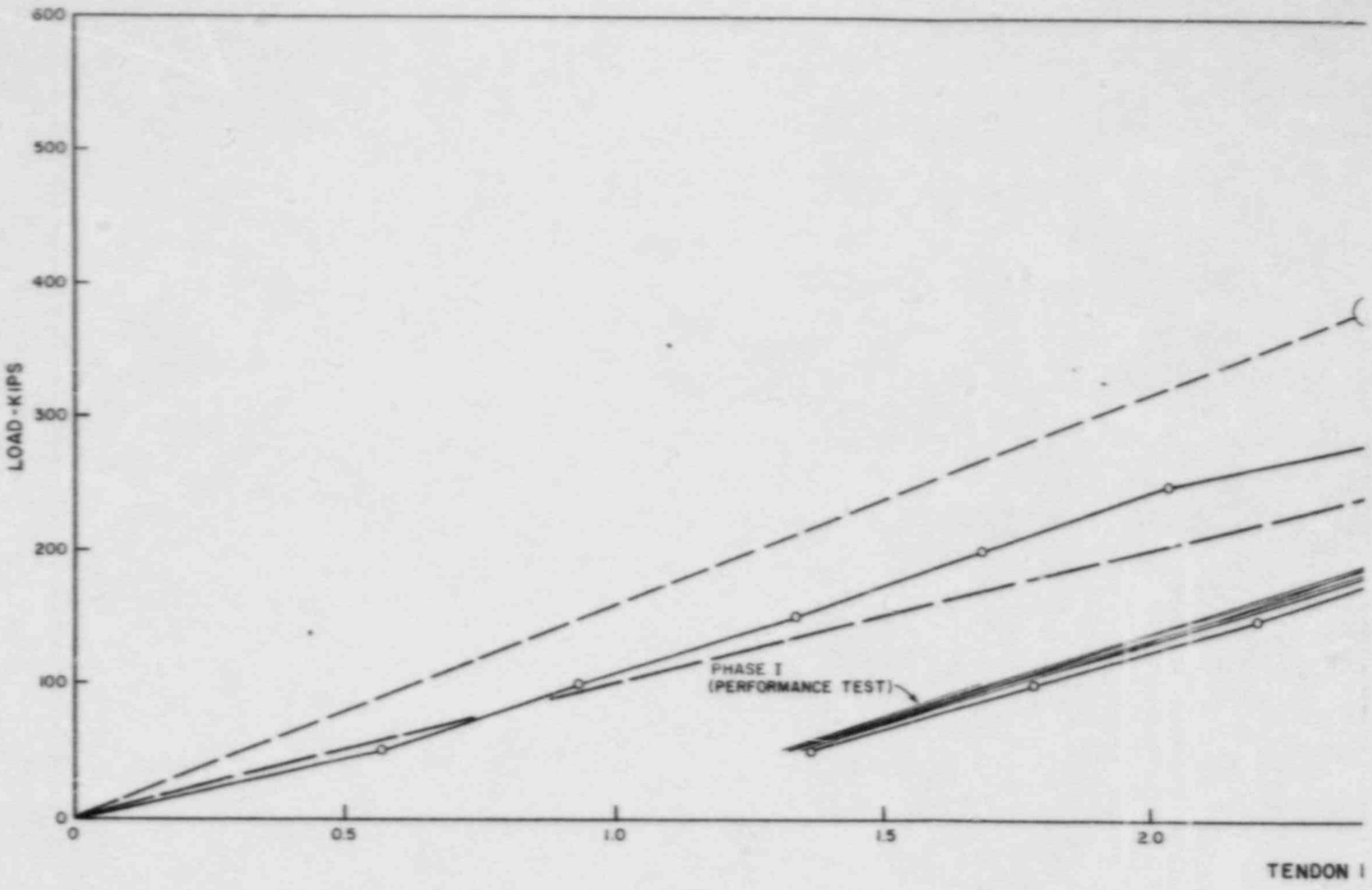
NOTE:
SEE FIGURES 1-5 AND 2-3 FOR LOCATION OF INSTRUMENTATION

FIGURE 3-15
ROCK MOVEMENT
PHASE I STRESSING
TEST ANCHOR 3, 45 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



NOTE:
SEE NOTE ON FIGURE 3-1

FIGURE 3-16
TENDON ELONGATION
PHASE Ⅴ STRESSING
TEST ANCHOR 3, 45 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM RERORT
NORTH ANNA UNITS 3 AND 4



SEE NOTE ON FIGURE 3-1.

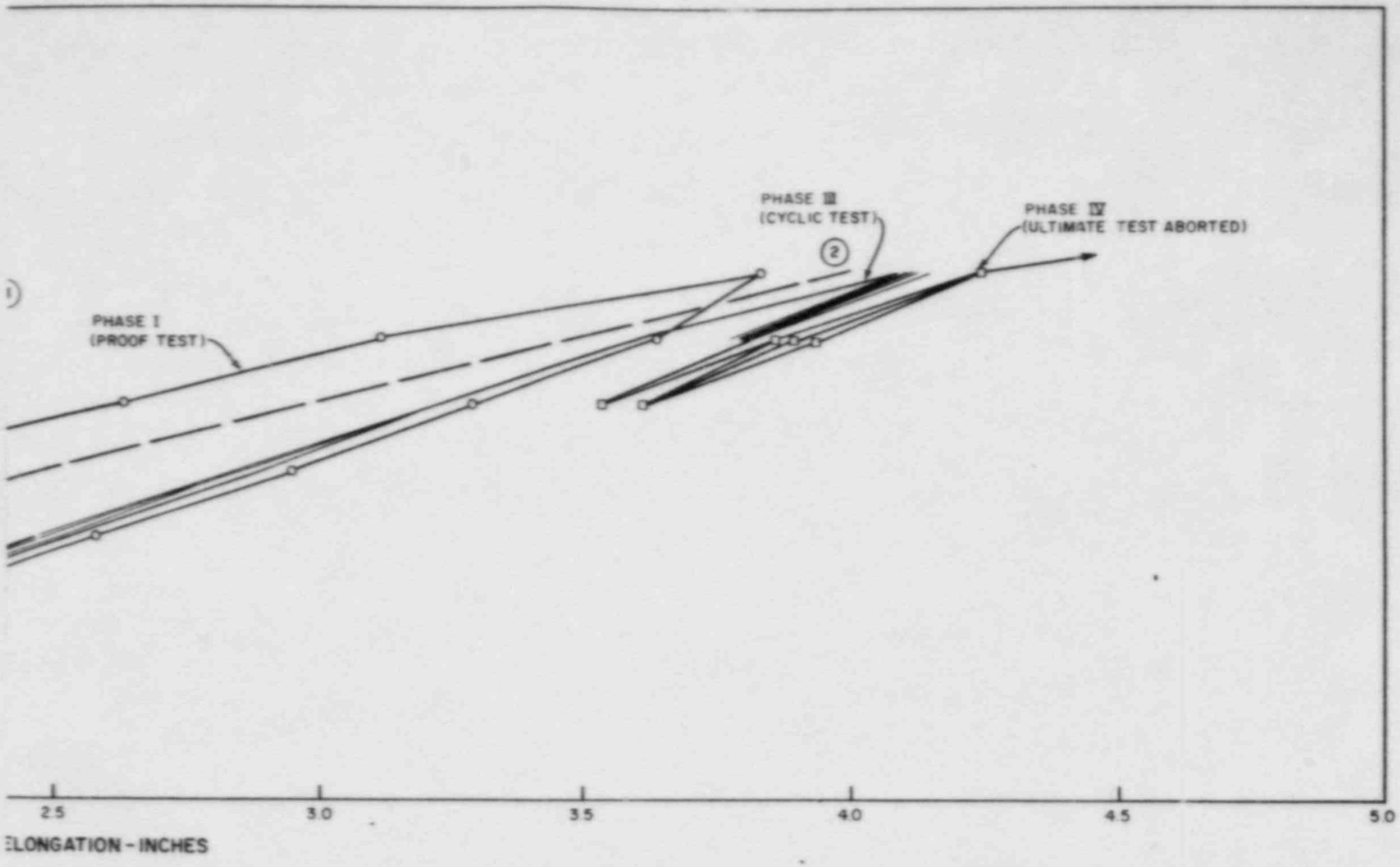
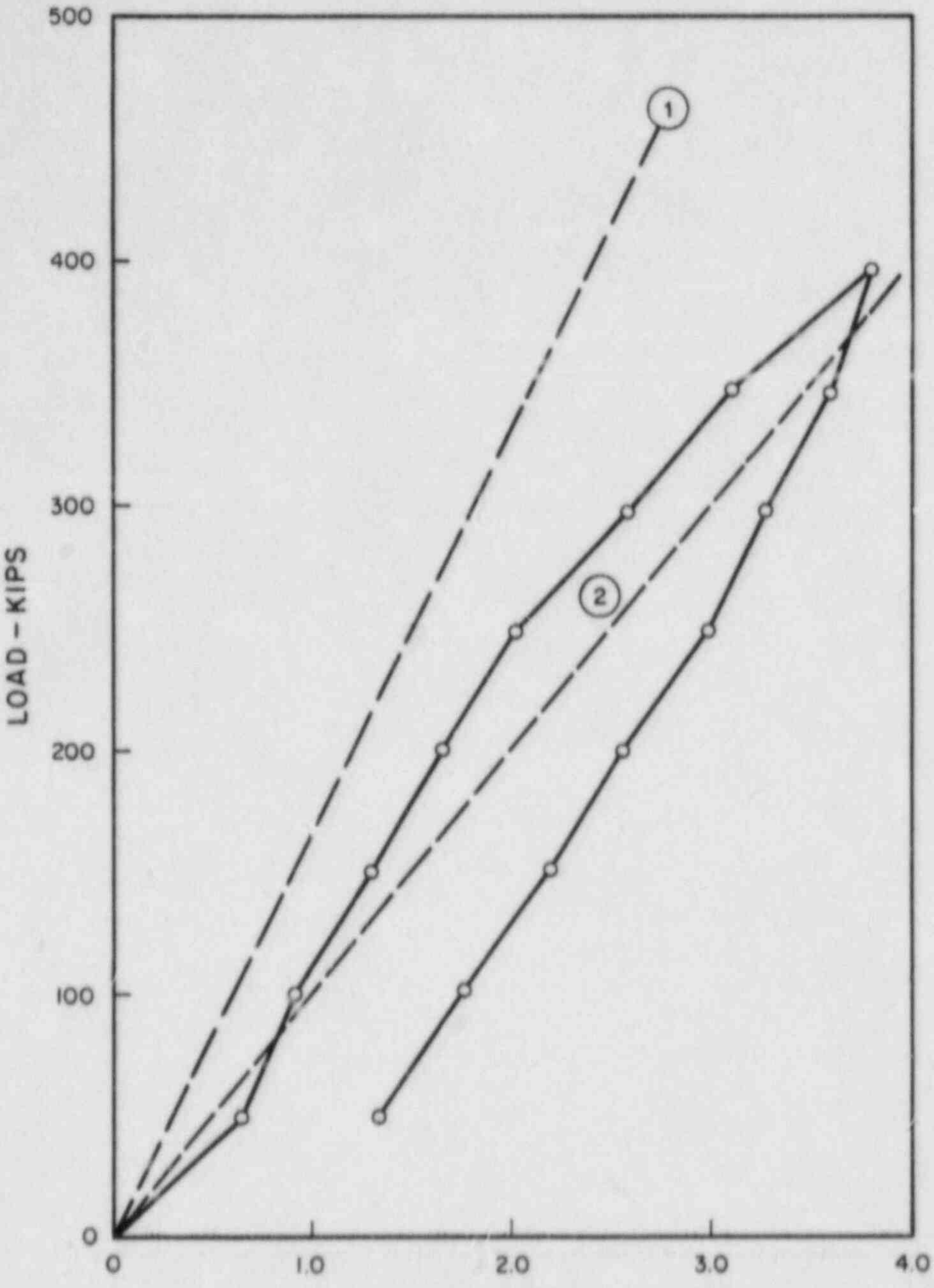
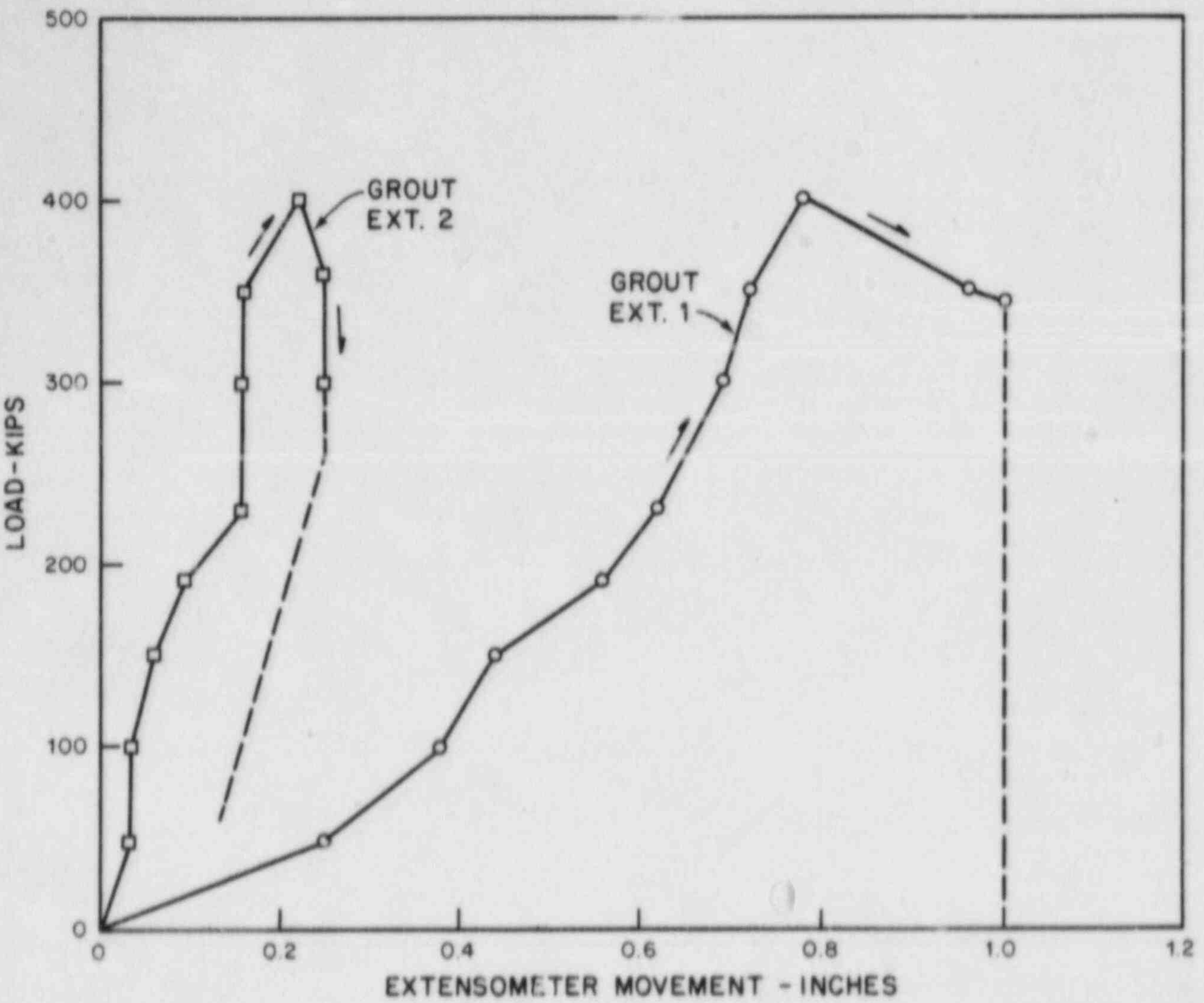


FIGURE 3-17
TENDON ELONGATION
PHASES I - IV - STRESSING
TEST ANCHOR 4, 45 FT. MULTISTRAND
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



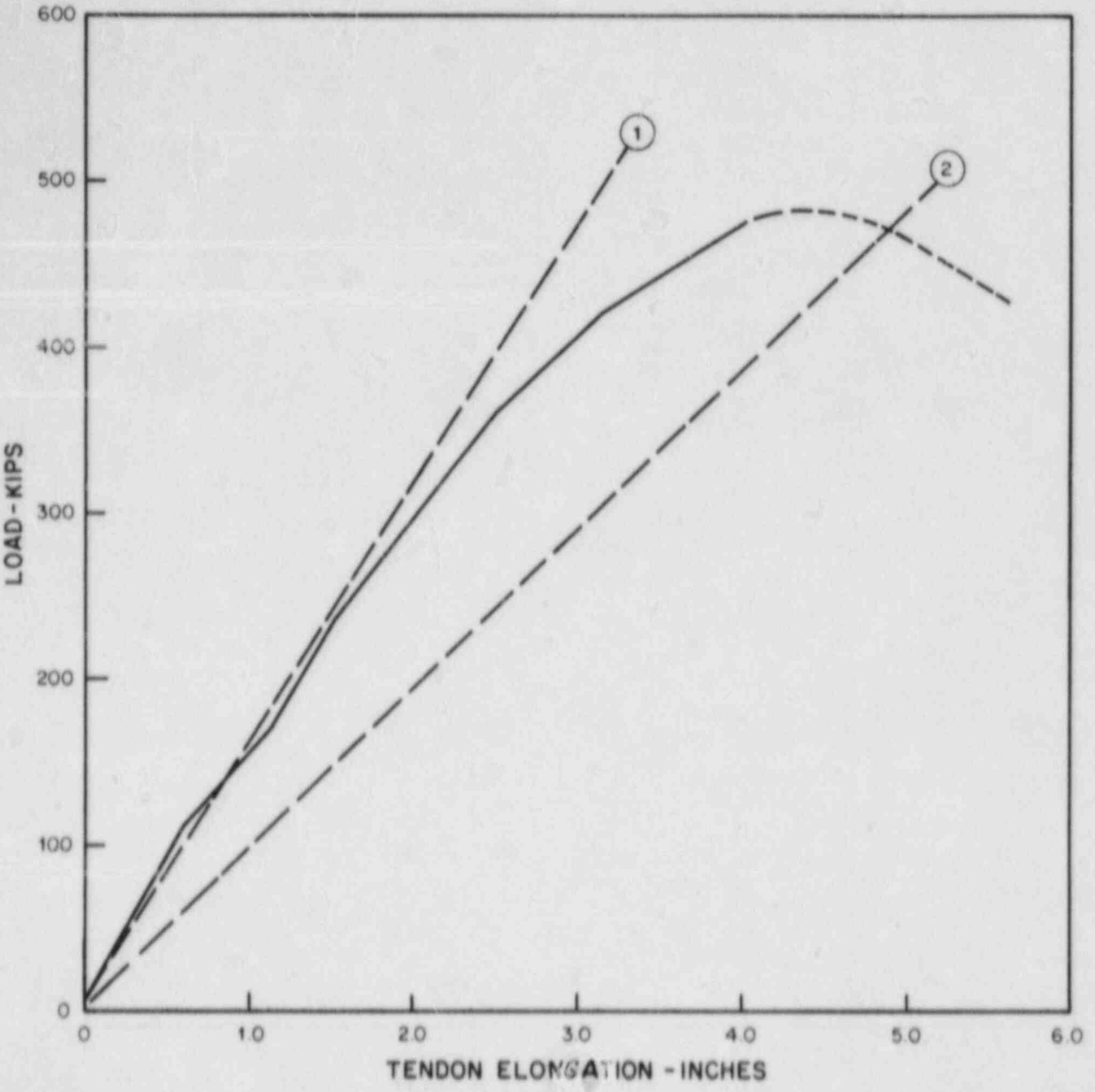
SEE NOTE ON FIGURE 3-1.

FIGURE 3-18
TENDON ELONGATION
PHASE I STRESSING
TEST ANCHOR 4, 45 FT. MULTISTRAND
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



NOTE:
SEE FIGURE 2-3 FOR
EXTENSOMETER LOCATIONS.

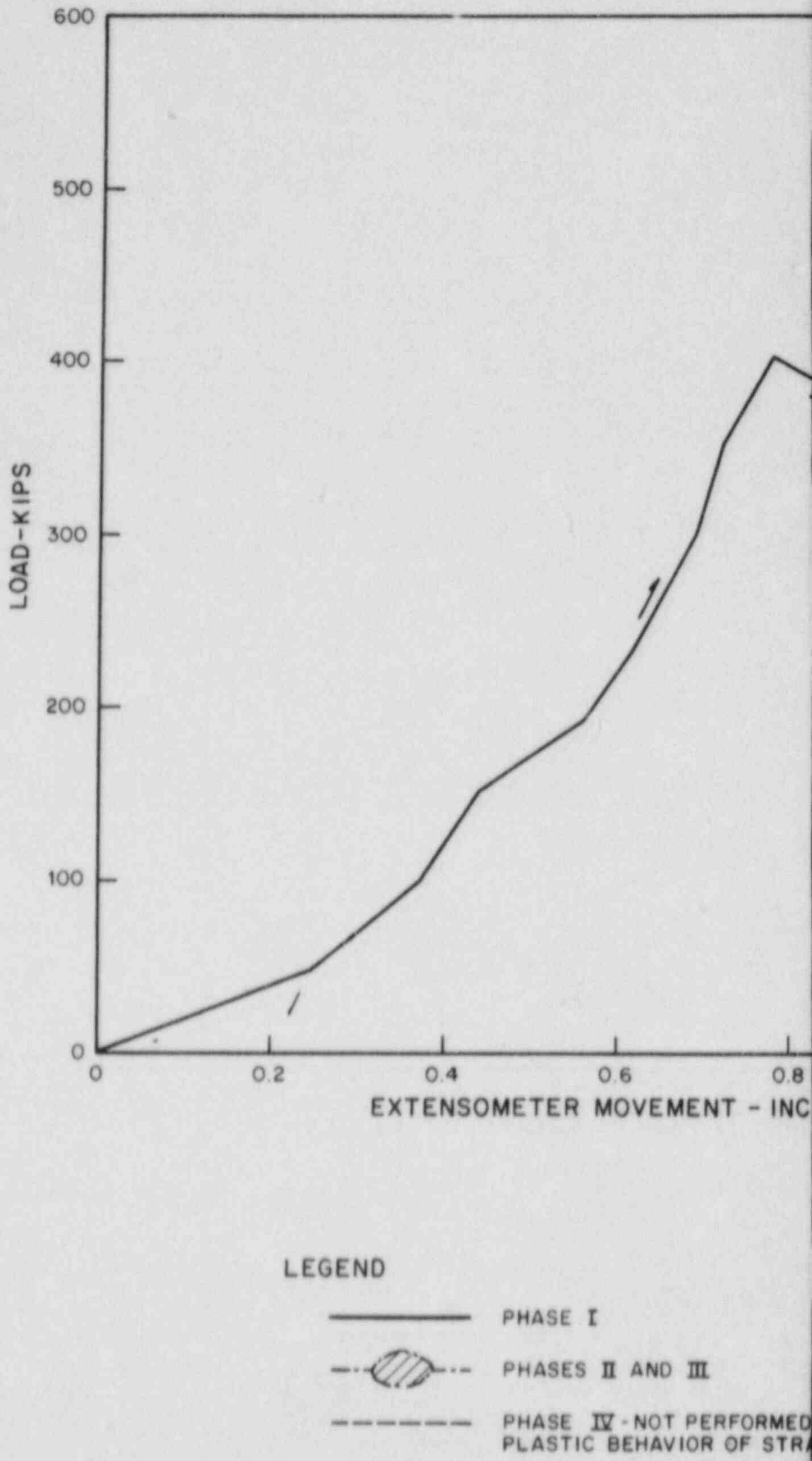
FIGURE 3-19
EXTENSOMETER MOVEMENT
PHASE I STRESSING
TEST ANCHOR 4, 45 FT. MULTISTRAND
ROCK ANCHOR TEST. PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



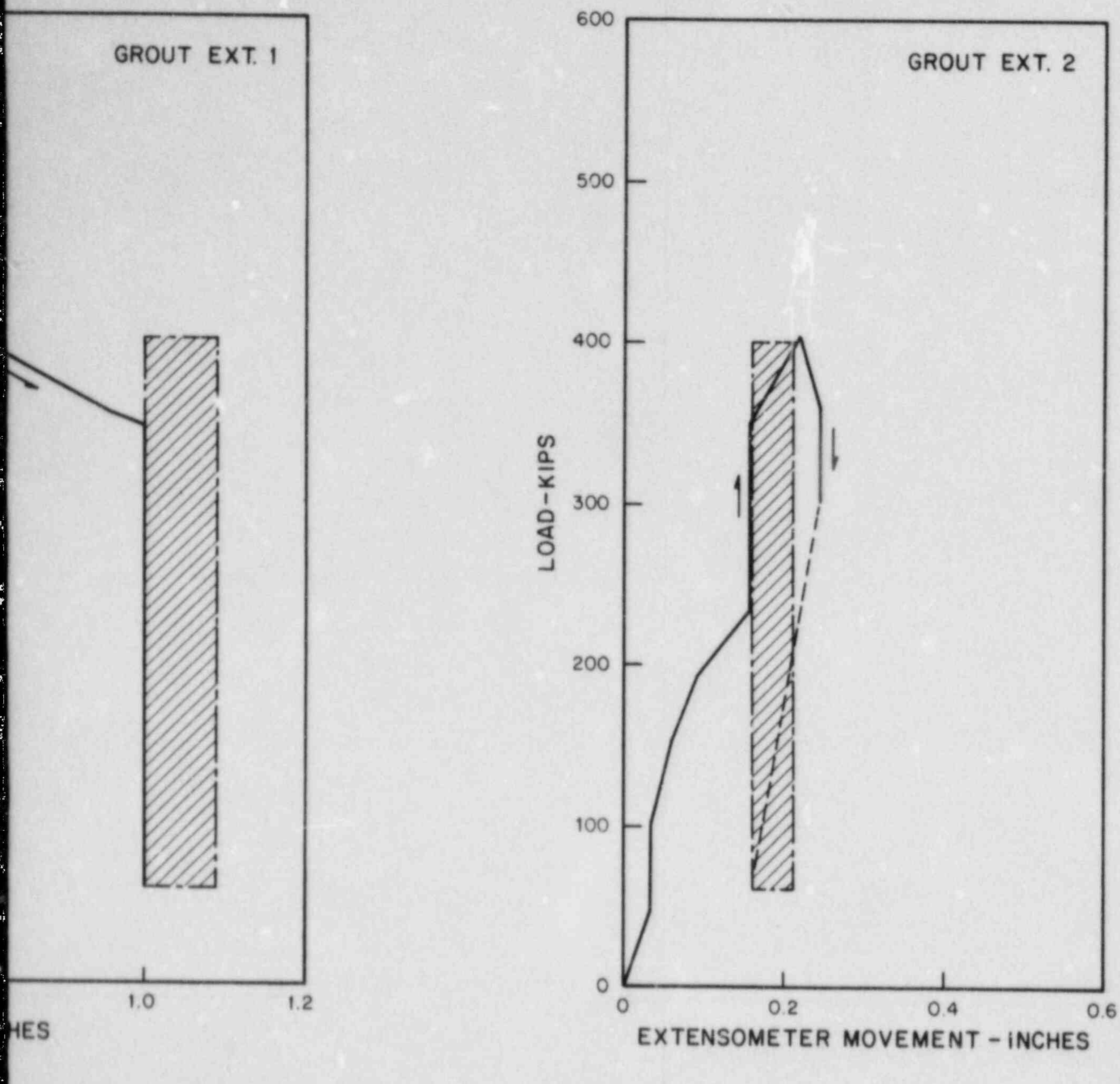
NOTE:
STRANDS STARTED TO BREAK
AT A LOAD OF 470 KIPS.

SEE NOTE ON FIGURE 3-1

FIGURE 3-20
TENDON ELONGATION
PHASE V STRESSING
TEST ANCHOR 4, 45 FT. MULTISTRAND
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



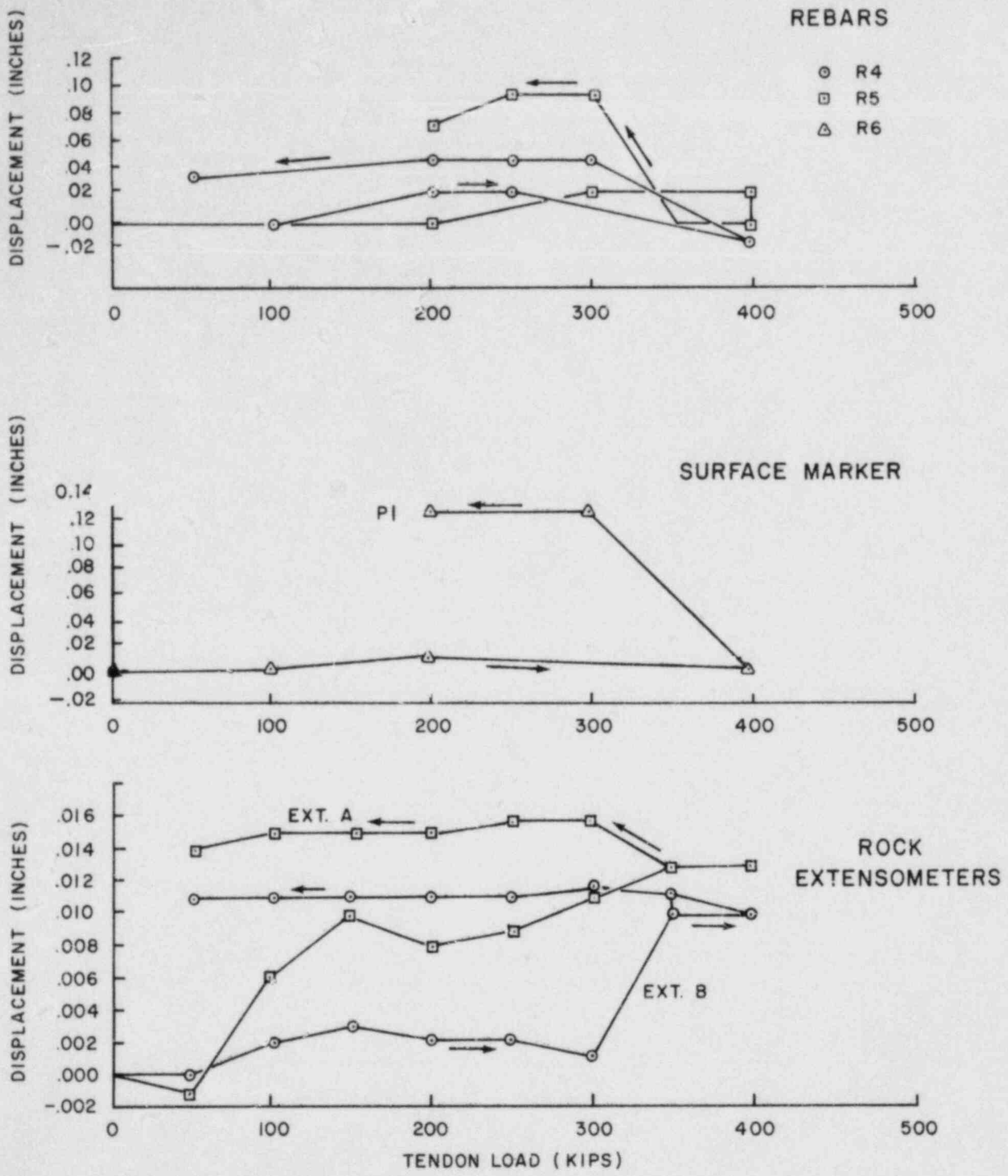
600
500
400
300
200
100
0
LOAD-KIPS
0
0.2
0.4
0.6
0.8
EXTENSOMETER MOVEMENT - INC
LEGEND
PHASE I
PHASES II AND III
PHASE IV - NOT PERFORMED
PLASTIC BEHAVIOR OF STR



DUE TO
ANDS

NOTE:
SEE FIGURE 2-3 FOR
EXTENSOMETER LOCATIONS.

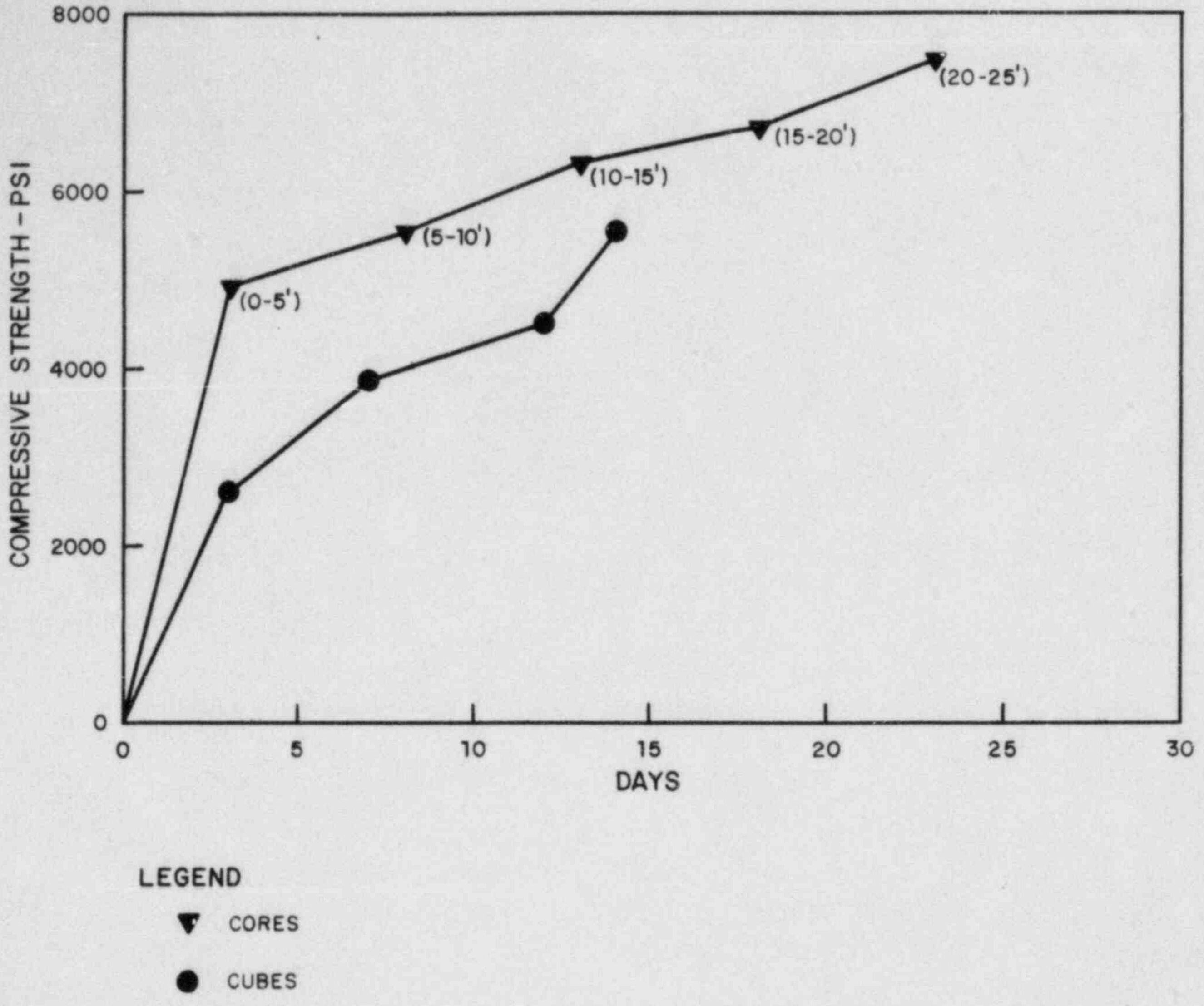
FIGURE 3-21
EXTENSOMETER MOVEMENT
PHASES I-IV STRESSING
TEST ANCHOR 4, 45 FT. MULTISTRAND
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



NOTE:

SEE FIGURES 1-5 AND 2-3 FOR LOCATION OF INSTRUMENTATION.

FIGURE 3-22
ROCK MOVEMENT
PHASE I STRESSING
TEST ANCHOR 4,45 FT. MULTISTRAND
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



NOTES:

1. NUMBERS IN PARENTHESIS REFER TO DEPTH INTERVAL FROM WHICH THE CORE WAS RECOVERED. TOP OF GROUT WAS 25 FT. BELOW GRADE.
2. GROUT TEST HOLE No. 1 FILLED WITH MB 814 GROUT.

FIGURE 3-23
GROUT STRENGTH
GROUT TEST HOLE No. 1
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4

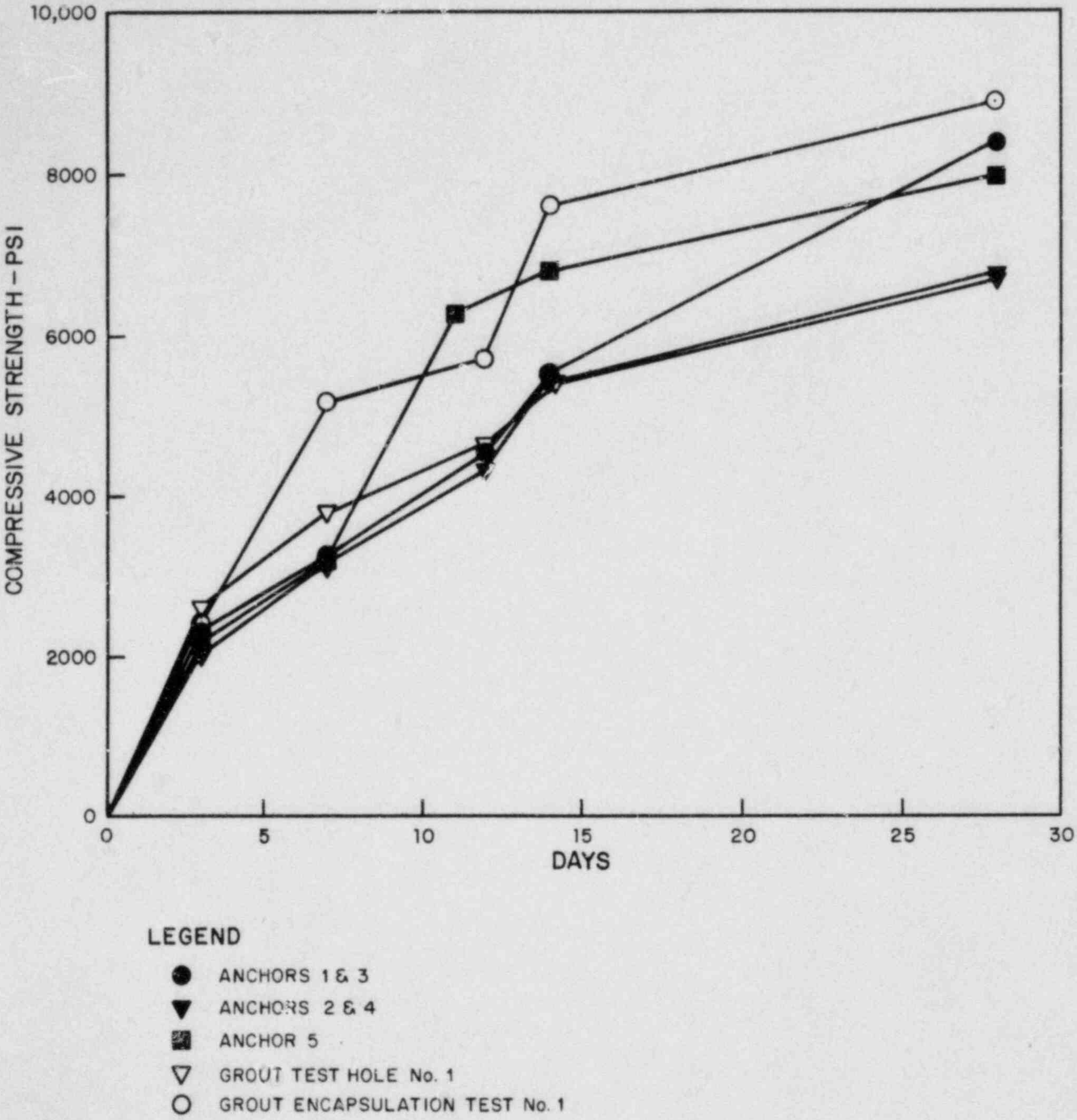
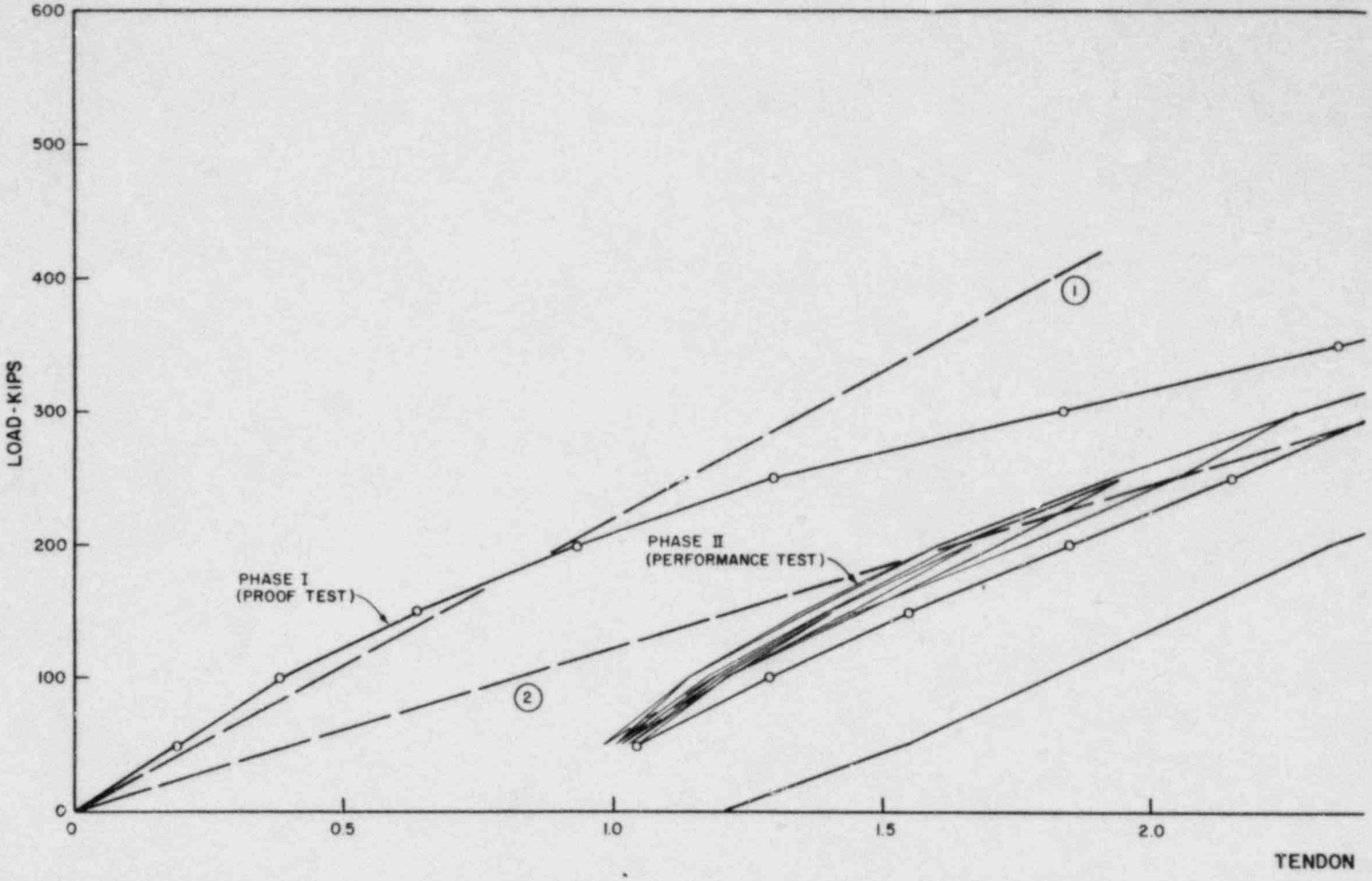


FIGURE 3 - 24
CUBE STRENGTHS - MB 814 GROUT
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



SEE NOTE ON FIGURE 3-1

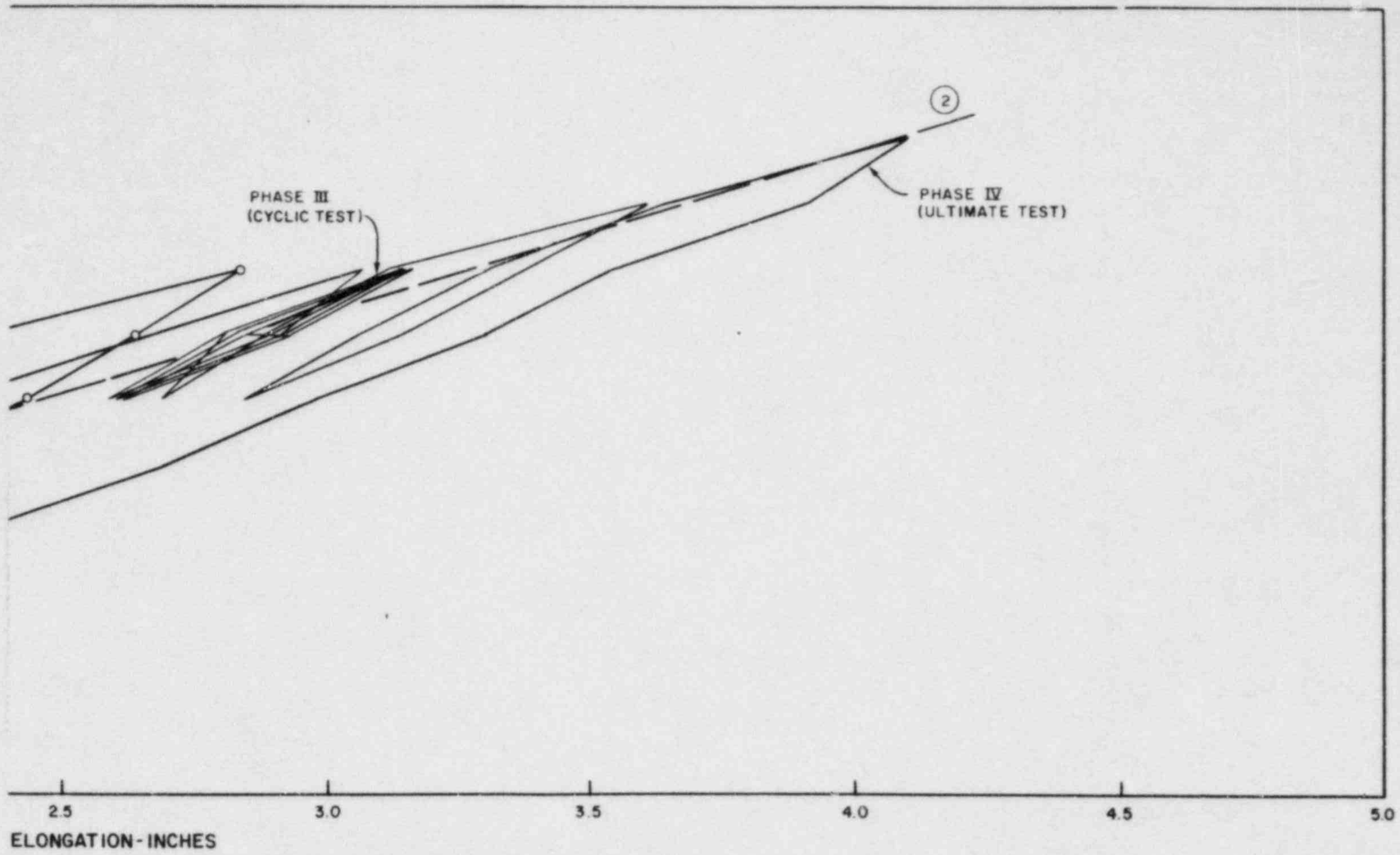
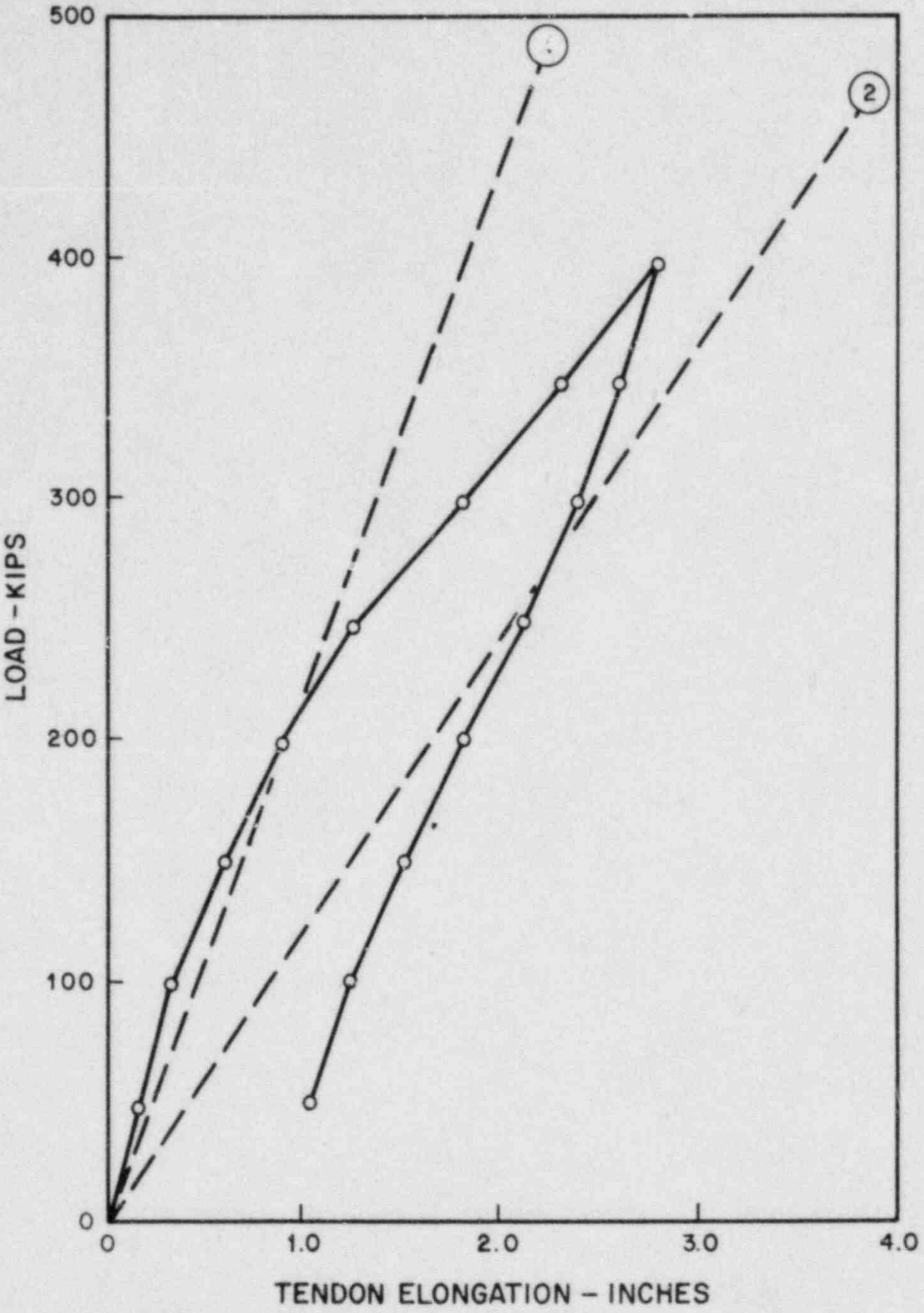
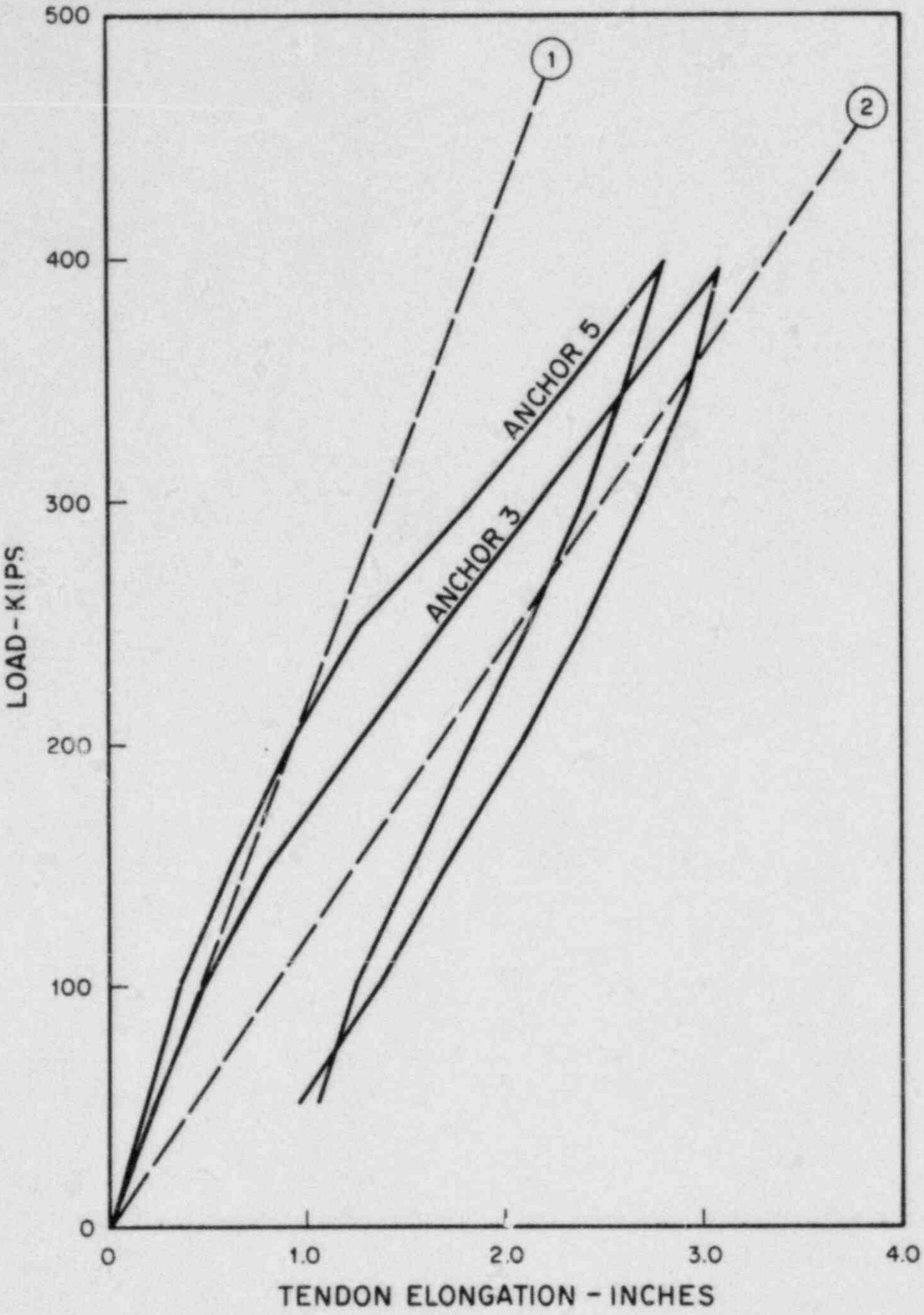


FIGURE 3-25
TENDON ELONGATION
PHASES I - IV - STRESSING
TEST ANCHOR 5, 45 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



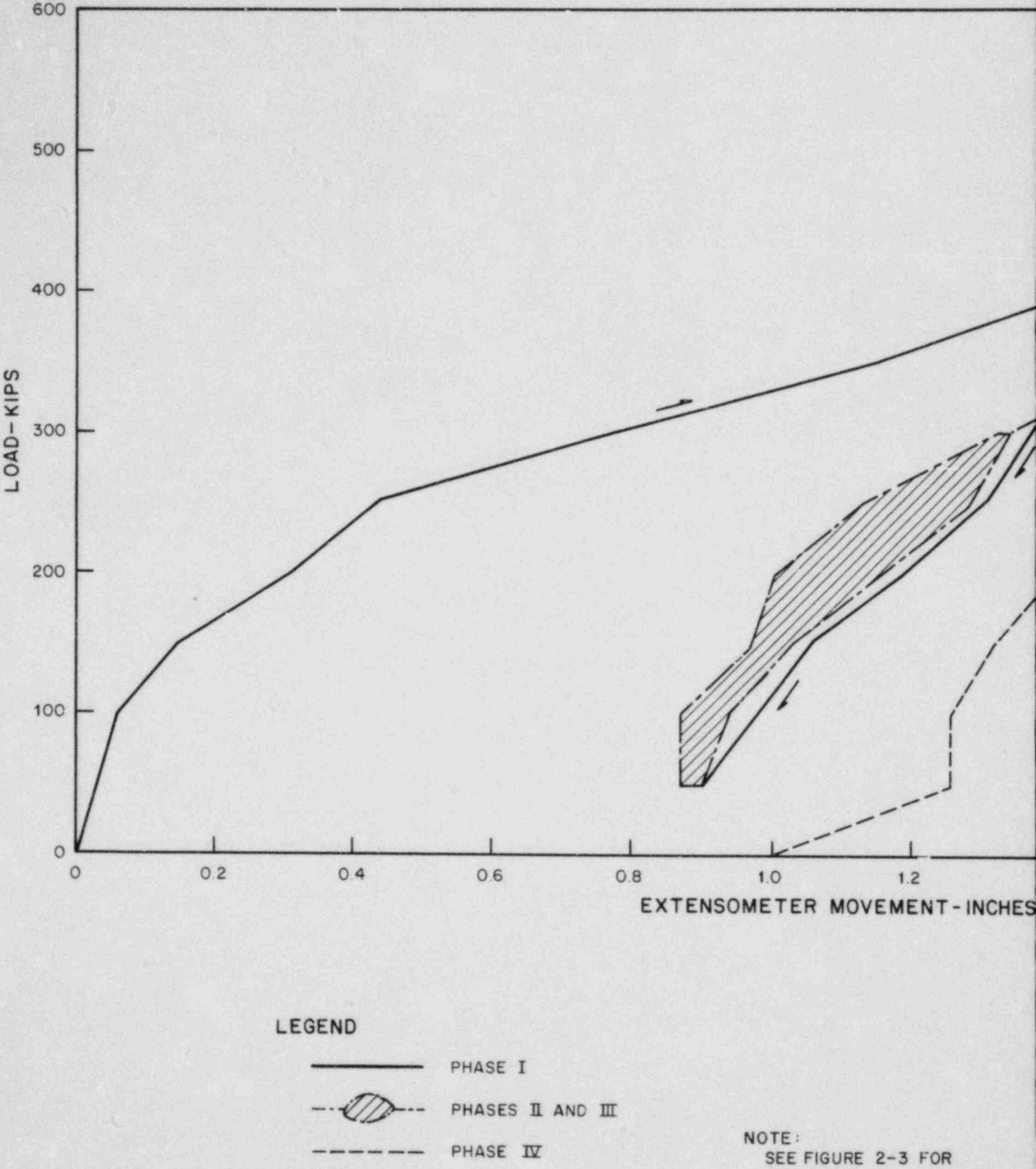
SEE NOTE ON FIGURE 3-1

FIGURE 3-26
TENDON ELONGATION
PHASE I STRESSING
TEST ANCHOR 5, 45 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



SEE NOTE ON FIGURE 3-1

FIGURE 3-27
TENDON ELONGATION
PHASE I STRESSING
TEST ANCHORS 3 & 5
45 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



NOTE:
SEE FIGURE 2-3 FOR
EXTENSOMETER LOCATIONS

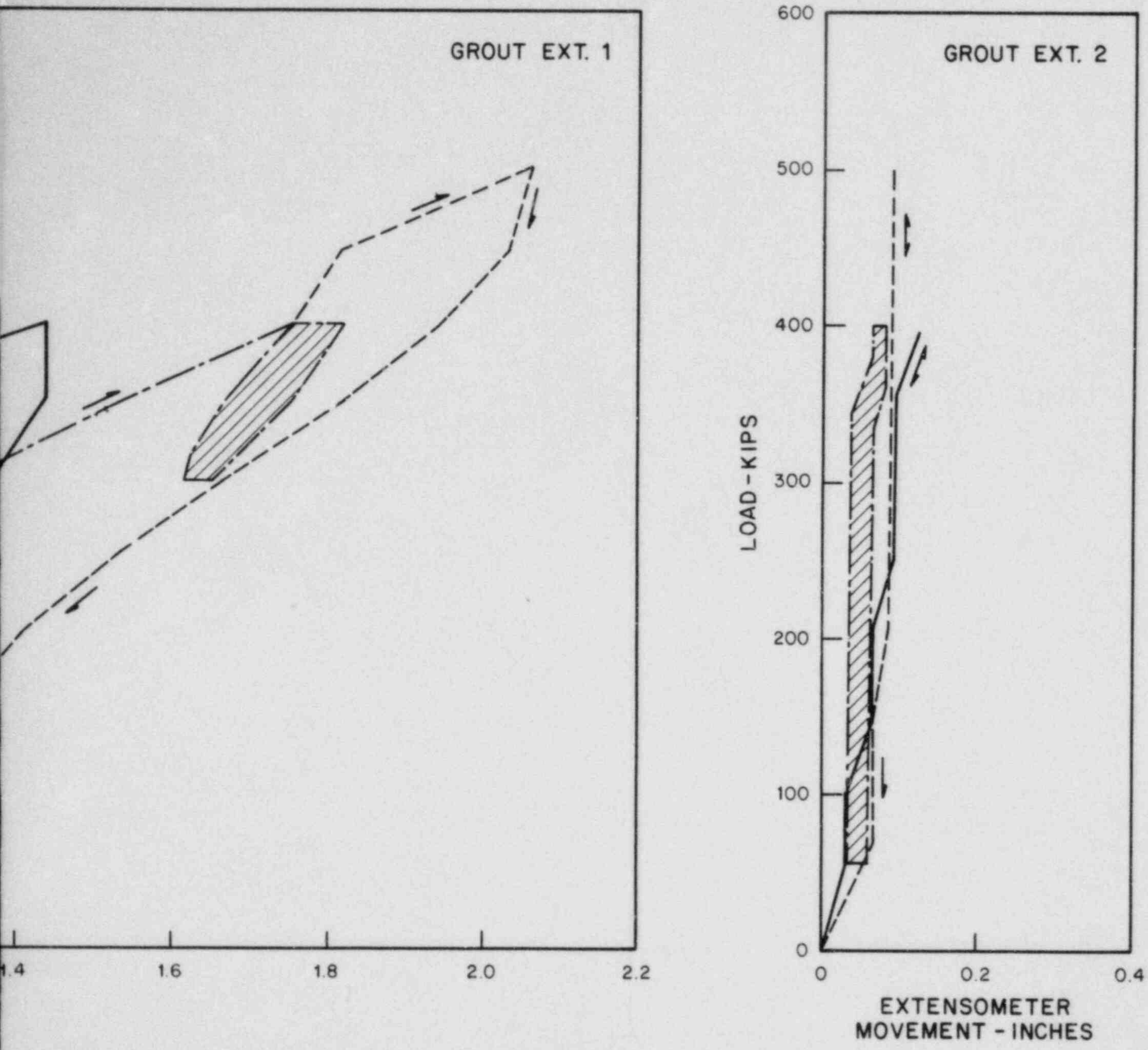


FIGURE 3-28
EXTENSOMETER MOVEMENT
PHASE I-IV STRESSING
TEST ANCHOR 5, 45 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4

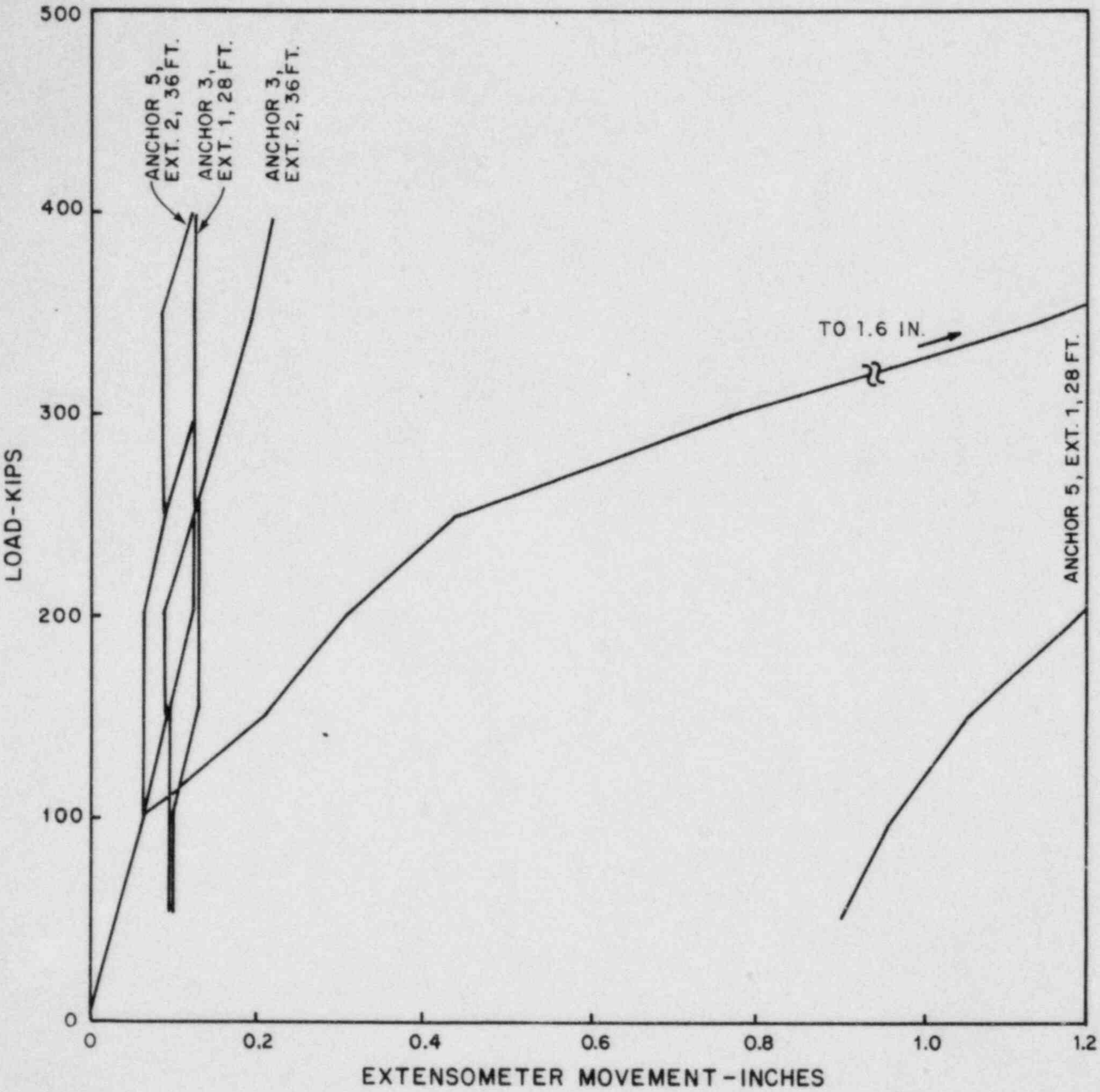
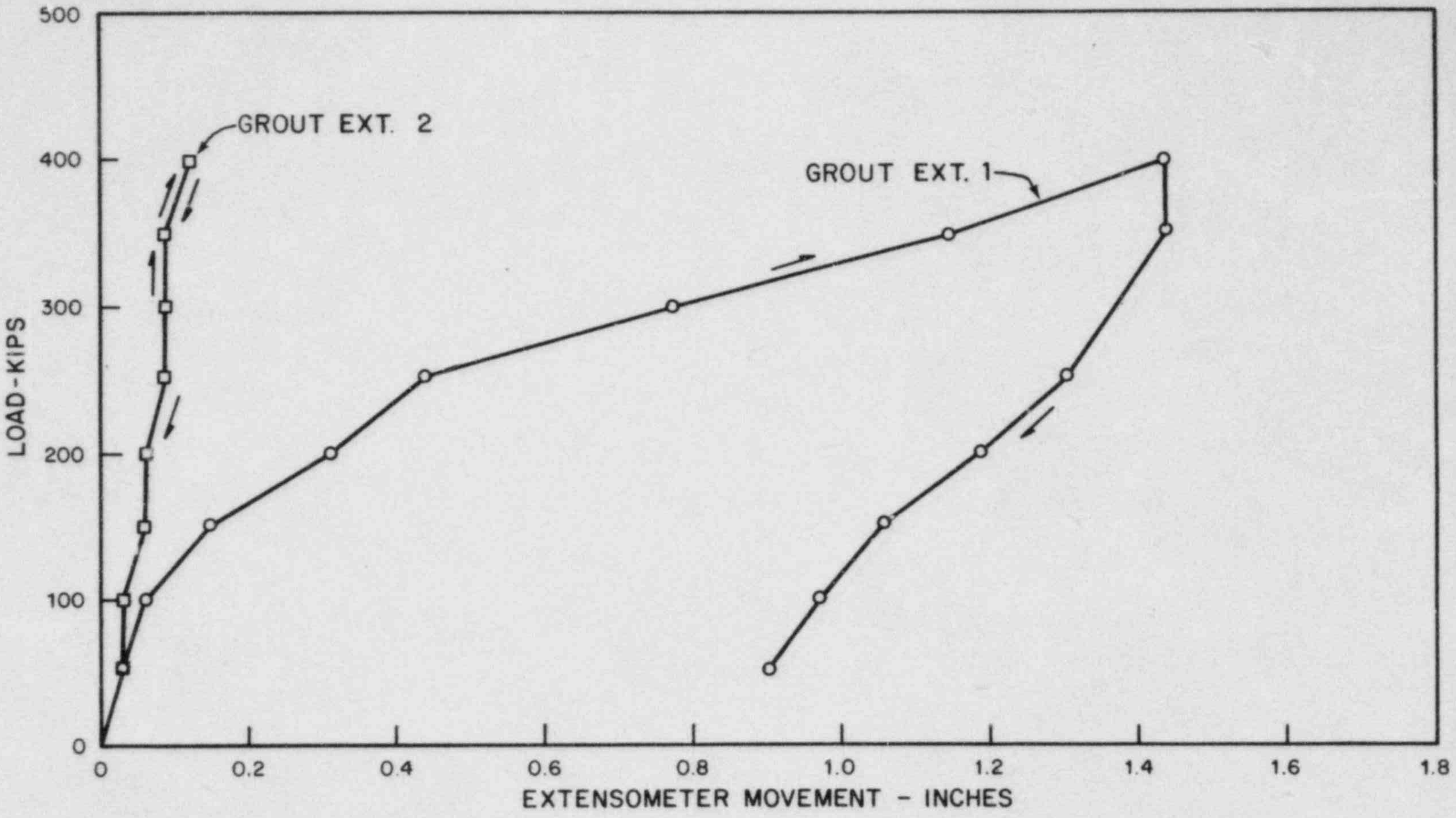


FIGURE 3-29
EXTENSOMETER MOVEMENT
PHASE I STRESSING
TEST ANCHORS 3 & 5, 45 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



NOTE:
SEE FIGURE 2-3 FOR
EXTENSOMETER LOCATIONS

FIGURE 3-30
EXTENSOMETER MOVEMENT
PHASE I STRESSING
TEST ANCHOR 5, 45 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4

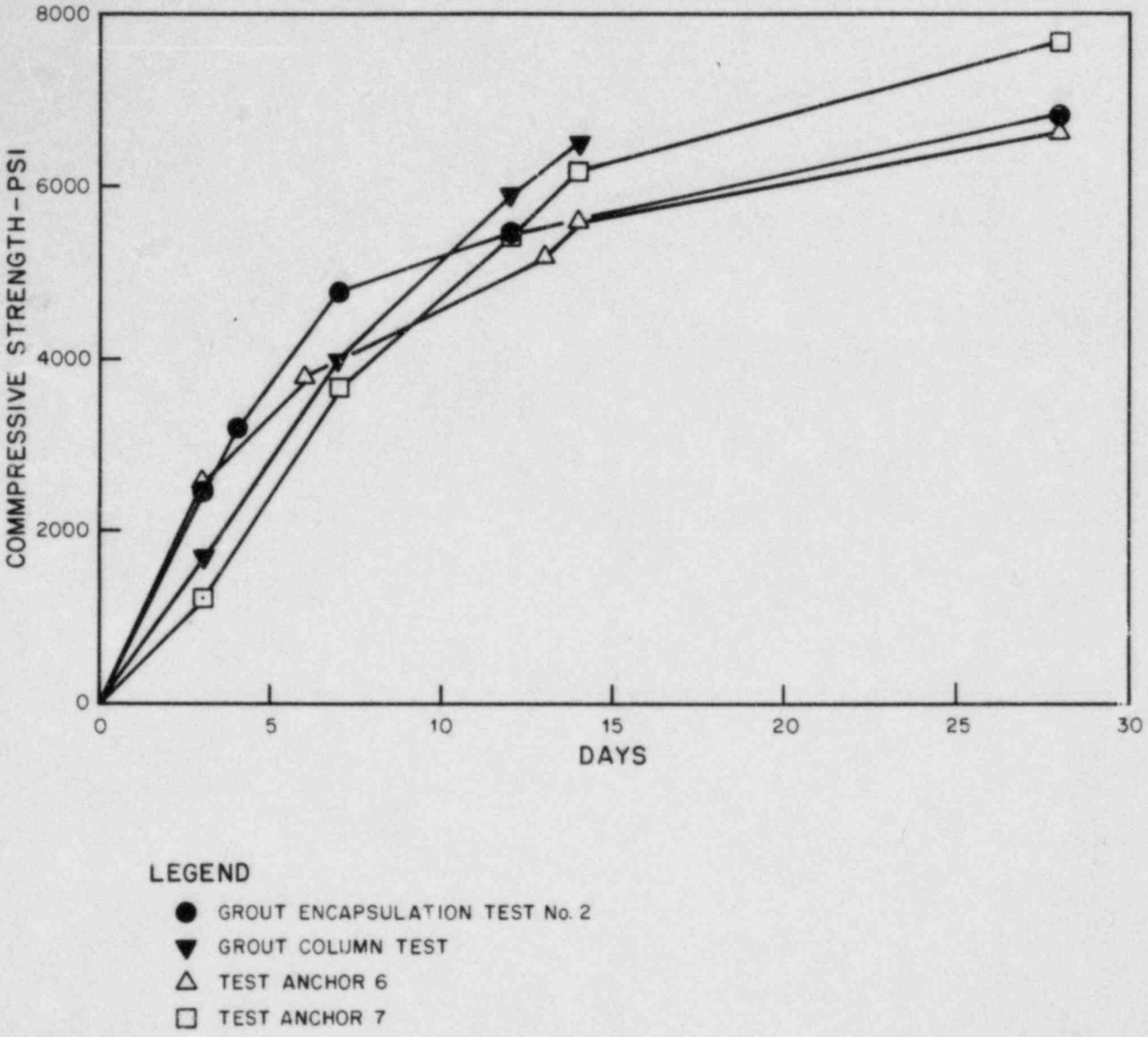
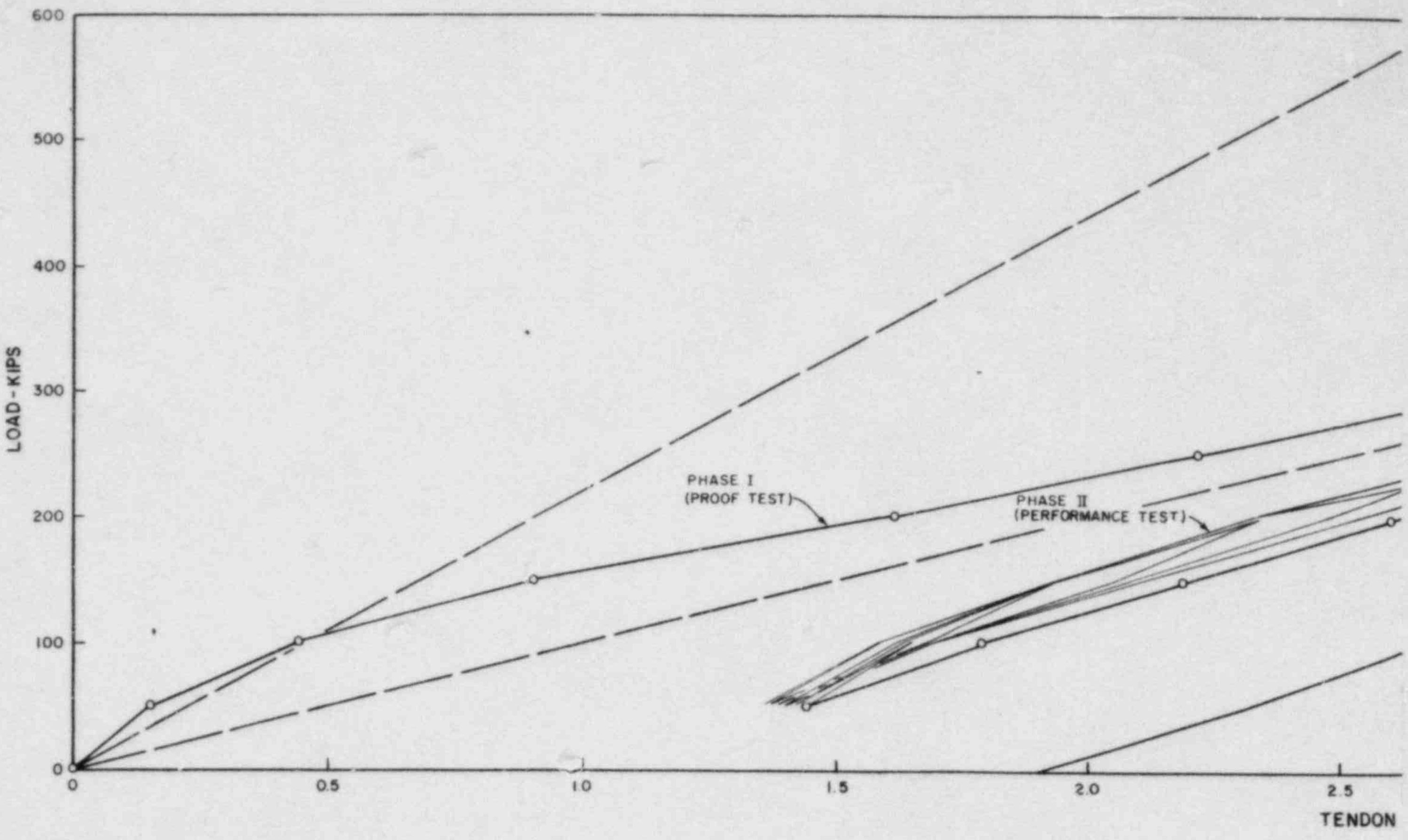


FIGURE 3-31
CUBE STRENGTHS
TYPE II CEMENT GROUT
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



SEE NOTE ON FIGURE 3-1.

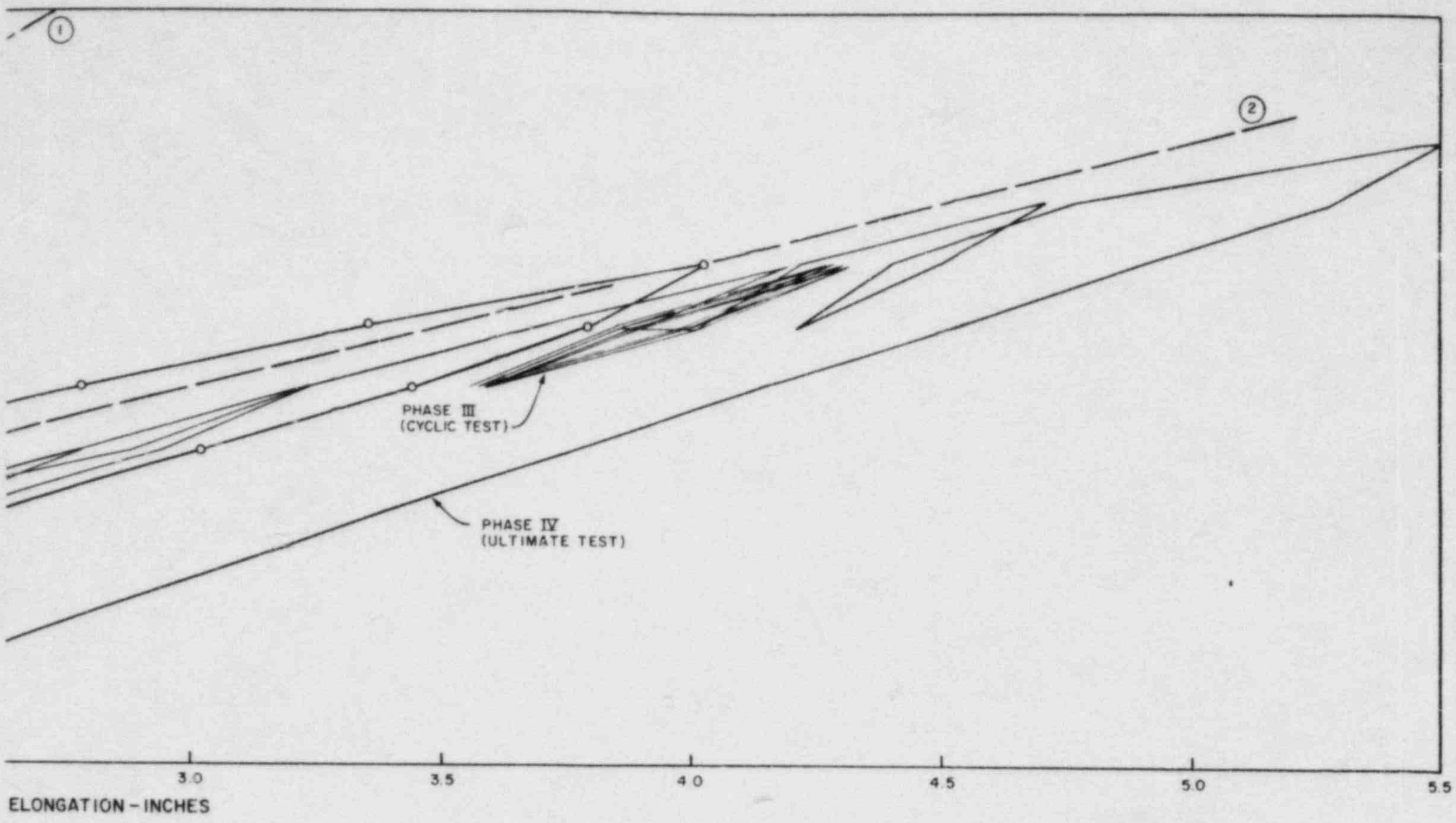


FIGURE 3-32
TENDON ELONGATION
PHASES I - IV STRESSING
TEST ANCHOR 6, 55 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4

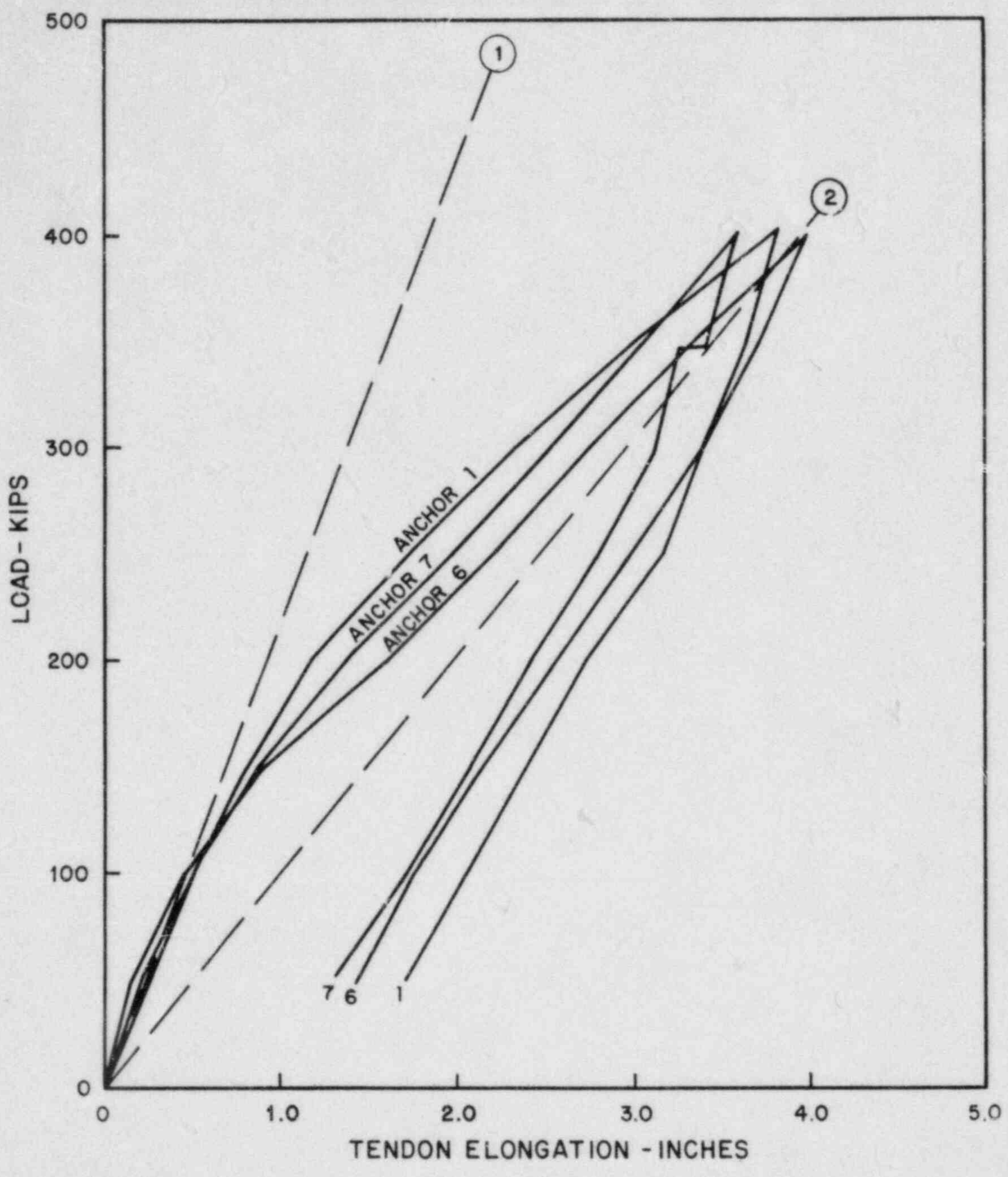
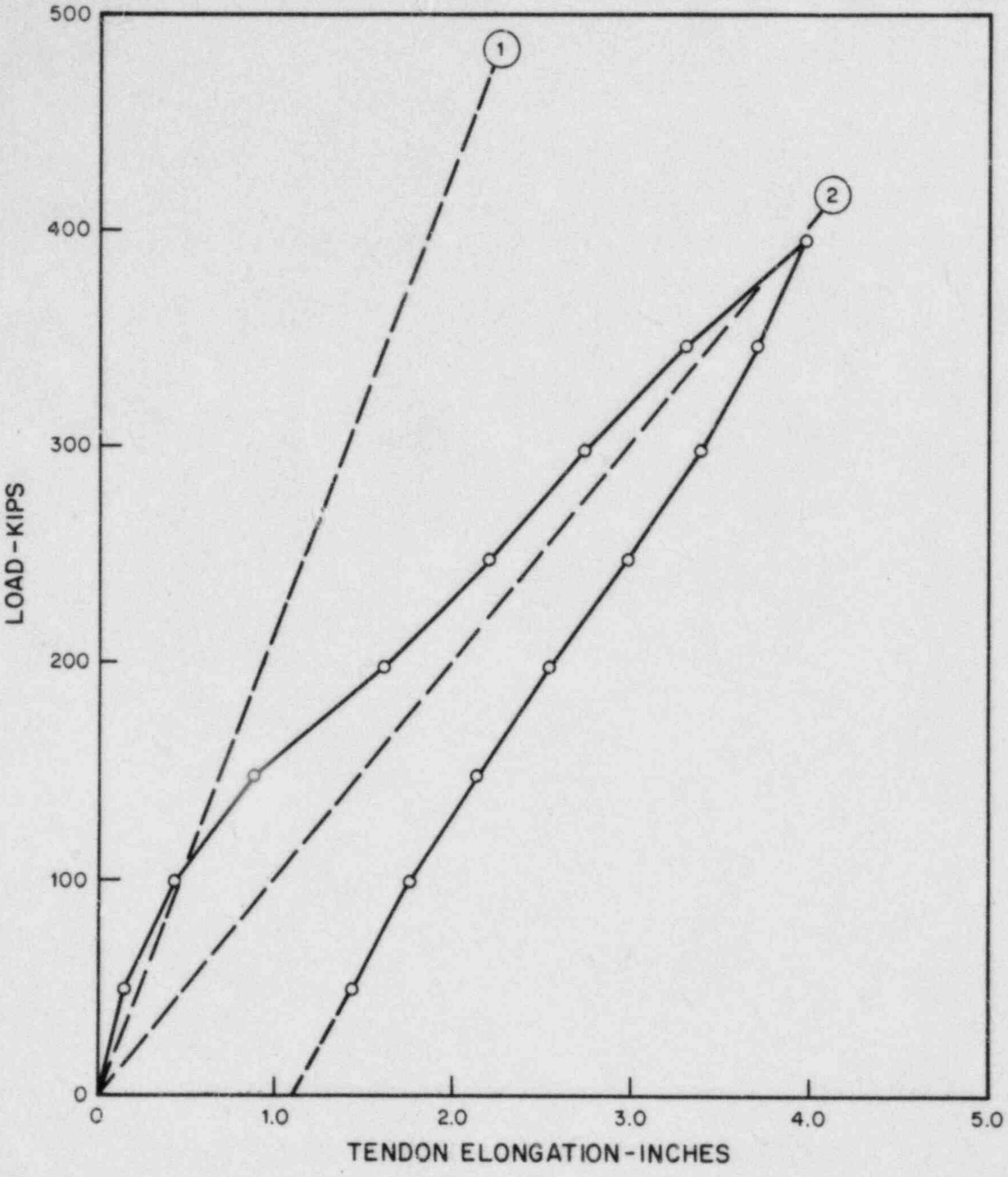


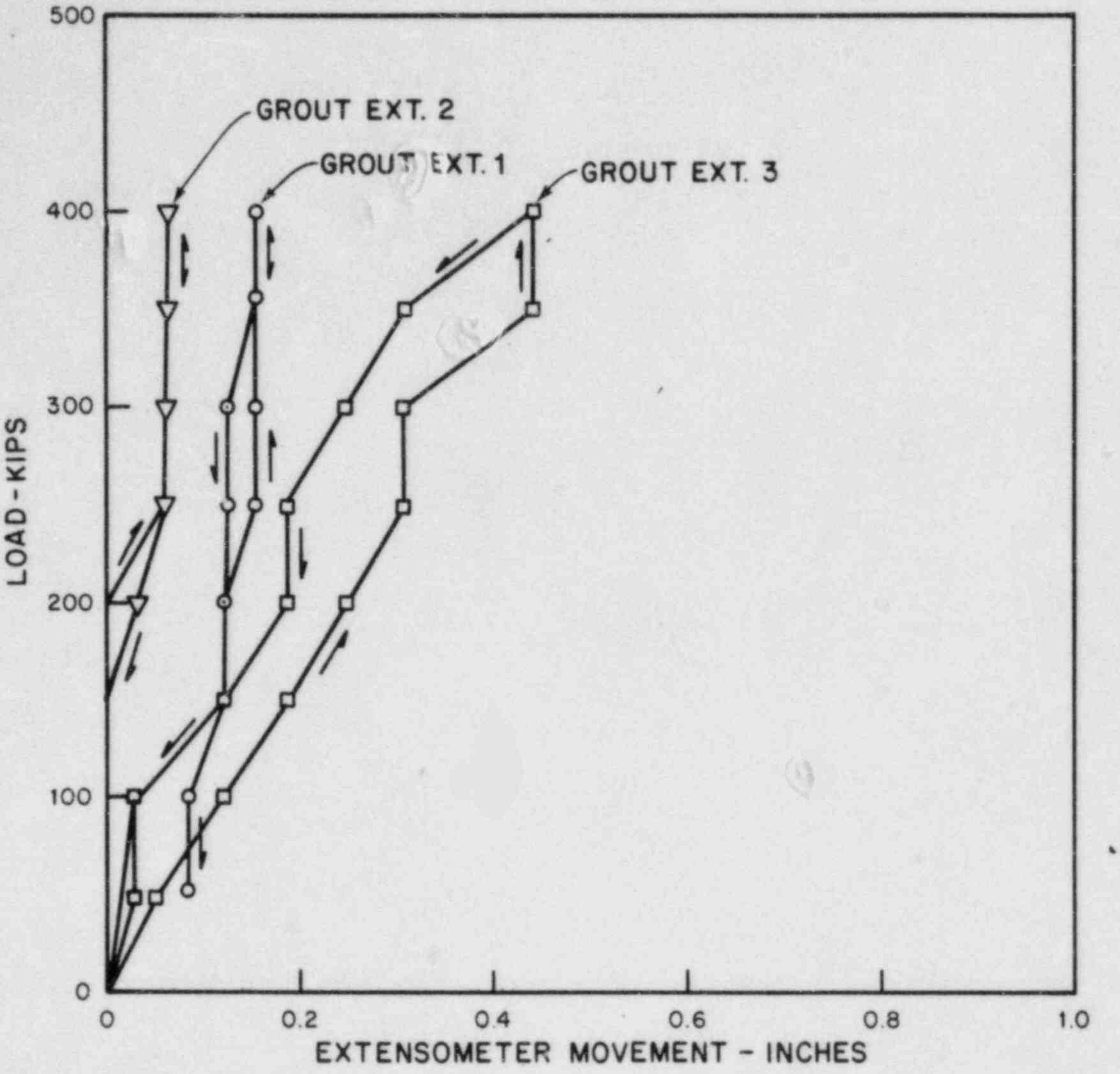
FIGURE 3-33
TENDON ELONGATION
PHASE I STRESSING
TEST ANCHORS 1, 6 & 7
55 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4

SEE NOTE ON FIGURE 3-1.



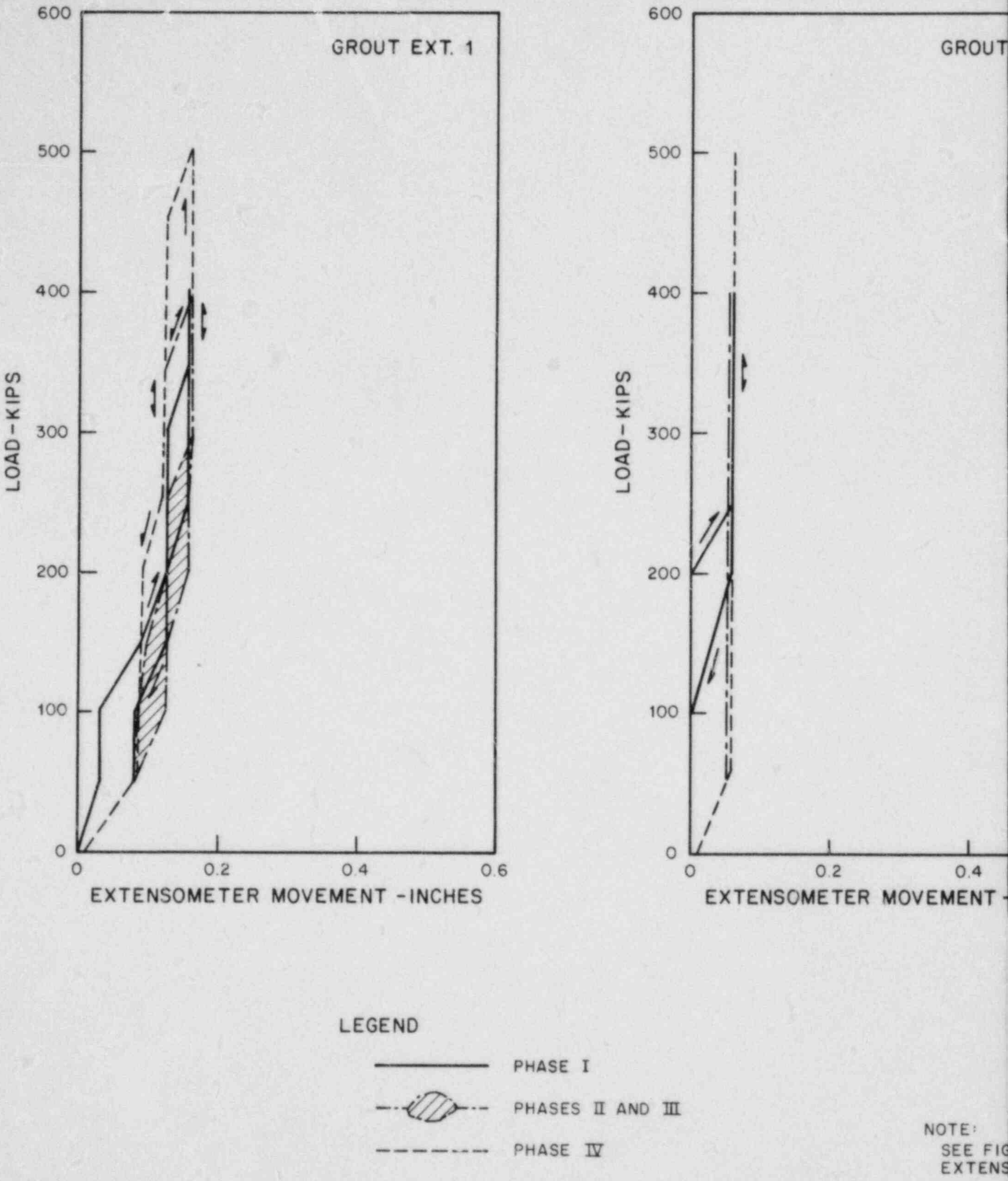
SEE NOTE ON FIGURE 3-1.

FIGURE 3-34
TENDON ELONGATION
PHASE I STRESSING
TEST ANCHOR 6, 55 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4

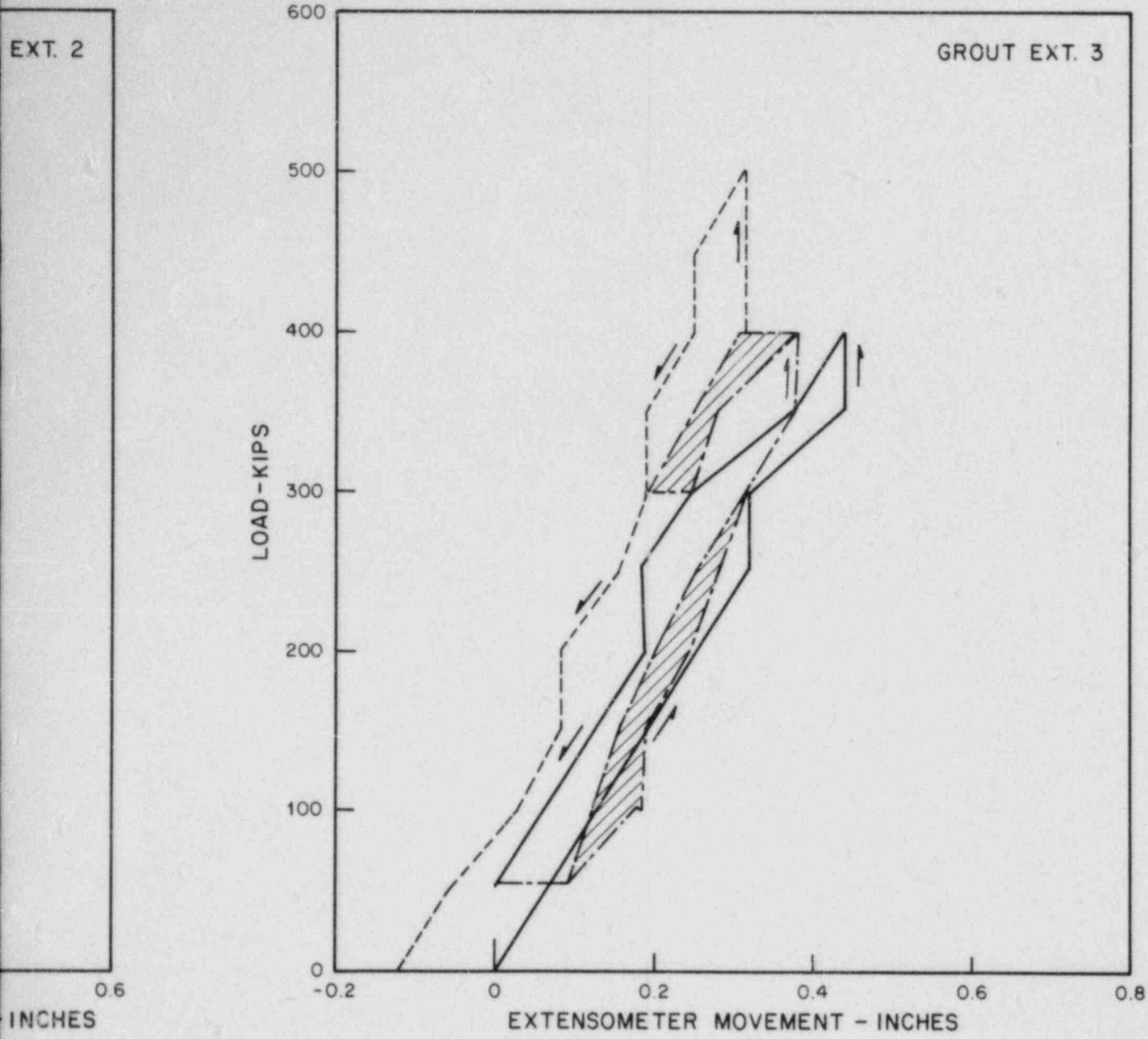


NOTE:
SEE FIGURE 2-3 FOR
EXTENSOMETER LOCATIONS.

FIGURE 3-35
EXTENSOMETER MOVEMENT
PHASE I STRESSING
TEST ANCHOR 6, 55 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4

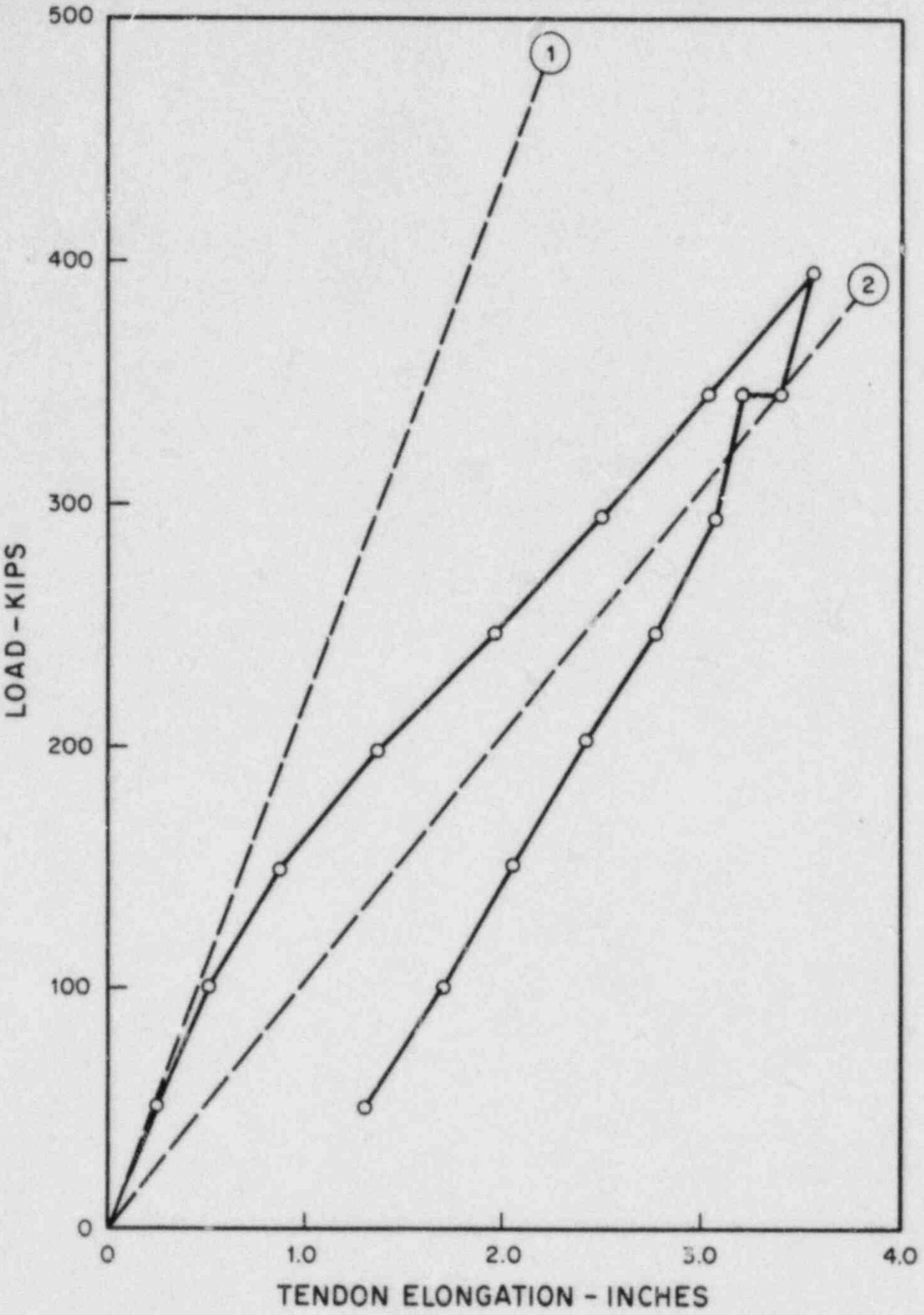


NOTE:
SEE FIG
EXTENS



URE 2-3 FOR
OMETER LOCATIONS.

FIGURE 3-36
EXTENSOMETER MOVEMENT
PHASES I-IV STRESSING
TEST ANCHOR 6, 55 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



SEE NOTE ON FIGURE 3-1

FIGURE 3-37

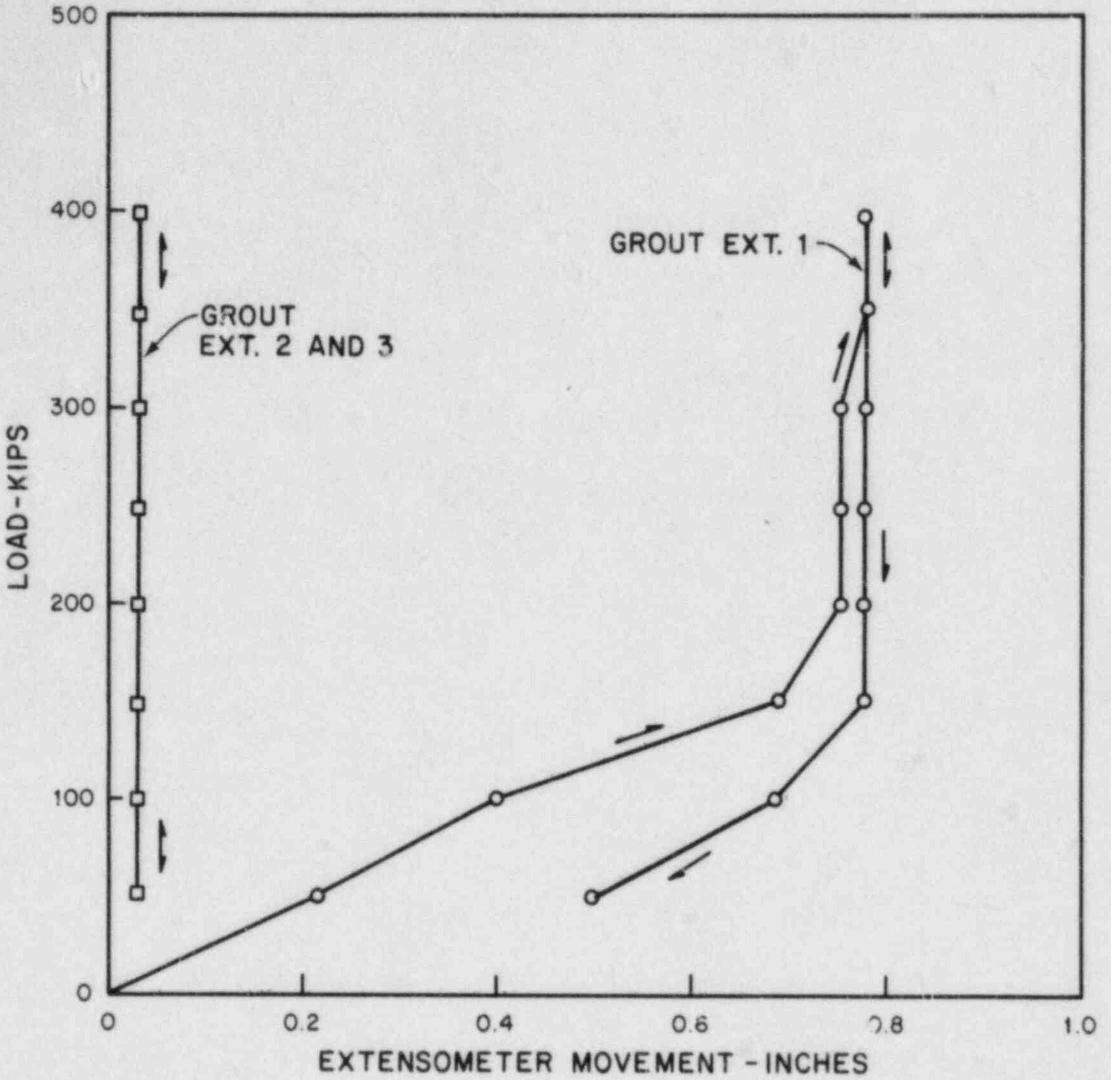
TENDON ELONGATION

PHASE I STRESSING

TEST ANCHOR 7, 55 FT. MULTIWIRE

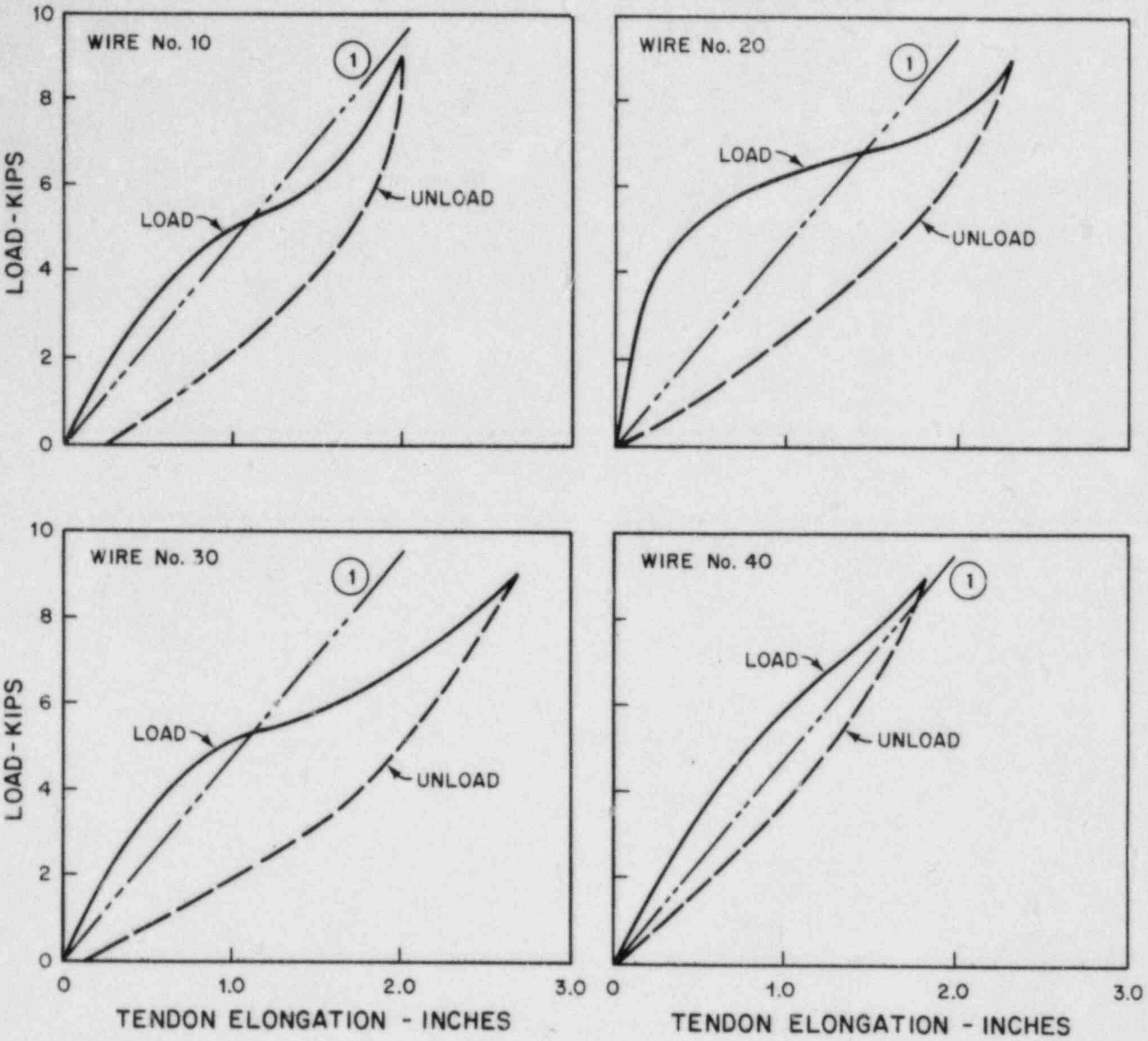
ROCK ANCHOR TEST PROGRAM REPORT

NORTH ANNA UNITS 3 AND 4



NOTE:
SEE FIGURE 2-3 FOR
EXTENSOMETER LOCATIONS.

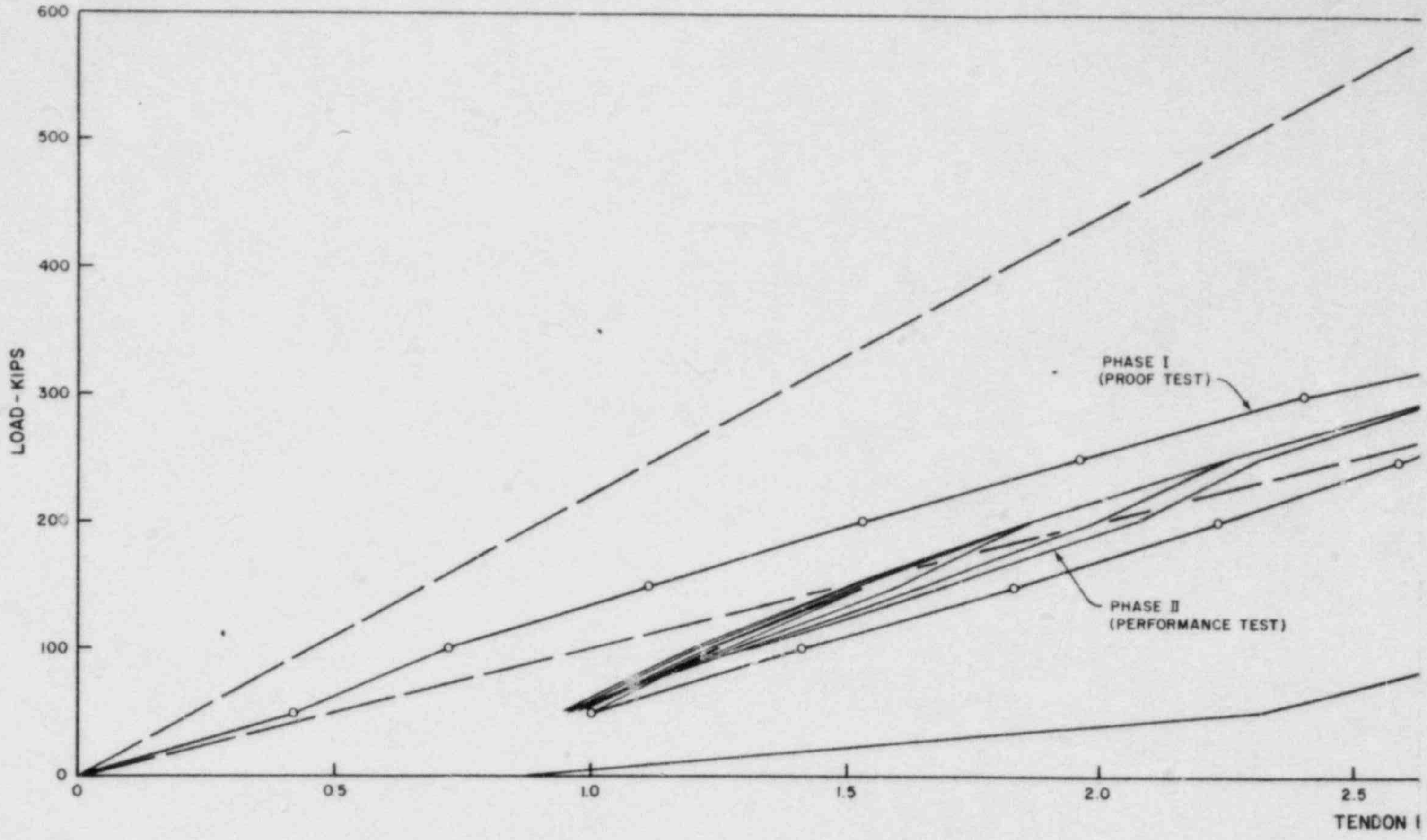
FIGURE 3-38
EXTENSOMETER MOVEMENT
PHASE I STRESSING
TEST ANCHOR 7, 55 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



NOTE:

① THEORETICAL ELONGATION
FOR A FULLY BONDED WIRE

FIGURE 3-39
LOAD-ELONGATION PLOTS
FOR SELECTED WIRES
TEST ANCHOR 7
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



SEE NOTE ON FIGURE 3-1.

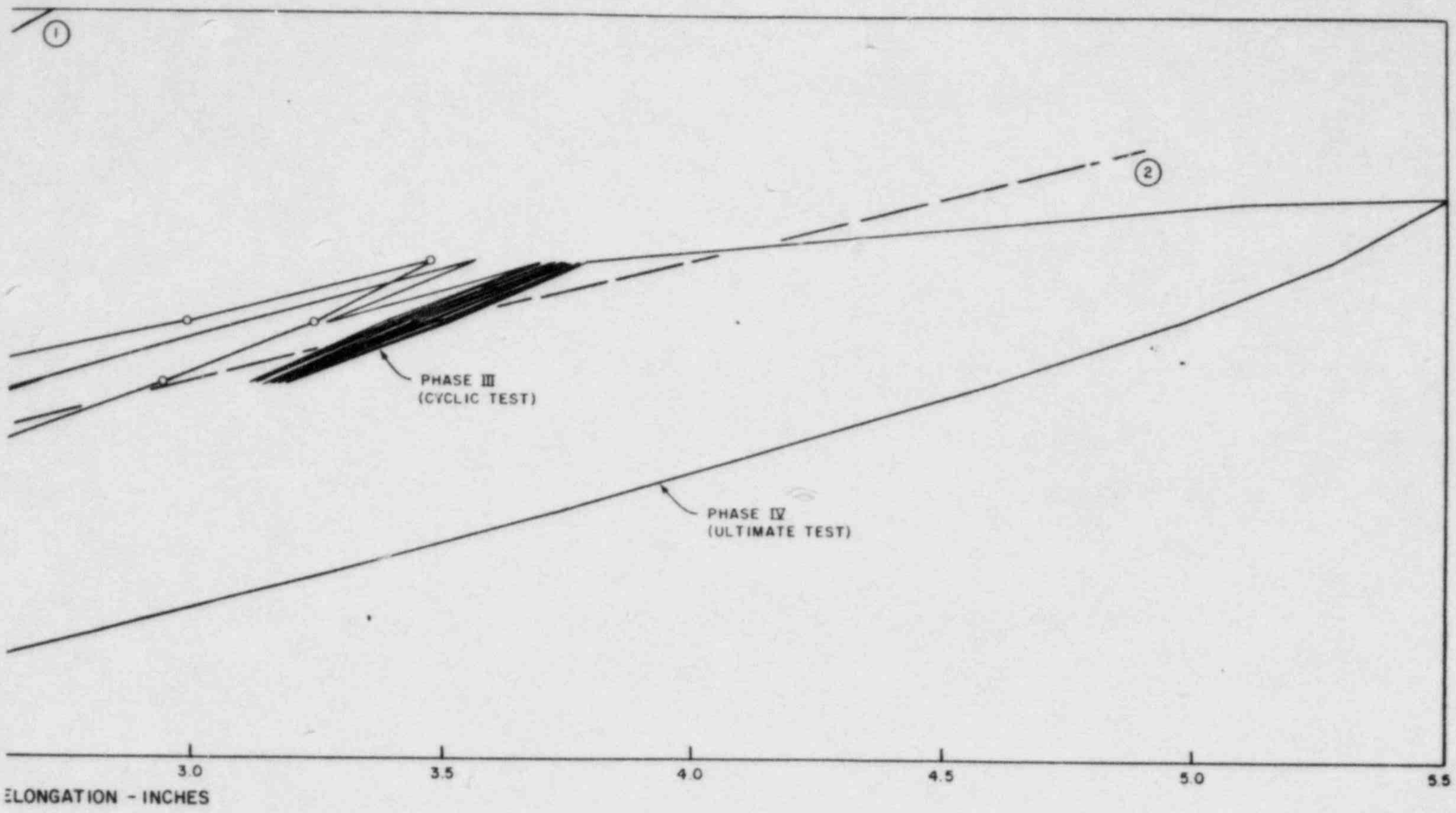
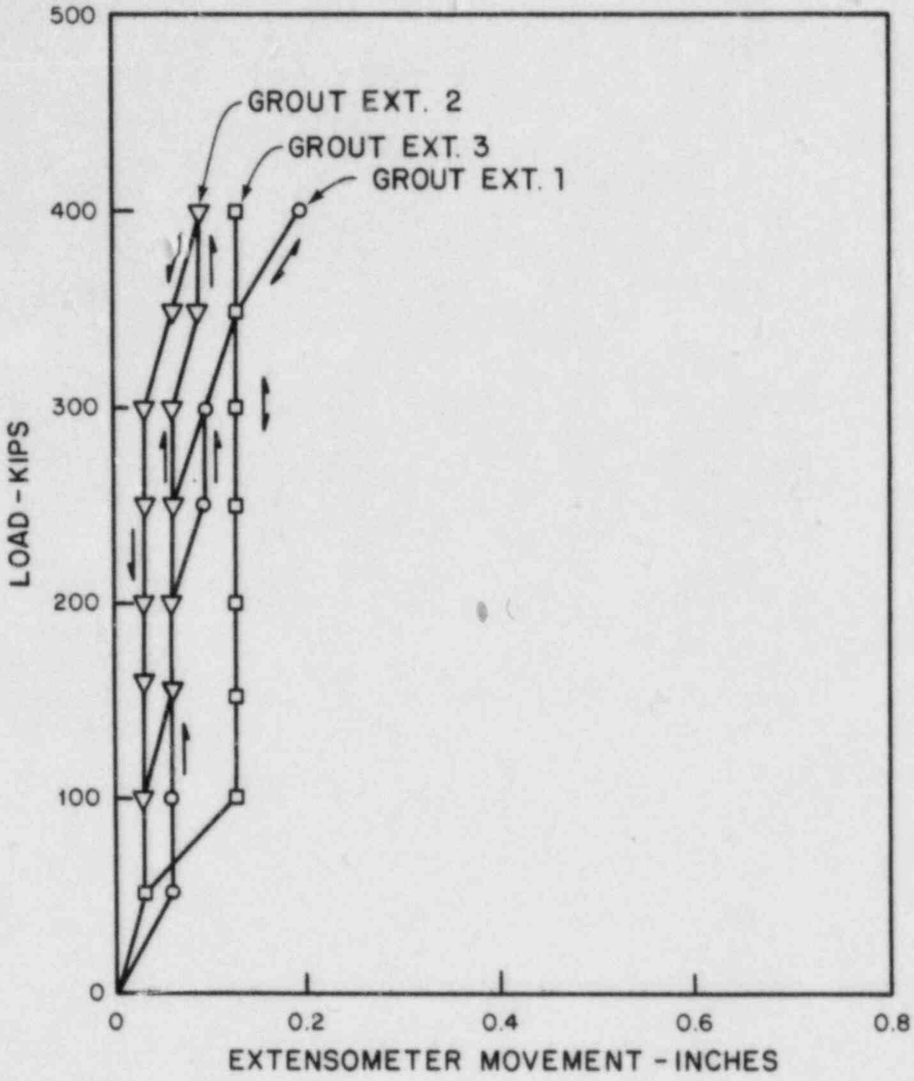
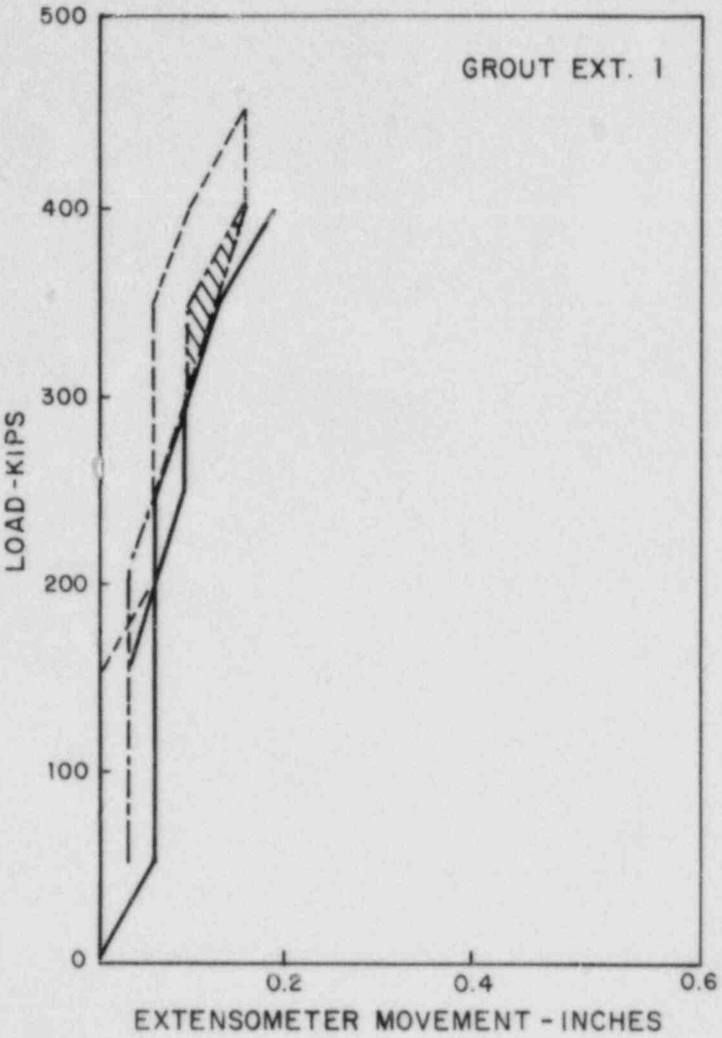


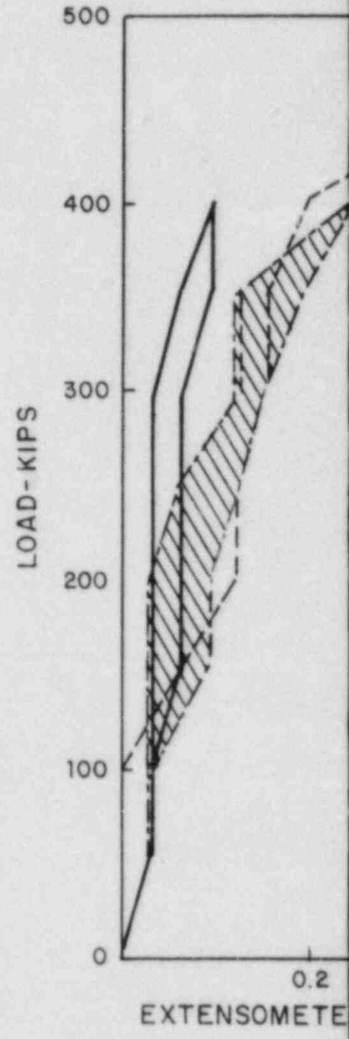
FIGURE 3-40
TENDON ELONGATION
PHASES I - IV STRESSING-RETEST
TEST ANCHOR 7, 55 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4



NOTE:
SEE FIGURE 2-3 FOR
EXTENSOMETER LOCATIONS.

FIGURE 3-41
EXTENSOMETER MOVEMENT
PHASE I STRESSING - RETEST
TEST ANCHOR 7, 55 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4





NOTE:
1. SEE FIGURE 2-3 FOR EXTENSOMETER LOCATIONS
2. GROUT EXTENSOMETER 3 DID NOT MOVE

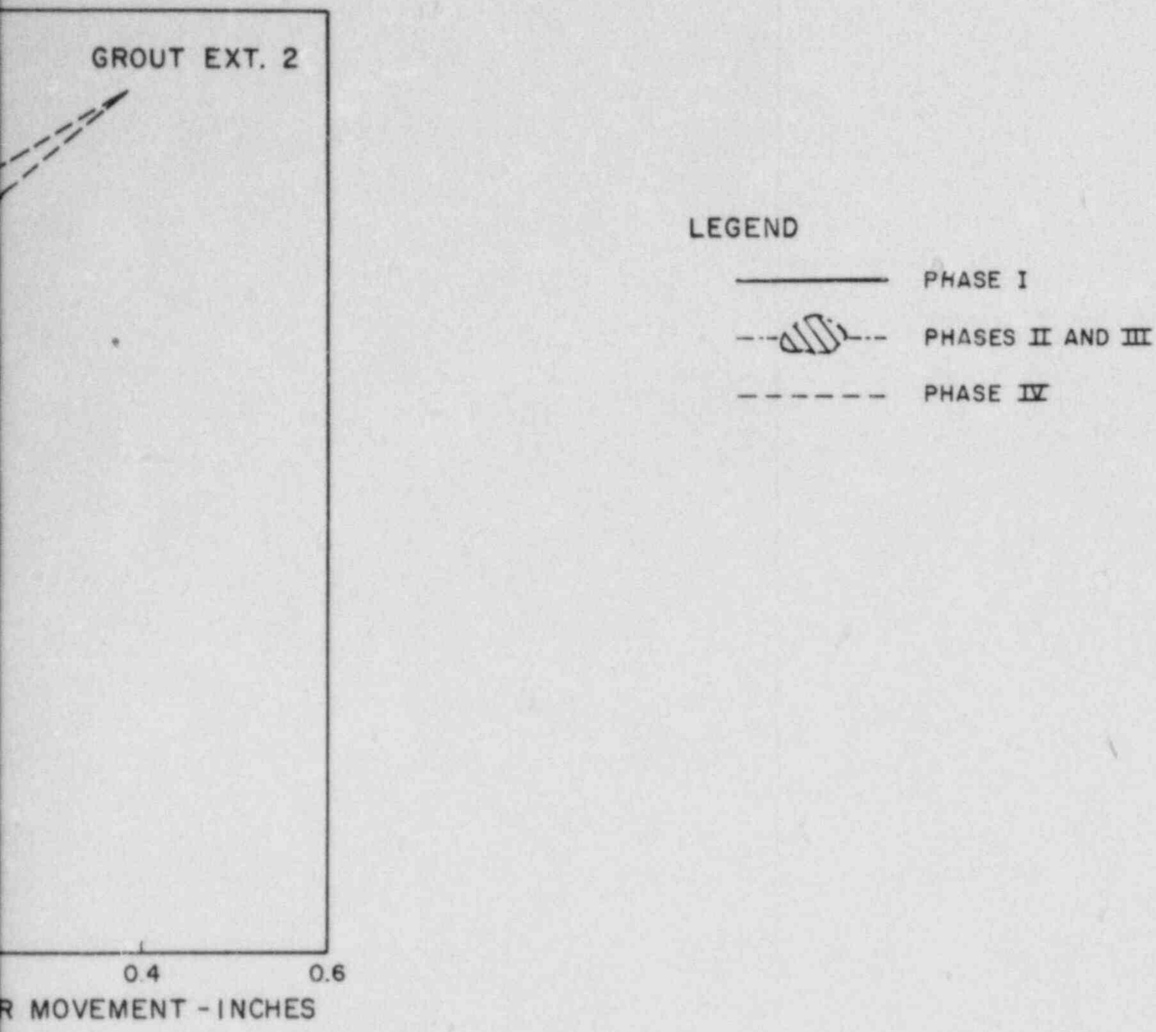


FIGURE 3-42
EXTENSOMETER MOVEMENT
PHASES I-IV STRESSING – RETEST
TEST ANCHOR 7, 55 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4

Figure 3-43*

GROUT COLUMN TEST

Column Height 41'3"

Column Diameter 4"

Date Tested 5-11-76

Distance Gauge 1 to Bottom 6'8"

Distance Gauge 2 to Bottom 2"-4"

Grout: Type II Portland Cement with .5% Interplast 'N' Expanding Agent. Water 5 gals/bag.

TIME	GAUGE 1 P.S.I.	GAUGE 2 P.S.I.	REMARKS
09:55	11.75	13.5	Column filled with water only
10:03	22.25	25.5	Grout filled and tremie pipe removed
10:05	23.5	26.5	Column topped up replacing tremie pipe volume.
10:10	23.25	26.0	
10:15	"	24.25	Slight clear water drips starting at both gauges
10:20	"	24.0	Bleed water forming; fluid level 3/4" below top of column
10:25	"	23.75	
10:30	"	23.5	Clear water drips continue at Gauges. Also at pipe joint 10'6" above bottom.
10:35	"	"	
10:40	"	"	2" deep bleed water formed; fluid level 1" below top of column
10:45	"	"	Clear water drips slowing
10:50	23.0	23.0	
10:55	"	23.0	
11:00	23.25	23.25	
11:05	"	23.0	Fluid level back to 3/4" below top of column.
11:10	23.5	"	
11:15	"	"	
11:20	23.25	"	Slow clear water drips continue.
11:25	23.0	"	
11:30	"	23.25	Bleed water starting to overtop column.
11:35	23.25	23.5	
11:40	"	"	
11:45	23.0	"	Grout risen to within 1/8" of top of column.
11:55	23.25	23.75	
12:00	23.5	"	Grout overtopping column. Visible signs of chemical reaction.
12:15	23.25	"	
12:30	"	23.5	Expansion rate approximately 1/8"/10 mins.
13:00	22.5	23.0	Slow clear water drips continue from gauges and joint.
13:15	"	22.75	
13:30	22.0	22.0	Grout continues to expand & overtop column.
14:00	20.5	20.5	Slow clearwater drips continuing.
14:00	20.5	20.5	Expansion virtually ceased. Bleedwater reforming at top of column. Foam on top of water.
14:30	16.5	17.0	No fluid overtopping column. Foam still forming. Clear water drips at gauges and joint ceased.

*This figure taken from Figure 64 of Appendix B

Figure 3-43

TIME	GAUGE 1 P.S.I.	GAUGE 2 P.S.I.	REMARKS
15:00	16.5	17.0	Foaming
15:20	15.0	16.0	"
15:40	14.5	17.5	"
16:10	13.0	23.5	"
16:30	12.5	22.0	Foaming ceased.
5/12/76 09:00	0.0	0.0	1.55 ft. of bleedwater at top of column Top of grout very soft.

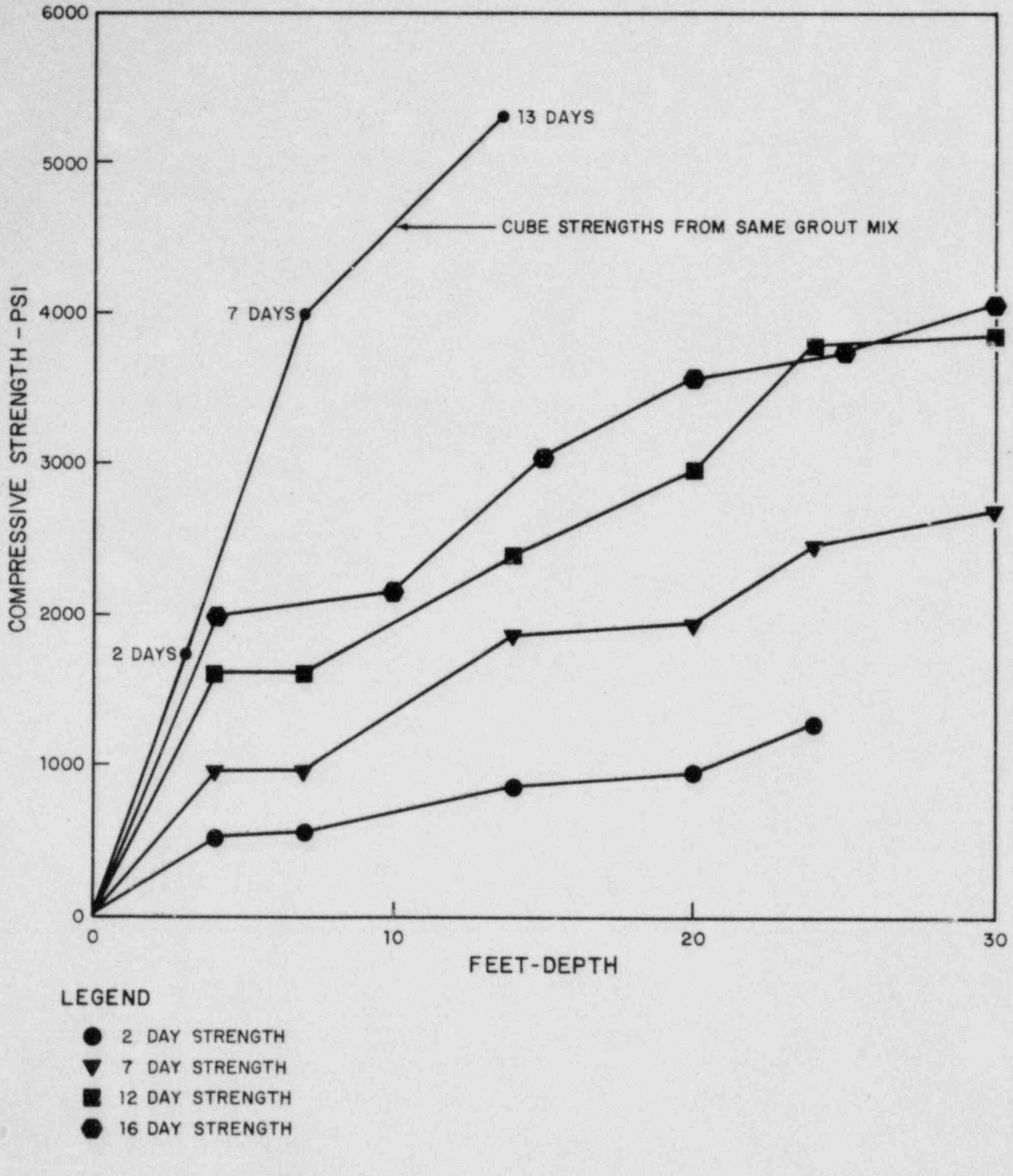


FIGURE 3-44

CYLINDER STRENGTHS GROUT COLUMN TEST

ROCK ANCHOR TEST PROGRAM REPORT
NORTH ANNA UNITS 3 AND 4

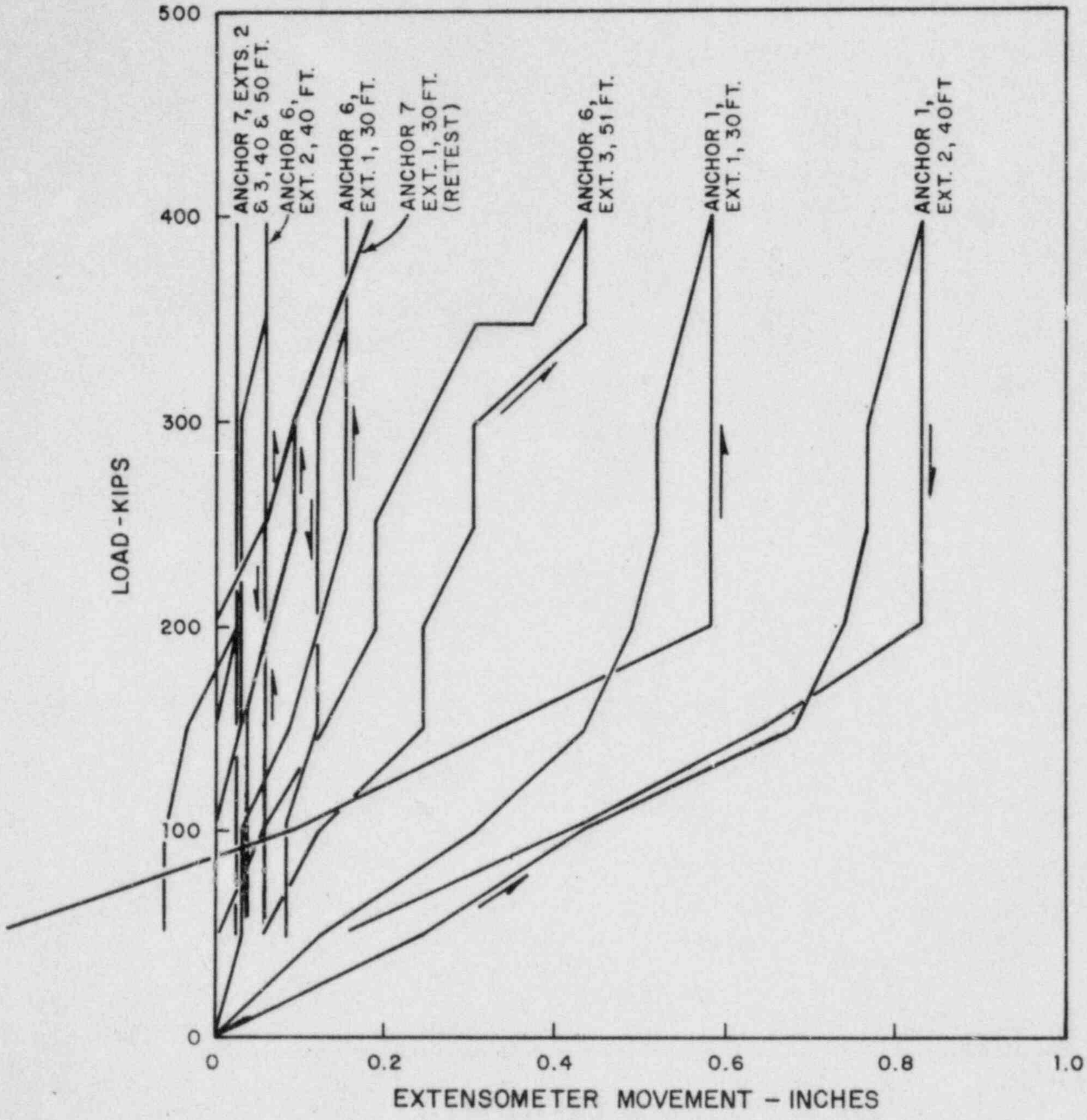


FIGURE 3-45
EXTENSOMETER MOVEMENT
PHASE I STRESSING
TEST ANCHORS 1, 6 & 7, 55 FT. MULTIWIRE
ROCK ANCHOR TEST PROGRAM REPORT'
NORTH ANNA UNITS 3 AND 4

APPENDIX A

SPECIFICATION NAS-30163 REV. 1

POSTTENSIONED ROCK ANCHOR PROOF TEST

J.O.Nos. 12180/12181
NAS-30163

Revision 1
December 1, 1975

Specification for

POSTTENSIONED ROCK ANCHOR PROOF TEST

North Anna Units 3 and 4
Virginia Electric and Power Company
Richmond, Virginia

Stone & Webster Engineering Corporation
Boston, Massachusetts

SECTION 4

QUALITY ASSURANCE PROGRAM

The contents of Section 4 are as follows:

GENERAL

These provisions shall apply to all work done as specified herein for the rock anchor test program for the North Anna Power Station, Units-3 and 4.

Submittal of Quality Assurance Program Summary

Each Bidder shall submit with his original proposal a summary of his Quality Assurance Program in sufficient detail to allow an evaluation of the quality control and quality assurance measures to be imposed by him on his own product and that of his subsuppliers. The Program shall be in compliance with the applicable requirements of 10CFP50--Appendix-B.

Quality Assurance Program Audits

The successful Bidder shall submit to the Engineers in advance, or make available at his plant, his Quality Assurance Program Manual for use in the audit of the program by the Engineers' representatives during the course of the performance of the work specified.

Authorized representatives of the Engineers shall be allowed access to the engineering offices, shops, and, working areas of the Contractor and his subsuppliers at all reasonable times for the purpose of auditing the Quality Assurance Program of the Contractor and his subsuppliers. Such audits will include examination of documentary evidence of activities affecting quality, and will be carried out on a planned, periodic basis during the course of the work to verify compliance with all aspects of the Program and to determine the effectiveness thereof.

If a Bidder has previously submitted his Quality Assurance Program to the Engineers, his proposal shall contain a separate signed document which indicates when his QA program was submitted and a statement that the program is still being followed as submitted.

Contract Documents at Production Location

The Contractor shall specifically ensure that a copy of this specification with all addenda thereto, and of all other contract documents, are readily available at each of his fabricating or production locations where work covered by this specification is in progress.

RESPONSIBILITY FOR INSPECTION

The work will be conducted under the general direction of the Engineers' Superintendent of Construction and will be subject to surveillance by his appointed representatives to ensure strict compliance with the terms of the contract and specifications. The Engineers' Field Inspectors will provide testing and audit functions to assure that the requirements of the specifications have been met. The Geotechnical Representative shall be present to supervise the various aspects of the test program, specifically, the rock drilling and core sampling, rock permeability tests, grouting procedures, stressing procedures and time duration of each load increment, measurements of load versus elongation and time, and to modify the test program, if necessary. The presence of Engineer's Field Inspector and the Geotechnical Representative shall not relieve the Contractor of responsibility for the proper execution of the work in accordance with the specifications.

The Contractor shall permit and provide for inspection, testing, and audit of the work at all times by the Superintendent's representatives, the Engineers' Field Inspector and the Geotechnical Representative.

The Contractor shall also allow sufficient and reasonable time for the Superintendent's representatives,

Geotechnical Representative, and Quality Control personnel
to perform the specified tests and inspections. 4.17

The Contractor shall provide the Engineers full 4.19
information concerning all materials or articles which he 4.20
contemplates incorporating in the work. The required 4.22
information supplied shall include, but not be limited to,
that summarized in the paragraph titled "Items for Quality 4.23
Control." Samples of materials shall be submitted for 4.24
approval when so directed. Equipment, material, and 4.25
articles installed or used without such approval shall be at 4.26
the risk of subsequent rejection.

All materials, supplies, and articles furnished and 4.28
incorporated in this work shall be of the highest grade, 4.29
free from defects and imperfections, of recent manufacture 4.30
and unused. Workmanship shall be of the highest quality and 5.1
in accordance with the best modern standard practice. 5.2

It shall be the Engineers' Field Inspector's 5.4
responsibility to review all tests and material 5.5
certification for completeness and accuracy and to maintain
a complete and orderly file of this information as part of 5.6
the Quality Control records for the project, including all 5.7
certifications and qualification papers.

It shall also be the Engineers' Field Inspector's 5.9
responsibility to review all rework in areas of 5.10
nonconformance to assure that all workmanship below the
standard set by these specifications has been satisfactorily 5.11
corrected.

The Engineers' Field Inspector has the authority 5.13
through the Engineers' Superintendent to prohibit the start 5.14
of various phases of work until inspection has been
provided; forbid the use of materials, equipment, or 5.15
workmanship which do not conform to specifications; and to
stop work which is not being done in accordance with plans 5.16
or specifications.

They may also require the removal or repair of 5.18
faulty construction or of construction performed without 5.19
inspection.

Implementation of the requirements contained in 5.21
this specification is the responsibility of the Engineers 5.22
and shall be permitted by the Contractor, without hindrance.

TESTS 5.26

Rock Anchors - Stressing 5.29

The stressing of the rock anchors shall be 6.1
conducted by the Contractor under the direction of the
Geotechnical Representative in accordance with the 6.3
"Stressing Procedure," Section 2 of this specification. The 6.4
Geotechnical Representative shall witness all stressing of
rock anchors.

Rock Permeability Tests 6.6

Rock permeability tests shall be conducted by the 6.8
Contractor under the direction of the Geotechnical 6.9
Representative. The tests shall be performed prior to the 6.10
installation of the anchors and in accordance with the "Rock 6.11
Permeability Tests," Section-2 of this specification.

Grout Tests 6.13

Two types of grout tests shall be performed by the 6.15
Engineers to verify the compressive strength of the grout 6.16
prior to stressing the tendons. The tests shall be 6.17
conducted in accordance with the "Grout Tests," Section-2 of
this specification. In addition, grout tests, as required 6.18
under "Performance Tests on Materials," shall be performed
by the Engineers.

Mixing Water 6.21

Chemical analyses of the mixing water for the grout 6.23
"Performance Tests on Materials," shall be performed by the 6.24
Engineers.

INSPECTION 6.29

As used herein, the term "Engineers' field 7.2
inspector" means the Senior Field Quality Control (FQC)
Engineer at the jobsite and the FQC inspectors assigned to 7.4
his staff.

The Engineers' or Purchaser's field inspector shall 7.6
have the right, at all reasonable times, to inspect the
Contractor's or subcontractor's work, material and 7.7
equipment, or inspection procedures as applicable to the
work covered by this specification to confirm that the 7.8
requirements of this specification are being complied with.
The Contractor or subcontractor shall provide all tools, 7.9
instruments, scaffolding, etc., necessary to facilitate
these inspections.

The Contractor shall cooperate with the Engineers' 7.11
field inspector in establishing when the various inspections
or tests will be performed during the progress of the work. 7.12
The Engineers' field inspector shall designate which of 7.13
these he is required to witness or participate in, and the
Contractor shall furnish an agreed-upon amount of 7.14
notification prior to the start of each.

The Engineers' field inspector shall: 7.16

1. Discuss it with the Contractor whenever he (the 7.18
inspector) notices anything that may lead to
rejection of the work. 7.19

2. At the appropriate time, witness or conduct the 7.21
various inspections, tests, verifications, etc., as
indicated on the enclosed "Field T.I.D. Report" 7.22
forms, and indicate that each is in conformance
with the requirements of this specification by 7.23
signing and dating in the spaces provided therefor.

It is not intended that the presence or activity of 7.25
the Engineers' field inspector shall relieve the Contractor
in any way whatsoever of his obligation to maintain an 7.26
adequate test, inspection, and documentation program of his
own, or of any other obligation under this specification. 7.27
Furthermore, the fact that the Engineers' field inspector 7.28
may inadvertently overlook a deviation from some requirement
of this specification shall not constitute a waiver of that 7.29
requirement, or of the Contractor's obligation to correct
the condition when it is discovered, or of any other 7.30
obligation under this specification.

By acting through established channels, the 8.2
Engineers' field inspector has the authority and
responsibility to stop any portion of the work which, if 8.3
continued, would make compliance with some requirement of
this specification difficult or impossible. 8.4

The specific duties assigned to the Engineers' 8.6
field inspector are as follows:

1. Anchor-Materials---Verify that certificates of 8.8
compliance and CMTRs from the manufacturer of the
following items and the required tests by the 8.9
Engineers on the following items are in accordance
with "MATERIALS" and "CORROSION PROTECTION," and
"Performance Tests on Materials" Section-2 of this 8.10
specification:

   a. complete tendon and anchorage assemblies, 8.12
   b. all grout ingredients and final mix, 8.14
   c. all lubricants 8.16

2. Equipment---verify that the various accessories used for drilling, grouting, stressing, and the rock permeability tests are in compliance with "EQUIPMENT" Section-2 of this specification.

3. Drilling-Boreholes---verify that the procedures specifed under "Borehole Alignment," "Drilling Technique," and "Washing and Cleaning Anchor Holes," Section-2 of this specification are complied with.

4. Rock Permeability Tests---witness the performance of the rock permeability tests and verify that the procedures specified under "Rock Permeability Tests" Section-2 of this specification are complied with.

5. Tendon-Installation---witness the complete tendon assembly installation and the grouting operations.

6. Grout-Tests---perform and/or verify the required grout tests as specified under "Grout Tests" and "Performance Tests on Materials," Section-2 of this specification.

7. Stressing-Procedure---witness all rock anchor stressing along with measurements of all instruments and verify the calibration requirements under "Accessories for Stressing and Testing Anchors," Section-2 of this specification.

8. Grout-Encapsulation-Test---witness the procedures under "Grout Encapsulation Test," Section 2 of this specification.

DOCUMENTATION

Documentation by Contractor

The basic documentation required by the Contractor is as follows:

Title	Distribution and No. of Copies	
	EFI	EPE
Documentation Checklist	3	3
Certified Material Test Reports	3	3
Calibration of Equipment and Instrumentation	3	3

	Distribution and No. of Copies		
	EFI	EPE	
Report on Rock Anchor Test Program	3	3	10.22 10.23

EFI - Engineers' Field Inspector 10.25
EPE - Engineers' Project Engineer, or designee 10.26

Based upon these basic documentation requirements, 11.5
and at the time indicated in the Schedule, the Contractor 11.6
shall submit to the Engineers a detailed "Documentation 11.7
Checklist"; this shall specifically itemize each individual 11.8
document that will be submitted, each identified in two 11.9
ways, namely:

1. Type of document such as mill test report, liquid 11.11
penetrant test report, etc. 11.12

2. The specific component(s), or parts thereof, to 11.14
which the document applies. 11.15

This Checklist shall be organized in a logical, 11.17
easy-to-follow format, so that for any shipment (whether it 11.18
be a partial or complete shipment), it will be readily 11.19
possible to ascertain from the Checklist the complete
consist of documentation applicable to that shipment. If 11.21
more than one shipment to the jobsite is involved, this
means the Checklist should be organized by components as 11.22
shipped, with all documents applicable to each component as
shipped separately itemized.

This Documentation Checklist, when submitted to the 11.25
Engineers, will be reviewed for its adequacy. If 11.27
satisfactory, it will be stamped "No Comment," and a copy
returned to the Contractor. If not satisfactory, it will be 11.29
returned with suggestions noted; it shall then be promptly 11.30
revised and resubmitted.

Documentation by the Engineers' Inspector 12.3

The Field T.I.D. Report included with this 12.5
specification, when signed and dated, comprises the specific 12.8
documentation required of the Engineers' inspector. The 12.9
T.I.D. Report shall be signed and dated at the appropriate
time by the Engineers' Field inspector as each item listed 12.10
thereon is verified, witnessed, or performed; it shall then 12.11
be submitted to the Engineers and to the Purchaser.

This T.I.D. Report is not meant to limit the 12.13
additional tests, inspections, or documentation required by 12.14
the specified codes or normally provided by the Contractor. 12.15

(See the "INSPECTION" section of this specification for the definition of the term "field inspector.")

The signature of the Engineers' inspector for each item on the T.I.D Report shall be his certification that, to the extent of his responsibility as delineated in the specification, the item signed for is in conformance with the requirements of the specification.

CERTIFICATION

Materials	Certification
Prestressing Steel	Manufacturer's specification of material conforming to ASTM A-416 including yield and ultimate stress and the requirements under "Performance Tests on Materials."
Nuts and Washers	Manufacturer's specification of material characteristics conforming to ASTM A-325 requirements and the requirements under "Performance Tests on Materials."
Bearing Plates	Manufacturer's specification of material requirements conforming to ASTM A-36 requirements and the requirements under Performance Tests of Materials."
Lubricant	Manufacturer's specification of material requirements and the requirements under "Performance Tests on Materials."
Tendon Anchorage	Manufacturer's specification of material requirements and the requirements under "Performance Tests on Materials."
Cement	Manufacturer's specification of material requirements conforming to ASTM C 150 and the requirements under "Performance Tests on Materials."
Hydraulic Jack	Manufacturer's specification indicating capacity and calibration certification of pumps and gages.

Materials	Certification	
Long Range Dial Indicators	Manufacturer's specification indicating capacity and calibration certification of gage.	14.26 14.27 14.28
Load Cells	Manufacturer's specification indicating capacity and calibration certifi-	15.1 15.2

SECTION 5

NONENGINEERED ITEMS

The QA Category-I items listed below shall be purchased using the following manufacturer's and catalog numbers. No equals shall be acceptable unless specifically identified below, or by addenda to this specification. Upon delivery, documentation accompanying the items listed below shall be checked for compliance with the subsection entitled "Materials" in Section-2 of this specification.

Multistrand Tendons

VSL Corp. Catalog Number ER5-16, 7-wire

-j3-strand prestressing steel, including the anchor head, bearing plate, wedges, sleeves, grease, and PVC sheathing or pipe for the unbonded length.

Inland-Ryerson Catalog Number CONA, 7-wire

-j2-strand (0.5-in.) prestressing steel and Catalog Number 12/.05-FG Anchor Assembly

Stressed steel 7-wire, 12-strand (0.5 in. diameter)

Prestressing steel with anchor assembly

PIC Corp. 7-wire, 13-strand (0.5 in. diameter)

Prestressing steel with anchor assembly

Prescon Corp. 7-wire, 13-strand (0.5 in. diameter)

Prestressing steel with anchor assembly

Multiwire Tendons

Inland-Ryerson Catalog Number BBRV 46-wire

(0.25-in.) prestressing steel and Catalog Number-46BG Anchorage Assembly

Prescon Corp. 46-wire (0.25 in.)

Prestressing steel with anchor assembly

APPENDIX B

REPORT ON THE CONSTRUCTION AND TESTING OF ROCK ANCHORS, PREPARED BY NICHOLSON ANCHORAGE COMPANY

NAC
GROUND ANCHORS
NICHOLSON ANCHORAGE COMPANY
P. O. BOX 308 BRIDGEVILLE, PENNSYLVANIA 15017
412/221-4500

REPORT ON THE CONSTRUCTION & TESTING OF ROCK ANCHORS

UNITS 3 & 4, NORTH ANNA POWER STATION
MINERAL, VIRGINIA

REPORT NO. 5007

FEBRUARY, 1977

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I GENERAL INTRODUCTION

North Anna Power Station is located near Mineral, Virginia in Louisa County, and is on the south bank of Lake Anna. The site is approximately 40 miles NNW of Richmond, Virginia and 70 miles SW of Washington, D.C. The power station is under construction by the Stone & Webster Engineering Corporation (S.W.E.C.) for the Virginia Electric & Power Company.

The containment auxiliary structures, contiguous to the Units 3 and 4 reactor containments, will require permanent rock anchors at the foundation mat level. A comprehensive anchor test program was required to:

a) verify design assumptions;

b) evaluate performance of two anchor tendon types to enable selection of the one more suited to the eventual application;

c) evaluate the effect of the anchor system upon the foundation mat of the structures and on the adjacent rock.

The test program was conducted using installation techniques, equipment and materials, and stressing sequences similar to those envisioned for the permanent rock anchors. The work was performed by Nicholson Anchorage Company (N.A.C.) under Contract No. NAC-30163. Site operations commenced on December 1, 1975 in accordance with S.W.E.C. specification No. NAS-30163 "Post-Tensioned Rock Anchor Proof Test."

Figure 1 shows the location of the test anchor together with the layout of the rock and concrete pad instrumentation and survey points. The first four anchors were located adjacent to the Unit 4 containment auxiliary structures, and the remainder within the area planned for the auxiliary building.

A total of seven separate anchor tests were performed together with two tendon encapsulation tests, three grout test holes, and one grout column test. The results of all these tests are presented in the body of the report, and show that the anchors, as designed, successfully withstood the severe test loading sequences specified.

II SITE GEOLOGY

The test anchor locations were chosen to simulate the geologic conditions that could be expected during the construction of the permanent anchors. The foundation rock at the test anchor locations is a slightly weathered to fresh grey granite gneiss. The cores recovered from each anchor borehole show the rock quality as excellent (95% to 100% recovery and 80% to 100% Rock Quality Designation). Complete descriptions of the recovered cores were made by the S.W.E.C. geotechnical representative. Core drilling was slow with poor rates of penetration and high bit wear due to the hardness and abrasive nature of the rock.

III ANCHOR TEST PROGRAM

Original Program

The original scope of work called for four anchors to be constructed and tested, two to be of a multi-strand system and two to be of a multi-wire system. One anchor of each pair was to be 45 ft. long and the other 55 ft. long. One additional 45 ft. long tendon of each type was to be supplied and used in a grout encapsulation test.

A fifth borehole to serve as a grout test hole was to be drilled and cement grouted. This grout was to be cored subsequently at specific intervals of time and depth with the cores being tested for crushing strength. The purpose of this test was to determine the grout quality and strength under actual field conditions.

Rock extensometers of the grout-in-place variety were to be provided at points equidistant between each pair of anchors. The extensometer anchor points were to be set at 12'6" and 25' below anchor head level in each case. Movements were to be monitored at the reference heads by means of a micrometer depth gauge at every loading stage.

Grout column extensometers were required at specified levels in each test anchor borehole. The anchor points were connected to the surface by stainless steel wires in a plastic sheath and grout column movements were detected by monitoring calibrated pulleys over which the connecting wires had been passed.

Continuous surveying of a number of fixed positions around the test anchor locations was carried out during the stressing sequences. The positions of these points are shown on Fig. 1 and the survey work was to be by S.W.E.C. personnel.

Modifications & Additions to the Program

The results obtained from the testing of the first four anchors indicated the need to extend the test program to obtain further information. This was done by installing one 45 ft. multi-wire tendon in a borehole using a similar type grout to the first four anchors but having a slightly higher water/cement ratio, and two 55 ft. multi-wire tendons using a different grout. The 45 ft. tendon originally intended for use in a grout encapsulation test was utilized in Test Anchor No. 5 and a 25 ft. multi-strand tendon was used for the encapsulation test.

To examine grout quality and strength to depth and age characteristics of the cement grout used in Test Anchors No. 6 & No. 7, a grout column test was carried out in simulation of the grouted borehole provided in the original program.

To further examine grout bond properties, two more shorter boreholes were drilled in an area adjacent to the three last anchors. These were filled with different types of cement grouts and, after allowing a suitable curing period, diamond drilling methods were used to recover rock/grout interfaces for the length of the boreholes.

Sufficient information had been obtained from the survey of the rock surface markers and from the monitoring of the rock extensometers during the first four tests to enable this part of the test procedures to be dispensed with for the last three anchor tests.

IV ANCHOR MATERIALS

Anchor Tendons

For the purposes of the test program the multi-strand type tendon materials were supplied by V.S.L. Corporation, and Prescon Corporation supplied the multi-wire type tendons. All tendons were assembled at the manufacturer's plants and then shipped to the site for installation. The unbonded length of each anchor tendon had each wire or strand coated with a corrosion inhibiting grease and encased in a plastic sheath. This provided permanent corrosion protection to the anchors and also precluded the transmission of anchor load into the grout column above the designed bond length. As-built details of each anchor tendon including type of grout are shown in Table I. When referring to this table, it should be noted that Test Anchors #1 to #4 were the original requirement and that Anchors #5, #6 and #7 were the additional ones. Also #7 was shortened from 55 ft. to 54'6-1/4" during the fitting of the replacement pulling head to that tendon.

Multi-Strand Tendons

These were as manufactured and supplied by the V.S.L. Corporation and all components were from their standard range of post-tensioning materials and associated matching

TABLE I

ANCHOR NO.	1	2	3	4	5	6	7
Tendon Type	Prescon 46x0.25"Ø Wires of 240 KSI Steel	V.S.L. 13x0.5"Ø Strands of 270 KSI Steel	Prescon 46x0.25"Ø Wires of 240 KSI Steel	V.S.L. 13x0.5"Ø Wires of 240 KSI Steel	Prescon 46x0.25"Ø Wires of 240 KSI Steel	Prescon 46x0.25"Ø Wires of 240 KSI Steel	Prescon 46x0.25"Ø Wires of 240 KSI Steel 45 Wires After Fitting New Head
Minimum Guaranteed Ultimate Tensile Strength (G.U.T.S.)	542.8 KIPS	537 KIPS	542.8 KIPS	537 KIPS	542.8 KIPS	542.8 KIPS	542.8 KIPS/531 KIPS After Fitting New Head
Overall Length	55'0"	55'0" Excl.Stress Tail	45'0"	45'0" Excl.Stress Tail	45'0"	55'0"	55'0"
Bonded Length	30'1"	29'6"	20'2"	19'6"	20'2"	29'9"	30'0"
Unbonded Length	24'11"	25'6"	24'10"	25'6"	24'10"	25'3"	25'0"
Borehole Length	57'0"	57'0"	47'0"	47'0"	56'0"	57'0"	57'0"
Bearing Plate Elevation	+236'	+236'	+245'	+245'	+245'	+245'	+245'
Distance Bearing Plate-Top of Grout	23'0"	24'3"	23'0"	24'4"	24'8"	23'4"	24'3"
Distance Bearing Plate-Bottom Spacer	45'0"	50'0"	45'0"	43'6"	45'0"	45'0"	45'5"
Distance Bearing Plate-Center Spacer	35'0"	40'0"	35'0"	36'0"	35'0"	35'0"	35'1"
Distance Bearing Plate-Top Spacer	25'0"	30'0"	25'0"	30'0"	25'0"	25'0"	25'0"
Distance Bearing Plate-Grout Extensometer Anchor Points	1@30'0" 1@40'0"	1@30'0" 1@40'0"	1@28'0" 1@36'0"	1@28'0" 1@36'0"	1@28'0" 1@36'0"	1@30'0" 1@40'0" 1@50'9"	1@30'0" 1@40'0" 1@50'0"
Grout Type	MasterFlow 814	MasterFlow 814	MasterFlow 814	MasterFlow 814	MasterFlow 814	Portland Cement Type II w/Intraplast 'N' @ 0.49#/bag	Portland Cement Type II w/Intraplast 'N' at 0.49#/bag
Water/Cement Ratio	2 Gals./55# Bag	2 Gals./55# Bag	2 Gals./55# Bag	2 Gals./55# Bag	2.4 Gals./55# Bag	5 Gals./94# Bag	5 Gals./94# Bag

components. The anchor tendons were made up as shown in Fig. 2. Figures 3 and 4 show the details of the heads and wedges used. Strand material certification is included as Fig. 5.

The strands were greased and sheathed with polyethylene for unbonded lengths shown in Table I. The grease was "Visconorust 3166", a corrosion resistant lubricant applied prior to application of the sheathing. This plastic coating was "Pi-mark" 80 psi 5/8" polyethylene tubing to ASTM 2239-71. A description of the grease is given in Fig. 6. After the strands had been greased and sheathed, the ends of each sheath tube were bound with duct tape to prevent leakage of grease onto the bonded portions of the tendon.

Multi-Wire Tendons

These were as manufactured and supplied by the Prescon Corporation and all components were from their standard range of post-tensioning materials and associated matching components. Fig. 7 shows the general arrangements of the Prescon tendons and Fig. 8 details of the stress heads and base plates. Stress is transferred from the stress heads to the tendon wires by means of button heads formed by upsetting the ends of each wire after completion of component assembly. A Materials Certification is attached as Fig. 9 for Anchors #1, #3 and #5. The base plate diameters were 5-1/4" for Anchors #1, #3 and #5 and 6" for Anchors #6 and #7. The stress head diameters were 6" for all tendons.

Similar greasing and sheathing was used for the multi-wire tendons as for the multi-strand except that the plastic sheathing was 3/8" I.D. and the grease was Shell C.C. Grease No. 1 as supplied by Shell Oil Company. Details of this grease are shown in Fig. 10.

Grout Materials

For Anchors #1 through #5, MasterFlow 814 Cable Grout as supplied by Master Builders of Cleveland was used. This is a material blended from selected cements and containing fluidifiers and thickening agents. It was designed to be a high strength, non-bleed material for grouting exhibiting thixotropy and giving good corrosion protection of highly stressed steel cables. Anchors #1 through #4 and Grout Test Hole #1 were filled with grout having a water/cement ratio of 2 gallons per 55 pound bag of MasterFlow 814 under the direction of a Master Builders field representative. After performance of various tests in which the developed grout to rock bond strength was found to be very low, Anchor #5 was grouted with a more fluid mix, the water/cement ratio being 2.4 gallons of water per 55 pound bag of cement. A similar mix was used for one tendon encapsulation test and for Grout Test Hole No. 2.

As there was no significant improvement in grout/rock bond due to this alteration in water content, Anchors #6 and #7 were grouted using Type II Portland Cement as supplied to the construction site batch plant, having a water/cement ratio of 5 gallons of water per 94 pound bag of cement and containing an expansive admixture. This admixture was 'Intraplast N' as supplied by Sika Chemicals and incorporated into the mix at the rate of 0.5% by weight of cement. Expansion of the grout results from the release of hydrogen gas caused by reaction of the aluminum powder of the admixture with the alkalis released by hydration of the cement. A similar grout was used in the grout column test.

The second tendon encapsulation and Grout Hole No. 3 were carried out using the Type II Portland Cement at a water/cement ratio of 5 gallons of water per 94 pound bag of cement without any admixture.

Test Reports

It was a specification requirement that Certified Material Test Reports (C.M.T.R.) be provided for all tendon and anchorage components and accessories. This included reports of actual results of chemical analyses, physical and mechanical tests, declarations of compliance, and methods of identification of materials, and was submitted by Nicholson Anchorage Company to S.W.E.C. in the form of documentation check lists referring to the documents numbered NAC-5007-1 to 89. It should be noted that some documents comprising the C.M.T.R. have also been used as Figures in this report. These may be identified by the presence of a C.M.T.R. documentation number NAC 5007-XX.

V ANCHOR CONSTRUCTION

Concrete Test Pads

In order to simulate actual working conditions, each test anchor was stressed against the reaction of a reinforced concrete block as shown in Fig. #11. These concrete pads were 4 ft. thick, the expected thickness of the structural mats at the future anchoring points, and were constructed by S.W.E.C. Each pad contained an 8" I.D. steel pipe sleeve to duplicate the borehole location and access method for the permanent rock anchors. In Anchors #1 to #4 the test pads were poured prior to start of anchor construction and access for the drilling equipment was gained by means of temporary earth ramps provided by S.W.E.C. For Anchors #5 to #7 the pads were poured after completion of borehole drilling but prior to installation of the anchor tendons.

Rock Drilling

Drilling of the anchor boreholes was commenced by taking a continuous rock core for the full depth except for Anchors #6 and #7 where only the bonded length was cored. Cores were of the specified NX size and accomplished using a Joy Type 12B core drill and a double tube core barrel having a split inner tube as manufactured by Christensen Diamond Products. Rock cores were placed in standard wooden core boxes and logged by the Stone & Webster geotechnical representative. Reference should be made to these logs for a detailed description of rock type and quality.

Upon completion of core drilling the boreholes were reamed out to the full designed diameter of 6-1/2". This was accomplished using 6" nominal diameter down-the-hole hammer fitted with a 6-1/2" button head bit on a Nicholson Anchorage Modified RDC-16 fully hydraulic rotary drill rig using compressed air as a flushing medium. A test was performed to ensure that the drill cuttings could be collected at a central location and without uncontrolled dispersal in the immediate working area. The cuttings from the first two boreholes (#3 & #4) were collected into a cap drum and piped away for disposal elsewhere. As the procedure proved successful, it was discontinued after drilling borehole #4. On completion of drilling, the boreholes were thoroughly cleaned by water and/or compressed air purge through the drill rods until the flushing water was clear of cuttings and the hole cleaned of debris.

Due to the hardness and abrasive nature of the rock, drilling generally was slow with poor rates of penetration and high bit wear. Drop center button bits were used but, even with redressing of the buttons, they were found to be under gauge and unusable after completion of two boreholes.

Alignment Checks

Vertical alignment of the boreholes was established by means of spirit level and square on the leading drill rods. Alignment was thereafter maintained by the use of stabilizer drill rods behind the drill bit. Starting tolerances using this method were proved to be better than ±1° which represents a deviation of 1.75 ft. in 100 ft. Thereafter the maximum deviation possible within the hole was limited to 1 inch in 10 feet by use of a special stabilizer rod 10 ft. long behind the drill bit and hammer that was but 1 inch less in diameter than the borehole. Following completion of drilling the boreholes were checked for verticality by means of a plumb line to establish that the alignment was within the permitted tolerance of ±3°. Only two boreholes were dry enough to permit precise measurements to be made. Grout Test Hole No. 1 was checked and found to have deviated from vertical by 5-1/2 inches at a depth of 46 ft. or 0.57°. Anchor Borehole No. 6 was measured as being 1/2 inch out of vertical. All other boreholes were estimated as being within these two figures.

Rock Permeability Tests

Following completion of drilling, each borehole was water tested to measure rock permeability. Two types of tests were conducted. The first, a falling head test, was carried out by filling the borehole with water and then checking the water level at specific time intervals by means of an electrical dip meter.

Following the falling head test a pressure test was carried out. For this test the borehole was filled with water and an inflatable packer was lowered into the hole to a depth determined by the geotechnical representative. A steel pipe connected the packer to the surface and, after inflation of the packer sleeve, water was pumped through this pipe at a constant pressure of 25 psi into the section of borehole isolated below the packer. Rate of flow was determined by use of a flow meter and pressure by 0-100 psi range Bourdon gauge. Calibration certificates were obtained for these and all other instruments used.

Results of the falling head and pressure packer tests are attached as Figures #12 to #23.

Preliminary Grout

In accordance with specification NAS-30163 if the leakage rate from the borehole during the pressure test exceeded 0.001 gallon per inch diameter per foot of depth per minute for the 10 minute test period, then preliminary grouting was specified. In addition, preliminary grouting was required if the leakage rate into the borehole during the falling head permeability tests exceeded 0.5 g.p.m.

Preliminary grouting of the boreholes was required to reduce rock permeability to specified levels and to minimize the possibility of ground water entering the boreholes after installation of anchor tendons. Grout consisted of Type II Portland Cement and water mixed at ratios as shown below. The grout was mixed in a high speed colloidal mixer and placed using a Moyno constant displacement pump. The mix was pumped into the borehole through a tremie tube running to the full depth until clean grout overtopped the hole at the surface. The tremie tube was then withdrawn and the grout allowed to set for a minimum of 24 hours. The boreholes were then redrilled using a 6-1/2 inch clearance rotary roller bit and the water tests repeated to ensure that the anchor holes were water-tight as specified. Grout mixes and quantities used were as follows:

T.A.1	Gravity Grouted	--	12 bags	6-1/2 gal/cu.ft.
T.A.2	" "	--	20 bags	" " "
T.A.3	" "	--	9 bags	" " "
T.A.4	" "	--	9 bags	" " "
T.A.5	Pressure Grouted	25 psi	8 bags	5 gals/bag
T.A.6	" "	25 psi	12 bags	" " "
T.A.7	" "	35 psi	13 bags	" " "
Grout Test Hole	Gravity Grouted	--	32-1/2 cu.ft. 6 cu.ft.(on the following day)	6-1/2 gal/cu.ft.

The grout test hole is included above as this required preliminary grouting due to high rock permeability and the necessity of preventing dilution or loss of the test grout. This procedure simulated the test anchor construction.

Tendon Installation

Tendons were delivered to the site ready assembled and stored in a heated warehouse until required for actual installation. At this time they were transported from warehouse to anchor location and unpacked ready for Quality Control inspection and documentation. Grout column extensometers were fitted at this stage and the telltale wires run to the anchor head. The 1/2" nominal I.D. 100 psi polypropylene grout tremie pipe was also positioned in the tendon at this time and checked for cleanliness and ease of subsequent removal.

Tendons were installed using the main reactor construction ringer crane which picked the tendons up at the head and then lowered them vertically into the boreholes. All tendons were placed with ease in spite of the close tolerances between tendon dimensions and borehole diameter. Each tendon was oriented correctly with respect to the grout extensometer wires and was allowed to hang in the borehole prior to grouting to ensure straightness of the strands or wires.

Tendon Grouting

Prior to the placing of the anchorage grout the borehole was filled and flushed with clean fresh water pumped through the tremie pipe in the tendon from the bottom of the hole. This ensured that the grout pipes were clean and unobstructed and that no foreign matter was present in the borehole. Upon completion of flushing the grout was then placed through the same tremie pipe. Grout was pumped to displace the water in the drilled hole until it overtopped the hole at the surface. Care was taken to see that clean and undiluted grout issued from the borehole before pumping was discontinued.

The grout level was then carefully monitored to see if any significant drop in level occurred or for other evidence of an unexpected anomalous grout condition such as rising air bubbles. In this circumstance additional grout would have been pumped into the borehole through the tremie pipe to ensure complete tendon encapsulation and a competent grout column. However, with all these anchors, when inspection revealed that no significant grout loss or other unusual condition was evident, the tremie pipe was withdrawn from the borehole.

Water testing and preliminary grouting of the boreholes virtually precludes the presence of untreated voids or fissures in the rock and the method outlined above is a valuable additional quality control procedure in safeguarding completely against the occurrence of anomalous conditions in the grout.

After allowing 20 to 30 minutes to elapse for grout level observation and to enable some set of the cement to take place the excess grout in each borehole above the top of the fixed anchor zone was washed out using a special flush pipe. This was 1" diameter steel with a closed lower end. Immediately above this cap, four vertical slots were cut approximately 2" long. The flushing water thus was directed into four horizontal pressure jets. In use the pipe was slowly lowered into the borehole and rotated. The water jets were able to wash the excess grout from around and between the tendon strands or wires to leave the borehole clean. The flush pipe was lowered to a predetermined level and water flushing continued until clean water overtopped the borehole at the surface. The end cap and horizontal jets prevented any disturbance of the grout below the bottom of the pipe and permitted a very accurate determination of the level of the top of the grouted fixed anchor zone.

The grouting technique employed for these anchors ensured complete embedment of the tendons, full compaction of the grout under hydrostatic head and accurate control of all stages of grout placement.

The level of the top of the grout was measured by probing after the grout had fully set. These measurements together with type of grout and water/cement ratios are shown in Table I. Grout cubes were taken of each grout mix and cube test results are included as Figs. 24 through 30.

VI INSTRUMENTATION

A comprehensive range of instrumentation was required to provide full information on the performance of the anchors. The instruments fell broadly into three groups which may be described as anchor instruments, rock instruments and other test instruments.

The anchor instrumentation consisted of electronic load cells, dial gauge movement indicators and grout column extensometers. For monitoring rock movements, grouted-in-place rock extensometers were installed and a comprehensive system of reference points were continually monitored for displacement during the test sequence by optical survey. The other instrumentation consisted of pressure gauges and flow meter associated with the rock permeability tests and pressure gauges used in grout column tests.

Anchor Instrumentation

Load cells were as manufactured by Remote Systems, Inc. and were of 500 kip capacity. The cells operated using bonded foil strain gauges and each cell contained two separate circuits--an active circuit for measuring load and a reference circuit. The active circuit consisted of eight strain gauges bonded at 45° intervals around the periphery of the cell body to form a Wheatstone Bridge with each leg of the bridge having two gauges 180° apart. This gauge arrangement automatically compensates for eccentric loading conditions and temperature changes. The reference circuit consisted of four strain gauges also forming a full Wheatstone Bridge. This circuit is isolated from the cell body so as to be free from strain effects due to loading or temperature changes, and provides a method of zeroing the strain meter to each individual load cell. The strain meter has also an inbuilt reference circuit which may be used to check instrument calibration.

The readings obtained from the strain meter are expressed in micro-inches per inch of the gauged cell area and, when multiplied by the given calibration factor, give the total load in pounds. This calibration factor is calculated from the readings obtained when tested in a laboratory load stand. The load stand had an accuracy of ±1% and the load cells had a sensitivity of approximately 50 lbs. to give a differentiation of 0.01% when at full load of 500 kips.

Hydraulic Jack Gauges

During testing all loads were set using the load cells but jack hydraulic pressures were recorded at the same time as a check should a malfunction of the cell occur. Hydraulic pressure gauges were marked in 100 psi intervals and were capable of being read to 50 psi.

Dial Gauges

The dial gauge movement indicators were Starrett #24-441J type having 4" travel and Starrett #24-4041J type having 6" travel. All indicators were constructed to read to 0.001" and were calibrated using gauge block sets traceable to the National Bureau of Standards. One 6" and one 4" travel gauge was used for each test positioned to determine the extension of the exact length of tendon under test. Readings from both gauges were recorded and averaged.

Grout Extensometers

Grout column extensometers each consisted of an anchor point made from 3/4" x 10" long rebar drilled and tapped at one end for special double tube fittings, and 0.047" stainless steel wire encased in 1/4" nylon tubing as the detector wire. At the surface a 3/64" flexible braided stainless steel wire cable was attached to the stiff detector wire. This was then passed round a pulley fixed to the underside of the jacking stool and then round a special 6" diameter pulley assembly which was graduated in 1/16" intervals enabling readings to an accuracy of 1/32" to be made. Movement of the wire and thus the grouted-in anchor point was detected by reading the graduated scale against a fixed pointer. The braided cable was passed completely round the special readout pulley to prevent slippage and tension was maintained in the system by means of a 9# weight on the end of the cable.

Anchors #1 through #5 were fitted with two grout column extensometers each and Anchors #6 and #7 with three. Anchor point levels were as shown in Table I. The results obtained during the tests provided a comprehensive record of grout column movements.

Rock Instrumentation

Rock extensometers were installed between Test Anchors #1 & #2 and #3 & #4. A 2-3/4" borehole was drilled and the anchor points set at 12'6" and 25' below bearing plate level and the borehole cement grouted to the ground surface. Each anchor point consisted of a 3/4" x 10" long rebar threaded one end for special pipe and tube couplings. Into the couplings were fitted the 1/4" stainless steel detector rod and the 1/2" polyethylene tube outer casing. At the surface a concrete block was poured on which was placed the 6" x 6" x 1/2" thick reference head plate. The plate was located on and elevation was altered by four 1/2" threaded studs set into the top of the concrete. Below the reference plate were two stainless steel tubes which passed over the 1/4" detector rods and acted as guides. The extensometers were read by use of a micrometer depth gauge bearing against the reference plate and with the probe inserted through a hole in the plate to touch the end of the detector rod within the guide tube (Fig. 35).

Surface Markers

Other monitoring of rock displacements was carried out by optical survey of various reference points in the area of the tests. During the various stressing sequences for Anchors #1 through #4 these were monitored continuously. The results obtained provided sufficient information to be able to dispense with this survey and other rock instrumentation for Anchors #5 through #7.

Other Instruments

Water pressure testing of boreholes was undertaken using a 1" Rockwell Manufacturing Company in-line flow meter to determine water take in any borehole. Calibration was by allowing water to flow through the meter into a collecting drum. The quantity so collected was then weighed on an accurate scale. Pressure was read on Weksler pressure gauges Type No. AA14P having an accuracy of ±0.5% of the full scale reading of the gauge. All gauges were tested for accuracy using a dead weight tester, the calibration of which was traceable to the National Bureau of Standards.

Smaller pressure gauges of a similar type were used in the grout column test to monitor fluid grout pressures at two points in a 45 ft. column. A cement grout containing an expanding agent was used for this test to check the effects of unrestrained expansion. The plots of pressure against time are appended.

VII ANCHOR TESTING

Stressing Equipment

To carry out the testing sequences specified, the manufacturer of each type of tendon provided hydraulic jacking equipment and accessories to apply the stress to the tendons. The jacks and other equipment were designed to be entirely compatible with the tendons and are part of the standard equipment and procedures used by the manufacturers for their normal stressing operations.

The V.S.L. stress jack was a Templeton-Kenley type of 500 ton capacity having a 7-1/4" I.D. center hole and ram stroke of 12". Strands were gripped by V.S.L. two part wedges in a standard E5-19 head. The jack was located on top of a special stressing stool standing on the anchor bearing plate. This stool was needed to give access to the grout column extensometer wires and is shown in the drawing of the V.S.L. tendon test arrangement, Fig. 33 An electronic load cell was placed on top of the jack but beneath the stressing head so that continuous monitoring of load was possible. A separate stress head was situated under the stressing stool immediately above the bearing plate which was used to lock off the load in the anchor during the seven day hold period.

For the multi-wire tendons the standard Prescon arrangement was used with a stool on top of the bearing plate supporting a Proceq 275 ton capacity hydraulic jack. The jack had a hollow ram with center hole of 3-3/8" diameter and a stroke of 4" (100 centimeters). Stress was applied to the anchor head by a 2-3/4" diameter pull bar which screwed into an internal thread in the head. The other end of this bar was also threaded and a circular stressing head used to transmit the jack force to the pull bar. Between this head and the jack ram was fitted an electronic load cell to enable loads to be monitored at all times the jack was in place. The anchors could be locked off and the jack reset or removed at any time by placing shims between the anchor head and the bearing plate. The stressing stool was a standard stressing item but was also used to give access to the grout column extensometer wires. A drawing of the Prescon type tendon test set up is also included as Fig. 34.

For the V.S.L. type of stressing equipment the hydraulic pump was gasoline powered and pressures were determined by a 6" diameter pressure gauge which had been calibrated with the ram. The Prescon hydraulic pump was electric but pressures were determined with a similarly calibrated gauge. In both types of equipment sufficient non-return valves were incorporated into the hydraulic lines to virtually eliminate loss of pressure due to internal bleeding and to permit maintenance of load for periods of time after removal of the hydraulic pump.

Stressing Sequences

Anchor testing was carried out in compliance with the following specification sequence with modifications as noted later. All load changes were in 50 kip increments and decrements.

1) Load to 400 kips and then reduce to 350 kips and lock-off.

2) Check load after 24 hrs. If prestress loss is less than 25 kips, restore load to 350 kips and proceed with Step 5. If loss exceeds 25 kips, restore load to 350 kips and hold for a further 24 hours.

3) Repeat Step 2. If loss of prestress load again exceeds 25 kips, then the anchor shall be rechecked at 24 hour intervals at the direction of the geotechnical representative to assess whether the loss of prestress is likely to continue.

4) If the anchor shows a continuous loss of prestress, the tendon shall be allowed to relax and will be monitored continuously at the direction of the geotechnical representative until a constant load is obtained.

5) Carry out cyclic loadings to 100 kips, 150 kips, 200 kips, 250 kips and 300 kips reducing the load to 50 kips between each load level.

6) Load to 400 kips and then reduce to 350 kips and lock-off.

7) Repeat Steps 2 and 3 or 4. If the loss of prestress is less than 25 kips, proceed with Step 8.

8) Perform 10 cycles of loading from 350 kips to 400 kips and back to 350 kips. Lock-off at 350 kips.

9) Perform lift-off load check seven days after the lock-off in Step 8.

10) Load to 500 kips and hold for 10 minutes. Then reduce load to zero.

Prior to start of stressing, Step 10 was modified to read:

> Reduce load to 300 kips. Then perform cyclic loadings to 350 kips, 400 kips, 450 kips, and 500 kips reducing load to 300 kips between each stage. Hold load of 500 kips for 10 minutes then reduce to zero.

For Test Anchors #5, #6 and #7, Step 8 was modified to read:

> Reduce load to 300 kips. Perform 25 cycles of loading from 300 kips to 400 kips and back to 300 kips. Then load to 350 kips and lock-off.

In Test Anchor #7 the lock-off in Step 8 and the whole of Step 9 was omitted.

For all stressing sequences loads were set using the load cell equipment. The hydraulic gauges were monitored constantly, however, as a precaution against malfunction of the electronic equipment. These cells were sensitive to load changes as small as 50 lbs. over the complete range of 0-500 kips. This represented a sensitivity of 0.1% of the specified load increment or decrement of 50 kips.

Tendon extensions were measured at each anchor head using dial gauges fastened to an independent reference platform. One 4" travel and one 6" travel gauge was used in each test, the gauges showing movement to 0.001".

Full records of all stressing were kept and included time, load, tendon extension and temperature. Readings were taken before and after any change in load to establish whether any movement had occurred at any load stage. The readings have been plotted in graph form and are included in the appendices.

Anchor Test Results

Anchor No. 1

This anchor was the first of the multi-wire installations to be tested and the results obtained demonstrated that the testing arrangements and instruments were working effectively and required no modification.

The seven day hold of the 350 kip test load showed that the anchor load had increased by 4.9 kips at the end of seven days from the locked off load level. Three days later the load increase was 3.2 kips above the original lock-off load.

The residual deflections recorded at the end of the testing sequences are not explainable in any detail but are the result of the sum of the movements due to plastic yield of the steel, short term creep of the steel during testing, residual stress retained by the steel within the grout column, and unrecovered upwards movements of the grout column. The load-extension graph is Fig. 36.

A load-time-extension graph is appended as Fig. 37 This is representative of the results of the other multi-wire anchors tested so no other graphs of this nature have been prepared.

No movements of the rock extensometers were noted during the testing of this anchor.

Anchor No. 2

This anchor was the second of the multi-strand tendons to be tested. Therefore, particular attention was able to be paid to the behaviour of this anchor at the load levels at which unexpected events occurred during the testing of the first.

During cycle 8 of the stressing sequence one individual wire of a seven wire strand was heard to part as the load was being increased from 350 kips to 400 kips. Examination of the tendon at the time of the occurrence did not reveal any indications of wire breakage so the test was continued. An examination of the tendon after completion of the test and dismantling of the stressing equipment showed that the wire had come into contact with a rough edge of the center hole in the stressing plate.

Load in the anchor during the seven day hold period dropped by 44 kips. No positive reason was established for this large loss other than the fact that some deformation of the horseshoe destressing shims was discovered after completion of the hold and lift-off cycle. This appeared to be due to some slippage in their position at the time of lock-off.

Cyclic loading of the anchor to 500 kips was attempted but excessive deflection was recorded at the 400 kip and 450 kip load levels indicating yield in the steel. Load was increased with care above this level in view of the experience with the first multi-strand anchor test. At a load of 471 kips wires in the strands were heard snapping and jacking was discontinued immediately. The load was reduced by stages to zero and the tendon recovery measured.

Grout extensometer movements are shown in Figures 41 and 42 and it can be seen that these movements started with the first load applied and continued virtually throughout all test sequences with the upper extensometer at -30', and up to a load of 400 kips with the lower extensometer at -40'.

The movements of the extensometers above the 350 kip load level as shown on Fig. 42 may or may not have been affected by the possible yielding of the strands as discussed below.

As with the first multi-strand anchor tested, Anchor No. 4, the test sequence up to Step 10 had been successfully completed when individual wires in the tendon strands again started to snap. On this occasion at a load of between 461 kips and 471 kips, the loading was discontinued and the load reduced progressively to zero and tendon recovery noted. After dismantling the test equipment, examination showed that with one exception the wires had broken level with the bottom of the grip wedges. The exception was the one wire that had come into contact with the edge of the bearing plate.

Following the similar events that occurred during the testing of the first strand anchor, a theory was proposed that the resetting of the grip wedges in the same position on a strand contributed to or caused failure of the strands at a significantly lower ultimate strength than stated by the manufacturer. Consequently, the strands for Anchor No. 2 were gripped at a lower level for Step 10 of the stressing cycle than for the previous steps. The test results show no change in behavioural pattern so indicating that the failures were due to other mechanisms. The manufacturers experience over the years together with our own observations confirms that, almost invariably, strand breakages occur at the bottom of the grip wedges.

To further explore these mechanisms and theories Anchor No. 4 was stressed to destruction after the completion of the testing of Anchor No. 2.

A load-time-extension graph for this anchor is appended as Fig. 40. This is also representative of the results of the other multi-strand anchor tested so no other graphs of this nature have been prepared.

There were no movements of the rock extensometers during the testing of this anchor.

Anchor No. 3

The results obtained from this test indicated a performance very similar to that of Anchor No. 1.

During the seven day hold period a slight increase in load in the tendon, 4-1/2 kips, was recorded.

At a load of 450 kips the movement in excess of the theoretical for the full tendon length was 1/4 inch. The load was not stable and had to be restored twice due to losses. Also audible sounds came from the tendon indicating some movement taking place. Plastic yield of the steel could not be the cause of the excess movement as the mill test report for the tendon wires showed that 460 kips would be the lowest load at which yield could be expected. The load-extension graph for this anchor is Fig. 43.

The readings of the grout column extensometers showed some minor movements at the start of testing with very little change thereafter.

Rock extensometer movements during the testing of this anchor are shown in Fig. 45 and shows that some small changes did occur during the test sequences. This indicates that some rock consolidation did take place though of a less magnitude than that recorded during the testing of Anchor No. 4, the first anchor tested on the site. A small residual movement was noted.

On completion of the normal test loading sequences on Anchor No. 3, a destruction test was attempted using the hydraulic gauge to set the load and measuring ram extension to determine strain. The load-extension graph for this test is shown on Fig. 43. The tendon did not fail but was yielded plastically and the test was discontinued at a load of 522 kips when the jack stool started to deform.

Anchor No. 4

This was the first anchor to be tested on the site and the first test sequences were carried out in severe weather conditions. This did not affect the load-extension records obtained but made the operation of the stressing equipment very difficult due to problems caused by freezing. One effect of the cold conditions was to cause a brittle failure of the tack welds holding part of the grout extensometer equipment in place. This is discussed more fully later but it was possible to obtain some meaningful results before the instruments went out of calibration.

The anchor stress/strain curve shows an immediate deviation from the theoretical line representing the extension of the 25 ft. free length at the start of the test sequence, then exhibiting linear behaviour to a load of 250 kips before showing additional deflections to cause nonlinearity. This is shown in Fig. 46.

The lift-off check at the end of the long term hold period, which for this anchor was twelve days, showed the anchor to have lost 1.8 kips in stress over the period.

As Step 10 of the test sequence was being carried out wires in the tendons started to snap at a load of between 435 kips and 445 kips, the tendon yielding and elongating too swiftly for the dial gauges to be reset and measurements taken. Consequently the load was reduced to zero and the equipment dismantled to permit inspection of the tendon.

It was found that one wire per strand in eight of the thirteen strands had snapped level with the bottom of the grip wedges. All the broken wires were on the same side of each strand and the eight strands were all on one side of the tendon.

A theory was proposed that the cyclic loadings together with the resetting of the grip wedges at the same position had contributed to or caused failure of the strands at approximately 83% of the manufacturer's guaranteed ultimate tensile strength. To test this theory, a final anchor stressing was carried out. In this the jack was replaced over the tendon and the pull head and grip wedges reset but at a lower level on the strand to avoid gripping the strand in the same place twice. The anchor was then restressed in 50 kip increments using the hydraulic gauge to set the load and measuring ram extension to determine strain. Wires in the strands commenced to break at a load of 470 kips. With progressive failure the load level dropped and ram extension increased. After all the strands had been failed, inspection revealed that the wires had all broken level with the bottom of the grip wedges. This confirmed that the position of the grip wedges had a minimal reducing effect upon tendon strength. Also, only two of the thirteen cables showed that the center straight wire had been broken with the other six wires of the strand. Eleven strands showed that the center wire was still intact and, therefore, had not been subject to the same stress level as the other wires forming the cable. The stress/strain graph for this anchor destruction test is appended as Fig. 47.

As a result of these tests, and from other information and observations that have been made at prior times, it is suggested that the reason for failure of the strand tendons at a load level significantly below stated ultimate strength is due to the fact that in ground anchors, the cables are held with barrels and wedges at one end only as opposed to each end in normal post-tensioning applications. The center wire in each cable is straight and load is transferred to it from the surrounding six wires through friction and the clamping force of the grip wedges. Any loss of friction or drop in clamping force will allow the center wire to slip and, therefore, to carry less load than the others forming the strand. Total slippage would mean a drop in strand ultimate strength to approximately 85% of the full strength. The figure compares very closely with the ultimate anchor capacities determined during these tests.

Similarly by taking 6/7 of the yield strength of the cables as taken from the test certificate then some plastic yield could be expected above a load of 390 kips approximately. Examination of the load-extension graph shows increasing deflections of the tendon above this load which could be due to the cause outlined.

This problem would seem to be unique to ground anchor usage and to require further investigation. Again, the effect of Poisson's Ratio would work against locking of the center wire when stress is applied after the grout is cured. In normal post-tensioning applications the cables are gripped both ends and stressed before the grout is placed, the effect of Poisson's Ratio then preventing slippage or destressing due to the constraining effect of the cured grout.

The grout extensometer at -30' showed movement right from the commencement of test loading, the large prime movements gradually becoming less until at 350 kips at the start of the first 24 hour hold period movement of the tell-tale had ceased after indicating a grout column displacement of 1". This is shown in Fig. 48.

The lower extensometer at -40' also indicated grout column movements but these were much slower, of much smaller magnitude and were recorded as totalling 1/4" at the start of the 24 hour hold.

During the off-loading after this hold period very severe weather occurred and the pulleys holding the extensometer read-out wires became dislodged from their usual alignment due to brittle failure of the welds holding them in position. After being refixed in their proper position, the extensometer read-outs were rezeroed and later readings were added to the readings obtained before the weld failure to assess grout column behaviour.

Thereafter during the balance of the test sequences the extensometers showed very small movements and virtually static conditions for the majority of load levels. The maximum total change for the upper extensometer after resetting the pulleys was 3/32 inch and for the lower 1/16 inch.

Fig. 49 shows the recorded movements of the rock extensometer. These show some consolidation taking place during the total test but of significance is the movement recorded at the 350 kip load level during the seven day hold period. This is directly attributable to the fact that the adjacent Anchor No. 3 was being test loaded during this period.

Anchor No. 5

Construction of this anchor commenced upon conclusion of the testing of Anchors #1 through #4. These anchors had been grouted using MasterFlow 814 grout mixed with two gallons of water to the bag at the direction of the grout manufacturer's representative and concern was expressed at the apparent low bond strengths achieved. Anchor No. 5 was grouted therefore with MasterFlow 814 grout but using 2.4 gallons of water to the bag which was the mix as originally specified as shown in the manufacturer's literature.

During the seven day hold period which was extended to 24 days for administrative reasons, the anchor lost a total load of 10 kips.

The grout extensometer readings for the upper tell-tale at -30' are included as Fig. 51. The lower extensometer is not shown as the maximum movement recorded was 3/32 inch.

During Cycle 1 of the test sequence fairly small movements of the upper extensometer were noted up to a load of 250 kips. At this point a loud crack was heard in the borehole and the detector wire jumped 1/8 inch. Thereafter movement increased in magnitude at each load application.

Anchor No. 6

Following completion of testing Anchors #1 through #5 and an assessment of the results, a decision was made to construct Anchor No. 6 using a multi-wire tendon but with grout changed to Type II Portland Cement obtained locally and as used in the construction batching plant, and containing a grout additive, Sika 'Intraplast N'. This is an expansive agent and the expansion is derived from the release of hydrogen gas formed by chemical reaction between the aluminum powder in the additive and the alkalis formed by hydration of the cement.

Due to an error at the time of manufacture, the tendon for Anchor No. 6 was found to have been assembled with a 6" diameter anchor and keeper plate at the bottom instead of the 5-1/4" diameter plates as fitted to previous anchor tendons. To avoid delays to the test program it was decided to use the tendon as delivered to site and then to make the tendon for Anchor No. 7 to similar dimensions. This would allow direct comparison of results to be made.

The load-extension curves obtained during testing show a pattern of behaviour very similar to the other multi-wire tendons tested. These are shown in Fig. 53.

During the seven day hold period which was extended to ten days, the stress in the anchor increased by 10 kips.

Final testing to 500 kips was accomplished successfully but was accompanied by some yield in the steel as was to be expected.

For this anchor, modifications to the bearing plate were made which permitted the fitting of an extra grout extensometer point within the borehole. The extensometers were, therefore, at -30', -40' and -50'9" levels and the recorded movements are shown in Fig. 54.

Anchor No. 7

During the assembly of the test apparatus for this anchor, it was found that the pull bar was very difficult to enter into the threaded anchor head. Visual inspection indicated some irregularities in the internal thread, however, sufficient thread was engaged eventually that was judged adequate to allow the first few cycles of test loading to proceed.

Step 1 was completed successfully as were Steps 2 through 5 and the load-extension graph is appended as Fig. 55. The graph shows behaviour very similar to the results of the other multi-wire anchors.

However, in increasing the load to 400 kips in Step 6, the thread in the anchor head failed thus causing a sudden destress of the anchor with attendant damage to the test setup and equipment and to the anchor head and tendon wires.

The damage to the test equipment was mainly caused by the pulling bar being catapulted through the jack stool and hollow jack and load cell thus dislodging the jack and cell and dial gauge stand and bracket. The wire connecting the read-out box to the load cell was parted. The sudden release of load caused the stress head to impact onto the lock-off shims left in place on the bearing plate and, not falling exactly square, hit the edge of one and projected the other sideways from beneath the stressing stool some distance. In impacting on the edge of one shim, one edge of the stress head was badly deformed and several wires bent or broken. These are noted on Fig. 56 attached. The extensometer wires were also broken.

Following a full inspection of the anchor head by all parties including the manufacturer it was decided to replace the damaged head with the head from Test Anchor #5. To enable this to be done a pocket had to be cut in the concrete test block and the 8" diameter steel pipe removed for a depth of 15". The old head was then driven down to permit the removal of the button heads on each wire with the aid of a set of hand operated metal shears. The old head was then removed from the tendon.

Using a single strand jack and 1/4" barrel and wedge to grip the wires, each wire was test loaded to 9 kips (76% of G.U.T.S.) to ensure that no damage had occurred to the tendon below the head and that the total tendon was competent and, therefore, able to be tested in accordance with the specification. These tests showed all wires to be sound and no anomalous conditions were recorded. Stress/strain graphs for four wires are shown in Fig. 57.

The head from Test Anchor #5 was then placed on the tendon wires and the wires all shortened by an equal amount to enable the three wires found broken below the old head to be fitted into the replacement head. Following shortening, the wires were then rebuttoned using a Proseq hydraulic field button-header. A miscalculation during the shortening of the wires left one too short to permit a button-head to be formed. Therefore, the anchor test proceeded with 45 wires of the 46 under stress and a total length of 54'6-1/4".

Following completion of the installation of the replacement head, the 8" I.D. steel pipe was replaced, the pocket in the concrete test block restored with 6000 psi concrete and the bearing plate set and levelled ready to receive the test equipment. The grout column extensometer wires were also repaired and checked for continuity.

After the concrete had been allowed to cure, the test equipment was reassembled and a new test carried out. This was for the total test sequences as specified with the exception of the lock-off in Step 8 and the hold period in Step 9. The anchor test was completed satisfactorily and the graphs obtained after the replacement head was fitted are included in the appendix as Fig. 58.

The second test on Anchor No. 7 showed graphs of stress versus strain that were to a large extent repeats of the results obtained from previous multi-wire tests.

During the test the unbutton-headed wire was visible and it seemed to be moving at the same rate as the other tendon wires during the stressing sequences. This indicated that it had become trapped in the tendon bundle and was, therefore, carrying some load. However, nothing in the load/extension curves can be seen to confirm or deny this. Differential movement of this wire was eventually noted as the anchor load was increased above 400 kips.

No seven day hold was carried out on this anchor and Step 10 followed immediately after Step 8 cyclic loadings. Jacking was discontinued at a high load of 459 kips as excessive movement was recorded due to plastic yield of the steel and the resulting proximity of the dial gauge reference plate to the underside of the jacking stool.

The experience gained with the repair and retesting of this anchor, whilst in no way premeditated, gave a valuable and convincing demonstration that, even after unexpected and potentially serious damage, the anchor could be restored in the field and proved to be of sound and viable construction and performance.

Three grout extensometers were fitted to this anchor at the -30', -40' and -50' levels. The two lower ones showed virtually no movement during the first test on this Anchor No. 7 but the upper detector exhibited large movements from commencement of the test. The readings, however, became erratic and an inspection revealed that the plastic sheath to the detector wire was not moving freely through the stress head thus giving false readings. In spite of the very confined conditions, it was found possible to cut the excess plastic tubing off the wire although a slight bend in the detector wire was unavoidably a result. Thereafter the upper extensometer registered no significant movements to indicate that the previous movements were probably erroneous and could be disregarded. Fig. 59 shows grout-extensometer movements.

The damage to the anchor due to the sudden destressing included the severing of the extensometer detector wires. They were reconnected after the anchor was repaired and operated satisfactorily. One point that should be emphasized here is that it was known that the detector wires did not stay in their former alignments in the tendon due to the disturbance of the wires that took place of necessity during the checking and repair of the anchor. Therefore, the very minor movements of the extensometers recorded during the second test on Anchor No. 7 are almost certainly associated with the alignment of the detector wires and not with any grout column movements.

Grout Test Hole No. 1

One additional borehole to the four test anchors was originally called for to enable a grouting test to be carried out. The hole was drilled to a depth of 46', water pressure tested, preliminary grouted and redrilled as the anchor boreholes. The grout test hole was then grouted using the stated technique of tremie tube to the bottom. Following withdrawal of the tremie pipe and after allowing the grout level to stabilize, the top 18'9" of grout was flushed out using the method previously described.

After allowing the grout to set, the column was cored with the diamond drill at specific intervals of depth and time. Sections of the cores so obtained were prepared and crushed in the laboratory in order to establish the strength-depth-time relationships of the in-situ grout as well as to verify that the construction procedures used were satisfactory. The record of grout cylinder strengths is appended as Fig. 32.

During the coring of the grout in this borehole, it was found that the core drill had been set up at a slightly different angle to the vertical than had the track drill.

Consequently the lower run of core extracted was found to have intersected the borehole wall thus provided a section of grout/rock interface for examination. Upon opening the split tube inner barrel to remove the sample, it was found that the rock section had not adhered to the grout but had fallen away.

No particular significance was attached to this at the time as testing of the first anchors had only just begun and no results were available for study. However, it became clear as testing progressed on the anchors that a completely unexpected problem was arising due to very poor bond between grout and rock. Nicholson Anchorage Company felt that there was, therefore, a need to investigate this problem further.

It should be noted that a full core of the grout column was obtained as specified for the length referred to above by a realignment of the drill rig.

Grout Test Holes #2 & #3

Following analysis of the results of the first four anchor tests, it was decided to compare the bonding characteristics of the MasterFlow 814 grout to Portland Type II cement without additive. Two more boreholes were, therefore, drilled to a depth of approximately 21 ft. and one was then tremie grouted with Type II cement mixed at five gallons of water per 94# bag of cement, and the other was grouted with MasterFlow 814 mixed at 2.4 gallons of water per 55# bag of cement. After allowing the grout to attain a minimum strength of 3000 psi, each hole was cored in such a manner that the grout/rock interface was recovered. In spite of the fact that the MasterFlow 814 grout obtained higher strengths earlier than the Portland Type II grout, it was found, from visual examination of the cores as the split inner barrel was opened, that the latter bonded to the rock significantly better than the former which, for a large part, was found to have fallen away from the rock.

Some difficulties were experienced with maintaining alignment of the diamond drill to obtain a continuous core of the rock/grout interface in Grout Test Hole No. 3; hence, several attempts were made. These are recorded and shown in Figs. #61 and #62. Fig. 60 described the cores obtained from Grout Test Hole No. 2 which was grouted with MasterFlow 814.

An extension of these tests was carried out in the Nicholson Anchorage Company shop in order to check whether grout shrinkage could be responsible for the low bond strengths. Two 5" I.D. steel pipes x 4 ft. long were set vertically and one filled and one half-filled by the tremie method with MasterFlow 814 grout mixed as specified. A third pipe was set up and tremie filled with an ordinary Portland Cement Type I grout with no additives and mixed at 5 gallons of water to the 94# bag. This was used as a control. All specimens had a single cable strand grouted into the center of the column within the pipe.

After a seven day curing period, it was noted that the MasterFlow 814 grout had shrunk away from the steel pipe, and feeler gauge measurements showed this reduction in diameter to be 1/32" at each end of the 4 ft. grouted pipe and at the accessible end of 2 ft. grouted pipe. Also water flowed freely through the pipe between grout and steel when the upper end was filled. No such shrinkage was detected with the Portland Cement grout nor was water able to flow through the pipe.

These tests were not witnessed by other than Nicholson Anchorage Company personnel.

The bonding characteristics were also checked by jacking with the single strand that was cast into the grout column. With the MasterFlow 814 grout the column moved within the steel pipe at the nominal jacking force whereas the cable in the Portland Cement was stressed to 80% of ultimate strength without causing any detectable grout column displacement.

Grout Encapsulation Tests

Two tendon encapsulation tests were also carried out to provide evidence that the grouting techniques used were sound and that no voids or weak areas in the grout column were formed. The tests were carried out by erecting two 6" diameter plastic pipes x 45 ft. long alongside the scaffold ladder tower into Reactor 4 excavation. V.S.L. multi-strand tendons were lowered into each pipe complete with integral grout tremie tube and the pipes were then filled with water. Grout was then pumped into each pipe through the tremie tube until it overtopped the top of the plastic pipe. After the grout had set the pipes were lifted out of the Reactor excavation and the lower portion cut up into sections using a rotary disc cutter. The plastic pipe was then cut off and the sections were examined.

One encapsulation test was carried out using MasterFlow 814 grout mixed at 2.4 gallons of water per 55# bag and the other with Portland Type II cement mixed at 5 gallons of water per 94# bag with no additives.

Examination of the sections of the encapsulated tendon showed that the Type II cement had bonded better to the steel cable tendons. The estimation of bonding was made from observation of the resistance of the encapsulated tendon to bending when being laid down for examination and from the ease with which sections of the grout could be lifted off the steel tendons.

The tendons were found to be generally centered well within the plastic pipe and the grout contained no significant voids or questionable areas with one exception. This was right at the bottom of the encapsulation test pipe filled with Portland Type II cement grout, and was due to a minor leakage of grout through an imperfect joint in the pipe end cap. The resulting void was about two inches long and one inch deep.

Grout Column Test

A grout column test was also carried out using the same technique of standing a pipe alongside the scaffold tower to simulate a borehole. In this test, a 4" pipe was used which was 42' long and fitted with two pressure gauges. One was centered 2'4" from the bottom and the other was 6" - 8" from the bottom. The pipe was attached to the same scaffold tower as was used for the encapsulation tests.

The 4" pipe was first filled with water to check for leaks in the joints and then filled with grout using the tremie pipe method. Grout mix was Portland Type II cement mixed at 5 gallons water per bag and containing Intraplast 'N' expansive admixture at the rate of 0.49# per bag.

After placing the grout, the pressure gauges were monitored continuously until the grout column had stabilized and any reaction or expansion due to the admixture had ceased. Pressures were recorded at frequent intervals and it was found that the grout did not become stable until 6-1/2 hours after placement. The plots of pressures versus time together with notes on grout behaviour are appended as Figs. 63 and 64.

After the grout had set, the 4" pipe was cut into sections and lifted out of the reactor excavation. From these sections cylinders were prepared for crushing to establish grout strengths and the relationships of strength to depth. These results are also appended as Fig. 31B. It should be noted that the top section of the grout column was found to be extremely soft and moldable in the hand even three weeks after pouring and the crush tests gave very low strengths.

VIII GENERAL COMMENTS

From the results of these tests, and from other information and observations that have been made it is suggested that the reason for failure of the V.S.L. tendons at a load level significantly below ultimate strand strength is due to the fact that, in ground anchor configurations, the cables are only held with barrels and wedges at one end. This has been discussed previously. It is therefore recommended that design parameters used in the future to calculate strand requirements should be altered to reflect the probability that the center wire will not be mobilized. Simply stated this will mean derating the guaranteed ultimate strength of the strand to 6/7 or 85% of the normal and adding, therefore, extra strands to a tendon to make up the required load. Alternatively the manufactureres could be requested to devise a tendon configuration that would be certain of developing the guaranteed ultimate tensile strength of the strand.

For the multi-wire tendon system it is recommended that the corrosion protection system be modified. At present each wire is greased and then encased in a plastic tube. However, during stressing the wires extend to leave a gap between the top of the plastic tubing and the anchor head. The plastic tubing is embedded in the top of the grout column and, being a slide fit over the wires, does not extend with the tendon. The gap formed exposes the tendon wires and some rust staining was noted in earlier tests. Anchor #7 had a modified plastic tube system which provided an oversize tube immediately below the anchor head. The regular plastic tube was fitted into this to provide a slip joint. However, in practice the epoxy cement holding the larger tube into the recesses provided in the head proved to be inadequate in strength and the plastic tubing again was found not to extend with the tendon. This problem is not expected to be difficult to solve but some more development work is needed to produce a satisfactory corrosion protection system.

All other techniques used in the test anchor program for drilling, placing, grouting and stressing worked well and gave no cause for revision of methods. The use of these techniques for construction of permanent anchors may, therefore, be safely recommended.

It must be emphasized that, apart from the unexpected behaviour of the MasterFlow 814 grout and the unique failure of the multi-strand tendons at loads in excess of 400 kips, all anchors proved to be dependable and stable even under the most rigorous and severe testing conditions.

Respectfully submitted,

NICHOLSON ANCHORAGE COMPANY

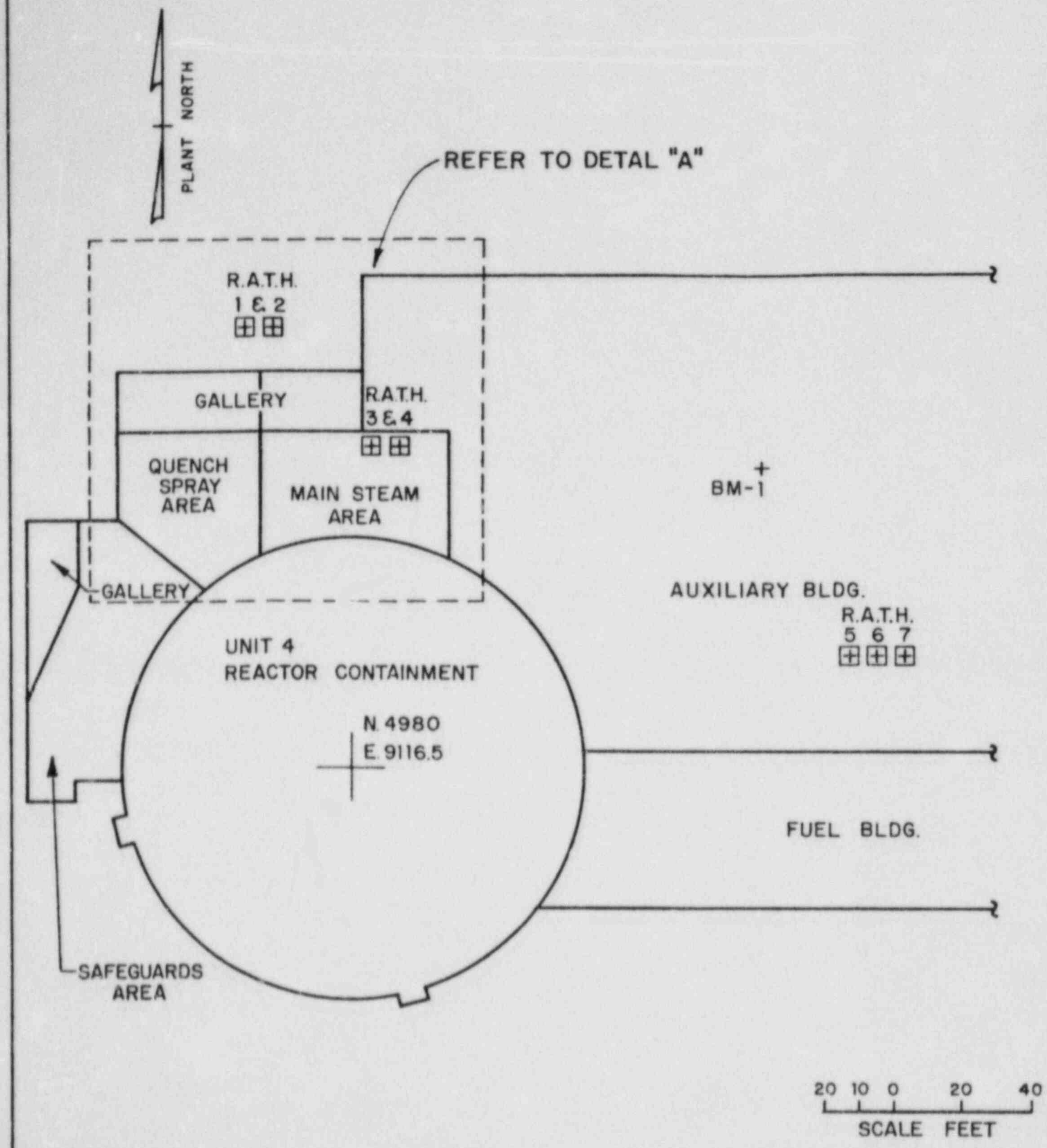
Wycliffe Jones

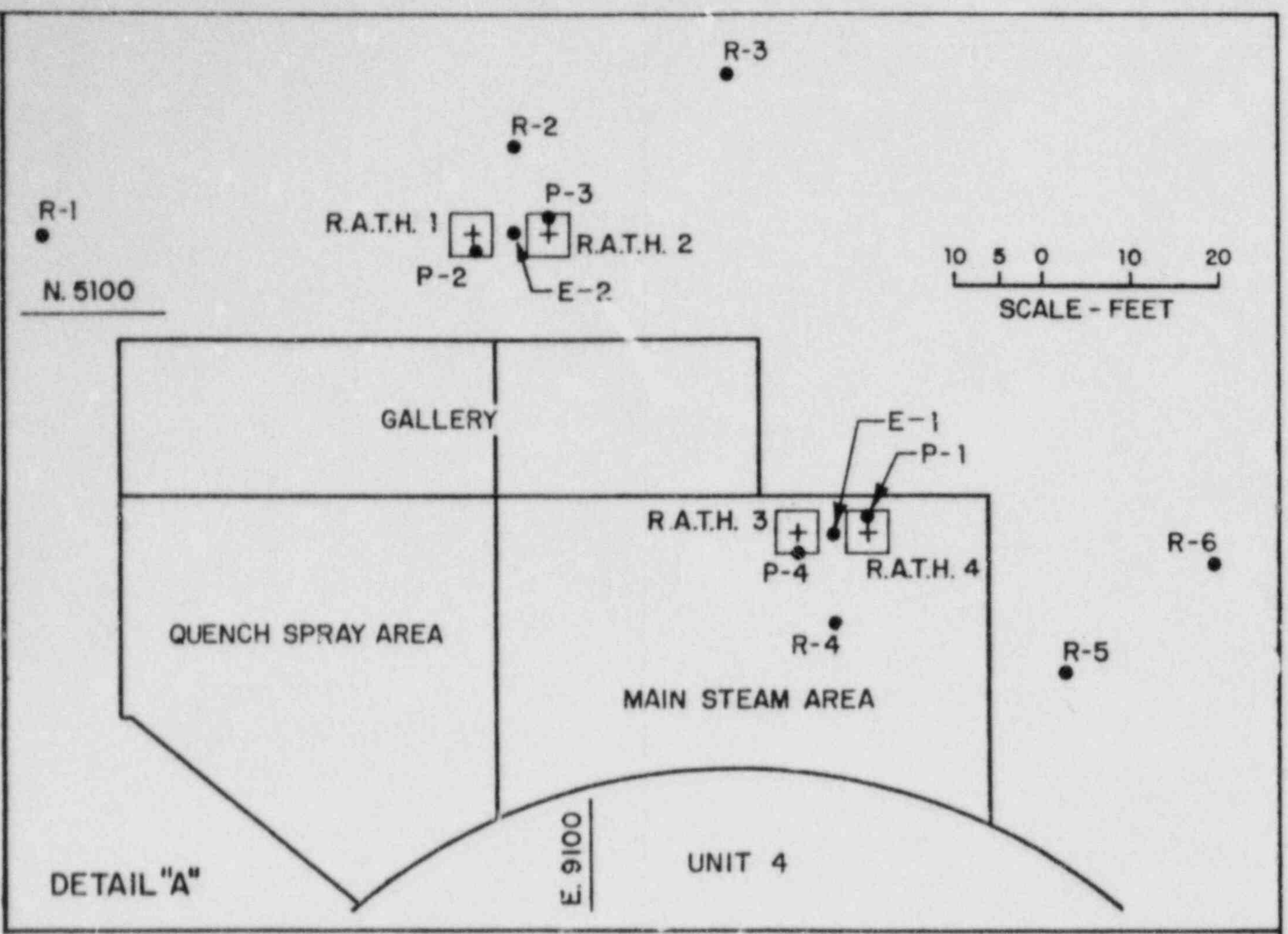
P. T. Wycliffe-Jones
Technical Consultant

PJ Nicholson

Peter J. Nicholson
President

January 12, 1977


PLANT NORTH
REFER TO DETAL "A"
R.A.T.H.
1 & 2
GALLERY
R.A.T.H.
3 & 4
QUENCH
SPRAY
AREA
MAIN STEAM
AREA
BM-1
GALLERY
AUXILIARY BLDG.
R.A.T.H.
5 6 7
UNIT 4
REACTOR CONTAINMENT
N. 4980
E. 9116.5
FUEL BLDG.
SAFEGUARDS
AREA
20 10 0 20 40
SCALE FEET



COORDINATES

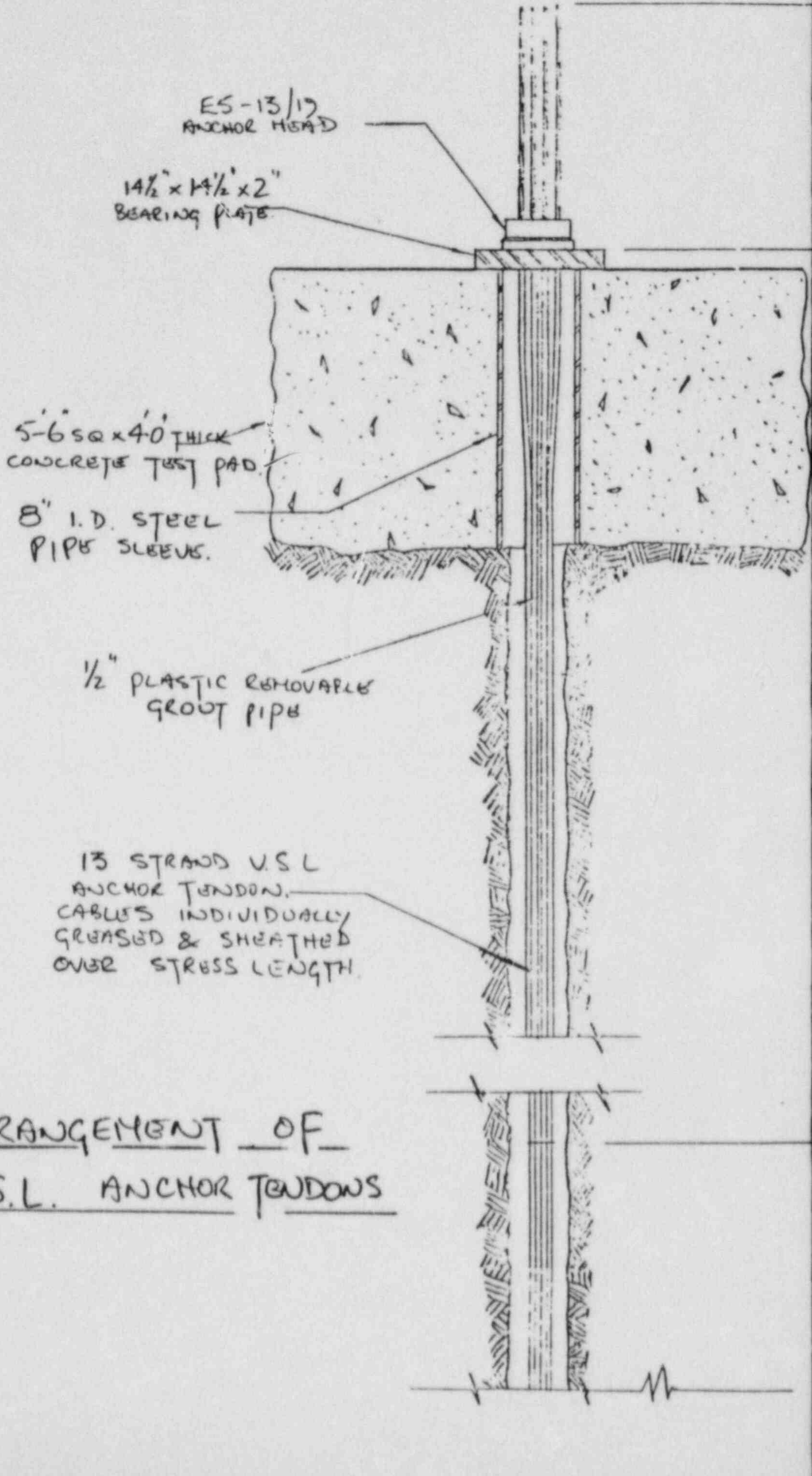
R.A.T.H. 1	N.5109	R - 1	N.5109	E - 1	N.5075
	E.9086.5		E.9037.5		E.9122.5
R.A.T.H. 2	N.5109	R - 2	N.5118.9	E - 2	N.5109
	E.9094.5		E.9091.3		E 9090.5
R.A.T.H. 3	N.5075	R - 3	N.5127.2	P - 1	N.5077
	E.9123.5		E.9114.4		E.9131.5
R.A.T.H. 4	N.5075	R - 4	N.5064.9	P - 2	N.5107
	E.9131.5		E.9126.9		E.9086.5
R.A.T.H. 5	N.5013	R - 5	N.5059.56	P - 3	N.5111
	E.9250		E.9152.9		E.9094.5
R.A.T.H. 6	N.5013	R - 6	N.5071.7	P - 4	N.5073
	E.9258		E.9170.5		E.9123.5
R.A.T.H. 7	N.5013	BM-1	N.5069.5		
	E.9266		E.9238.11		

NOTE: R-1 COORDINATE APPROXIMATE

LEGEND:
R = REBAR
BM = BENCH MARK
E = EXTENSOMETER
P = TEST PAD POINT

FIG.
ROCK ANCHOR TEST PROGRAM
ANCHOR & SURVEY
MONUMENT LOCATIONS
NORTH ANNA POWER STATION
UNITS 3 & 4

Fig. 1



ARRANGEMENT OF V.S.L. ANCHOR TENDONS

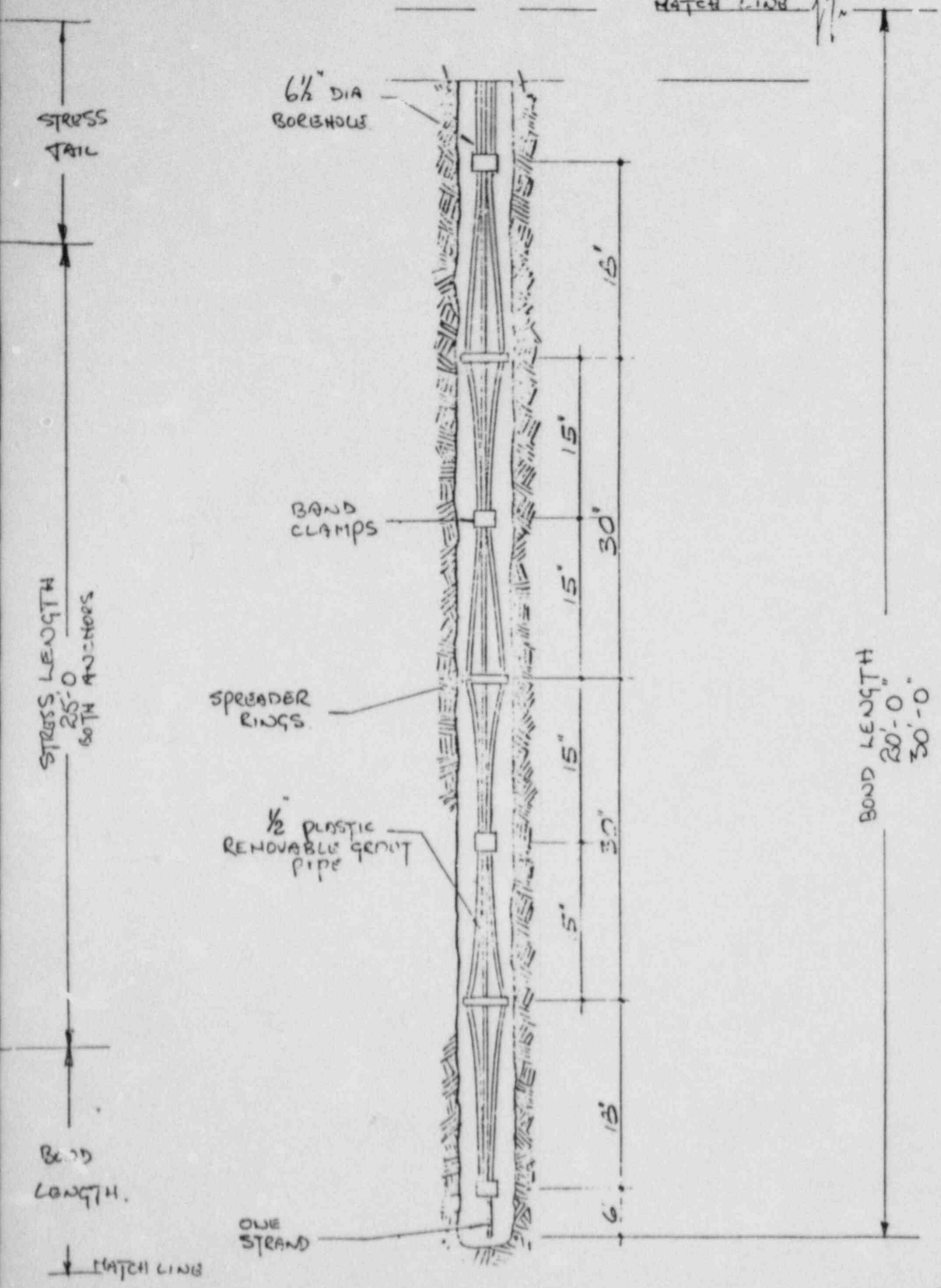
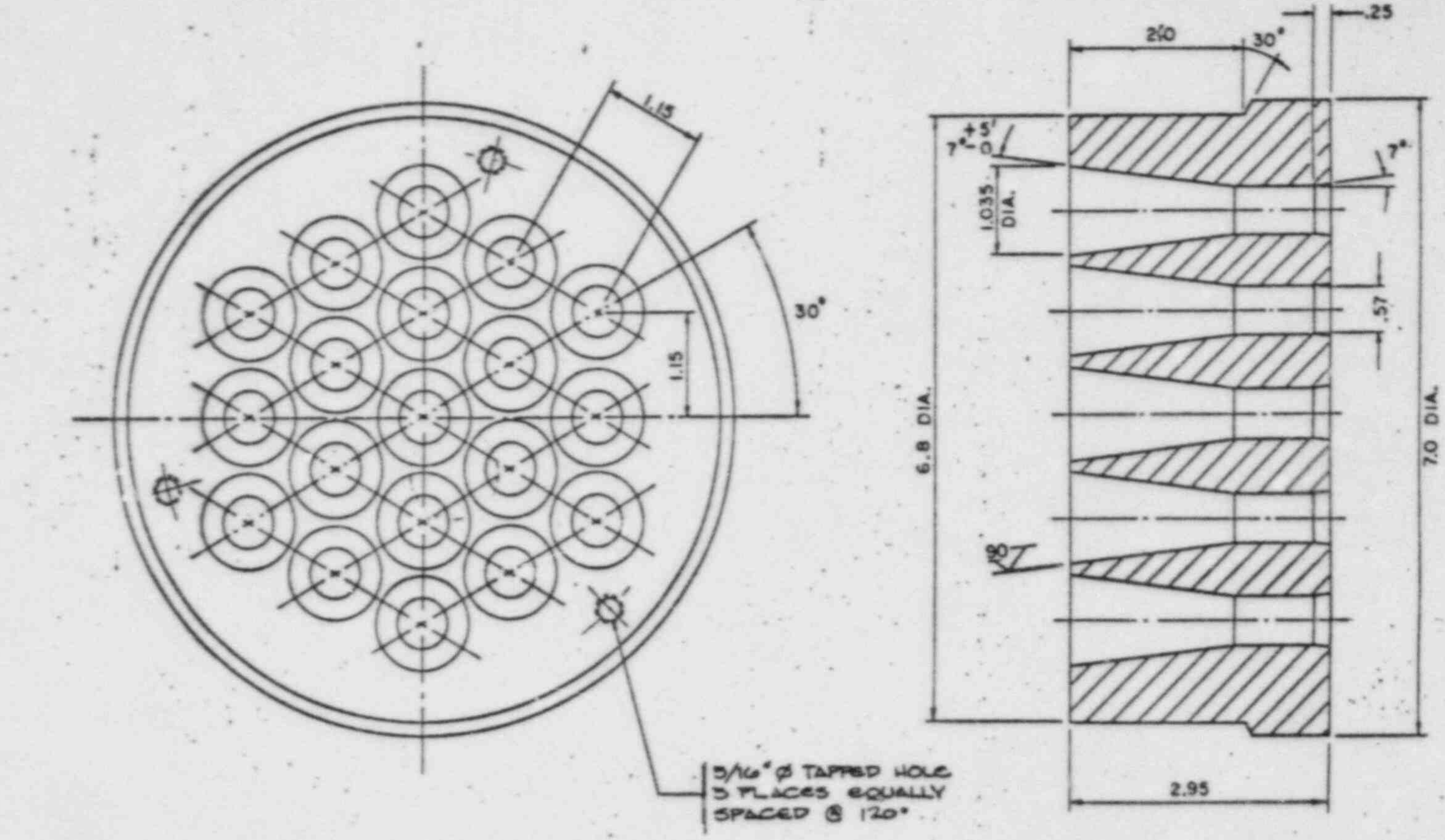

MATCH LINE
STRESS TAIL
6½" DIA BOREHOLE
15'
15"
BAND CLAMPS
30"
15"
SPREADER RINGS
15"
½" PLASTIC REMOVABLE GROUT PIPE
30"
5"
15'
6"
ONE STRAND
STRESS LENGTH 25'-0 BOTH ANCHORS
BOND LENGTH.
MATCH LINE
BOND LENGTH 20'-0" 30'-0"

Fig. 2.

FIG. 5

FIG. 5.

NAE 5807-6



MATERIAL: C 1050 HR

MACHINED SURFACES 125 UNLESS OTHERWISE SPECIFIED

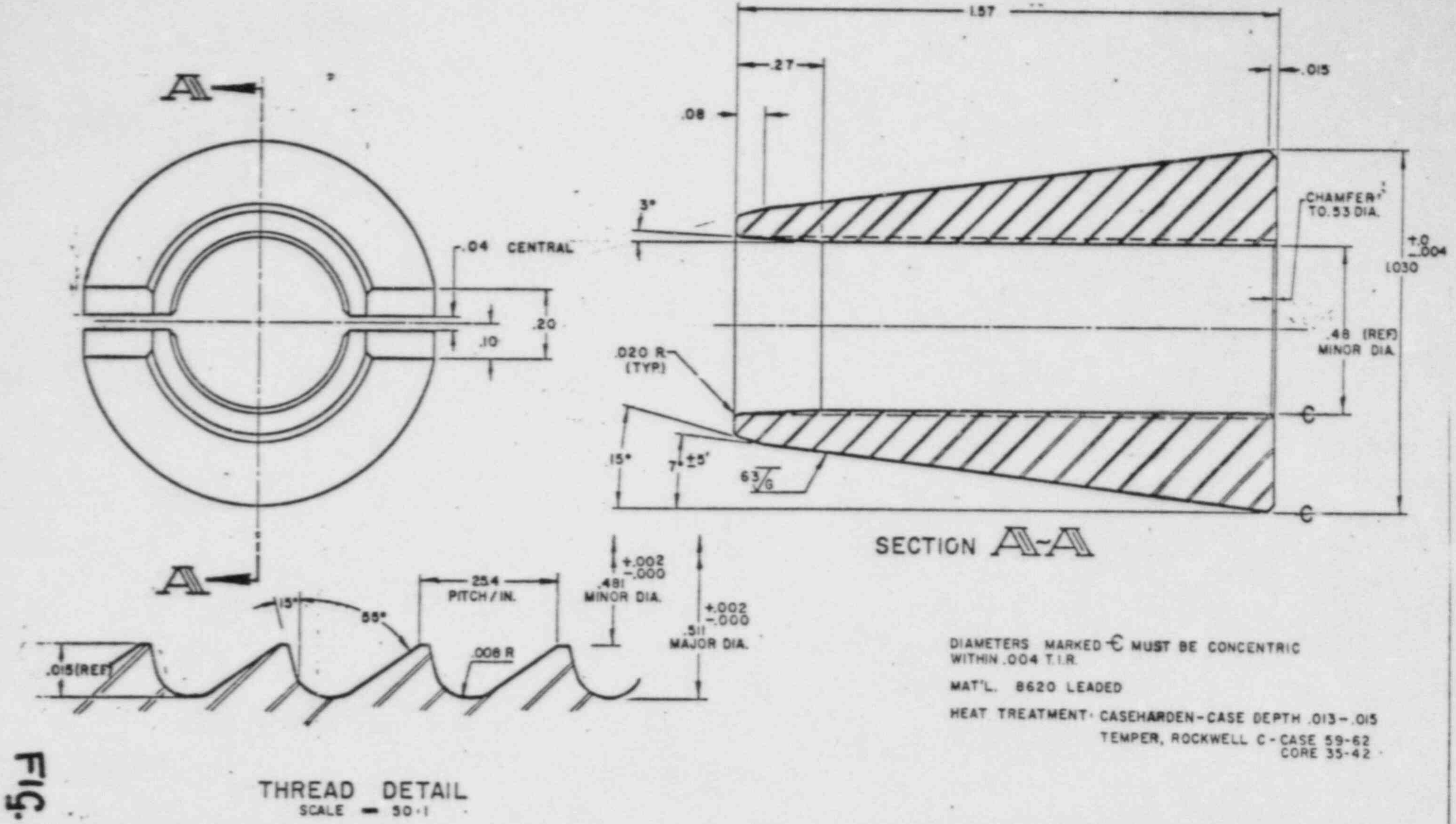
TOLERANCES
.X .030
.XX .010
.XXX .005
ANGLES 1/2°

VSL CORPORATION LOS GATOS, CALIFORNIA

ANCHOR HEAD E5-19

DRWN.
CHKD.
SCALE: FULL
APPROVED
DWG. NO. 19.01

REV CHANGE DATE BY



DIAMETERS MARKED C MUST BE CONCENTRIC WITHIN .004 T.I.R.

MAT'L. 8620 LEADED

HEAT TREATMENT: CASEHARDEN - CASE DEPTH .013 - .015
TEMPER, ROCKWELL C - CASE 59-62
CORE 35-42

FIG. 4.

REV	CHANGE	DATE	BY

TOLERANCES UNLESS SPECIFIED
.X ± .030
.XX ± .010
.XXX ± .005
ANGLES ± 1/2°

V S L CORPORATION LOS GATOS, CALIFORNIA

1/2" VSL WEDGE

DRAWN Dion
SCALE 5:1 & NOTED
APPROVED
DWG NO G.00.01
REV

AMERICAN SPRING WIRE CORPORATION
P. O. BOX 46585 BEDFORD HEIGHTS, OHIO 44146
TEL. (216) 292-4620

CERTIFICATION

VSL Corporation
P.O. Box 866
Springfield, Virginia 22150

P. O.	SIZE	GRADE	WEIGHT	DATE
7335	1/2	- 270K Strand per ASTM A416 Spec. in 12,000 ft. center pull Reel-less Paks	----	9-24-75

HEAT #	CHEMICAL ANALYSIS				TENSILE STRENGTH
Y 532984	C	.78	S	.022	-----
	Mn	.83	Si	.27	
	P	.022			

Michael J. Urbancic
Laboratory

NAC 5007-27

FIG. 5A.

#5.44

CC-[illegible]

AMERICAN SPRING WIRE CORPORATION
P. O. BOX 46585 - BEDFORD HEIGHTS, OHIO 44146
TELEPHONE: (216) 292-4620

CUSTOMER VSL
CUSTOMER ORDER NO. 7335
ITEM 1/2 270K
HEAT NO. Y532984
QUANTITY 21 PAKS TOTAL

STRESS RELIEVED P.C. STRAND
TEST CERTIFICATE

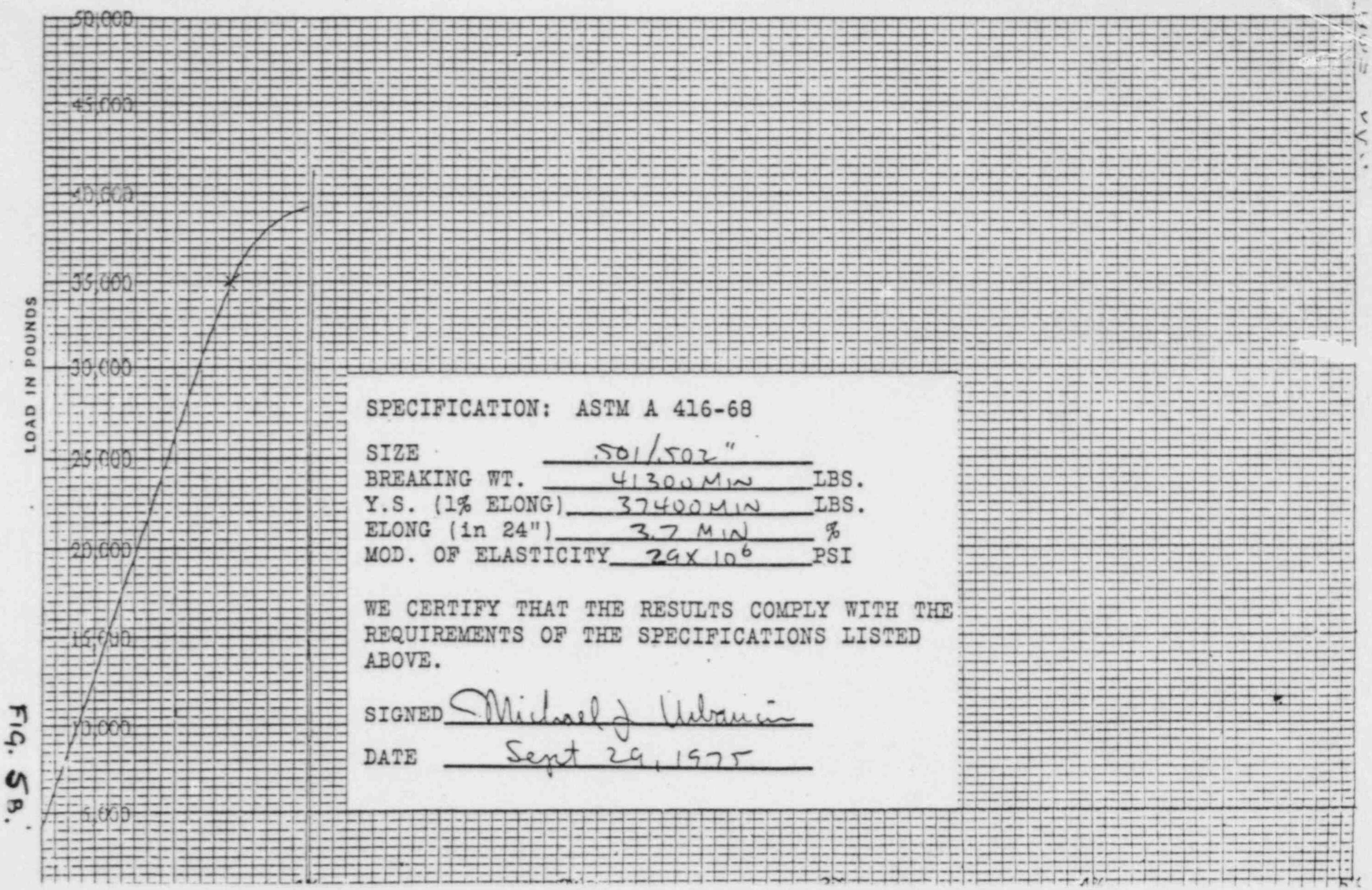


FIG. 5B.

SPECIFICATION: ASTM A 416-68

SIZE .501/.502"
BREAKING WT. 41300 MIN LBS.
Y.S. (1% ELONG) 37400 MIN LBS.
ELONG (in 24") 3.7 MIN %
MOD. OF ELASTICITY 29 X 10⁶ PSI

WE CERTIFY THAT THE RESULTS COMPLY WITH THE REQUIREMENTS OF THE SPECIFICATIONS LISTED ABOVE.

SIGNED Michael J. Urbancic
DATE Sept 29, 1975

VISCOR
Established 1892
VISCOSITY OIL COMPANY

PRODUCT DATA SHEET

VISCONORUST 3166

SCOPE

Visconorust 3166 is Compounded from highly refined petroleum oils, and long chain metallic hydro-carbons. It is designed to provide a film that is both flexible and stable from below 0°F to 300°F. Visconorust 3166 can be applied at ambient plant temperature without additional heat and be applied to the wire strand either by an automated pumping system or manually.

END USE

A Rust Protective and Lubricant coating to be applied to strand tendons prior to covering with a plastic casing for use in post tensioning construction.

PHYSICAL SPECIFICATION AND CHARACTERISTICS:

A. Color: Dark Amber

B. Dropping Point (Melting Point ASTM D-566) 350°F Minimum. No flowing or leakage off wires at high ambient temperatures.

C. Flash Point (Cleveland Open Cup Test): 350° F. Min.

D. Film: Grease type - soft, pliable, self healing

E. ASTM Cone Penetration @ 77°F (265-295)

F. Viscosity of oil @ 100°F 300-800

G. Excellent barrier against possible water contamination, and insulation against stray electrical currents.

H. Lubrication

Visconorust 3166 by remaining pliable and greasy throughout its operating temperature range will provide the required lubrication between the tendon and the plastic casing.

PK FIG. 6

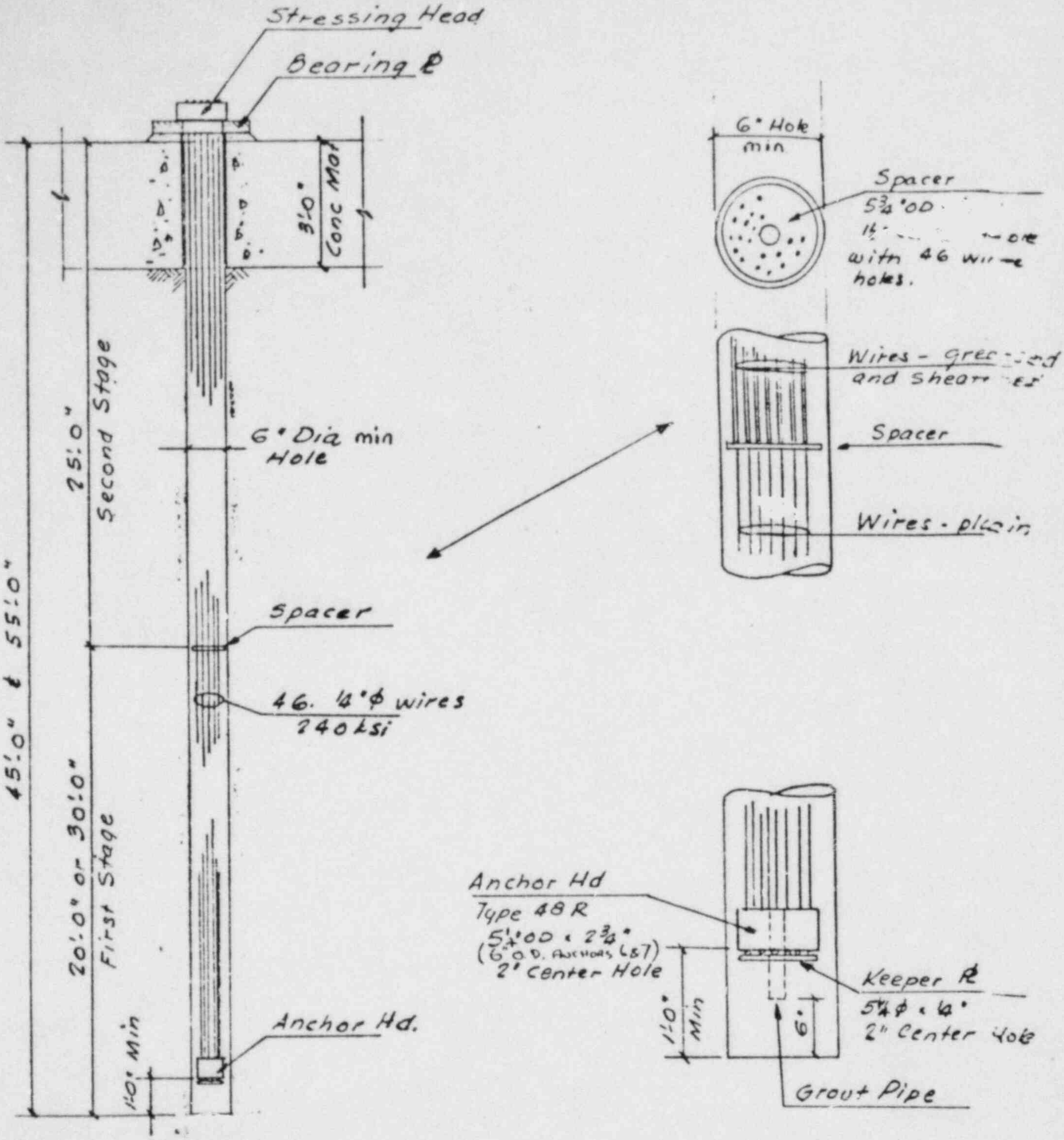
[illegible] SOUTH WESTERN AVENUE
CHICAGO, ILLINOIS 60608
U S A

TELEPHONE
AREA CODE 312
[illegible]

NAC 5007-37

I.	Rust Test - ASTM D-1743	Pass
J.	Water Soluble Chlorides, PPM	Less than 2
	Water Soluble Nitrates, PPM	Less than 2
	Water Soluble Sulfides, PPM	Less than 2

NAC 5007-37
FIG. 6.A



TYPICAL ROCK ANCHOR

ARRANGEMENT OF PRESCON TENDONS

FIG. 7.

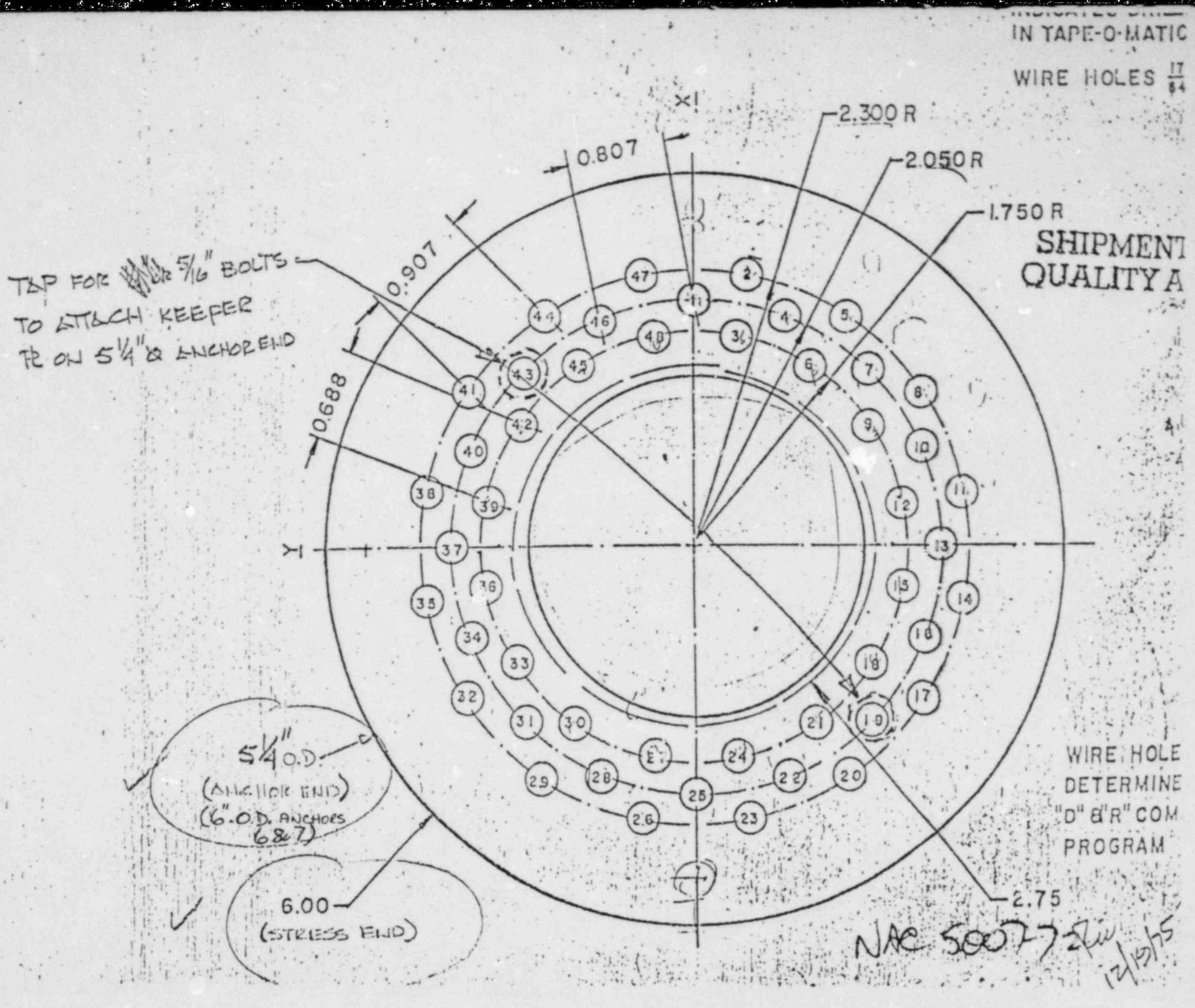

INDICATES DRI
IN TAPE-O-MATIC
WIRE HOLES 17/64
X
2.300 R
0.807
2.050 R
1.750 R
SHIPMENT
QUALITY A
0.907
TAP FOR 5/16" BOLTS TO ATTACH KEEPER PL ON 5 1/4" Ø ANCHOR END
0.688
Y
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48
5 1/4" O.D. (ANCHOR END) (6" O.D. ANCHORS 6 & 7)
6.00 (STRESS END)
2.75
WIRE HOLE
DETERMINE
"D" & "R" COM
PROGRAM
NAC 50077 Rev 12/15/75

FIG. 9

Armco Steel Corporation
Kansas City, Mo. 64125

CERTIFICATION OF TEST
TUFWIRE

DEC 12 1975

DATE December 9, 1975

CUSTOMER: The Prescon Corporation

NOMINAL WIRE DIA. .250"

MIN. SPECIFICATIONS

REQ'D. BREAKING STRENGTH 11,784 LBS. 240,000 P.S.I.

MINIMUM ELONGATION IN 10" 4.00 PERCENT

Customer Order No. 4377

COIL NO.	DIAMETER		BREAKING STRENGTH LBS.		ULT. TENSILE STR. PSI		% ELONG 10"	BENDS	STRAIGHT-NESS	BUTTON HEAD	YIELD STRENG
	FRONT	BACK	FRONT	BACK	FRONT	BACK					
5	.251		12,660		255,800		5.00				11,14
7	.251		12,400		250,600						
9	.251		12,720		257,000						
31	.251		12,360		249,700						
32	.251		12,820		259,000						
33	.251		12,660		255,800						
34	.251		12,540		253,400						
39	.251		12,240		247,300						
74)	.251		12,2[illegible]		246,900						
25	.251		12,580		254,200						
28	.251		12,520		253,000						
100	.251		12,500		252,600						
104	.251		12,420		251,000						
105	.251		12,500		252,600		5.60				11,04
106	.251		12,440		251,400						
108)	.251		12,340		249,300						
114	.251		12,820		259,000						
116	.251		12,400		250,600						
120	.251		12,560		253,800						
121	.251		12,700		256,600						

HEAT NO. 27368

ANALYSIS: .79 C .85 MN .010 P .017 S .25 SI

THE PHYSICAL OR MECHANICAL TEST REPORTED ABOVE ARE CORRECT AS CONTAINED IN THE RECORDS OF THE CORPORATION.

FIG. 9A.

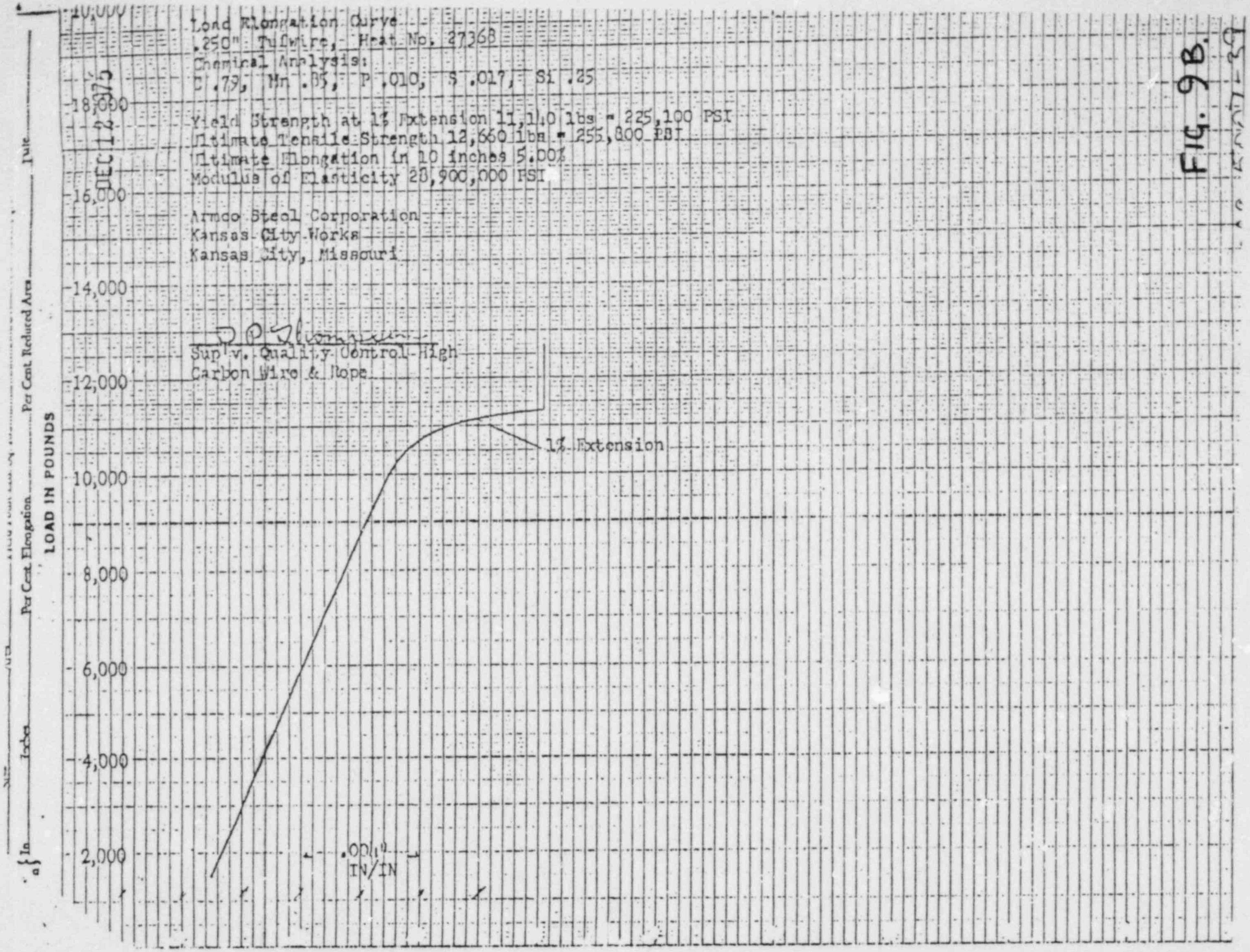
BY J P Thompson

TITLE

.1AA 5807- 39

MU-2588 REV. 2/72

FIG. 9B.



Load Elongation Curve
.250" Tufwire, Heat No. 27368
Chemical Analysis:
C .79, Mn .85, P .010, S .017, Si .25

Yield Strength at 1% Extension 11,140 lbs = 225,100 PSI
Ultimate Tensile Strength 12,660 lbs = 255,800 PSI
Ultimate Elongation in 10 inches 5.00%
Modulus of Elasticity 28,900,000 PSI

Armco Steel Corporation
Kansas City Works
Kansas City, Missouri

D P Thompson
Sup'v. Quality Control-High
Carbon Wire & Rope

DEC 12 1975

SHELL OIL COMPANY
P. O. BOX 1422
HOUSTON, TEXAS 77001

April 2, 1973.

Prescon Corp.
1338 N. W. W. White Road
San Antonio, Texas

Attention: Mr. Harvey Penshorn

Dear Mr. Penshorn:

The purpose of this letter is to familiarize you with the specifications of Shell Cable Coating Grease Number 1 (C. C. Grease #1).

Shell C. C. Grease is a smooth, buttery textured calcium based grease with a rust inhibitor blended in, which gives the grease a reddish appearance. Calcium based greases have a superior water resistance quality in comparison to sodium based greases. Calcium based greases' useful temperature is 180°F compared to 200°F for sodium based greases. The National Lubricating Grease Institute (NLGI) grade number 1 grease will allow good penetration of the grease into the strands along with better pumpability.

Properties of Shell C. C. Grease Number 1 are as follows:

ASTM Worked Penetration @ 77°F (60 strokes)	320
Dropping Point °F	205
Soap	Calcium
% Soap	12
Appearance	Smooth
Color	Reddish
Mineral Oil Vis. SSU @ 100°F	305
Mineral Oil Pour Point	-15
Rust inhibited	Yes

I wish to thank you for the courtesy extended to me during our recent phone conversation. I'm working toward getting you a written quotation, however, I may have to contact you again personally or by phone before writing you a quotation.

If you have any questions concerning this product, please feel free to contact me at area code 713-526-4631, ext. 47.

Very truly yours,

JWSmith

J. W. Smith, Senior Representative
Industrial Sales Department

FIG. 10

JAC 5007-42

CALCULATION SHEET

Item

J1

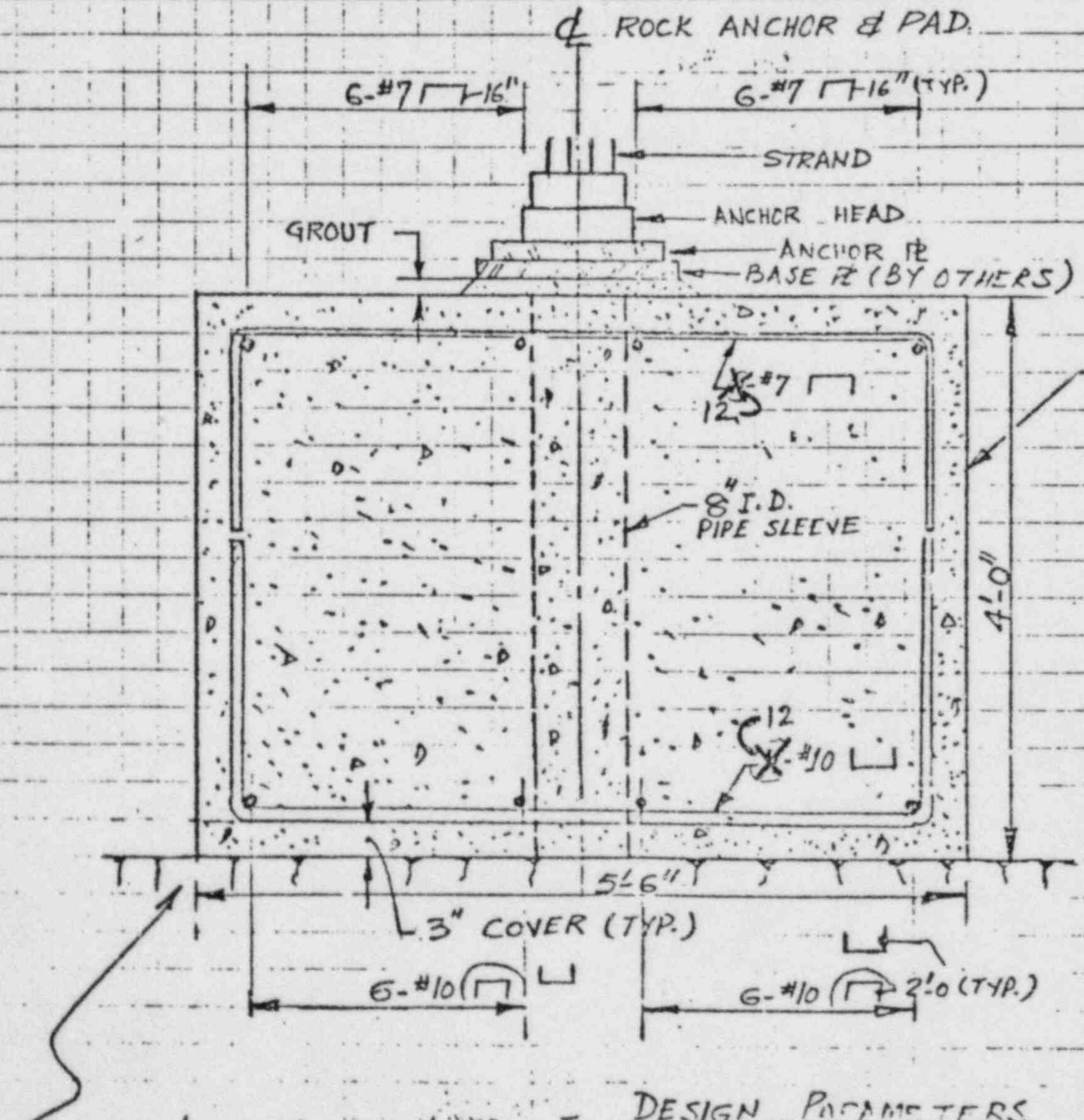
Client VEPCO Location N. ANNA #3 Est. No J.O. No 12180

Subject ROCK ANCHOR TEST PAD Date 10/21/75 By P.S. GUPTA

M... / G.S. AREA (VERTICAL ANCHOR) Checked 10/21/75 By S.H. Patel

Based on Revised By

NOTED OCT 23 1975 F.F.Chin



DESIGN PARAMETERS

$f_c' = 3000$ PSI.

SIZE OF BASE ℄ $16\frac{3}{8}" \times 16\frac{3}{8}"$ (Assumed)

DESIGN LOAD = 550^k.

CONC. TO ROCK BEARING = $10^T/\square'$

8" I.D PIPE SLEEVE CENTERED OVER THE PAD.

FIG. 11.

WATER PRESSURE TESTS

NORTH ANNA POWER STATION
ROCK ANCHOR TESTS

ANCHOR No. GROUT TEST. GAUGE EL. —
DATE TESTED 12-4-75, SURFACE EL. +230'
S.W.L. EL. +224·5 PACKER EL. —

TEST ZONE	GAUGE P.S.I.	FLOW GAL.	TIME MIN.	HEAD DROP FT	REMARKS.
	FALLING HEAD TEST		0	0	READINGS TAKEN
			·25	·27	FROM TOP OF
			·5	·54	CASING 1'-5" ABOVE
			·75	·77	GROUND SURFACE.
			1·0	·89	
			1·25	1·06	
			1·5	1·22	
			1·75	1·35	
			2·0	1·46	
			2·25	1·57	
			2·5	1·69	
			2·75	1·76	
			3·0	1·84	
			3·25	1·92	
			3·5	1·99	
			3·75	2·05	
			4·0	2·10	
			4·25	2·15	
			4·5	2·21	
			4·75	2·23	
			5·0	2·28	
			5·25	2·33	
			5·5	2·36	
			5·75	2·39	
			6·0	2·46	
			6·25	2·48	
			6·5	2·50	
			6·75	2·53	
			7·0	2·56	
			8·0	2·67	

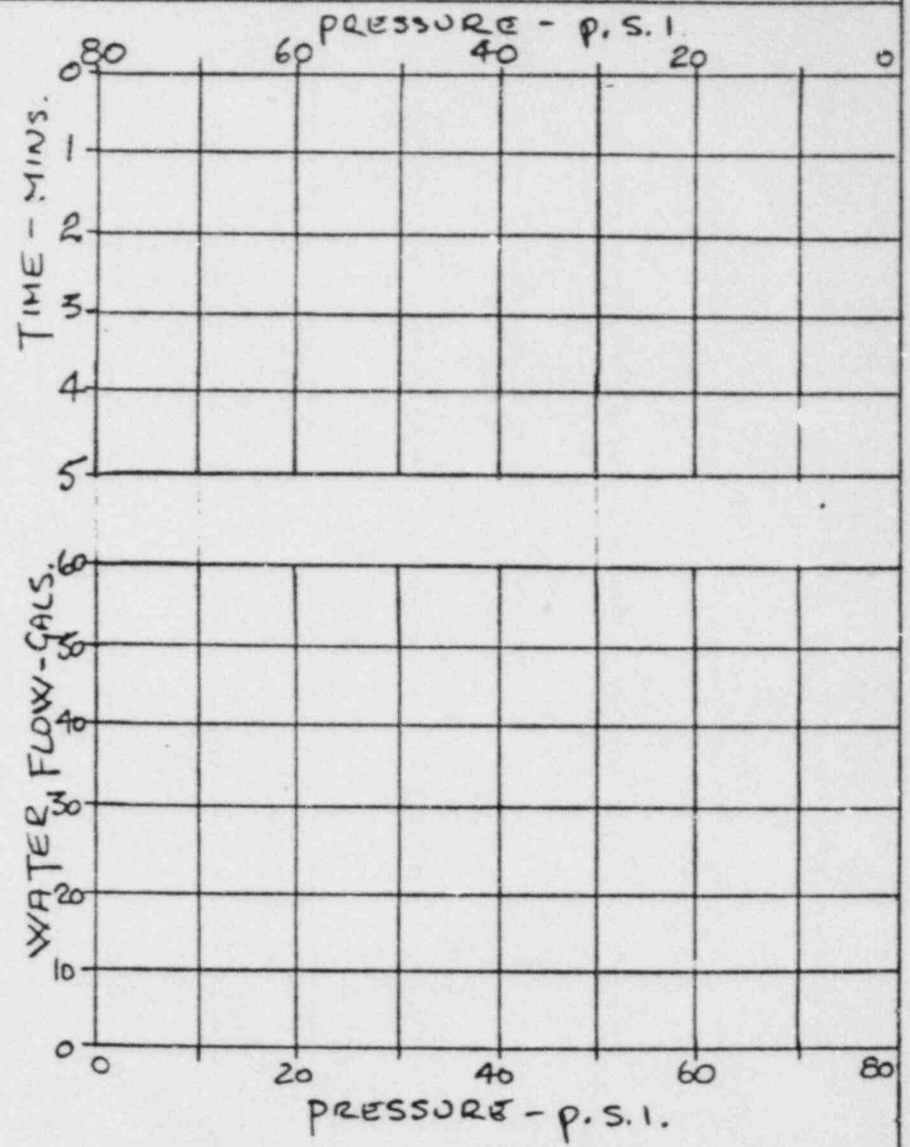


FIG. 12.

WATER PRESSURE TESTS

NORTH ANNA POWER STATION
ROCK ANCHOR TESTS

ANCHOR Nº GROUT TEST
DATE TESTED 12-5-75'
S.W.L. EL. ±225'

GAUGE EL. +234'
SURFACE EL. ±230'
PACKER EL. +195'

TEST ZONE	GAUGE P.S.I.	FLOW GAL.	TIME MIN.	HEAD DROP FT	REMARKS
+195' +185'	25.2		0		WATER FLOW
	20.5		.15		STOPPED AND
	20.0		.30		PRESSURE GAUGE
	19.5		.45		MONITORED FOR
	18.8		1.00		DROP IN PRESSURE
	18.0		1.15		FOR TEST
	17.8		1.30		INTERVAL.
	17.2		1.45		
	17.0		2.00		
	16.8		2.15		
	16.5		2.30		
	16.0		2.45		
	15.8		3.00		
	15.5		3.15		
	15.1		3.30		
	15.0		3.45		
	14.8		4.00		
	14.5		4.15		
	14.4		4.30		
	14.0		4.45		
	13.8		5.00		
+215' +185'	25	.75	10		TESTED 12-17-75' AFTER HOLE PRELIMINARY GROUTED.

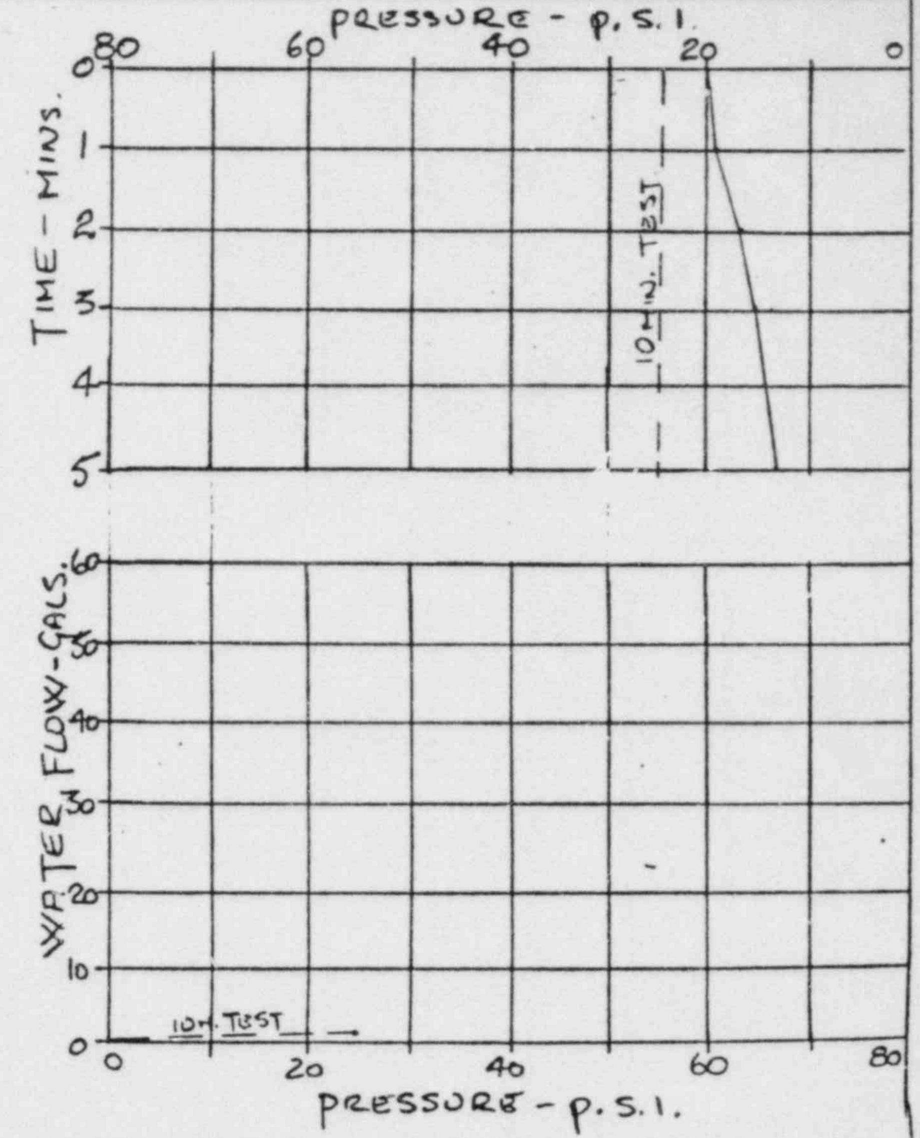


FIG. 13.

WATER PRESSURE TESTS

NORTH ANNA POWER STATION
ROCK ANCHOR TESTS

ANCHOR Nº 1
DATE TESTED 12-22-75
S.W.L. EL. +228'
GAUGE EL. –
SURFACE EL. +237'
PACKER EL. –

TEST ZONE	GAUGE P.S.I.	FLOW GAL.	TIME MIN.	HEAD DROP FT	REMARKS.
	FALLING		0	0	READINGS TAKEN
	HEAD	TEST	·5	·15	FROM TOP OF
			1	·26	CONCRETE PAD.
			1·5	·36	
			2	·50	
			2·5	·62	
			3	·75	
			3·5	·86	
			4	·99	
			4·5	1·11	
			5	1·23	
			5·5	1·33	
			6	1·45	
			6·5	1·56	
			7	1·67	
			7·5	1·79	
			8	1·88	
			8·5	1·99	
			9	2·10	
			9·5	2·19	
			10	2·30	
			10·5	2·40	

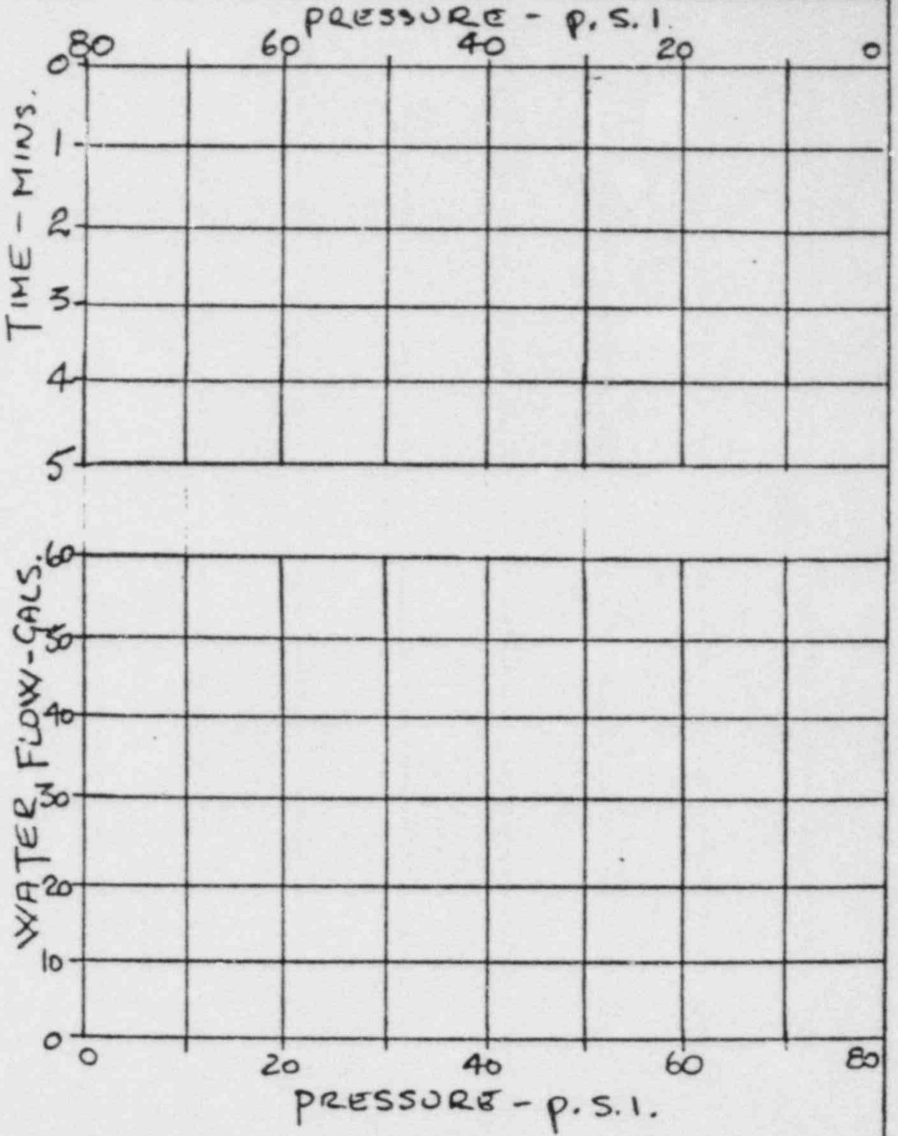


FIG. 14.

WATER PRESSURE TESTS.

NORTH ANNA POWER STATION
ROCK ANCHOR TESTS.

ANCHOR Nº 1
DATE TESTED 12-23-75
S.W.L. EL. +229.2'

GAUGE EL. +241'
SURFACE EL. +237'
PACKER EL. +227'

TEST ZONE	GAUGE P.S.I.	FLOW GAL.	TIME MIN.	HEAD DROP FT	REMARKS
+227' - +180'	6.5	8.4	1		
"	"	4.3	3		
"	"	4.1	5		
"	12.5	5.7	1		
"	"	8.1	3		
"	"	7.2	5		
"	25	13.9	1		TEST STOPPED AS WATER WAS FLOWING INTO REACTOR PIT 4 FROM FISSURE.
"	"	15.6	3		
"	"	26.3	5		
+226.2 - +180'	15	0	5		TESTED 12-30-75 AFTER HOLE WAS PRELIMINARY GROUTED.

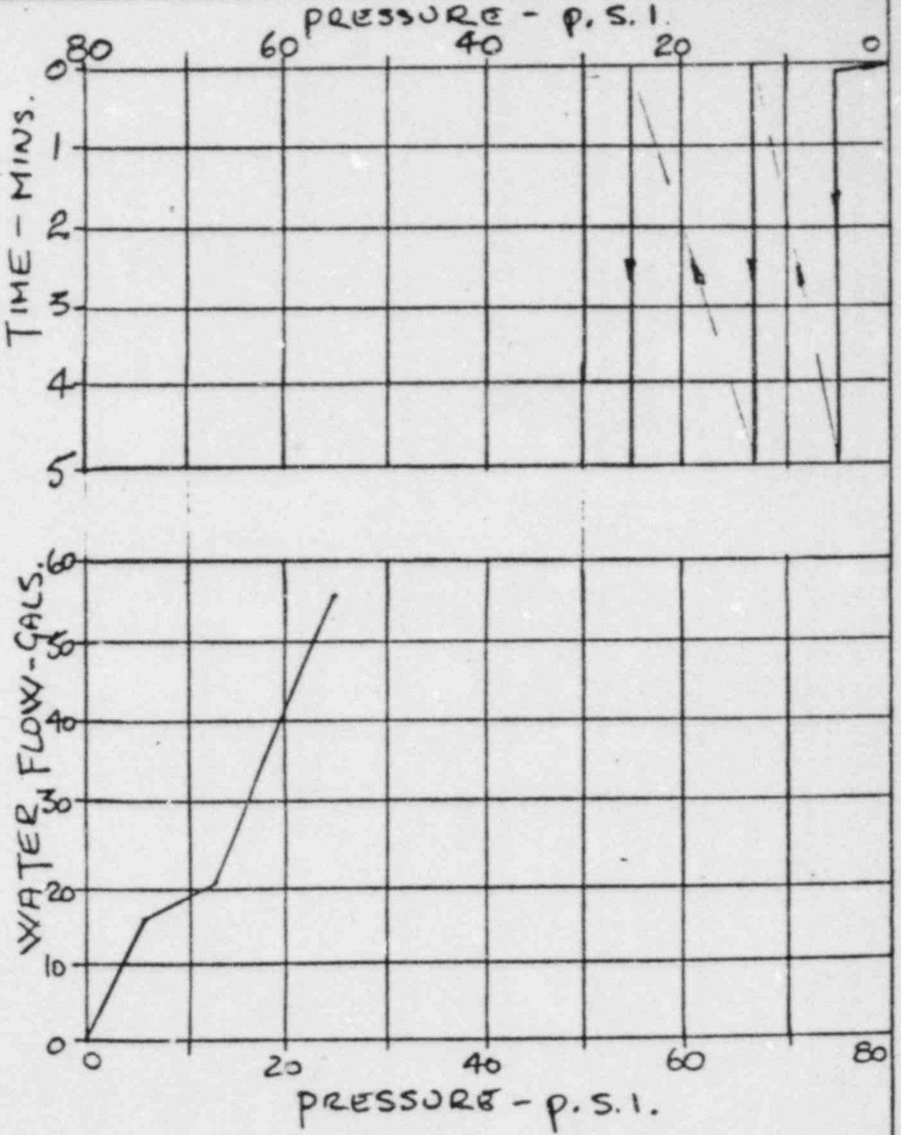


FIG. 15.

WATER PRESSURE TESTS

NORTH ANNA POWER STATION
ROCK ANCHOR TESTS

ANCHOR Nº 2
DATE TESTED 12-22-75
S.W.L. EL. +228'

GAUGE EL. —
SURFACE EL. +237'
PACKER EL. —

TEST ZONE	GAUGE P.S.I.	FLOW GAL.	TIME MIN.	HEAD DROP FT	REMARKS.
	FALLING HEAD TEST		0	0	READINGS TAKEN FROM TOP OF CONCRETE PAD.
			·5	·12	
			1	·23	
			1·5	·35	
			2	·47	
			2·5	·58	
			3	·69	
			3·5	·80	
			4	·92	
			4·5	1·01	
			5	1·12	
			5·5	1·22	
			6	1·33	
			6·5	1·46	
			7	1·54	
			7·5	1·64	
			8	1·73	
			8·5	1·86	
			9	1·95	
			9·5	2·05	
			10·0	2·12	

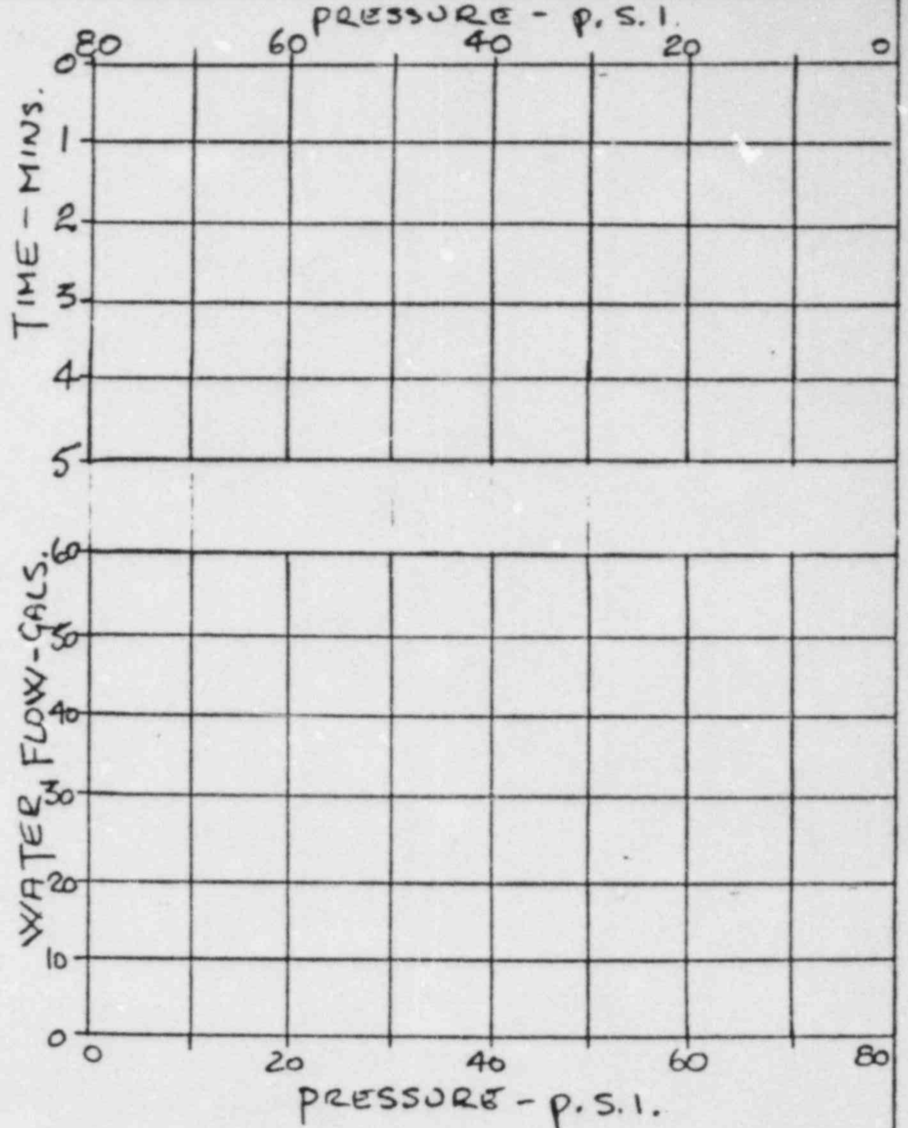


FIG. 16.

WATER PRESSURE TESTS

NORTH ANNA POWER STATION
ROCK ANCHOR TESTS

ANCHOR Nº 2
DATE TESTED. 12-23-75
S.W.L. EL. ±229.2'

GAUGE EL. +241'
SURFACE EL. ±237'
PACKER EL. +221'

TEST ZONE	GAUGE P.S.I.	FLOW GAL.	TIME MIN.	HEAD DROP FT	REMARKS
+221' +180'	6.25	8.6	1		
"	"	8.4	2		
"	"	1.0	3		
"	"	0.4	4		
"	"	0.3	5		
"	12.5	9.3	1		
"	"	5.7	2		
"	"	3.8	3		
"	"	3.7	4		
"	"	0	5		
"	25	14.8	1		
"	"	11.4	2		
"	"	8.8	3		
"	"	-	4		PACKER LOST AIR & DEFLATED
+220.2' +180	15	0	5		TESTED 12-30-75 AFTER PRELIMINARY GROUTING.

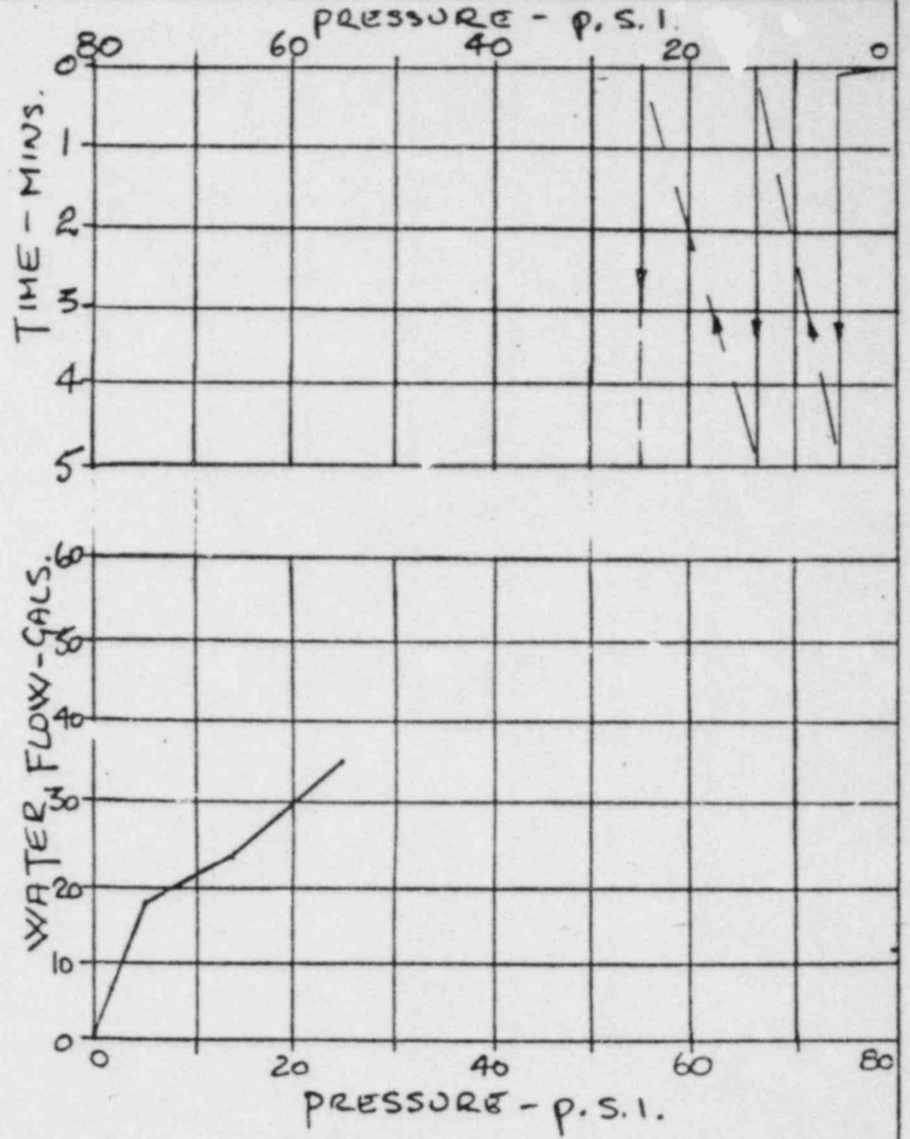


FIG. 17.

WATER PRESSURE TESTS

NORTH ANNA POWER STATION
ROCK ANCHOR TESTS

ANCHOR Nº 3
DATE TESTED 12-22-75
S.W.L. EL. ±230
GAUGE EL. -
SURFACE EL. ±246
PACKER EL. -

TEST ZONE	GAUGE P.S.I.	FLOW GAL.	TIME MIN.	HEAD DROP FT	REMARKS.
	FALLING		0	0	READINGS TAKEN
	HEAD TEST		·5	0	FROM TOP OF
			1	·03	CONCRETE PAD
			1·5	·03	
			2	·04	
			2·5	·05	
			3	·05	
			3·5	·07	
			4	·09	
			4·5	·1[illegible]	
			5	·10	
			5·5	·10	
			6	·11	
			6·5	·12	
			7	·13	
			7·5	·14	
			8	·15	
			8·5	·15	
			9	·16	
			9·5	·17	
			10	·18	

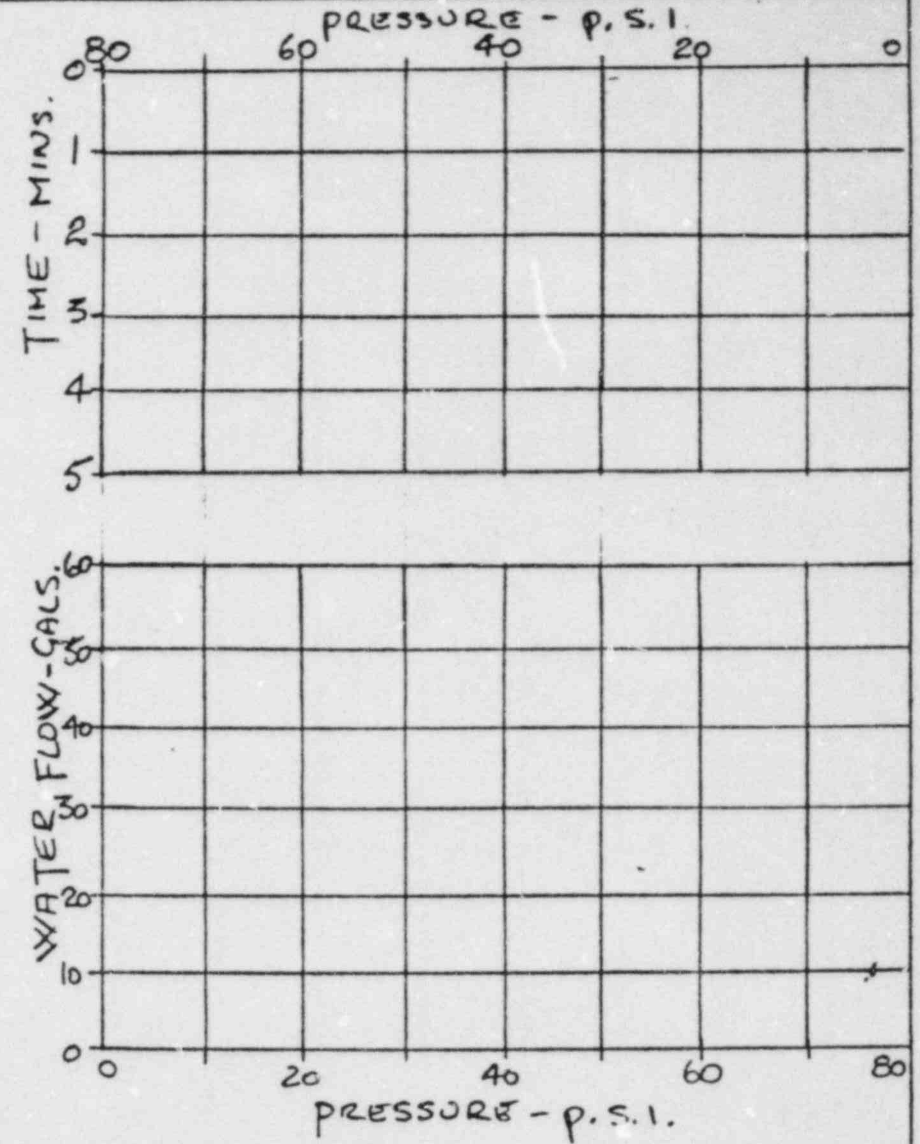


FIG. 18.

WATER PRESSURE TESTS

NORTH ANNA POWER STATION
ROCK ANCHOR TESTS

ANCHOR No 3
DATE TESTED 12-23-75
S.W.L. EL. +227'

GAUGE EL. +250'
SURFACE EL. +246'
PACKER EL. +236'

TEST ZONE	GAUGE P.S.I.	FLOW GAL.	TIME MIN.	HEAD DROP FT	REMARKS.
+236' +199'	6.25	0.4	1		WATER ROSE
"	"	0.8	3		IN B.H. No4
"	"	0.6	5		
"	12.5	0.7	1		
"	"	1.3	3		
"	"	0.8	5		INTERCONNECTED
"	25	1.0	1		TO BH No 4.
"	"	1.7	3		NO PRESSURE
"	"	1.8	5		TEST ON No 4.
"	12.5	0.5	1		
"	"	0.6	3		
"	"	0.8	5		
"	6.25	0	1		
"	"	0	3		
"	"	0	5		
"	15	0	5		TESTED 12-30-75 AFTER PRELIMINARY GROUTING.

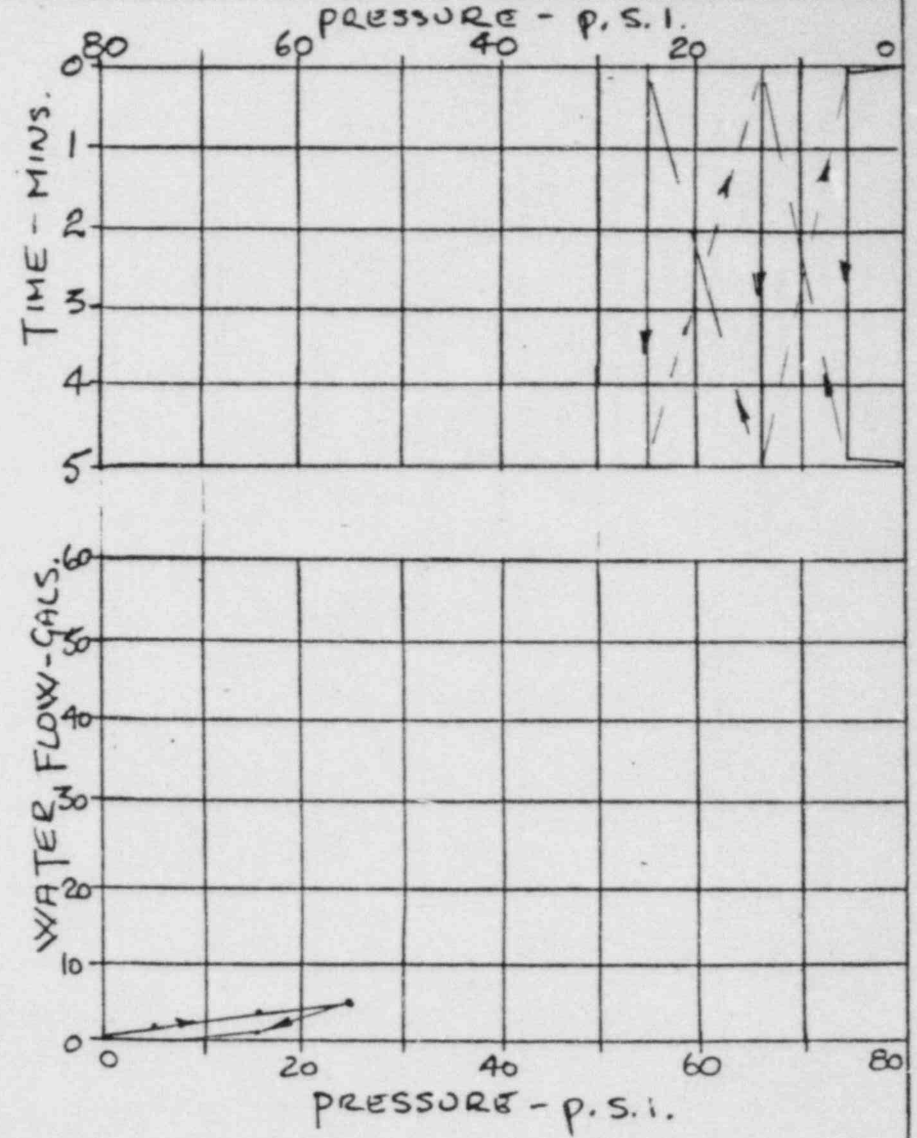


FIG. 19.

WATER PRESSURE TESTS

NORTH ANNA POWER STATION
ROCK ANCHOR TESTS

ANCHOR Nº 4
DATE TESTED 12-22-75
S.W.L. EL. +230'

GAUGE EL. -
SURFACE EL. +246'
PACKER EL. -

TEST ZONE	GAUGE P.S.I.	FLOW GAL.	TIME MIN.	HEAD DROP FT	REMARKS.
FULL	FALL	ING	0	0	READINGS TAKEN
HOLE	HEAD	TEST	·5	·04	FROM TOP OF
			1	·08	CONCRETE PAD
			1·5	·13	
			2	·16	
			2·5	·18	
			3	·23	
			3·5	·26	
			4	·30	
			4·5	·32	
			5	·36	
			5·5	·40	
			6	·42	
			6·5	·50	
			7	·52	
			7·5	·54	
			8	·56	
			8·5	·60	
			9	·64	
			9·5	·65	
			10	·69	

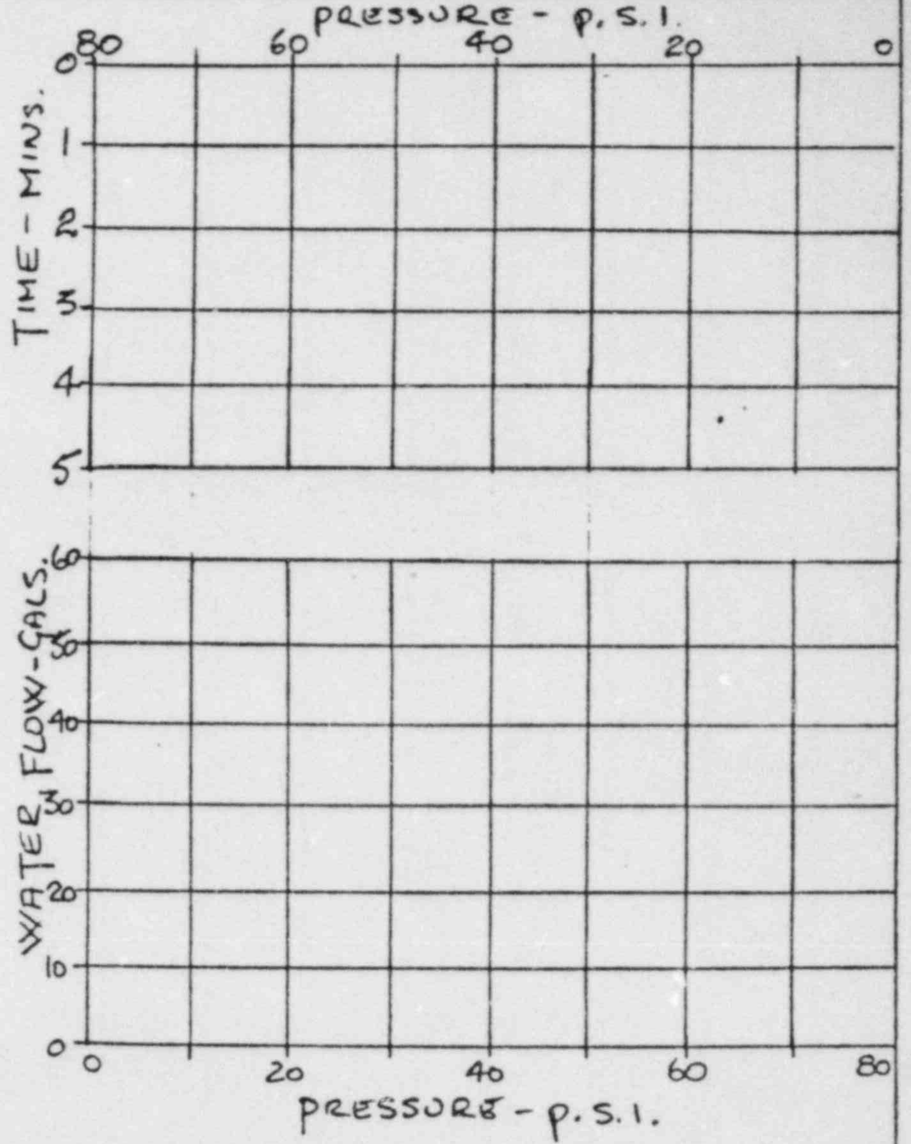


FIG. 20.

WATER PRESSURE TESTS

NORTH ANNA POWER STATION
ROCK ANCHOR TESTS

ANCHOR Nº 5
DATE TESTED 2-16-76.
S.W.L. EL. –

GAUGE EL. +214.5'
SURFACE EL. +241'
PACKER EL. +234'

TEST ZONE	GAUGE P.S.I.	FLOW GAL.	TIME MIN.	HEAD DROP FT	REMARKS.
+234' | +189'	6.25	0	5		
"	12.5	0.1	5		
"	25	0	5		25 P.S.I HELD FOR 15 MIN WITHOUT WATER LOSS
FALLING HEAD TEST			5	'01'	

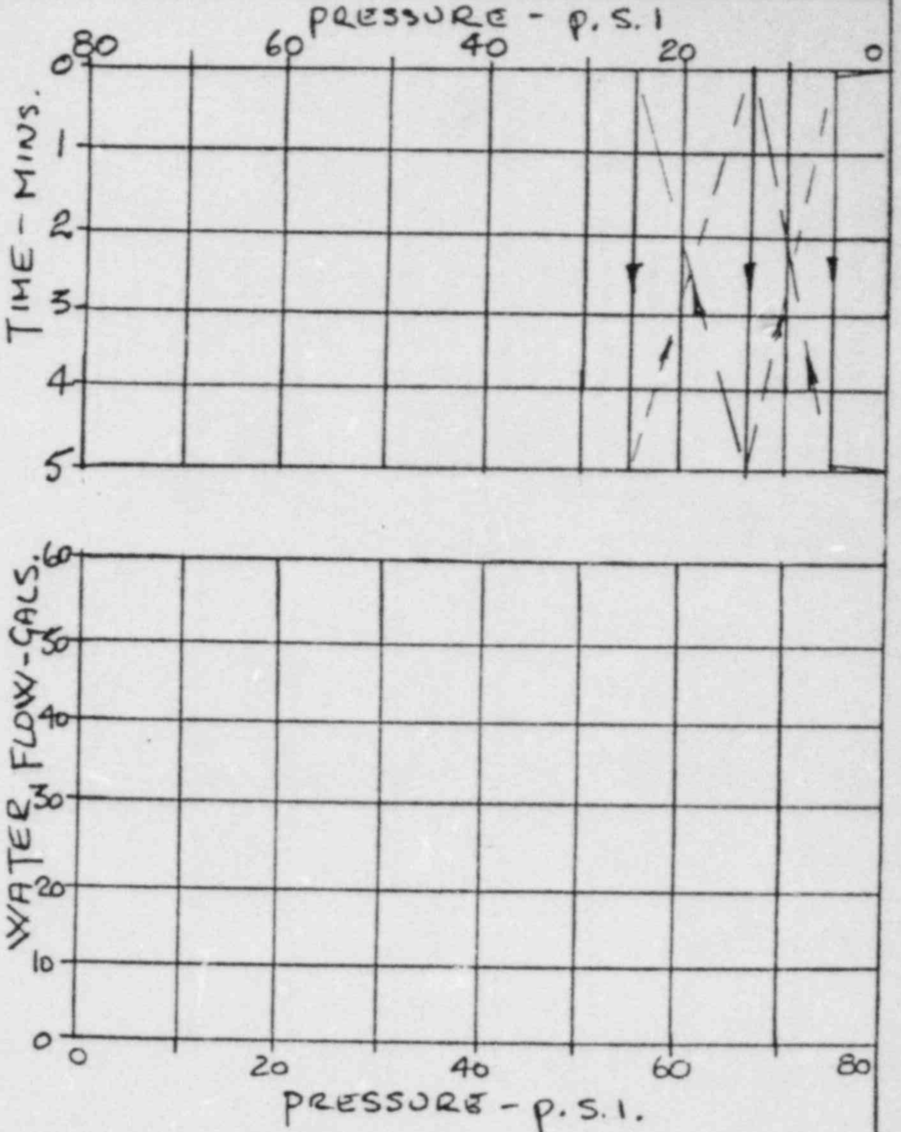


FIG. 21.

WATER PRESSURE TESTS

NORTH ANNA POWER STATION
ROCK ANCHOR TESTS

ANCHOR Nº 6
DATE TESTED 2-27-76
S.W.L. EL. –

GAUGE EL. +244.5'
SURFACE EL. +241'
PACKER EL. +240'

TEST ZONE	GAUGE P.S.I.	FLOW GAL.	TIME MIN.	HEAD DROP FT	REMARKS.
+240' +188'	25	0	1		
"	30	·1	1		SLIGHT PACKER LEAK.
"	25	0	4		
"	25	0	5		NO WATER LOSS.
FALLING HEAD TEST			10	0	NO WATER LOSS

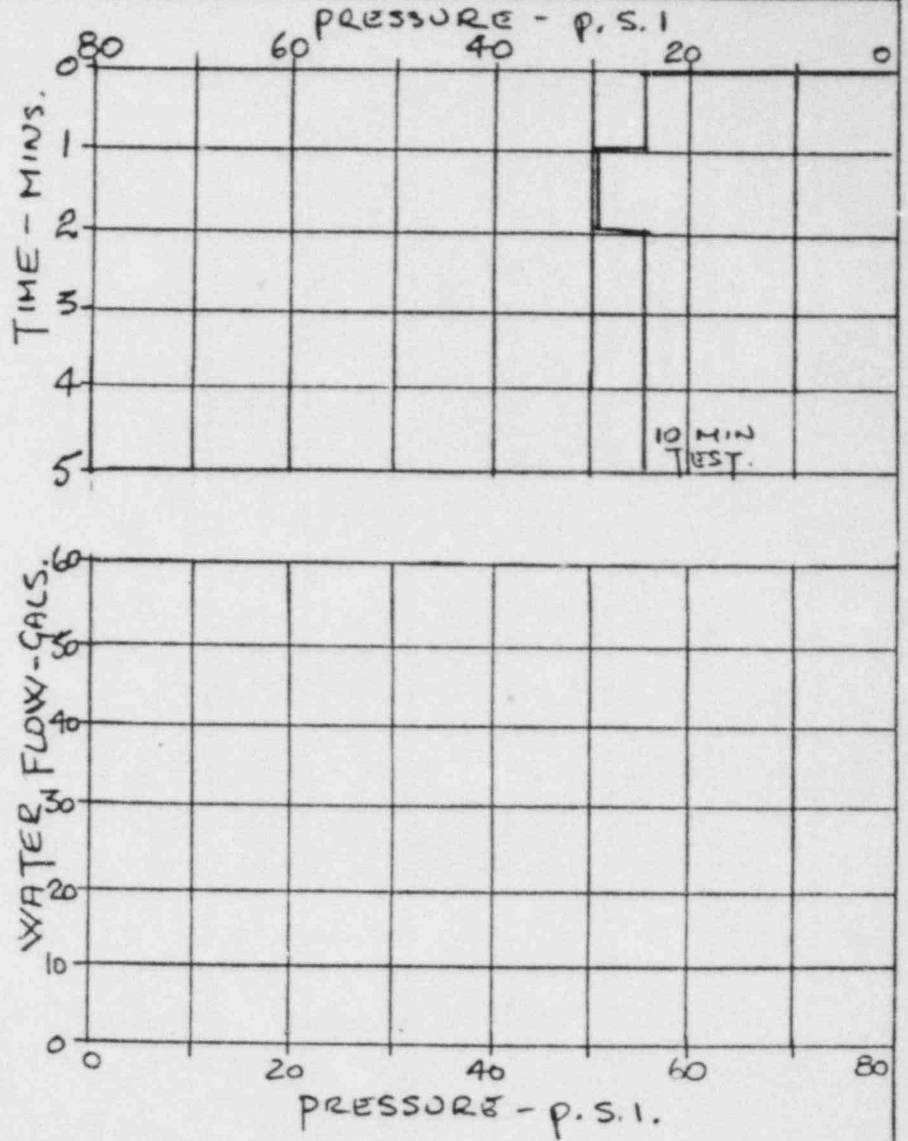


FIG. 22.

WATER PRESSURE TESTS

NORTH ANNA POWER STATION
ROCK ANCHOR TESTS

ANCHOR Nº 7
DATE TESTED 4-24-76.
S.W.L. EL. +200'

GAUGE EL. +244.5'
SURFACE EL. +241'
PACKER EL. +240'

TEST ZONE	GAUGE P.S.I.	FLOW GAL.	TIME MIN.	HEAD DROP FT	REMARKS.
+240' +188'	25	0	10		NO WATER LOSS
FULL HOLE FALLING HEAD TEST			10	0	NO WATER LOSS

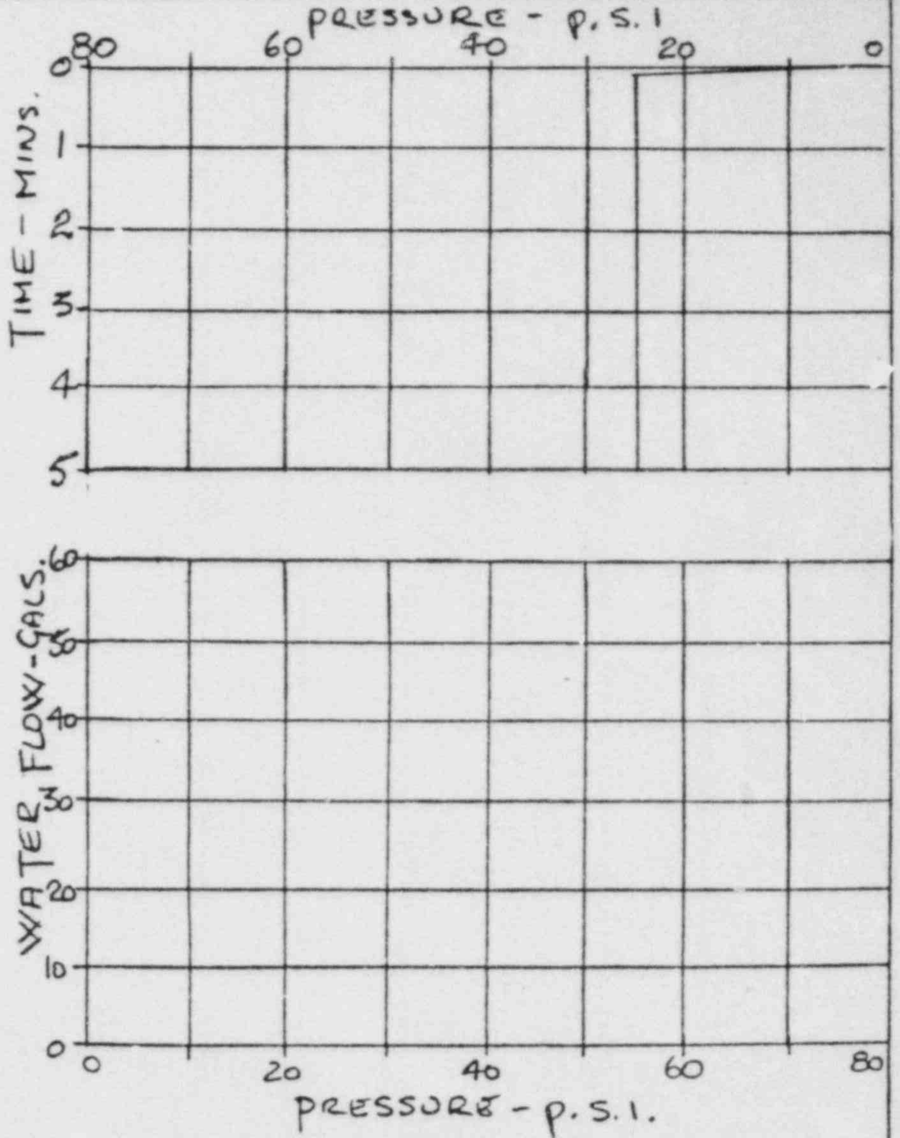


FIG. 23.

THIS FORM MUST BE COMPLETELY FILLED IN AND MUST ACCOMPANY EACH TEST SPECIMEN SENT TO THE LABORATORY

INFORMATION COMMON TO ALL AND DESIGN

North Anna Power Station

PROJECT LOCATION: Louisa County, Virginia

JOB ORDER: 12180 .50

CONCRETE MIXED BY: ~~[illegible]~~

CYLINDERS TAKEN BY: W.B. WILLARD

SPECIFIED CONCRETE STRENGTH: 3000 psi 28 DAYS

DESIGN MIX NO.: MASTERFLOW GROUT

DATE SAMPLED: 1-8-76

HOUR: 12:45 PM

WEATHER: CLEAR

LOCATION OF CONCRETE PLACEMENT: TEST ROCK ANCHORS 1 & 5 NORTH PC #4

CONCRETE MIXED: ☐ CENTRAL ☐ TRANSIT MIXED N/A ☐ AGITATED ☐ NON-AGITATED

TRUCK TICKET NO.: N/A

TRUCK NO.: N/A

TRUCK MIXER CAP: N/A CU YD

LOAD SIZE: N/A CU YD

TIME BATCHED AT CENTRAL PLANT: N/A

REVOLUTIONS OF TRUCK MIXER OR AGITATOR: N/A

SAMPLE TAKEN AT: ☐ POINT OF PLACEMENT ☐ TRUCK ☑ MIXER

METHOD OF PLACING: ☑ PUMP ☐ TREMIE ☐ CHUTES ☐ BUGGIES ☐ CONVEYORS ☐ BUCKET

ENGINEER APPROVED MIX DESIGN SPECIFICATIONS

MAX WATER/CEMENT RATIO: N/A

SLUMP: N/A

AIR CONTENT: ~~3 to 5~~ %

UNIT WEIGHT: N/A PCF

METHOD OF PACKING SAMPLE FOR SHIPMENT: N/A

CURING AT JOB: See Remarks Below

METHOD: [✓ A] or B DAYS

LOCATION OF SAMPLED BATCH WITHIN THE PLACEMENT: + Water Added @ Placement. = N/A lbs.

SECOND STAGE GROUTING

MIX — QUANTITY PER BATCH

COMPLETE (A) AND (B)

(A) MOIST AGGREGATE — FINE AGGREGATE: N/A LBS; WATER CONTAINED: N/A LBS; COARSE AGGREGATE: N/A LBS; WATER CONTAINED: N/A LBS; ICE ADDED: N/A LBS; WATER ADDED: 20 LBS

(B) SURFACE DRY AGGREGATE — FINE AGGREGATE: N/A LBS; COARSE AGGREGATE: N/A LBS; TOTAL WATER USED: 20 LBS

QUANTITY OF CEMENT: 55 LBS

FLY ASH: N/A LBS

WATER/CEMENT RATIO: N/A

MEASURED SLUMP: N/A IN

AIR CONTENT: N/A %

UNIT WT: N/A PCF

TEMPERATURES, °F: AIR 48° CONCRETE 55°

REMARKS: A. Cyl's. prepared in temporary Lab., cured 1 Day in Plastic Bags & remaining 6 & 27 Days In Moist Room. (See Remarks Below)

ADMIXTURES: AIR ENTRAINING None N/A OZ; WATER REDUCING OZ

MATERIALS USED

CEMENT BRAND/TYPE: ~~Citadel/II~~

FLY ASH BRAND: N/A

COARSE AGGREGATE SIZE/SOURCE: NO. N/A A.H. Smith

FINE AGGREGATE SOURCE: ~~Kingsland Reach.~~

REMARKS: B. Cyl's prepared in Field cured 1 day in Insulated Box & Remaining 6 & 27 days in Moisture Room .

ADMIXTURE BRANDS: AIR ENTRAINING ~~M.B.V.R.~~; WATER REDUCING None

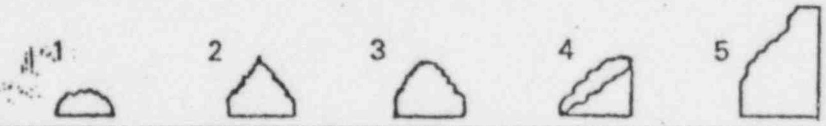
TESTS (TO BE FILLED IN BY TESTING LABORATORY)

TESTING LABORATORY: S&W Quality Control

LOCATION: Plant Site Q.C. Lab.

DATE RECEIVED: 1-8-76

TYPE OF BREAK



TEST RESULTS

SPECIMEN NO.	DIAMETER (in.)*	AREA (sq. in.)	BREAKING LOAD (lb.)	STRENGTH (psi)	TEST DATE	AGE AT TEST	TYPE OF BREAK
TAB 2+1 2A	2.00	4.00	9,000	2250	1-11	3	N/A
2B 2C	2.00 2.00	4.00 4.00	13,000 13,000	3250 3250	1-15	7	N/A
2D 2E	2.00 2.00	4.00 4.00	16,500 20,000	4125 5000	1-20	12	N/A
2F 2G	2.00 2.00	4.00 4.00	21,000 23,000	5250 5750	1-22	14	N/A
2H 2J	2.00 2.00	4.00 4.00	37,500 30,000	9375 7500	2-5	28	N/A

AVERAGE 3250 : 7 days / 4563 LBS PER SQ INCH AT 12 DAYS

AVERAGE 5500 14 days / 8438 LBS PER SQ INCH AT 28 DAYS

REMARKS

SIGNED 1-15-76 FOR TESTING LAB DATE

SIGNED 2-5-76 FOR TESTING LAB DATE

CONDITION OF SPECIMENS: SATISFACTORY ☐ UNSATISFACTORY ☐

FIG. 24.

LOCATION OF POUR

VALUE (PSI)

DATE SAMPLED

TEST NO.

7-DAY

28-DAY

TEST REPORT

STONE & WEBSTER ENGINEERING CORPORATION

THIS FORM MUST BE COMPLETELY FILLED IN AND MUST ACCOMPANY EACH TEST SPECIMEN SENT TO THE LABORATORY

INFORMATION OBTAINED AT STD AND DESIGN MIX INFORMATION

PROJECT: North Anna Power Station
PROJECT LOCATION: Louisa County, Virginia
JOB ORDER: 12180 .50
CONCRETE MIXED BY: ~~[illegible], Inc.~~
SPECIMENS TAKEN BY: W. G. WILLARD
SPECIFIED CONCRETE STRENGTH: 5000 PSI 28 DAYS
DESIGN MIX NO: MASTERFLOW 819 GROUT
DATE SAMPLED: 1-6-76
HOUR: 3:15 AM [✓ PM]
WEATHER: CLEAR
LOCATION OF CONCRETE PLACEMENT: TEST ROCK ANCHORS 2+4 NORTH RC #4
CONCRETE MIXED: ☐ CENTRAL ☐ TRANSIT MIXED N/A ☐ AGITATED ☐ NON AGITATED
TRUCK TICKET NO: N/A
TRUCK NO: N/A
TRUCK MIXER CAP.: N/A CU YD
LOAD SIZE: N/A CU YD
TIME OF MIXING AT CENTRAL PLANT: N/A MIN
REVOLUTIONS OF TRUCK MIXER OR AGITATOR: N/A
SAMPLE TAKEN AT: ☐ POINT OF PLACEMENT ☐ TRUCK ☑ MIXER
METHOD OF PLACING: ☑ PUMP ☐ TREMIE ☐ CHUTES ☐ BUGGIES ☐ CONVEYORS ☐ BUCKET

ENGINEER APPROVED MIX DESIGN SPECIFICATIONS
MAX. WATER/CEMENT RATIO: N/A
SLUMP: N/A IN
AIR CONTENT: ~~3 to 5~~ %
UNIT WEIGHT: N/A PCF
METHOD OF PACKING SAMPLE FOR SHIPMENT: N/A
CURING AT JOB: See Remarks Below
METHOD: [✓ A] or B DAYS
LOCATION OF SAMPLED BATCH WITHIN THE PLACEMENT: SECOND STAGE GROUTING
+ Water Added @ Placement. = N/A lbs.

GIVE QUANTITY PER BATCH

COMPLETE (A) AND (B)
(A) MOIST AGGREGATE: FINE AGGREGATE N/A LBS; WATER CONTAINED N/A LBS; COARSE AGGREGATE N/A LBS; WATER CONTAINED N/A LBS; ICE ADDED N/A LBS; WATER ADDED 20 LBS
(B) SURFACE DRY AGGREGATE: FINE AGGREGATE N/A LBS; COARSE AGGREGATE N/A LBS; TOTAL WATER USED 20 LBS
QUANTITY OF CEMENT: 55 LBS
FLY ASH: N/A LBS
WATER/CEMENT RATIO: N/A
MEASURED SLUMP: N/A IN
AIR CONTENT: N/A %
UNIT WT: N/A PCF
TEMPERATURES, °F: AIR 45° CONCRETE 50°
REMARKS: A. Cyl's. prepared in temporary Lab., cured 1 Day in Plastic Bags & remaining 6 & 27 days in Moist Room. (See Remarks Below)
ADMIXTURES: AIR ENTRAINING None OZ; WATER REDUCING N/A OZ

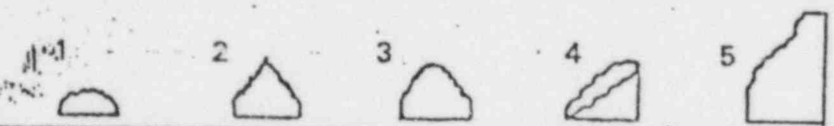
USED

CEMENT, BRAND/TYPE: ~~Citadel/II~~
FLY ASH BRAND: N/A
COARSE AGGREGATE SIZE/SOURCE: NO N/A A.H. Smith
FINE AGGREGATE SOURCE: ~~Kingsland Reach~~.
REMARKS: B. Cyl's prepared in Field cured 1 day in Insulated Box & Remaining 6 & 27 days in Moisture Room .
ADMIXTURE BRANDS: AIR ENTRAINING ~~M.B.V.R.~~; WATER REDUCING None

TO BE FILLED IN BY TESTING LABORATORY

TESTING LABORATORY: S&W Quality Control
LOCATION: Plant Site Q.C. Lab.
DATE RECEIVED:

TYPE OF BREAK



TEST RESULTS

SPECIMEN NO.	DIAMETER (in.)*	AREA (sq. in.)	BREAKING LOAD (lb)	STRENGTH (psi)	TEST DATE	AGE AT TEST	TYPE OF BREAK
TAB 29-4 1A	2.00	4.00	8000	2,000	1-9	3	N/A
1B 1C	2.00	4.00	12000 13,500	3375 3000	1-13	7	N/A
1D 1E	2.00	4.00	17,500 17,000	4375 4250	1-18	12	N/A
1F 1G	2.00	4.00	23,500 21,000	5875 5250	1-20	14	N/A
1H 1I	2.00	4.00	26,500 27,500	6625 6875	2-3	28	N/A

AVERAGE 3188 / 4313 LBS PER SQ INCH AT 7 days / 13 DAYS
AVERAGE 5563 / 6750 LBS PER SQ INCH AT 14 days / 28 DAYS

REMARKS:

CONDITION OF SPECIMENS: SATISFACTORY ☐ UNSATISFACTORY ☐

*CUBE SIZE

SIGNED: DATE 1-13-76 FOR TESTING LAB
SIGNED: DATE 2-3-76 FOR TESTING LAB

FIG. 25

JOB NO.
LOCATION OF POUR
VALUE (PSI)
VALUE (PSI)
VALUE (PSI)
DATE SAMPLED
TEST NO.
7-DAY
28-DAY

COMPRESSIVE STRENGTH
TEST REPORT

STONE & WEBSTER ENGINEERING CORPORATION

A 5045 13 *THIS FORM MUST BE COMPLETELY FILLED IN AND MUST ACCOMPANY EACH TEST SPECIMEN SENT TO THE LABORATORY*

PROJECT: North Anna Power Station
PROJECT LOCATION: Louisa County, Virginia
JOB ORDER: 12181 .50

SAMPLING (INFORMATION OBTAINED AT SITE AND DESIGN MIX INFORMATION)

CONCRETE MIXED BY: ~~Champion, Inc.~~
SPECIMENS TAKEN BY: WG WILLARD
SPECIFIED CONCRETE STRENGTH: 3000 PSI 28 DAYS
DESIGN MIX NO: MASTERBUILDERS 814 GROUT
DATE SAMPLED: 2-13-76
HOUR: 3:30 AM [✓ PM]
WEATHER: CLEAR & MILD
LOCATION OF CONCRETE PLACEMENT: TEST ROCK ANCHOR #5 EAST OF RC#9
CONCRETE MIXING: ☐ CENTRAL ☐ SHRINK ☒ AGITATED ☐ TRANSIT MIXED ☐ NON AGITATED
TRUCK TICKET NO: N/A
TRUCK NO: N/A
TRUCK MIXER CAP.: N/A CU YD
LOAD SIZE: N/A CU YD
TIME OF MIXING AT CENTRAL PLANT: N/A MIN
REVOLUTIONS OF TRUCK MIXER OR AGITATOR: N/A
SAMPLE TAKEN AT: ☒ POINT OF PLACEMENT ☐ TRUCK ☐ MIXER
METHOD OF PLACING: ☒ PUMP ☐ TREMIE ☐ CHUTES ☐ BUGGIES ☐ CONVEYORS ☐ BUCKET

ENGINEER APPPOVED MIX DESIGN SPECIFICATIONS
MAX. WATER/CEMENT RATIO: N/A
SLUMP: N/A IN
AIR CONTENT: ~~3 to 5~~ %
UNIT WEIGHT: N/A PCF
METHOD OF PACKING SAMPLE FOR SHIPMENT: N/A
CURING AT JOB: See Remarks Below
METHOD: [✓ A] or B DAYS
LOCATION OF SAMPLED BATCH WITHIN THE PLACEMENT: BEGINNING OF PLACEMENT
+ Water Added @ Placement. = N/A lbs.

MIX (GIVE QUANTITY PER BATCH)

COMPLETE (A) AND (B)
(A) MOIST AGGREGATE: FINE AGGREGATE N/A LBS; WATER CONTAINED N/A LBS; COARSE AGGREGATE N/A LBS; WATER CONTAINED N/A LBS; ICE ADDED N/A LBS; WATER ADDED 20 LBS
(B) SURFACE DRY AGGREGATE: FINE AGGREGATE N/A LBS; COARSE AGGREGATE N/A LBS; TOTAL WATER USED 20 LBS
QUANTITY OF CEMENT: 55 LBS
FLY ASH: N/A LBS
WATER/CEMENT RATIO: N/A
MEASURED SLUMP: N/A IN
AIR CONTENT: N/A %
UNIT WT: N/A PCF
TEMPERATURES, °F: AIR 70° CONCRETE 70°
REMARKS: A. Cyl's. prepared in temporary Lab., cured 1 Day in Plastic Bags & remaining 6 & 27 days in Moist Room. (See Remarks Below)
ADMIXTURES: AIR ENTRAINING None OZ; WATER REDUCING N/A OZ

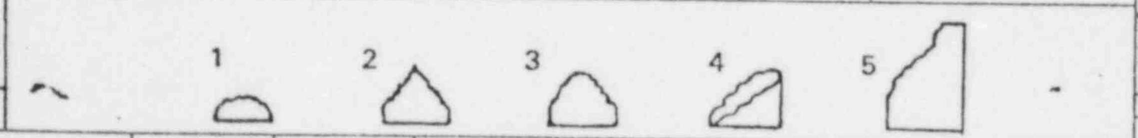
MATERIALS USED

CEMENT BRAND/TYPE: M.B. ~~Citadel/II~~ 814
FLY ASH BRAND: N/A
COARSE AGGREGATE SIZE/SOURCE: NO. N/A A.H. Smith
FINE AGGREGATE SOURCE: ~~Kingsland Reach.~~
REMARKS: B. Cyl's prepared in Field cured 1 day in Insulated Box & Remaining 6 & 27 days in Moisture Room .
ADMIXTURE BRANDS: AIR ENTRAINING ~~M.B.V.R.~~ N/A; WATER REDUCING None

TESTS (TO BE FILLED IN BY TESTING LABORATORY)

TESTING LABORATORY: S&W Quality Control
LOCATION: Plant Site Q.C. Lab.
DATE RECEIVED:

TYPE OF BREAK



TEST RESULTS

SPECIMEN NO.	DIAMETER (in.)*	AREA (sq in.)	BREAKING LOAD (lb.)	STRENGTH (psi)	TEST DATE	AGE AT TEST	TYPE OF BREAK
TA #5							
5A	2.00	4.00	9,000	2250	2-16	3	N/A
5B	2.00	4.00	14,000	3500			
5C	2.00	4.00	13,000	3250	2-20	7	N/A
5D	2.00	4.00	19,000	4750			
5E	2.00	4.00	20,500	5125	2-25	12	N/A
5F	2.00	4.00	22,000	5500			
5G	2.00	4.00	22,500	5625	2-27	14	N/A
5H	2.00	4.00	28,000	7000			
5I	2.00	4.00	28,500	7125	3-12	28	N/A

AVERAGE: LBS PER SQ INCH AT DAYS
AVERAGE: LBS PER SQ INCH AT DAYS

REMARKS:

SIGNED: W. J. Harris DATE: 2-16-76
FOR TESTING LAB
SIGNED: DATE: 3-12-76

CONDITION OF SPECIMENS: SATISFACTORY ☐ UNSATISFACTORY ☐

FIG. 26

JOB NO
LOCATION OF POUR
VALUE (PSI)
VALUE (PSI)
VALUE (PSI)
DATE SAMPLED
TEST NO
DAY
DAY

STONE & WEBSTER ENGINEERING CORPORATION

QUALITY CONTROL
NSPECTION REPORT

2040.26

JOB NUMBER: 12180

DATE: 4-27-76

SYSTEM(S) OR PART(S) NAME: T.A. #6

Poured 3-30-76

LOCATION(S): North Anna Power Station

Q.C. Lab

REFERENCE DOCUMENT(S):

DWG. NO. OR P.O. | ITEM | QTY. | DESCRIPTION(S) AND INSPECTION REMARK(S)

	PSI
3 days	2750 2500
6 days	3750 3875
13 days	5000 5375
14 days	5625 5625
28 days	6750 6625

FIG. 27.

QUALITY CONTROL INSPEC./ENG.: H. J. Harris

DATE: 4-27-76

PAGE 1 OF 1

STONE & WEBSTER ENGINEERING CORPORATION

QUALITY CONTROL
INSPECTION REPORT

JOB NUMBER 12180

DATE 5-25-76

SYSTEM(S) OR PART(S) NAME	LOCATION(S)	REFERENCE DOCUMENT(S)
1A # 7 Poured 5-10-76	North Anna Power Station Q. C. Lab.	

DWG. NO. OR P.O.	ITEM	QTY.	DESCRIPTION(S) AND INSPECTION REMARK(S)
			Compressive strength Results

	PSI	AVG
3 days	1250 1000	{1125}
7 days	3750 3625	{3688}
12 days	5250 5625	{5438}
14 days	6250 6250	{6250}
28 days	8375 8000	{8188}

FIG. 28

QUALITY CONTROL INSPEC/ENG.

DATE 5-25-76

PAGE 1 of 1

STONE & WEBSTER ENGINEERING CORPORATION

QUALITY CONTROL
NSPECTION REPORT

3040.28

JOB NUMBER	DATE
12180	3-17-76

SYSTEM(S) OR PART(S) NAME	LOCATION(S)	REFERENCE DOCUMENT(S)
Grout Encapsulation Test #1 Poured 2-18-76	North Anna Power Station Q.C. Lab	

WG. NO. OR P.O.	ITEM	QTY.	DESCRIPTION(S) AND INSPECTION REMARK(S)

Compressive Strength Test Results

	PSI
3 days -	2375
7 days -	5125 5125
12 days -	5750 5625
14 days -	7375 7750
28 days -	8500 9000

FIG. 29.

QUALITY CONTROL INSPEC./ENG.	DATE	PAGE
A. J. Harris	3-17-76	1 OF 1

COMPRESSIVE STRENGTH
TEST REPORT

STONE & WEBSTER ENGINEERING CORPORATION

THIS FORM MUST BE COMPLETELY FILLED IN AND MUST ACCOMPANY EACH TEST SPECIMEN SENT TO THE LABORATORY

SAMPLING (INFORMATION OBTAINED AT SITE) AND DESIGN MIX INFORMATION

PROJECT: North Anna Power Station
PROJECT LOCATION: Louisa County, Virginia
JOB ORDER: 12181 .50
CONCRETE MIXED BY: ~~Champion, Inc.~~
SPECIMENS TAKEN BY: WGWILLARD
SPECIFIED CONCRETE STRENGTH: 3000 PSI 28 DAYS
DESIGN MIX NO: GROUT
DATE SAMPLED: 3-1-76
HOUR: 3:15 AM ~~PM~~
WEATHER: CLEAR & MILD
LOCATION OF CONCRETE PLACEMENT: GROUT ENCAPSULATION TEST #2
CONCRETE MIXING: HANDMIX
☐ CENTRAL ☐ ~~[illegible]~~ AGITATED
☐ TRANSIT MIXED ☐ NON-AGITATED
TRUCK TICKET NO: N/A
TRUCK NO.: N/A
TRUCK MIXER CAP.: N/A CU YD
LOAD SIZE: N/A CU YD
TIME OF MIXING AT CENTRAL PLANT: N/A MIN
REVOLUTIONS OF TRUCK MIXER OR AGITATOR: N/A
SAMPLE TAKEN AT: ☒ POINT OF PLACEMENT ☐ TRUCK ☐ MIXER
METHOD OF PLACING: ☒ PUMP ☐ TREMIE ☐ CHUTES ☐ BUGGIES ☐ CONVEYORS ☐ BUCKET
ENGINEER APPROVED MIX DESIGN SPECIFICATIONS
MAX. WATER/CEMENT RATIO: N/A
SLUMP: N/A IN
AIR CONTENT: ~~3 to 5~~ N/A
UNIT WEIGHT: N/A PCF
METHOD OF PACKING SAMPLE FOR SHIPMENT: N/A
CURING AT JOB: See Remarks Below
METHOD [✓ A] or B DAYS
LOCATION OF SAMPLED BATCH WITHIN THE PLACEMENT: + Water Added @ Placement. = 0 lbs.
TOP QUARTER OF PVC PIPE

MIX GIVE QUANTITY PER BATCH

COMPLETE (A) AND (B)
(A) MOIST AGGREGATE: FINE AGGREGATE N/A LBS; WATER CONTAINED N/A LBS; COARSE AGGREGATE N/A LBS; WATER CONTAINED N/A LBS; ICE ADDED N/A LBS; WATER ADDED 42 LBS
(B) SURFACE DRY AGGREGATE: FINE AGGREGATE N/A LBS; COARSE AGGREGATE N/A LBS; TOTAL WATER USED 42 LBS
QUANTITY OF CEMENT: 94 LBS
FLY ASH: N/A LBS
WATER/CEMENT RATIO: N/A
MEASURED SLUMP: N/A IN
AIR CONTENT: N/A %
UNIT WT.: N/A PCF
TEMPERATURES, °F: AIR 70 CONCRETE 80
REMARKS: A. Cyl's. prepared in temporary Lab., cured 1 Day in Plastic Bags & remaining 6 & 27 days in Moist Room. (See Remarks Below)
ADMIXTURES: AIR ENTRAINING ~~None~~ N/A OZ; WATER REDUCING OZ

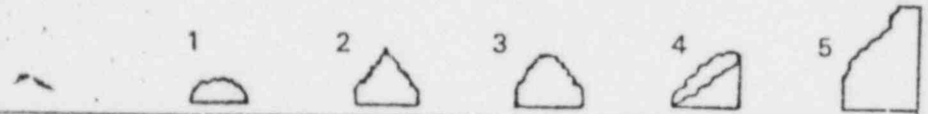
MATERIALS USED

CEMENT BRAND/TYPE: Citadel/II
FLY ASH BRAND: N/A
COARSE AGGREGATE SIZE/SOURCE: NO. N/A A.H. Smith
FINE AGGREGATE SOURCE: ~~Kingsland Reach.~~
REMARKS: B. Cyl's prepared in Field cured 1 day in Insulated Box & Remaining 6 & 27 days in Moisture Room .
ADMIXTURE BRANDS: AIR ENTRAINING ~~M.B.V.R.~~ N/A; WATER REDUCING None

TESTS (TO BE FILLED IN BY TESTING LABORATORY)

TESTING LABORATORY: S&W Quality Control
LOCATION: Plant Site Q.C. Lab.
DATE RECEIVED: 3-1-76

TYPE OF BREAK



TEST RESULTS

SPECIMEN NO.	DIAMETER (in)*	AREA (sq. in)	BREAKING LOAD (lb)	STRENGTH (psi)	TEST DATE	AGE AT TEST	TYPE OF BREAK
#2 2A	2.00	4.00	10,000	2500	3-4	3	N/A
2B 2C	2.00	4.00 4.00	13,000 13,000	3250 3250	3-5 ~~3-8~~	4 ~~5~~	N/A
2D 2E	2.00	4.00 4.00	19,000 19,500	4750 4875	3-8	7	N/A
2F 2G	2.00	4.00 4.00	22,000 22,000	5500 5500	3-13	12	N/A
2H 2I	2.00	4.00 4.00	27,500 27,000	6875 6750	3-15	14	N/A

AVERAGE LBS PER SQ INCH AT DAYS
AVERAGE LBS PER SQ INCH AT DAYS

REMARKS

SIGNED W. J. Harris DATE 3-4-76 FOR TESTING LAB
SIGNED DATE

CONDITION OF SPECIMENS SATISFACTORY ☐ UNSATISFACTORY ☐

FIG. 30

JOB NO
LOCATION OF POUR
VALUE (PSI) 33 34 35
VALUE (PSI) 27 28 29
VALUE (PSI) 21 22 23
DATE SAMPLED 14 15 16 17
TEST NO. 6 7 8 9 1
DAY DAY
72 73

STONE & WEBSTER ENGINEERING CORPORATION

QUALITY CONTROL
NSPECTION REPORT

2040.26

JOB NUMBER: 12180

DATE: 5-25-76

SYSTEM(S) OR PART(S) NAME	LOCATION(S)	REFERENCE DOCUMENT(S)
Grout "Encapsulation" (COLUMN.) Poured 5-11-76	North Anna Power Station	

DWG. NO. OR P.O.	ITEM	QTY.	DESCRIPTION(S) AND INSPECTION REMARK(S)

	PSI	
3 days	1750 1625	{1688}
7 days	4250 3875	{4063}
13 days	6000 5750	{5875
14 days	6500 6500	{6500}

CUBES TAKEN FROM GROUT USED IN GROUT COLUMN TEST.

FIG. 31 A.

QUALITY CONTROL INSPEC./ENG.: W. J. Harris

DATE: 5-25-76

PAGE: 1 of 1

STONE & WEBSTER ENGINEERING CORPORATION

UALITY CONTROL
NSPECTION REPORT
9040.28

JOB NUMBER	DATE
12180	5-25-76

SYSTEM(S) OR PART(S) NAME	LOCATION(S)	REFERENCE DOCUMENT(S)
Grout Test cylinders from PVC pipe POURED 5-11-76	North Anna Power Station Q.C. Lab	

DWG. NO. OR P.O. | ITEM | QTY. | DESCRIPTION(S) AND INSPECTION REMARK(S)

3 days old	PSI
# 21	557
11	518
31	836
41	955
51	1274

NOTE: THESE SPECIMENS CONSISTED OF PORTLAND TYPE II CEMENT AND ½% by WT. OF SIKA INTRAPLAST "N"

7 days old	
# 12	955
22	955
32	~~23~~ 1871
42	1911
52	2548
62	2667

12 days	
# 13	1592
23	1613
33	2389
43	2946
53	3782
63	3822

16 days	
# 14	1990
24	2150
34	3025

17 days	
# 44	3583
54	3742
64	4061

40'-8"	3742
35'-4 35'-8	3344
2'-4 2'-8	398

NOTE: THESE SPECIMENS WERE NOTICABLY SOFTER THAN THE REST. ALSO, TWO SPECIMENS BROKE IN HALF WHILE TRYING TO APPLY CAPPING COMPOUND (# 3'-4 To 3'-8 & 2'-0 To 2'-4), DUE TO SOFTNESS.

FIG. 31B

QUALITY CONTROL INSPEC./ENG.	DATE	PAGE
J. Harris	5-25-76	1 OF 1

NORTH ANNA POWER STATION
UNITS 3 & 4
ROCK ANCHOR TEST PROGRAM

RESULTS OF CRUSHING TESTS ON NX SIZE CORE SAMPLES OF GROUT FROM GROUT TEST HOLE #1

DATE TESTED	DEPTH OF SAMPLE BELOW GROUND	MAX. LOAD ON SAMPLE	CRUSHING STRENGTH, P.S.I.	AGE DAYS
1/9/76	20 ft.	15,000 Lbs.	4777	2-1/2
(9:30 A.M.)	25 ft.	15,500 Lbs.	4936	
1/9/76	25 ft.	17,500 Lbs.	5573	3
(3:00 P.M.)	30 ft.	17,500 Lbs.	5573	
1/12/76	30 ft.	20,000 Lbs.	6369	6
	35 ft.	19,500 Lbs.	6210	
1/19/76	35 ft.	21,000 Lbs.	6688	13
	40 ft.	21,500 Lbs.	6847	
1/20/76	40 ft.	24,000 Lbs.	7643	14
	45 ft.	23,000 Lbs.	7325	

Grout was MasterFlow 814 mixed with 2 gallons water/55# bag.

Report by North Anna Power Station Quality Control Laboratory dated 1/22/76 by W. J. Harris.

Job No. 12180-12181
Reference Documents NAS 30163 dated 12/1/75.

FIG. 32.

GENERAL ARRANGEMENT FOR TEST SET-UP FOR THE V.S.L. ANCHORS

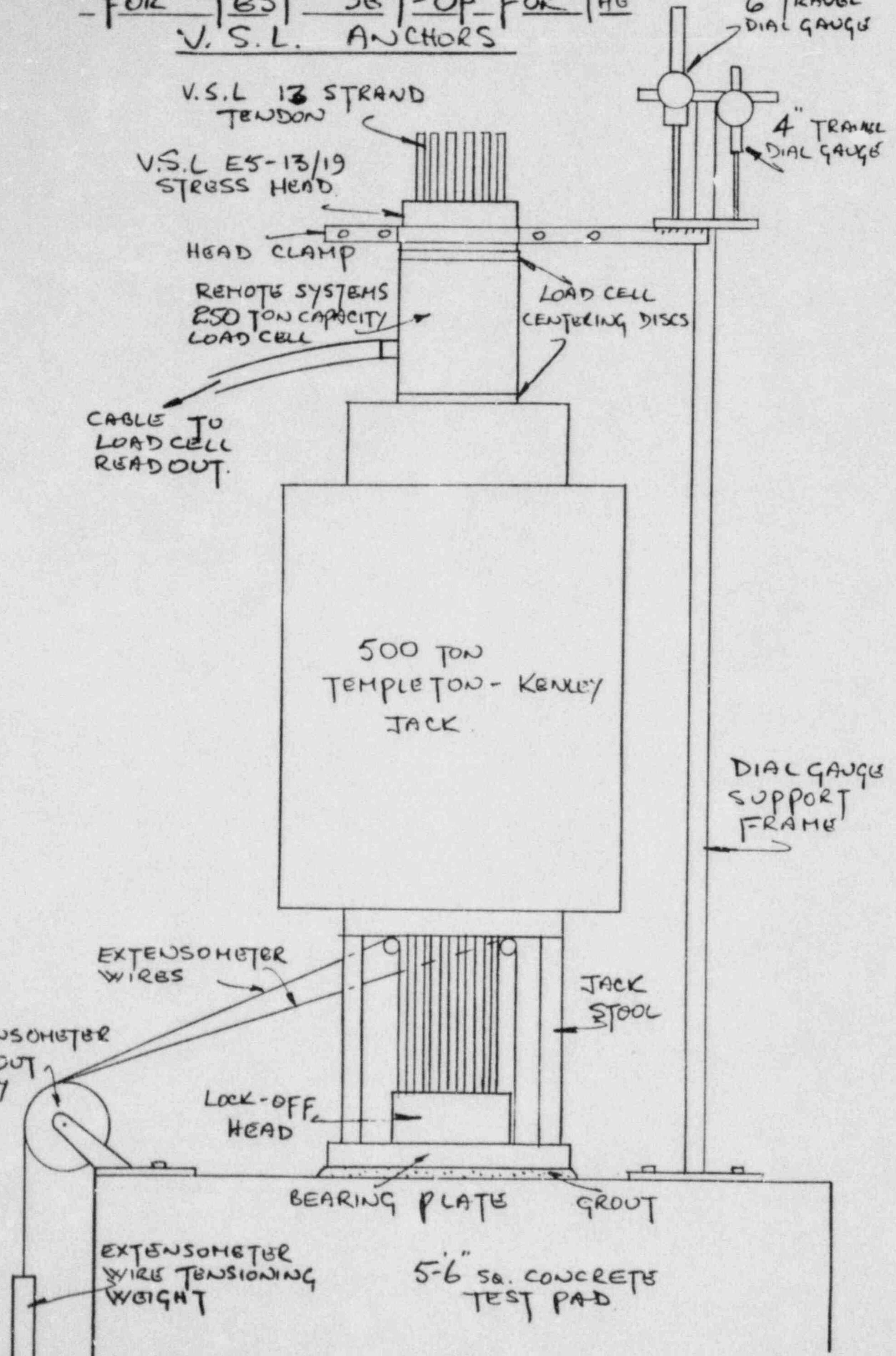


FIG. 33.

GENERAL ARRANGEMENT OF TEST SET-UP FOR THE PRESCON ANCHORS

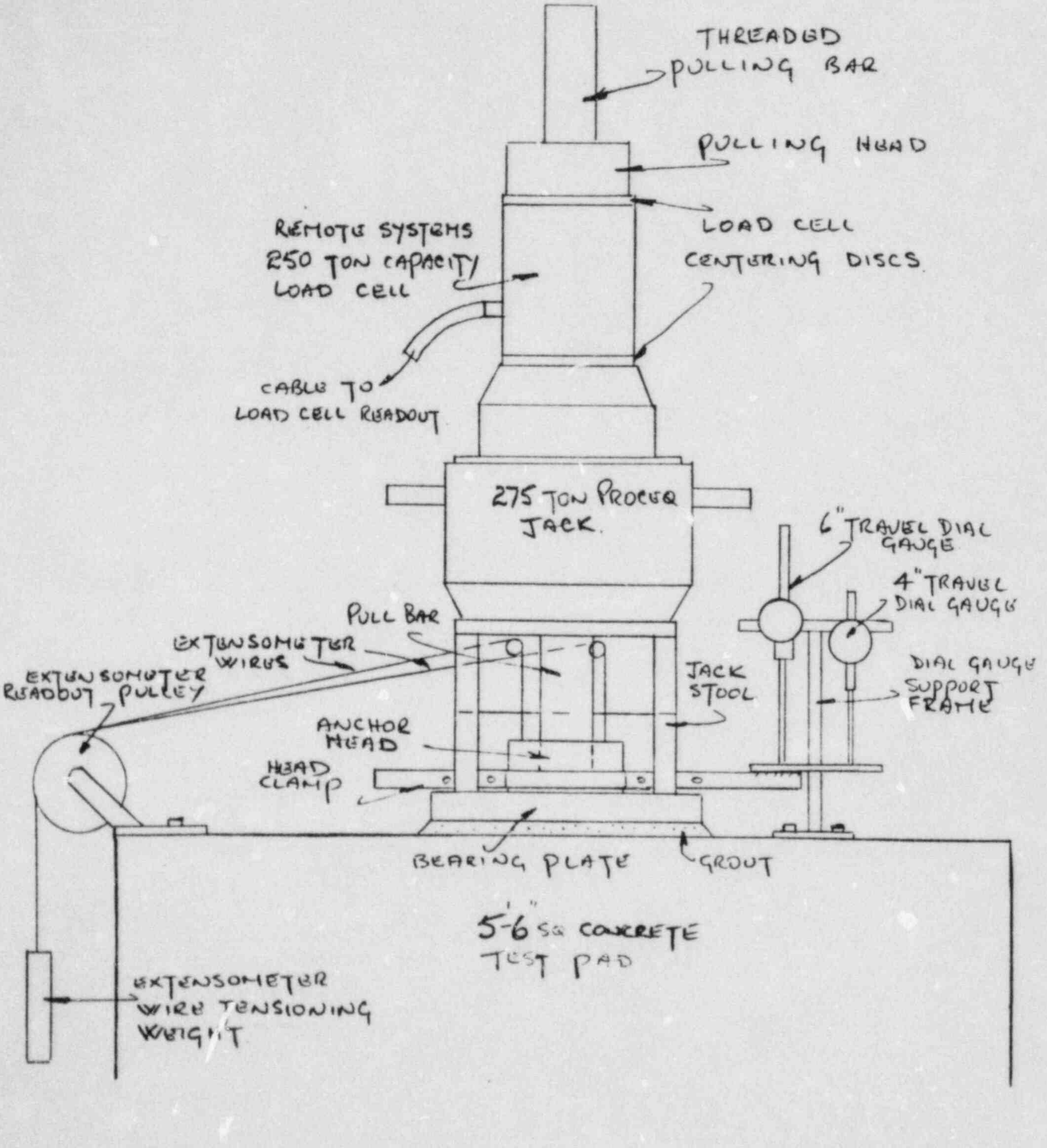
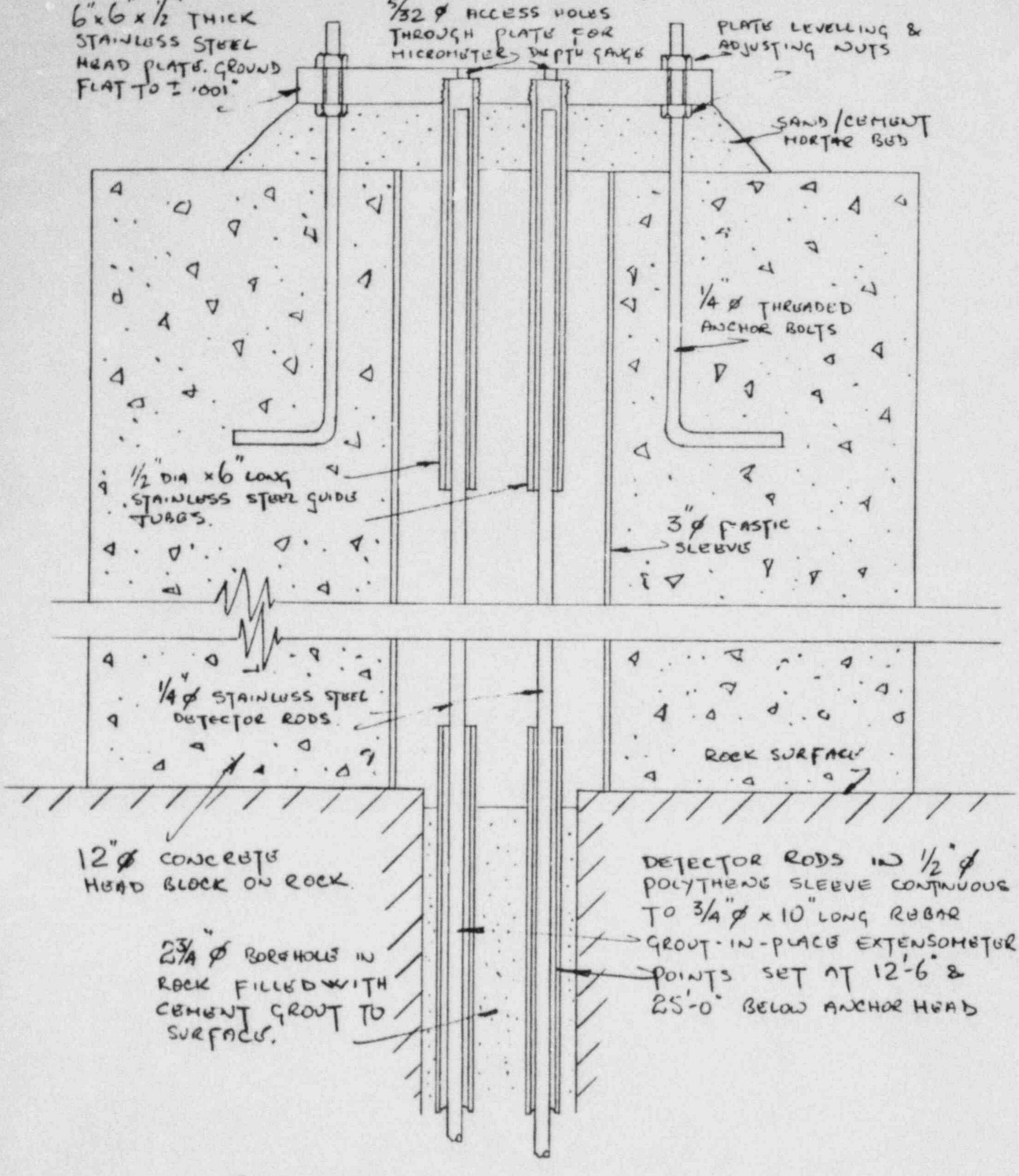


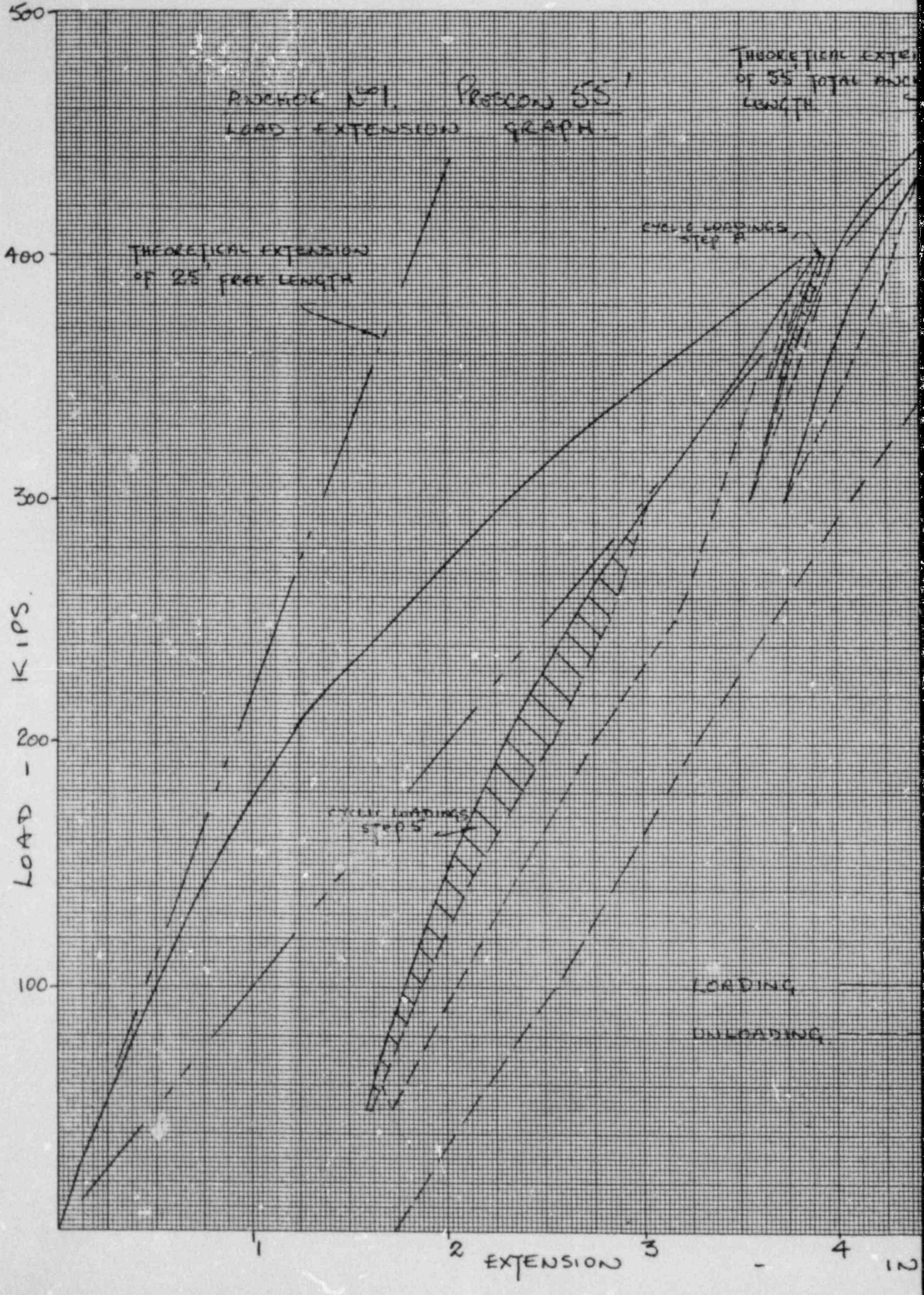
FIG. 34.



GENERAL ARRANGEMENT OF ROCK EXTENSOMETER INSTALLATION.

SCALE: ½ FULL SIZE

FIG. 35.



ANCHOR No1. PRESCON 55'
LOAD-EXTENSION GRAPH.

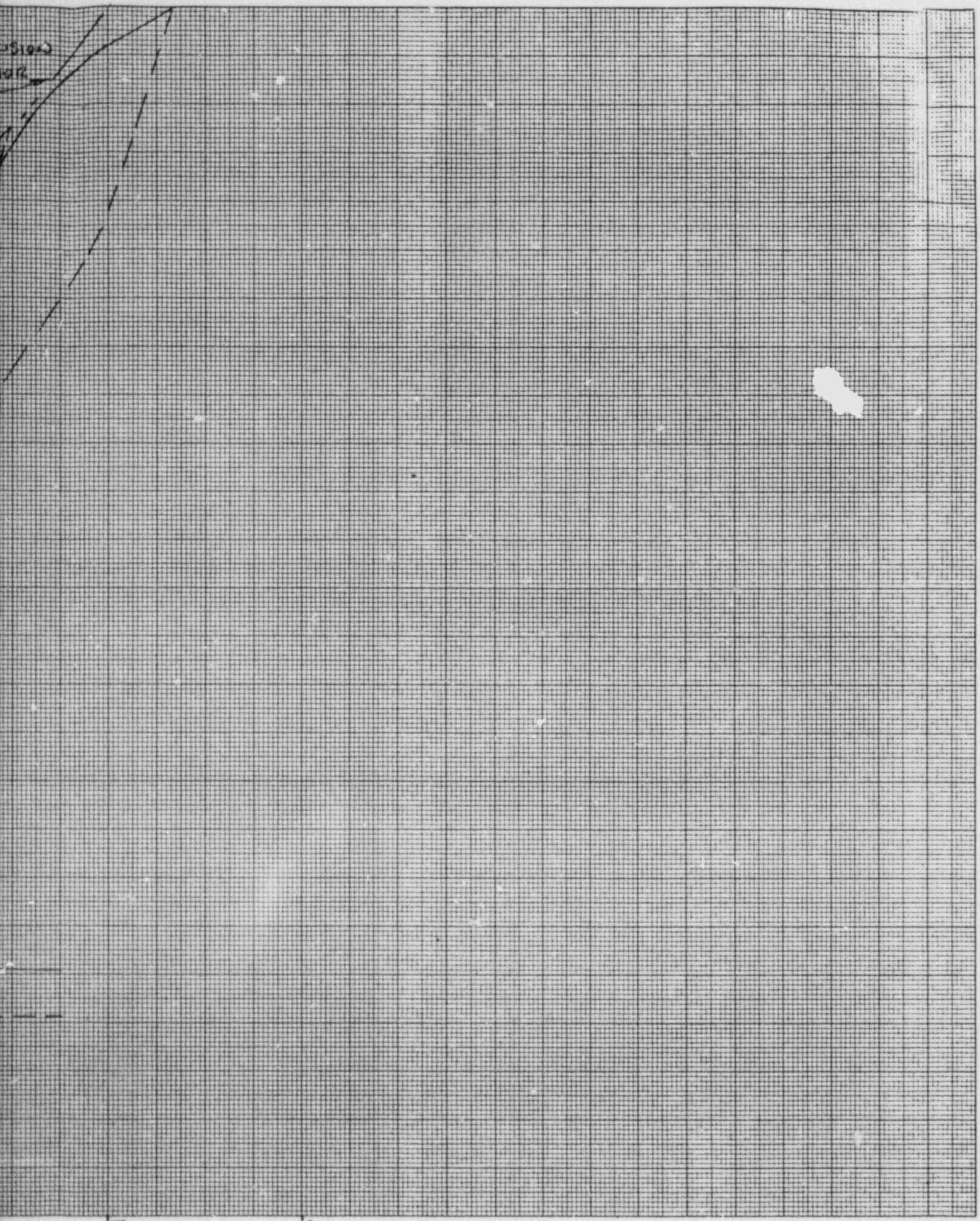


FIG. 36

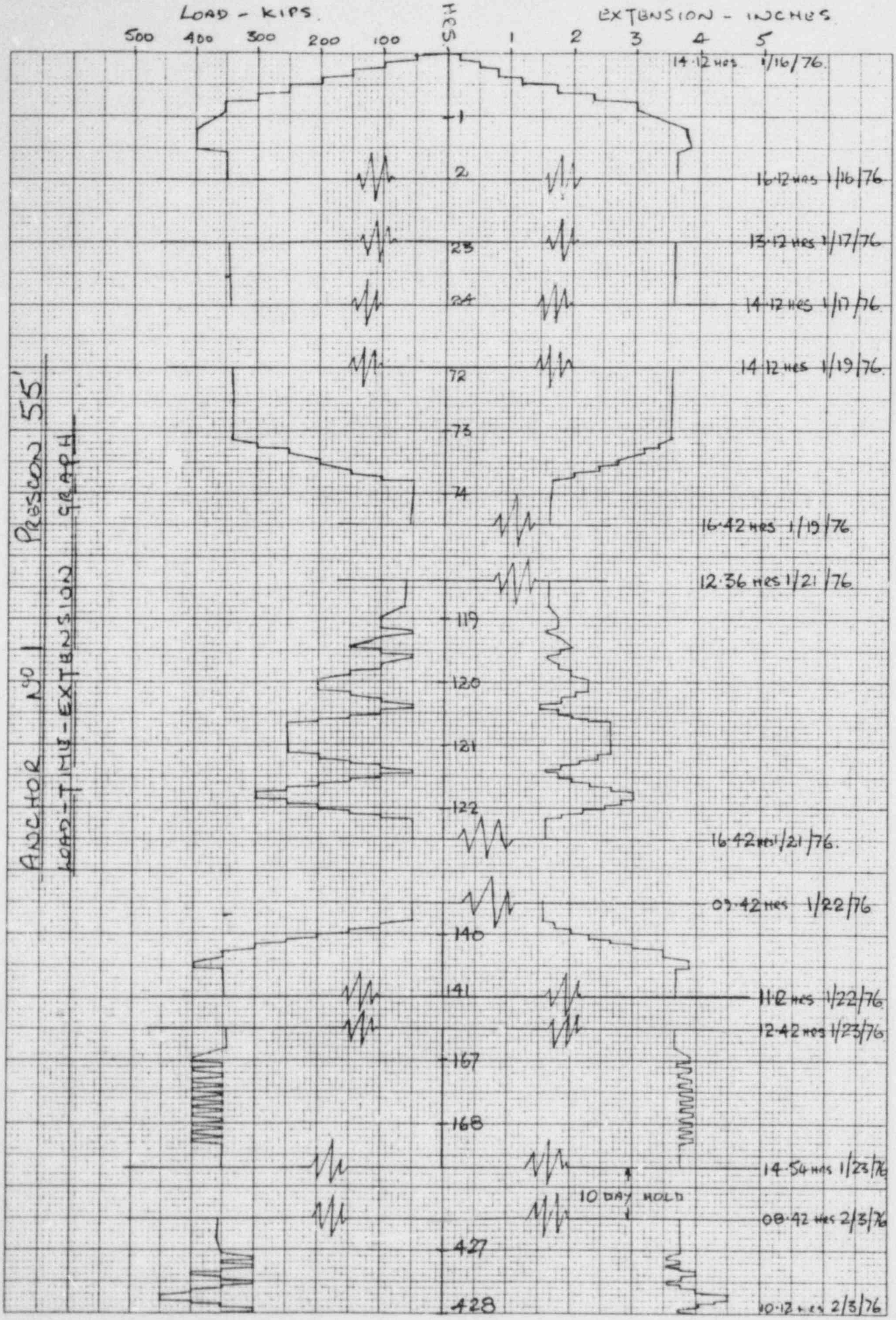
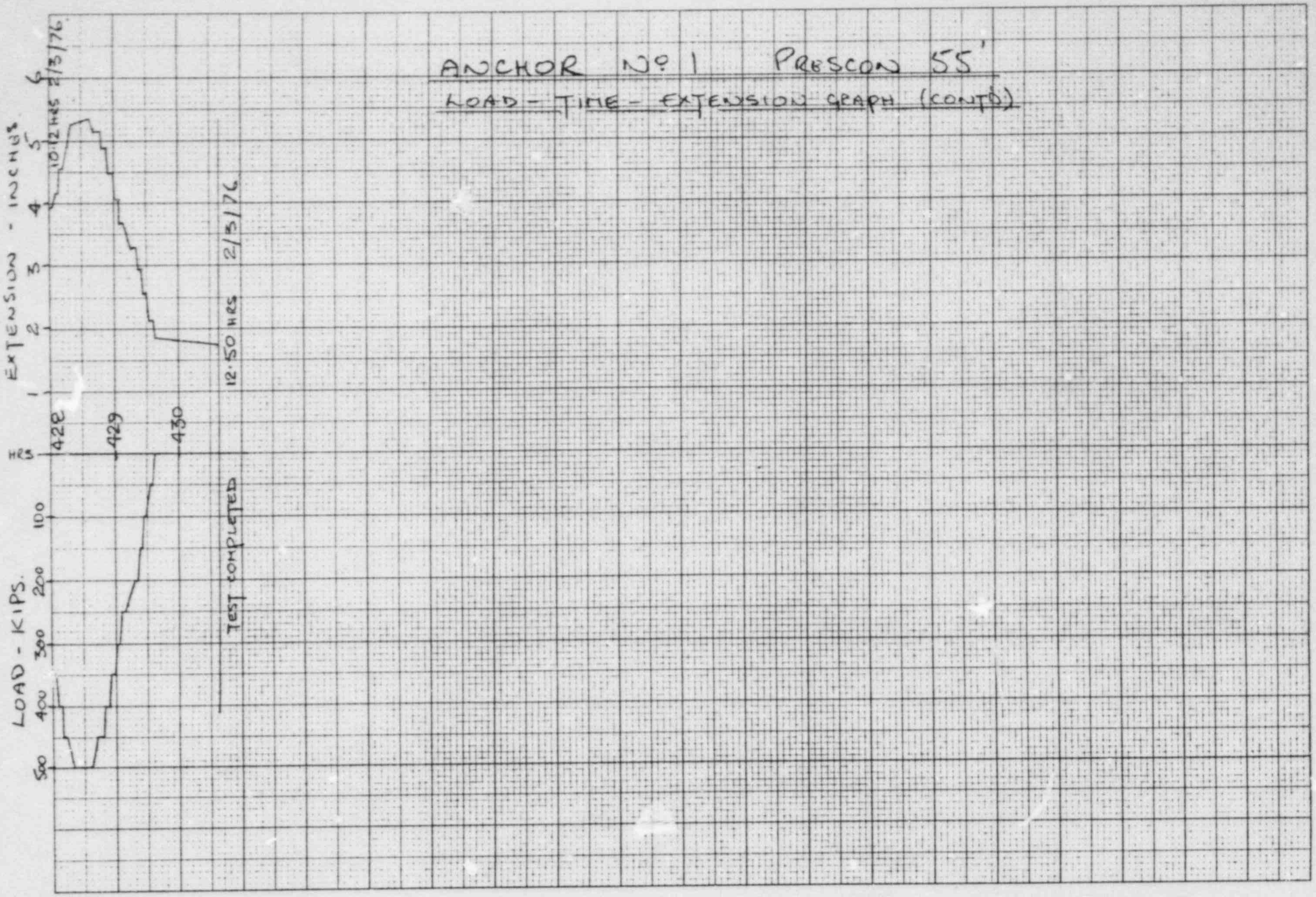

ANCHOR No 1 PRESCON 55'
LOAD-TIME-EXTENSION GRAPH
LOAD - KIPS
500
400
300
200
100
HRS
EXTENSION - INCHES
1
2
3
4
5
14·12 HRS 1/16/76
1
2
16·12 HRS 1/16/76
23
13·12 HRS 1/17/76
24
14·12 HRS 1/17/76
72
14 12 HRS 1/19/76
73
74
16·42 HRS 1/19/76
12·36 HRS 1/21/76
119
120
121
122
16·42 HRS 1/21/76
09·42 HRS 1/22/76
140
141
11·12 HRS 1/22/76
12·42 HRS 1/23/76
167
168
14·54 HRS 1/23/76
10 DAY HOLD
08·42 HRS 2/3/76
427
428
10·12 HRS 2/3/76

FIG. 37

FIG. 37A



ANCHOR No 1 PRESCON 55'

LOAD - TIME - EXTENSION GRAPH (CONTD)

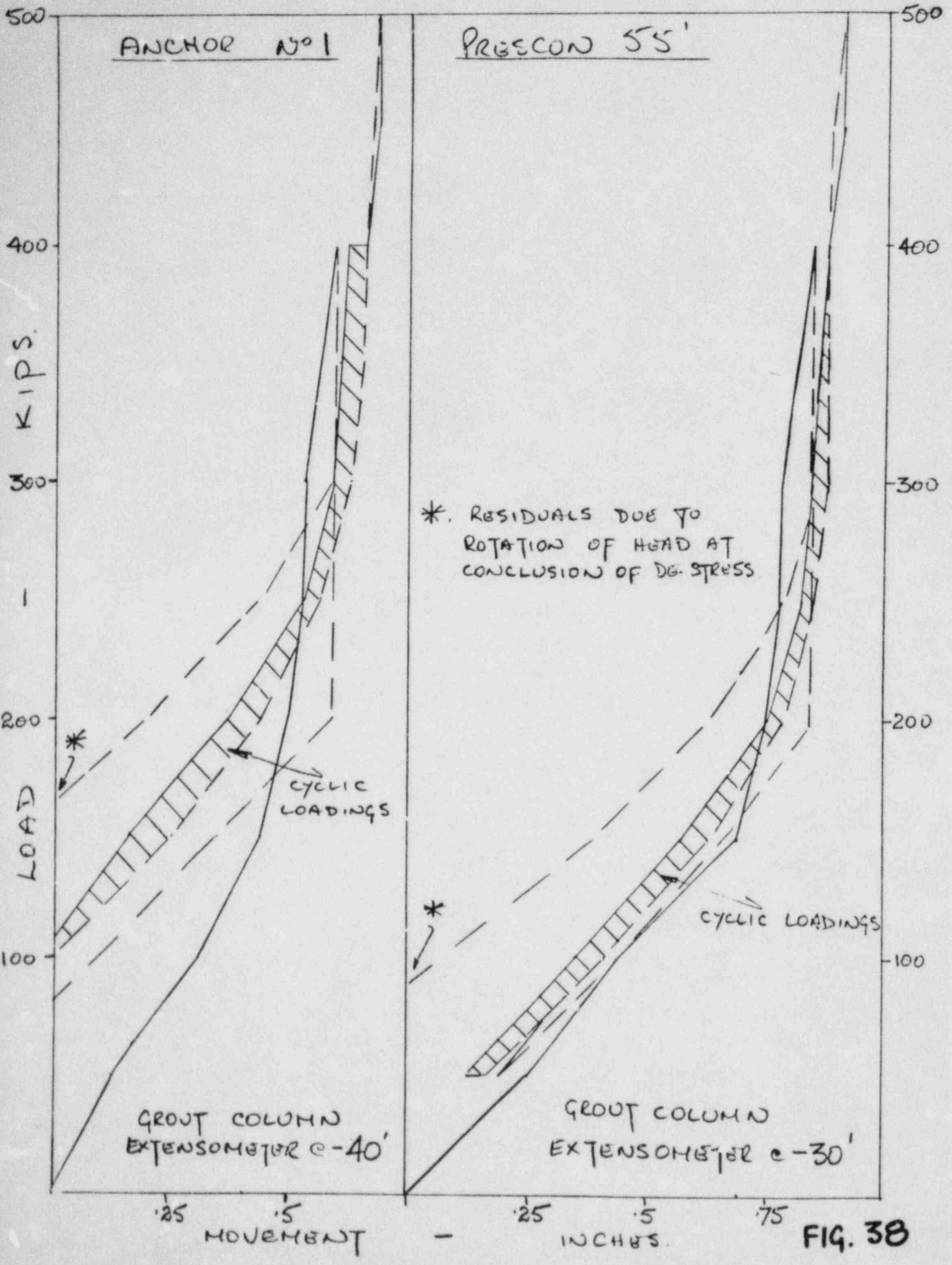

ANCHOR Nº1
PRESCON 55'
LOAD - KIPS.
500
400
300
200
100
500
400
300
200
100
*
*. RESIDUALS DUE TO ROTATION OF HEAD AT CONCLUSION OF DG. STRESS
CYCLIC LOADINGS
*
CYCLIC LOADINGS
GROUT COLUMN EXTENSOMETER @ -40'
GROUT COLUMN EXTENSOMETER @ -30'
·25
·5
·25
·5
·75
MOVEMENT - INCHES.

FIG. 38

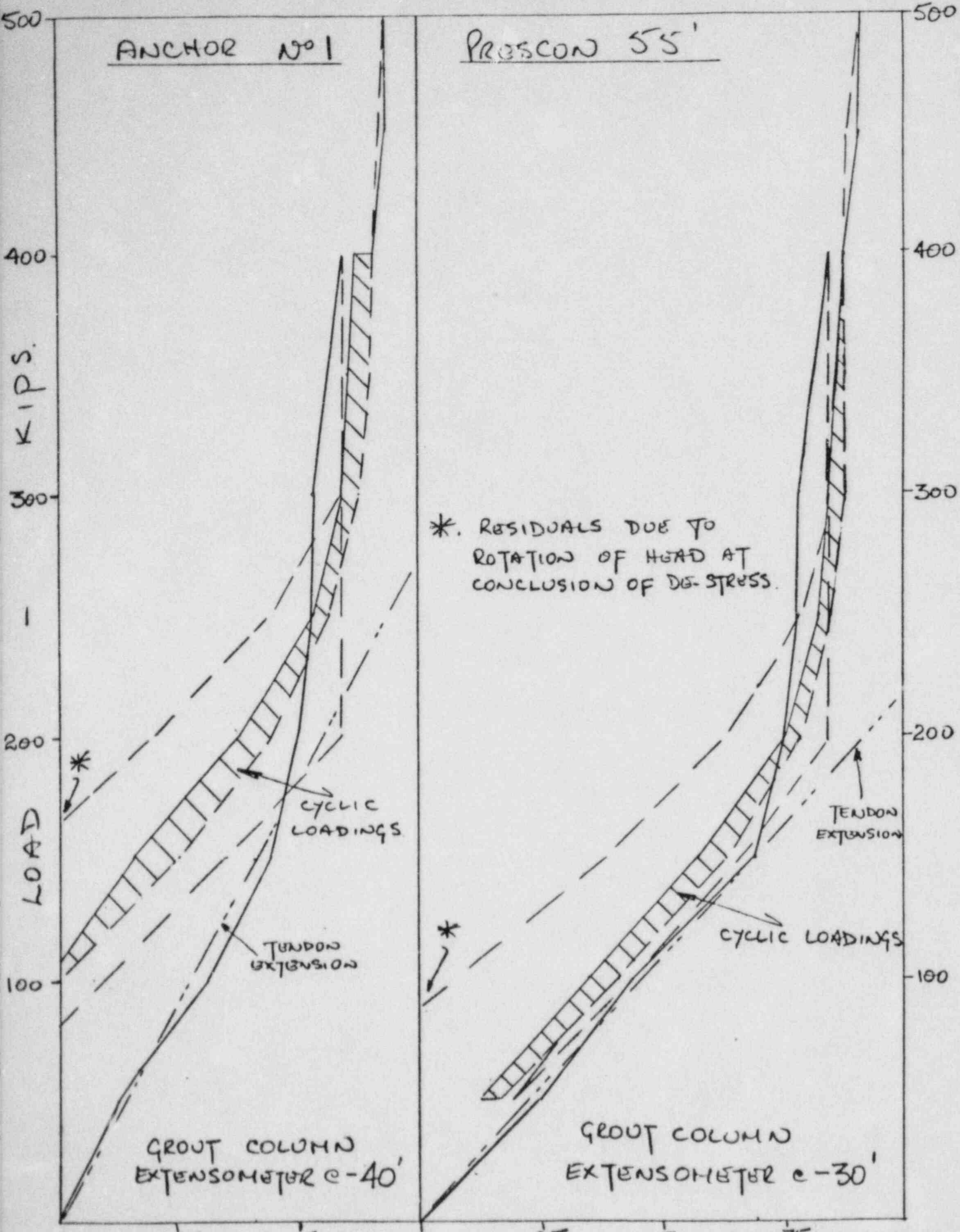
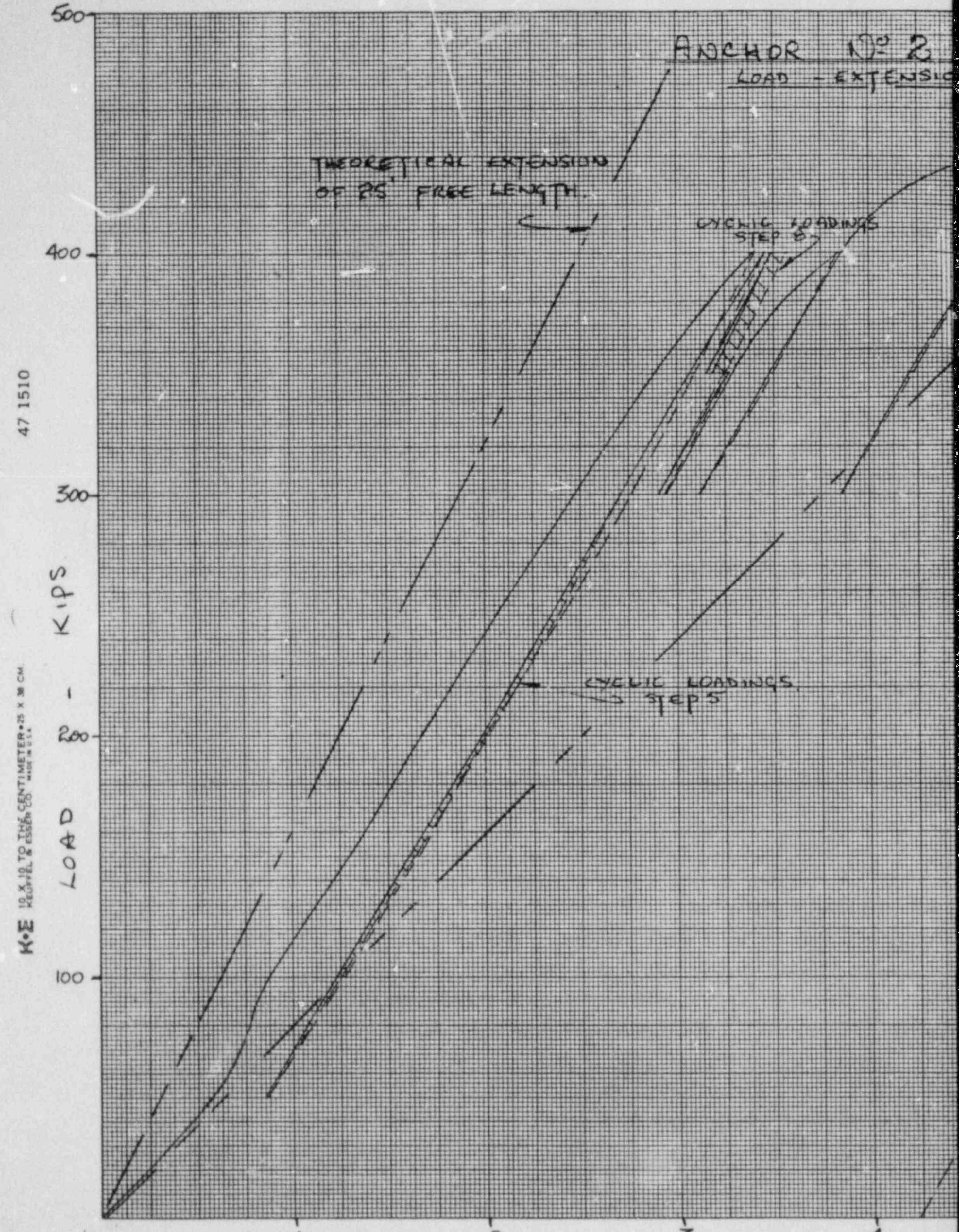

ANCHOR No 1
PRESCON 55'
LOAD - KIPS.
500
400
300
200
100
*
CYCLIC LOADINGS
TENDON EXTENSION
*. RESIDUALS DUE TO ROTATION OF HEAD AT CONCLUSION OF DE-STRESS.
*
CYCLIC LOADINGS
TENDON EXTENSION
GROUT COLUMN EXTENSOMETER @ -40'
GROUT COLUMN EXTENSOMETER @ -30'
·25
·5
·25
·5
·75
MOVEMENT - INCHES.

FIG. 38A


ANCHOR No 2
LOAD - EXTENSIO
THEORETICAL EXTENSION OF 25' FREE LENGTH
CYCLIC LOADINGS STEP 8
CYCLIC LOADINGS STEP 5
LOAD - KIPS
500
400
300
200
100
EXTENSION
1
2
3
4
47 1510
K•E 10 X 10 TO THE CENTIMETER • 25 X 38 CM. KEUFFEL & ESSER CO. MADE IN U.S.A.

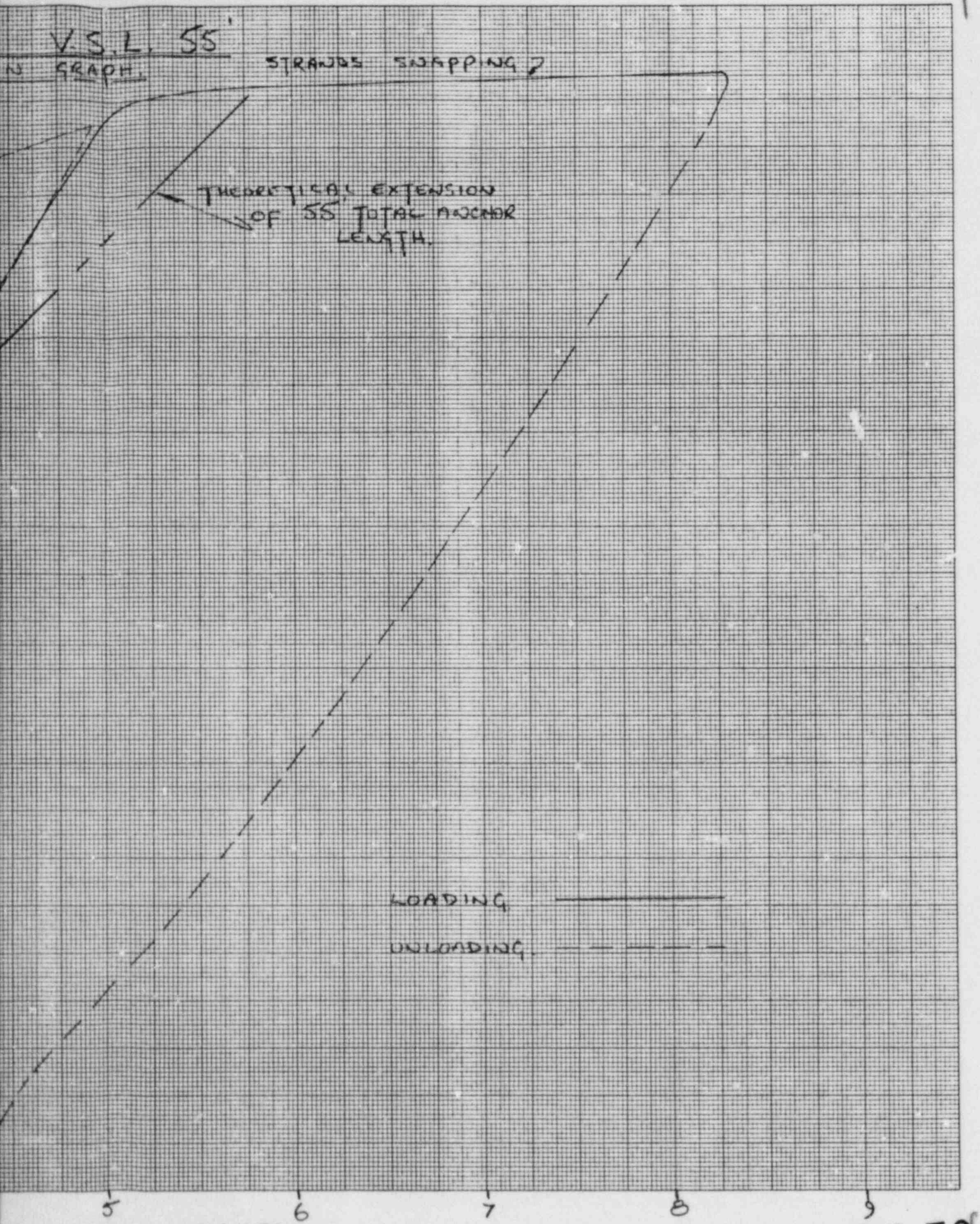
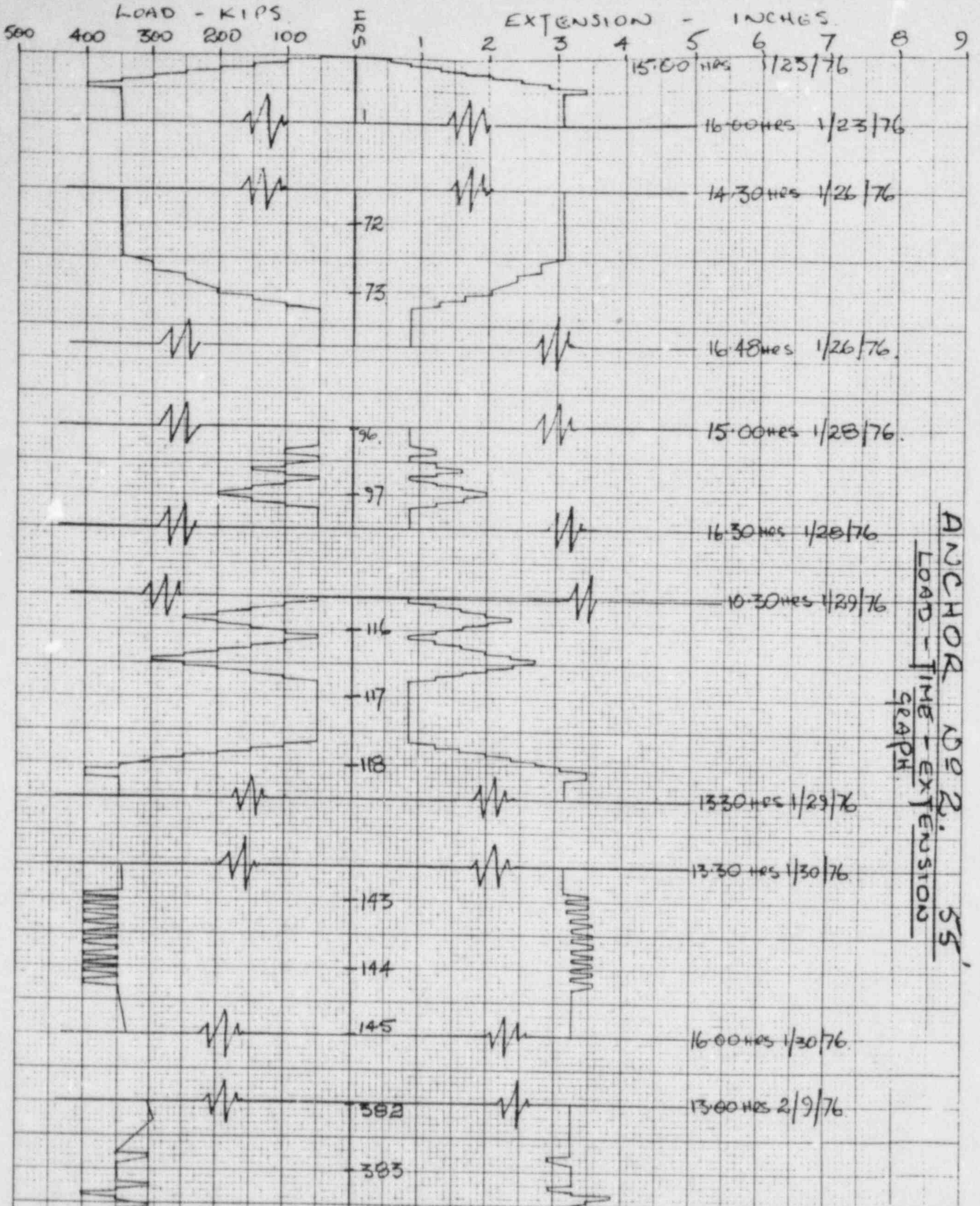

V.S.L. 55'
N GRAPH.
STRANDS SNAPPING ?
THEORETICAL EXTENSION OF 55' TOTAL ANCHOR LENGTH.
LOADING
UNLOADING
5
6
7
8
9
INCHES.

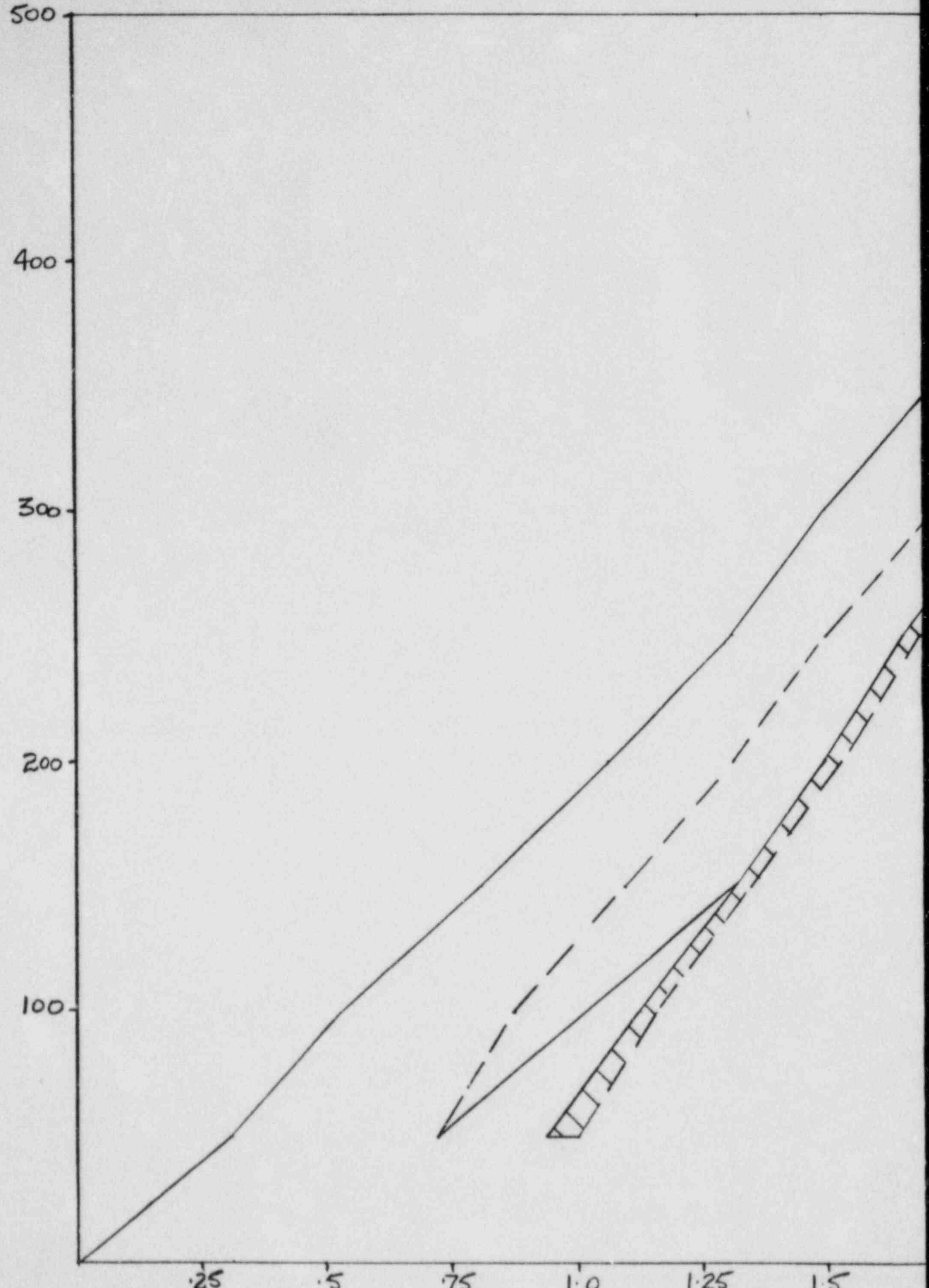
FIG. 39



ANCHOR No 2. 55'

LOAD - TIME - EXTENSION GRAPH

FIG. 40


500
400
300
200
100
·25
·5
·75
1·0
1·25
1·5
MOVEMENT

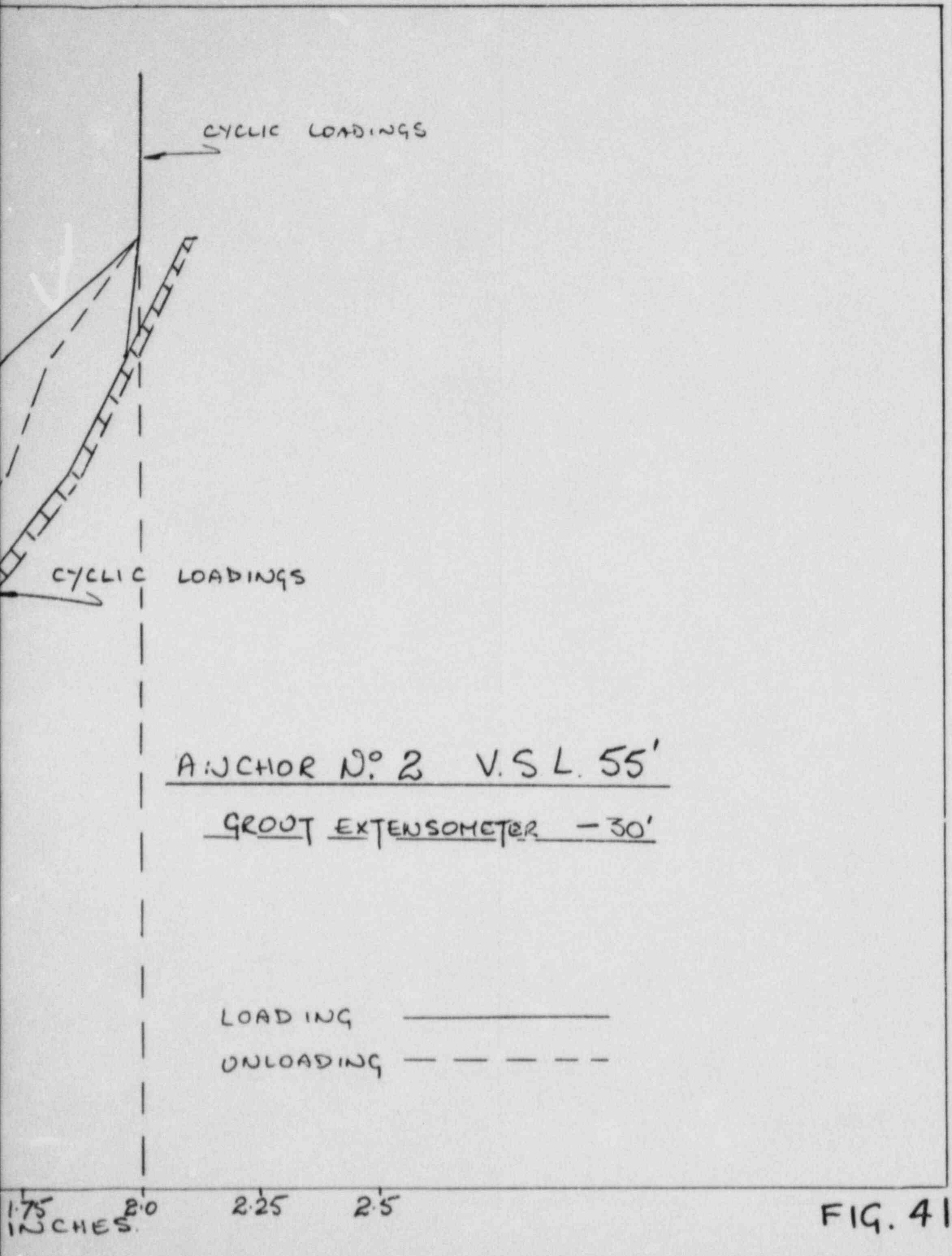
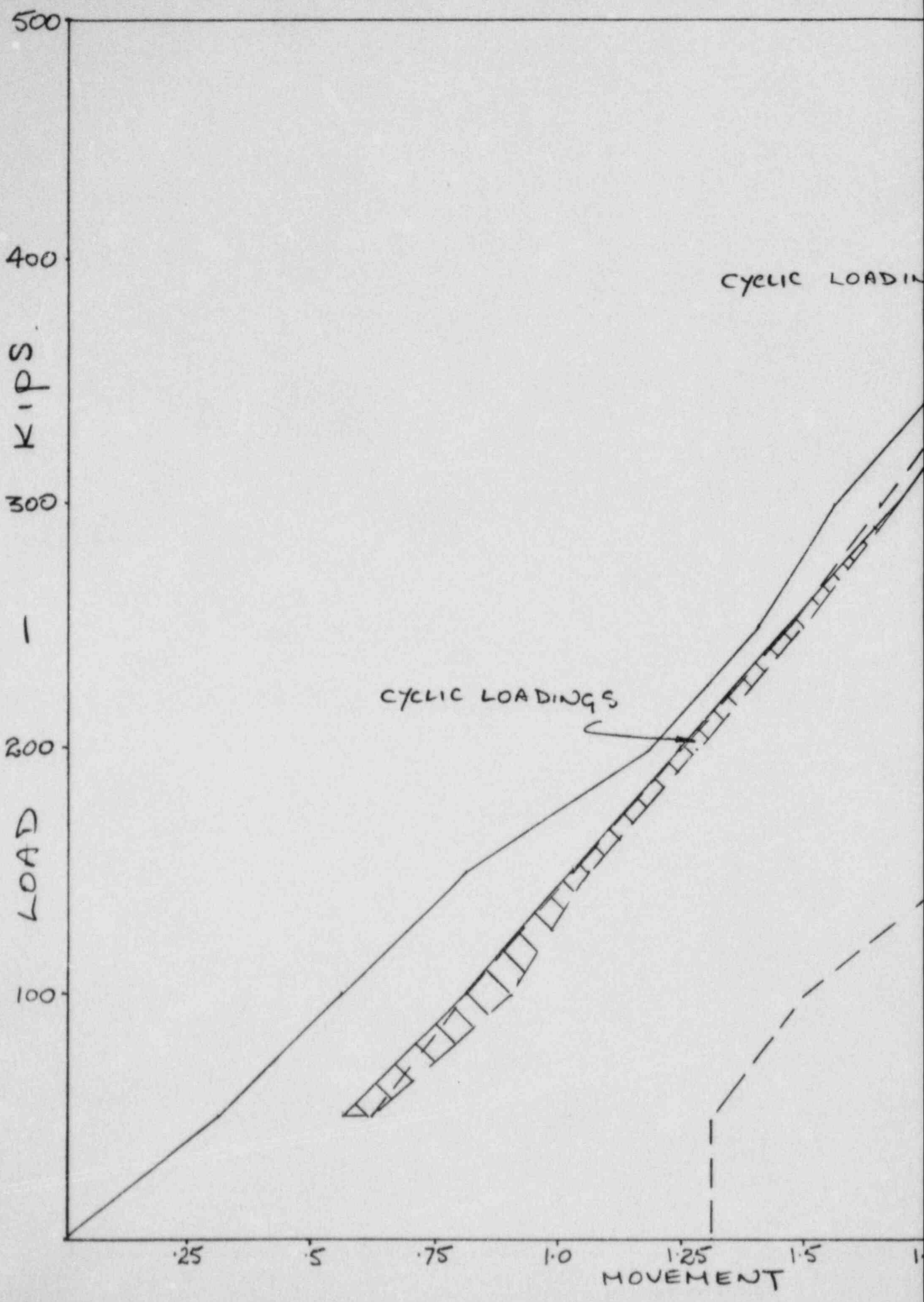
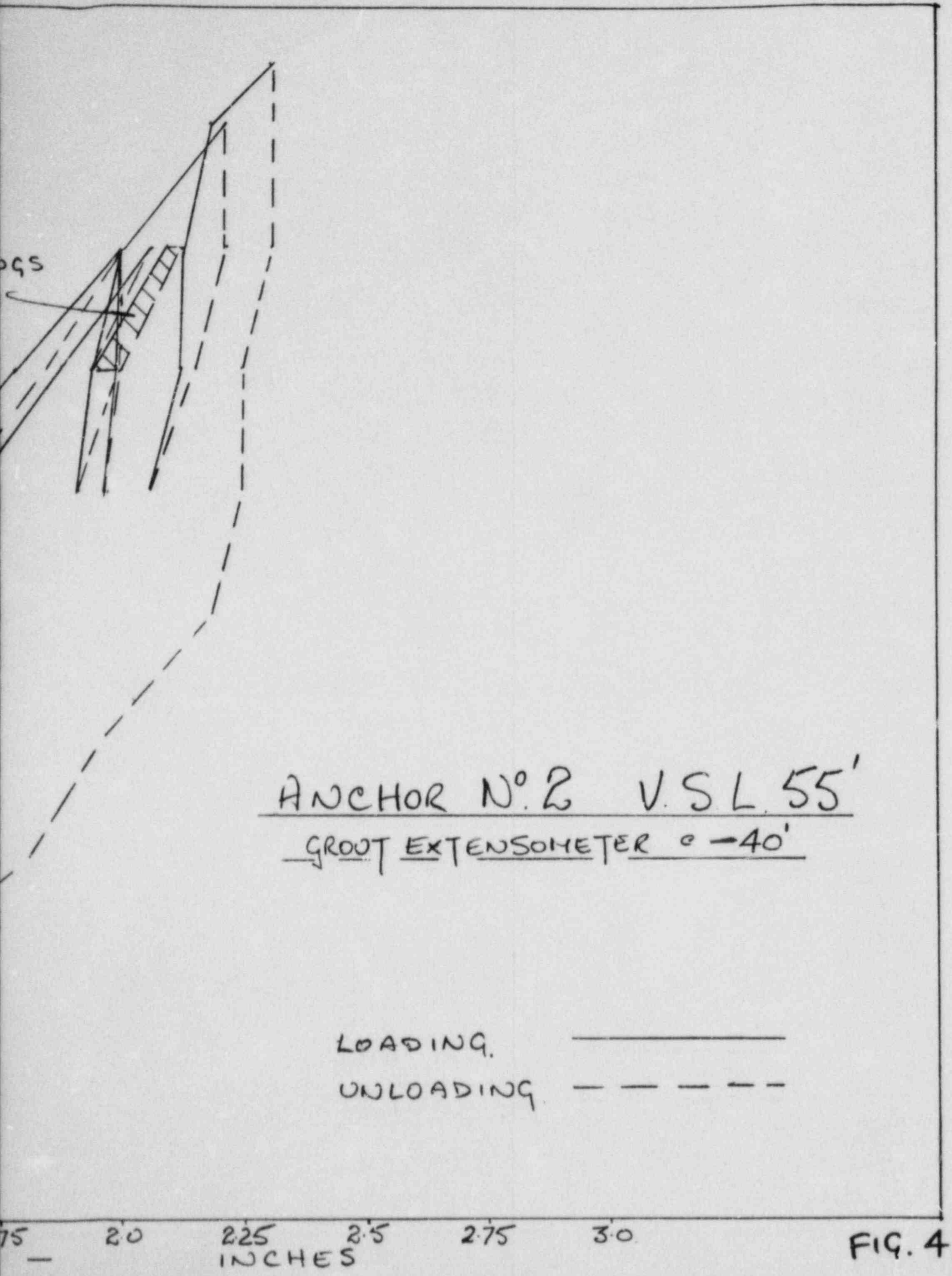

CYCLIC LOADINGS
CYCLIC LOADINGS
ANCHOR No. 2 V.S.L. 55'
GROUT EXTENSOMETER – 30'
LOADING
UNLOADING
1.75
2.0
2.25
2.5
INCHES.

FIG. 41

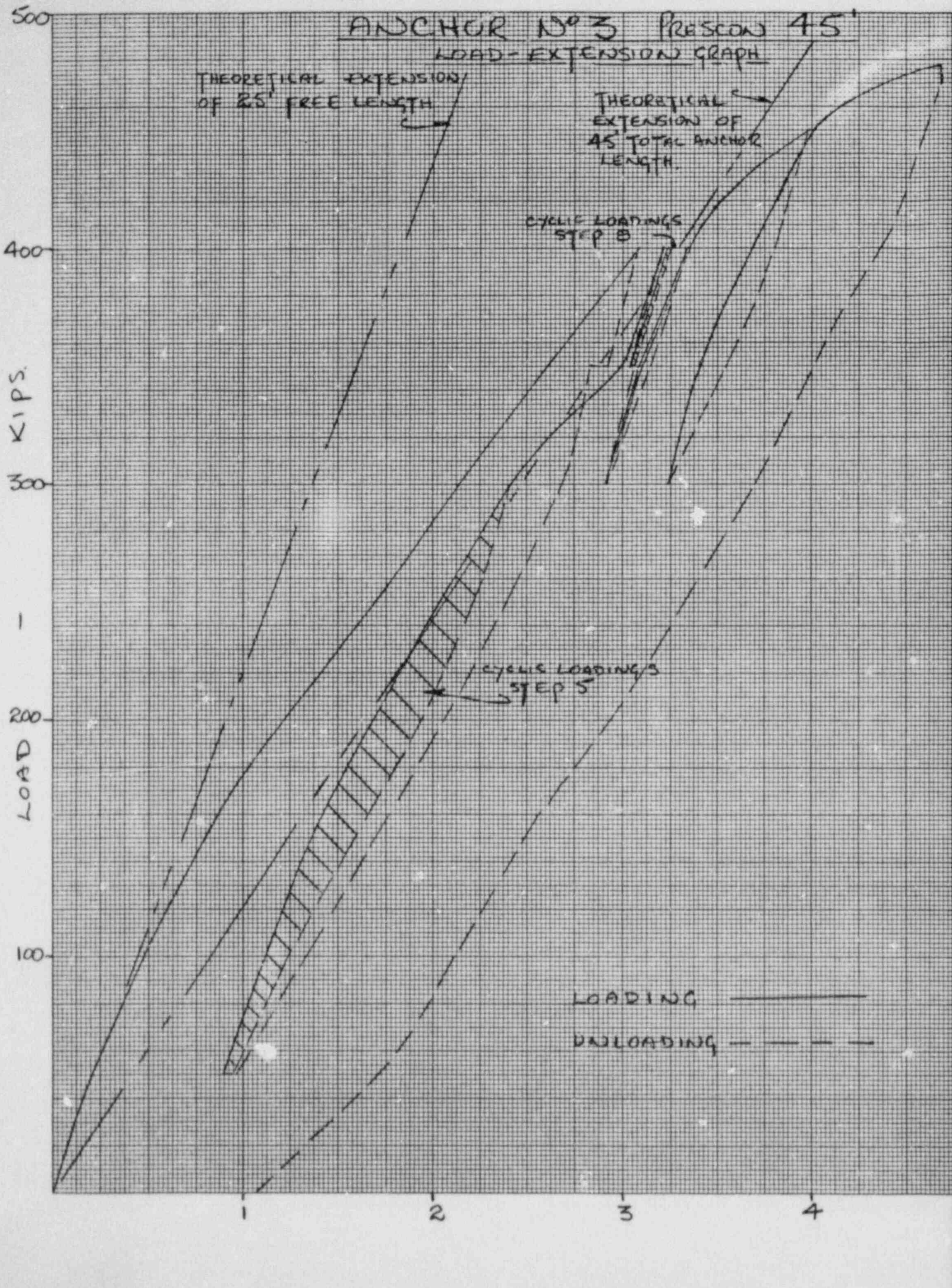

500
400
300
200
100
LOAD - KIPS
·25
·5
·75
1·0
1·25
1·5
1·
MOVEMENT
CYCLIC LOADINGS
CYCLIC LOADIN



ANCHOR No. 2 V.S.L. 55'

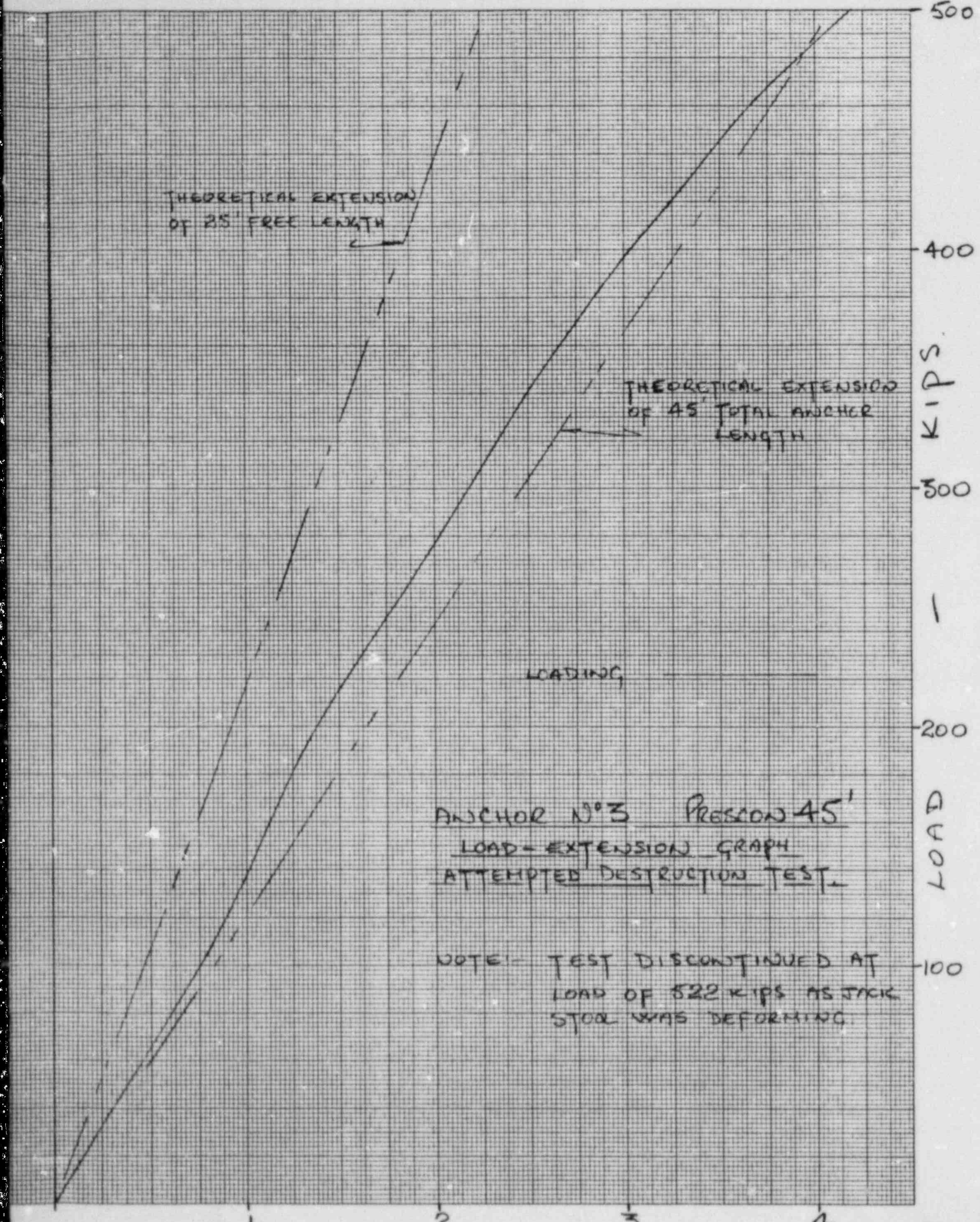
GROUT EXTENSOMETER @ -40'

FIG. 42



ANCHOR Nº 3 PRESCON 45'

LOAD-EXTENSION GRAPH



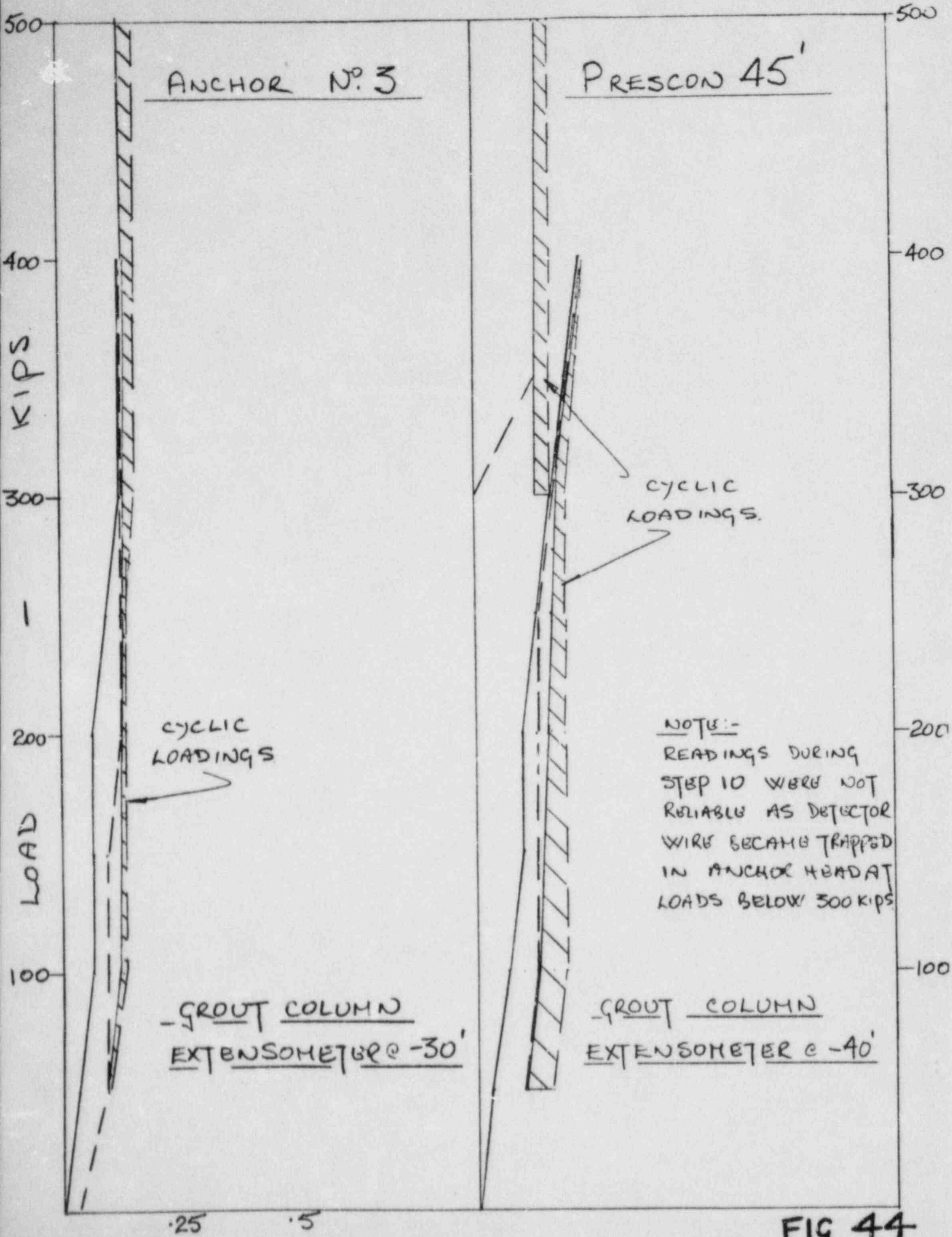
ANCHOR N°3 PRESCON 45'

LOAD – EXTENSION GRAPH

ATTEMPTED DESTRUCTION TEST.

NOTE:- TEST DISCONTINUED AT LOAD OF 522 KIPS AS JACK STOOL WAS DEFORMING.

FIG. 43.



ANCHOR No. 3

PRESCON 45'

CYCLIC LOADINGS.

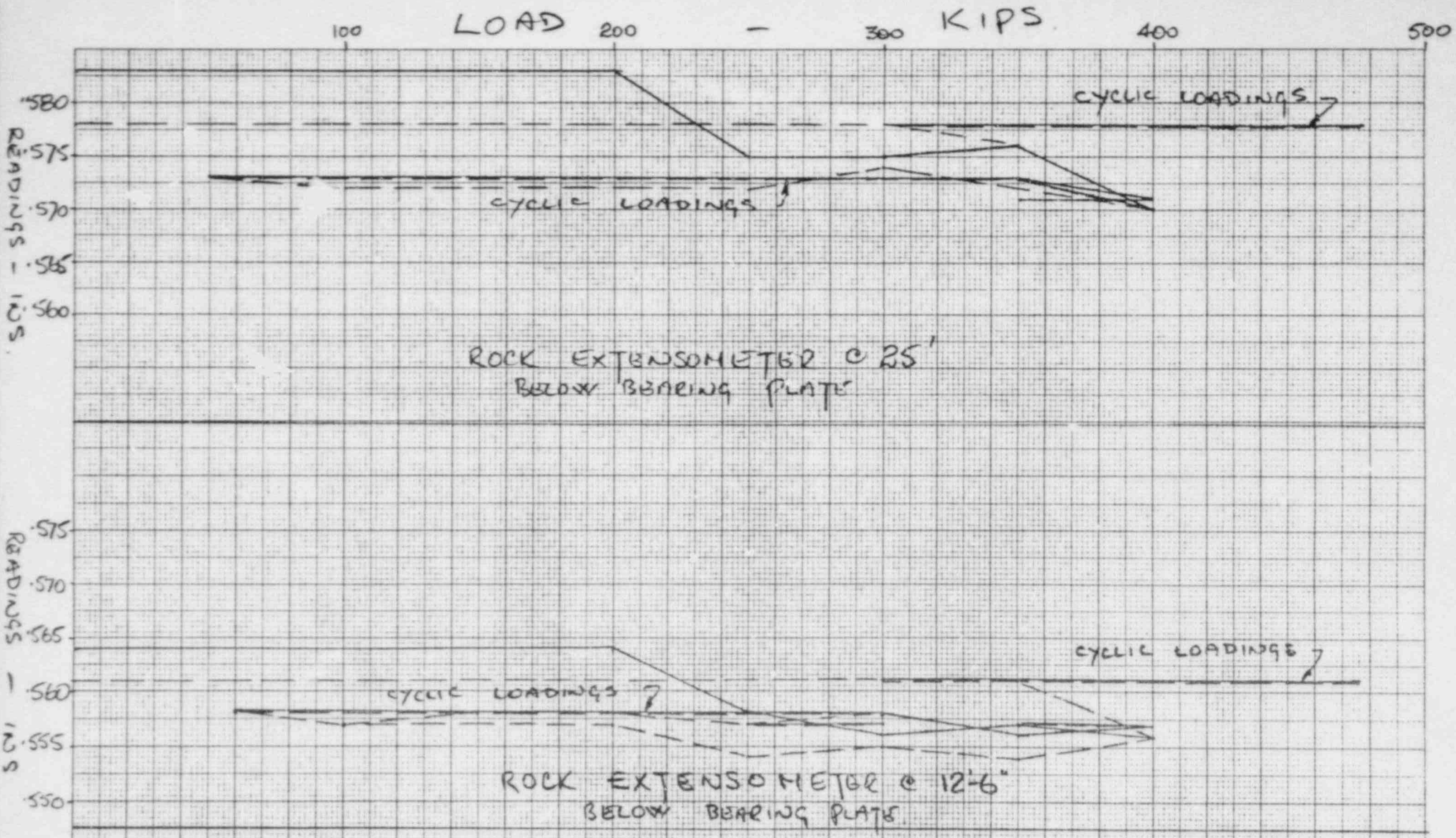
CYCLIC LOADINGS

NOTE:-
READINGS DURING STEP 10 WERE NOT RELIABLE AS DETECTOR WIRE BECAME TRAPPED IN ANCHOR HEAD AT LOADS BELOW 300 KIPS

-GROUT COLUMN EXTENSOMETER @ -30'

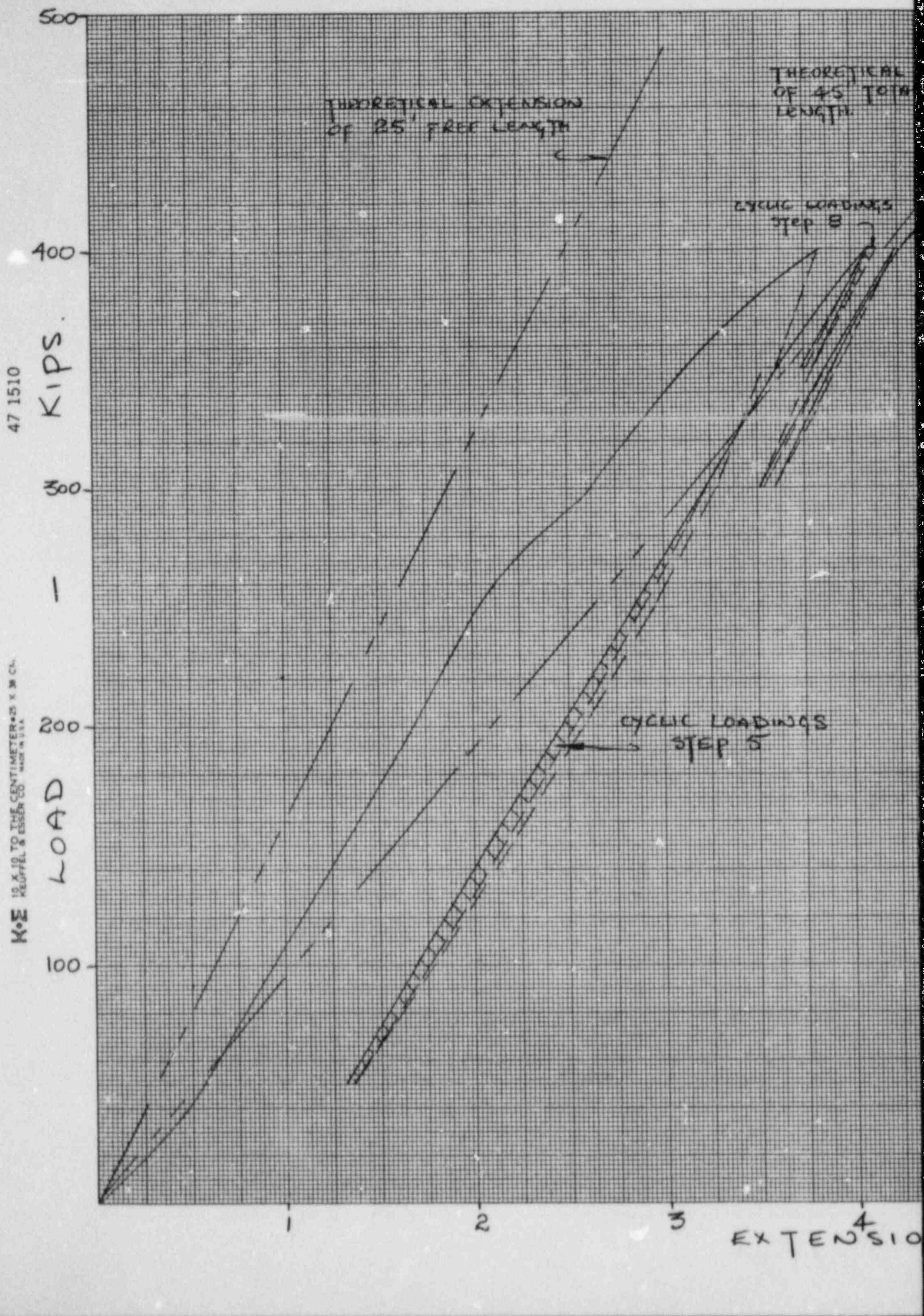
-GROUT COLUMN EXTENSOMETER @ -40'

FIG. 44.



READINGS OF ROCK EXTENSOMETER E-1
DURING TESTING OF ANCHOR N°3.

FIG. 4F

THEORETICAL EXTENSION OF 25' FREE LENGTH
THEORETICAL
OF 45' TOTA
LENGTH
CYCLIC LOADINGS STEP 8
CYCLIC LOADINGS STEP 5
LOAD - KIPS.
500
400
300
200
100
1
2
3
4
EXTENSIO
47 1510
K&E 10 X 10 TO THE CENTIMETER 18 X 25 CM.
KEUFFEL & ESSER CO. MADE IN U.S.A.

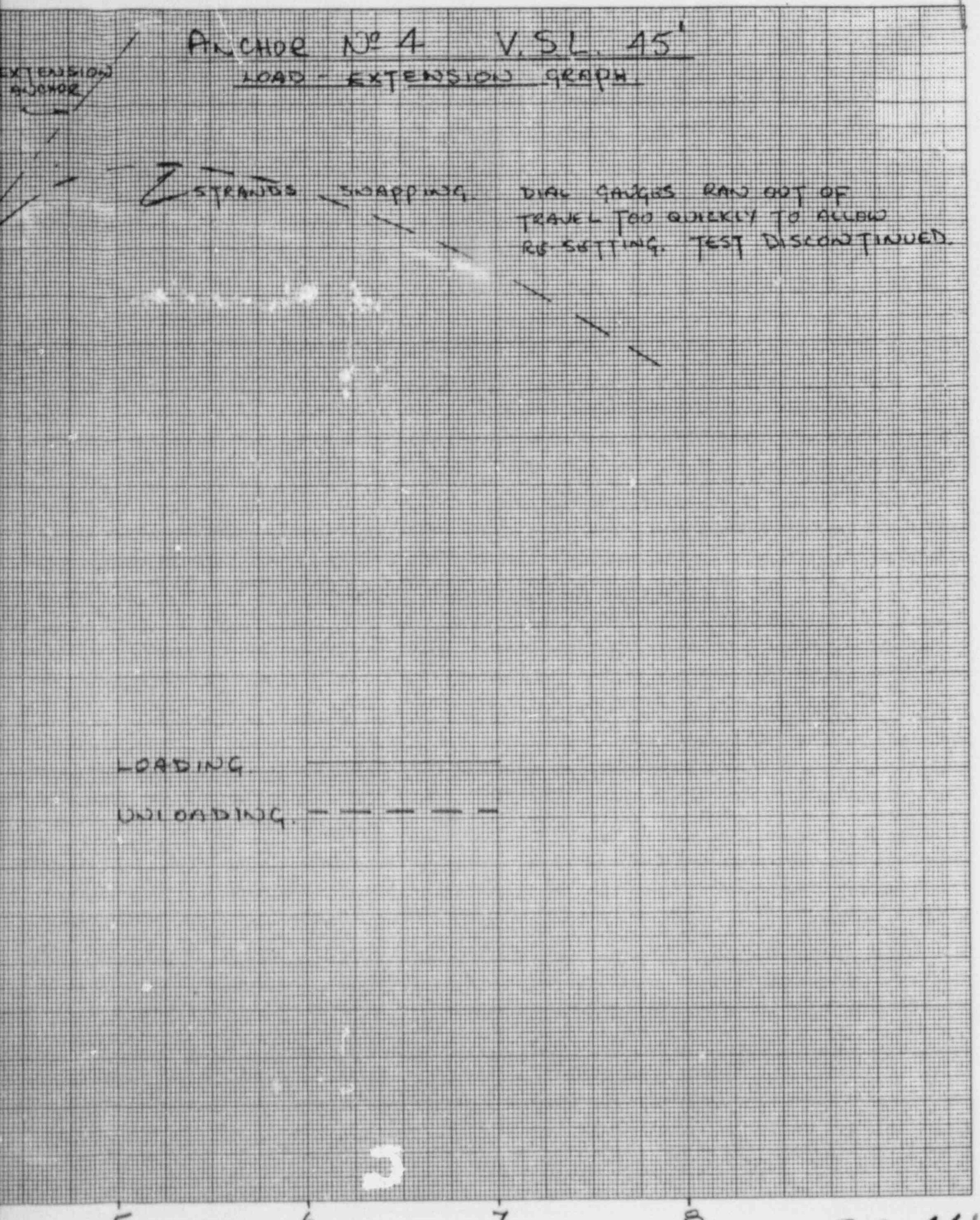
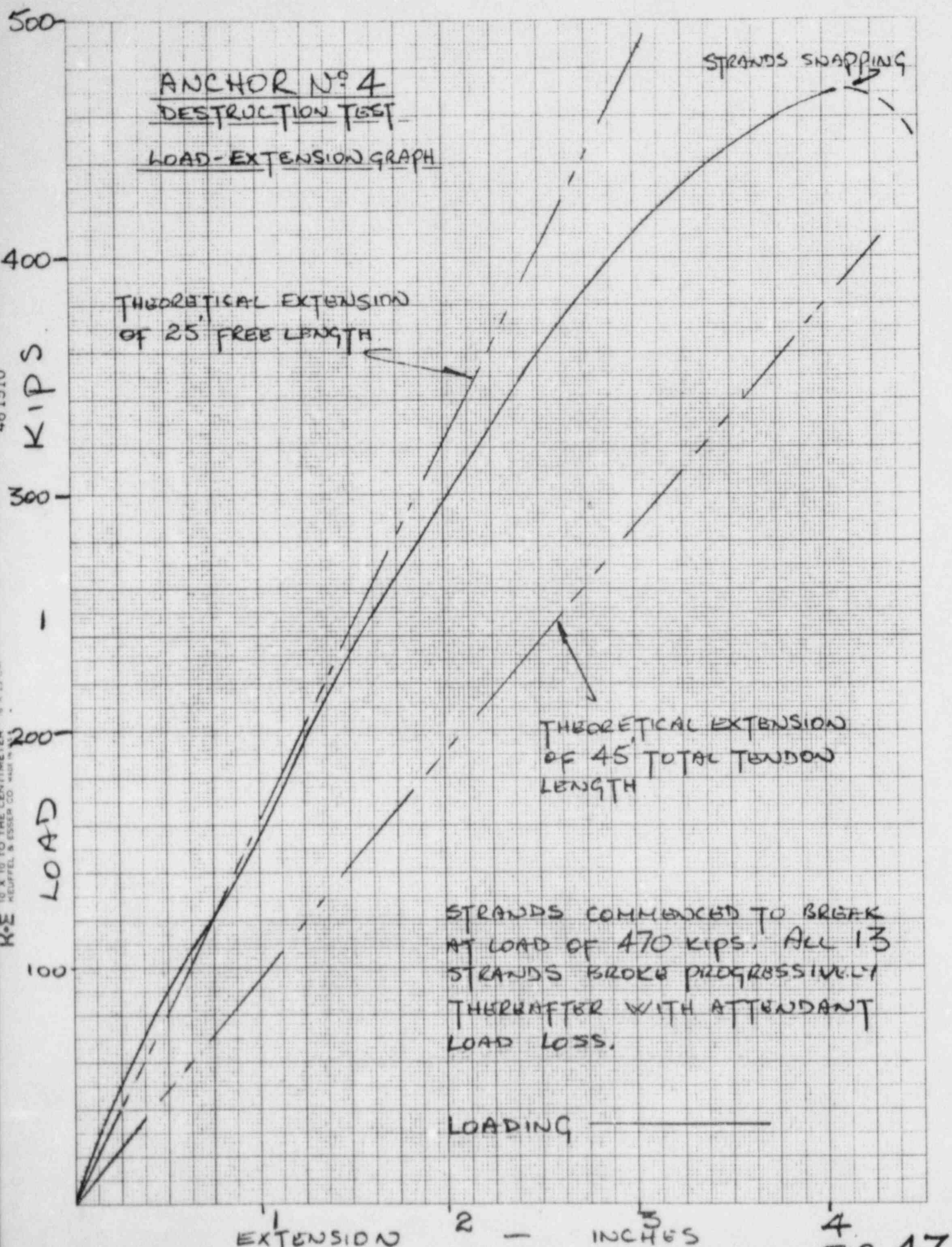

ANCHOR No 4 V.S.L. 45'
LOAD - EXTENSION GRAPH
EXTENSION ANCHOR
2 STRANDS SNAPPING.
DIAL GAUGES RAN OUT OF TRAVEL TOO QUICKLY TO ALLOW RE-SETTING. TEST DISCONTINUED.
LOADING.
UNLOADING.
5
6
7
8
INCHES.

FIG. 46



ANCHOR No 4
DESTRUCTION TEST
LOAD-EXTENSION GRAPH

STRANDS SNAPPING

THEORETICAL EXTENSION OF 25' FREE LENGTH

THEORETICAL EXTENSION OF 45' TOTAL TENDON LENGTH

STRANDS COMMENCED TO BREAK AT LOAD OF 470 KIPS. ALL 13 STRANDS BROKE PROGRESSIVELY THEREAFTER WITH ATTENDANT LOAD LOSS.

FIG. 47

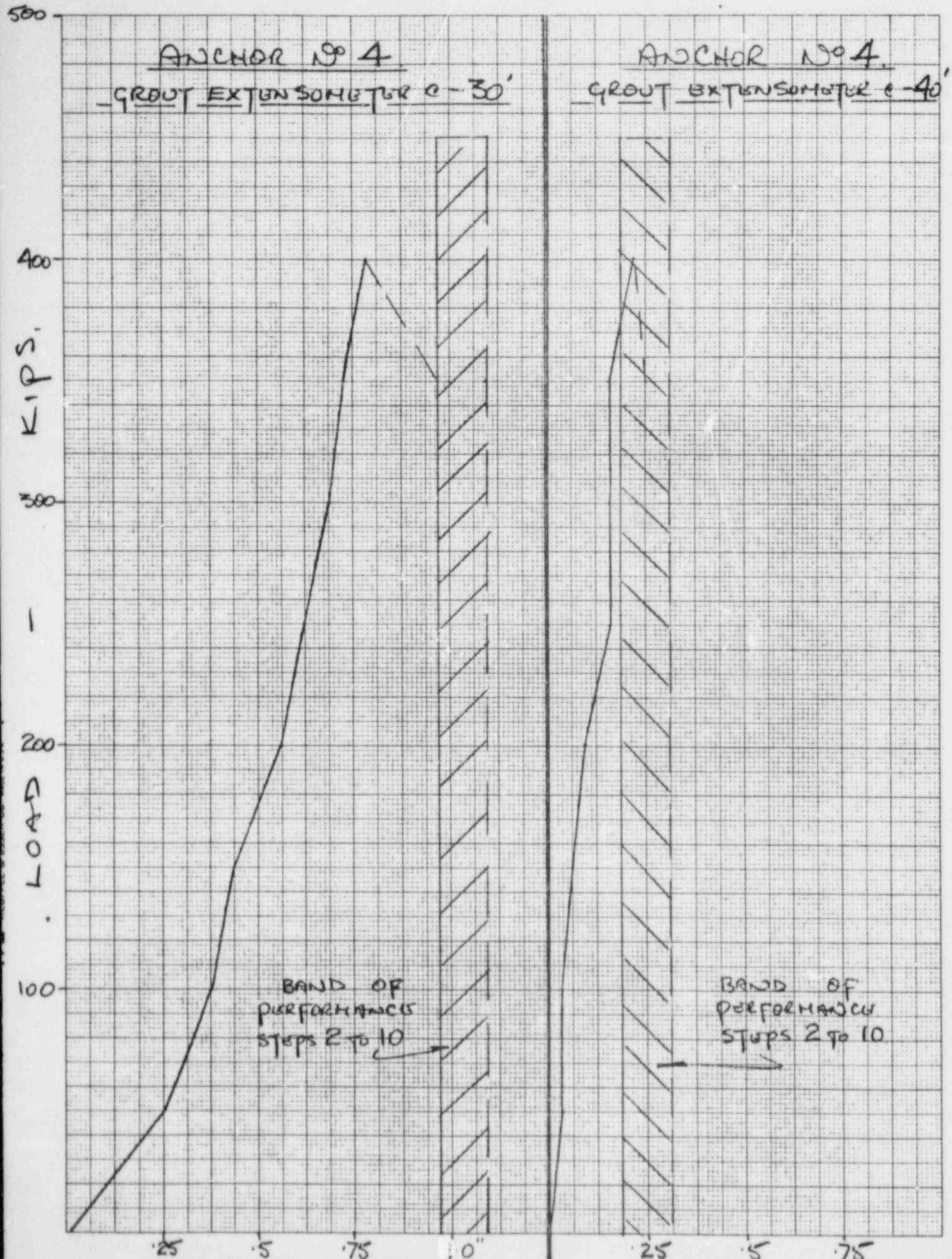
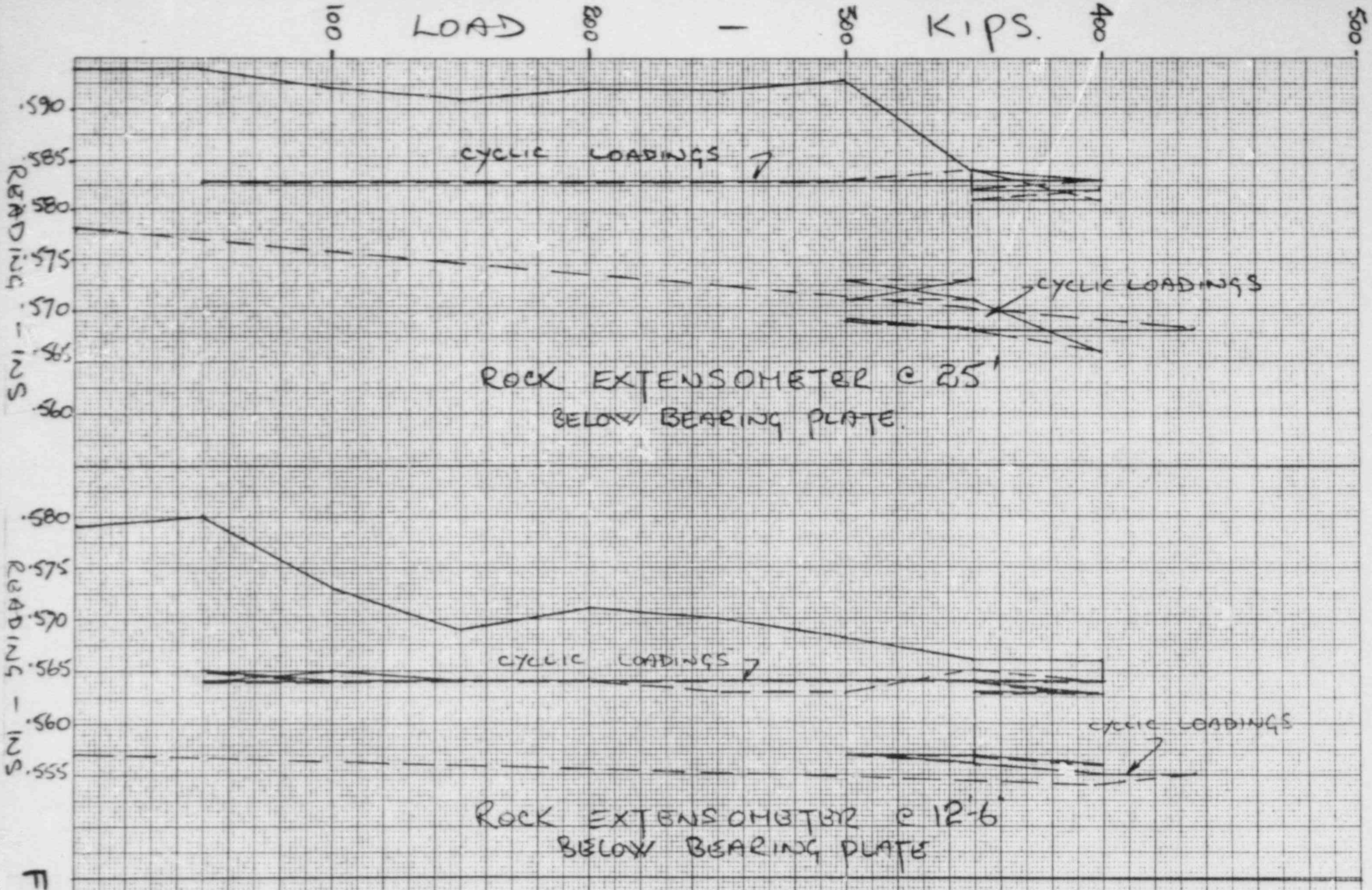

ANCHOR No 4
GROUT EXTENSOMETER @ -30'
ANCHOR No 4.
GROUT EXTENSOMETER @ -40'
500
400
300
200
100
LOAD - KIPS.
BAND OF PERFORMANCE STEPS 2 TO 10
BAND OF PERFORMANCE STEPS 2 TO 10
·25
·5
·75
1·0"
·25
·5
·75
MOVEMENT — INCHES.

FIG. 48



READINGS OF ROCK EXTENSOMETER E-1 DURING TESTING OF ANCHOR N°4

FIG. 49

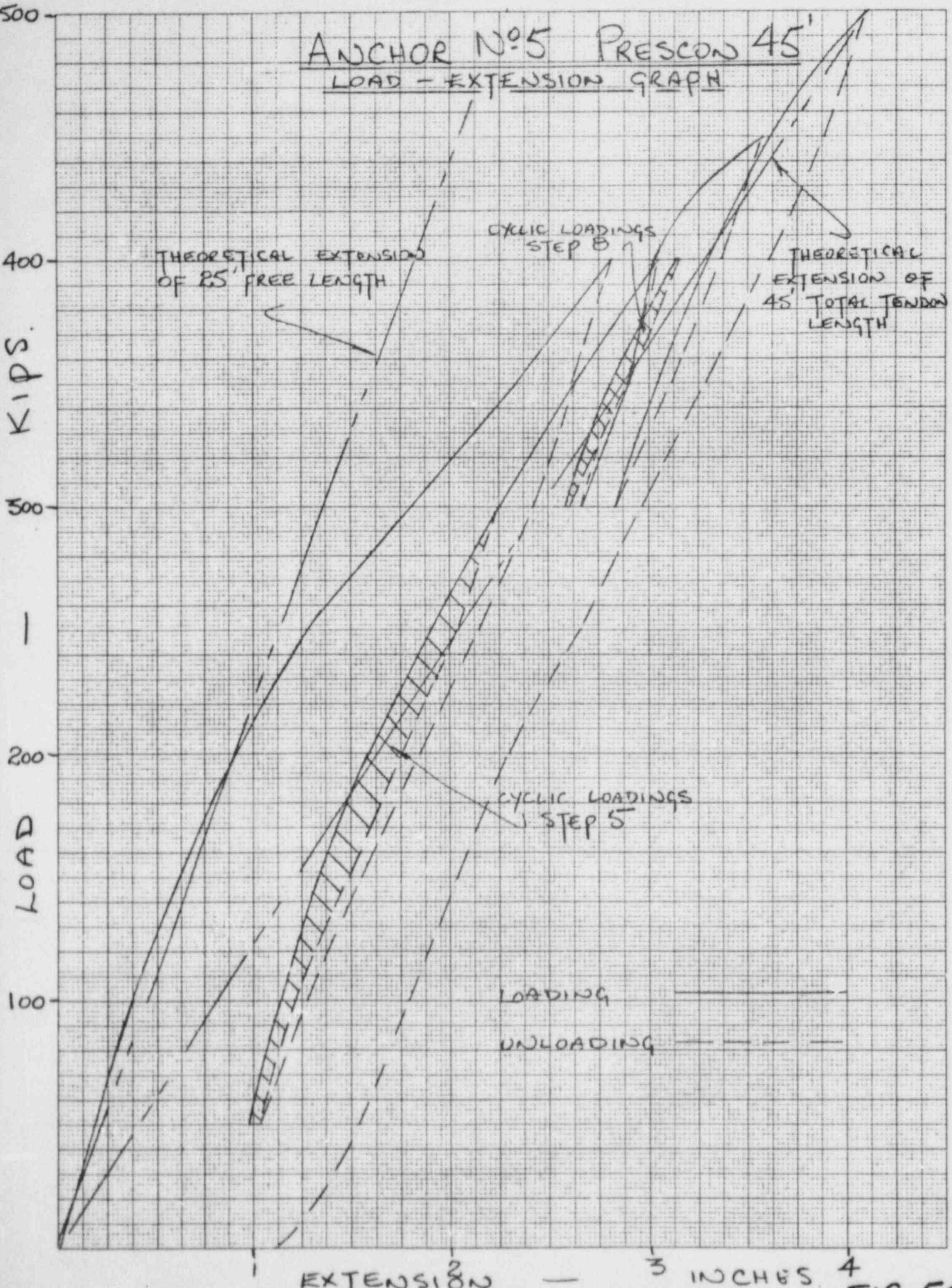
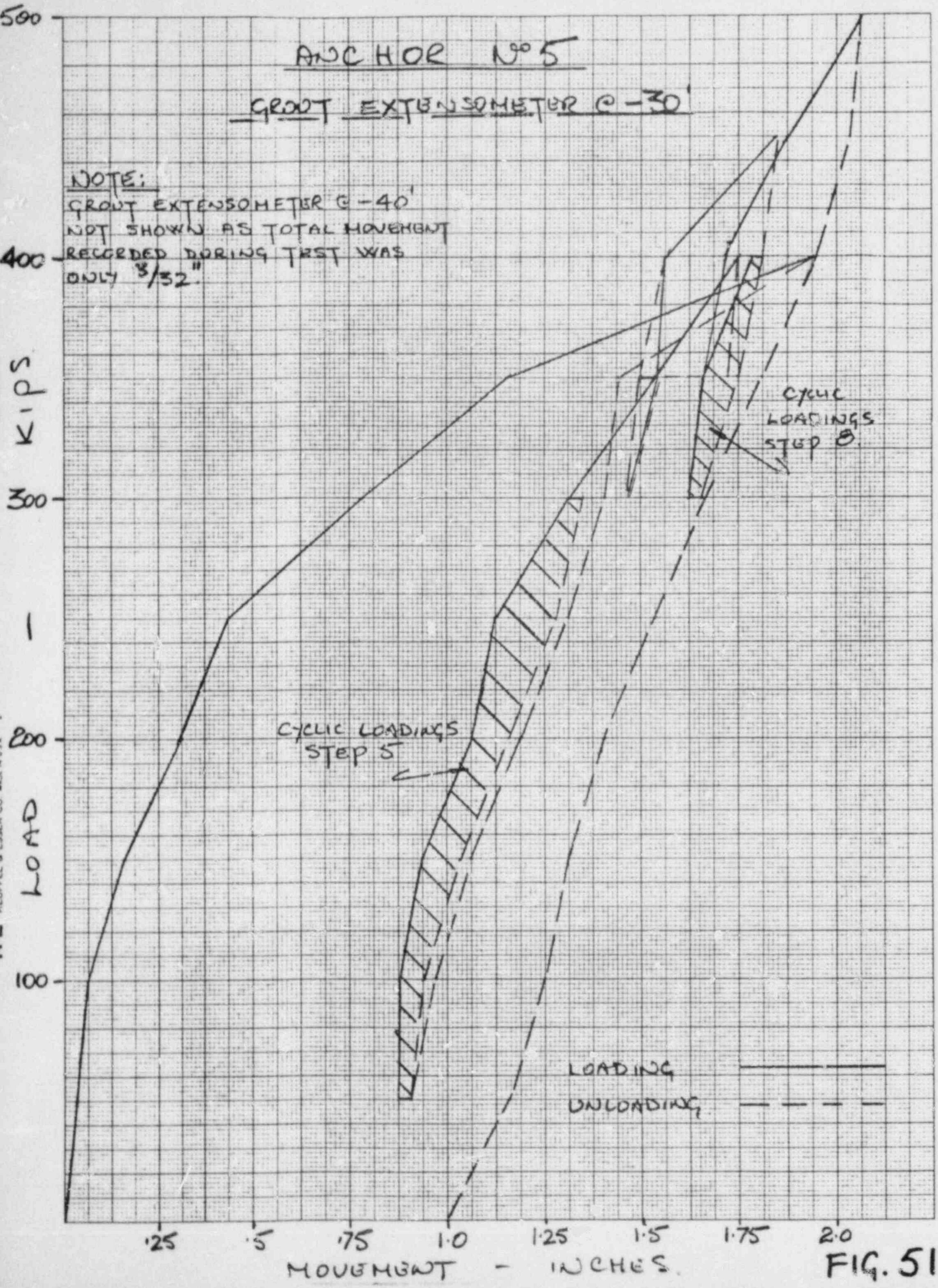

ANCHOR No.5 PRESCON 45'
LOAD - EXTENSION GRAPH
THEORETICAL EXTENSION OF 25' FREE LENGTH
CYCLIC LOADINGS STEP 8
THEORETICAL EXTENSION OF 45' TOTAL TENDON LENGTH
CYCLIC LOADINGS STEP 5
LOADING
UNLOADING
LOAD — KIPS.
500
400
500
200
100
1
2
3
4
EXTENSION — INCHES
K+E KEUFFEL & ESSER CO. MADE IN U.S.A.

FIG.50



ANCHOR N° 5

GROUT EXTENSOMETER @ -30'

NOTE:
GROUT EXTENSOMETER @ -40'
NOT SHOWN AS TOTAL MOVEMENT
RECORDED DURING TEST WAS
ONLY 3/32"

FIG. 51

ARMCO
Armco Steel Corporation
Kansas City, Mo. 64125

CERTIFICATION OF TEST
TUFWIRE

DATE February 23, 1976

CUSTOMER: The Presco Corporation
1338 North W. W. White Road
San Antonio, Texas 78205

NOMINAL WIRE DIA. .250"

MIN. SPECIFICATIONS

REQ'D. BREAKING STRENGTH 11,786 LBS. 240,000 P.S.I.

MINIMUM ELONGATION IN 10" 4.00 PERCENT

Customer Order No. 4619

COIL NO.	DIAMETER		BREAKING STRENGTH LBS.		ULT. TENSILE STR. PSI		% ELONG 10"	BENDS	STRAIGHT-NESS	BUTTON HEAD	YIELD STRENGTH
	FRONT	BACK	FRONT	BACK	FRONT	BACK					
	.251		12,200		246,500						
	.251		12,180		246,100						
1	.251		12,380		250,100		4.70				10,860#
5	.251		12,040		243,300	←	5.0 PKB-6-03				
8	.251		12,440		251,400						
7	.251		12,180		246,100						
1	.251		12,240		247,300						
3	.251		12,320		248,900						
7	.251		12,120		244,900						
8	.251		12,360		249,700						
0	.251		12,540		253,400						
7	.251		12,200		246,500						
9	.251		12,360		249,700						
55	.251		12,040		243,300		5.20				10,640
56	.251		12,540		253,400		50. 01-04-39				
57	.251		12,220		246,900						
72	.251		12,260		247,700						
73	.251		12,260		247,700		5.0.				
78	.251		12,080		244,100	←	PKB-6-03				

35 Coils, 30,530 lbs

A S T M Specification A 421-74

QC Received Inspection
Date 2-26-76

PCW 280

HEAT NO. 26472

ANALYSIS:				
.82	.86	.010	.026	.24
C	MN	P	S	SI

FIG. 52A.

THE PHYSICAL OR MECHANICAL TEST REPORTED ABOVE ARE CORRECT AS CONTAINED IN THE RECORDS OF THE CORPORATION.

[illegible] and sworn to before me, A Notary Publ
in and for Jackson County, the State of Missouri.
this 23rd day of February 1976
Virgil G. [illegible]
2-26-76

BY J P [illegible]
Sup'v. Quality Control High Caron Wire & Rope

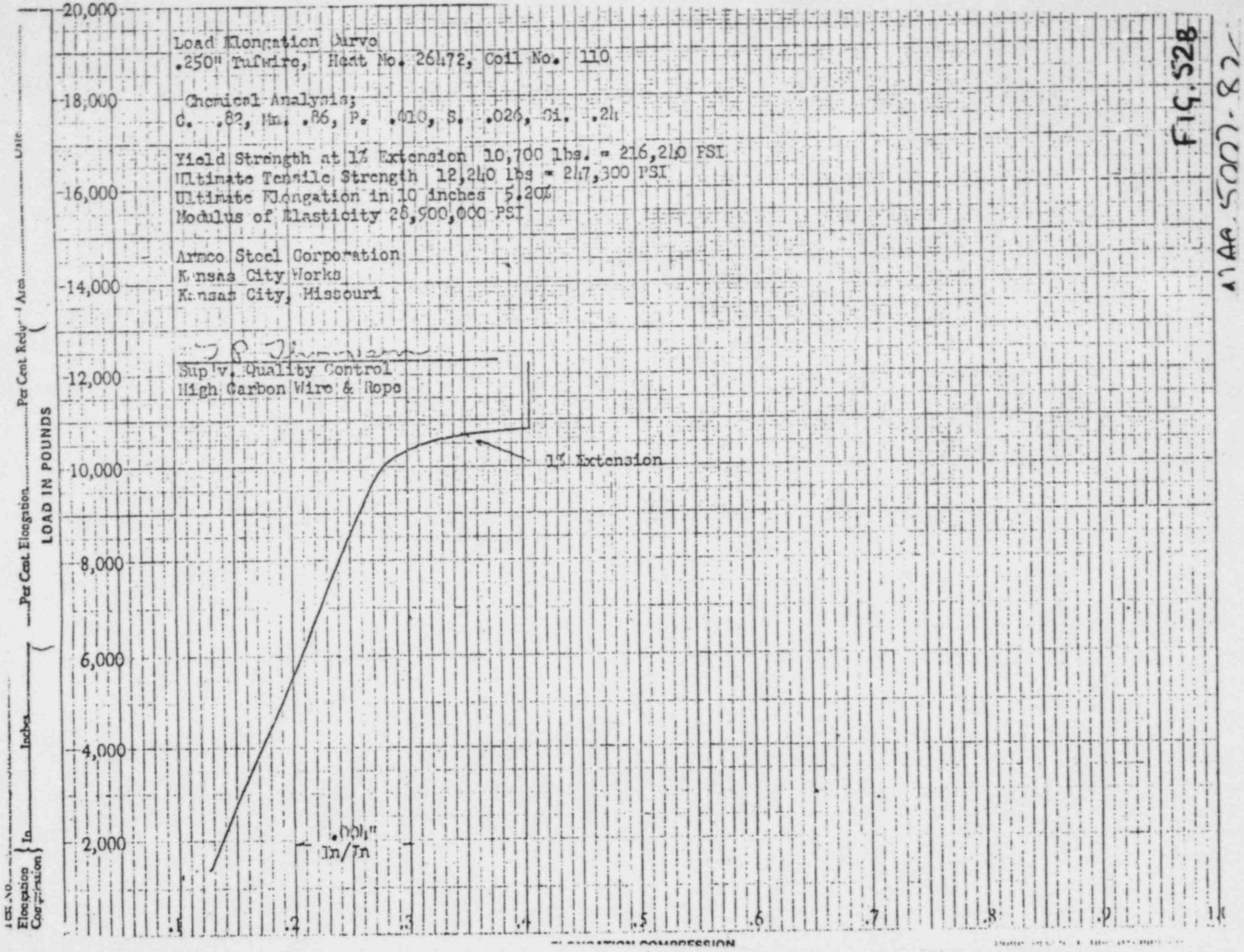
TITLE

NAC (00)-82

MU-2508 REV. 3/72

FIG. 528

11AA 5007-82



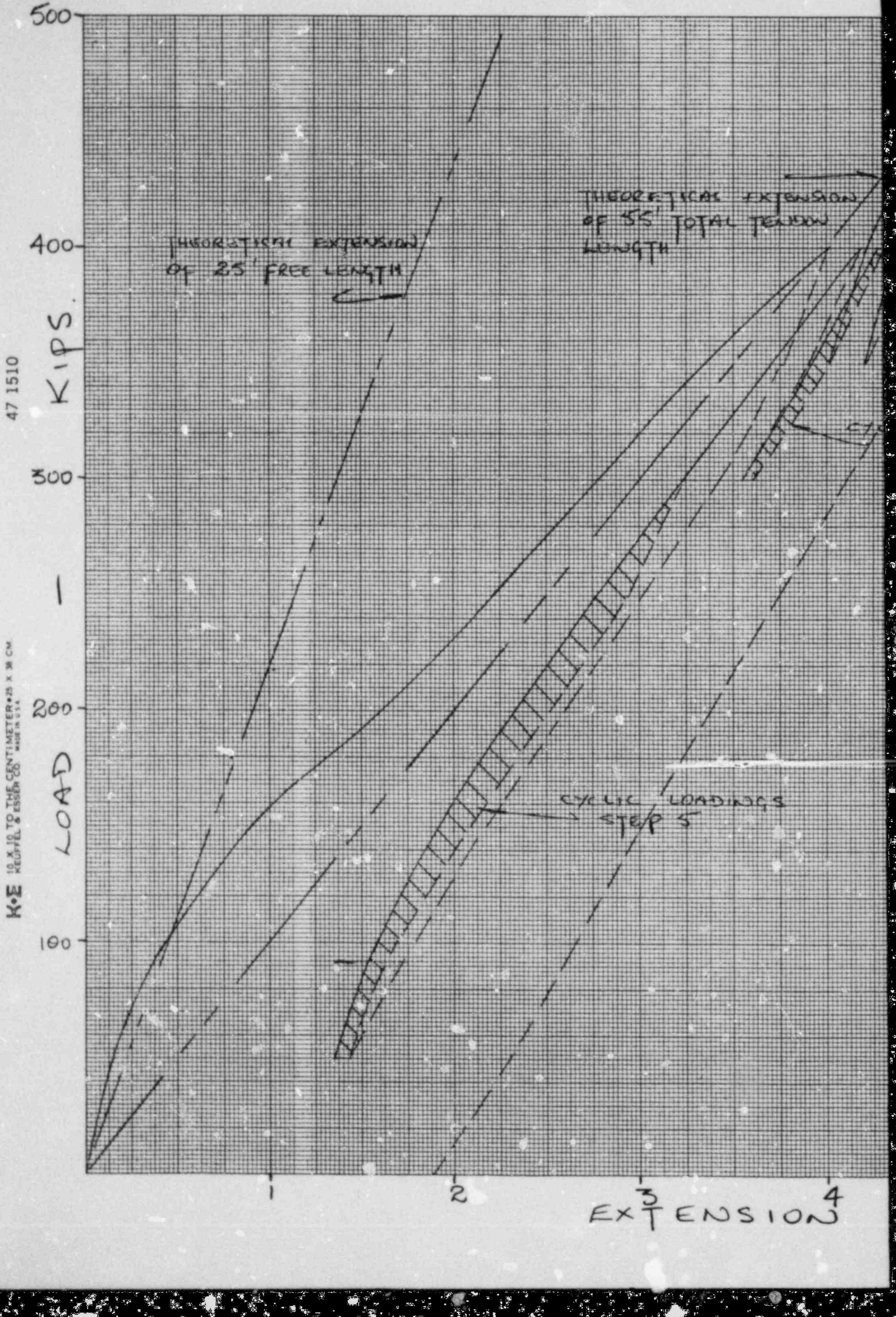
Load Elongation Curve
.250" Tufwire, Heat No. 26472, Coil No. 110

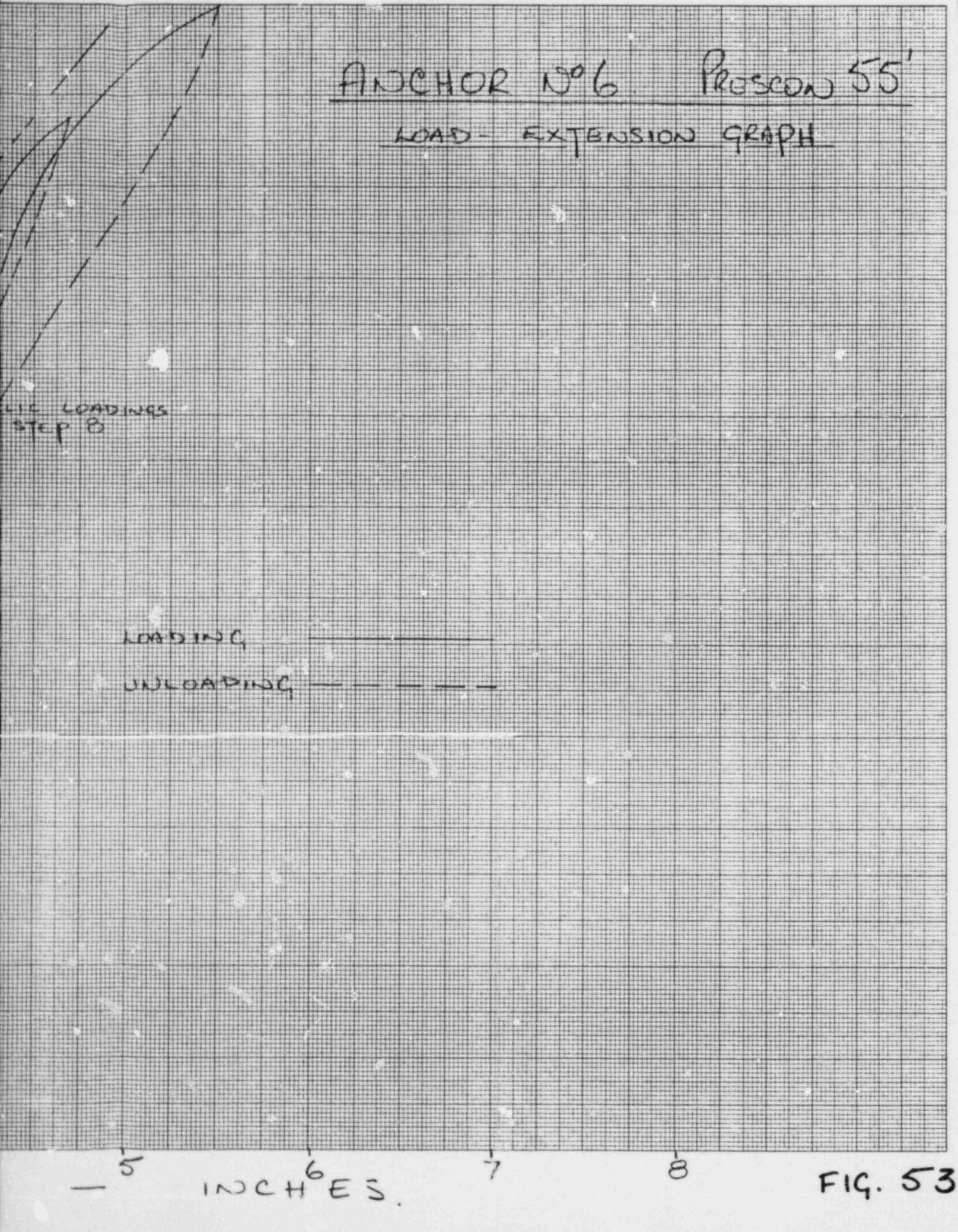
Chemical Analysis;
C. .82, Mn. .86, P. .010, S. .026, Si. .24

Yield Strength at 1% Extension 10,700 lbs. = 216,240 PSI
Ultimate Tensile Strength 12,240 lbs = 247,300 PSI
Ultimate Elongation in 10 inches 5.20%
Modulus of Elasticity 28,900,000 PSI

Armco Steel Corporation
Kansas City Works
Kansas City, Missouri

Sup'v. Quality Control
High Carbon Wire & Rope


500
400
300
200
100
LOAD — KIPS.
1
2
3
4
EXTENSION
THEORETICAL EXTENSION OF 25' FREE LENGTH
THEORETICAL EXTENSION OF 55' TOTAL TENDON LENGTH
CYCLIC LOADINGS STEP 5
CY
K•E 10 X 10 TO THE CENTIMETER • 25 X 38 CM KEUFFEL & ESSER CO. MADE IN U.S.A.
47 1510



ANCHOR N°6 PROSCON 55'

LOAD- EXTENSION GRAPH

FIG. 53

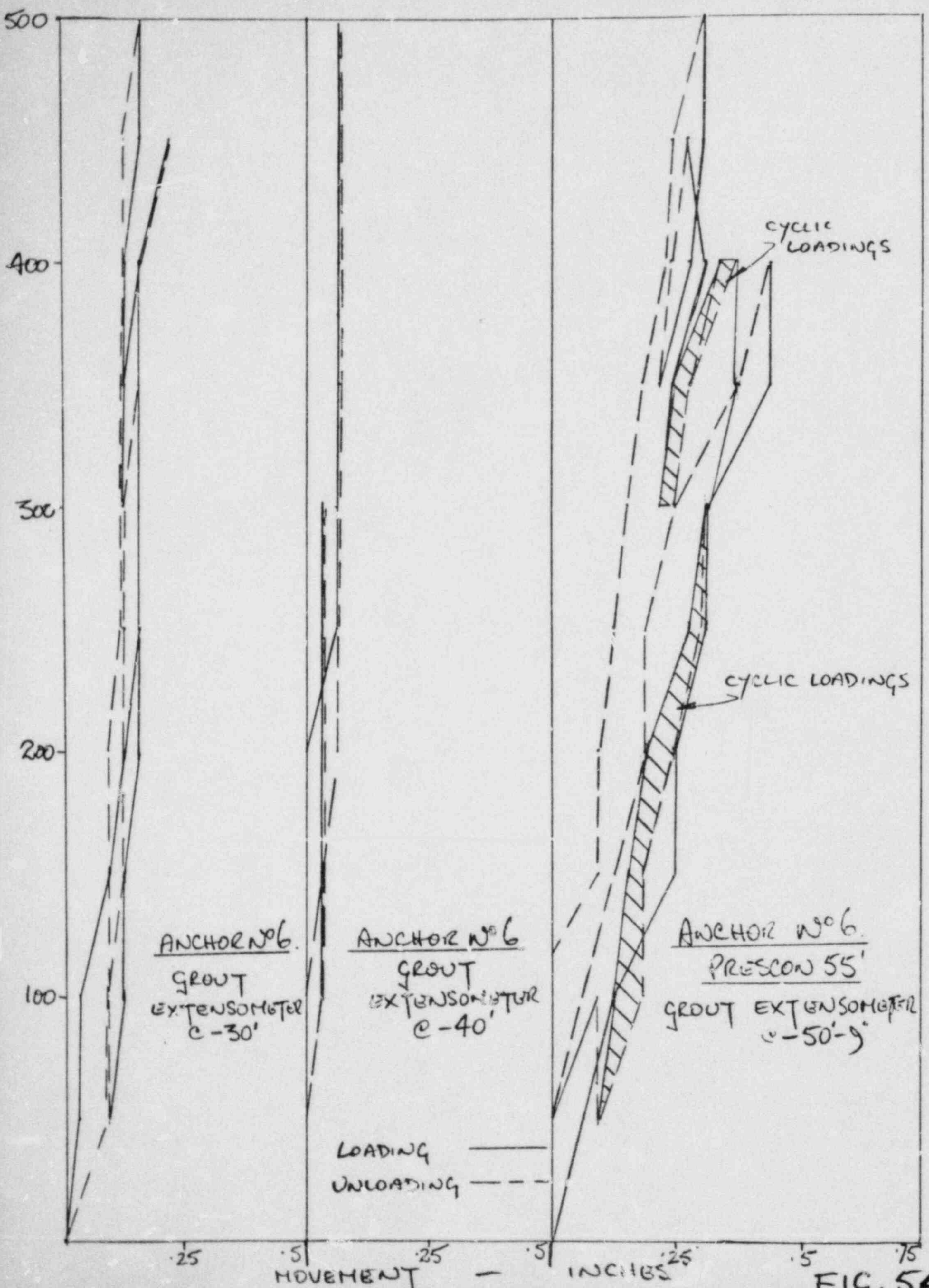
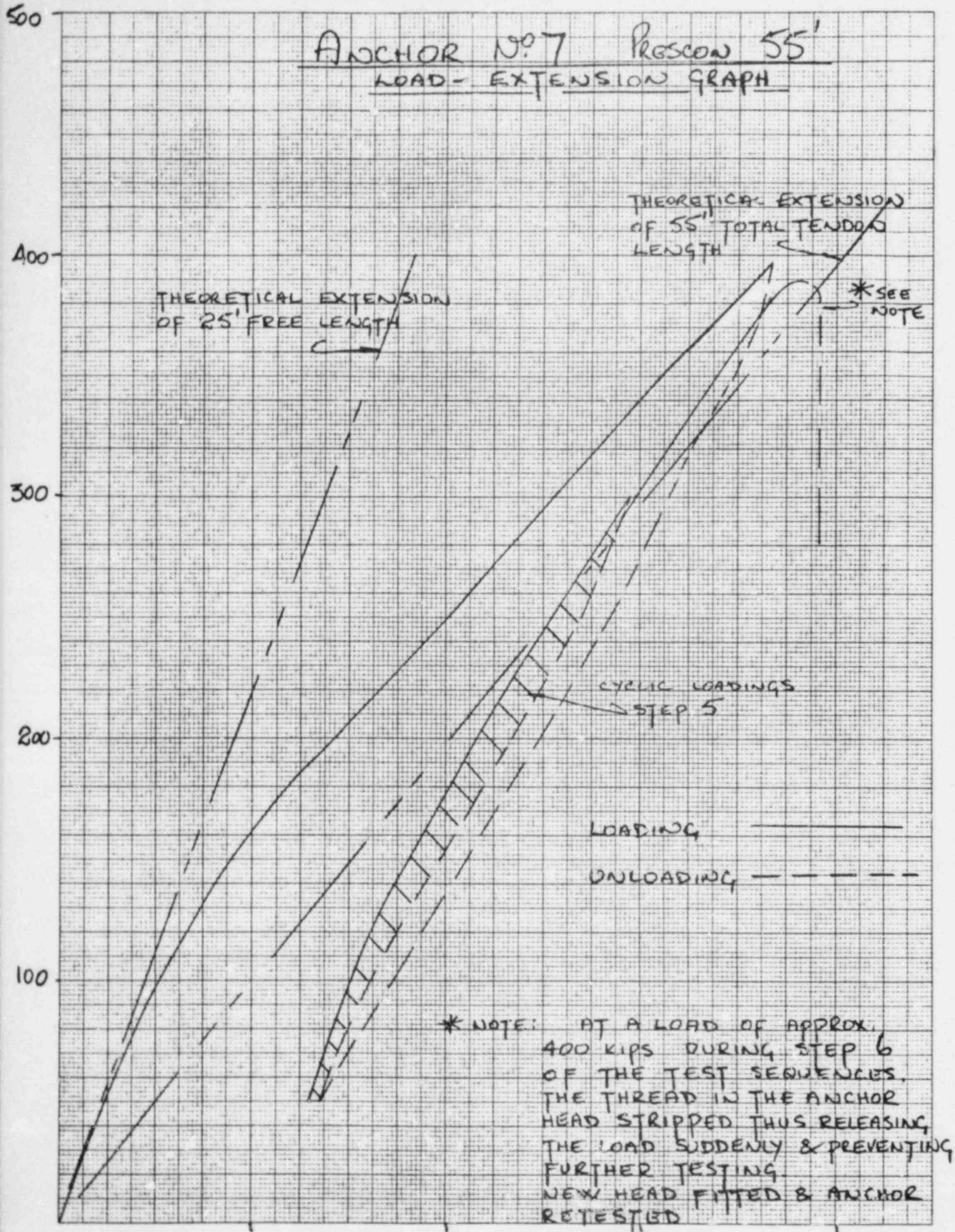
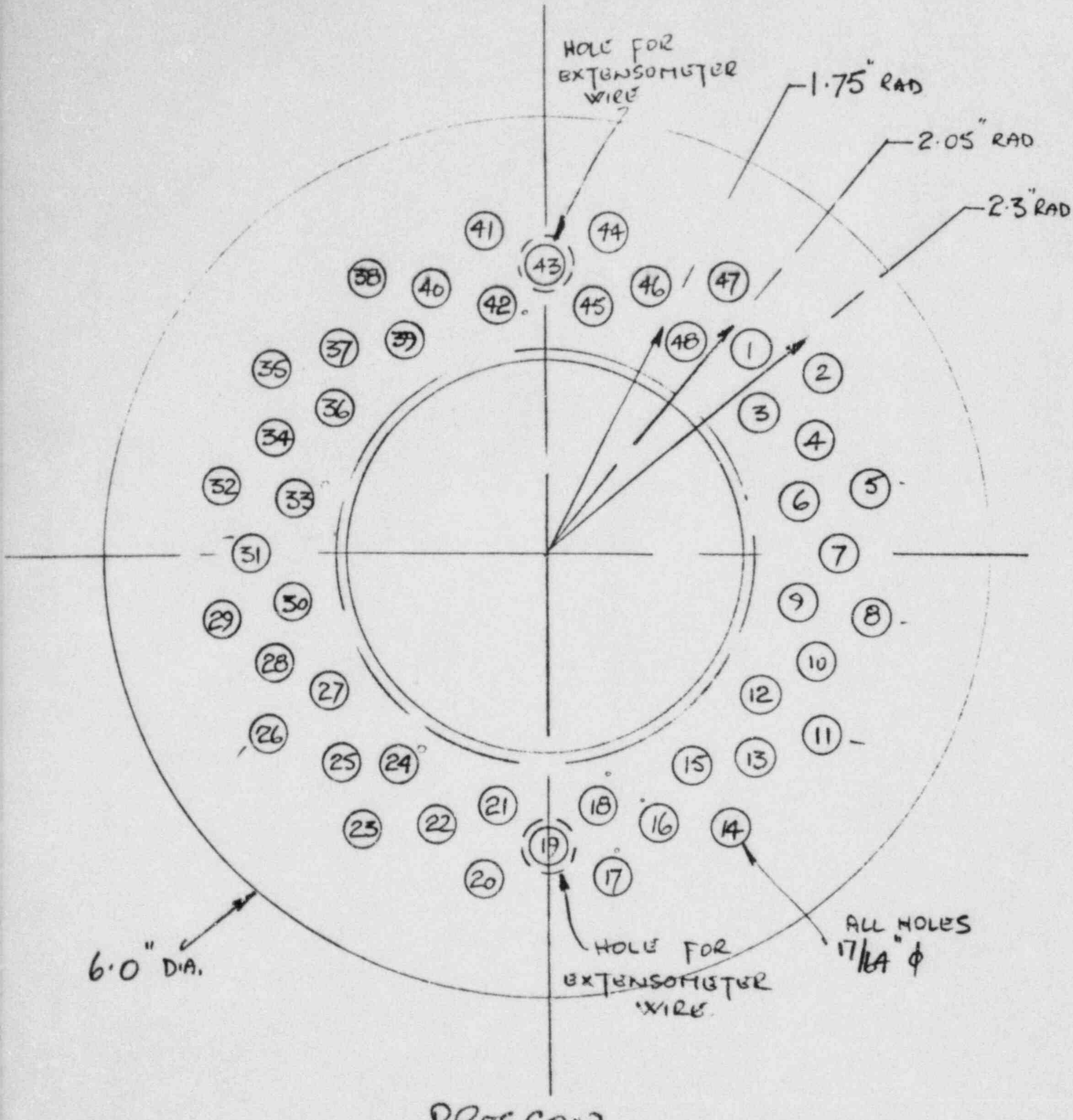

500
400
300
200
100
·25
·5
·25
·5
·25
·5
·75
MOVEMENT — INCHES
ANCHOR Nº6.
GROUT
EXTENSOMETER
@ -30'
ANCHOR Nº6
GROUT
EXTENSOMETER
@ -40'
ANCHOR Nº6.
PRESCON 55'
GROUT EXTENSOMETER
@ -50'-9"
CYCLIC LOADINGS
CYCLIC LOADINGS
LOADING
UNLOADING

FIG. 54



* NOTE: AT A LOAD OF APPROX. 400 KIPS DURING STEP 6 OF THE TEST SEQUENCES, THE THREAD IN THE ANCHOR HEAD STRIPPED THUS RELEASING THE LOAD SUDDENLY & PREVENTING FURTHER TESTING. NEW HEAD FITTED & ANCHOR RETESTED

FIG. 55



PRESCON

STRESS HEAD.

MATERIAL: C-1045

BROKEN WIRES, 5, 8, 11, 26, 31
BENT WIRES. 24, 33, 48.

FIG. 56

ANCHOR No 7 PRESCON 55'

LOAD - EXTENSION GRAPHS

FOR REPRESENTATIVE INDIVIDUAL
TENDON WIRES TESTED PRIOR
TO FITTING NEW ANCHOR HEAD.

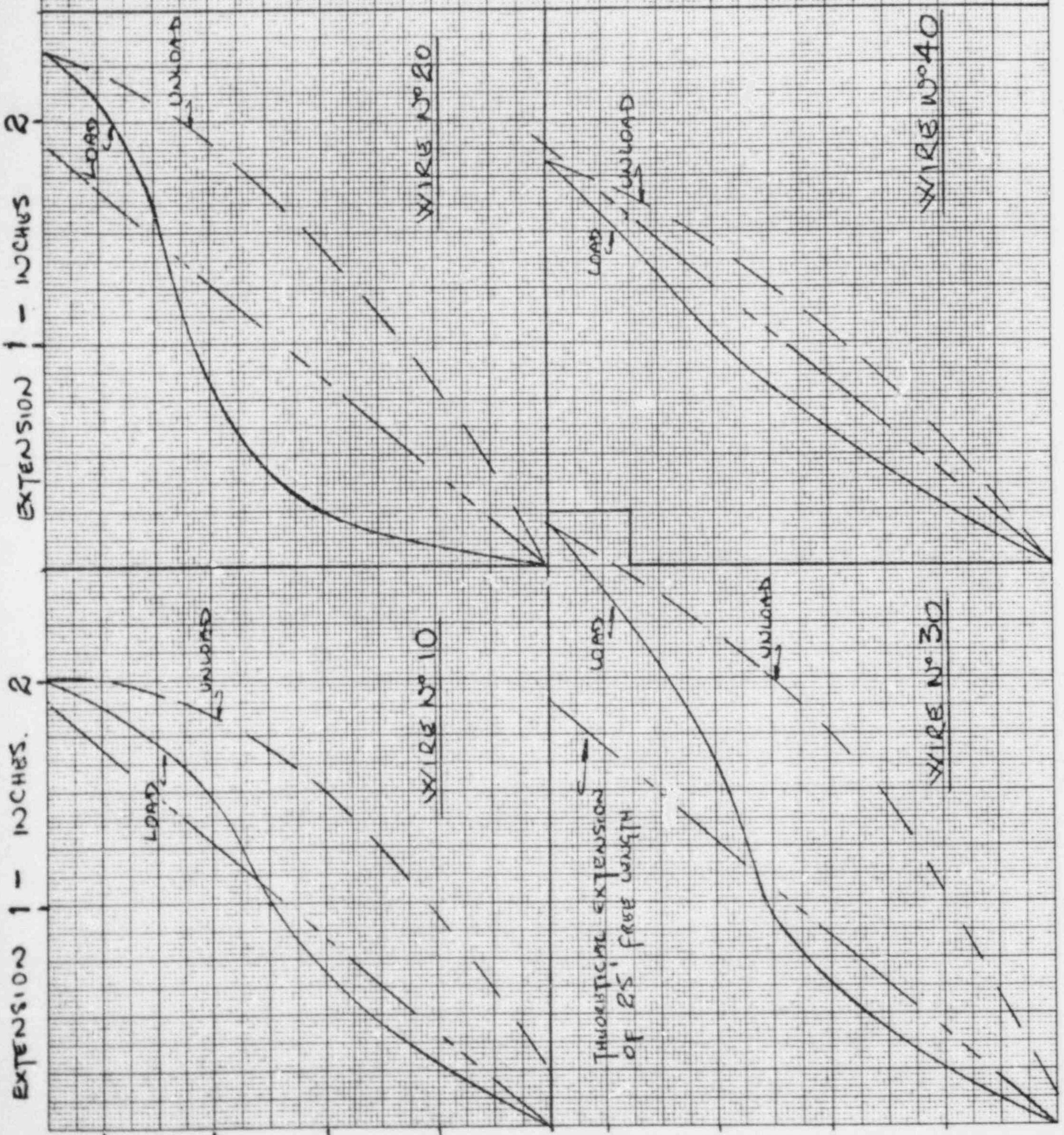
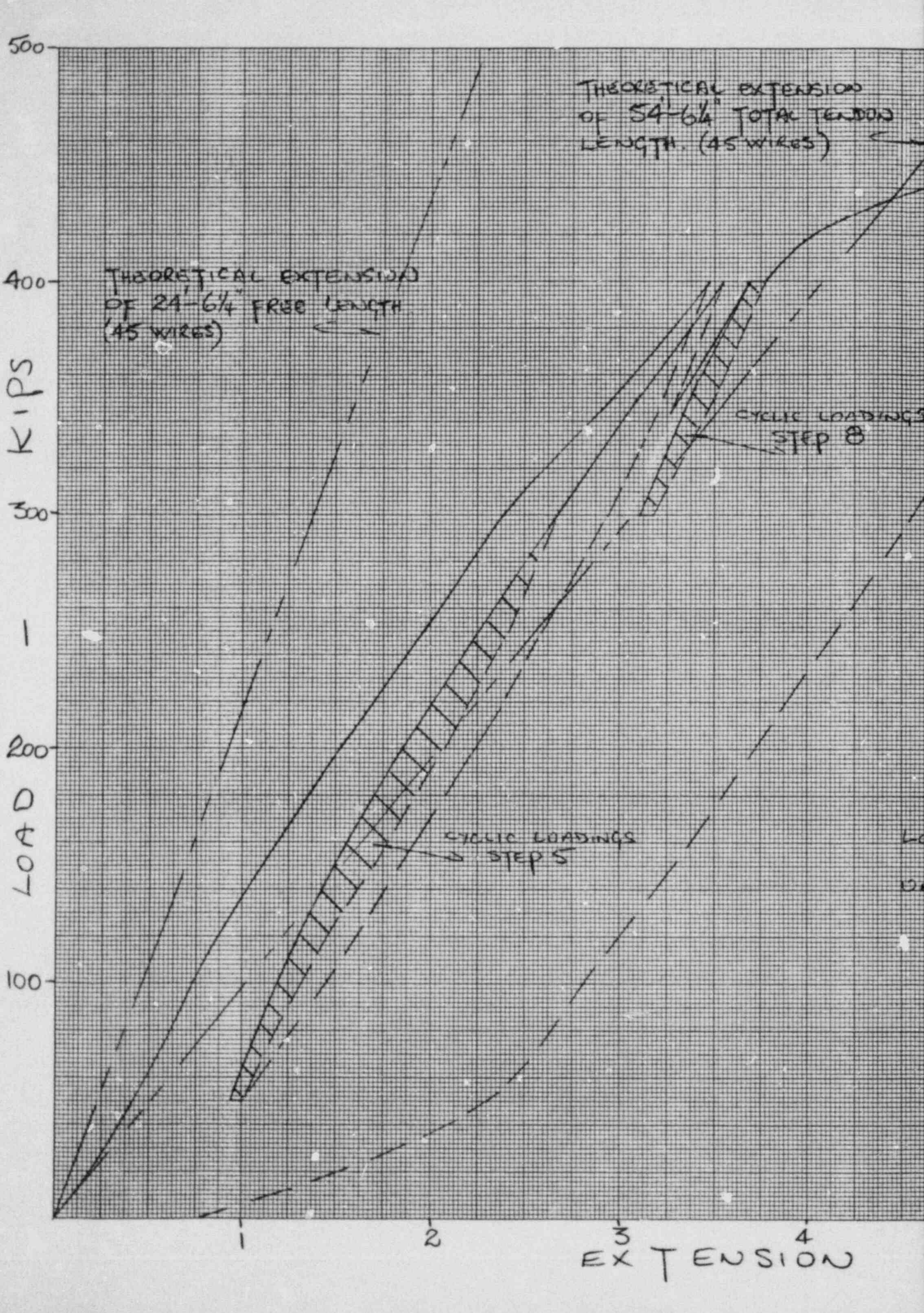
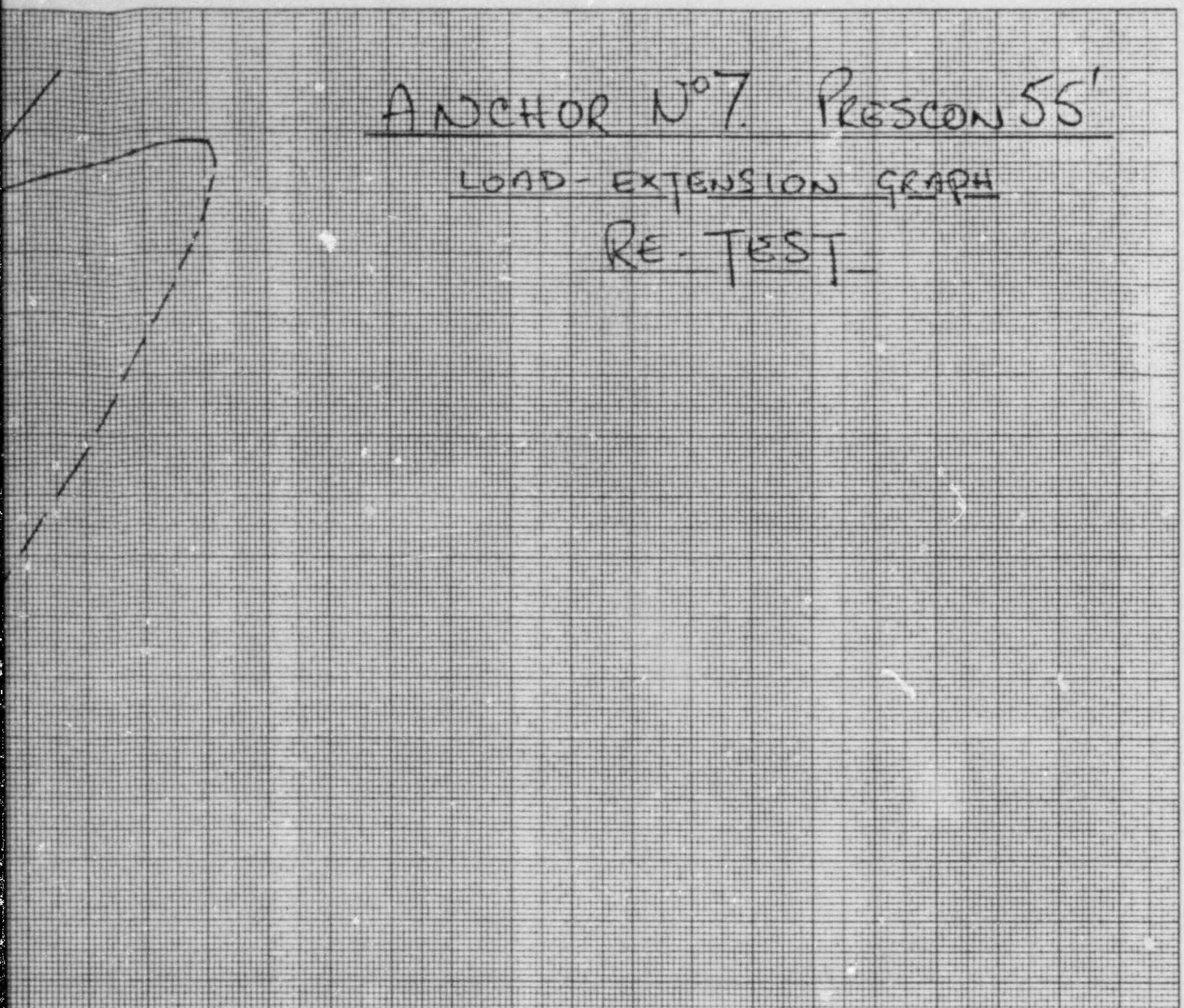


FIG. 57


500
400
300
200
100
LOAD - KIPS
1
2
3
4
EXTENSION
THEORETICAL EXTENSION OF 54'-6¼" TOTAL TENDON LENGTH. (45 WIRES)
THEORETICAL EXTENSION OF 24'-6¼" FREE LENGTH (45 WIRES)
CYCLIC LOADINGS STEP 8
CYCLIC LOADINGS STEP 5
LO
UA



ANCHOR N°7. PRESCON 55'

LOAD-EXTENSION GRAPH

RE-TEST

FIG. 58

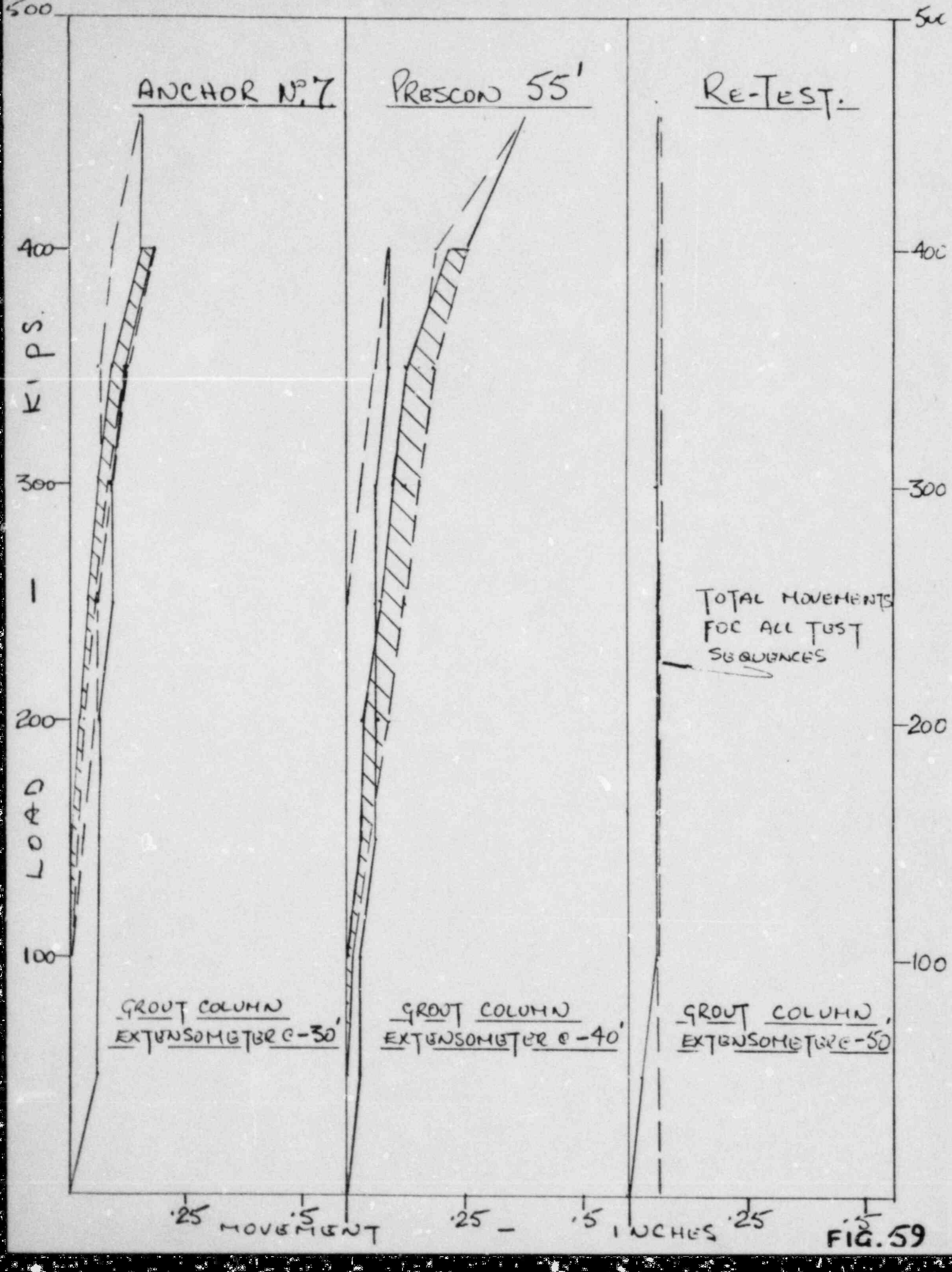

500
400
300
200
100
LOAD — KIPS.
5…
400
300
200
100
ANCHOR No. 7
PRESCON 55'
RE-TEST.
TOTAL MOVEMENTS FOR ALL TEST SEQUENCES
GROUT COLUMN EXTENSOMETER @ -30'
GROUT COLUMN EXTENSOMETER @ -40'
GROUT COLUMN, EXTENSOMETER @ -50
·25
·5
·25
·5
·25
·5
MOVEMENT — INCHES

FIG. 59

NAC
GROUND ANCHORS

NICHOLSON ANCHORAGE COMPANY

P. O. BOX 308 BRIDGEVILLE, PENNSYLVANIA 15017
412/221-4500

For North Anna Power Station
Masterflow 814 Grout
2.4 gals/55# bag.

Record of Hole No. Grout Test Hole #2

Location

Surface Elevation

Date Started 2/18/76

Driller Rick Triplett

Date Completed 2/18/76

STRATA	FEET	INCHES	TOTAL
Concrete (Roller Bit)	4	4	4' 4"
Concrete	3	6	7'10"
Grout/Rock Interface Bonded		11	8' 9"
" " " Unbonded		2	8'11"
" " " Bonded	4	2	13' 1"
" " " Unbonded	1	1	14' 2"
" " " Bonded		6	14' 8"
" " " Unbonded	7	4	22' 0"

FIG. 60

A Duplicate of this Record may be obtained by writing to the Above Address

NAC
GROUND ANCHORS
NICHOLSON ANCHORAGE COMPANY
P. O. BOX 308 BRIDGEVILLE, PENNSYLVANIA 15017
412/221-4500

For North Anna Power Station

Record of Hole No. Grout Test Hole #3

Portland Cement Type II Grout
5 gals/bag. No additive.

Location

Surface Elevation

Date Started 2/19/76

Driller Rick Triplett

Date Completed 2/19/76

STRATA	FEET	INCHES	TOTAL
Run I Concrete	1	0	1' 0"
Grout	13	0	14' 0"
Grout/Rock Interface	4	0	18' 0"
Bonded			
Grout		4	18' 4"
Run II Concrete	1	0	1' 0"
Grout	17	0	18' 0"
Run III Concrete	1	0	1' 0"
Grout	12	6	13' 6"
Run IV Concrete	1	0	5' 0"
Grey Granite	4	0	With Roller Bit
Grout/Rock Interface	4	6	9' 6"
Bonded			✓

Fig. 61

A Duplicate of this Record may be obtained by writing to the Above Address

NAC
GROUND ANCHORS

NICHOLSON ANCHORAGE COMPANY

P. O. BOX 308 BRIDGEVILLE, PENNSYLVANIA 15017
412/221-4500

For North Anna Power Station

Record of Hole No. Grout Test Hole #3

Portland Cement Type II Grout
5 gals/bag. No additive

Location

Surface Elevation

Date Started 2/20/76

Driller Rick Triplett

Date Completed 2/26/76

STRATA	FEET	INCHES	TOTAL
Run V Concrete	4	--	4' 0"
Grey Granite	8	3	12' 3"
Grout/Rock Interface	3	8	16' 0"
Bonded			
Grout	6		22' 0"
Grey Granite	3		25' 0"

FIG. 62

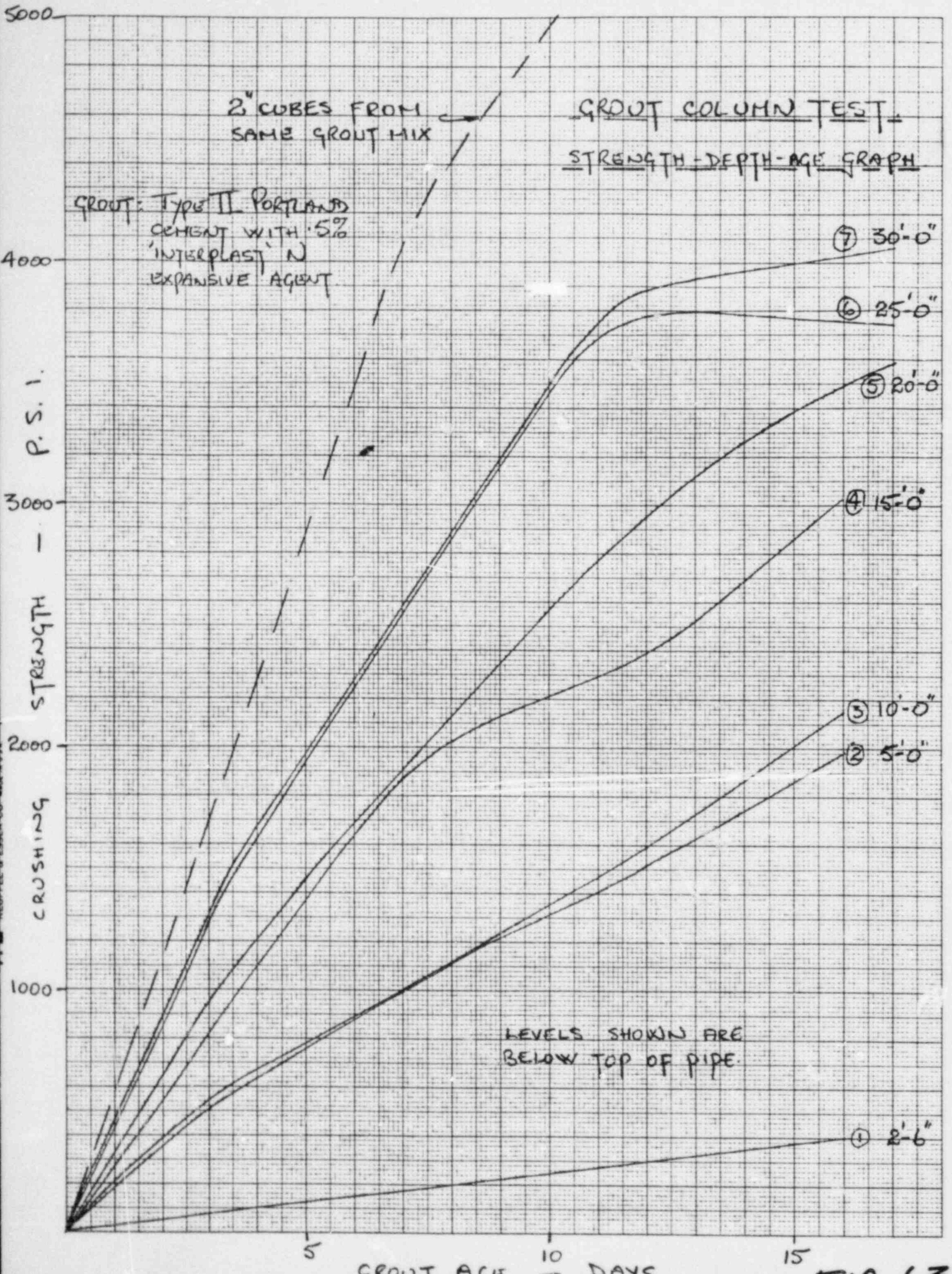

GROUT COLUMN TEST.
STRENGTH-DEPTH-AGE GRAPH
2" CUBES FROM SAME GROUT MIX
GROUT: TYPE II PORTLAND CEMENT WITH ·5% 'INTERPLAST' N EXPANSIVE AGENT
⑦ 30'-0"
⑥ 25'-0"
⑤ 20'-0"
④ 15'-0"
③ 10'-0"
② 5'-0"
① 2'-6"
LEVELS SHOWN ARE BELOW TOP OF PIPE.
5000
4000
3000
2000
1000
CRUSHING STRENGTH – P.S.I.
5
10
15
GROUT AGE – DAYS

FIG. 63.

GROUT COLUMN TEST

Column Height 41'3"

Column Diameter 4"

Date Tested 5-11-76

Distance Gauge 1 to Bottom 6'8"

Distance Gauge 2 to Bottom 2"-4"

Grout: Type II Portland Cement with .5% Interplast 'N' Expanding Agent. Water 5 gals/bag.

TIME	GAUGE 1 P.S.I.	GAUGE 2 P.S.I.	REMARKS
09:55	11.75	13.5	Column filled with water only
10:03	22.25	25.5	Grout filled and tremie pipe removed
10:05	23.5	26.5	Column topped up replacing tremie pipe volume.
10:10	23.25	26.0	
10:15	"	24.25	Slight clear water drips starting at both gauges
10:20	"	24.0	Bleed water forming; fluid level 3/4" below top of column
10:25	"	23.75	
10:30	"	23.5	Clear water drips continue at Gauges. Also at pipe joint 10'6" above bottom.
10:35	"	"	
10:40	"	"	2" deep bleed water formed; fluid level 1" below top of column
10:45	"	"	Clear water drips slowing
10:50	23.0	23.0	
10:55	"	23.0	
11:00	23.25	23.25	
11:05	"	23.0	Fluid level back to 3/4" below top of column.
11:10	23.5	"	
11:15	"	"	
11:20	23.25	"	Slow clear water drips continue.
11:25	23.0	"	
11:30	"	23.25	Bleed water starting to overtop column.
11:35	23.25	23.5	
11:40	"	"	
11:45	23.0	"	Grout risen to within 1/8" of top of column.
11:55	23.25	23.75	
12:00	23.5	"	Grout overtopping column. Visible signs of chemical reaction.
12:15	23.25	"	
12:30	"	23.5	Expansion rate approximately 1/8"/10 mins.
13:00	22.5	23.0	Slow clear water drips continue from gauges and joint.
13:15	"	22.75	
13:30	22.0	22.0	Grout continues to expand & overtop column.
14:00	20.5	20.5	Slow clearwater drips continuing.
14:00	20.5	20.5	Expansion virtually ceased. Bleedwater reforming at top of column. Foam on top of water.
14:30	16.5	17.0	No fluid overtopping column. Foam still forming. Clear water drips at gauges and joint ceased.

FIG. 64.

TIME	GAUGE 1 P.S.I.	GAUGE 2 P.S.I.	REMARKS
15:00	16.5	17.0	Foaming
15:20	15.0	16.0	"
15:40	14.5	17.5	"
16:10	13.0	23.5	"
16:30	12.5	22.0	Foaming ceased.
5/12/76			
09:00	0.0	0.0	1.55 ft. of bleedwater at top of column Top of grout very soft.

FIG. 64A

APPENDIX C

BORING LOGS FOR RATH 1 THROUGH 7

VIRGINIA ELECTRIC AND POWER COMPANY

SH 1 OF 2

SITE NORTH ANNA POWER STATION . UNITS 3 & 4 J.O. No. 12180/12181 BORING No. RATH-1
TYPE OF BORING NX CORE LOCATION N 5109 E 9087 GROUND ELEV. 236 FT
DATE DRILLED 12/11-12/75 DRILLED BY NICHOLSON/TRIPLETT LOGGED BY DPH
SUMMARY OF BORING GROUNDWATER @ 21.0 FT DEPTH 12/11/75. CORED WITH SPLIT INNER CORE BARREL.
STRIKES AND DIPS OF JOINTS OBTAINED WITH GONIOMETER USING CONSISTENT
FOLIATION PLANES WITH ASSUMED STRIKE OF N50E.

ELEV. FEET	DEPTH FEET	OVERALL WEATHERING AND RQD 0 25 50 75 100	SAMPLE BLOWS RECOV.	SAMPLE TYPE	GRAPHIC LOG	SOIL OR ROCK DESCRIPTION: FIELD AND LABORATORY TEST RESULTS; OR JOINTING, BEDDING AND FAULTING DESCRIPTIONS	SOIL OR ROCK DESCRIPTION: SOIL STRATA DESCRIPTION; LITHOLOGY AND TEXTURE
	0.0						
235						CONCRETE TEST PAD	
	5.0						
230	6.2					TOP OF ROCK @ 6.2'	GRAY GRANITE GNEISS, FRESH, VERY HARD, SLIGHTLY FOLIATED @ 45°, MGD. CLOSE TO CLOSELY JOINTED, FINE GRAINED, SOME BIOTITE.
		100%	100%	NX 1			
225	10.7						
		100%	100%	NX 2		12.1 FEET N50E, 55NW FOLIATION JOINT CLEAN	
	15.4						
220		100%	100%	NX 3		17.5 FEET N23E, 40SE, SLIGHTLY IRREGULAR JOINT	
	19.8					19.8 FEET N38E, 59SE, IRREGULAR JOINT, IRON OXIDE STAIN	
215						20.5 N28W, 63SW, SLIGHTLY IRREGULAR JOINT, SLIGHT CHLORITE COATING	
		33%	100%	NX 4		23.1 N5E, 58W JOINT, SLIGHT CHLORITE AND HEMATITE COATING	
						23.5 N30E, 44NW IRREGULAR JOINT, WHITE CLAY COATING	
	24.3					23.6 AND 23.9 N15W, 65SW, JOINTS, WHITE CLAY COATING	
	25.0					24.1 N10E, 58NW JOINT	
210						24.2 N15E, 30NW-0° JOINT, IRON OXIDE STAINING	
		85%	100%	NX 5		26.1 N30E, 30-80°NW, JOINT, IRREGULAR IRON OXIDE STAIN	
						26.3 N20E, 67 NW, JOINT, IRON OXIDE STAIN	
	28.8					27.0 N09E, 62 N . JOINT, IRON OXIDE STAIN	
						28.0 N25E, 43 NW, CURVED JOINT	
	30.0	80%	94%	NX 6		29.0 N55E, 50SE JOINT	
						30.4 AND 30.5 N40E, 35SE JOINTS, SLIGHT IRON OXIDE STAIN	
205						30.8 N60E, 15-20 SE JOINT IRON OXIDE STAIN	
	32.7					33 3 N38E, 21SE STAIN, IRON OXIDE STAIN	
						35.0 N50E, 58 NW, IRON OXIDE STAIN, CLEAN	
	35.0	87%	100%	NX 7		36.3 N-S, VERTICAL JOINT AND N35E, 42SE JOINT, BOTH IRON OXIDE STAINED	
200						36.5 AND 36.8 N37E, 20SE JOINTS, IRON OXIDE STAIN	36.8 FEET-46.3 FEET MASSIVE AND FRESH
	37.3						
		97%	97%	NX 8			
	40.0						

1. FIGURES IN BLOW OR RECOVERY COLUMN OPPOSITE SOIL SAMPLE DENOTE THE NUMBER OF BLOWS OF A 140 LB HAMMER FALLING 30" REQUIRED TO DRIVE A 2" OD SAMPLE SPOON 12" OR THE DISTANCE SHOWN. FIGURES SHOWN OPPOSITE ROCK CORES DENOTE THE PERCENT OF CORE RECOVERED.
2. ■ INDICATES LOCATION OF UNDISTURBED SAMPLE.
   ◤ INDICATES LOCATION OF SPLIT-SPOON SAMPLE.
   ☐ INDICATES LOCATION OF SAMPLING ATTEMPT ◤ WITH NO RECOVERY.
   SUBSCRIPT NEXT TO SYMBOL INDICATES SAMPLE NUMBER.
3. ▽ INDICATES LOCATION OF NATURAL GROUND WATER TABLE.
4. RQD - ROCK QUALITY DESIGNATION.
5. [] INDICATES DEPTH & LENGTH OF NX CORING RUN.
6. DATUM IS MEAN SEA LEVEL

4	
3	
2	
1	

BORING LOG RATH-1
NORTH ANNA POWER STATION
UNITS 3 & 4
VIRGINIA ELECTRIC AND POWER COMPANY
STONE & WEBSTER ENGINEERING CORPORATION

VIRGINIA ELECTRIC AND POWER COMPANY

SH 2 OF 2

SITE NORTH ANNA POWER STATION , UNITS 3 & 4 J.O. No. 12180/12181 BORING No. BATH-1
TYPE OF BORING NX CORE LOCATION N 5109 E9087 GROUND ELEV. 236 FT
DATE DRILLED 12/11-12/75 DRILLED BY NICHOLSON/TRIPLETT LOGGED BY DRW
SUMMARY OF BORING GROUNDWATER @ 21.0 FT DEPTH 12/11/75, CORED WITH SPLIT INNER CORE BARREL,
STRIKES AND DIPS OF JOINTS OBTAINED WITH GONIOMETER USING CONSISTENT
FOLIATION PLANES WITH ASSUMED STRIKE OF N50E

ELEV. FEET	DEPTH FEET	OVERALL WEATHERING AND RQD 0 25 50 75 100	SAMPLE BLOWS RECOV.	SAMPLE TYPE	GRAPHIC LOG	SOIL OR ROCK DESCRIPTION: FIELD AND LABORATORY TEST RESULTS; OR JOINTING, BEDDING AND FAULTING DESCRIPTIONS / SOIL STRATA DESCRIPTION; LITHOLOGY AND TEXTURE
195	40.0	97%	97%	NX 8		
	41.8	100%	100%	NX 9		
190	45.0 46.3	97%	97%	NX 10		49.1 N50E, 55NW,FOLIATION JOINT 50.1 N10W, 77NE JOINT, CLEAN
185	50.5	100%	100%	NX 11		
	51.8	100%	100%	NX 12		54.8 N10W, 50-55 SW JOINT, CLEAN
180	55.0	100%	100%	NX 13		
	56.6					END OF BORING AT 56.6'

1. FIGURES IN BLOW OR RECOVERY COLUMN OPPOSITE SOIL SAMPLE DENOTE THE NUMBER OF BLOWS OF A 140 LB HAMMER FALLING 30" REQUIRED TO DRIVE A 2" OD SAMPLE SPOON 12" OR THE DISTANCE SHOWN. FIGURES SHOWN OPPOSITE ROCK CORES DENOTE THE PERCENT OF CORE RECOVERED.
2. ■ INDICATES LOCATION OF UNDISTURBED SAMPLE.
   ◩ INDICATES LOCATION OF SPLIT-SPOON SAMPLE.
   ☐ INDICATES LOCATION OF SAMPLING ATTEMPT WITH NO RECOVERY.
   SUBSCRIPT NEXT TO SYMBOL INDICATES SAMPLE NUMBER.
3. ▽ INDICATES LOCATION OF NATURAL GROUND WATER TABLE.
4. RQD - ROCK QUALITY DESIGNATION.
5. [] INDICATES DEPTH & LENGTH OF NX-CORING RUN.
6. DATUM IS MEAN SEA LEVEL

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BORING LOG BATH-1
NORTH ANNA POWER STATION
UNITS 3 & 4
VIRGINIA ELECTRIC AND POWER COMPANY
STONE & WEBSTER ENGINEERING CORPORATION

VIRGINIA ELECTRIC AND POWER COMPANY

SH 1 OF 2

SITE NORTH ANNA POWER STATION . UNITS 3 & 4 J.O. No. 12180/12181 BORING No. BATH-2
TYPE OF BORING NX CORE LOCATION N 5109 E 9095 GROUND ELEV. 236
DATE DRILLED 12/15/75 DRILLED BY NICHOLSON/TRIPLETT LOGGED BY D.P.H.
SUMMARY OF BORING CORED WITH SPLIT INNER CORE BARREL. STRIKES AND DIPS OF JOINTS WERE OBTAINED WITH A GONIOMETER USING CONSISTANT FOLIATION PLANES WITH ASSUMED STRIKE OF N50E.

ELEV. FEET	DEPTH FEET	OVERALL WEATHERING AND RQD 0 25 50 75 100	SAMPLE BLOWS RECOV.	SAMPLE TYPE	GRAPHIC LOG	SOIL OR ROCK DESCRIPTION: FIELD AND LABORATORY TEST RESULTS, OR JOINTING, BEDDING AND FAULTING DESCRIPTIONS	SOIL OR ROCK DESCRIPTION: SOIL STRATA DESCRIPTION, LITHOLOGY AND TEXTURE
230	0.0						CONCRETE TEST PAD
	5.5					TOP OF ROCK AT 5.5'	
225		88%	100%	NX 1		5.6 N46W, 20NE JOINT 6.7 N17W, 69SW JOINT, WEATHERED FELDSPAR COATING 8.1 HORIZONTAL JOINTS, IRON OXIDE STAINING	GRAY GRANITE GNEISS, SLIGHTLY WEATHERED TO FRESH, MODERATELY CLOSE JOINTING, FINE GRAINED, 45° FOLIATION, HARD, SOME BIOTITE.
	9.0 10.0						
220		95%	97%	NX 2		11.5 N55W, 19NE, IRREGULAR JOINT, IRON OXIDE STAINING 12.3 N50E, 45NW, FOLIATION JOINT 13.0 N15W, 21NE, JOINT, IRON OXIDE STAINING	
	13.0					14.3 N50E, 57NW, FOLIATION JOINT 14.4 N27E, 65NE, JOINT	
	15.0	100%	100%	NX 3			
215						15.9 N 0E, 62NW FOLIATION JOINT, ROUGH, IRON OXIDE STAINING 17. N70E, 38SE JOINT, IRON OXIDE STAINING	
	17.7					19.7 N25E, 40SE JOINT, SLIGHT IRON OXIDE STAINING	19.7-2" MODERATELY WX. ZONE ADJACENT TO JOINT.
	20.0	100%	100%	NX 4			
210							
	22.0					22.6 N75E, 31NW JOINT, SLIGHT IRON OXIDE STAINING 23.5 N80W, 18NE JOINT, SLIGHT IRON OXIDE STAINING	
		97%	97%	NX 5		24.9 N33W, 25NE JOINT 25.3 N35E, 18SE JOINT, SLIGHT IRON OXIDE STAINING	
	25.0					25.7 N35E, 20SE JOINT, SLIGHT WEATHERING 26.1 N20E, 20SE JOINT, IRON OXIDE STAINING	26.5-27.1 PEGMATITE VEIN WITH MAGNETITE INCLUSIONS, VERY HARD.
205	26.8					27.3 N50E, 46NW, FOLIATION JOINT, IRON OXIDE STAINING	
		100%	100%	NX 6		28.8 N33E, 25SE JOINT, SLIGHT IRON OXIDE STAINING	
200	31.4					31.4 N35E, 40SE JOINT, CLEAN	31.4-36.8 MASSIVE AND FRESH.
		100%	100%	NX 7			
195	36.0					36.8 N10W, 35NE JOINT, IRON OXIDE STAINING	
		100%	100%	NX 8		37.4 & 37.7 N60E, 48SE JOINTS, IRON OXIDE STAINING 37.8 & 38.8 N35E, 30SE JOINTS, IRON OXIDE STAINING	
	40.0						

1. FIGURES IN BLOW OR RECOVERY COLUMN OPPOSITE SOIL SAMPLE DENOTE THE NUMBER OF BLOWS OF A 140 LB HAMMER FALLING 30" REQUIRED TO DRIVE A 2" OD SAMPLE SPOON 12" OR THE DISTANCE SHOWN. FIGURES SHOWN OPPOSITE ROCK CORES DENOTE THE PERCENT OF CORE RECOVERED.
2. ■ INDICATES LOCATION OF UNDISTURBED SAMPLE.
   ◤ INDICATES LOCATION OF SPLIT-SPOON SAMPLE.
   ☐ INDICATES LOCATION OF SAMPLING ATTEMPT
   ◸ WITH NO RECOVERY.
   SUBSCRIPT NEXT TO SYMBOL INDICATES SAMPLE NUMBER.
3. ▽ INDICATES LOCATION OF NATURAL GROUND WATER TABLE.
4. RQD - ROCK QUALITY DESIGNATION.
5. [] INDICATES DEPTH & LENGTH OF NX CORING RUN.
6. DATUM IS MEAN SEA LEVEL.

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BORING LOG BATH-2

NORTH ANNA POWER STATION
UNITS 3 & 4
VIRGINIA ELECTRIC AND POWER COMPANY

STONE & WEBSTER ENGINEERING CORPORATION

VIRGINIA ELECTRIC AND POWER COMPANY

SH 2 OF 2

SITE NORTH ANNA POWER STATION - UNITS 3 & 4 J.O. No. 12180/12181 BORING No. BATH-2
TYPE OF BORING NX CORE LOCATION N 5209 E 9095 GROUND ELEV. 236
DATE DRILLED 12/15/75 DRILLED BY NICHOLSON/TRIPLETT LOGGED BY D.P.H.
SUMMARY OF BORING CORED WITH SPLIT INNER CORE BARREL. STRIKES AND DIPS OF JOINTS WERE OBTAINED WITH A GONIOMETER USING CONSISTANT FOLIATION PLANES WITH ASSUMED STRIKE OF N50E.

ELEV. FEET	DEPTH FEET	OVERALL WEATHERING AND RQD 0 25 50 75 100	SAMPLE BLOWS RECOV.	SAMPLE TYPE	GRAPHIC LOG	SOIL OR ROCK DESCRIPTION FIELD AND LABORATORY TEST RESULTS, OR JOINTING, BEDDING AND FAULTING DESCRIPTIONS; SOIL STRATA DESCRIPTION; LITHOLOGY AND TEXTURE
190	40.0	100%	100%	NX 9		
	[illegible].3					
185	45.0	100%	100%	NX 10		
	49.1					49.2 N70E, 18SE JOINT, 1/2" PEGMATITE ADJACENT TO JOINT
180	50.0	100%	100%	NX 11		
	53.7					
175	55.0	100%	100%	NX 12		56.0 N60E, 46SE JOINT, IRON OXIDE STAINING 56.8 N50W, 75NE JOINT, SLIGHT CHLORITE COATING
	57.1					END OF BORING AT 57.1'

1. FIGURES IN BLOW OR RECOVERY COLUMN OPPOSITE SOIL SAMPLE DENOTE THE NUMBER OF BLOWS OF A 140 LB HAMMER FALLING 30" REQUIRED TO DRIVE A 2" OD SAMPLE SPOON 12" OR THE DISTANCE SHOWN. FIGURES SHOWN OPPOSITE ROCK CORES DENOTE THE PERCENT OF CORE RECOVERED.
2. ■ INDICATES LOCATION OF UNDISTURBED SAMPLE.
   ◤ INDICATES LOCATION OF SPLIT-SPOON SAMPLE.
   ☐ INDICATES LOCATION OF SAMPLING ATTEMPT ◸ WITH NO RECOVERY.
   SUBSCRIPT NEXT TO SYMBOL INDICATES SAMPLE NUMBER.
3. ▽ INDICATES LOCATION OF NATURAL GROUND WATER TABLE.
4. RQD - ROCK QUALITY DESIGNATION.
5. ⌶ INDICATES DEPTH & LENGTH OF NX CORING RUN.
6. DATUM IS MEAN SEA LEVEL.

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BORING LOG BATH-2

NORTH ANNA POWER STATION
UNITS 3 & 4
VIRGINIA ELECTRIC AND POWER COMPANY

STONE & WEBSTER ENGINEERING CORPORATION

VIRGINIA ELECTRIC AND POWER COMPANY

SH 1 OF 2

SITE NORTH ANNA POWER STATION . UNITS 3 & 4 J.O. No. 12180/12181 BORING No. BATH-1
TYPE OF BORING NX CORE LOCATION N 5075 E 9124 GROUND ELEV. 245
DATE DRILLED 12/9-10/75 DRILLED BY NICHOLSON/TRIPLETT LOGGED BY D.P. HAGSTROM
SUMMARY OF BORING

ELEV. FEET	DEPTH FEET	OVERALL WEATHERING AND RQD (0 25 50 75 100)	SAMPLE: BLOWS RECOV.	SAMPLE: TYPE	GRAPHIC LOG	SOIL OR ROCK DESCRIPTION: FIELD AND LABORATORY TEST RESULTS; OR JOINTING, BEDDING AND FAULTING DESCRIPTIONS	SOIL OR ROCK DESCRIPTION: SOIL STRATA DESCRIPTION; LITHOLOGY AND TEXTURE
245	0.0						
						CONCRETE TEST PAD TO 5.2'	
						FILL CONCRETE FROM 5.2' TO 6.6'	
240	5.0						
	6.6					TOP OF ROCK AT 6.6'	
		64%	85%	NX 1			GRAY GRANITE GNEISS, MODERATELY WEATHERED, MODERATELY CLOSE JOINTING, FINE GRAINED, FOLIATION AT 45° MODERATELY HARD, SOME BIOTITE. MODERATELY WEATHERED TO 9.8' BECOMING SLIGHTLY WEATHERED.
	9.8					9.5 & 9.8 N20E, 30SE JOINTS, IRON OXIDE STAINING	
235							
		100%	100%	NX 2		12.1 N5E, 15SE JOINT, CLAY COATING	@ 13.3 MAGNETITE CRYSTALS
						13.4 N48W, 63NE, JOINT, MODERATELY WEATHERED	
	14.5					14.9 N35E, 20SE JOINT, IRON OXIDE STAINING, 1/2" SERVERLY WEATHERED ZONE ABOVE JOINT	14.9' - ROCK BECOMES FRESH WITH MODERATELY CLOSE JOINTING
230	15.0						
		100%	100%	NX 3			
	19.2						
225	20.0						
		92%	100%	NX 4		21.3 N35E, 26SE JOINT	
						22.5 & 22.6 N29E, 22SE JOINTS IRON OXIDE STAINING	23.5-2" PEGMATITE
	23.5						
220	25.0	92%	100%	NX 5		25.6 N50E, 10SE JOINT	26.8-27.5 - DISSEMINATED PYRITE
						26.6 N25E, 55NW JOINT	
						27.5 N45E, 45NW JOINT, ADULARIA COATING	
	27.9						
215	30.0	100%	100%	NX 6			
	31.0					32.2 & 32.3 N55E, 35NW FOLIATION JOINTS, IRON OXIDE STAINING	
		97%	100%	NX 7			
210	35.0						
	35.7					37.2 N55E, 55NW FOLIATION JOINT, 1/4" SOFT GREEN CHLORITE COATING	
						37.3 N65E, 30SE JOINT, SLIGHTLY CHLORITE COATING	
		94%	100%	NX 8		37.5 N15W, 30NE JOINT, SLIGHTLY SANDY COATING	
						37.6 N45E, 52NW JOINT	
						38.8 N40E, 45SE JOINT, IRON OXIDE STAINING	
						39.0 N5W, 53SW JOINT, SLIGHTLY CHLORITE COATING	
205	40.0						

1. FIGURES IN BLOW OR RECOVERY COLUMN OPPOSITE SOIL SAMPLE DENOTE THE NUMBER OF BLOWS OF A 140 LB HAMMER FALLING 30" REQUIRED TO DRIVE A 2" OD SAMPLE SPOON 12" OR THE DISTANCE SHOWN. FIGURES SHOWN OPPOSITE ROCK CORES DENOTE THE PERCENT OF CORE RECOVERED.
2. ■ INDICATES LOCATION OF UNDISTURBED SAMPLE.
   F INDICATES LOCATION OF SPLIT-SPOON SAMPLE.
   ☐ INDICATES LOCATION OF SAMPLING ATTEMPT
   F WITH NO RECOVERY.
   SUBSCRIPT NEXT TO SYMBOL INDICATES SAMPLE NUMBER.
3. ▽ INDICATES LOCATION OF NATURAL GROUND WATER TABLE.
4. RQD - ROCK QUALITY DESIGNATION.
5. ▯ INDICATES DEPTH & LENGTH OF NX CORING RUN.
6. DATUM IS

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BORING LOG BATH-1

NORTH ANNA POWER STATION
UNITS 3 & 4
VIRGINIA ELECTRIC AND POWER COMPANY

STONE & WEBSTER ENGINEERING CORPORATION

VIRGINIA ELECTRIC AND POWER COMPANY

SH 2 OF 2

SITE NORTH ANNA POWER STATION . UNITS 3 & 4 J.O. No 12180/12181 BORING No. RATH-3
TYPE OF BORING NX CORE LOCATION N 5075 E 9124 GROUND ELEV. 245
DATE DRILLED 12/9-10/75 DRILLED BY NICHOLSON/TRIPLETT LOGGED BY D.P. HAGSTROM
SUMMARY OF BORING

ELEV. FEET	DEPTH FEET	OVERALL WEATHERING AND RQD 0 25 50 75 100	SAMPLE BLOWS RECOV.	SAMPLE TYPE	GRAPHIC LOG	SOIL OR ROCK DESCRIPTION FIELD AND LABORATORY TEST RESULTS, OR JOINTING, BEDDING AND FAULTING DESCRIPTIONS / SOIL STRATA DESCRIPTION; LITHOLOGY AND TEXTURE
205	40.4					40.6 N70E, 55NW JOINT, IRON OXIDE STAINING AND ADULARIA COATING
		100%	100%	NX 9		42.0 N50E, 50NW, FOLIATION JOINT, SLIGHT ADULARIA COATING
200	45.0					44.6 N15E, 30SE, JOINT, IRON OXIDE STAINING
		100%	100%	NX 10		
	47.0					END OF BORING AT 47.0'

1. FIGURES IN BLOW OR RECOVERY COLUMN OPPOSITE SOIL SAMPLE DENOTE THE NUMBER OF BLOWS OF A 140 LB HAMMER FALLING 30" REQUIRED TO DRIVE A 2" OD SAMPLE SPOON 12" OR THE DISTANCE SHOWN. FIGURES SHOWN OPPOSITE ROCK CORES DENOTE THE PERCENT OF CORE RECOVERED.
2. ■ INDICATES LOCATION OF UNDISTURBED SAMPLE.
   ◤ INDICATES LOCATION OF SPLIT-SPOON SAMPLE.
   ☐ INDICATES LOCATION OF SAMPLING ATTEMPT
   ◸ WITH NO RECOVERY.
   SUBSCRIPT NEXT TO SYMBOL INDICATES SAMPLE NUMBER.
3. ▽ INDICATES LOCATION OF NATURAL GROUND WATER TABLE.
4. RQD - ROCK QUALITY DESIGNATION.
5. ☐ INDICATES DEPTH & LENGTH OF NX CORING RUN.
6. DATUM IS MEAN SEA LEVEL.

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BORING LOG RATH-3
NORTH ANNA POWER STATION
UNITS 3 & 4
VIRGINIA ELECTRIC AND POWER COMPANY
STONE & WEBSTER ENGINEERING CORPORATION

VIRGINIA ELECTRIC AND POWER COMPANY

SH 1 OF 2

SITE NORTH ANNA POWER STATION . UNITS 3 & 4 J.O. No 12180/12181 BORING No. RATH-4
TYPE OF BORING NX CORE LOCATION N 5075 E 9132 GROUND ELEV. 245
DATE DRILLED 12/5-6/75 DRILLED BY NICHOLSON/TRIPLETT LOGGED BY DPN
SUMMARY OF BORING GROUNDWATER AT 20.0 FT DEPTH. CORED WITH SPLIT INNER CORE BARREL. STRIKES AND DIPS OF JOINTS WERE OBTAINED WITH A GONIOMETER USING CONSISTENT FOLIATION PLANES WITH AN ASSUMED STRIKE OF N50E

ELEV. FEET	DEPTH FEET	OVERALL WEATHERING AND RQD 0 25 50 75 100	SAMPLE BLOWS RECOV.	SAMPLE TYPE	SOIL OR ROCK DESCRIPTION: FIELD AND LABORATORY TEST RESULTS; OR JOINTING, BEDDING AND FAULTING DESCRIPTIONS	SOIL OR ROCK DESCRIPTION: SOIL STRATA DESCRIPTION, LITHOLOGY AND TEXTURE
245	0.0				CONCRETE TEST PAD TO 5.2'	
240	5.0				FILL CONCRETE FROM 5.2-6.6'	
	6.6				TOP OF ROCK AT 6.6'	GRAY GRANITE GNEISS, FRESH, CLOSELY JOINTED, FINE GRAINED, FOLIATION DIPS 45°, VERY HARD.
		72%	85%	NX 1		
235	9.8				10.1 N50E, 55NW FOLIATION JOINT	6.6-6.8' PEGMATITE, SLIGHT TO MODERATE WEATHERING
		72%	100%	NX 2	14.1' N50E, 38SE JOINT, IRON OXIDE STAINED	
	14.3				14.3-16.8 E-W, VERTICAL JOINT, IRON OXIDE COATING OPEN TO 0.2"	
230	15.0	46%	97%	NX 3		
	19.0					
225	20.0	92%	100%	NX 4	22.1' N75E, 48SE JOINT 22.3' N80E, 35SE JOINT 23.0' N25E, 25SE JOINT, IRON OXIDE STAINED	
	23.7					
220	25.0	100%	100%	NX 5	24.9' N20E, [illegible] JOINT, IRON OXIDE STAINED	
	27.7				28.4 N70E, 5-10NW JOINT, SLIGHT IRON OXIDE STAINED	
215	30.0	100%	100%	NX 6		
	31.8				31.8 N50E, 45NW FOLIATION JOINT	
		84%	97%	NX 7	33.6 N50E, 40NW FOLIATION JOINT	
210	35.0				35.1 N40E, 22SE JOINT 35.4 and 35.5 N50E, 55NW FOLIATION JOINTS, ADULARIA COATINGS 36.7 N50E, 50NW FOLIATION JOINT, CHLORITE COATING.	
	36.5	89%	100%	NX 8	39.0 N20W, 5-10NE JOINT, IRON OXIDE STAINED 39.1 N75W, 70SW JOINT, IRON OXIDE STAINED 39.2 N50W, 15NE JOINT, IRON OXIDE STAINED	
205	40.0					

1. FIGURES IN BLOW OR RECOVERY COLUMN OPPOSITE SOIL SAMPLE DENOTE THE NUMBER OF BLOWS OF A 140 LB HAMMER FALLING 30" REQUIRED TO DRIVE A 2" OD SAMPLE SPOON 12" OR THE DISTANCE SHOWN. FIGURES SHOWN OPPOSITE ROCK CORES DENOTE THE PERCENT OF CORE RECOVERED.
2. ■ INDICATES LOCATION OF UNDISTURBED SAMPLE.
   ◤ INDICATES LOCATION OF SPLIT-SPOON SAMPLE.
   ☐ INDICATES LOCATION OF SAMPLING ATTEMPT
   ◤ WITH NO RECOVERY.
   SUBSCRIPT NEXT TO SYMBOL INDICATES SAMPLE NUMBER.
3. ⊻ INDICATES LOCATION OF NATURAL GROUND WATER TABLE.
4. RQD - ROCK QUALITY DESIGNATION.
5. ⌷ INDICATES DEPTH & LENGTH OF NX CORING RUN.
6. DATUM IS MEAN SEA LEVEL

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BORING LOG RATH-4

NORTH ANNA POWER STATION
UNITS 3 & 4
VIRGINIA ELECTRIC AND POWER COMPANY

STONE & WEBSTER ENGINEERING CORPORATION

VIRGINIA ELECTRIC AND POWER COMPANY

SH 2 OF 2

SITE NORTH ANNA POWER STATION . UNITS 3 & 4 J.O. No 12180/12181 BORING No RATH-4

TYPE OF BORING NX CORE LOCATION N 5075 E 9132 GROUND ELEV 245

DATE DRILLED 12/5-6/73 DRILLED BY NICHOLSON/TRIPLETT LOGGED BY DEN

SUMMARY OF BORING GROUNDWATER AT 20.0 FT DEPTH

ELEV. FEET	DEPTH FEET	OVERALL WEATHERING AND RQD 0 25 50 75 100	SAMPLE BLOWS RECOV.	SAMPLE TYPE	GRAPHIC LOG	SOIL OR ROCK DESCRIPTION FIELD AND LABORATORY TEST RESULTS; OR JOINTING, BEDDING AND FAULTING DESCRIPTIONS / SOIL STRATA DESCRIPTION; LITHOLOGY AND TEXTURE
205	40.1					40.1 E-W, 45S JOINT, ADULARIA COATING
						40.2 N65W, 40SW JOINT, ADULARIA COATING
						41.[illegible] N50E, 50NW FOLIATION JOINT, SLIGHT IRON OXIDE STAINING
		100%	100%	NX 9		
	43.6					
						44.5 N15E, 25SE JOINT, SLIGHT IRON OXIDE STAINING
200	45.0	92%	100%	NX 10		
						46.1-47.2 E-W, VERTICAL TO 80S JOINT, IRON OXIDE STAINED
	47.2					END OF BORING AT 47.2'

1. FIGURES IN BLOW OR RECOVERY COLUMN OPPOSITE SOIL SAMPLE DENOTE THE NUMBER OF BLOWS OF A 140 LB HAMMER FALLING 30" REQUIRED TO DRIVE A 2" OD SAMPLE SPOON 12" OR THE DISTANCE SHOWN. FIGURES SHOWN OPPOSITE ROCK CORES DENOTE THE PERCENT OF CORE RECOVERED.
2. ■ INDICATES LOCATION OF UNDISTURBED SAMPLE.
   ▼ INDICATES LOCATION OF SPLIT-SPOON SAMPLE.
   ☐ INDICATES LOCATION OF SAMPLING ATTEMPT
   ▽ WITH NO RECOVERY.
   SUBSCRIPT NEXT TO SYMBOL INDICATES SAMPLE NUMBER.
3. ▽ INDICATES LOCATION OF NATURAL GROUND WATER TABLE.
4. RQD - ROCK QUALITY DESIGNATION.
5. ▯ INDICATES DEPTH & LENGTH OF NX CORING RUN.
6. DATUM IS MEAN SEA LEVEL

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BORING LOG RATH-4

NORTH ANNA POWER STATION
UNITS 3 & 4
VIRGINIA ELECTRIC AND POWER COMPANY

STONE & WEBSTER ENGINEERING CORPORATION

VIRGINIA ELECTRIC AND POWER COMPANY

SH 1 OF 2

SITE NORTH ANNA POWER STATION . UNITS 3 & 4 J.O. No. 12180/12181 BORING No. RATH-5
TYPE OF BORING NX CORE LOCATION N 5013 E 9250 GROUND ELEV. 245'
DATE DRILLED 3/11/76 DRILLED BY NICHOLSON/TRIPLETT LOGGED BY DPR
SUMMARY OF BORING CORED WITH SPLIT INNER CORE BARREL

ELEV. FEET	DEPTH FEET	OVERALL WEATHERING AND RQD 0 25 50 75 100	SAMPLE BLOWS RECOV.	SAMPLE TYPE	GRAPHIC LOG	SOIL OR ROCK DESCRIPTION: FIELD AND LABORATORY TEST RESULTS; OR JOINTING, BEDDING AND FAULTING DESCRIPTIONS	SOIL STRATA DESCRIPTION; LITHOLOGY AND TEXTURE
245	0.0					0-4.0' CONCRETE TEST PAD	
240	5.0					HOLE REAMED TO 24.0' WITH 6.5" ROTARY BIT BEFORE START OF CORING	
235	10.0						
230	15.0						
225	20.0						
221	24.0					START OF CORING AT 24.0'	
220	25.0	78%	100%	NX 1		24.1 45° JOINT, SLIGHT IRON OXIDE STAINING 25.5 HEALED 60° JOINT, CLEAN 26.5 45° FOLIATION JOINT, SLIGHTLY WEATHERED, SLIGHTLY OPEN, IRON OXIDE STAINING 27.0 60° JOINT, SLIGHT IRON OXIDE STAINING 27.5, 27.6, & 27.7 60° JOINTS, SLIGHT EPIDOTE COATINGS	GRAY GRANITE GNEISS, SLIGHTLY WEATHERED TO FRESH, CLOSE TO MODERATELY CLOSE JOINTING, MEDIUM GRAINED, HARD TO VERY HARD, FOLIATION DIPS 45°
	28.5					28.2 45° FOLIATION JOINT, SLIGHT IRON OXIDE STAINING 28.9-30.3' CLOSELY SPACED 45° FOLIATION JOINTS	
215	30.0	91%	100%	NX 2		29.8 30° JOINT, SLIGHT EPIDOTE COATING 31.0 20° JT., SLIGHTLY WEATHERED, SLIGHTLY VUGGY 31.3 80° JT. SL. WEATHERED, EPIDOTE COATING 31.7 20° JOINTS, MODERATELY WEATHERED	30.9 - ROCK BECOMES SLIGHTLY TO MODERATELY WEATHERED AND SLIGHTLY VUGGY WITH QUARTZ CRYSTALS
	33.0					33.0 20° JOINT, IRON OXIDE STAINING, VUGGY 34.3 80° JOINT, EPIDOTE COATING	
210	35.0	87%	100%	NX 3		35.4, 36.2, & 37.4 45° FOLIATION JOINTS, CHLORITE COATINGS	33.1 - ROCK BECOMES FRESH TO SLIGHTLY WEATHERED
	37.5					38.0 30° JOINT, SLIGHT EPIDOTE COATING	
		89%	100%	NX 4		38.7 60° JOINT, SLIGHT EPIDOTE COATING 39.2 [illegible] JOINT, SLIGHT EPIDOTE COATING	
205	40.0					40.0 45° JOINT, SLIGHT EPIDOTE COATING	

1. FIGURES IN BLOW OR RECOVERY COLUMN OPPOSITE SOIL SAMPLE DENOTE THE NUMBER OF BLOWS OF A 140 LB HAMMER FALLING 30" REQUIRED TO DRIVE A 2" OD SAMPLE SPOON 12" OR THE DISTANCE SHOWN. FIGURES SHOWN OPPOSITE ROCK CORES DENOTE THE PERCENT OF CORE RECOVERED.
2. ■ INDICATES LOCATION OF UNDISTURBED SAMPLE.
   ▼ INDICATES LOCATION OF SPLIT-SPOON SAMPLE.
   ☐ INDICATES LOCATION OF SAMPLING ATTEMPT WITH NO RECOVERY.
   SUBSCRIPT NEXT TO SYMBOL INDICATES SAMPLE NUMBER.
3. ▼ INDICATES LOCATION OF NATURAL GROUND WATER TABLE.
4. RQD - ROCK QUALITY DESIGNATION.
5. ☐ INDICATES DEPTH & LENGTH OF NX CORING RUN.
6. DATUM IS MEAN SEA LEVEL

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BORING LOG RATH-5

NORTH ANNA POWER STATION
UNITS 3 & 4
VIRGINIA ELECTRIC AND POWER COMPANY
STONE & WEBSTER ENGINEERING CORPORATION

VIRGINIA ELECTRIC AND POWER COMPANY

SH 2 OF 2

SITE NORTH ANNA POWER STATION . UNITS 3 & 4 J.O. No. 12180/12181 BORING No. RATH-5
TYPE OF BORING NX CORE LOCATION N 5013 E 9250 GROUND ELEV. 245'
DATE DRILLED 2/11/76 DRILLED BY B.CRULSON/TRIPLETT LOGGED BY DPH
SUMMARY OF BORING

SOIL OR ROCK DESCRIPTION

ELEV. FEET	DEPTH FEET	OVERALL WEATHERING AND RQD 0 25 50 75 100	SAMPLE BLOWS RECOV.	SAMPLE TYPE	FIELD AND LABORATORY TEST RESULTS, OR JOINTING, BEDDING AND FAULTING DESCRIPTIONS	SOIL STRATA DESCRIPTION; LITHOLOGY AND TEXTURE
205	40.0	89%	100%	NX 4	40.6 70° JOINT, EPIDOTE COATING 40.8 45° JOINT, EPIDOTE COATING 41.1 45° JOINT, EPIDOTE COATING	
200	42.0 45.0	64%	100%	NX 5	42.3 45° JOINT, EPIDOTE COATING 42.8 45° JOINT, EPIDOTE COATING 43.6 20° JOINT, ROUGH, CLEAN 44.8 45° JOINT, IRON OXIDE STAINING 44.9 35° JOINT, SLIGHT EPIDOTE COATING 45.2 50° JOINT 45.7 60° JOINT, SLIGHT EPIDOTE COATING 46.5 45° JOINT, SLIGHT EPIDOTE CTG.	46.5-49.7 HEALED BROKEN ZONE, SLIGHTLY WEATHERED, IRON OXIDE STAINING
195	46.5 50.0	60%	100%	NX 6	48.6 80° JOINT, OPEN TO 0.1", GREEN CLAY FILLING 50.4 45° JOINT, EPIDOTE COATING 50.7 TWO 45° JOINTS, EPIDOTE COATINGS	
190	51.0 55.0	100%	100%	NX 7	51.4 45° JOINT, EPIDOTE COATING 52.8 50° JOINT, SLIGHTLY WEATHERED, IRON OXIDE STAINING 53.2 20° JOINT 54.9 25° JOINT, IRON OXIDE STAINING, EPIDOTE COATING 55.5 40° JOINT, IRON OXIDE STAINING, EPIDOTE COATING	
189.5	55.5				END OF BORING AT 55.5'	

1. FIGURES IN BLOW OR RECOVERY COLUMN OPPOSITE SOIL SAMPLE DENOTE THE NUMBER OF BLOWS OF A 140 LB HAMMER FALLING 30" REQUIRED TO DRIVE A 2" OD SAMPLE SPOON 12" OR THE DISTANCE SHOWN. FIGURES SHOWN OPPOSITE ROCK CORES DENOTE THE PERCENT OF CORE RECOVERED.
2. ■ INDICATES LOCATION OF UNDISTURBED SAMPLE.
   ◤ INDICATES LOCATION OF SPLIT-SPOON SAMPLE.
   ☐ INDICATES LOCATION OF SAMPLING ATTEMPT
   ◸ WITH NO RECOVERY.
   SUBSCRIPT NEXT TO SYMBOL INDICATES SAMPLE NUMBER.
3. ▽ INDICATES LOCATION OF NATURAL GROUND WATER TABLE.
4. RQD - ROCK QUALITY DESIGNATION.
5. [] INDICATES DEPTH & LENGTH OF NX CORING RUN.
6. DATUM IS MEAN SEA LEVEL

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BORING LOG RATH-5
NORTH ANNA POWER STATION
UNITS 3 & 4
VIRGINIA ELECTRIC AND POWER COMPANY
STONE & WEBSTER ENGINEERING CORPORATION

VIRGINIA ELECTRIC AND POWER COMPANY

SH 1 OF 2

SITE NORTH ANNA POWER STATION . UNITS 3 & 4 J.O. No. 12180/12181 BORING No. RATH-6
TYPE OF BORING NX CORE LOCATION N 5013 E 9258 GROUND ELEV. 245
DATE DRILLED 2/24/76 DRILLED BY NICHOLSON/TRIPLETT LOGGED BY DPH
SUMMARY OF BORING

ELEV. FEET	DEPTH FEET	OVERALL WEATHERING AND RQD 0 25 50 75 100	SAMPLE BLOWS RECOV.	SAMPLE TYPE	GRAPHIC LOG	SOIL OR ROCK DESCRIPTION: FIELD AND LABORATORY TEST RESULTS; OR JOINTING, BEDDING AND FAULTING DESCRIPTIONS	SOIL STRATA DESCRIPTION; LITHOLOGY AND TEXTURE
245	0.0					0-5.0' CONCRETE TEST PAD	
240	5.0					HOLE REAMED TO 25.0' WITH 6.5" ROTARY BIT BEFORE START OF CORING	
235	10.0						
230	15.0						
225	20.0						
220	25.0					START OF CORING AT 25.0'	
		82%	100%	NX 1		25.3 45° FOLIATION JOINT, SLIGHT ADULARIA COATING 25.7 45° FOLIATION JOINT 26.0, 26.1, & 26.2 45° FOLIATION JTS. 26.7 60° JOINT, SLIGHT EPIDOTE COATING 28.6 & 28.7 45° FOL. JTS., SL. TO MOD. WX., SL. VUGGY, SL. CLAY COATING	GRAY GRANITE GNEISS, SLIGHTLY WEATHERED TO FRESH, CLOSE TO MODERATELY CLOSE JOINTING, MEDIUM GRAINED, HARD TO VERY HARD, FOLIATION DIPS 45°
	29.5						
215	30.0	100%	100%	NX 2		30.0 50° JOINT, ADULARIA COATING 32.7 50° JOINT, EPIDOTE COATING 33.8 45° JOINT, EPIDOTE COATING, IRON OXIDE STAINING	30.0-31.1 PEGMATITE VEIN, FRESH, VERY HARD
	34.0						
210	35.0	100%	100%	NX 3		35.2 80° JOINT, ADULARIA COATING 38.2, 38.6, & 38.9 45° JOINTS, ADULARIA AND EPIDOTE COATINGS	
	38.8						
205		89%	100%	NX 4			

1. FIGURES IN BLOW OR RECOVERY COLUMN OPPOSITE SOIL SAMPLE DENOTE THE NUMBER OF BLOWS OF A 140 LB HAMMER FALLING 30" REQUIRED TO DRIVE A 2" OD SAMPLE SPOON 12" OR THE DISTANCE SHOWN. FIGURES SHOWN OPPOSITE ROCK CORES DENOTE THE PERCENT OF CORE RECOVERED.
2. ■ INDICATES LOCATION OF UNDISTURBED SAMPLE.
   ◤ INDICATES LOCATION OF SPLIT-SPOON SAMPLE.
   ☐ INDICATES LOCATION OF SAMPLING ATTEMPT ◸ WITH NO RECOVERY.
   SUBSCRIPT NEXT TO SYMBOL INDICATES SAMPLE NUMBER.
3. ▽ INDICATES LOCATION OF NATURAL GROUND WATER TABLE.
4. RQD - ROCK QUALITY DESIGNATION.
5. ▯ INDICATES DEPTH & LENGTH OF NX CORING RUN.
6. DATUM IS MEAN SEA LEVEL

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BORING LOG RATH-6

NORTH ANNA POWER STATION
UNITS 3 & 4
VIRGINIA ELECTRIC AND POWER COMPANY
STONE & WEBSTER ENGINEERING CORPORATION

VIRGINIA ELECTRIC AND POWER COMPANY

SH 2 OF 2

SITE NORTH ANNA POWER STATION . UNITS 3 & 4 J.O. No. 12180/12181 BORING No. RATH-6
TYPE OF BORING NX CORE LOCATION N 5013 E 9258 GROUND ELEV. 245'
DATE DRILLED 2/24/76 DRILLED BY NICHOLSON/TRIPLETT LOGGED BY DPH
SUMMARY OF BORING

ELEV. FEET	DEPTH FEET	OVERALL WEATHERING AND RQD 0 25 50 75 100	SAMPLE BLOWS RECOV.	SAMPLE TYPE	GRAPHIC LOG	SOIL OR ROCK DESCRIPTION FIELD AND LABORATORY TEST RESULTS, OR JOINTING, BEDDING AND FAULTING DESCRIPTIONS / SOIL STRATA DESCRIPTION; LITHOLOGY AND TEXTURE
205	40.0	89%	100%	NX 4		40.2 56° JOINT, FeO STAINING, SLIGHT EPIDOTE COATING 40.9 40° JOINT, IRON OXIDE STAINING 41.1 70° JOINT, SLIGHTLY WEATHERED, ADULARIA COATING 41.6 45° FOLIATION JOINT, IRON OXIDE STAINING 42.3 50° JOINT, SLIGHT IRON OXIDE STAINING
	43.3					44.6 50° JOINT, IRON OXIDE STAINING
200	45.0	100%	100%	NX 5		45.2 30° JOINT, SLIGHT ADULARIA COATING 45.8 45° JOINT, ADULARIA COATING 47.0 50° JOINT, EPIDOTE COATING
	47.8					
195	50.0	100%	100%	NX 6		49.9 45° JOINT, SLIGHT IRON OXIDE STAINING 51.4 60° JOINT, SLIGHT CHLORITE COATING
	52.6					52.8 60° JOINT ADULARIA COATING
190	55.0	96%	100%	NX 7		55.5 30° JOINT, EPIDOTE COATING
	57.7	80%	100%	NX 8		57.4 COMPOUND 30° JOINTS, ADULARIA COATING 57.9 20° JOINT, EPIDOTE COATING
185.9	59.1					END OF BORING AT 59.1

1. FIGURES IN BLOW OR RECOVERY COLUMN OPPOSITE SOIL SAMPLE DENOTE THE NUMBER OF BLOWS OF A 140 LB HAMMER FALLING 30" REQUIRED TO DRIVE A 2" OD SAMPLE SPOON 12" OR THE DISTANCE SHOWN. FIGURES SHOWN OPPOSITE ROCK CORES DENOTE THE PERCENT OF CORE RECOVERED.
2. ■ INDICATES LOCATION OF UNDISTURBED SAMPLE.
   ◤ INDICATES LOCATION OF SPLIT-SPOON SAMPLE.
   ☐ INDICATES LOCATION OF SAMPLING ATTEMPT
   ◸ WITH NO RECOVERY.
   SUBSCRIPT NEXT TO SYMBOL INDICATES SAMPLE NUMBER.
3. ▽ INDICATES LOCATION OF NATURAL GROUND WATER TABLE.
4. RQD - ROCK QUALITY DESIGNATION.
5. [] INDICATES DEPTH & LENGTH OF NX CORING RUN.
6. DATUM IS MEAN SEA LEVEL

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BORING LOG RATH-6

NORTH ANNA POWER STATION
UNITS 3 & 4
VIRGINIA ELECTRIC AND POWER COMPANY
STONE & WEBSTER ENGINEERING CORPORATION

VIRGINIA ELECTRIC AND POWER COMPANY

SH 1 OF 2

SITE NORTH ANNA POWER STATION UNITS 3 & 4 J.O. No. 12180/12181 BORING No. RATH-7
TYPE OF BORING NX CORE LOCATION N5013, E9266 GROUND ELEV. 245'
DATE DRILLED 4/12-13/76 DRILLED BY NICHOLSON/TRIPLETT LOGGED BY DPH
SUMMARY OF BORING

ELEV. FEET	DEPTH FEET	OVERALL WEATHERING AND RQD (0 25 50 75 100)	SAMPLE BLOWS RECOV.	SAMPLE TYPE	GRAPHIC LOG	SOIL OR ROCK DESCRIPTION: FIELD AND LABORATORY TEST RESULTS; OR JOINTING, BEDDING AND FAULTING DESCRIPTIONS	SOIL STRATA DESCRIPTION; LITHOLOGY AND TEXTURE
245	0.0					0-4.0' CONCRETE TEST PAD HOLE REAMED TO 8.3' WITH 6.5" ROTARY BIT BEFORE START OF CORING	
240	5.0						
	8.3						
235	10.0	91%	100%	NX 1		8.7' 45° JOINT, SLIGHT EPIDOTE COATING 9.4 7° JOINT, ADULARIA COATING 9.8 45° JOINT, 10.6 45° FOLIATION JOINT, ADULARIA COATING	GRAY GRANITE GNEISS, FRESH, CLOSE TO MODERATE CLOSE, JOINTING, MEDIUM GRAINED, VERY HARD, FOLIATION DIPS 45°
	12.8						
230	15.0	93%	100%	NX 2		15.1 30° JOINT, IRON OXIDE STAINING 15.8 60° FOLIATION JOINT	
	17.3					17.1 AND 17.3 60° JOINTS, SLIGHTLY WEATHERED ¼" SILTY FILLING	17.0 - 17.3' SLIGHTLY WEATHERED ZONE
225	20.0	95%	100%	NX 3		18.6, 19.2 AND 19.4 45° FOLIATION JOINTS 20.0 50° JOINT	
	21.3						
220	25.0	71%	100%	NX 4		22.9, 23.1, 23.2, 23.4, 23.8 24.1, 25.2 45° FOLIATION JOINTS EPIDOTE COATING	
	26.8					26.4 60° JOINT 26.9 70° JOINT, SLIGHTLY WEATHERED, SILT FILLING EPIDOTE COATING 27.9 60° JOINT, SLIGHT EPIDOTE COATING	
215	30.0	93%	100%	NX 5		29.1 60° JOINT, SLIGHT EPIDOTE COATING 29.3 45° FOLIATION JOINT, IRON OXIDE STAINING 30.0 45° FOLIATION JOINT, EPIDOTE COATING	
		89%	10?%	NX 6		32.3 80° JOINT, SLIGHTLY WEATHERED, IRON OXIDE STAINING	
	34.5					34.? AND 34.3 45° FOLIATION JOINTS, EPIDOTE COATING	
210	35.0	82%	100%	NX 7			
	39.0					37.6-39.7 VERTICAL JOINT, SLIGHTLY WEATHERED, ADULARIA COATING	37.3-37.8 SL. WEATHERED ZONE
205	40.0						

1. FIGURES IN BLOW OR RECOVERY COLUMN OPPOSITE SOIL SAMPLE DENOTE THE NUMBER OF BLOWS OF A 140 LB HAMMER FALLING 30" REQUIRED TO DRIVE A 2" OD SAMPLE SPOON 12" OR THE DISTANCE SHOWN. FIGURES SHOWN OPPOSITE ROCK CORES DENOTE THE PERCENT OF CORE RECOVERED.
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   ☐ INDICATES LOCATION OF SAMPLING ATTEMPT
   ◸ WITH NO RECOVERY.
   SUBSCRIPT NEXT TO SYMBOL INDICATES SAMPLE NUMBER.
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6. DATUM IS MEAN SEA LEVEL

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BORING LOG RATH-7

NORTH ANNA POWER STATION
UNITS 3 & 4
VIRGINIA ELECTRIC AND POWER COMPANY

STONE & WEBSTER ENGINEERING CORPORATION

VIRGINIA ELECTRIC AND POWER COMPANY

SH 2 OF 2

SITE NORTH ANNA POWER STATION . UNITS 3 & 4 J.O. No. 12180/12181 BORING No. RATH-7

TYPE OF BORING NX CORE LOCATION N4013, E9266 GROUND ELEV. 245'

DATE DRILLED 4/12-13/76 DRILLED BY NICHOLSON/TRIPLETT LOGGED BY DPH

SUMMARY OF BORING

ELEV. FEET	DEPTH FEET	OVERALL WEATHERING AND RQD (0 25 50 75 100)	SAMPLE BLOWS RECOV.	SAMPLE TYPE	GRAPHIC LOG	SOIL OR ROCK DESCRIPTION (FIELD AND LABORATORY TEST RESULTS, OR JOINTING, BEDDING AND FAULTING DESCRIPTIONS; SOIL STRATA DESCRIPTION, LITHOLOGY AND TEXTURE)
205	40.0	78%	93%	NX 8		40.6 50° JOINT, ADULARIA COATING 41.0 70° JOINT, IRON OXIDE STAINING 41.5 45° FOLIATION JOINT, IRON OXIDE STAINING 42.0 60° JOINT, SLIGHT EPIDOTE COATING
200	43.5 45.0	78%	100%	NX 9		43.9, 44.0 50° JOINTS, SLIGHT IRON OXIDE STAINING, ADULARIA COATINGS 44.3, 44.6 50° JOINTS, ADULARIC FILLING 47.6 60° JOINT, ADULARIA AND EPIDOTE FILLING
195	48.0 50.0	93%	100%	NX 10		49.5 60° JOINT, ADULARIA AND EPIDOTE COATINGS 51.1 60° JOINT, ADULARIA AND EPIDOTE COATINGS
190	52.5 55.0 58.0	100%	100%	NX 11		53.1 60° JOINT, ADULARIA AND EPIDOTE COATINGS 54.0 15° JOINT, SLIGHT IRON OXIDE STAINING 54.3-55.[illegible] VERTICAL JOINT, ADULARIA COATING 56.6 45° JOINT

END OF BORING AT 58.0'

1. FIGURES IN BLOW OR RECOVERY COLUMN OPPOSITE SOIL SAMPLE DENOTE THE NUMBER OF BLOWS OF A 140 LB HAMMER FALLING 30" REQUIRED TO DRIVE A 2" OD SAMPLE SPOON 12" OR THE DISTANCE SHOWN. FIGURES SHOWN OPPOSITE ROCK CORES DENOTE THE PERCENT OF CORE RECOVERED.
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BORING LOG RATH-7

NORTH ANNA POWER STATION
UNITS 3 & 4
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APPENDIX D

EXAMINATION OF THIN SECTIONS FROM GROUT TEST HOLES

APPENDIX D

EXAMINATION OF THIN SECTIONS FROM GROUT TEST HOLES

Samples

Four thin sections were prepared from cores of the grout-rock interface in two grout test holes. Specimens 1 and 2 were taken from Grout Test Hole No. 2 which was filled with MB 814 grout, and specimens 3 and 4 were taken from Grout Test Hole No. 3 which was filled with a Type II cement grout.

Method of Observation

The thin sections were examined under an Olympia polarizing petrographic microscope. Mineral identifications were made using standard petrographic characteristics of the minerals. In addition, a slide projector was used to project the full slide on a viewing screen so that overall patterns on the thin sections could be seen.

Observations

1. Specimen 1 - MB 814 grout - Granite gneiss rock

Rock Composition:	Quartz, plagioclase (two types), microcline, biotite, minor chlorite, and opaques

Grout-Rock Contact: A nearly continuous 0.05 mm wide crack is present along the grout-rock interface, and at several places along it the crack splays off into the grout (see Figure 1). The cracks at the grout-rock interface and in the grout are filled with a late phase birefringent mineral showing a first order gray interference color and undulatory extinction. These properties would imply some type of silicate mineral.

Adjacent to the grout-rock interface, a 0.15 mm thick band is present in the grout. This band contains none of the coarse particles (filler) that are present in the grout mass. Under crossed nicols, the band appeared browner in color than the majority of the grout. No observable alteration or difference was found in the rock near the contact.

Adjacent to this 0.15 mm thick band is a 7.0 mm thick band which is discolored and coarser grained (see Figure 1). Under crossed polaroid filters, the unaffected grout is gray, and the discolored band becomes browner toward each filled crack.

2. Specimen 2 - MB 814 grout - Granite gneiss rock

   Rock Composition: Same as Specimen 1

   Grout-Rock Contact: Similar to Specimen 1. A 0.05 mm crack is present along the interface. The crack filling is not as complete as in Specimen 1. Several cracks which are not filled exist in the grout but these could have occurred during preparation of the sample. The 0.15 mm thick discolored band and the wider (7 mm) band were again noted.

3. Specimen 3 - Neat cement (Type II) grout - Granite gneiss rock

   Rock composition: Same as Specimen 1

   Grout-Rock Contact: Along most of the contact, no cracks are present and the bond is tight. Where cracks are found in the grout, they are not filled. A discolored band of grout is present along the contact, but it is much thinner (0.03 mm) than that seen on specimens 1 and 2.

4. Specimen 4 - Neat cement (Type II) grout-granite gneiss

   Rock Composition: Same as Specimen 1

   Grout-Rock Contact: Similar to Specimen 3

Conclusions

The samples of the MB 814 grout-rock interface show the development of fine cracks along the interface. The cracking probably occurred while the grout cured and may represent shrinkage cracking of the grout. The cracks have since been healed by a late phase silicate mineral. It is postulated that the formation of the mineral filling was caused by a chemical reaction within the grout itself or by a reaction between the groundwater and the grout. The granite gneiss rock shows no signs of chemical alteration and, therefore, does not contribute to the chemical reaction. This conclusion is supported by the fact that the cracks extending into the grout mass are healed by the same mineral filling. Also, the 7 mm discolored band parallels both the cracks at the interface and the cracks extending into the grout. This wide discolored band adjacent to cracks may have been caused by the chemical reaction which resulted in the formation of the mineral filling. The 0.15 mm band of fine grained material is present in the grout at the interface. This thin band may have been caused by the migration of bleed water toward the rock after grout placement.

The samples of the neat cement grout-rock interface generally showed little cracking at the interface. No evidence of a chemical reaction was present other than a very thin discolored band of grout. No mineral fillings are present in cracks.

Based upon the observed differences in the thin section the following conclusions can be made:

1. MB 814 cable grout apparently cracked during curing.

2. The cracks in the MB 814 grout are healed by a mineral resulting from a chemical reaction. This chemical reaction at the rock-grout interface probably has a tendency to weaken the bond. (Note: It has been established by field and laboratory tests that the MB 814 grout does not bond as well to the rock as neat cement grout.)

3. Neat cement grout shows none of the above characteristics.

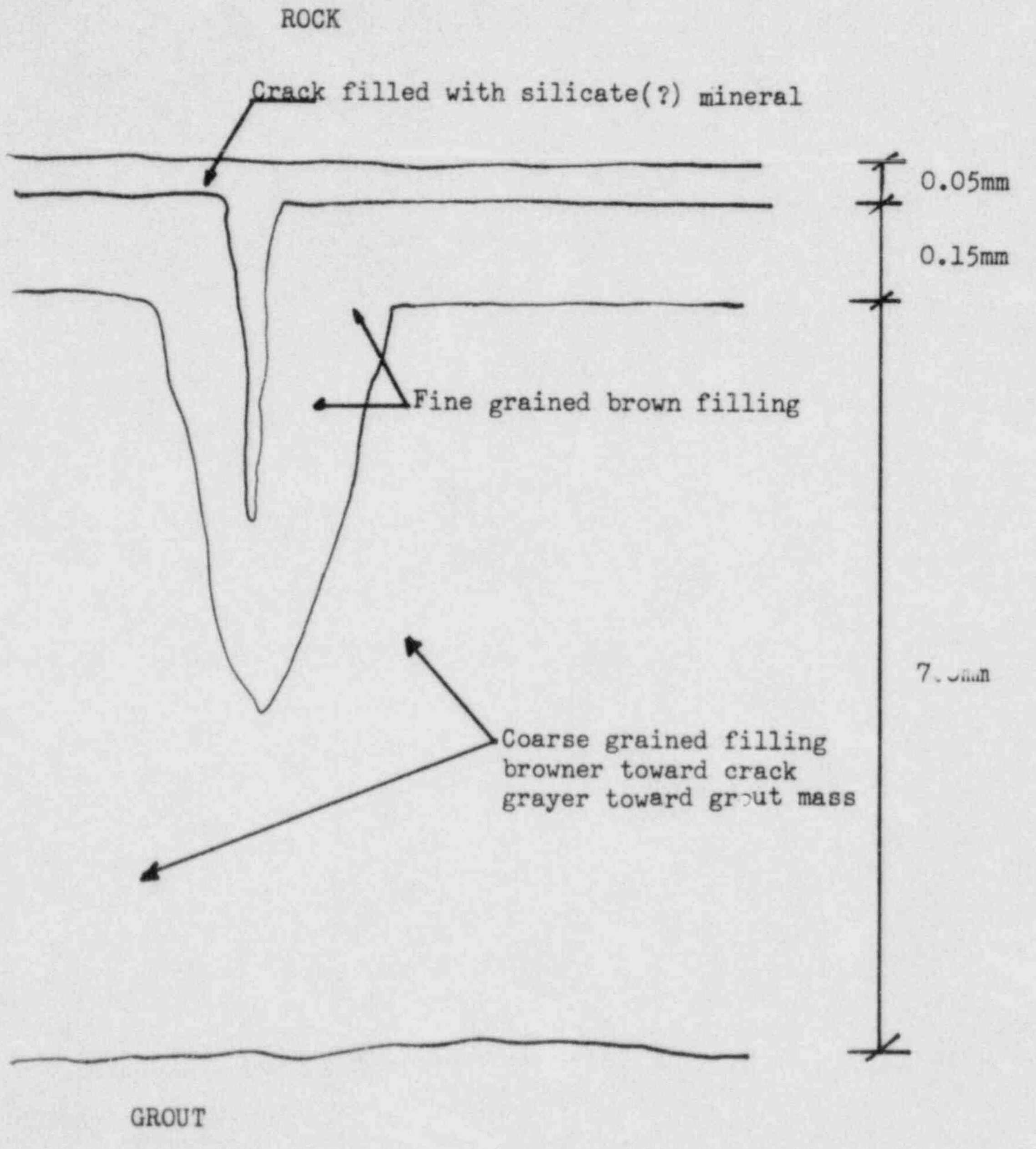
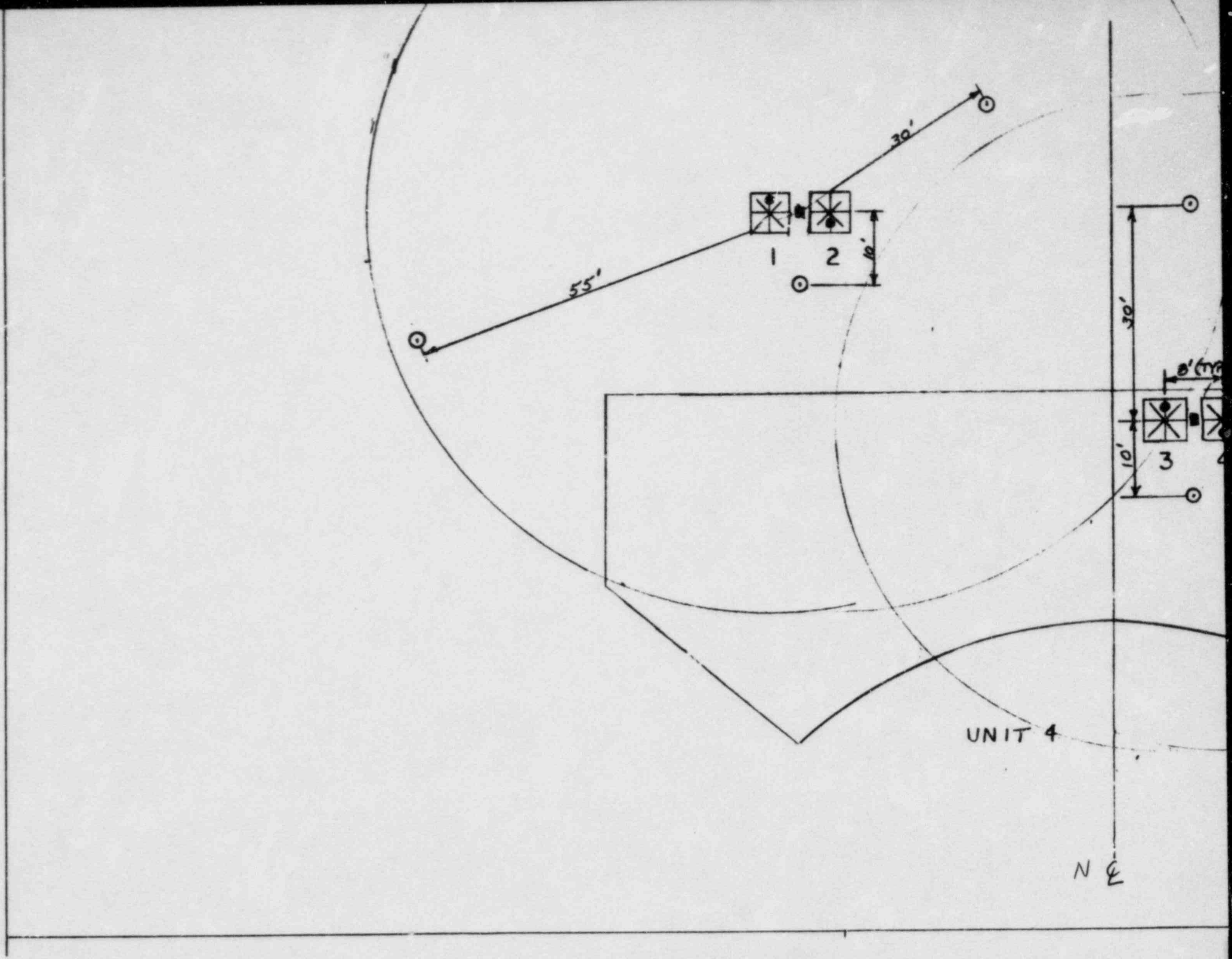


Figure 1 Typical MB 814 Grout/Rock Interface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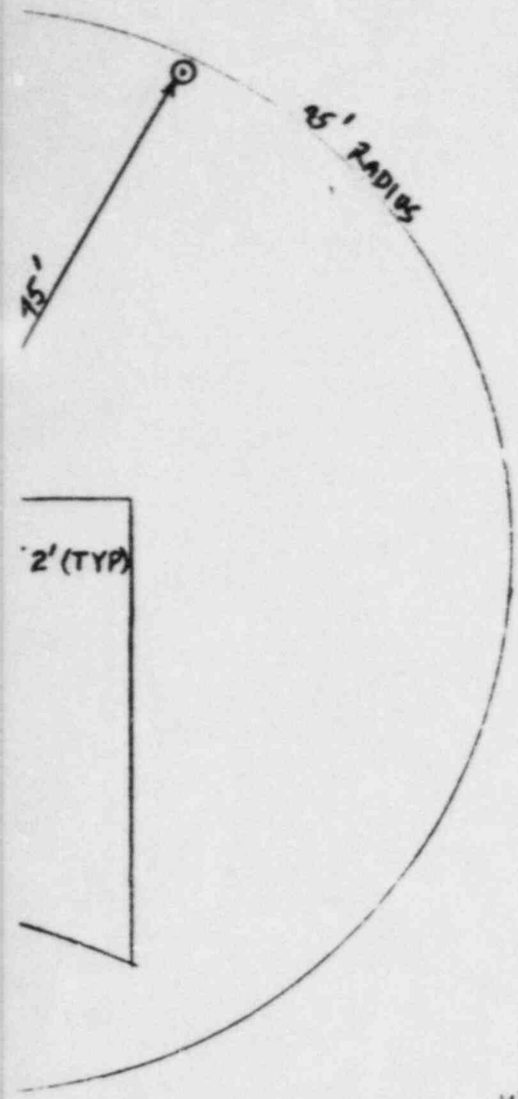


-12.5 extensometer
-25

NOTES:

1. ANCHORS 1 AND 2 WILL BE 55' LONG
   ANCHORS 3 AND 4 WILL BE 45' LONG
2. SCALE: 1/16" = 1'
3. X ANCHOR LOCATION
4. ● DOUBLE-POSITION EXTENSOMETER LOCATION
   1&2 WILL BE ANCHORED @ 30' & 40' DEPTHS
   3&4 WILL BE ANCHORED @ 28' & 36' DEPTHS
5. ■ DOUBLE POSITION EXTENSOMETERS @ 12.5' & 25' DEPTH
6. ⊙ REBAR GROUTED AT LEAST 3 FT. INTO ROCK AND EXTENDING NO LESS THAN 2 FT. ABOVE GRADE
7. THE ELEVATION OF THE BASE PLATE ADJACENT TO THE ANCHOR, THE TOP OF THE CONCRETE TEST PADS, AND THE TOP OF THE RE-BARS SHALL BE SURVEYED IMMEDIATELY AFTER EACH INCREMENT AND/OR DECREMENT OF LOAD.

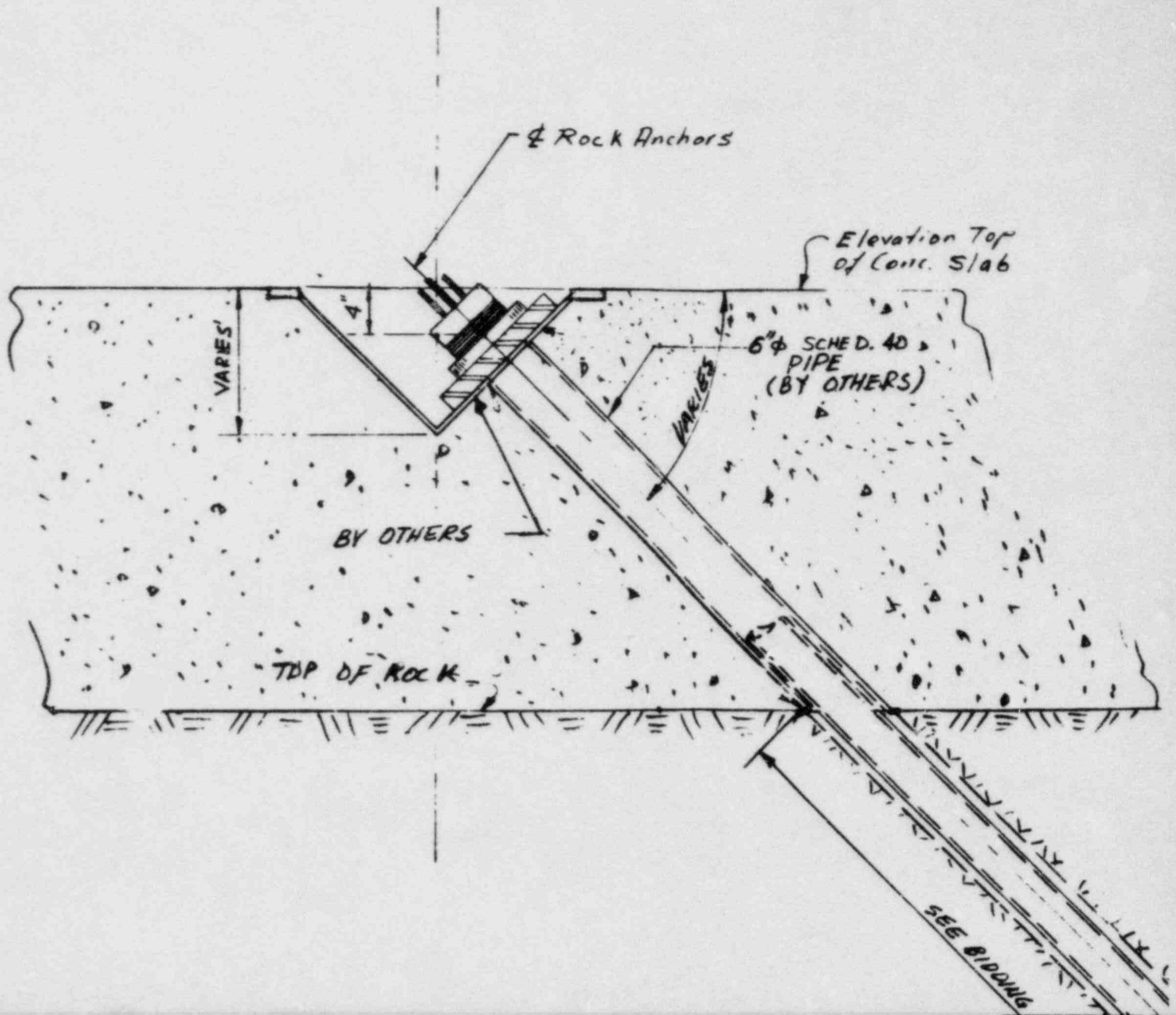
7		6		5		4		3		2		1	

INSTRUMENTATION FOR ROCK
ANCHOR TEST PROGRAM

STONE & WEBSTER ENGINEERING CORPORATION

12180 GSK-19

▲ 5215 4


℄ Rock Anchors
Elevation Top of Conc. Slab
4"
VARIES'
6"φ SCHED. 40 PIPE (BY OTHERS)
VARIES
BY OTHERS
TOP OF ROCK
SEE BIDDING

5010 28

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

Page No.

Preliminary

Item

12181

Client VEPCO Unit#3&4 Location No Anna Est No. J.O. No 12180

Subject or Apparatus Rock Anchors Safeguards, Date 5/22/74 By E Medina

Quench Spray & Main Steam Valve Checked 5/23/74 By P. Gupta

Based on Buildings & Intake Structure Revised By