1/26/81

ADDITIONAL INFORMATION ON GINNA
ROCK ANCHOR DESIGN

PREPARED FOR

ROCHESTER GAS & ELECTRIC CORP.

BY

GILBERT ASSOCIATES, INC.

PREPARED BY: R. E. Pages
R. E. Pages

REVIEWED BY: R. E. Pages for
J. F. Fulton

APPROVED BY: F. L. Moreadith
F. L. Moreadith

8012180394

11/26/80

At the meeting of October 21, 1980 (RG&E/GAI and USNRC), Dr. John Chen expressed concern about the condition of the rock anchors for the containment at Ginna. He further stated that his understanding of the FSAR description of the rock anchor design led him to conclude that the design was deficient. As a result of that meeting, RG&E/GAI agreed to provide additional information to support their position that the rock anchor design is acceptable and does not constitute a safety hazard.

A review of the design, the FSAR, and current literature indicates that the design assumptions used were acceptable for rock anchor design at that time, and that these assumptions are still considered to be acceptable. The effects of overlapping of the "reaction cones" of the anchors were accounted for in the original design calculations (see Tab 2) and in the recent calculations (see Tab 3). A scale prototype test was conducted during the original design and four anchors were checked for stress losses during construction. The loads applied to the anchors during installation (i.e., 0.8 GUTS) exceed current anchor loads as well as any load they will see in the future.

The assumed condition of the anchors for the controlling load combination (i.e., 1.5 x accident pressure) neglects the overburden, the weight of containment internal structures and equipment as well as the tensile capacity of the rock. The safety margins are adequate even without the use of these additional factors, all of which would increase the margin.

Rock creep data was not obtained during the original design work. However, at a later date, additional rock cores and tests were made in an adjacent area on the site. The results of these tests are provided in Tab 1 (Lucius Pitkin Report dated September 6, 1973) as well as an estimate of the creep over 40 years which was .21 x 10^{-6} inches per inch at a compressive stress of 10000 psi. The maximum stress in the rock was estimated at approximately 400 psi. The relaxation of the tendon wires themselves is estimated at 690 x 10^{-6} inches per inch. These calculations indicate that the rock creep is insignificant relative to other potential sources of force loss. ?

Except during containment pressurization, the resultant uplift force on the rock wedge surrounding the rock anchors is zero. As Figures 1 and 2 illustrate, the upward force which the rock anchor tendons exert on the rock (at the grout-rock interface) is always in equilibrium with a downward reaction force on the rock at the footing-rock interface. This equilibrium condition existed at the various stages of rock anchor stressing and wall tendon stressing, and it is not changed by the lift off tests of the past surveillances, nor by the recently completed retensioning program.

The tension force in the rock anchor, and hence the shear forces at the tendon-grout interface and at the grout-rock interface, increases when there is lift off of the upper rock anchor head from its shims during the application of a force to the wall tendon. The amount of force increase in the rock anchor is the difference in the final force applied to the wall tendon and the force required to lift off the rock anchor head. If a conservatively low anchor head lift off value of 0.5 GUTS is assumed, then the recently completed retensioning operations would have increased the force in the rock anchors by approximately 0.24 GUTS (0.735 GUTS minus 0.5 GUTS), assuming zero friction loss in the wall. Since the rock anchors were originally stressed to 0.8 GUTS and locked off at 0.7 GUTS, the retensioning program increased the rock anchor force at most by 30% of the largest force which has been successfully applied to the rock anchors, 0.8 GUTS. Considering these conditions, there is no basis for postulating an "anchor failure", particularly in light of the close agreement between predicted and measured tendon elongations for all 133 tendons.

In addition, the phenomena which RG&E has been concerned with at the site, that is, greater than predicted tendon force losses with time, would not be explained by a "failure" of the rock anchors. Anchorage "failure" is a phenomena which would have occurred very rapidly and while the highest loads were being applied, i.e., 0.8 GUTS. In all cases, even the 6% overload applied during each retension, the load we are applying to the anchor is below the initial installation and test load of 0.8 GUTS.

Tabs 1 through 8 of the Attachment provide additional information relative to the rock anchor design and construction.

A second concern expressed during the meeting was related to the connecting sleeve between the upper and lower tendons. Tab 9 of the Attachment includes three documents: (1) a telex relating the results of lab tests on the connector; (2) a telex relating the assembly procedure for the coupling (note item (5)); and (3) the design criteria for the coupling. Although there is no specific record that the anchors' treads were fully engaged, it would seem that since the procedure required full engagement, lack of this would have been reported.

To summarize, the review has not uncovered either faulty assumptions or calculation errors of an extent that would be of concern.

Gilbert Associates, Inc.
Reading, Pennsylvania

ANALYSIS/CALCULATION

SUBJECT		CISID		PAGE ___ OF ___ PAGES
REV. 0	1	2	3	
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ROCK EQUILIBRIUM AT STRESSING ROCK ANCHOR TO $0.80 F_{pu}$ and LOCKING OFF AT $0.70 F_{pu}$

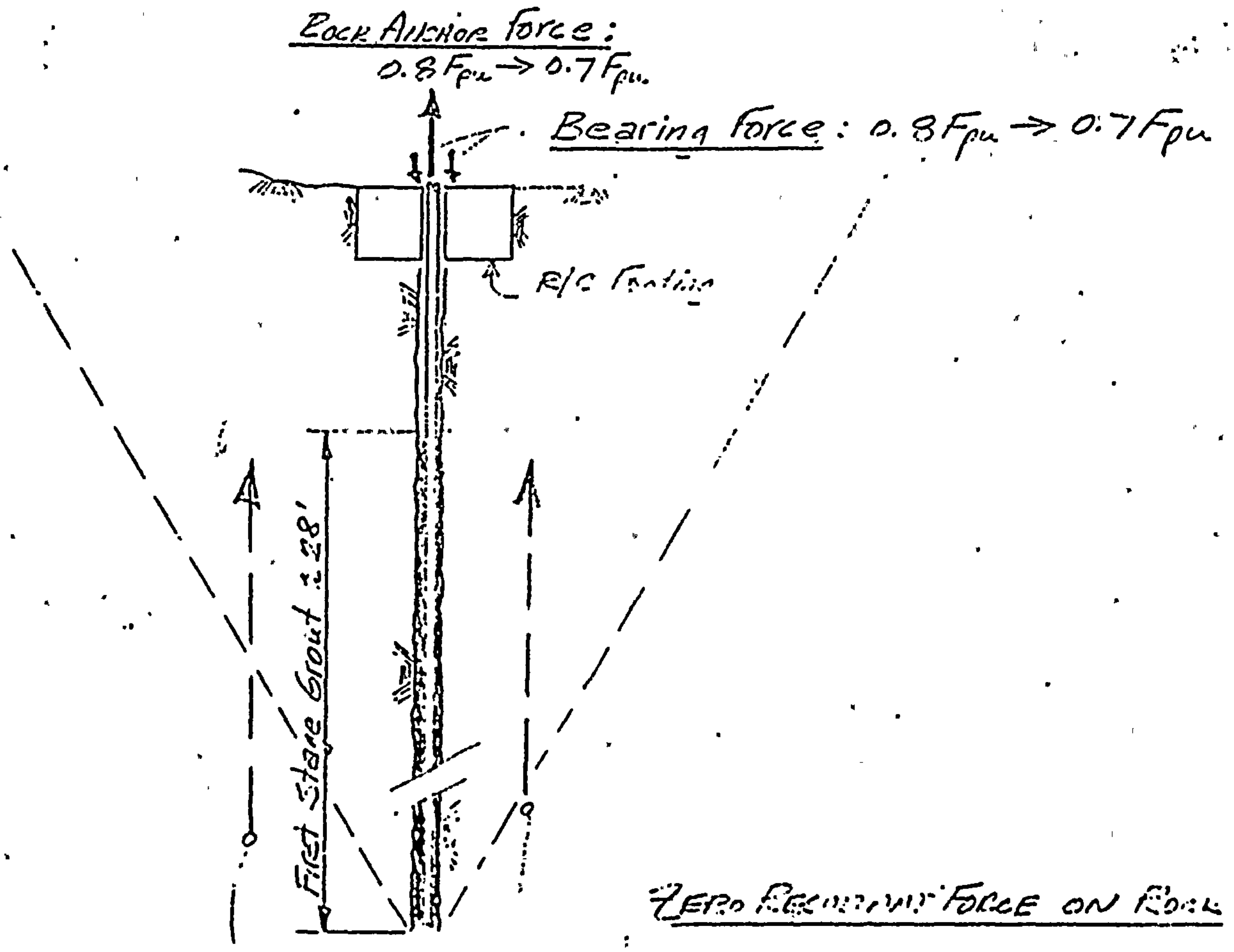

FIGURE 1.

POOR ORIGINAL

Gilbert Associates, Inc. Reading, Pennsylvania	SUBJECT			CISID		PAGE
ANALYSIS/CALCULATION	REV. 0	1	2	3		OF
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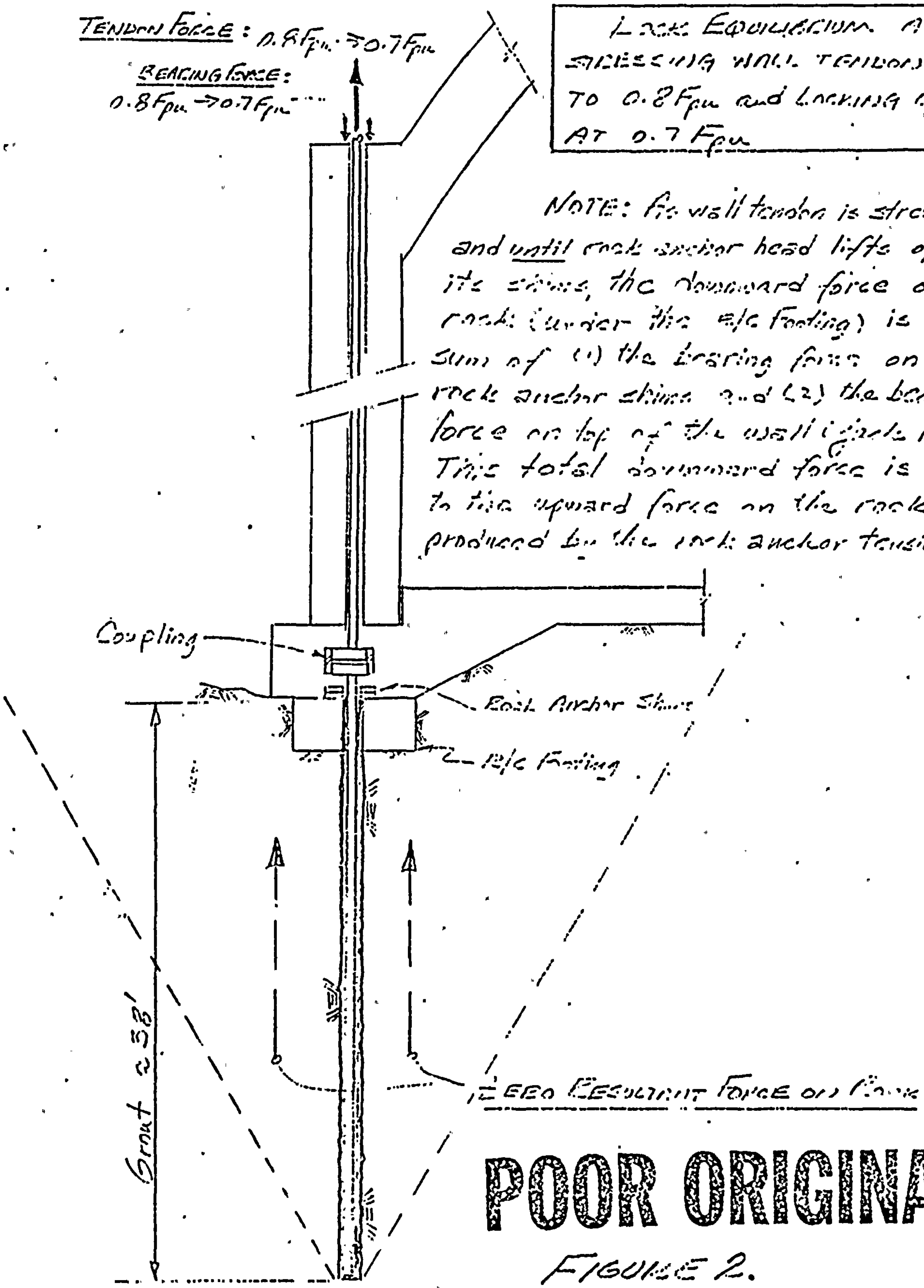
LOCK EQUILIBRIUM AT STRESSING WALL TENDONS TO $0.8F_{pu}$ and LOCKING OFF AT $0.7F_{pu}$



NOTE: As wall tendon is stressed and until rock anchor head lifts off its shims, the downward force on the rock (under the R/C Footing) is the sum of (1) the bearing force on the rock anchor shims and (2) the bearing force on top of the wall (jack reaction). This total downward force is equal to the upward force on the rock produced by the rock anchor tension.

POOR ORIGINAL

FIGURE 2.

ATTACHMENT

TABLE OF CONTENTS

2.8 GEOLOGY

2.8.1 SUMMARY

A geological program involving a regional geological survey, borings, and other tests at the site was conducted to provide information needed to assess foundation conditions, seismic activity and ground water conditions. The details of these investigations which were performed by Dames & Moore are reported in detail in Volume 1, Appendix D of the PSAR and in Appendix 2 B of this report.

These results and subsequent information discussed below indicate that the rock and compact granular soil on the site provide a suitable foundation for plant structures with allowable bearing pressures in the range of 3 to 6 tons per square foot for spread or mat foundations on the compact granular soils and of 30 to 40 tons per square foot on bedrock.

2.8.2 REGIONAL GEOLOGY

The site lies within the Erie-Ontario lowlands physiographic province which is characterized by an erosional topography of low relief modified by glacial features. The land rises gradually to the south where it meets the Appalachian Uplands at the Portage Escarpment.

Geologic formations in the region include Lower and Middle Paleozoic sédiments overlying the pre-Cambrian basement rocks. The pre-Cambrian surface dips to the south at approximately 60 feet per mile with local variations.

The youngest formation occurring at the site is the Queenston formation of Upper Ordovician Age. The Queenston is roughly 1,000 feet thick in this area and overlies approximately 80 feet of Oswego sandstone, approximately 600 feet of Lorraine shales and probably less than 30 feet of Potsdam sandstone.. The pre-Cambrian surface is roughly 2,600 to 2,700 feet deep at the site.

2.8.3 LOCAL GEOLOGY

The major nuclear station structures are supported in the Queenston Formation or atop a thin layer of natural or compacted granular soils immediately above the bedrock. The Queenston Formation, which is generally found at depths of 30 to 40 feet, is composed of alternating strata of thinly to thickly bedded, dense, fine grained sandstone, silty sandstone, and sandy siltstone, with occasional thin beds of fissile shale. Bedding is essentially horizontal with occasional cross-bedding and shaly partings. The color is predominately red, but random green blotches and layers occur throughout the depths explored. Occasional continuous vertical joints were noted in the borings and during our site inspections.

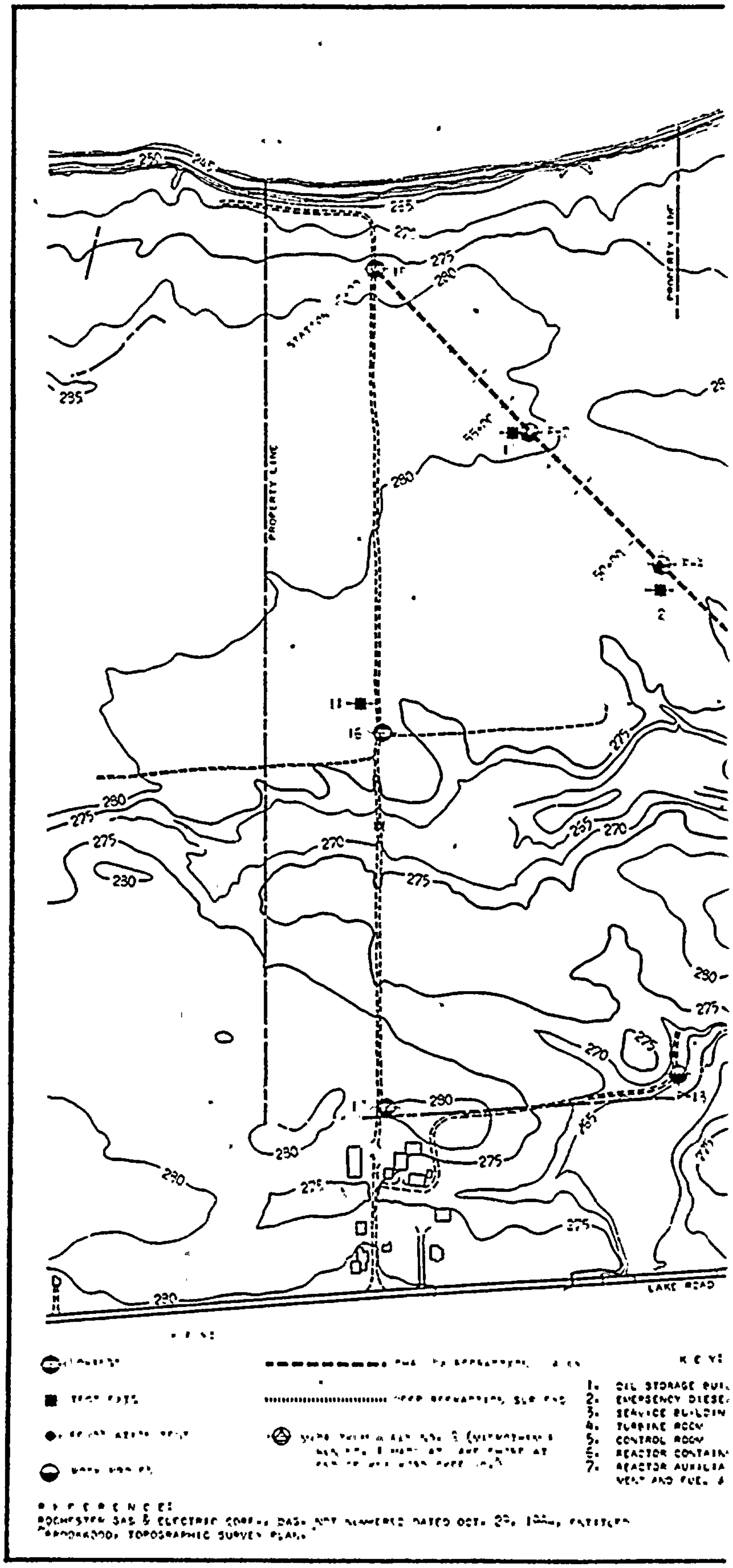
Subsequent to the initial environmental studies, seven additional borings were drilled to depths between 35 and 90 feet in the reactor area for a supplementary foundation study. The location of these borings are shown on Figure 2.8-1. The soil and rock encountered in the seven borings were similar in all respects to the on-site materials described in the PSAR.

Nine borings were drilled for the proposed intake and discharge tunnels. As shown on Figure 2.8-1, these borings extended from the shore to a distance of about 3,000 feet into Lake Ontario.

Prior to Construction of the plant foundations, the soil overburden (30 to 40 feet of glacial drift) was removed. The exposed rock surface was observed to be similar to that examined in nearby outcrops. Bedding was horizontal and occasional crossbedding and shaly partings were evident. A pattern of vertical joints of limited vertical extent was evident in the out-croping rock, particularly along the lake shore side of the excavation. The observed joints continued to depths of from 20 to 30 feet from the top of the rock, but no evidence of movement along the joints was found. The major joint systems were found to be in accordance with those trends reported in the PSAR. Some minor exfoliation noted in the bottom of the excavation is believed to have been caused primarily by the heavy equipment traffic on the excavation floor and the drying effects of exposure to air.

The cores extracted in the nine borings drilled for the intake structure investigation were compared with the cores of the previous borings drilled at the site. As expected, the rock encountered below the lake was consistent with the rock encountered in on-shore borings.

The on-shore shaft and the tunnels were inspected during construction as well as after completion of the tunneling. Examination of the exposed rock revealed conditions consistent with those encountered during the previous studies. No zones of defective rock were found and no weathered rock was evident in the tunnels. The rock in both tunnels is sound. Water flow was practically non-existent, being essentially limited to scattered areas of minor moisture infiltration. The actual conditions found in the tunnel excavations are in agreement with those encountered in all previous borings drilled during the initial subsurface investigation and the other supplementary investigations.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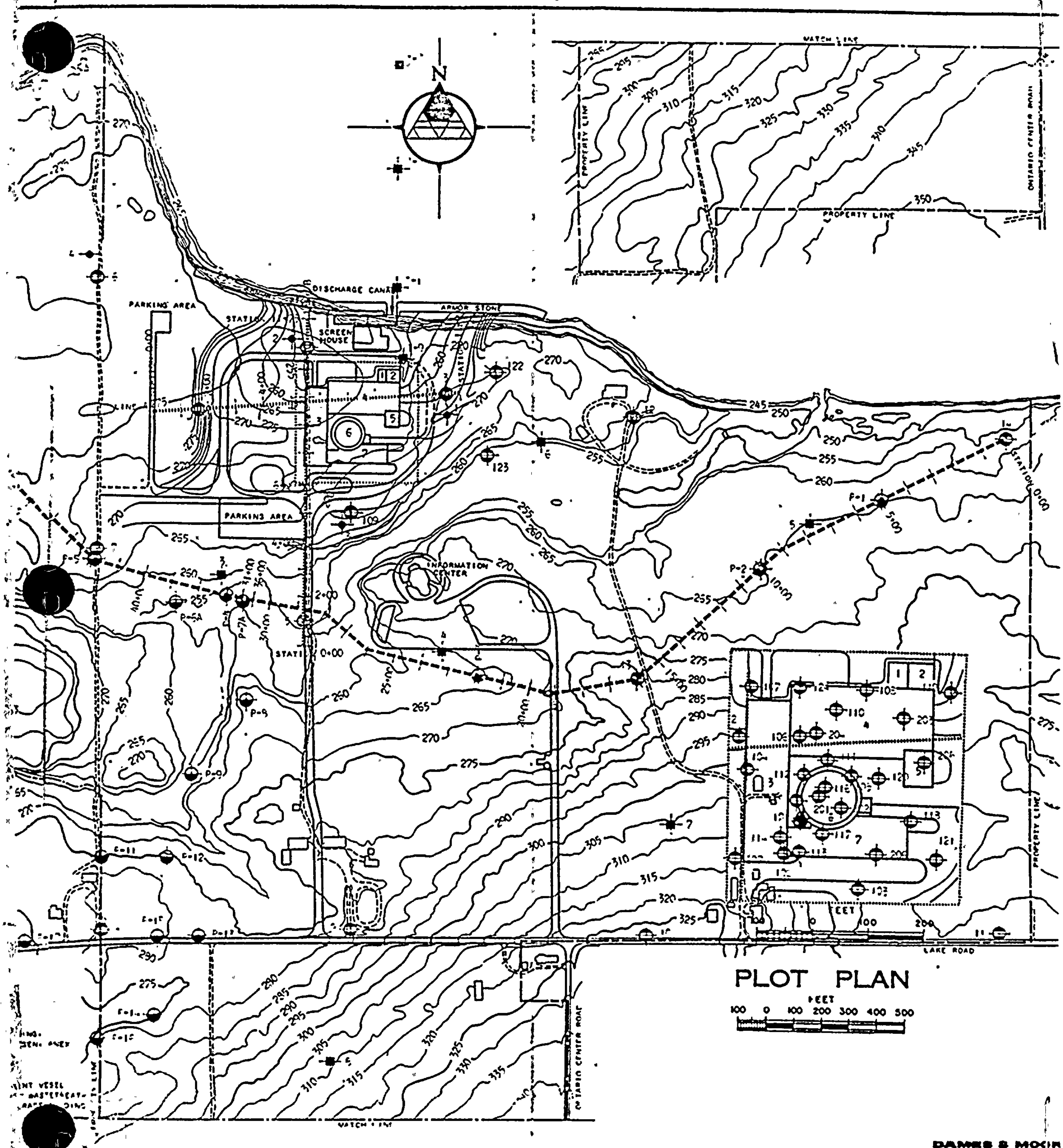
KEY
1. OIL STORAGE BUI
2. EMERGENCY DIESE
3. SERVICE BUILDIN
4. TURBINE ROOM
5. CONTROL ROOM
6. REACTOR CONTAIN
7. REACTOR AUXILIA
VENT AND FUEL



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2-8-1

FIG. 2.8

EARTH SCIENCES

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TEHRAN, IRAN

100 CHURCH STREET · NEW YORK 7, NEW YORK · CORTLANDT 7-1810

PARTNERS: GARDNER M. REYNOLDS · ROBERT M. PERRY · JOSEPH A. FISCHER

ASSOCIATE: FRANCIS E. RANFT

June 2, 1966

Gilbert Associates, Incorporated
Engineers and Consultants
525 Lancaster Avenue
Reading, Pennsylvania 19603

Attention: Mr. Hans Lorenz

Gentlemen:

We submit herewith ten copies of our "Report, Supplementary Foundation Studies, Proposed Brookwood Nuclear Power Plant, Ontario, New York, Rochester Gas and Electric Corporation."

The scope of our studies was planned in cooperation with Mr. D. K. Croneberger of Gilbert Associates, Incorporated. Our preliminary conclusions were transmitted verbally to Messrs. Croneberger and H. Lorenz during the course of our studies.

Yours very truly,

DAMES & MOORE

Robert M. Perry

Robert M. Perry, P.E.

RMP:ts

REPORT

SUPPLEMENTARY FOUNDATION STUDIES

PROPOSED BROOKWOOD NUCLEAR POWER PLANT

ONTARIO, NEW YORK

ROCHESTER GAS AND ELECTRIC CORPORATION

INTRODUCTION

GENERAL

This report presents the results of our supplementary foundation studies for the proposed Brookwood Nuclear Power Plant presently under construction near Ontario, New York, for the Rochester Gas and Electric Corporation. Detailed information relative to environmental conditions, site and subsurface features, and general foundation recommendations are presented in our report* dated June 14, 1965.

PURPOSE

The purpose of our supplementary studies was to:

1) recommend specific bearing pressures for use in the design of foundations supported by the natural compact granular soils, compacted granular fill and sound bedrock;

2) present more detailed information on the depths at which the compact natural granular soils and the bedrock are encountered;

3) further explore the condition of the bedrock in the reactor area; and

4) evaluate the effects of the dynamic load imposed by the turbine-generator on the soil-foundation system.

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* "Report, Site Evaluation Study, Proposed Nuclear Power Plant, Ontario, New York, Rochester Gas and Electric Corporation"

SCOPE OF WORK

The field phase of our supplementary studies consisted of drilling seven test borings. Two of the borings were drilled in the reactor area and extended 50 feet into the bedrock. The remaining five borings were terminated when bedrock was encountered. Undisturbed soil samples, suitable for laboratory testing, were extracted from each test boring. Rock cores were recovered from the two borings in the reactor area.

The locations of the borings drilled for these studies are shown in relation to the proposed construction and previously drilled borings on the Plot Plan, Plate 1. The field explorations were performed under the technical direction of a Dames & Moore Engineering Geologist.

The results of the field explorations and laboratory tests, which provide the basis for our engineering analyses and recommendations, are presented in the Appendix to this report.

SITE CONDITIONS

The plant will be located in a relatively level meadow area with surface elevations* on the order of +275 feet. Grading operations were underway during our field explorations.

The subsurface conditions encountered in the borings drilled during this investigation are similar to those previously encountered in the plant area. In general, the plant area is underlain by four basically different types of material. These are, in order of increasing depth:

1) firm brown surficial silty and clayey soils;
2) soft gray silty clay;
3) compact sandy and gravelly soils; and
4) bedrock.

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* All elevations presented in this report refer to United States Coast and Geodetic Survey Datum.

Detailed descriptions of the materials encountered in the plant area are shown on the boring logs presented in the Appendix. In general, the compact granular soils were encountered at depths ranging from about five feet to 35 feet below the original ground surface. Bedrock generally was observed at depths ranging from about 34 feet to 40 feet below the surface. The southwest corner of the proposed plant revealed bedrock at somewhat shallower depths.

Contours of the surface of the compact granular soils and the underlying bedrock are presented on the Plot Plan. This contour map was prepared by interpolation between borings. Consequently, local variations may occur between the boring locations which are not indicated by the contours.

DISCUSSION AND RECOMMENDATIONS

GENERAL

It is understood that foundations for the major plant facilities will be installed at depths of 25 or more feet below the original ground surface. In our prior report, we recommended that spread or mat foundations be installed on the natural compact granular soil, compacted granular backfill or sound bedrock.

Spread and mat foundation installation and design criteria are presented in subsequent sections of this report. The results of our analyses evaluating the effects of the turbine-generator on the soil-foundation system are presented in the final section of this report.

FOUNDATION INSTALLATION PROCEDURES

Natural Soils: Spread or mat foundations can be installed directly on the compact granular soils at elevations below those indicated by the contours on the Plot Plan. We recommend that the sand and gravel at foundation depth be proof rolled with heavy pneumatic-tired equipment. The proof rolling will recompact soils which are disturbed during excavation operations. Any local pockets of loose or soft material requiring additional excavation also will be revealed by the proof rolling operations. Soils removed below proposed foundation grade should be replaced with compacted structural fill or lean concrete.

Compacted Backfill: Foundations which are to be installed above the elevation of the surface of the natural granular soils should be supported by compacted granular backfill placed after the clayey soils are removed. Prior to placing the backfill, the exposed underlying natural granular soil should be proof rolled. The structural fill then should be placed in layers approximately eight inches in thickness. Each layer should be compacted to a density of at least 95 percent of the maximum density obtainable by the Modified AASHO* Method of Compaction, Test Designation T180-57. We suggest that large vibratory or heavy pneumatic-tired equipment be used to compact the granular backfill soils.

We believe that most of the natural granular soils excavated in the plant area below the elevations indicated on Plate 1 can be reused as backfill. The upper silty and clayey soils should not be used as structural fill.

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* American Association of State Highway Officials

It will be necessary to dewater all deep excavations. Information regarding ground water levels and soil permeability was presented in our previous report. We recommend that adequate dewatering measures be taken prior to final excavation and that the dewatering be continuously maintained during:

1) final excavation;
2) proof rolling operations;
3) placement of structural backfill;
4) foundation installation; and
5) general backfilling operations.

We recommend that an experienced Soils Engineer be present during site preparation in order to inspect the excavation and proof rolling operations and to technically supervise the placement of structural backfill.

FOUNDATION DESIGN CRITERIA

Soil: Based upon the results of our field explorations and laboratory tests, we recommend that spread and mat foundations be designed utilizing the net bearing pressures presented on Plate 2, Foundation Design Data. The bearing pressures presented on Plate 2 are applicable for the compact natural granular soil and structural granular fill compacted in accordance with our aforementioned recommendations. The recommended bearing pressures apply to the total of all design loads, dead and live. The term "net bearing pressures" refers to the foundation pressure that can be imposed in excess of the lowest adjacent overburden pressure. The recommended bearing pressures apply to foundations at least ten feet in width.

We recommend that the maximum net bearing pressures imposed on the natural compact soils and the compacted structural fill should be limited to 10,000 and 8,000 pounds per square foot, respectively. Although, from a stability standpoint, greater bearing pressures could be used in the design of large spread and mat foundations, we recommend that these limiting values be maintained in order to restrict foundation movements to small elastic deformations.

Rock: We recommend that foundations installed on the underlying sound rock be designed utilizing a bearing pressure not in excess of 35 tons per square foot. This pressure applies to the total of all design loads, dead and live. It is possible that weathered rock may be encountered at the soil-rock interface. Our field explorations indicate that the weathered zone is relatively thin, generally less than one to two feet in thickness.

We understand that the bedrock in the reactor area will be required to provide resistance to lateral forces. We believe that a lateral resistance of 25,000 pounds per square foot of vertical contact area can be relied upon in the sound rock. This lateral resistance applies only to foundations poured in "neat" excavations directly against the exposed rock faces. The 25,000 pounds per square foot value does not take into account the additional resistance which would be provided by any adjacent overburden above the surface of the bedrock.

The exposed bedrock should be inspected by a qualified Engineering Geologist in order to examine the condition of the foundation material and to check for any unusual or unanticipated joint patterns.

TURBINE-GENERATOR FOUNDATION

The turbine-generator will be supported on a mat foundation approximately 40 feet by 150 feet in plan dimensions. The base of the mat will be installed at approximately Elevation +243 feet, some four to seven feet above the rock surface. The center-line of the turbine-generator will be approximately 50 feet above the base of the mat foundation. The dead weight of the equipment and the foundation will impose a pressure of about 4,000 pounds per square foot on the foundation soils.

We understand that the turbine-generator will operate at approximately 1,800 revolutions per minute. During start-up and operation, an unbalanced moment on the order of 2,000,000 foot-pounds will be transmitted to the soils at the base of the mat. This moment is a steady-state condition and does not vary with the operating speed. Unbalanced dynamic forces will be negligible. A torque approximately ten times the operating torque will result from a short-circuit load. This short-circuit torque will be balanced within the equipment foundation and will not be transmitted to the foundation soil.

Our analyses indicate that the deflection resulting from the unbalanced moment will be on the order of 0.004 inches at the edge of the unit. We believe that there will be no influence from any small unbalance in the equipment since the operating frequency is well above the resonant frequency of the soil-foundation system.

-oOo-

The following Plates and Appendix are attached and complete this report:

Plate 1	-	Plot Plan
Plate 2	-	Foundation Design Data
Appendix	-	Field Explorations and Laboratory Tests

Respectfully submitted,

DAMES & MOORE

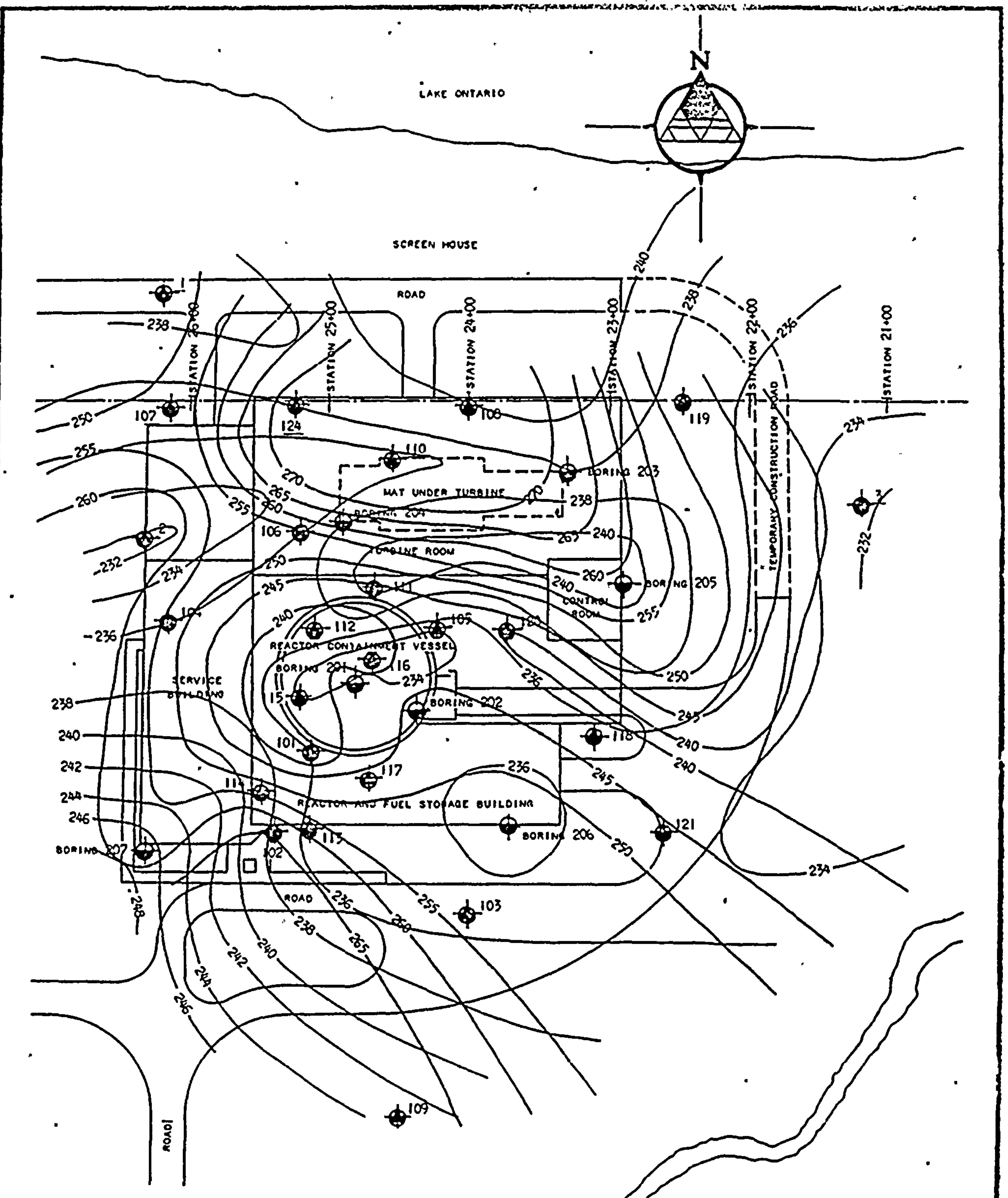
Robert M Perry

Robert M. Perry
State of New York
P.E. Registration No. 35284

Arthur Rothman

RMP-AR:ts

Arthur Rothman



KEY:

BORINGS DRILLED FOR THIS INVESTIGATION

BORINGS DRILLED PREVIOUSLY

PLOT PLAN

SHOWING CONTOURS OF COMPACT GRANULAR SOIL AND UNDERLYING ROCK

FEET

50 0 50 100

REFERENCE:

GILBERT ASSOCIATES, INC., DWG. NO. D-746-001, DATED 2-9-66, ENTITLED "ROCHESTER GAS AND ELECTRIC CORPORATION, CIVIL, SUB-SURFACE EXPLORATION, PLOT PLAN- TEST HOLE LOCATIONS."

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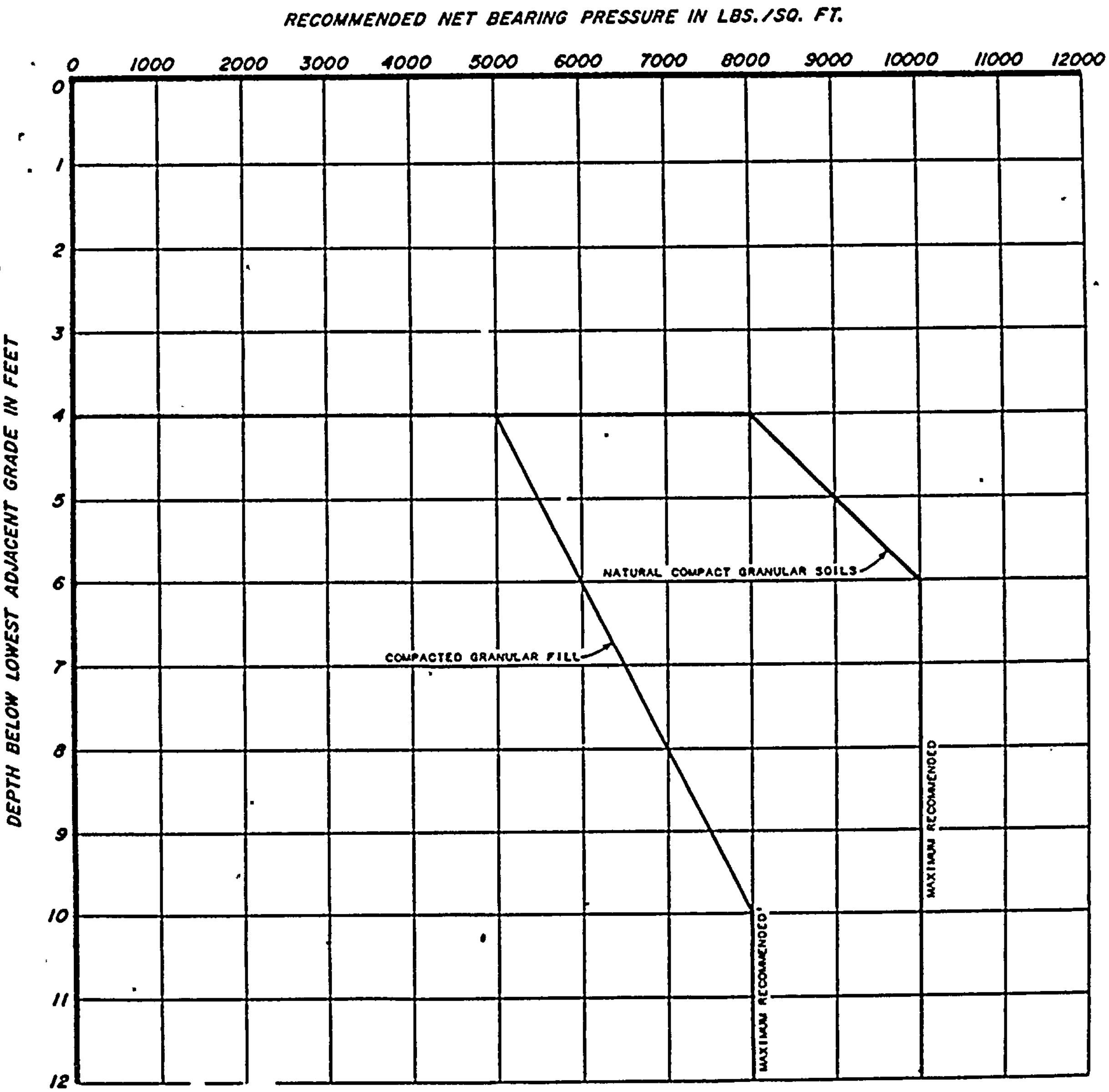
PLATE I

POOR ORIGINAL



NOTE:

SEE TEXT OF REPORT FOR USE OF THIS PLATE.

FOUNDATION DESIGN DATA

NATURAL AND FILL SOILS

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APPENDIX

FIELD EXPLORATIONS AND LABORATORY TESTS

FIELD EXPLORATIONS

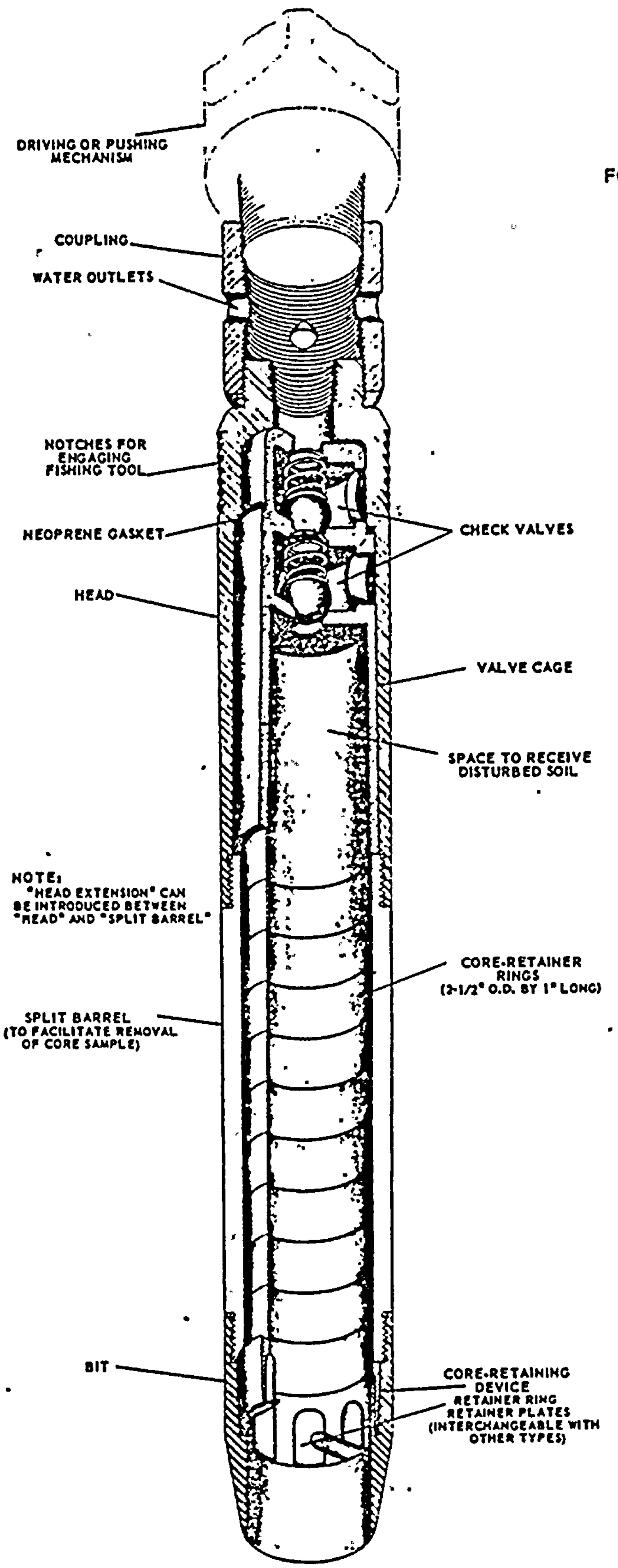
The subsurface conditions in the plant area were explored during this investigation by drilling 7 supplementary test borings to depths ranging from 35 feet to 90 feet below the ground surface. The locations of the borings are shown on the Plot Plan. The field exploration program was conducted under the technical direction of a Dames & Moore Engineering Geologist. The borings were drilled approximately four inches in diameter utilizing truck-mounted rotary drilling equipment. Driller's mud was used where necessary to prevent the walls of the borings from caving.

Continuous observations of the materials encountered in the borings were recorded in the field during drilling operations. Undisturbed soil samples, suitable for laboratory testing, were extracted from the borings utilizing the Dames & Moore sampler illustrated on Page A-2 of this Appendix. The sampler is three and one-quarter inches in outside diameter and approximaterly two and one-half inches in inside diameter. Rock cores were obtained from the two test borings in the reactor area to a depth of 50 feet below the rock surface utilizing a Series NX core barrel. The cores recovered are two and one-eight inches in diameter. The soil samples and rock cores were shipped to our New York office and laboratory where they were further examined and subjected to appropriate laboratory tests.

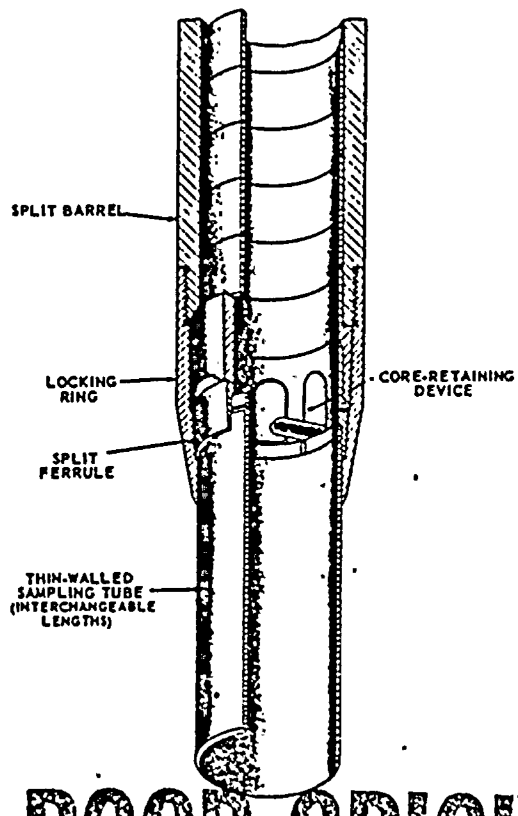
Detailed descriptions of the soils and rock encountered in the borings are presented on Plates A-1A and A-1B, Log of Borings. The soils were classified in accordance with the Unified Soil Classification System described on Plate A-2.

SOIL SAMPLER TYPE U

FOR SOILS DIFFICULT TO RETAIN IN SAMPLER
U. S. PATENT NO. 2,318,062



ALTERNATE ATTACHMENTS



The number of blows required to drive the sampler a distance of one foot into the soil utilizing a 500-pound drive weight falling a distance of 18 inches is presented in the column at the left of the log of each boring. The percent of core recovery obtained during coring operations is also presented in this column.

The elevations which appear at the top of each boring log refer to United States Coast and Geodetic Survey Datum and were determined by representatives of Rochester Gas and Electric Company.

LABORATORY TESTS

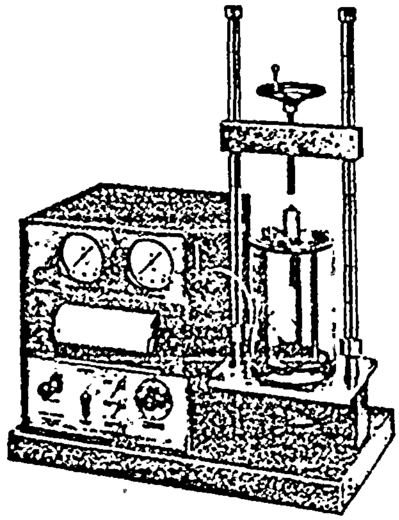
Soil: A number of undisturbed samples of the natural compact granular soils were tested to evaluate their strength characteristics. Triaxial compression tests were performed on the soil samples in the manner described on Page A-4. In addition to the tests on samples of the natural undisturbed soils, triaxial compression tests were performed on samples of remolded and recompacted granular material. These tests were used in our compacted fill studies to evaluate the variation in strength characteristics with changes in density.

A load-deflection curve was plotted for each strength test and the shearing strength of the soil was determined from this curve. Determinations of the moisture content and dry density of the soils were made in conjunction with each strength test.

The results of the strength tests and the corresponding moisture and density determinations are tabulated on Page A-5, Summary of Soil Strength Test Data.

METHODS OF PERFORMING UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS

THE SHEARING STRENGTHS OF SOILS ARE DETERMINED FROM THE RESULTS OF UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS. IN TRIAXIAL COMPRESSION TESTS THE TEST METHOD AND THE MAGNITUDE OF THE CONFINING PRESSURE ARE CHOSEN TO SIMULATE ANTICIPATED FIELD CONDITIONS.



TRIAXIAL COMPRESSION TEST UNIT

UNCONFINED COMPRESSION AND TRIAXIAL COMPRESSION TESTS ARE PERFORMED ON UNDISTURBED OR REMOLDED SAMPLES OF SOIL APPROXIMATELY SIX INCHES IN LENGTH AND TWO AND ONE-HALF INCHES IN DIAMETER. THE TESTS ARE RUN EITHER STRAIN-CONTROLLED OR STRESS-CONTROLLED. IN A STRAIN-CONTROLLED TEST THE SAMPLE IS SUBJECTED TO A CONSTANT RATE OF DEFLECTION AND THE RESULTING STRESSES ARE RECORDED. IN A STRESS-CONTROLLED TEST THE SAMPLE IS SUBJECTED TO EQUAL INCREMENTS OF LOAD WITH EACH INCREMENT BEING MAINTAINED UNTIL AN EQUILIBRIUM CONDITION WITH RESPECT TO STRAIN IS ACHIEVED.

YIELD, PEAK, OR ULTIMATE STRESSES ARE DETERMINED FROM THE STRESS-STRAIN PLOT FOR EACH SAMPLE AND THE PRINCIPAL STRESSES ARE EVALUATED. THE PRINCIPAL STRESSES ARE PLOTTED ON A MOHR'S CIRCLE DIAGRAM TO DETERMINE THE SHEARING STRENGTH OF THE SOIL TYPE BEING TESTED.

UNCONFINED COMPRESSION TESTS CAN BE PERFORMED ONLY ON SAMPLES WITH SUFFICIENT COHESION SO THAT THE SOIL WILL STAND AS AN UNSUPPORTED CYLINDER. THESE TESTS MAY BE RUN AT NATURAL MOISTURE CONTENT OR ON ARTIFICIALLY SATURATED SOILS.

IN A TRIAXIAL COMPRESSION TEST THE SAMPLE IS ENCASED IN A RUBBER MEMBRANE, PLACED IN A TEST CHAMBER, AND SUBJECTED TO A CONFINING PRESSURE THROUGHOUT THE DURATION OF THE TEST. NORMALLY, THIS CONFINING PRESSURE IS MAINTAINED AT A CONSTANT LEVEL, ALTHOUGH FOR SPECIAL TESTS IT MAY BE VARIED IN RELATION TO THE MEASURED STRESSES. TRIAXIAL COMPRESSION TESTS MAY BE RUN ON SOILS AT FIELD MOISTURE CONTENT OR ON ARTIFICIALLY SATURATED SAMPLES. THE TESTS ARE PERFORMED IN ONE OF THE FOLLOWING WAYS:

<u>UNCONSOLIDATED-UNDRAINED:</u> THE CONFINING PRESSURE IS IMPOSED ON THE SAMPLE AT THE START OF THE TEST. NO DRAINAGE IS PERMITTED AND THE STRESSES WHICH ARE MEASURED REPRESENT THE SUM OF THE INTERGRANULAR STRESSES AND PORE WATER PRESSURES.

<u>CONSOLIDATED-UNDRAINED:</u> THE SAMPLE IS ALLOWED TO CONSOLIDATE FULLY UNDER THE APPLIED CONFINING PRESSURE PRIOR TO THE START OF THE TEST. THE VOLUME CHANGE IS DETERMINED BY MEASURING THE WATER AND/OR AIR EXPELLED DURING CONSOLIDATION. NO DRAINAGE IS PERMITTED DURING THE TEST AND THE STRESSES WHICH ARE MEASURED ARE THE SAME AS FOR THE UNCONSOLIDATED-UNDRAINED TEST.

<u>DRAINED:</u> THE INTERGRANULAR STRESSES IN A SAMPLE MAY BE MEASURED BY PERFORMING A DRAINED, OR SLOW, TEST. IN THIS TEST THE SAMPLE IS FULLY SATURATED AND CONSOLIDATED PRIOR TO THE START OF THE TEST. DURING THE TEST, DRAINAGE IS PERMITTED AND THE TEST IS PERFORMED AT A SLOW ENOUGH RATE TO PREVENT THE BUILDUP OF PORE WATER PRESSURES. THE RESULTING STRESSES WHICH ARE MEASURED REPRESENT ONLY THE INTERGRANULAR STRESSES. THESE TESTS ARE USUALLY PERFORMED ON SAMPLES OF GENERALLY NON-COHESIVE SOILS, ALTHOUGH THE TEST PROCEDURE IS APPLICABLE TO COHESIVE SOILS IF A SUFFICIENTLY SLOW TEST RATE IS USED.

AN ALTERNATE MEANS OF OBTAINING THE DATA RESULTING FROM THE DRAINED TEST IS TO PERFORM AN UNDRAINED TEST IN WHICH SPECIAL EQUIPMENT IS USED TO MEASURE THE PORE WATER PRESSURES. THE DIFFERENCES BETWEEN THE TOTAL STRESSES AND THE PORE WATER PRESSURES MEASURED ARE THE INTERGRANULAR STRESSES.

SUMMARY OF SOIL STRENGTH TEST DATA

BORING	DEPTH (feet)	DRY DENSITY (pcf)	MOISTURE CONTENT (percent)	CELL PRESSURE (psf)	ONE-HALF DEVIATOR STRESS (psf)	REMARKS
202	30½	114	11.2	1,500	3,900	Natural
		110	11.0	1,500	2,100	Recompacted
				2,000	2,900	Recompacted
				3,000	4,400	Recompacted
		115	10.6	1,500	3,300	Recompacted
				2,000	3,750	Recompacted
		127	10.5	1,500	4,150	Recompacted
203	10½	117	10.8	500	1,400	Natural
				1,500	2,700	Natural
		124	11.2	500	2,700	Recompacted
				1,500	4,300	Recompacted
203	15½	120	12.1	1,500	1,600	Natural
				3,000	3,800	Natural
				6,000	8,300	Natural
		111	11.5	500	800	Recompacted
				1,000	1,800	Recompacted
				3,000	4,000	Recompacted
204	20½	112	7.3	2,000	4,000	Natural
		111	7.6	2,000	3,500	Recompacted
205	15½	125	11.6	1,000	3,000	Natural
		122	11.8	1,000	1,800	Recompacted
207	16½	144	6.5	2,000	5,200	Natural

Rock: Unconfined compression, triaxial compression and tension tests were performed on selected rock cores extracted from the borings. These tests were performed by subjecting rock cores approximately two and one-eight inches in diameter and four to six inches in height to an axial strain and recording the resisting stress developed by the rock. A stress-strain curve was plotted for each of the compression tests and the shearing strength of the rock was determined from this curve. The results of the strength tests on the rock cores are presented below:

BORING	DEPTH (feet)	CELL PRESSURE (psi)	ONE-HALF DEVIATOR STRESS (psi)	TYPE OF TESI
201	42	-	50*	Tension
201	45	-	50*	Tension
201	49	1,000	4,700	Triaxial Compression
202	47	-	3,900	Unconfined Compression
202	50½	1,500	4,400	Triaxial Compression

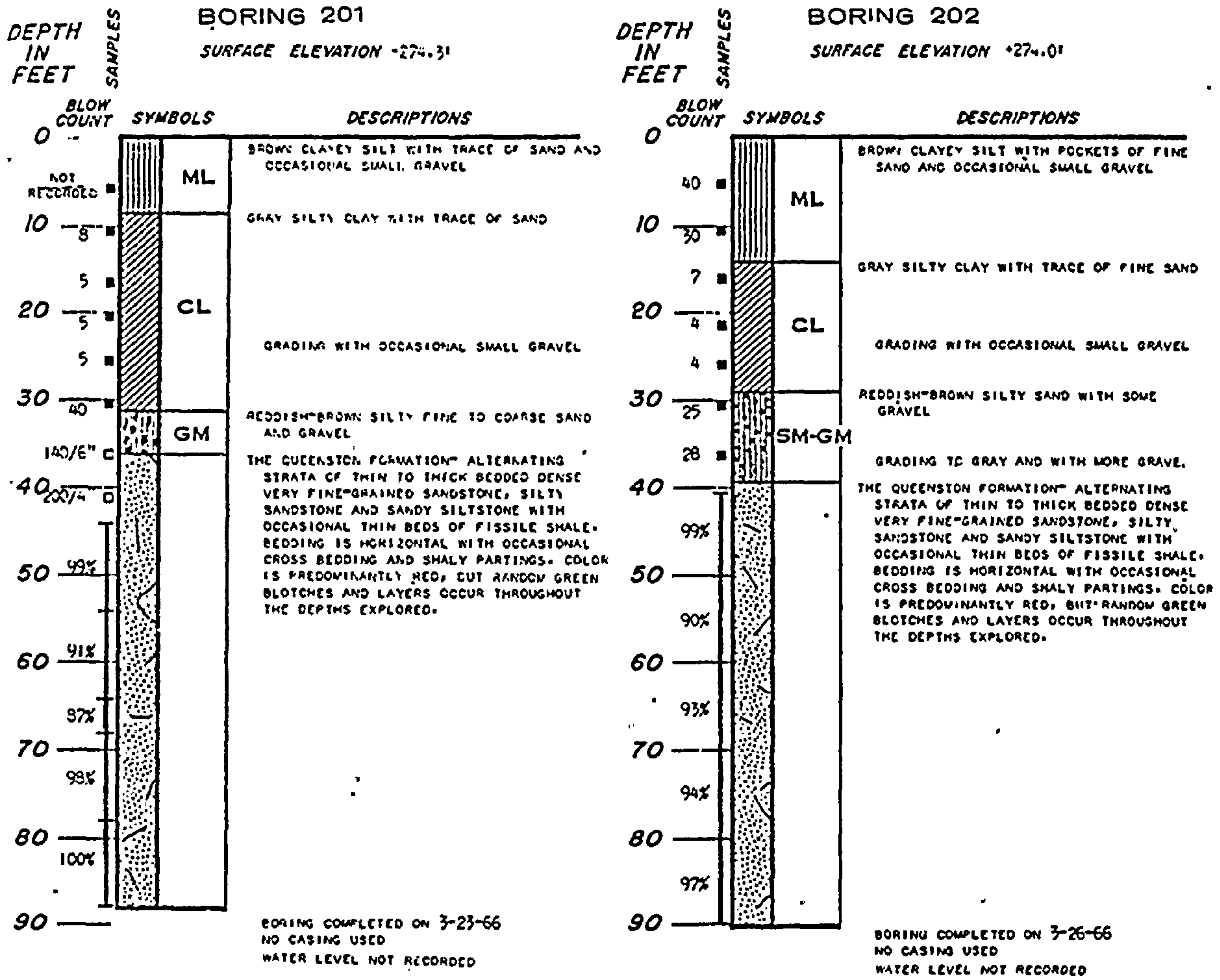
* Indicates peak tensile stress normal to bedding planes.

The following Plates are attached and complete this Appendix:

Plate A-1A - Log of Borings (Borings 201 and 202)

Plate A-1B - Log of Borings (Borings 203 through 207)

Plate A-2 - Unified Soil Classification System and Key to Test Data



LOG OF BORINGS

NOTES:

FIGURES UNDER THE COLUMN ENTITLED "BLOW COUNT" REFER TO THE NUMBER OF BLOWS REQUIRED TO DRIVE THE DAMES & MOORE SAMPLER ONE FOOT INTO THE OVERBURDEN WITH A [illegible]-LB. DRIVE WEIGHT FALLING A DISTANCE OF 18 INCHES. THE DAMES & MOORE SAMPLER IS 3¼" O.D. AND 2½" I.D.

THE PERCENT OF CORE RECOVERED IN A CORING RUN IS ALSO INDICATED IN THE COLUMN ENTITLED "BLOW COUNT". AN [illegible]-SIZE CORE BARREL WAS USED TO CORE THE BEDROCK. THE CORE RECOVERED IS 2⅛" IN DIAMETER.

SURFACE ELEVATIONS REFER TO USC & GS DA[illegible]

DAMES & MOORE

PLATE A-1A

BORING 203

SURFACE ELEVATION +273.6'

DEPTH IN FEET	BLOW COUNT	SYMBOLS	DESCRIPTIONS
0		ML	BROWN CLAYEY SILT WITH TRACE OF SAND AND OCCASIONAL SMALL GRAVEL
	102	SM·GM	REDDISH-BROWN SILTY FINE TO MEDIUM SAND WITH GRAVEL AND ROCK FRAGMENTS
10	102		WITH LESS GRAVEL
	74		
20	180/6"	GM	REDDISH-BROWN SILTY SAND AND GRAVEL WITH ROCK FRAGMENTS
	150/6"		
	100/2"		WITH MORE ROCK FRAGMENTS
30	2C0/7"		
			WITH MORE ROCK FRAGMENTS
	150/4"		THE QUEENSTON FORMATION- RED SANDSTONE
40			

BORING COMPLETED ON 3-25-66
NO CASING USED
WATER LEVEL NOT RECORDED

BORING 204

SURFACE ELEVATION +276.4'

DEPTH IN FEET	BLOW COUNT	SYMBOLS	DESCRIPTIONS
0		CL	BROWN SILTY CLAY WITH OCCASIONAL SMALL GRAVEL
	39		
10	18		
	106	GM	REDDISH-BROWN SILTY FINE SAND AND GRAVEL
20	38	SM	REDDISH-BROWN SILTY FINE TO MEDIUM SAND WITH OCCASIONAL SMALL GRAVEL
	84		MEDIUM SAND GRADING OUT
30	154		
	185/6"	SM·GM	REDDISH-BROWN SILTY FINE SAND WITH GRAVEL AND ROCK FRAGMENTS
40			THE QUEENSTON FORMATION- RED SANDSTONE

BORING COMPLETED ON 3-24-66
NO CASING USED
WATER LEVEL NOT RECORDED

BORING 205

SURFACE ELEVATION +273.7'

DEPTH IN FEET	BLOW COUNT	SYMBOLS	DESCRIPTIONS
0		CL	BROWN SILTY CLAY WITH OCCASIONAL SMALL GRAVEL
	35		
10	21		
	60	SM·GM	REDDISH-BROWN SILTY FINE SAND WITH GRAVEL
20	86		GRADING WITH LESS SILT
	152/9"	GC	REDDISH-BROWN CLAYEY SAND AND GRAVEL WITH SEAMS OF FINE SAND
30	194/10"		WITH ROCK FRAGMENTS
	200/6"		THE QUEENSTON FORMATION- RED SANDSTONE
40			

BORING COMPLETED ON 3-25-66
NO CASING USED
WATER LEVEL NOT RECORDED

BORING 206

SURFACE ELEVATION +271.5'

DEPTH IN FEET	BLOW COUNT	SYMBOLS	DESCRIPTIONS
0		CL	BROWN SILTY CLAY WITH LITTLE FINE SAND AND OCCASIONAL SMALL GRAVEL
	16		
10	17		
	15	CL	GRAY SILTY CLAY
20	100	GM	REDDISH-BROWN SILTY FINE SAND AND GRAVEL WITH ROCK FRAGMENTS
	98		
	130/3"		
30	88/6"		
	150/2"		THE QUEENSTON FORMATION- RED SANDSTONE
40			

BORING COMPLETED ON 3-28-66
NO CASING USED
WATER LEVEL NOT RECORDED

BORING 207

SURFACE ELEVATION +273.1'

DEPTH IN FEET	BLOW COUNT	SYMBOLS	DESCRIPTIONS
0		SM	BROWN SILTY FINE SAND
	9	CL	BROWN SILTY CLAY WITH OCCASIONAL SMALL GRAVEL
10	14		
	101	GM	REDDISH-BROWN SILTY FINE SAND AND GRAVEL WITH ROCK FRAGMENTS
20	142		
			WITH MORE ROCK FRAGMENTS
	154/7"		THE QUEENSTON FORMATION- RED SANDSTONE
30	200/2½"		
	140/1"		
40			

BORING COMPLETED ON 3-23-66
NO CASING USED
WATER LEVEL NOT RECORDED

LOG OF BORINGS

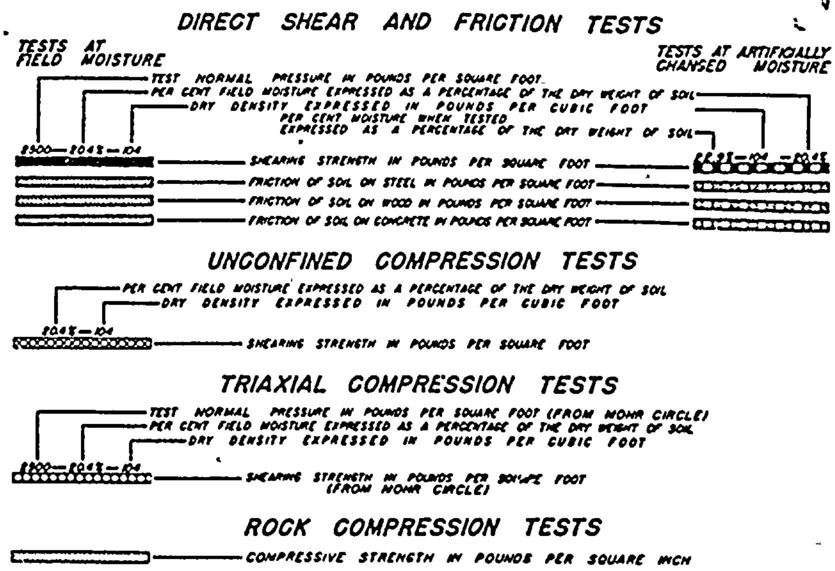
DAMES & MOORE

POOR ORIGINAL

PLATE A-1B

MAJOR D	
COARSE GRAINED SOILS	GRAVEL AND GRAVELL SOILS
	MORE THAN 5 OF COARSE F TION RETAIN ON NO. 4 SIE
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS
	MORE THAN 5(OF COARSE FA TION PASSIN NO. 4 SIEVE
FINE GRAINED SOILS	SILTS AND CLAYS
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS
HIGHLY ORGANIC	

NOTE: DUA



KEY TO TEST DATA

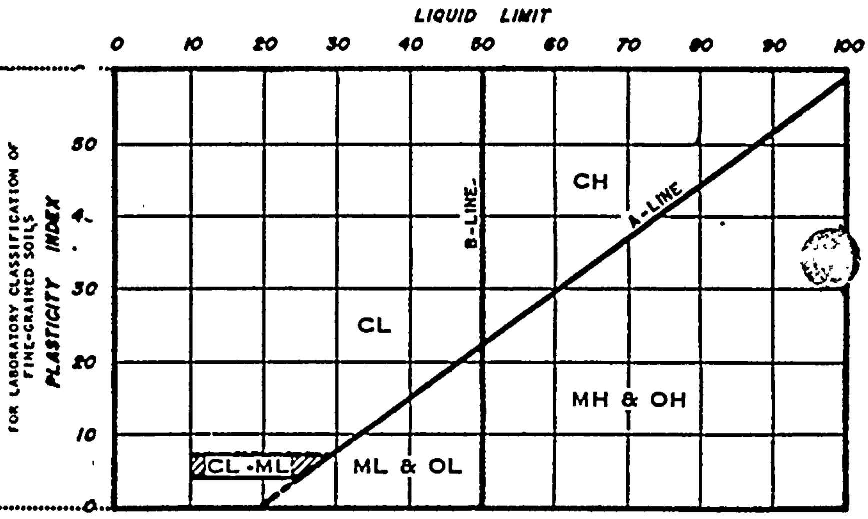
- ■ INDICATES DEPTH OF UNDISTURBED SAMPLE
- ⊠ INDICATES DEPTH OF DISTURBED SAMPLE
- □ INDICATES DEPTH OF SAMPLING ATTEMPT WITH NO RECOVERY
- ⊟ INDICATES DEPTH OF SPLIT-SPOON SAMPLE
- I INDICATES DEPTH AND LENGTH OF CORING RUN

KEY TO SAMPLES

...ONS	GRAPH SYMBOL	LETTER SYMBOL	TYPICAL DESCRIPTIONS
CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
		GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
CLEAN SAND (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SP	POORLY-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
SANDS ...ES (APPREC... AMOUNT OF FINES)		SM	SILTY SANDS, SAND-SILT MIXTURES
		SC	CLAYEY SANDS, SAND-CLAY MIXTURES
LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
		OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
...S		PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

...MBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS.

...IL CLASSIFICATION CHART



PLASTICITY CHART

UNIFIED SOIL CLASSIFICATION SYSTEM

Gilbert Associates, Inc. Reading, Pennsylvania ANALYSIS/CALCULATION	SUBJECT GINNA ROCK CREEP				1
	REV. 0	1	2	3	PAGES
	MICROFILMED				
	ORIGINATOR [illegible]				
	DATE 4.22.80				

POOR ORIGINAL

Reference: Lucius Pitkin report dated Sept. 6, 1973
"Compression Tests of Cored Rock Samples"

From test data
At 10 min. $\epsilon = 3785\ \mu$"/"
At 250 min. $\epsilon = 3870\ \mu$"/"

$\therefore\ \Delta\epsilon = 85\ \mu$"/" in 240 min. (14400 sec)

$$\epsilon_{total} = \epsilon_{eo} + a \ln t$$

$$\boxed{\epsilon_{cr} = \epsilon_{total} - \epsilon_{eo} = a \ln t} \quad \text{—— (1)}$$

$$\boxed{\text{Also}\quad a = (\sigma/E)^n}$$ from Eng. Properties of Rock by Farmer —— (2)

Eqn (1) gives

$$a = \frac{\epsilon_{cr}}{\ln t} = \frac{85}{\ln (14400)} = 8.88 \times 10^{-6} \text{ for 10000 psi stress}$$

For Eqn (2), use $E = 1.89 \times 10^5$ TSF as per Tab Iqbal
$= 2.63 \times 10^6$ psi

$$\therefore\ 8.88 \times 10^{-6} = \left(\frac{10000}{2.63 \times 10^6}\right)^n = (3.80228 \times 10^{-3})^n$$

$\therefore\ n = 2.09$ — use

or $(10000/2.63 \times 10^6)^{2.09} = 8.76 \times 10^{-6} \approx 8.88 \times 10^{-6}$
OK

Rock Anchors @ 2'-1" = 2.083'
lock off force = 168 ksi × 4.42 in² = 743^k
∴ Force per FT = 743^k/2.083' = 357^k/FT.
Rock anchors bear on 6'-3" wide footing (Dwg D-421-001)
∴ Rock stress under footing : 357'/(12 × 75) = 397 psi

Gilbert Associates, Inc. Reading, Pennsylvania ANALYSIS/CALCULATION	SUBJECT GINNA Rock CREEP				CISID	PAGE 2 OF PAGES
	REV.	0	1	2	3	
	MICROFILMED					
	ORIGINATOR	[illegible]				
	DATE	4.22.80				

∴ Rock creep Strain just under footing =

$\epsilon_{cr} = (\sigma/E)^n \ln t = \left(\frac{377\,psi}{2.63\times10^6}\right)^{2.09} \ln t =$

$\epsilon_{cr} = 0.0103\times10^{-6} \ln t$

At 15 yrs., $t = 15\times365\times24\times60\times60 = 473\times10^6$ sec

$\therefore \epsilon_{cr} = (.0103\times10^{-6}) \ln 473\times10^6$
$= (.0103\times10^{-6})(19.97) = \boxed{.2\ \mu''/''}$

insignificant

✓ C11010

At 40 yrs., $t = 1261\times10^6$ sec

$\therefore \epsilon_{cr} = (.0103\times10^{-6}) \ln 1261\times10^6 = \underline{0.21\ \mu''/''}$

Weight of Reactor Bldg Containment produces 70 k/FT on footing

POOR ORIGINAL

Lucius Pitkin INCORPORATED EST. 1885 Analytical, Metallurgical and Research Laboratories

50 HUDSON STREET, NEW YORK, N. Y. 10013 • (212) BE 3-2737

CABLE ADDRESS: NIKTIP

REPORT

September 6, 1973

M-2871

Your Ref.: File GINNA

Dames & Moore
14 Commerce Drive
Cranford, New Jersey 07016

Attention: Mr. Adekunle Oguntala

Subject: COMPRESSION TESTS OF CORED ROCK SAMPLES

Two cored rock specimens (204-114-1 & 204-104) were prepared and tested in unconfined axial compression. Strain gages of X-Y configuration were employed to determine axial and lateral strain.

One cored rock specimen (204-125) was prepared and tested in unconfined axial compression-creep. This specimen was loaded to 10,000 psi for four hours. The specimen was then loaded from 10,000 psi to failure.

One cored rock specimen (204-114-2) was prepared and tested in cyclic triaxial compression at a confining pressure of 100 psi. The specimen was loaded and unloaded 10 times to a stress of 6000 psi; 10 times to a stress of 9000 psi, and 10 times to a stress of 12,000 psi. At the completion of cyclic loading, the specimen was tested to failure. Strain gages of X-Y configuration were employed to determinal axial and lateral strain.

One cored specimen was prepared from sample 204-125 and returned to Dames & Moore. Cored rock sample 204-86 was cut into two samples and returned to Dames & Moore.

Poisson's ratio for each specimen tested in unconfined axial compression was calculated from the linear portion of the stress-strain curve.

Complete results of all tests performed are appended.

Respectfully submitted,

LUCIUS PITKIN, INC.

Approved: A. J. Vecchio
Asst. Chief Metallurgist

J. Crosson
Metallurgist

FORM 107-11/68

Dame[illegible] Moore
Attn: M[illegible] Oguntala

Luc[illegible]tkin
inc[illegible]orated

Date: Sept [illegible]
M-2871 F[illegible]

UNCONFINED COMPRESSION TESTS

Sample	Wt. (gms)	Diam. (in)	Length (in)	Area (sq.in.)	Volume (cu.in.)	Density (gms/cu.in.)	Ult. Load (lbs)	Ult. Str., psi	Remarks
204-114-1	491	1.87	4.30	2.75	11.83	41.50	52,500	19,100	violent shat
204-104	489	1.86	4.28	2.72	11.64	42.01	60,500	22,240	violent shat
204-125	483	1.86	4.25	2.72	11.56	41.78	50,500	18,560	violent shat
204-125	485	1.85	4.33	2.69	11.65	41.63	returned to Dames and Moore		

TRIAXIAL COMPRESSION TESTS

Sample	Wt. (gms)	Diam. (in)	Length (in)	Area (sq.in.)	Volume (cu.in)	Density (gms/cu.in.)	Ult. Load (lbs)	Ult. Str., psi	Remarks
204-114-2	497	1.87	4.34	2.75	11.94	41.62	52,000	18,910	violent shat Confining Pressure-100

Lucius Pitkin
incorporated

Dames & Moore
Attn: Mr A. Oguntala

September 6, 1973
M-2871 File: Ginna

POISSON'S RATIO

Sample	Stress Range For Poisson's Ratio	Poisson's Ratio
204-114	5090-12,730	.25
204-104	5150-11,030	.19

MOISTURE CONTENT

Sample	MOISTURE CONTENT, %
204-114	0.0951
204-125	0.0958

Lucius Pitkin
Incorporated

Dames & Moore
Attn: Mr A.. Oguntala

M-2871 File: Ginna
September 6, 1973

Sample: 204-125

UNCONFINED COMPRESSION-CREEP

Axial Load (lbs)	Axial Stress (psi)	Axial Strain (u-in/in)	Lateral Strain (u-in/in)	Elapsed Time (min.)
0	0	0	0	0
1000	370	180	0	0.3
2000	740	390	0	0.7
3000	1100	630	5	1.0
4000	1470	830	10	1.3
5000	1840	1030	15	1.7
6000	2210	1250	25	2.0
7000	2570	1440	45	2.3
8000	2940	1630	55	2.7
9000	3310	1795	65	3.0
10,000	3680	1955	75	3.3
11,000	4040	2110	95	3.7
12,000	4410	2250	110	4.0
13,000	4780	2370	125	4.3
14,000	5150	2500	140	4.7
15,000	5510	2605	155	5.0
16,000	5880	2715	175	5.3
17,000	6250	2820	190	5.7
18,000	6620	2920	205	6.0
19,000	6990	3020	225	6.3
[illegible],000	7350	3120	245	6.7
21,000	7720	3220	265	7.0
22,000	8090	3310	280	7.3

Lucius Pitkin

incorporated

Dames & Moore
Attn: Mr A. Oguntala

M-2871 File: Ginn.
September 6, 1973

POOR ORIGINAL

Sample: 204-125

UNCONFINED COMPRESSION-CREEP

Axial Load (lbs)	Axial Stress (psi)	Axial Strain (u-in/in)	Lateral Strain (u-in/in)	Elasped Time (min.)
23,000	8460	3400	300	7.7
24,000	8820	3490	320	8.0
25,000	9190	3580	345	8.3
26,000	9560	3670	375	8.7
27,000	9930	3760	390	9.0
27,200	10,000	(3785)	400	10
27,200	10,000	3830	425	20
27,200	10,000	3835	435	30
27,200	10,000	3845	440	40
27,200	10,000	3845	445	50
27,200	10,000	3850	445	60
27,200	10,000	3860	450	70
27,200	10,000	3860	455	80
27,200	10,000	3860	455	90
27,200	10,000	3860	455	100
27,200	10,000	3860	455	110
27,200	10,000	3860	455	120
27,200	10,000	3860	455	130
27,200	10,000	3860	455	140
27,200	10,000	3860	455	150
27,200	10,000	3860	455	160
27,200	10,000	3860	455	170
27,200	10,000	3860	[illegible]	[illegible]

Dames & Moore
Attn: Mr A. Oguntala

M-2871 File: Gin
September 6, 1973

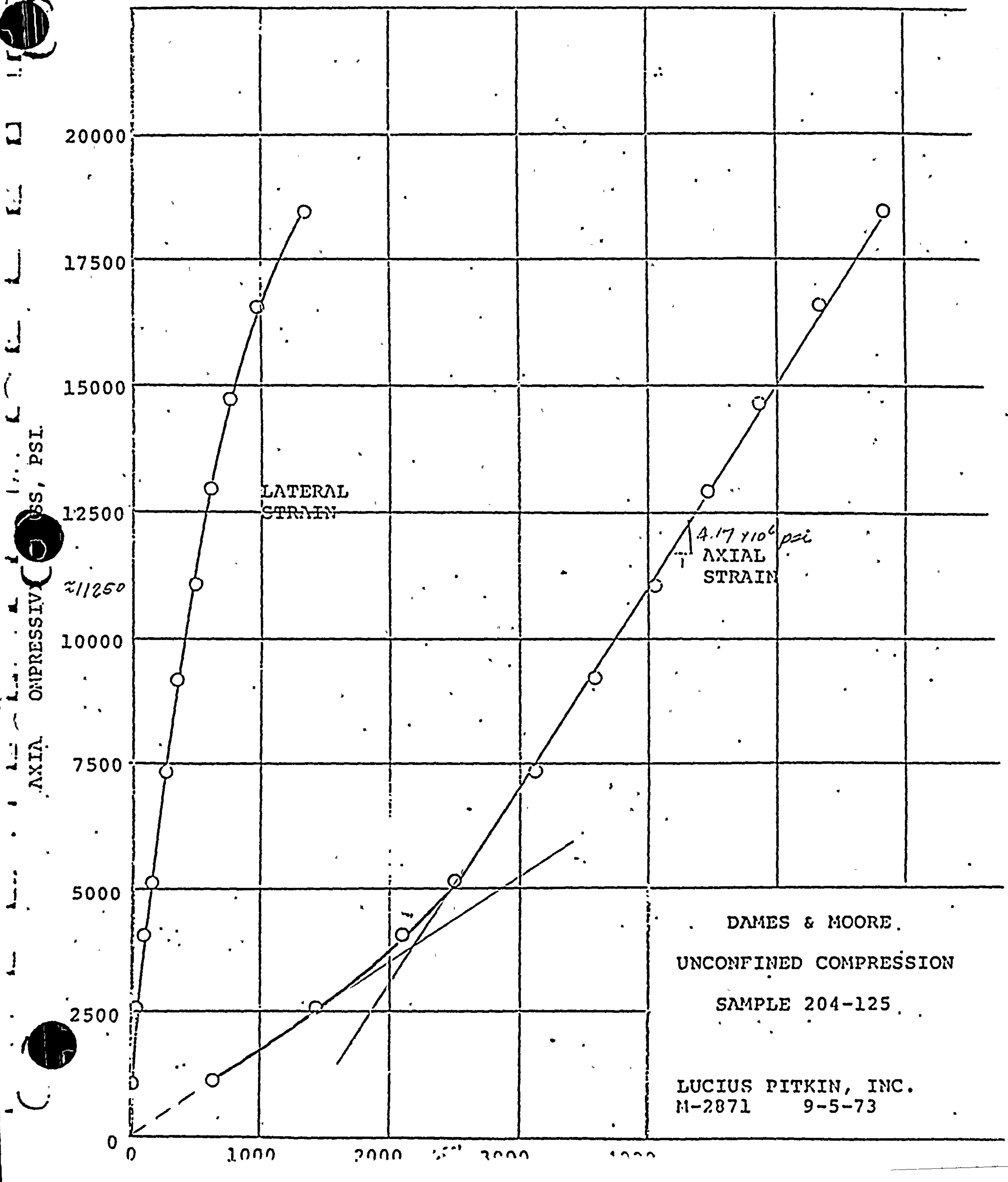
Sample: 204-125

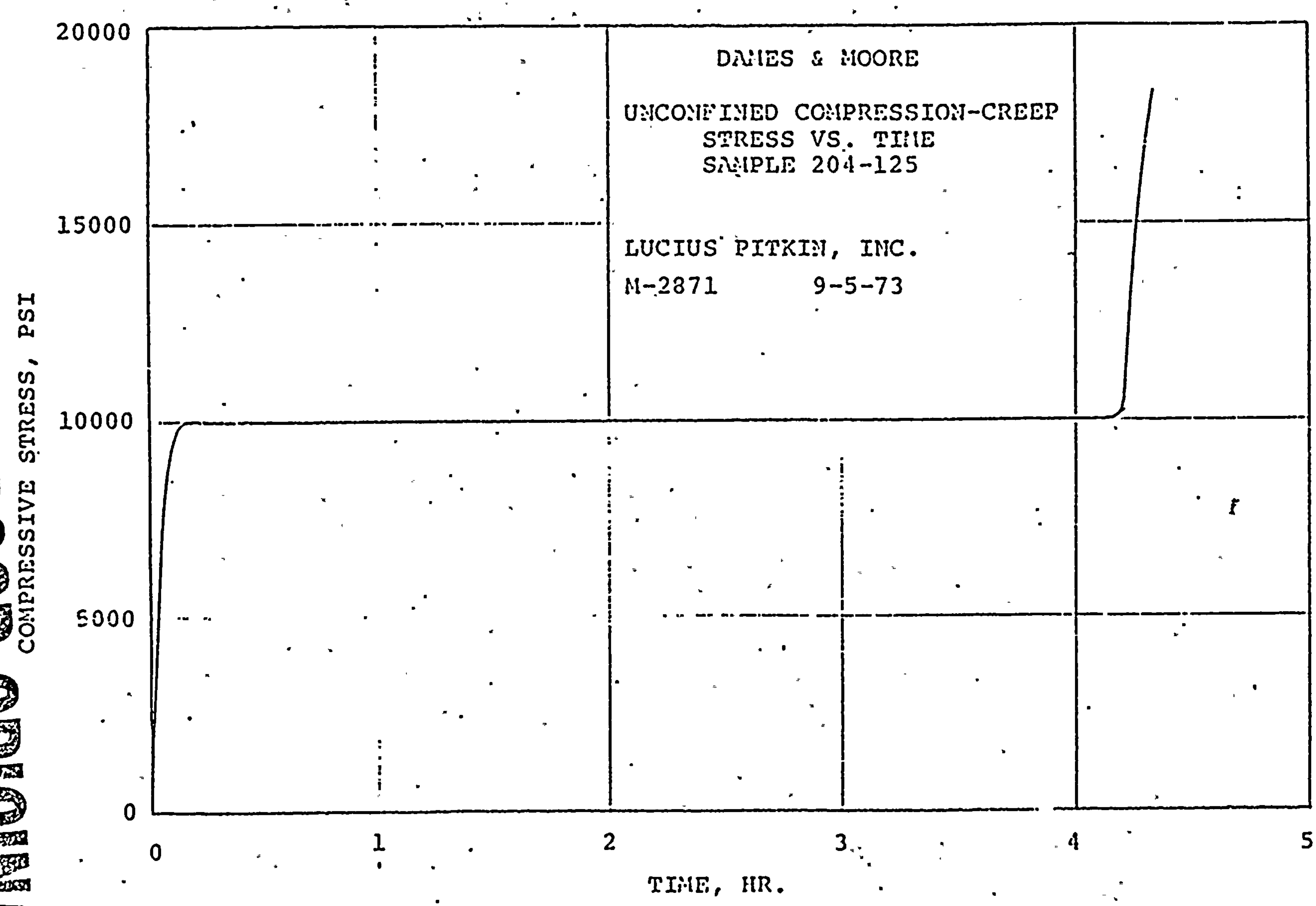
UNCONFINED COMPRESSION-CREEP

Axial Load (lbs)	Axial Stress (psi)	Axial Strain (u-in/in)	Lateral Strain (u-in/in)	Elapsed Time (min.)
27,200	10,000	3860	455	190
27,200	10,000	3860	455	200
27,200	10,000	3865	460	210
27,200	10,000	3865	460	220
27,200	10,000	3865	460	230
27,200	10,000	3865	460	240
27,200	10,000	3870	460	250
28,000	10,290	3925	470	250.5
29,000	10,660	3995	485	251.0
30,000	11,030	4060	505	251.5
35,000	12,870	4450	600	253.5
40,000	14,710	4880	755	255.5
45,000	16,540	5335	985	257.5
50,000	18,380	5860	1340	259.5
50,500	18,560			

Ult. Load

3870
3785


AXIA[L] [C]OMPRESSIV[E STRE]SS, PSI
20000
17500
15000
12500
≈11250
10000
7500
5000
2500
0
0
1000
2000
3000
LATERAL STRAIN
4.17 x10⁶ psi
1
AXIAL STRAIN
DAMES & MOORE
UNCONFINED COMPRESSION
SAMPLE 204-125
LUCIUS PITKIN, INC.
M-2871 9-5-73



DAMES & MOORE
UNCONFINED COMPRESSION-CREEP
STRESS VS. TIME
SAMPLE 204-125
LUCIUS PITKIN, INC.
M-2871 9-5-73
COMPRESSIVE STRESS, PSI
20000
15000
10000
5000
0
0
1
2
3
4
5
TIME, HR.

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

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

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

Rock Anchors

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

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

1. The Raising and Strengthening of Steenbrar Dam, By S. S. Morris and W. S. Garrett, Proceedings, I.C.E., Vol. 1., Pt. 1, No. 1, p. 23; Discussion, Vol. 1, Pt. 1, No. 4, 1956, p. 399.

2. Stress Analysis and Special Problems of Prestressed Dams, O. C. Zienkiewica and R. W. Gerstner, Journal of the Power Division, ASCE, January, 1961.

3. 1300 - Ton Capacity Prestressed Anchors Stabilize Dam, A. Eberhardt and J. A. Veltrop, Journal of the Prestressed Concrete Institute, Vol. 10, No. 4, August, 1965.

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

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

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

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

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

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

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

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

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

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

For 0.60 f_u = 635 kips

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

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

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

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

Weight of rock in kips per ft. circumference = $0.096d^2$ = 67.4

Internal Pressure in kips per ft. circumference

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

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

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

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

why not 240

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

why 75 psig and not 60 psig

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

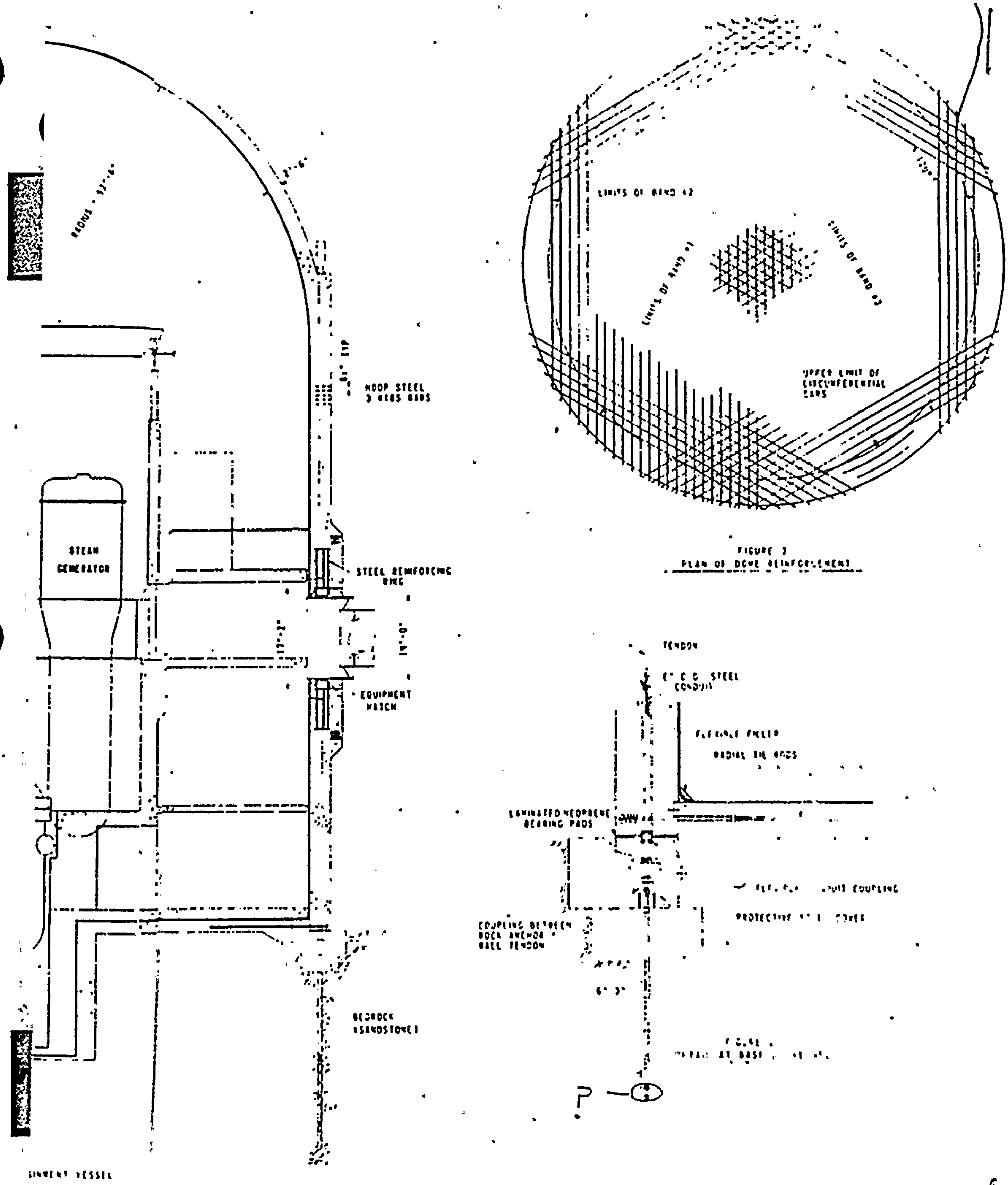
Consideration was also given for seismic loading without internal pressure. For the 0.1g ground motion (vertical and horizontal components considered to occur simultaneously and the effects added algebraically) there is no uplift. Minimum downward component is 0.9 kips per ft. The factor of safety against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

But we have only 67.4 kps/ft.

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

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

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



CONTAINMENT VESSEL CROSS SECTION AND DETAILS
FIG. 5.1.2-1

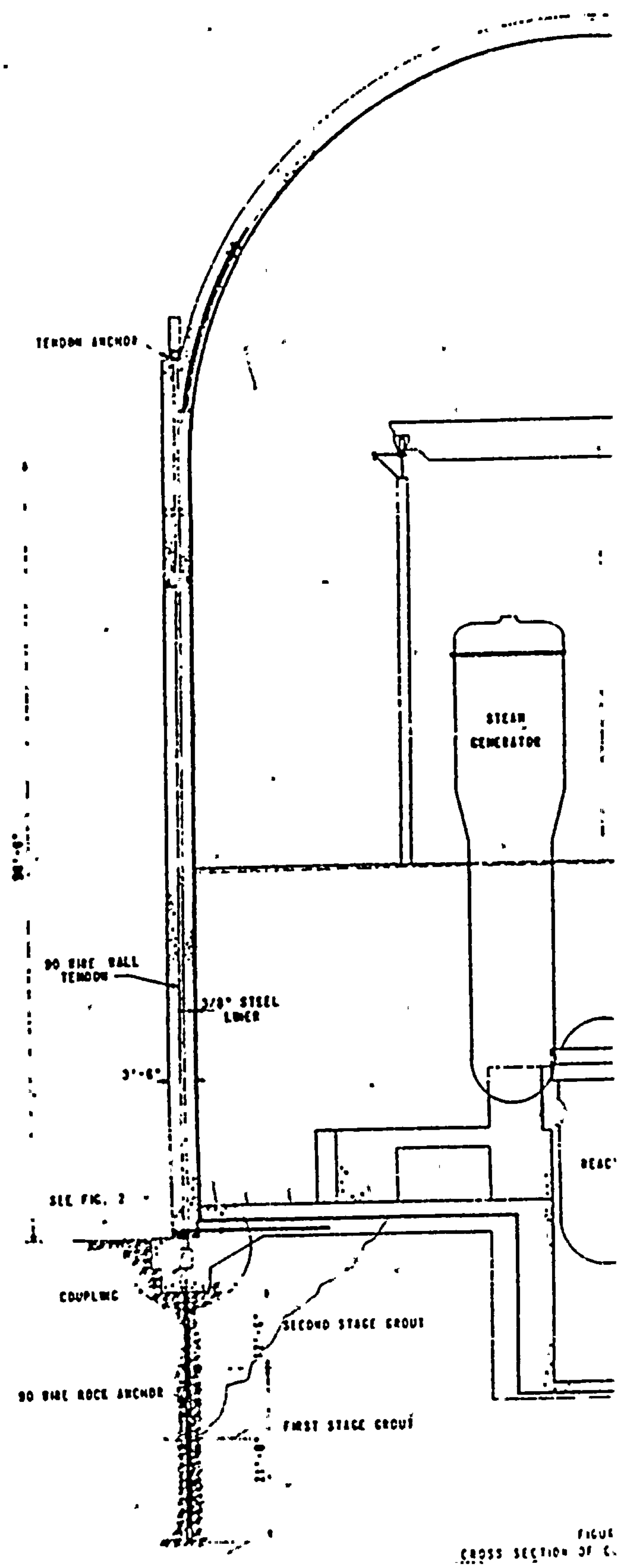
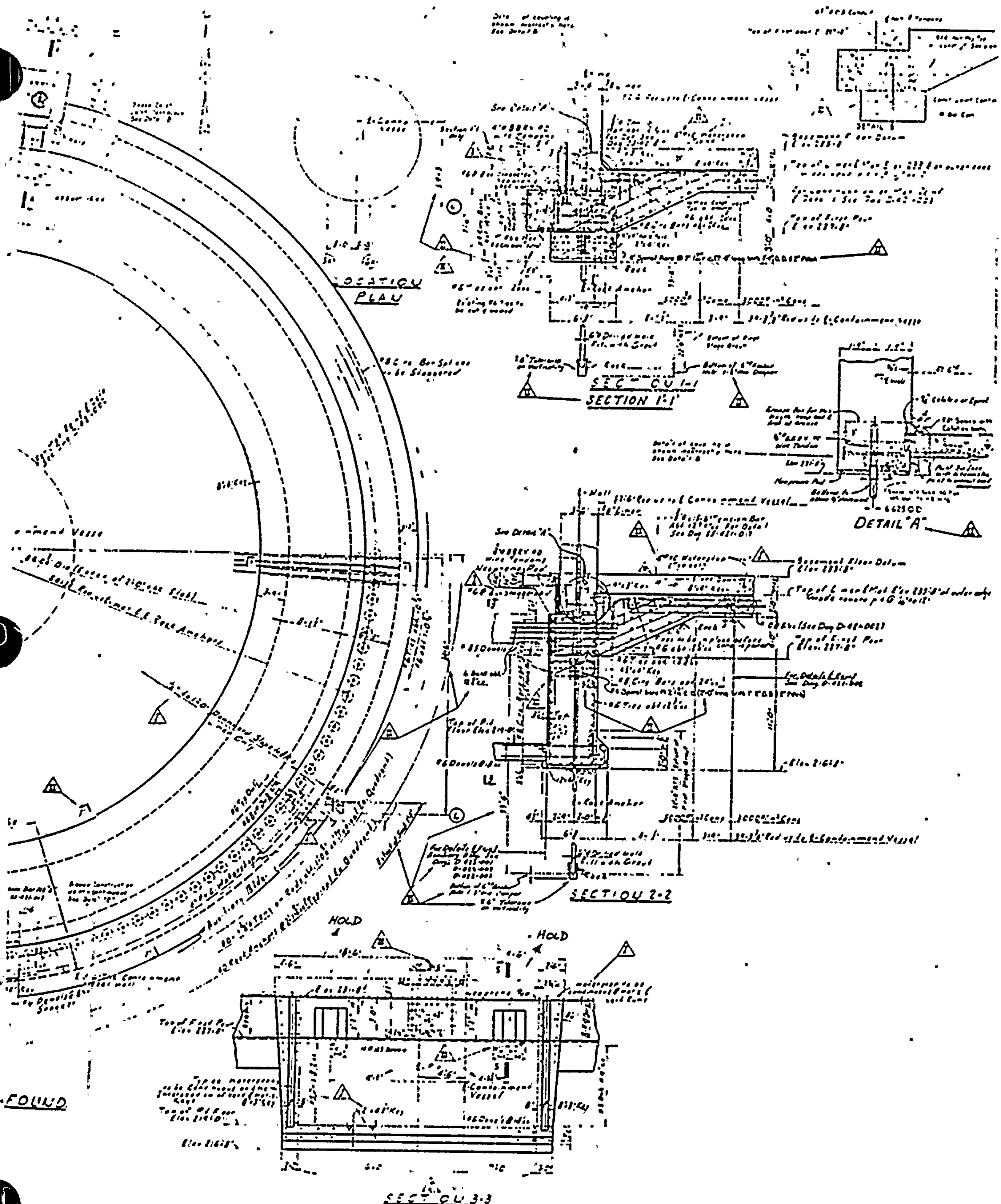


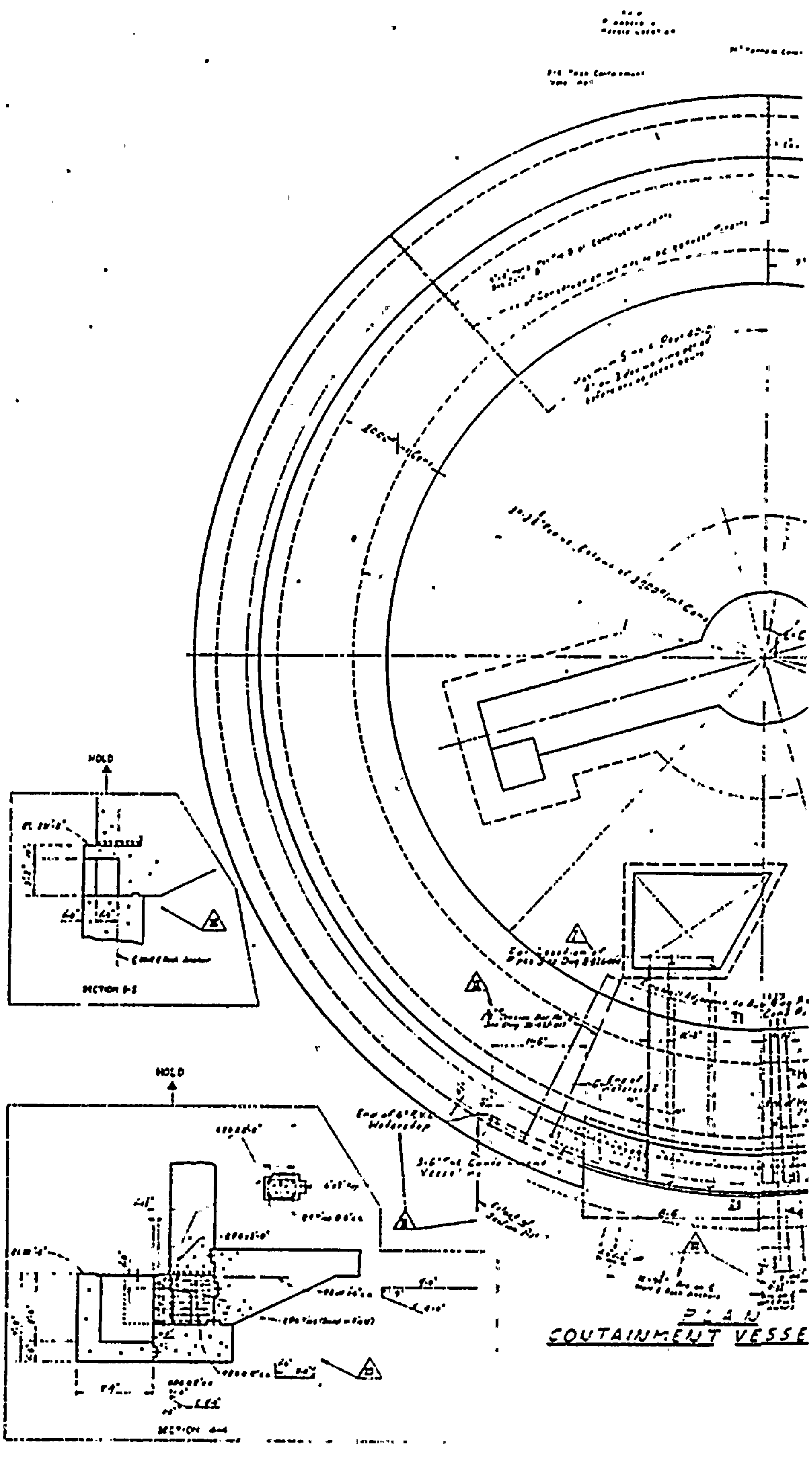
FIGURE
CROSS SECTION OF C
SCALE

POOR ORIGINAL



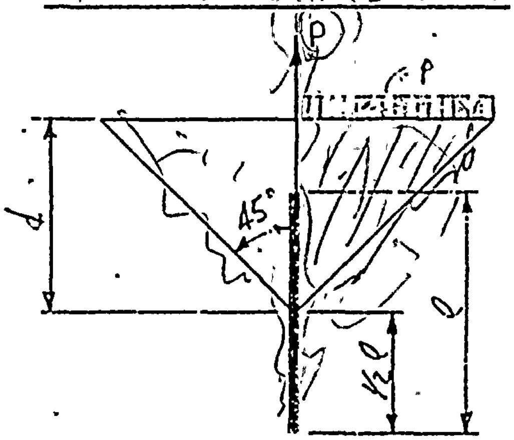
REACTOR CONTAINMENT VESSEL MAT FOUNDATION AND RING GIRDER

FIG. 5.1.2-2



POOR ORIGINAL

ANCHOR DESIGN

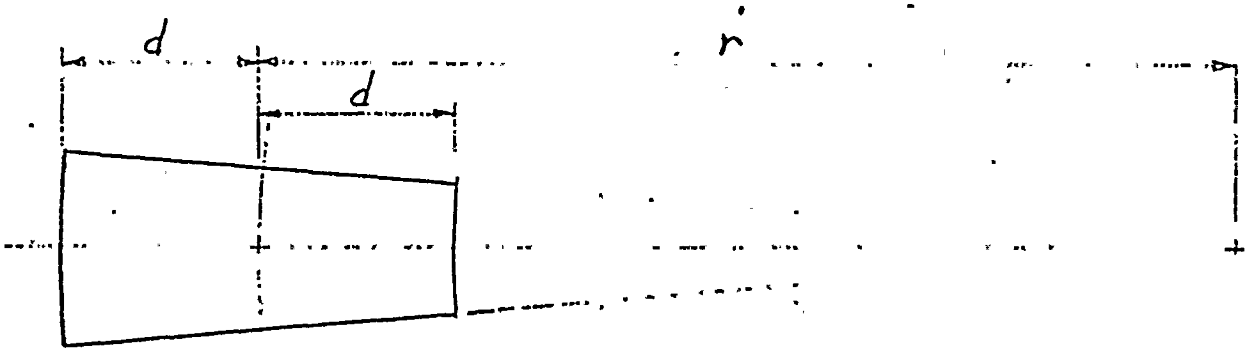


BULK SPECIFIC GRAVITY OF $Rock = 2.54$

SUBMERGED WEIGHT OF ROCK =
$1.54 \times 62.45 = 96$ pcf

p = internal pressure (psi)

$r = \frac{1}{2}(105 + 3.5) = 54.25'$



WT. OF ROCK (kips) / FT. CIRCUMFERENCE $= .096\, d^2$?

INTERNAL PRESSURE (kips) / FT. CIRCUMFERENCE $= .144\, p\left[\pi r^2 - \pi (r-d)^2\right] \cdot \frac{1}{2\pi r}$

$= \frac{.072\, p d (2r - d)}{r}$

BOND STRESS BETWEEN GROUT AND ROCK = 170 psi AT JACKING LOAD
HOLE = 6"φ   90-¼"φ WIRE UNIT 635^k @ 60% f_u

∴ EMBEDMENT LENGTH $= \frac{80/60 \times 635{,}000}{\pi \times 6 \times 170 \times 12} = \underline{22.0'}$

POOR ORIGINAL

BROOKWOOD UNIT NO. 1	GILBERT ASSOCIATES INC — ENGINEERS AND CONSULTANTS — NEW YORK, WASHINGTON
CONTAINMENT VESSEL	SHEET C-.
ROCK ANCHORS	W.O. 4155

108.5
26.5
82.0

FOR $d = 26.5'$ WT. OF ROCK $= .096 \times \overline{26.5}^2 = 67.4$ k/'

FOR "p" = 60 psi INT. PRESSURE $= \dfrac{.072 \times 60 \times 26.5 \times 82.0}{54.25} = 173.0$ k/'

= 69 = 199.0 k/'

= 75 = 216.3 k/'

= 90 = 259.6 k/'

WT. OF ROCK + INT. PRESSURE =

@ 0 psi = 67.4 k/'

@ 60 = 240.4 k/'

@ 69 = 266.4 k/'

@ 75 = 283.7 k/'

@ 90 = 327.0 k/'

FOR LOAD COMBINATION I $C = \frac{1}{0.95}(0.95D + 1.5P + 1.0T + 1.0TL)$

UPLIFT / FT CIRCUMFERENCE = 289.2 k/' < 327.0 ok.

$F.S. = \frac{327.0}{289.2} = 1.13$

FOR 69 psig TEST C = 182 k/' < 266.4 ok.

$F.S. = \frac{266.4}{182} = 1.47$

TOTAL PRESTRESS FORCE = 297.5 k/'

POOR ORIGINAL

BROOKWOOD UNIT NO. 1	READING PENNA	GILBERT ASSOCIATES INC ENGINEERS AND CONSULTANTS	NEW YORK WASHINGTON
CONTAINMENT VESSEL ROCK ANCHORS	MADE DC	CHKD / SO CF / CF ON / APPD	SIZE / DRAWING NO. / REV. / SHEET — W.O. 4155 / DATE

FOR LOAD COMBINATION II : $C = \frac{1}{0.95}(0.95D + 1.25P + 1.0T' + 1.0TL' + 1.25E)$

MAX. UPLIFT / FT CIRCUMFERENCE = 274.1 k/

MIN UPLIFT / FT CIRCUMFERENCE = 150.5 k/

F.S. (AGAINST OVERTURNING) = $\frac{71.4\,\pi\,108.5 \times 56.0}{237 \times .667 \times 24{,}220 \times 149.5}$ = 2.38 ok

FOR 0.1g GROUND MOTION - NO LOSS-OF-COOLANT ACCIDENT

MAX. UPLIFT / FT. CIRCUMFERENCE = -0.9 k/ NO UPLIFT

F.S. (AGAINST OVERTURNING) = $\frac{138.4\,\pi\,108.5 \times 56.0}{237 \times .667 \times 24{,}220 \times 149.5}$ = 4.62 ok.

FOR LOAD COMBINATION III $C = \frac{1}{0.95}(0.95D + 1.0P + 1.0T + 1.0TL + 1.0E')$

MAX. UPLIFT / FT. CIRCUMFERENCE = 289.2 k/

MIN. UPLIFT / FT. CIRCUMFERENCE = 125.4 k/

F.S. (AGAINST OVERTURNING) = $\frac{117.4\,\pi\,108.5 \times 56.0}{474 \times .667 \times 24{,}220 \times 149.5}$ = 1.96 ok.

FOR 0.2g GROUND MOTION - NO LOSS-OF-COOLANT ACCIDENT

MAX. UPLIFT / FT. CIRCUMFERENCE = 69.2 k/

F.S. (AGAINST OVERTURNING) = $\frac{138.4\,\pi\,108.5 \times 56.0}{474 \times .667 \times 24{,}220 \times 149.5}$ = 2.31 ok

Margin: 274.1 / 223.6 / 50.5 — 283.7 / 212.3 / 71.4 — 71 / 70.1 / 0.9 — 67.4 / 71 / 138.4 — 289.2 / 140.2 / 149.0 — 220 / 71 / 149.0 / 125.6 / 254 — 266.4 / 149.0 / 117.4 — 140.2 / 71 / 69.2

POOR ORIGINAL

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ROCK ANCHORS		

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DEFINITION OF PRESSURE AREA INSIDE CONTAINMENT (PA)

REF: FIGS. 1 & 2 PAGES 11 & 12

FOR ONE RUNNING FT. OF CIRCUMFERENCE AT THE ℄ OF THE CONTAINMENT WALL:

$$\frac{1'}{r_{℄}} = \frac{S_1}{r_i} = \frac{S_2}{r_{℄} - d}$$

WHERE S_1 = ARC LENGTH AT INSIDE FACE OF CONT. WALL

S_2 = ARC LENGTH AT INNER EXTENT OF ROCK WEDGE

$$\therefore\ S_1 = \frac{52.5'(1')}{54.25'} = 0.968'$$

$$S_2 = \frac{27.75'(1')}{54.25'} = 0.512'$$

$$PA = \frac{1}{2}\underbrace{(S_1 + S_2)}_{1.48'}\underbrace{\left(26.5' - \frac{3.5'}{2}\right)}_{24.75'}$$

$$= 18.32\ \text{FT}^2/\text{FT. OF CIRC. AT ℄ OF CONT. WALL}$$

$0.1505\ ^{k}/_{FT^3} \times 4^{FT} \times 18.32\ FT^2 \approx 11^k$

p (PRESSURE INSIDE CONTAINMENT) psi		DOWNWARD FORCE ON ROCK WEDGE, KIPS (INCLUDING 4' OF CONC.)
0.	$\frac{p \times 144\ ^{IN^2}/_{FT^2} \times 18.32^{FT^2}}{1000\ ^{\#}/_{K}} + 11^k =$	11.0
60.		169.3
69.		193.0
75.		208.9
90.		248.4

DETERMINE:
TOTAL ROCK WEIGHT ENGAGED BY ROCK ANCHOR BELOW BASE OF CONTAINMENT MAT. BASE CALCULATIONS ON PER FT. OF CIRCUMFERENCE AT ℄ OF CONTAINMENT WALL.

SUBMERGED WT. OF ROCK = $0.096\ ^{K}/_{FT^3}$
(PG 5.1.2-21 FSAR)

$$\text{ROCK WT.} \simeq \frac{1}{2}(2d)(d) \times 1' \times 0.096\ ^{K}/_{FT^3}$$

$$\simeq 0.096\,d^2 \text{ PER FT. OF CIRC.}$$

(PG 5.1.2-22 FSAR)

$$\simeq 0.096\ ^{K}/_{FT^3}(26.5')^2 = \underline{\underline{67.4\ ^{K}/_{FT.}}}$$

TOTAL DOWNWARD FORCE

p (PSI)					
0.	67.4 K/FT	+	11.0	=	78.4
60.			169.3		236.7
69.			193.0		260.4
75.			208.9		276.3
90.			248.4		315.8

NOTE: THESE VALUES DO NOT AGREE WITH TABLE VALUES IN FSAR, PG 5.1.2-22

DETERMINE:
RESULTANT FORCE AT BASE OF CONTAINMENT DUE TO

$$1.5P - 0.95D$$

$D = 71\ ^{K}/_{FT.}$ (pg C2 OF ORIGINAL CALCS.)

FORCE AT BASE DUE TO PRESSURE

$$= \frac{p\,r_i}{2} = p\left(\#/_{IN^2}\right)\frac{r_i}{2}(FT)\frac{144\ ^{IN^2}/_{FT^2}}{1000\ \#/_K} = KIPS/FT.$$

p	$\frac{p\,r_i}{2}$ (KIPS/FT.)	$-0.95D$ (K/FT.)	= RESULTANT FORCE AT BASE K/FT
0	0	−67.4	−67.4
60	226.8		159.4
69	260.8		193.4
75	283.5		216.1
90	340.2		272.8

CORRESPONDS TO 1.5P − 0.95D

FSAR VALUE ON PG 5.1.2-23 IS 259. K/FT.

CHECK ORIGINAL CALCULATION OF D.

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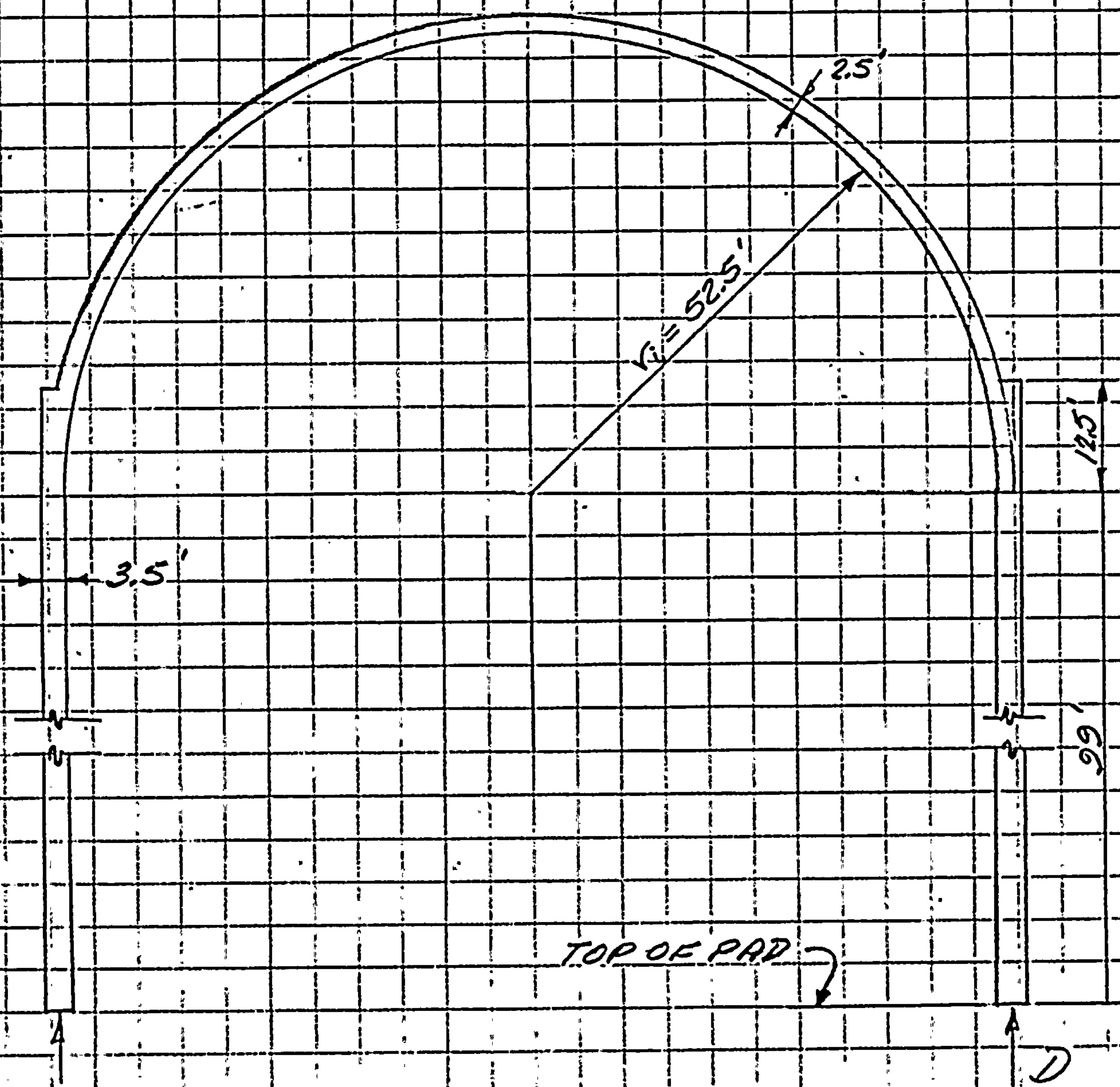
DETERMINE:
CONTAINMENT D IN FEET AT
℄ OF CONTAINMENT BASE AT
ELASTOMERIC PAD ELEVATION



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VOLUME OF CONTAINMENT

$$V_{DOME} = 2\pi r_{\mathrm{C}_{DOME}}^2 t_{DOME}$$

$$= 2\pi (53.75\,FT)^2 (2.5')$$

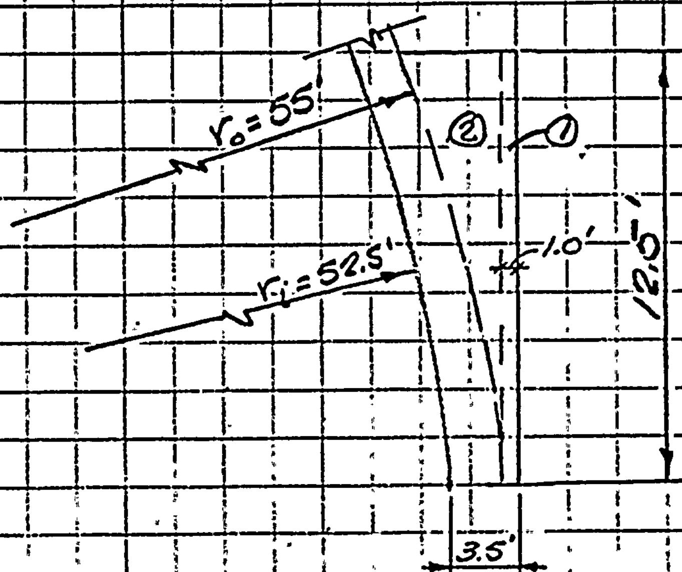
$$= 45{,}380\,FT^3 \quad (VS.\ 45{,}500\,FT^3\ ORIG.\ CALCS.)$$

$$V_{CYL} = 2\pi r_{\mathrm{C}_{CYL}}\ 99'\ t_{CYL}$$

$$= 2\pi (54.25\,FT)(99')(3.5')$$

$$= 118{,}109\,FT^3 \quad (VS.\ 116{,}000\,FT^3)$$

RING GIRDER



$$\begin{array}{r} 52.5 \\ +\ 3.5 \\ \hline 56.0 \\ -\ .5 \\ \hline 55.5' \end{array}$$

$$V_{ORG} = 1'(12.5')\,2\pi\,(55.5')$$

$$= 4359.\,FT^3$$

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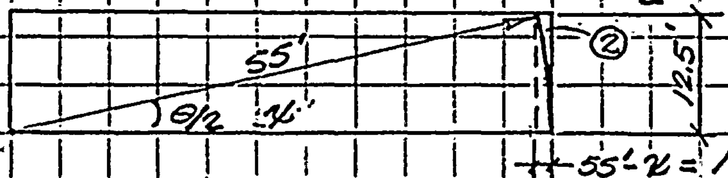
$$\theta/2 = \sin^{-1}\frac{12.5}{55} \quad \therefore \sin(\theta/2) = 0.227$$

$$\& \ \theta/2 = 13.14°$$



$$r = \sqrt{(55)^2 - (12.5)^2} = 53.561'$$

$$A_{②} = \underset{17.99\,FT^2}{(1.439)(12.5')} - \frac{1}{2}\left[\underbrace{\frac{\pi(55')^2\overset{(26.28°)}{\theta}}{360°} - \frac{(55')^2\overset{0.443}{\sin\theta}}{2}}_{23.70\,FT^2}\right]$$

$$= 6.14\ FT^2$$

ASSUME C.G. OF $A_{②}$ AT $\simeq 53.56 + \frac{2}{3}(1.439)$
$= 54.52'$

$$V_{②RG} = 6.14\ FT^2 \times 2\pi(54.52)$$
$$= 2103\ FT^3$$

$$V_{RG} = V_{①RG} + V_{②RG} = 4359 + 2103$$
$$= 6462\ FT^3$$

$$\left.\begin{array}{l}\text{TOTAL}\\ \text{VOL. CONT.}\end{array}\right\} = \left.\begin{array}{r}45380\\118109\\6462\end{array}\right\} \simeq 170{,}000\ FT^3$$

$$D\ (K/FT) = \frac{170{,}000\ FT^3 \times .150\ K/FT^3}{2\pi(54.25')}$$

$$= 74.8\ K/FT \simeq \text{ORIGINAL CALC.}$$

DETERMINE:
WEIGHT OF 3/8" STEEL LINER
AT ℄ OF CONTAINMENT BASE

$$Vol_{DOME} = 2\pi r_i^2 t_{LINER}$$

$$= 2\pi (52.5')^2 \frac{3}{8}'' \frac{1}{12''/FT}$$

$$= 541 \ FT^3$$

$$Vol_{CYL.} = 2\pi r_i \, 99' \, t_{LINER}$$

$$= 2\pi (52.5') \, 99' \, \frac{3}{8}'' \frac{1}{12''/FT}$$

$$= 1021 \ FT^3$$

$$D_{LINER} = \frac{1562 \ FT^3 \times 0.49 \ K/FT^3}{2\pi (54.25')}$$

$$= 2.2 \ K/FT.$$

TOTAL D = 74.8
+2.2
77.0

.95D = 73.1 K/'

USE CURRENT CALCS. TO COMPARE TO FSAR INFO.

P (psi)	UPLIFT FORCE (KIPS) USING NEW D & $\frac{P r_i}{2}$ FROM PG 3	RESISTING FORCE (KIPS) FROM PG 8 2	RATIO (IE, F. OF S.)
0	-73.1 K/FT	78.4	—
60.	153.7	236.7	1.54
69.	187.7	260.4	1.39
75.	210.4	276.3	1.31
90.	267.1	315.8	1.18

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COMPARISONS OF CURRENT CALCS USING FSAR ASSUMPTIONS WITH FSAR VALUES

ITEM	CURRENT CALCS.	FSAR VALUE	FSAR PG.
UPLIFT FOR L.C.(a) IN FSAR SEC. 5.1.2.3 (Load Combination)	267.1 K/FT	259. K/FT	(5.1.2-23
ANCHOR CAP. FOR L.C.(a)	(444.6)* 315.8 K/FT	327. K/FT	"
F. OF S. FOR L.C. (a)	[1.66]** 1.18	1.26	"
UPLIFT FOR SIT, IE; 69 PSI	187.7 K/FT	182. K/FT	"
ANCHOR CAP. FOR SIT	(375.5)* 260.4 K/FT	266.4 K/FT	"
F. OF S. FOR SIT	[2.00]** 1.39	1.47	

* SEE PG. 10 FOR VALUES IN PARENTHESES

** BASED ON PARENTHETICAL VALUE

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DATE 11/8/80				[illegible]

RECALCULATE RESISTING FORCES ASSUMING THE ROCK IS ENGAGED AT THE BOTTOM ANCHOR HEAD.

$\therefore d \simeq \begin{array}{r}26.5\\+11.0\end{array} = 37.5^{FT}$

$PA \simeq \frac{1}{2}(S_1 + S_2)(37.5 - \frac{3.5}{2})$ SEE PG 1

$S_1 = 0.968'$ NO CHANGE

$S_2 = \frac{16.75'(1')}{54.25'} = 0.309'$

$\therefore PA = \frac{1}{2}(0.968' + 0.309')(35.75')$

$= 22.83\ FT^2/FT$ OF CIRC.

WT. OF 4' {MAT & FILL CONC} $= 22.83 FT^2/FT \times 4^{FT} \times .150^{K}/_{FT^3}$

$= 13.7^K$

p. (PSI)	$\frac{p \times 144\ IN^2/FT^2}{1000\ \#/K} \times 22.83\ FT^2/FT + 13.7^K$ = RESISTING FORCE ON ROCK WEDGE
0	$13.7^K/FT$
60	211
69	240.5
75	260.3
90	309.6

$$ROCK\ WT. = 0.096\ {}^{K}/_{FT^3}\ (37.5^{FT})^2$$ SEE PG. 2

$$= 135\ {}^{K}/FT.\ OF\ CIRC.$$

TOTAL RESISTING FORCE = INTERIOR FORCE ON ROCK WEDGE + ROCK WT.

φ (PSI)	TOTAL RESIS. FORCE (K/FT)	UPLIFT PG. 7 (K/FT)	RATIO IE, F.S.
0.	148.7	-73.1	—
60.	346.	153.7	2.25
69.	375.5	187.7	2.00
75.	395.3	210.4	1.88
90.	444.6	267.1	1.66

NONE OF THE ORIGINAL OR CURRENT CALCS. INCLUDES INTERIOR STRUCTURE DEAD WEIGHT IN THE RESISTING FORCE

$$\begin{array}{r} 135 \\ 309 \\ \hline 444.6 \end{array}$$

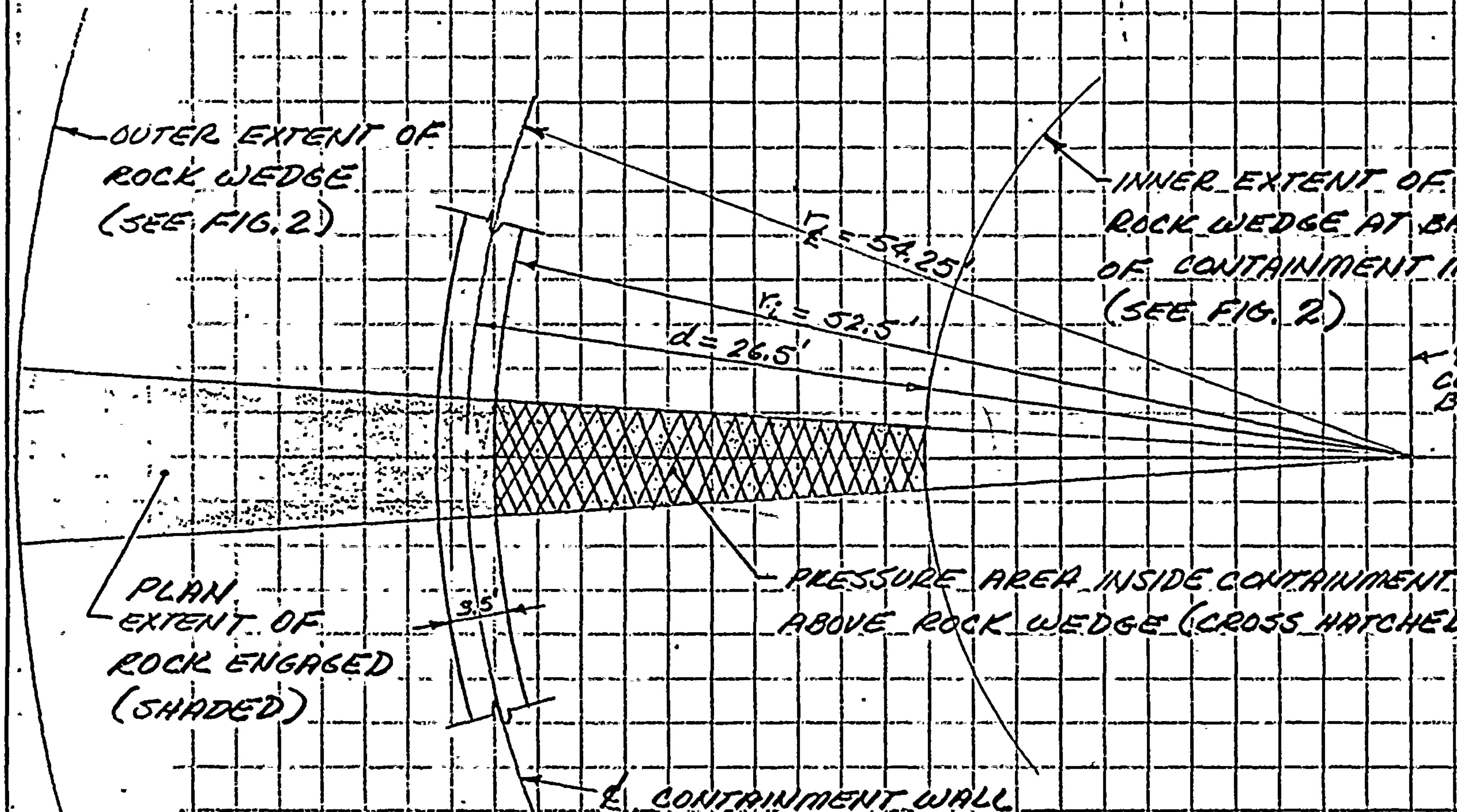

FIGURE 1 - PLAN VIEW OF ROCK AND PRESSURE AREA

SUBJECT				
REV.	0	1	2	3
MICROFILMED				
ORIGINATOR	FJH			
DATE	11/18/80			

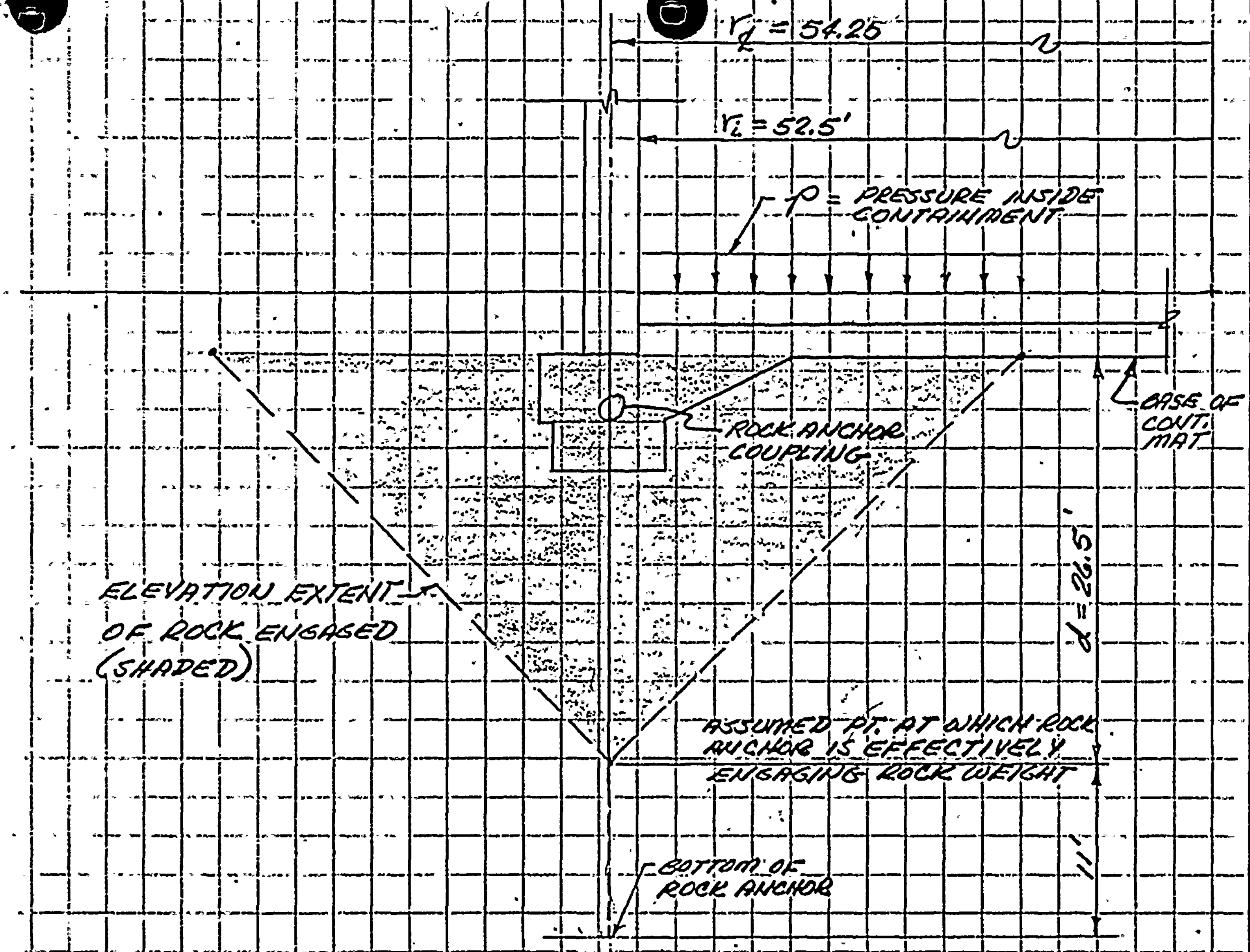

FIGURE 2 – ELEVATION OF ROCK AND PRESSURE AREA

POOR ORIGINAL

Robert E. Ginna
Nuclear Power Station
Ontario - New York

ROCK ANCHOR # 46

HOLE 18

Analysis of Stresses
and Elongations

(According to measurements
made on Aug. 26 1966.)

POOR ORIGINAL

ROCK ANCHOR # 10 - Hole 16

90 WIRES ϕ 1/4"

Differences in length of wires (Measurement Aug. 26 [illegible])

Within 1/16" (0.0625") ⟶ 65 wires ⟶ A_s = 3.190
2/16" (0.1250") ⟶ 15 " 0.735
3/16" (0.1875") ⟶ 7 " 0.[illegible]
7/16" (0.4375") ⟶ 1 " 0.049
8/16" (0.5000") ⟶ 2 " 0.098

ΣA_s = 4.415 in²

E = 29 200 000 psi

Free length of the tendon (5 Different Assumptions)

l = 11.5 — 17 — 22.5 — 28.[illegible] — [illegible]

The first 65 wires are assumed to carry linearly up to [illegible]

Operations	l = 11.5	17.0	22.5	28.0	[illegible]
$\epsilon_1 = \frac{0.0625}{l}$	2.28 × 10⁻⁴	1.[illegible] × 10⁻⁴	1.16 × 10⁻⁴	0.93 × 10⁻⁴	0.7[illegible]
$F_1 = A_1 \cdot E \cdot 3.190$	21.1 K	14.7 K	10.[illegible] K	[illegible] K	[illegible]
$\epsilon_2 = \frac{0.0625}{l}$	4.53 × 10⁻⁴	3.06 × 10⁻⁴	2.32 × 10⁻⁴	1.86 × 10⁻⁴	1.5[illegible]
$\Delta F_2 = \epsilon_2 \cdot E \cdot 3.190$	49.2 K	2[illegible].5 K	21.6 K	17.3 K	14[illegible]
$F_2 = F_1 + \Delta F_2$	63.3 K	43.[illegible] K	31.[illegible] K	26.0 K	2[illegible]
$\epsilon_3 = \frac{0.0625}{l}$	4.53 × 10⁻⁴	3.06 × 10⁻⁴	2.32 × 10⁻⁴	1.86 × 10⁻⁴	1.5[illegible]
$\Delta F_3 = \epsilon_3 \cdot E \cdot 3.925$	51.4 K	35.[illegible] K	26.6 K	[illegible]	[illegible]
$F_3 = F_2 + \Delta F_3$	115.2 K	7[illegible] K	57.[illegible] K	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
[illegible]	[illegible] K	[illegible]	[illegible] K	[illegible]	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

POOR ORIGINAL

[illegible] AT [illegible]

Assuming [illegible]

[illegible] P_E = [illegible]

[illegible] P_E = [illegible]

Operation [illegible]	11.6	17.6	22.5	27.6	32.[illegible]
P_E	378.4 K	269.3 K	205.4 K	163.5 K	137.[illegible]
[illegible]	345.6 K	472.7 K	528.5 K	578.5 K	624.2 K
Δ = [illegible]	0.369"	0.748"	1.128"	1.507"	[illegible]
[illegible]	0.768"	1.248"	0.688"	2.0[illegible]1"	[illegible]
[illegible]	1146.6 K	578.2 K	499.6 K	474.5 K	110[illegible]
[illegible]	0.481"	0.915"	1.350"	1.782"	[illegible]
[illegible]	0.981"	1.415"	1.850"	2.282"	[illegible]

[illegible]

[illegible]

POOR ORIGINAL

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Operations:	11.5'	17.0'	[illegible]	26.0'	33.5'
wire with Δ = [illegible] 6'	114.9 ksi	[illegible]	[illegible] ksi	160.2 ksi	165.2 ksi
" " Δ = 2 1/6'	101.7 "	[illegible]	[illegible]	[illegible] "	160.[illegible] "

CONCLUSIONS :

— The difference in length [illegible] calculation tolerance [illegible] analysed Rock Anchor [illegible] from 0.70 UTS ([illegible] ksi) to [illegible] in the shortest wire. This [illegible] condition of free length (only [illegible]), and a [illegible] modulus of elasticity [illegible] increase during the stressing [illegible] ces the highest stress [illegible] ([illegible] ksi for l = 22.5 ft.) The influence of the curvature of the σ – ε Diagram of the wire is [illegible] this stress level.

— At overstressing stress (0.80 UTS) the maximum stress in the shortest wire reaches 207.6 ksi (0.[illegible] UTS.) This is also for the shortest free length of 11.5'. For a free length of 22.5' the stress reduces to 200 ksi. Here the curvature of the stress - strain diagram of the wire has a sensible influence resulting in a decrease of the calculated max. stresses.

— The difference in elongation of the tendon is small, for 0.7 UTS. and l = 22.5' it is:

$$\Delta l = 1.528'$$
$$168 \times \frac{22.5 \times 12}{[illegible] 200} = 1.552$$
$$\Delta\ elong. = 0.024'$$

AMERICAN BBR INC.
89TH AND ROCKWELL STS.
CHICAGO 8, ILLINOIS

Chicago
[illegible]

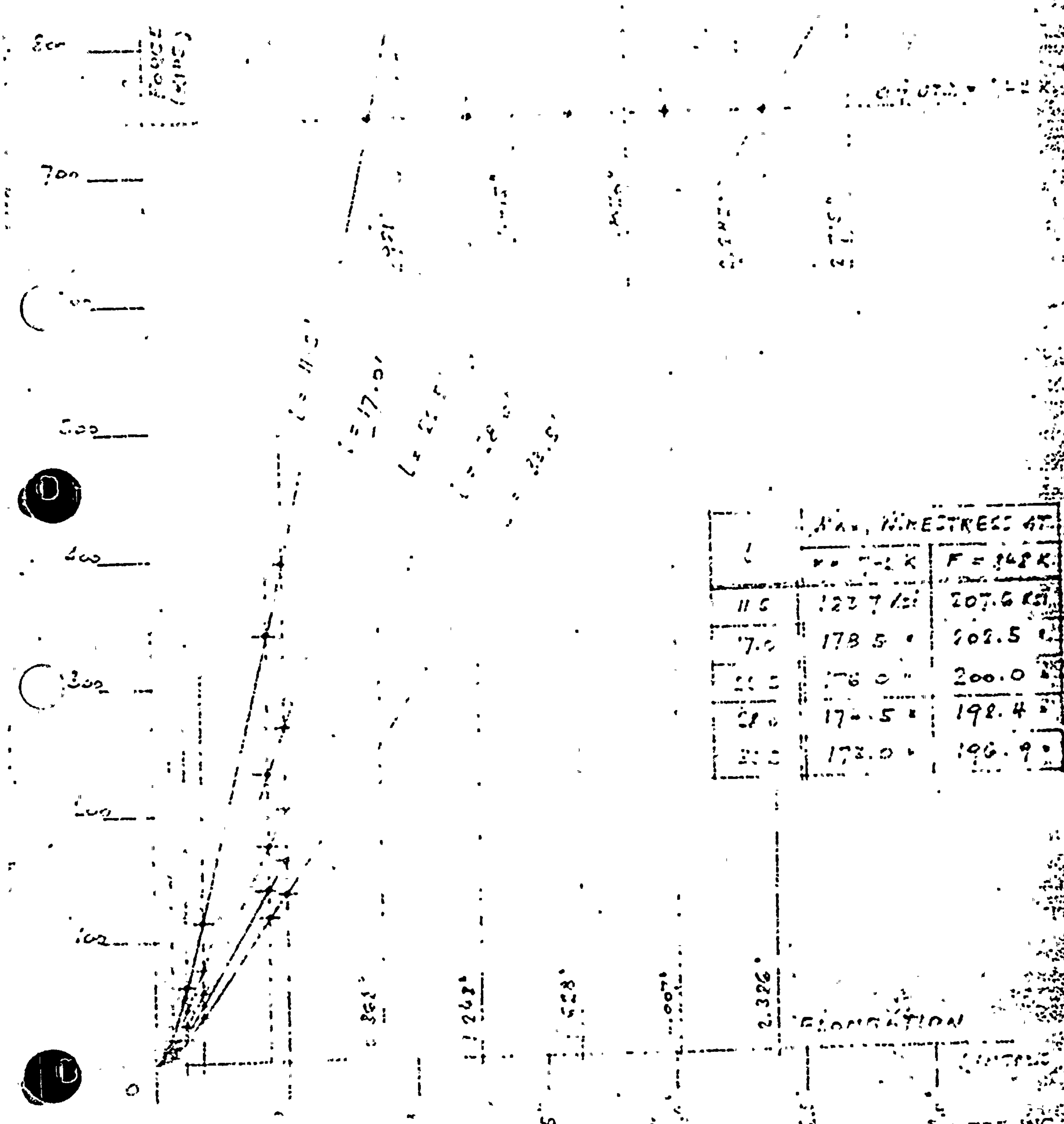
POOR ORIGINAL

Rock Anchor # 46

Hole 1E

FORCE - ELONGATION DIAGRAM

(90 WIRES ⌀ 1/4")



ℓ	Max. Wirestress at	
	F = ..42 K	F = 848 K
11.5	123.7 ksi	207.6 ksi
17.0	178.5 "	202.5 "
22.0	176.0 "	200.0 "
28.0	174.5 "	198.4 "
33.0	173.0 "	196.9 "

ELONGATION

E = const. = 29 200 000 psi

AMERICAN BBR INC.

Mr. E. U. Powell, ...
Cinn. Project
Westinghouse Atomic Power ...
P. O. Box 355
Pittsburgh, Pennsylvania ...

Re: ...

Dear Mr. Powell:

Reference is made to ... letter to ... dated September 20, 1966 which provided an analysis of the effect of mis-length wires in Rock Anchor #6 based upon the data obtained during the tensioning operation which was performed on September 6, 1966.

The following are comments regarding this analysis:

1. The unbonded length of wire is assumed to be uniform for all wires and is established by a trial and error solution. This assumption is conservative but is the only one that would logically be made with the available data, since bond slippage is a function of wire stresses, the higher stressed wires would in all probability have a greater free length than the lower stressed wires. This would tend to produce a more uniform distribution of load among the wires.

2. The proportional limit, as defined by the stress-strain curves for this material, was passed during the tensioning operation. This fact was considered in the analysis.

3. The analysis assumed no slack in the wires prior to tensioning.

The data have been reviewed and do substantiate the fact that the maximum wire stresses both during jacking and initially when shimmed in place are essentially within the limits of the specification (i.e. 0.80 fu and 0.70 fu respectively).

Six copies are attached of an analysis which we had performed immediately after the test and before the material data were made available. This analysis also provides calculations to determine normal and maximum shim heights. This latter data was requested by your letter EWS-400 dated September 22, 1966.

POOR ORIGINAL

Mr. E. U. Powell - BOK-1041
December 20, 1966
Page 2

The test also included determining the lift-off force after a 24 hour delay. This jacking operation provided information on initial prestress losses. By copy of this letter Ryerson is requested to submit this data for incorporation with other test results.

Very truly yours,

D. K. Cronebarger
bac

D. K. Cronebarger
Structural Engineer

DKC:bac
Enclosures

cc: E. U. Powell (5)
R. M. Luken
H. A. Parzick
T. M. Brown

	ACT.	INF.	SIG.	DATE
PROJ. SUPT.			[illegible]	[illegible]
CIVIL DEPT.		X	[illegible]	[illegible]
ELECT. DEPT.				
MECH. DEPT.				
PIPING DEPT.				
COST DEPT.		X	[illegible]	[illegible]
SCHE. DEPT.		X	[illegible]	[illegible]
ADM. DEPT.				
J. G. [illegible]		X	[illegible]	1/10/66
FILE [illegible]				

XC: To Hole #46 Folder

POOR ORIGINAL

227.67 ELEV.

Sept. 6, 1966

Initial Tensioning

9-6-66 Max. Force = 743K Max. Hydraulic Pressure = 5750 psi

Time	Required Load (kip)	Required Hydraulic Gage (psi)	Actual Load (kip)	Actual Hydraulic Gage (psi)	Dial Gage (inches)	Total Elongation (inches)	PL Elevation	Remarks	
10 AM	0	0					10.43 10.44 ←	227.68 227.67	
10:20	100	775		775	.132"				5/16
10:25	200	1550		1550			10.43 10.44		9/16
10:30	300	2320		2320	.388 .001 10:35				9/16
1035	400	3100		3100	.531 .001				3/4
10:40	500	3870		3870	.6715		10-43 10-44		7/8
10:45	600	4650		4650	.831 .061				1"
10:50	700	5420		5350	.978 .061	.185	10-435 10-445		1 1/8"
10:55	743	5750		5756	1.004				1 3/16
11:00	700	5420		5420	1.065				1 1/8
11:05	600	4650		4650	[illegible]	0.982			1 1/16
11:10	500	3870		3870	.819	.755			1"
11:15	400	3100		3100	.685	.520	10.438 10.495		7/8
11:20	300	2320		2320	.548	.484			3/4
11:25	200	1550		1550	.399	.334	10.438 10.445		9/16
11:30	100	775		775	.337	.269			
11:35	0	0		0			10. 10.		

RAM AREA : 129.3 in²

9-6-66

FULL TENSION

TIME	REQ'D. HYDRAUL GAUGE (psi)	ACTUAL LOAD	ACTUAL HYDRAUL. GAUGE LOW	ACTUAL HYDRAUL. GAUGE HIGH	DIAL GAUGE (IN)	TOTAL ELONGAT. (IN)	PLATE ELEVAT.	REMARKS
1^{30}	0				.000			
	25				.002		10.48 N 10.48 S	
1^{35}	50				.008			
	75				.014			
1^{40}	100				.018			
	150				.028			
1^{45}	200				.038			
	250				.047			
1^{50}	300				.057			
— CHANGE GAUGE —	500				.093			
2^{00}	700	✓	✓	✓	.116			
	1100				.185			
2^{05}	1500				.262		10.475 10.48	227.675 227.67
	1900				.331			
2^{10}	2300				.405			
	2700				.480			
2^{15}	3100				.559		10.48 N 10.4835	227.67 227.667
	3500				.627			
2^{20}	3900				.700			
	4300				.774			

Pile 18 - Anchor # 46 - 9-6-66 $\frac{9}{4}$ $\frac{9.9}{1.162}$

Time	Ram Hydraulic Gage (PSI)	Actual Load	Hydraulic Gage Low	Hydraulic Gage High	Dial Gage (inches)	Total Elongation (in.)	R. El.	Remarks	R.A.
2^{25}	4700				.843		10.48 10.485	227.67 227.665	
	4900				.8825				
2^{30}	5100.				.914				
	5300		5280		.9525	.16			
2^{35}	5500		5425		.989		10.475 10.483	227.675 227.667	
	5900				1.067				
2^{40}	6100				1.114				
2^{45}	6300				1.162				
2^{50}	6570				1.234		10.48 10.45	227.67 227.665	1 1/2
3^{00}	6300	6,900							
	6100				1.222				
	5900				1.198				
	5500	5750							
	5300								
	5100								
3^{05}	4900	4800					10.485 10.48	LIFT OFF SHIMS LOOSE 227.675 227.67	1 3/[illegible]
	4700	6500							
	4300	4200					"		
3^{15}	3900	5650	LIFT OFF [illegible] SHIMS						1 3/[illegible]
	3500								
	3100								
	2700								

ROCK ANCHORS By John Gilbert   Friday - 8-26-66 - 1. . . A.M.

SUBJECT: Test on Rock Anchor Number 46, Hole Number 18

Equipment Used: One 50 ton jack w/18" dia. jacket resting on rock anchor base plate. Hydraulic Gages - Two in parallel had a maximum reading of 10000 lbs/sq. inch. - Computed area of ram = 13.29 square inch.

Witnesses: Jack Edgren, Bill Wilkinson, Ted Brown - J. T. Ryerson & Sons
Ross M. Luken, John Gilbert - Bechtel Corporation

Initial Test - 10 A.M.

Buttonheads were checked from top of button to top of anchor head. Average height on approximately 80 tendons was 5/16" minus 1/16". About five tendons were within tolerance - 5/16" above anchor head. The remaining five tendons were marked w/yellow chalk to indicate any subsequent movement. Measurements and location were taken thus:

1 - N. W. = Second row - 11/16"
2 - N. E. = First row - 11/16"
3 - S. E. = First row - 1/2"
4 - S. W. = Fifth row - 7/16"
5 - S. W. = Second row - 3/8"

Pressure was applied at 11 A.M. - Gage reading 5,000 lbs/sq. inch when 1/16" shims were located under anchor head = 34 Tons.
Approximately twenty tendons were loose - prising with a screwdriver.

Second Stage - 1:45 P.M.

Pressure applied to loosen 1/16" shims = 2,100 lbs/sq.inch = 14 ton.
1/8" shims under anchor - Pressure 8,400 lbs/sq. inch = 56 ton

1 = 5/8"
2 = 5/8"
3 = 7/16"
4 = 3/8"
5 = 5/16"

Approximately 15 tendons were loose at this stage.

Third Stage:

Pressure applied to loosen 1/8" shims. 4,100 lbs/sq.inch = 28 ton.
Efforts to install 1/16" shims with 1/8" shims were partially successful . Pressure applied 9,500 lbs/sq.inch = 64 tons. Measurements on buttonheads were:

1 = 9/16"
2 = 7/16"
3 = 1/2"
4 = 5/16"
5 = 1/4"

Approximately ten tendons were loose - screwdriver used.

Further tests were not conclusive as 3/16" shims were not aligned at true level.

Removed 1/8" shims - left 1/16" shims under anchor when Ted Brown intimated conclusion.
4:45 P.M.

John Gilbert

P.O. BOX 157 August 25, 1966 ONTARIO CENTER, N.Y.

Mr. E. U. Powell, Project Manager
Ginna Nuclear Plant
Westinghouse Atomic Power Division
P.O. Box 355
Pittsburgh, Pennsylvania 15230

BBW-617
RE: R.E. Ginna Nuclear Plant
Unit #1
Prestressed Rock Anchors

Gentlemen:

We are transmitting, herewith, one (1) copy each of the following data:

1. Depth of holes
2. Placing sequence of tendons
3. Tensioning sequence of tendons

This information was requested by Mr. W. Berg of GAI on August 19, 1966, for evaluation by his organization.

Sincerely yours,

BECHTEL CORPORATION

R. M. LUKEN

RML:cs
Enclosures

CC: W. Berg
T. Brown
K. T. Momose
H. Parzick
J. Stull / J. Shryock

POOR ORIGINAL

POOR ORIGINAL

REPORT ON JOB PROGRESS

POST TENSIONING DIVISION

PL[redacted]

JOSEPH T. RYERSON & SON, INC.

Report Number 27

Job BECHTEL CORP. 21T341591

Plant

Date 8/22/66

Weather:

	Number Of Men	T & M Hours		Number Of Men	T & M Hours

REMARKS — Progress of Work

PLACING SEQUENCE: PER INSTRUCTIONS FROM
HANS LORENZER AT JOBSITE MEETING)
PLACE ANCHORS AND GROUT IN TWO HOLES
WITH VISIBLY FLOWING WATER (92 & 113)
IMMEDIATELY PROCEED TO PLACE & GROUT
HOLES IN DRIEST SECTION UNTIL ANCHORS IN
92 & 113 CAN BE STRESSED.
AFTER 92 & 113 ARE STRESSED SUCCESSFULLY,
CONTINUE PLACING AND GROUTING ALL HOLES.
NO SEQUENCE SPECIFIED

STRESSING SEQUENCE:
1ST HOLES 92 & 113
2ND PER SPECIFICATIONS
STRESS EVERY FOURTH ROCK ANCHOR.
STRESS ANOTHER QUARTER - EQUAL SPACING
STRESS REMAINING HALF.

Superintendent [signature]

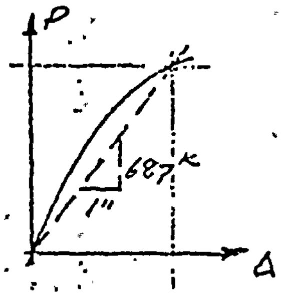
BEAVER VALLEY UNIT No. 1 CONTAINMENT VESSEL	READING PENNA	GILBERT ASSOCIATES INC ENGINEERS AND CONSULTANTS	NEW YORK WASHINGTON
ROCK ANCHOR TEST	MADE: [illegible] CHKD: SQ CF: CF DN: APPD:	SIZE: DRAWING NO.: REV: W.O. [illegible] DATE [illegible]	SHEET 1

TENDON #46 IN HOLE #18

1. Free length of tendon.

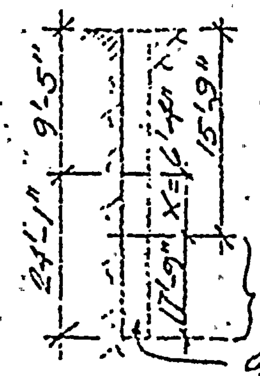
The slope of the load-deformation curve will indicate the amount of bond slipage.
For tendon in hole #18 the slope was $848/1.234 = 687\ ^K/in$

$$\Delta = \frac{PL}{AE} \; ; \; L = \frac{\Delta AE}{P} = \frac{1(4.41)(29{,}500)}{687} = 189'' = \underline{15'\text{-}9''}$$



2. Bond slipage

Total bond slipage = 6'-1"



3. Wire stresses

Max tendon elongation at 0.8(ULTS) was $\Delta = 1.234$ in.

$$\frac{P}{A} = \frac{\Delta E}{L} = \Delta \frac{29{,}500}{189} = 156$$

Diff. in wire length.	Total deform.	0.8(ULTS) = 156 Δ	0.7 ULTS
65 wires within 1/16"	Δ = 1.234 in	192.5 ksi	168.5 ksi
15 " +1/8" too long	Δ = 1.234 − 0.125 = 1.109 in	173. ksi	151.5 ksi
7 " +3/16 " "	Δ = 1.234 − 0.188 = 1.046 in	163. ksi	142.5 ksi
1 " +7/16 " "	Δ = 1.234 − 0.438 = 0.796 in	124. ksi	108.5 ksi
2 " +1/2 " "	Δ = 1.234 − 0.5 = 0.734 in	114.5 ksi	105.0 ksi

Max req'd ultimate strength for tendon in hole #18;

$$ULTS = \frac{192.5}{0.8} = 241\ ksi.$$

Previous tests run on the wires from the same reel as used in tendon #46 (hole #18) indicate an ultimate strength of 244 and 250 ksi.
Therefore tendon #46 in hole #18 meets the code requirements and is acceptable.
Note: Max tendon stress variation as a result of the shim thickness = 1/32"; $\frac{P}{A} = \pm \frac{29{,}500}{32(189)} = \pm 5.0$ ksi.

[illegible]WOOD UNIT NO. 1
CONTAINMENT VESSEL

ROCK ANCHOR TEST

READING PENNA	GILBERT ASSOCIATES INC ENGINEERS AND CONSULTANTS	NEW YORK WASHINGTON

MADE	CHKD	SO CF	CF DN	APPD	SIZE DRAWING NO. REV.	SHEET
KEN					W. O. [illegible] DATE [illegible]	2
REV.						

TENDON # 61 IN HOLE # 113

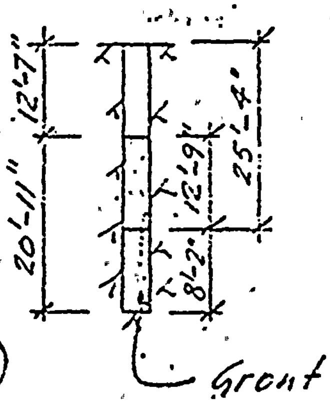
1. Free length of tendon

$$K = P/\Delta = \frac{6350(129.3)}{1927} = 427^{K}/in$$

$$L = \frac{\Delta A E}{P} = \frac{4.41(29{,}500)}{427} = 304'' = 25'\text{-}4''$$

2. Bond slipage

Total bond slipage = 12'-9"



3. Max wire stress

Max. tendon elongation at 0.8 (ULTS) was

$$\Delta = 1.927\left(\frac{6558}{6350}\right) = 1.99\,in$$

Max wire stress $\frac{P}{A} = \frac{\Delta E}{L} = \frac{1.99(29{,}500)}{304} = 193\,ksi \approx 192\,ksi$

O.K.

4. Shims

$t = 1\frac{13}{16}''$

Lift off $1\frac{13}{16}''$ shim @ 5850 psi, 5740 psi req'd

O.K.

POOR ORIGINAL

[illegible] U. + No. 1 CONTAINMENT VESSEL	READING PENNA	GILBERT ASSOCIATES INC ENGINEERS AND CONSULTANTS				NEW YORK WASHINGTON	
	MADE	CHKD	SO CF	CF DN	APPD	SIZE DRAWING NO. REV	SHEET
	KEH						3
						W.O. [illegible] DATE [illegible]	
ROCK ANCHOR TEST	REV.						

TENDON #58 IN HOLE #92

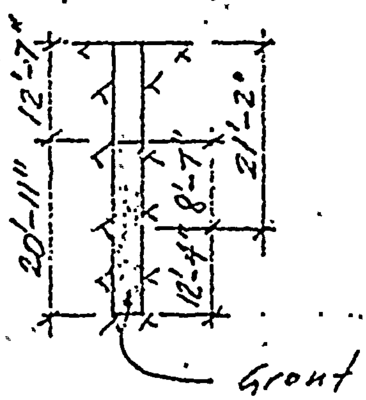
1. Free length of tendon

$$K = \frac{6550(129.3)}{1.654} = 512\ ^{K}/in$$

$$L = \frac{\Delta AE}{P} = \frac{AE}{K} = \frac{4.41(29{,}500)}{512} = 254'' = \underline{21'-2''}$$

2. Bond slipage

Total bond slipage = 8'-7"



3. Max wire stress

Max. tendon elongation at 0.8 (ULTS) was $\Delta = 1.654''$ @ 847^K

Max wire stress : $\frac{P}{A} = \frac{1.654(29{,}500)}{254} = \underline{192\ ksi}$ O.K.

4. Shims

$t = \frac{3}{4} + \frac{1}{2} + \frac{1}{8} = \underline{1\frac{3}{8}''}$

Lift off $1\frac{3}{8}''$ shim @ 5675 psi , 5740 psi req'd O.K.

POOR ORIGINAL

BROOKWOOD UNIT No. 1
CONTAINMENT VESSEL

ROCK ANCHOR TEST

READING PENNA | GILBERT ASSOCIATES INC | NEW YORK WASHINGTON
ENGINEERS AND CONSULTANTS

MADE	CHKD	SQ CF	CF DN	APPD	SIZE, DRAWING NO. REV	SHEET
KEN						4

W.O. — DATE —

REV.

TENDON # 64 IN HOLE # 10

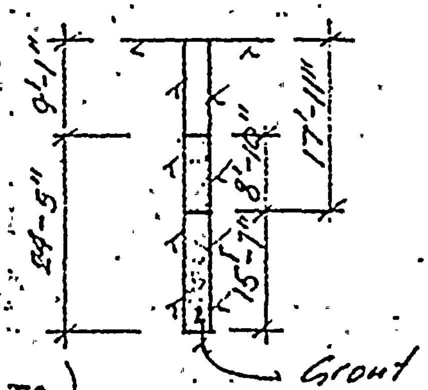
1. Free length of tendon

$$K = \frac{849}{1.404} = \underline{605\ K/in}$$

$$L = \frac{AE}{K} = \frac{4.41(29,500)}{605} = 215'' = \underline{17'-11''}$$

2. Bond slipage

Total bond slipage =



3. Max. wire stress

Max. tendon elongation at 0.8(ULTS) was $\Delta = 1.404$ in @ 849^K

Max wire stress $\frac{P}{A} = \frac{1.404(29,500)}{215} = \underline{193\ ksi \approx 192\ ksi\ o.k.}$

4. Shims

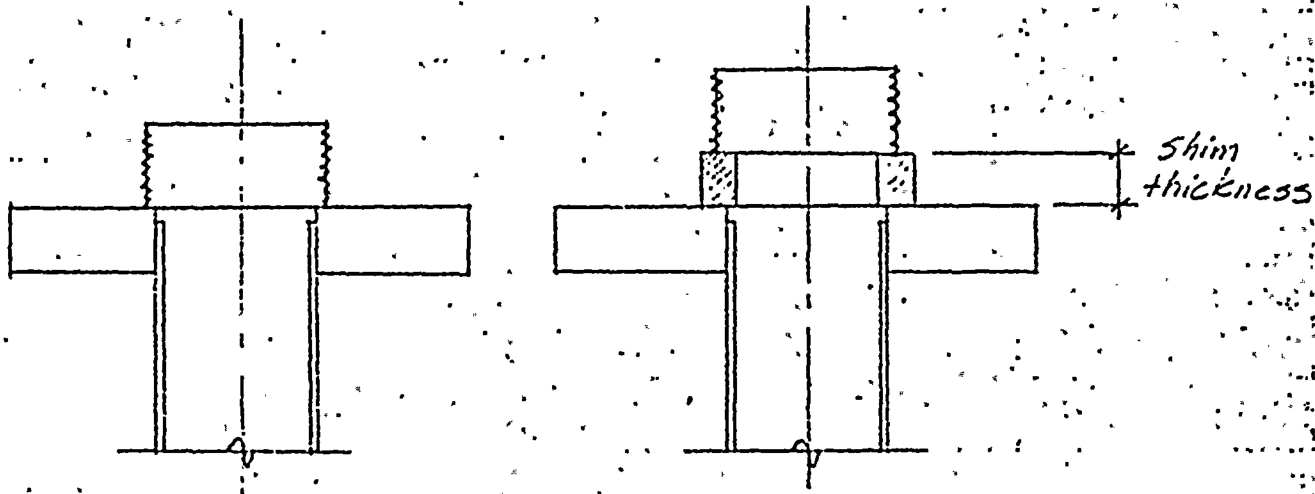
$t = \frac{3}{4} + \frac{1}{4} + \frac{1}{8} + \frac{1}{16} = 1\frac{3}{16}''$

Lift off $1\frac{3}{16}''$ shim @ 6000 psi, 4.5% over 5740 psi

POOR ORIGINAL

[illegible] UNIT No. 1	READING PENNA	GILBERT ASSOCIATES INC ENGINEERS AND CONSULTANTS	NEW YORK WASHINGTON
CONTAINMENT VESSEL	MADE [illegible]	SIZE DRAWING NO. REV	SHEET 5
ROCK ANCHOR TEST	REV.	W.O. [illegible]	DATE [illegible]

SHIMS



Rock Anchor as placed

Rock anchor stressed

Tendon elongation at lock off stress $0.7(ULTS) = \underline{168 \text{ ksi}}$

a) No bond slipage $L = 11.5' = 138''$ slack neglected

$$\Delta = \frac{PL}{AE} = \frac{168(138)}{29{,}500} = 138(5.7)(10^{-3}) = 0.785'' \qquad \underline{\Delta = 13/16''}$$

b) Normal bond slipage $L = 11.5 + 7.5 = 19.0 = 228$ in

$\Delta = 228(5.7)(10^{-3}) = 1.3''$

slack neglected

shims: $\underline{\Delta = 1\,5/16''}$

c) Total bond slipage $L = 33.5' = 420$ in

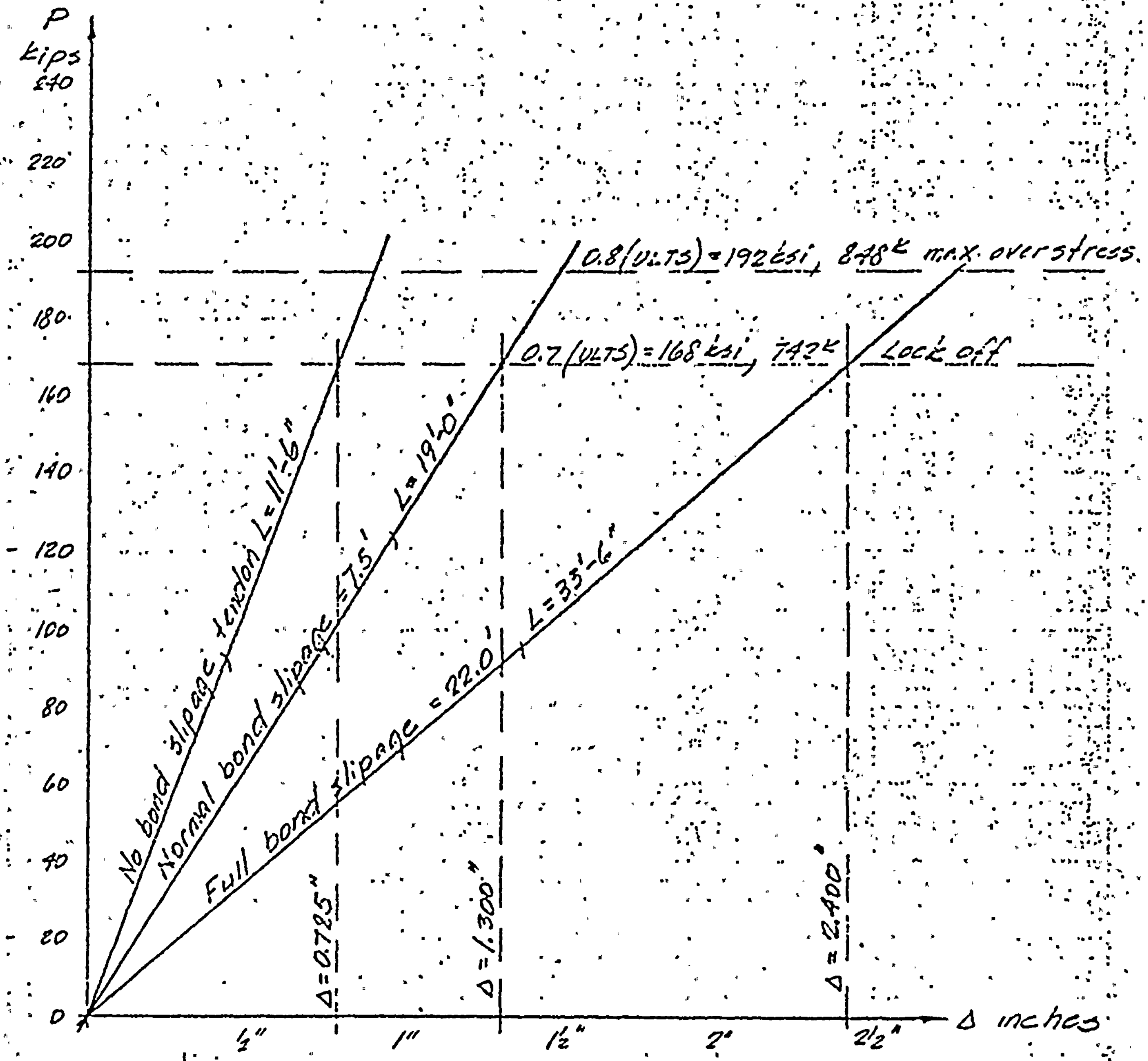
$\Delta = 420(5.7)(10^{-3}) = 2.4''$

slack neglected

Shims: $\underline{\Delta = 2\,3/8''}$

POOR ORIGINAL

BROOKWOOD UNIT No. 1 CONTAINMENT VESSEL	READING PENNA	GILBERT ASSOCIATES INC ENGINEERS AND CONSULTANTS	NEW YORK WASHINGTON
ROCK ANCHOR TEST	MADE: KEN	W.O.	SHEET 6



LOAD - ELONGATION CURVE
for ROCK ANCHORS
90 wire tendon

POOR ORIGINAL

INSTALLATION OF ROCK ANCHORS 10-20-66

16 EQUALLY SPACED ELEVATIONS WERE TAKEN PRIOR TO TENSIONING OF ROCK ANCHORS – A SECOND READING WAS TAKEN AFTER TENSIONING. RESULTS APPEAR BELOW

	8-16-66	10-20-66	
HOLE No.	1ST READING	2ND READING	
3	227.66'	227.66'	
13	227.66'	227.64"	
23	227.67'	227.66'	
32	227.70'	227.68'	
42	227.70'	227.70	
52	227.71'	227.67'	
62	227.71'	227.69'	
72	227.72'	227.68'	
82	227.72'	227.68'	
92	227.71'	227.67'	
102	227.71'	227.69'	
112	227.68'	227.66'	
122	227.67'	227.65'	
132	227.68'	227.68'	
142	227.70'	227.69'	
152	227.70'	227.68'	

POOR ORIGINAL

ROCK ANCHORS 10-21-66

FOUR ROCK ANCHORS WERE GIVEN A SECOND LIFT-OFF - THE READINGS WERE RECORDED AS FOLLOWS. GAGE PRESSURES INDICATED:

HOLE No. 1. = 5,700

HOLE No. 41 = 5,700

HOLE No. 81 = 5,600

HOLE No. 121 = 5,550

PREVIOUSLY THE LIFT-OFF WAS RECORDED AS FOLLOWS

HOLE No. 1. = 5800 - 10-1-66

HOLE No. 41 = 5850 - 10-14-66

HOLE No. 81 = 5700 - 10-7-66

HOLE No. 120 = 5700 - 10-3-66

J.G.

POOR ORIGINAL

ype II cement, modified for low heat of hydration, is used to minimize rinkage.

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

Elastometer Bearing Pads

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

Rock Anchor Tests

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

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

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

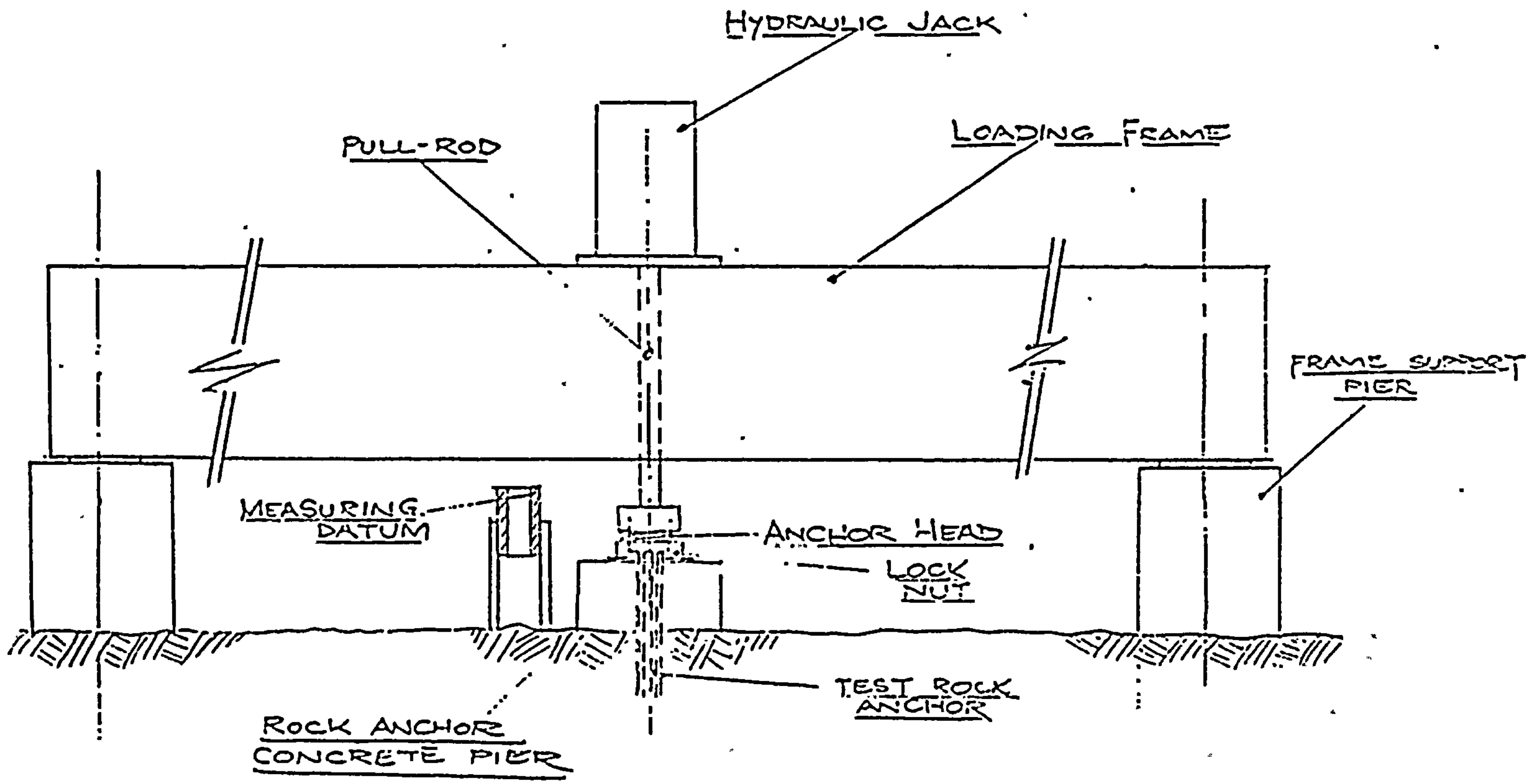
The test procedure for test A-1 was as follows:

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

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

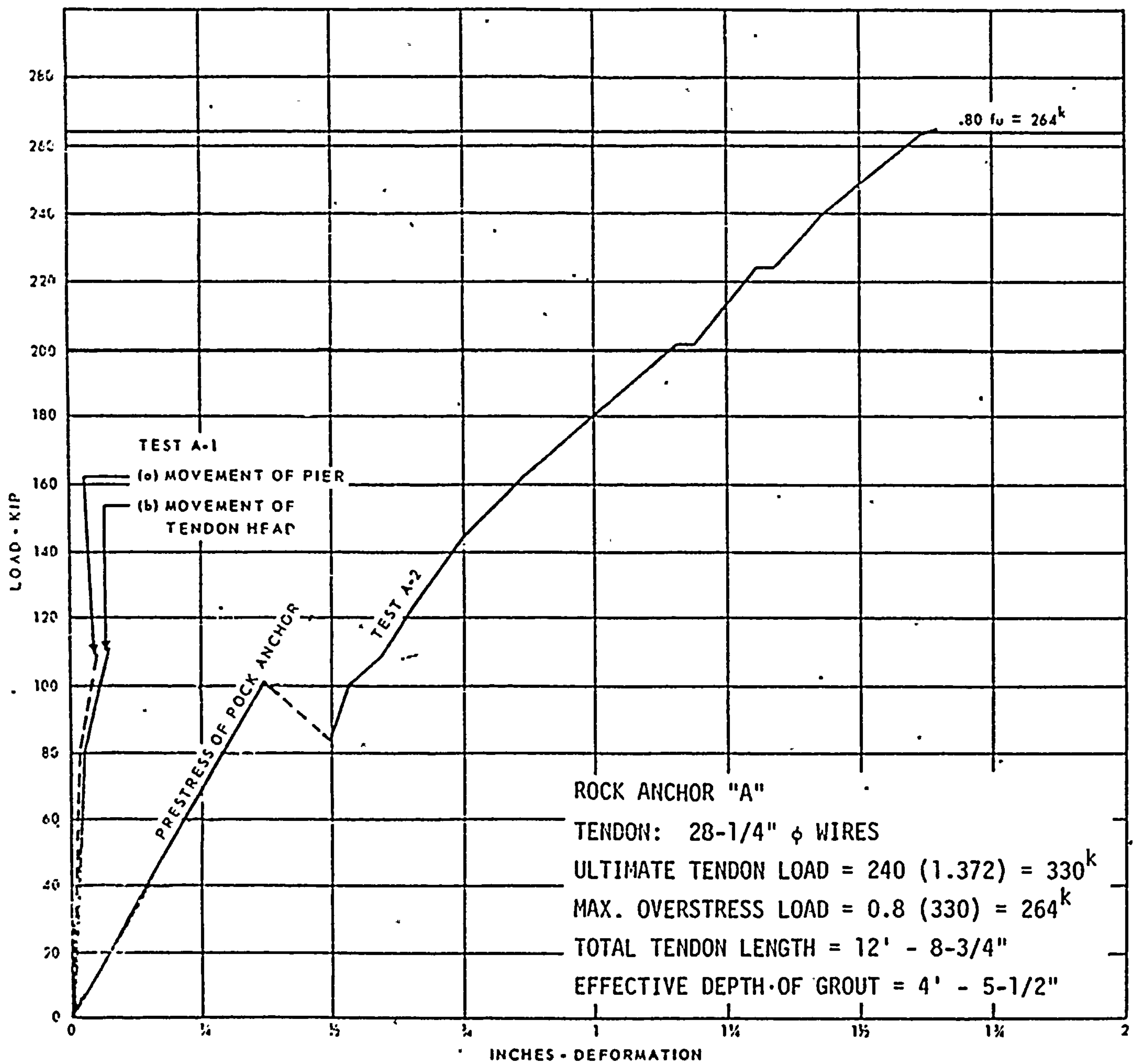
Table 5.6.1-1 lists measurements taken during test A-1. figures 5.6.1-6, 5.6.1-7 and 5.6.1-8 show plots of load vs. elongation deflection for all tests.

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



ROCK ANCHOR TEST A-1

ROCK ANCHOR TEST A-1
FIG. 5.6.1-5



CONTAINMENT VESSEL
ROCK ANCHOR TEST
SHEET 1

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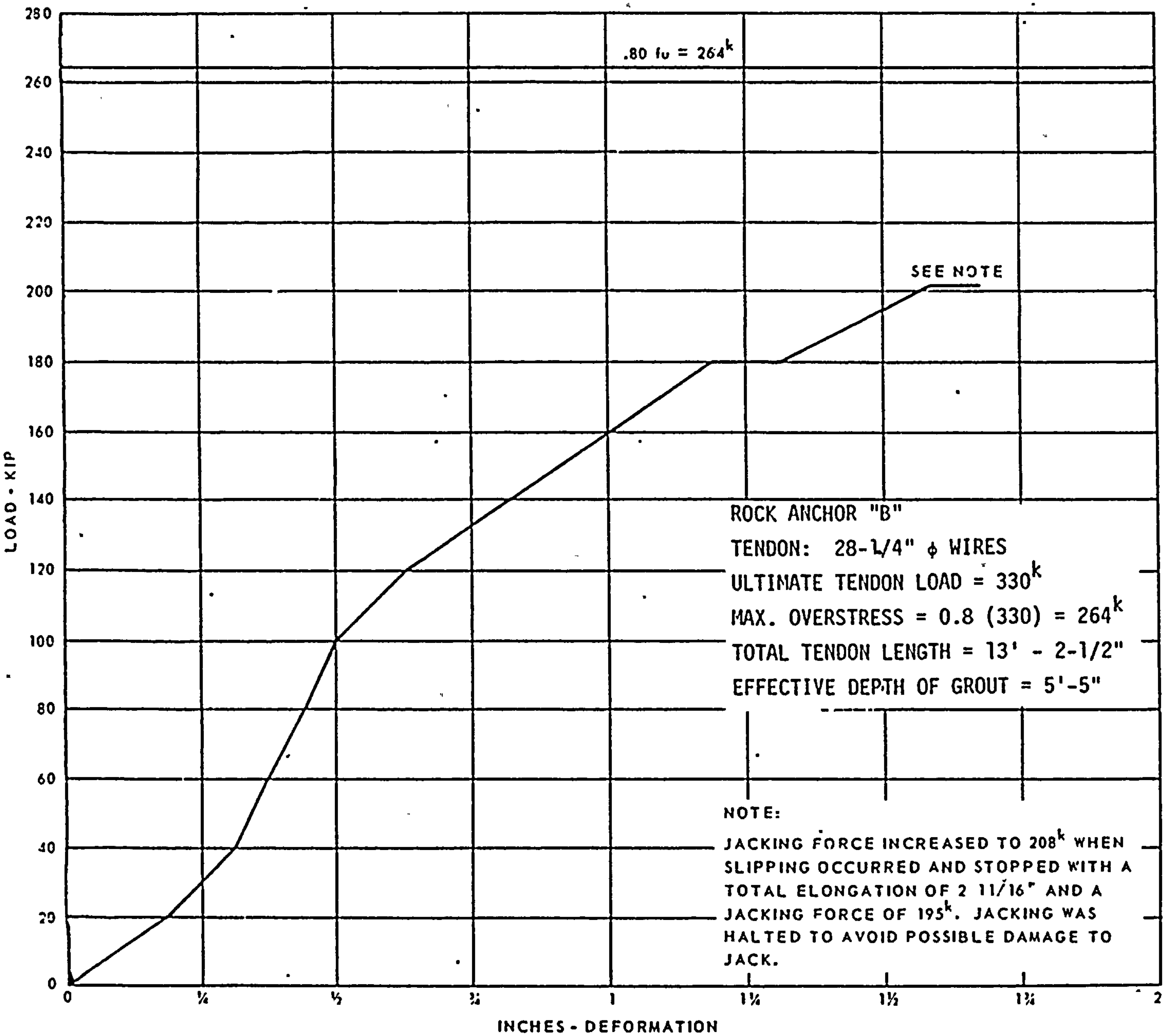
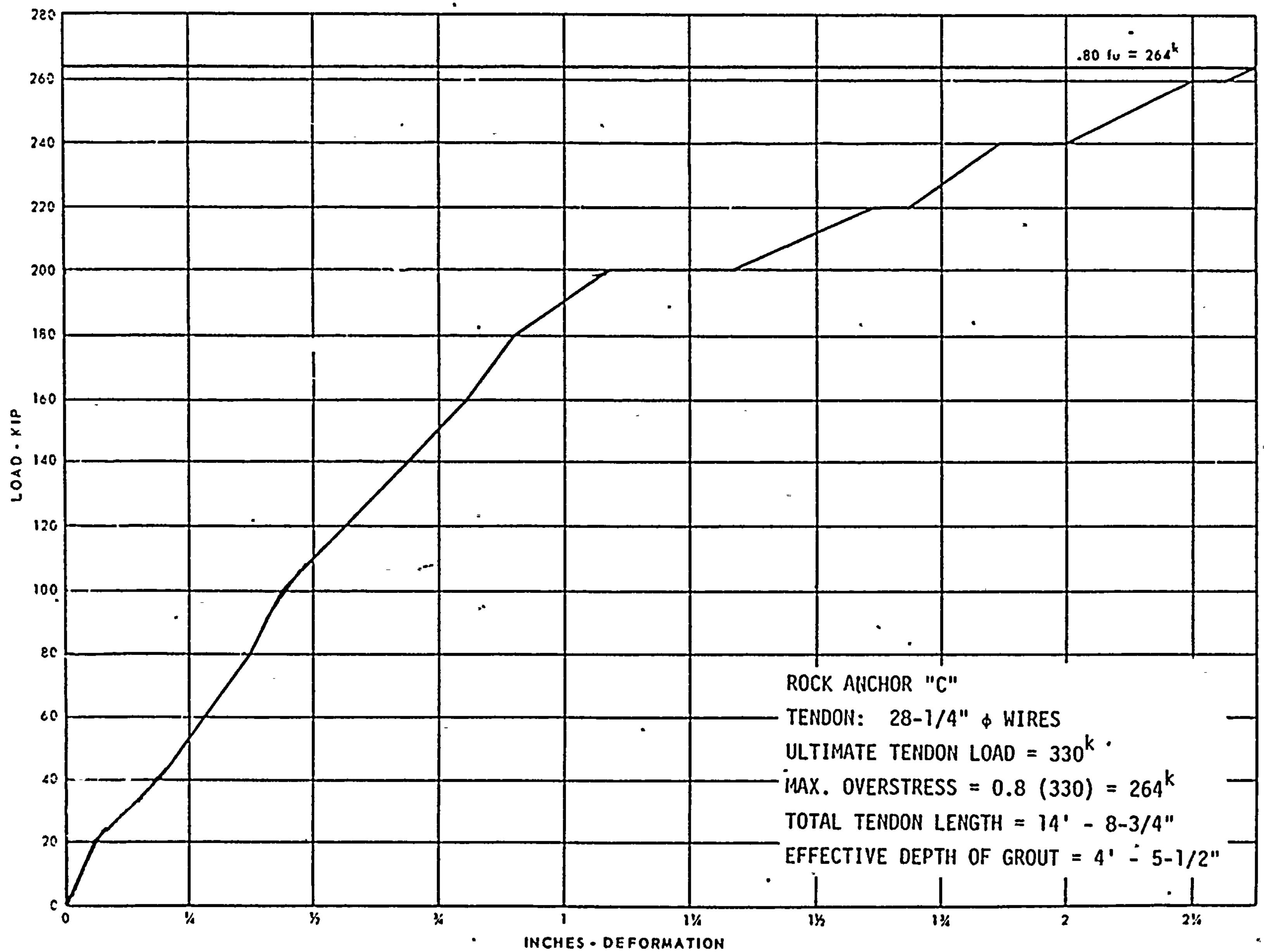
CONTAINMENT VESSEL - ROCK ANCHOR TEST - SHEET 1
FIG. 5.6.1-6



CONTAINMENT VESSEL
ROCK ANCHOR TEST

CONTAINMENT VESSEL - ROCK ANCHOR TEST - SHEET 2
FIG. 5.6.1-7



CONTAINMENT VESSEL
ROCK ANCHOR TEST

CONTAINMENT VESSEL - ROCK ANCHOR TEST - SHEET 3
FIG. 5.6.1-8

TABLE 5.6.1-1

GINNA STATION - UNIT NO. 1

ROCK ANCHOR "A" - UPLIFT TEST WITH JACKING FRAME

DATE OF TEST - MAY 19, 1966

TIME	LOAD KIPS	PIER DIALS N.E. CORNER INCHES	PIER DIALS S.W. CORNER INCHES	HEAD DIAL INCHES	AVERAGE DEFORMATION TOP OF PIER INCHES	ROCK SURFACE PEGS NORTH INCHES	ROCK SURFACE PEGS INTERMEDIATE INCHES	ROCK SURFACE PEGS SOUTH INCHES
0840	0	.300	0	.700	0	7-1/4	7-5/8	9-3/4
0955	20	.304	.005	.705	.0045			
1010	40	.308	.009	.709	.0085			
1025	60	.311	.012	.714	.0115			
1040	80	.318	.019	.723	.0185			
1055	100	.354	.031	.752	.0425	7-1/4	7-9/16	9-5/8
					LIFT OFF APPARENT			
1105	110	.380	.039	.767	.0595			
	80	.349	.025	739	.037			
	60	.334	.016	.724	.025			
	40	.326	.010	.715	.018			
	20	.318	.003	.706	.0105			
	0	.312	-.002	.699	.005	7-1/4	7-9/16	9-5/8

This copy to: R. M. LUKEN
Bechtel

GILBERT ASSOCIATES, INC.
READING, NEW YORK

RECEIVED
SEP 19 1966
BECHTEL CORP.

FILE			ADM. DEPT.	SCHE. DEPT.	COST DEPT.	PIPING DEPT.	MECH. DEPT.	ELEC. DEPT.	CIVIL DEPT.	PROJ. SUPT.	
		/									ACT.
											INFO.
											SIG.
											DATE

September 14, 1966

GGW-646

Mr. E. U. Powell, Manager
Ginna Project
Westinghouse Atomic Power Division
P. O. Box 355
Pittsburgh, Pennsylvania 15230

Please distribute

Re: Ginna Station Unit No. 1
Containment Vessel - Rock Anchors
GAI-4155

File
File. R.O.
etc.

Dear Mr. Powell:

Forwarded herewith are seven (7) copies of the revised specifications for installation of Rock Anchors.

This specification is a retyping of the entire installation procedure including the revision which was prepared and agreed upon at the job site meeting on September 7, 1966. This specification also includes the deletion of the requirement for performing the consistency (flow cone) test. Your attention is invited to the fact that the requirement as detailed in this specification and actually performed in the field is to determine the proportions of grout which have the lowest possible water-cement material ratio as is consistent with pump requirements. These proportions are then approved by the Engineer (Mr. Henry Walker). The testing laboratory field personnel must then insure that actual proportions remain as approved.

By copy of this letter this specification is also distributed as follows:

Addressee	Quantity
J. A. McConnell (RG&E)	1
R. R. Koprowski (RG&E)	3
R. M. Luken (Bechtel)	6
H. A. Parzick (WAPD)	1
T. M. Brown (Ryerson)	3

At the job site meeting on September 7, 1966 certain deviations were also requested from the tensioning procedure stipulated on the specification. This will confirm approval of these deviations. For your record six (6) copies of the installation and tensioning sequence are attached.

POOR ORIGINAL

By copy of this letter Mr. Ted Brown (Ryerson) is requested to forward to us one sepia and one print of the erection drawing showing identification of rock anchors by number.

Very truly yours,

D. K. Croneberger
Structural Engineer

DKC:bac
Enclosures

cc: E. U. Powell (5)
J. A. McConnell
R. R. Koprowski
R. M. Luken
H. A. Parzick
T. M. Brown

POOR ORIGINAL

DKC/HL
9-12-66
Revision III

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

1.0 SCOPE

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

2.0 REFERENCE CODES AND SPECIFICATIONS

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

3.0 MATERIALS FOR GROUT

3.1 Portland Cement

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

3.2 Water

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

3.3 Admixtures

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

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

4.0 MIXING PROPORTIONS

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

5.0 LABORATORY TESTS

5.1 General

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

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

5.2 Plastic Volume Change

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

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

5.3 Compressive Strength

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

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

6.0 MIXING

Care shall be taken to remove lumps, oversize material and foreign matter prior to introducing materials into the mixer. The temperature of the cement when placed into the mixer shall not exceed 140°F. To produce a uniform grout mixture with a minimum of mixing time, a mixer that produces a shearing action in mixing shall be used. This may be accomplished by paddles, discs, or drums running at high speed. in a vertical or horizontal position. Mixing time shall be approximately 1/2 minute per bag of cement. Due to the shearing action, a considerable amount of heat is generated and mixing at high speed shall be limited to about 2 minutes. Standard mortar and concrete mixers or hand mixing is not satisfactory. The grout mixture shall be screened with an 8-mesh strainer immediately after it leaves the mixer. Under no circumstances shall grout be retempered. The proportions of the grout shall be such as to have as low a water-cementing material ratio as is consistent with pumping requirements. These proportions are subject to approval by the Engineer.

7.0 PLACING GROUT

7.1 General

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

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

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

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

When the shipment of anchors is received at the jobsite, all tendons shall be examined for shipping damages and location of spacer. As the tendon is lowered into the hole, the tendon shall also be examined for any inconsistencies apparent in the arrangement of the wires. After the anchor is inserted into the hole and prior to placing the first stage grout, the button heads shall be examined to insure wire lengths are as specified.

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

7.2 First Stage Grout

The depth of each hole shall be determined and the volume of grout computed to provide the required embedment length. The grout shall be placed by gravity in a manner to displace any ground water in the hole upward without producing any significant dispersion of the grout. Special care shall be exercised to grout approximately the first two feet at a slow speed to avoid excessive mixing of the grout with the water. Grout shall be placed to an elevation approximately one foot above the desired level. Within thirty (30) minutes and after the grout feeder hose has been withdrawn, the pump shall be lowered to an elevation of 11'-0" below the top of the jacking plate and operated slowly until it is evident that no grout is being removed. A hydrostatic head of water shall be maintained constantly to within one foot (1') of the jacking plate by adding water through the hole in the anchor head. Two hours later the pumps shall be lowered to nine inches (9") above the previously pumped grout level (i.e., 10'-3" below the top of the jacking plate) and, while pumping, shall be lowered slowly until the grout level is detected. If grout level is more than six inches (6") below the original level, adjacent holes shall be inspected for grout. If grout level is within the specified tolerances, no further action is required pending tensioning except to maintain the hole full of water. Periodic checks shall be made to ensure that the hole remains filled with water. If the grout level is not within the specified tolerances, the tendon shall be removed and the hole flushed. The cause of abnormal grout rise or fall shall be determined before re-grouting.

7.3 <u>Second Stage Grout</u>

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

POOR ORIGINAL

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

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

8.0 <u>TENSIONING SEQUENCE</u>

Rock anchors shall not be tensioned until the concrete supporting the tendon hardware (baseplate, movable head and shins) has attained a minimum ultimate compressive strength of 4000 psi. When a tendon is located within 3'-0" of a construction joint in the ring girder, the minimum ultimate compressive strength of the abutting pour shall also be 4000 psi. No rock anchors shall not be tensioned until the grout specimens exhibit a minimum confined compressive strength of 4000 psi unless otherwise approved by the Engineer nor shall they be tensioned before the grout has cured a minimum of 10 days. The sequence for tensioning the anchors shall be as follows:

a. Initially, tension every fourth anchor. There are no limitations on the sequence for tensioning these anchors.

b. Secondly, tension the anchors located mid way between the tensioned anchors. There are no limitations on the sequence for tensioning these anchors.

c. Thirdly, tension remaining anchors. Again, there are no limitations on the sequence for tensioning these anchors.

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

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

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

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

INSTALLATION AND TENSIONING SEQUENCE

INSTALLATION

9-16-66

123	9·9 × 28 37'6	1559-16 121 35'6	69
16	9·9 31	25" 118 35'6	71
157	9·9 × 34	153" 114 38'3	73
13	9·9 × 38	86" 111 37'0	75
11	9·9 × 42	125" 108 35'9	77
134	9·9 × 46 37'9	9·19 104 ×	79
114	9·12 50 39'6	104" 100 38'6	81
121	9·12 54 38'6	138 96 38'3	83
106	9·12 58 41'9	143 93 37'9	85
127	9·12 62	165 90 38'3	88
118	9·12 66	24 87 36'3	91
126	9·12 70 38'9	9-19 84 ×	92
112	9·12 74 38'3	" 80 ×	94
101	9·12 78 37'3	" 76 ×	97
109	9·12 82 38'6	" 72 ×	99
	9·12 86 38'6	68	101
147 11am	9·13 89 37'0	64	103
22	9·13 95 39'0	9·19 60 ×	105
132	9·13 98 39'0	" 56	107
99	9·13 102 39'0	" 52	109
135	9·13 106 35'6	" 48	112
G	9·13 110 38'3	" 44	113
	9·13 116 38'0	" 40	115
R	9·13 119 38'6	" 36	117
144	9·13 122 35'0	" 32	120
O	9/14 126 37'6	29	123
	" 130 38'0	27	125
	" 134 36'9	33	127
U	" 138 38'9	35	129
	" 141 34'9	37	131
	" 144 36'6	39	133
	" 150 38'0	41 30	135
T	97-9·16 153 37'3	43	137
	" 156 36'9	45	140
	" 158 37'6	47	143
	" 154 38'6	49	146
G	9·14 " 152 36'0	51	147
	" 148 37'9	53	149
R	" 145 36'3	55	151
O	" 142 38'6	57	155
	9·14 " 139 35'0	59	157
U	23 - 9·14 136	61	159
T	108 " 132	63	~~260~~
	137 " 128	65	
	150 " 124	67	

TENSION

18 × 9·7	54	132
10 9·7	58	128
92 × 9·7	62	124
113 × 9·7	66	121
147	70	118
2 × 9·7	74	114
6 × 9·7	78	111
14 × 9·8	82	108
22 "	86	104
26 "	89	100
24 "	95	96
20	98	93
16	102	90
12	106	87
8	110	84
4	116	80
160 9·8	119	76
3 9·9	122	72
5 9·9	126	68
7 9·9	130	64
9 9·12	134	60
11 9·12	138	56
13 9·22	141	52
15 9·22	144	48
17 9·22	150 ×	44
19 9·26	153 ×	40
21 9·26	156 ×	36
23 9·27	158 ×	32 10/6
28	1 ×	29
25	154 ×	27
31	152 ×	33
34	148	35
38	145	37
42	142	39
46 9/27	139	41
50	136	43

POOR ORIGINAL

FORM E-6 200M

SIGNATURE

DATE 6/23/66

TITLE RYERSON STEEL FIELD RECORD

JOB No.

SUBJECT POST TENSIONING JOB PROGRESS

SHEET No. 1 OF 3

NO.	DEPTH	ANCH	GROUT DATE	GROUT LEVEL	STRESS		NO.	DEPTH	ANCH	GROUT DATE	GROUT LEVEL	STRESS
1	38'9	77	8/16	10'9			36	38'0				
2	38'4	65	12	11'0			37	38'2				
3	37'6	71	12	9'8			38	38'3				
4	38'0	6	15	9'8			39	38'4				
5	37'8	60	15	9'4			40	38'5				
6	38'4	69	12	9'8			41	38'5				
7	38'0	44	15	9'3			42	38'5				
8	38'0	4	12	10'7			43	38'8				
9	37'10	70	15	9'1			44	38'0				
10	38'0	64	12	9'1			45	38'11				
11	37'9	68	12	9'6			46	37'11				
12	38'0	62	12	8'9			47	37'9				
13	37'6	5	15	9'1			48	38'6				
14	37'9	72	15	9'8			49	38'6				
15	37'9	63	16	10'11			50	39'3				
16	37'8	66	15	9'10			51	34'3				
17	37'11	75	15	9'4			52	38'8				
18	37'9	46	15	9'5			53	38'0				
19	37'8	45	15	8'9			54	38'9				
20	37'6	84	16	10'3			55	39'0				
21	37'6	102	16	11'0			56	38'8				
22	37'7	93	16	11'8			57	37'10				
23	38'0	92	16	11'5			58	42'0				
24	37'3	87	16	11'6			59	37'10				
25	37'6	16	16	10'8			60	38'8				
26	36'1	100	16	11'0			61	38'0				
27	37'8						62	39'1				
28	37'4						63	36'10				
29	37'10						64	38'9				
30	37'10						65	38'6				
31	37'10						66	37'0				
32	37'10						67	39'0				
33	37'11						68	38'8				
34	38'3						69	37'9				
35	38'1						70	39'2				

POOR ORIGINAL

FORM E-6 300M

SIGNATURE ______ DATE ______

TITLE RYERSON STEEL FIELD RECORD JOB No. ______

SUBJECT POST TENSIONING JOB PROGRESS SHEET No. 2 of 3

NO.	DEPTH	ANCH	GROUT DATE	GROUT LEVEL	STRESS		NO.	DEPTH	ANCH	GROUT DATE	GROUT LEVEL	STRESS
71	39'2						106	38'4				
72	38'8						107	38'3				
73	38'5						108	38'0				
74	38'2						109	38'2				
75	38'8						110	38'5				
76	36'6						111	38'5				
77	38'1						112	38'2				
78	38'0						113	37'9	61	11	12'7	
79	38'3						114	38'5				
80	38'8						115	38'7				
81	38'6						116	38'4				
82	38'8						117	38'8				
83	38'8						118	38'2				
84	38'8						119	38'3				
85	38'8						120	38'9				
86	38'3						121	38'3				
87	38'8						122	36'10				
88	38'5						123	38'8				
89	38'8						124	37'8				
90	38'6						125	37'7				
91	38'10						126	37'9				
92	38'9	62	11	12'7			127	37'10				
93	38'7						128	38'0				
94	38'12						129	38'2				
95	39'2						130	38'4				
96	38'9						131	38'3				
97	39'1						132	38'5				
98	39'2						133	38'4				
99	39'0						134	38'0				
100	38'11						135	38'2				
101	38'0						136	38'5				
102	39'2						137	38'9				
103	37'0						138	38'9				
104	39'0						139	38'9				
105	39'2						140	38'5				

FORM 5-6 200M

SIGNATURE ______ DATE ______

TITLE RYERSON STEEL FIELD RECORD JOB No. ______

SUBJECT POST TENSIONING JOB PROGRESS SHEET No. 3 OF 3

NO.	DEPTH	ANCH	GROUT DATE	GROUT LEVEL	STRESS		NO.	DEPTH	ANCH	GROUT DATE	GROUT LEVEL	STRESS
141	38' 9											
142	38 3											
143	37'10											
144	38 4											
145	38 2											
146	38'1											
147	38'0	79	16	9'8								
148	38'5											
149	37'6											
150	37'10											
151	37'6											
152	38'1											
153	38'0											
154	38'0											
155	38'6											
156	38'0											
157	37'3											
158	36'0											
159	36'9											
160	38'9	59	16	10'2								

ROCHESTER GAS AND ELECTRIC CORP. SHEET NO. 1

DATE August 19, 1966

TO Mr. John Arthur

Subject: Ryerson Rock Anchors (Grouting Operation)

Space remaining for Second Stage Grout below top of sleeve.

Hole Number	Tendon Number	Height Recorded	HIGH	LOW	GOOD
1	77	10' 4½"	1½"	1½"	
2	65	11' 0"			X
3	71	9' 0½"	1' 5½"		
4	6	9' 7½"	9½"		
5	60	9' 3½"	1' 2½"		
6	69	9' 8½"	9½"		
7	44	9' 3"	1' 3"		
8	4	10' 7"			X
9	70	9' 3"	1' 3"		
10	64	9' 0"	1' 6"		
11	68	9' 0"	1' 6"		
12	62	8' 8"	1' 10"		
13	5	9' 0½"	1' 5½"		
14	72	9' 8"	10"		
15	63	11' 0"			X
16	66	9' 11"	7"		
17	75	9' 4½"	1' 1½"		
18	46	9' 5½"	1' 0½"		
19	45	8' 7"	1' 11"		
20	84	10' 10"			X
21	102	11' 0½"			X
22	93	11' 10"		1"	
23	92	11' 6"			X
24	87	11' 6"			X
25	16	10' 5"	1"		
26	100	11' 2½"			X
92	58	12' 7"		10"	
113	61	12' 7"		10"	
147	79	9' 8"	10"		
160	59	10' 1½"	5½"		

Plans and Specifications indicate height of Second Stage Grout to be 11' 6" with an allowable tollerance of plus one foot to minus three inches of this design requirment- 10' 6" to 11' 9"

Measurments taken this date by R. Murray , J. Boniface

R. G. & E. Field Office
Robert Emmett Ginna Nuclear Power Station

POOR ORIGINAL

...ENG WORK SHEET

SHEET____OF____

B[…]CHTEL CORPORATION

JOB No. 5807 — R.E. GINNA NUCLEAR POWER PLANT

BY: J.G. — ONTARIO CENTER, N.Y.

DATE: 12-10-66 — REPORT ON ROCK ANCHOR 1ST STAGE GROUT CUBE TESTS.

SHEET-1

PLA[…] WALL TENDON	HOLE No.	TENDON No.	HOLE DEPTH.	DATE PLACED	DATE GROUTED	SPACE REMAINING	7 DAYS 1ST CUBE	7 DAYS 2ND CUBE	3RD CUBE →	DAYS
1-11-68	1	77	38'-9"	8-16-66	8-16-66	10'-9"	3,790	BAD CYL.		
1-11-68	2	65	38'-4	8-12-66	8-12-66	11'-4"	2950	3100	4000	35 DAYS
	3	71	37'-6	8-12-66	8-12-66	9'-5	3325	1562.	4125	35
	4	6	38'-0	8-15-66	8-15-66	10'-0	3500	3225		
-11-68	5	60	37'-8	8-15-66	8-15-66	9'-4	3637	3700		
-12-68	6	69	38'-4	8-12-66	8-12-66	10'-0	CUBES	DEFECTIVE		
-11-68	7	44	38'-0	8-15-66	8-15-66	9'-7	3962	4025		
-12-68	8	4	38'-0	8-12-66	8-12-66	10'-11	2675	2400	4125	35
-12-68	9	70	37'-10	8-15-66	8-15-66	9'-5	3575	3636		
-11	10 G	64	38'-0	8-12-66	8-12-66	9'-5	2535	2913		
-11-68	11 G	68	37'-9	8-12-66	8-12-66	9'-10"	3000	2800	3800	35
-12-68	12 G	62	38'-0	8-12-66	8-12-66	9'-1"	2000	2025	3125	35
1-10-68	13 G	5	37'-8	8-15-66	8-15-66	9'-5"	DESTROYED	3165		
1-10-68	14 G	72	37'-9	8-15-66	8-15-66	10'-0	DESTROYED	2937		
	15	63	37'-9	8-16-66	8-16-66	11'-3	3150	3312		
	16	66	37'-8	8-15-66	8-15-66	10'-2	3387	3417		
	17	75	37'-11	8-15-66	8-15-66	9'-8	3440	3615		
	18	46	37'-8	8-15-66	8-15-66	9'-9	3150	3212		
1-10-68	19 G	45	37'-8	8-15-66	8-15-66	9'-1	3305	3100		
	20	84	37'-6	8-16-66	8-16-66	11'-2	3000	3000		
1-11	21 G	102	37'-6	8-16-66	8-16-66	11'-4	2687	3050		
	22	93	37'-7	8-16-66	8-16-66	12'-2"	3500	3687		
	23	92	37'-3	8-16-66	8-16-66	11'-9	2625	3425		
	24	87	37'-3	8-16-66	8-16-66	11'-10	2250	2500		
1-10-68	25 G	16	37'-6	8-16-66	8-16-66	10'-8	2555	2717		
1-10-68	26 G	100	36'-1	8-16-66	8-16-66	11'-6	2450	2950		
	27	8A	38'-0	9-20-66	9-20-66	11'-0"	3450	3625	4500	16
1-25-68	28	129	37'-6	9-9-66	9-9-66	11'-4"	4358	4200	4625	18
	29	56A	36'-3	9-20-66	9-20-66	10'-6	3750	4125	5125	16
	30	43A	37'-0	9-20-66	9-20-66	11'-0"	4000	4025	5050	28

POOR ORIGINAL

SHEET____OF____

BECHTEL CORPORATION

JOB No. 5807

BY: [illegible]

DATE: 12-10-66

R.E. GINNA NUCLEAR POWER PLANT

ONTARIO CENTER, N.Y.

REPORT ON ROCK ANCHOR 1ST STAGE GROUT CUBE TESTS SHEET.

WALL TENDON PLACED	HOLE No.	TENDON No.	HOLE DEPTH	DATE PLACED	DATE GROUTED	SPACE REMAINING	7 DAYS 1ST CUBE	7 DAYS 2ND CUBE	3RD CUBE	DAYS
	31	96	38'-0	9-9-66	9-9-66	11'-2	3513	3000	4625	18
	32	95	37'-0	9-19-66	9-20-66	11'-2	4062	3875	4800	16
	33	28A	37'-6	9-20-66	9-20-66	11'-0"	4650	4525	5300	16
	34	107	38'-6	9-9-66	9-9-66	11'-5½	3325	3463	4300	18
	35	39A	38'-4	9-20-66	9-20-66	11'-1	3738	3925	5000	18
	36	146	37'-6	9-19-66	9-20-66	11'-1	4100	4062	4312	18
	37	26	37'-9	9-20-66	9-20-66	11'-4	4487	4500	4800	18
	38	133	39'-0	9-9-66	9-9-66	11'-7½	3350	3825	4050	18
	39	42A	37'-3	9-20-66	9-20-66	11'-0	4100	4250	5675	25
	40	115	38'-6	9-19-66	9-20-66	11'-2	3562	3000	4375	16
	41	17	37'-6	9-20-66	9-20-66	11'-0	3600	3925	5500	25
1-22-68	42	111	38'-9	9-9-66	9-9-66	11'-3	3650	3375	4150	18
1-23-68	43	2	36'-9	9-20-66	9-20-66	11'-0	4500	3875	5425	17
1-23-68	44	41	37'-9	9-19-66	9-20-66	11'-1	4112	4100	4225	17
	45	51A	37'-0	9-20-66	9-20-66	11'-1	3400	3250	4125	17
	46	134	37'-9	9-9-66	9-9-66	11'-2½	2875	3125	3750	18
	47	20	38'-3	9-23-66	9-23-66	10'-11	4175	3800	4675	17
	48	158	37'-0	9-19-66	9-20-66	10'-9	3950	3700	4250	17
	49	48	38'-0	9-23-66	9-23-66	10'-11	3125	3350	4125	17
1-2?	50	114	39'-6"	9-12-66	9-12-66	10'-6	3250	4313	4525	17
1-22-68	51	29	38'-0	9-23-66	9-23-66	10'-9	3400	2900	3575	16
1-23-68	52	120	37'-3	9-19-66	9-20-66	11'-3	4075	4087	4625	17
1-23-68	53	34	38'-0	9-23-66	9-23-66	10'-11	3675	4150	5125	17
1-23-68	54	121	38'-6	9-12-66	9-12-66	11'-2	4750	4350	4513	17
	55	9A	37'-9	9-23-66	9-24-66	10'-5	4250	3750	5000	16
	56	140	39'-3	9-19-66	9-20-66	10'-11	4050	4400	5500	17
	57	15A	37'-6	9-23-66	9-24-66	10'-9	3550	3550	5425	16
	58	106	41'-9	9-12-66	9-12-66	11'-1	3500	3438	4375	17
1-17-68	59 G	52A	38'-0	9-23-66	9-24-66	10'-8	3275	4125	4575	16
1-17-68	60 G	154	35'-3	9-19-66	9-19-66	11'-3				

POOR ORIGINAL

ENG WORK SHEET V

SHEET ___ OF ___

BECHTEL CORPORATION

JOB: 5227

BY: [illegible]

DATE: 12-12-66

R E Ginna Nuclear Power Plant

Ontario Center New York

Report on Rock Anchor 1st stage Grout Cube Tests

SHEET 3-6

	Hole No	Tendon No	Hole Depth	Tube Placed	Date Grouted	Space Remaining	7 Days 1st Cube	7 Days 2nd Cube	3rd Cube	Days
-17-68	61 G	1A	38'-0"	9-23-66	9-24-66	10'-11"	4150	4000	5425	16
	62	127	39'-0"	9-12-66	9-12-66	11'-2"	3300	3100	3775	17
-22-68	63	3A	37-10	9-23-66	9-24-66	10'-9"	3360	4125	5625	16
-22-68	64 G	136	36'-4"	9-19-66	9-20-66	10'-10"	4237	3975	5375	17
	65	54A	38'-9"	9-24-66	9-24-66	10'-9"	3200	4500	5350	16
	66	118	39'-0"	9-12-66	9-12-66	11'-3"	3350	3150	4375	17
	67	14A	38'-4"	9-24-66	9-24-66	10'-6"	4325	4275	4375	16
REPEAT →	68	40	39'-0"	9-23-66	9-23-66	10'-9"	4200	3625	5400	26
	69	55A	38'-6"	9-24-66	9-24-66	10'-7"	3650	4000	4700	11
	70	126	38'-9"	9-12-66	9-12-66	11'-4½"	3400	3550	3750	17
	71	21A	39'-0"	9-24-66	9-24-66	11'-0"	3700	4000	4525	11
	72	152	36'-7"	9-19-66	9-19-66	11'-5"	4425	4188	5300	17
	73	27A	38'-3"	9-24-66	9-24-66	10'-10"	3250	2825	4775	16
	74	112	38'-6"	9-12-66	9-12-66	11'-½"	3450	3563	4000	16
	75	19A	38'-3"	9-26-66	9-26-66	10'-11"	4125	3950	3975	10
	76	122	36'-9"	9-19-66	9-19-66	11'-4"	4113	4200	5525	17
	77	10A	36'-3"	9-26-66	9-26-66	10'-10"	3925	3750	4300	10
	78	101	37'-3"	9-12-66	9-12-66	11'-5"	5063	5575	5450	16
	79	149	38'-3"	9-26-66	9-26-66	11'-0"	3800	3375	4510	11
	80	110	34'-9"	9-19-66	9-19-66	11'-3"	4313	4500	4900	17
	81	7A	38'-3"	9-26-66	9-26-66	10'-10"	3275	3950	4600	11
	82	109	38'-6"	9-12-66	9-12-66	11'-2½"	3188	3000	4625	16
	83	12A	37'-0"	9-26-66	9-26-66	10'-10"	3575	3200	4490	11
	84	148	37'-6"	9-19-66	9-19-66	11'-3"	5850	5300	5100	16
	85	12B	37'-6"	9-26-66	9-26-66	10'-11"	3875	4175	4825	11
	86	130	38'-6"	9-12-66	9-12-66	11'-6"	4450	5088	5375	18
	87	24	36'-3"	9-16-66	9-16-66	11'-0"	4300	4000	4625	18
	88	53A	37'-9"	9-26-66	9-26-66	11'-2"	4175	4450	5250	11
	89	147	37'-0"	9-12-66	9-13-66	11'-0"	3650	4088	4825	18
	90	25	32'-3"	9-16-66	9-16-66	11'-8"	4275	4500	4950	18

POOR ORIGINAL

ENG WORK SHEET V

SHEET ____ OF ____

BECHTEL CORPORATION

JOB No. 5507 — R.E. Ginna Nuclear Power Plant

BY: [illegible] — Ontario Center, New York

DATE: 12-10-66 — Report on Rock Anchor 1st stage Grout Cube Tests Sheet 5

Hole No.	Tendon No.	Hole Depth	Date Placed	Date Grouted	Space Remaining	7 Days 1st cube	7 Days 2nd cube	3rd cube	Days
121	155	35'6"	9-16-66	9-16-66	10'-11"	4000	4113	4925	18
122	144	35'-0"	9-13-66	9-14-66	10'-6"	4625	4700	5125	18
123	73A	38'-3"	9-29-66	9-29-66	10'-6"	4000	4050	5000	19
124	150	37'-6"	9-16-66	9-16-66	10'-7"	4525	4700	4825	18
125	67A	37'-9"	9-29-66	9-29-66	10'-6"	3925	3900	4750	19
126	142	37'-6"	9-14-66	9-14-66	11'-2½"	4375	4075	4925	16
127	94A	37'-9"	9-29-66	9-29-66	10'-11"	4225	3375	4900	19
128	137	37'-9"	9-16-66	9-16-66	10'-8"	4113	4225	4300	18
129	91A	37'9"	9-29-66	9-29-66	10'-11"	4350	4400	4575	19
130	159	38'-0"	9-14-66	9-14-66	11'-0"	5375	4600	6150	16
131	90A	38'-3"	9-29-66	9-29-66	10'-9"	4175	4425	4600	11
132	108	37'-0"	9-16-66	9-16-66	11'-1"	5188	5050	5625	18
133	47A	38'-6"	9-29-66	9-29-66	10'-11"	4450	4075	4925	11
134	124	36'-9"	9-14-66	9-14-66	11'-2"	4875	4325	4425	16
135	50A	36'-0"	9-29-66	9-29-66	10'-11"	4250	4150	4825	11
136	23	38'-0"	9-16-66	9-16-66	11'-3"	4350	5000	4325	18
-25-68 137	49A	38'-3"	9-29-66	9-29-66	10'-10"	4425	4450	5400	11
-23-68 138	119	38'-9"	9-14-66	9-14-66	11'-0"	4475	4500	5200	16
139	128	35'-0"	9-14-66	9-16-66	9'-0"	4400	3750	4475	17
140	76A	38'-3"	9-29-66	9-29-66	10'-6"	4250	3150	5250	11
-23-68 141	139	34'-9"	9-14-66	9-14-66	10'-9"	4250	4875	5375	16
142	141	38'-6"	9-14-66	9-16-66	11'-1"	3600	4350	4375	17
143	74A	38'-3"	9-29-66	9-29-66	10'-7"	4525	4200	5425	11
144	113	36'-6"	9-14-66	9-14-66	10'-11"	4125	3800	5325	16
145	123	36'-3"	9-14-66	9-16-66	11'-6"	4500	4375	4725	17
146	78A	36'-9"	9-29-66	9-29-66	10'-6"	3850	3575	4825	11
-25-68 [illegible] 147	79	38'-3"	8-19-66	8-19-66	10'-0"				
-25-68 148	103	37'-9"	9-14-66	9-16-66	11'-3"	4175	3825	4825	17
149	30A	36'-9"	9-20-66	9-20-66	11'-1"	4187	4700	5000	17
150	117	38'-0"	9-14-66	9-14-66	11'-0"	4438	4000	5375	16

POOR ORIGINAL

SHEET ____ OF ____

[illegible]ECHTE[illegible] [illegible]ORATION

JOB No. 5807 — R[illegible] Nuclear Power Plant

BY: [illegible] — Ont[illegible]o Center, New York

DATE: 12-10-66 — Report on Rock Anchor 1st stage Grout Cube Tests

Sheet 4-6

	Hole No.	Tendon No.	Hole Depth	Date Placed	Date Grouted	Space Remaining	7 Days 1st cube	7 Days 2nd cube	3rd cube	→ Days
	91	11A	38'-0"	9-26-66	9-26-66	10'-8"	4175	3675	4700	11
	92	58	38'-3"	8-11-66	8-11-66	12'-1"	2000	1856	3125	34
	93	143	37'-9"	9-16-66	9-16-66	11'-0"	4425	4675	5325	18
	94	13A	37'-0"	9-26-66	9-26-66	10'-11"	3825	4050	4425	11
	95	22	39'-0"	9-13-66	9-13-66	11'-0"	4913	5313	5125	18
	96	138	38'-3"	9-16-66	9-16-66	11'-2"	4675	4200	4700	18
	97	35A	38'-9"	9-26-66	9-26-66	11'-2"	3700	3700	4560	11
	98	133	39'-0"	9-13-66	9-13-66	11'-5½"	4550	4938	5175	18
	99	85A	38'-9"	9-28-66	9-29-66	11'-1"	4925	4550	4900	19
	100	104	38'-6"	9-16-66	9-16-66	11'-2"	4700	4700	4925	18
2-68	101	80A	37'-3"	9-28-66	9-29-66	11'-0"	4675	4725	5500	19
2-68	102	99	39'-0"	9-13-66	9-13-66	10'-6"	4900	4700	4875	18
2-68	103	88A	36'-3"	9-28-66	9-29-66	10'-11"	4175	4550	4875	19
2-68	104	156	36'-6"	9-19-66	9-19-66	10'-7"	4913	4688	5400	15
	105	86A	38'-9"	9-28-66	9-29-66	10'-9"	4400	4450	5375	19
	106	135	38'-6"	9-13-66	9-14-66	11'-1"	4275	4875	4962	18
	107	81A	38'-0"	9-28-66	9-29-66	10'-9"	4250	4375	4700	19
	108	125	35'-9"	9-16-66	9-16-66	10'-9"	4525	4588	5150	18
	109	57A	37'-9"	9-28-66	9-29-66	10'-7"	3900	4425	4900	19
	110	131	38'-3"	9-13-66	9-14-66	11'-½"	4850	4025	4962	18
	111	86	37'-0"	9-16-66	9-16-66	11'-1"	4575	4525	5100	18
	112	33A	37'-0"	9-28-66	9-29-66	10'-8"	4400	4250	4825	19
	113	61	37'-9"	8-11-66	8-11-66	12'-1"	No	test		
	114	153	38'-3"	9-16-66	9-16-66	11'-2"	4200	4475	4550	18
	115	83A	38'-3"	9-28-66	9-28-66	10'-11"	4500	4425	5400	19
	116	36	38'-0"	9-13-66	9-14-66	10'-9½"	4375	4000	4800	18
	117	88A	38'-3"	9-28-66	9-29-66	11'-2"	4925	4550	5500	19
	118	25	35'-[illegible]"	9-16-66	9-16-66	11'-0"	4000	4600	5000	18
	119	115	38'-6"	9-13-66	9-14-66	10'-7½"	5225	5500	6725	18
	120	99A	35'-2"	9-28-66	9-28-66	10'-0"	4250	4450	4750	19

POOR ORIGINAL

ENG WORK SHEET

SHEET____OF____

BECHTEL CORPORATION

JOB No. 5807

BY: [illegible]

DATE: 12-10-66

R.E. Ginna Nuclear Plant

Ontario Center, New York

Report on Rock Anchors, 1st Stage Grout Cube Tests

Sheet 6-1

	Hole No.	Tendon No.	Hole Depth	Date Placed	Date Grouted	Space Remaining	7 Days 1st Cube	7 Days 2nd Cube	3rd Cube	→ Days
	151	37A	35'-3"	9-20-66	9-20-66	10'-11"	3475	3475	4940	17
	152	116	36'-0"	9-14-66	9-16-66	11'-2"	4175	4275	4825	17
	153	97	37'-3"	9-16-66	9-16-66	11'-0"	3625	4038	4250	17
	154	151	38'-6"	9-14-66	9-14-66	11'-0"	3438	3750	4750	16
	155	38A	36'-0"	9-20-66	9-20-66	11'-0"	3700	3850	4320	11
	156	160	32'-9"	9-14-66	9-14-66	10'-11"	4375	4500	5000	16
	157	32	36'-6"	9-20-66	9-20-66	10'-9"	4187	4625	4750	11
	158	157	37'-6"	9-14-66	9-14-66	10'-10"	4050	4275	4550	16
-22	159	31A	34'-3"	9-20-66	9-20-66	10'-11"	3725	4037	4150	11
-23 Support	160	59	35'-6	8-16-66	8-16-66	10'-6"	2875	2437	No Record	

POOR ORIGINAL

W.R. GRACE & CO. DIVISION

DEARBORN CHEMICAL

LABORATORY:
320 GENESEE STREET, LAKE ZURICH, ILLINOIS 60047

DATE November 20, 1967

AFFIDAVIT

SHIPPED TO
Joseph T. Ryerson & Sons, Inc.
2558 West 16th Street
Chicago, Illinois 60616
Attention: Mr. Frank Bialas

ORDER NO. 21T111-10

VAT OR ROLL NO. NO-OX-ID® 490 Nuclear Grade-Batch 4775

DATE SHIPPED Nov. 6, 1967

AMOUNT SHIPPED 4 - 55 gallon drums; 1500 lbs. net

We hereby certify that the above identified material was tested in accordance with Gilbert Associates, Inc., Technical Specification SP-5357 and that the results of such tests have met the requirements as follows:

Chlorides	6 ppm
Nitrates	No detectable amount
Sulfides	No detectable amount

R. A. Larrick

R.A. Larrick - Director,
Analytical Services

Subscribed and sworn to before me this 20
day of November 1967

NOTARY PUBLIC

POOR ORIGINAL

Form 515-B

DEARBORN CHEMICAL DIVISION
W. R. GRACE & CO.

SUBJECT: INITIAL PUMPING OF NO-OX-ID® CM CASING FILLER - NUCLEAR GRADE

CITY Chicago, Ill.

NAME OF COMPANY: ROBERT EMMETT GINNA NUCLEAR POWER PLANT ONTARIO, NEW YORK

DATE Feb. 8, 1968

TO: Those concerned with this project *

IN REPLY TO YOURS OF

FROM: Paul E. France

COPY TO:

My report covering the initial pumping of NO-OX-ID® CM Casing Filler - Nuclear Grade at the Ginna Brookwood Project is enclosed. Its intent is to keep all interested parties abreast of developments.

Paul E. France

Paul E. France
Product Manager
Maintenance and Production Coatings

PEF:ej
Enc.

*

Mr. Ross Lukens Mr. Charles Huston Mr. John Gilbert Mr. Don Lindsey	Bechtel Corp. " " "	% Ginna Nuclear Power Station Lake and Ontario Center Roads Box 157 Ontario Center, New York 1452(
Mr. James Hood	Gilbert Associates	"
Mr. Ed Cantabene	Rochester Iron & Metal Co.	"
Mr. David Tate	Westinghouse Electric Corp.	"
Mr. R. Kaprowski Mr. Jack Boniface Mr. John Arthur	Rochester Gas & Electric Corp. " "	89 East Avenue Rochester, New York 14604
Mr. [illegible]. M. Brown	Jos. T. Ryerson & Son, Inc.	P. O. Box 8000-A Chicago, Illinois 60680

POOR ORIGINAL

POOR ORIGINAL 9-21-66

SHEET No 1

Rock Anchors

Hole	Tendon	Placed	Grout	Tensioned	Grout No 2
1	77	8-16-66	8-16-66	10-1-66	10-21-66
2	65	8-12-66	8-12-66	9-7-66	9-30-66
3	71	8-12-66	8-12-66	9-9-66	9-30-66
4	6	8-15-66	8-15-66	9-8-66	9-30-66
5	60	8-15-66	8-15-66	9-9-66	9-30-66
6	69	8-12-66	8-12-66	9-7-66	9-30-66
7	44	8-15-66	8-15-66	9-9-66	9-30-66
8	4	8-12-66	8-12-66	9-8-66	9-30-66
9	70	8-15-66	8-15-66	9-12-66	9-30-66
0	64	8-12-66	8-12-66	9-7-66	9-30-66
11	68	8-12-66	8-12-66	9-12-66	9-30-66
12	62	8-12-66	8-12-66	9-8-66	9-30-66
13	5	8-15-66	8-15-66	9-22-66	9-30-66
14	72	8-15-66	8-15-66	9-8-66	9-30-66
[illegible]	63	8-16-66	8-16-66	9-22-66	9-30-66
[illegible]	66	8-15-66	8-15-66	9-8-66	9-30-66
17	75	8-15-66	8-15-66	9-22-66	9-30-66
18	46	8-15-66	8-15-66	9-7-66	9-8-66
19	45	8-15-66	8-15-66	9-26-66	9-30-66
20	84	8-16-66	8-16-66	9-8-66	9-30-66
1	102	8-16-66	8-16-66	9-26-66	9-30-66
22	93	8-16-66	8-16-66	9-8-66	9-30-66
23	92	8-16-66	8-16-66	9-27-66	9-30-66
24	87	8-16-66	8-16-66	9-8-66	9-30-66
25	16	8-16-66	8-16-66	9-27-66	9-30-66
26	100	8-16-66	8-16-66	9-8-66	9-30-66
27	8A	9-20-66	9-20-66	10-6-66	10-20-66
28	129	9-9-66	9-9-66	9-27-66	10-5-66
29	56A	9-20-66	9-20-66	10-6-62	10-20-66
30	43A	9-20-66	9-23-66	10-14-66	10-20-66
31	96	9-9-66	9-9-66	9-27-66	10-5-66
32	95	9-19-66	9-20-66	10-6-66	10-20-66
3	28A	9-20-66	9-20-66	10-14-66	10-20-66
4	107	9-9-66	9-9-66	9-27-66	10-5-66
35	39A	9-20-66	9-20-66	10-14-66	10-20-66
36	146	9-19-66	9-20-66	10-6-66	10-20 66

POOR ORIGINAL 9-21-66

SHEET No 2

ROCK ANCHORS

HOLE	TENDON	PLACED	GROUT	TENSIONED	GROUT No 2
37	26	9-20-66	9-20-66	10-14-66	10-20-66
38	133	9-9-66	9-9-66	9-27-66	10-5-66
39	42A	9-20-66	9-23-66	10-14-66	10-20-66
40	115	9-19-66	9-20-66	10-14-66	10-20-66
41	17	9-20-66	9-23-66	10-14-66	10-21-66
42	111	9-9-66	9-9-66	9-27-66	10-5-66
43	2	9-20-66	9-23-66	10-14-66	10-20-66
44	41	9-19-66	9-20-66	10-14-66	10-20-66
[illegible]	51A	9-20-66	9-23-66	10-18-66	10-20-66
46	134	9-9-66	9-9-66	9-27-66	10-5-66
47	20	9-23-66	9-23-66	10-18-66	10-20-66
48	158	9-19-66	9-20-66	10-5-66	10-20-66
49	48	9-23-66	9-23-66	10-11-66	10-20-66
50	114	9-12-66	9-12-66	9-28-66	10-5-66
[illegible]	29	9-24-66	9-24-66	10-11-66	10-20-66
[illegible]	120	9-19-66	9-20-66	10-5-66	10-20-66
53	34	9-23-66	9-23-66	10-11-66	10-20-66
54	121	9-12-66	9-12-66	9-28-66	10-5-66
55	9A	9-24-66	9-24-66	10-11-66	10-20-66
56	140	9-19-66	9-20-66	10-5-66	10-20-66
57	15A	9-23-66	9-24-66	10-11-66	10-20-66
58	106	9-12-66	9-12-66	9-28-66	10-5-66
59	52A	9-23-66	9-24-66	10-11-66	10-20-66
60	154	9-19-66	9-19-66	10-5-66	10-20-66
61	1A	9-23-66	9-24-66	10-8-66	10-20-66
62	127	9-12-66	9-12-66	9-28-66	10-5-66
63	3A	9-23-66	9-24-66	10-8-66	10-20-66
64	136	9-19-66	9-20-66	10-5-66	10-20-66
65	54A	9-24-66	9-24-66	10-8-66	10-20-66
66	118	9-12-66	9-12-66	9-28-66	10-5-66
67	14A	9-24-66	9-24-66	10-8-66	10-20-66
68	40	9-23-66	9-23-66	10-5-66	10-20-66
V [illegible]	55A	9-24-66	9-24-66	10-8-66	10-20-66
70	126	9-12-66	9-12-66	9-28-66	10-5-66
V 71	21A	9-24-66	9-24-66	10-8-66	10-20-66
72	152	9-19-66	9-19-66	10-5-66	10-20-66

SHEET No 3 POOR ORIGINAL 9-21-66

ROCK ANCHORS

HOLE	TENDON	PLACED	GROUT	TENSIONED	GROUT No. 2.
73	27A	9-24-66	9-24-66	10-8-66	10-20-66
74	112	9-12-66	9-12-66	9-28-66	10-5-66
V 75	19A	9-26-66	9-26-66	10-6-66	10-20-66
76	122	9-19-66	9-19-66	10-5-66	10-5-66
V 77	10A	9-26-66	9-26-66	10-6-66	10-20-66
78	101	9-12-66	9-12-66	9-28-66	10-5-66
79	149	9-26-66	9-26-66	10-17-66	10-20-66
80	110	9-19-66	9-19-66	10-5-66	10-5-66
X 81	7A	9-26-66	9-26-66	10-7-66	10-21-66
82	109	9-12-66	9-12-66	9-28-66	10-5-66
83	12A	9-26-66	9-26-66	10-17-66	10-20-66
84	148	9-19-66	9-19-66	10-5-66	10-5-66
X 85	18A	9-26-66	9-26-66	10-7-66	10-20-66
86	130	9-12-66	9-12-66	9-29-66	10-5-66
87	24	9-16-66	9-16-66	10-4-66	10-5-66
88	53A	9-26-66	9-26-66	10-7-66	10-20-66
89	147	9-13-66	9-13-66	9-29-66	
90	105	9-16-66	9-16-66	10-4-66	
V 91	11A	9-26-66	9-26-66	10-7-66	
92	58	8-11-66	8-11-66	9-7-66	
93	143	9-16-66	9-16-66	10-4-66	
94	13A	9-26-66	9-26-66	10-7-66	
95	22	9-13-66	9-13-66	9-29-66	10-20-66
96	138	9-16-66	9-16-66	10-4-66	
97	35A	9-26-66	9-26-66	10-13-66	
98	132	9-13-66	9-13-66	9-29-66	
99	85A	9-28-66	9-29-66	10-13-66	
100	104	9-16-66	9-16-66	10-4-66	
101	80A	9-28-66	9-29-66	10-13-66	
102	99	9-13-66	9-13-66	9-29-66	
103	88A	9-28-66	9-29-66	10-13-66	
104	156	9-19-66	9-19-66	10-4-66	
105	86A	9-28-66	9-29-66	10-13-66	10-20-66
106	135	9-13-66	9-14-66	9-29-66	10-19-66
107	81A	9-28-66	9-29-66	10-13-66	10-19-66
108	125	9-16-66	9-16-66	10-4-66	10-19-66

SHEET No. 4. POOR ORIGINAL 9-21-66

ROCK ANCHORS

HOLE	TENDON	PLACED	GROUT	TENSIONED	GROUT No. 2.
109	57A	9-28-66	9-29-66	10-13-66	10-19-66
110	131	9-13-66	9-14-66	9-29-66	
111	86	9-16-66	9-16-66	10-4-66	
112	33A	9-28-66	9-29-66	10-13-66	
113	61	8-11-66	8-11-66	9-7-66	
114	153	9-16-66	9-16-66	10-4-66	
115	83A	9-28-66	9-29-66	10-13-66	
116	36	9-13-66	9-14-66	9-29-66	
117	82A	9-28-66	9-29-66	10-13-66	
118	25	9-16-66	9-16-66	10-4-66	
119	145	9-13-66	9-14-66	9-30-66	
120	89A	9-28-66	9-29-66	10-13-66	
121	155	9-16-66	9-16-66	10-3-66	10-21-66
122	144	9-13-66	9-14-66	9-30-66	
123	73A	9-29-66	9-29-66	10-12-66	
124	150	9-16-66	9-16-66	10-3-66	
125	67A	9-29-66	9-29-66	10-18-66	
126	142	9-14-66	9-14-66	9-30-66	
127	94A	9-29-66	9-29-66	10-18-66	
128	137	9-16-66	9-16-66	10-3-66	10-19-66
129	91A	9-29-66	9-29-66	10-18-66	
130	159	9-14-66	9-14-66	9-30-66	
131	90A	9-29-66	9-29-66	10-8-66	
132	108	9-16-66	9-16-66	10-3-66	
133	47A	9-29-66	9-29-66	10-8-66	
134	124	9-14-66	9-14-66	9-30-66	
135	50A	9-29-66	9-29-66	10-8-66	
136	23	9-16-66	9-16-66	10-3-66	
137	49A	9-29-66	9-29-66	10-7-66	
138	119	9-14-66	9-14-66	9-30-66	
139	128	9-14-66	9-16-66	10-3-66	
140	76A	9-29-66	9-29-66	10-7-66	
141	139	9-14-66	9-14-66	9-30-66	
142	141	9-14-66	9-16-66	10-3-66	
143	74A	9-29-66	9-29-66	10-7-66	
144	113	9-14-66	9-14-66	9-30-66	10-19-66

SHEET No 5. 9-21-66

ROCK ANCHORS

HOLE	TENDON	PLACED	GROUT	TENSIONED	GROUT No 2
145	123	9-14-66	9-16-66	10-3-66	10-19-66
146	78A	9-29-66	9-29-66	10-7-66	
147	79	8-19-66	8-19-66	9-30-66	
148	103	9-14-66	9-16-66	10-3-66	
149	30A	9-20-66	9-20-66	10-7-66	
150	117	9-14-66	9-14-66	10-1-66	
151	37A	9-20-66	9-20-66	10-17-66	
152	116	9-14-66	9-16-66	10-3-66	
153	97	9-16-66	9-16-66	10-1-66	
154	151	9-14-66	9-14-66	10-1-66	
155	38A	9-20-66	9-20-66	10-1-66	
156	160	9-14-66	9-14-66	10-1-66	
157	32	9-20-66	9-20-66	10-1-66	
158	157	9-14-66	9-14-66	10-1-66	10-19-66
159	31A	9-20-66	9-20-66	10-1-66	10-19-66
160	59	8-16-66	8-16-66	9-8-66	10-19-66
		T: 116	T: 109	TOTAL: 21	TOTAL: 1

TENDONS ON SITE 119

POOR ORIGINAL

-26-

INSTALLATION AND TENSIONING SEQUENCE

INSTALLATION

9-9-66	28	9-16-66	121	9-24-66	69
9-9-66	31	9-16-66	118	9-24-66	71
9-9-66	34	9-16-66	114	9-24-66	73
9-9-66	38	9-16-66	111	9-26-66	75
9-9-66	42	9-16-66	108	9-26-66	77
9-9-66	46	9-19-66	104	9-26-66	79
9-12-66	50	9-16-66	100	9-26-66	81
9-12-66	54	9-16-66	96	9-26-66	83
9-12-66	58	9-16-66	93	9-26-66	85
9-12-66	62	9-16-66	90	9-26-66	88
9-12-66	66	9-16-66	87	9-26-66	91
9-12-66	70	9-19-66	84	8-11-66	92
9-12-66	74	9-19-66	80	9-26-66	94
9-12-66	78	9-19-66	76	9-26-66	97
9-12-66	82	9-19-66	72	9-28-66	99
9-12-66	86	9-23-66	68	9-28-66	101
9-13-66	89	9-20-66	64	9-28-66	103
9-13-66	95	9-19-66	60	9-28-66	105
9-13-66	98	9-19-66	56	9-28-66	107
9-13-66	102	9-19-66	52	9-28-66	109
9-13-66	106	9-19-66	48	9-28-66	112
9-13-66	110	9-19-66	44	8-11-66	113
9-13-66	116	9-19-66	40	9-28-66	115
9-13-66	119	9-19-66	36	9-28-66	117
9-13-66	122	9-19-66	32	9-28-66	120
9-14-66	126	9-20-66	29	9-29-66	123
9-14-66	130	9-20-66	27	9-29-66	125
9-14-66	134	9-20-66	33	9-29-66	127
9-14-66	138	9-20-66	35	9-29-66	129
9-14-66	141	9-20-66	37	9-29-66	131
9-14-66	144	9-20-66	39	9-29-66	133
9-14-66	150	9-20-66	41	9-29-66	135
9-16-66	153	9-20-66	43	9-29-66	137
9-14-66	156	9-20-66	45	9-29-66	140
9-14-66	158	9-23-66	47	9-29-66	143
9-14-66	154	9-23-66	49	9-29-66	146
9-14-66	152	9-23-66	51	8-16-66	147
9-14-66	148	9-23-66	53	9-20-66	149
9-14-66	145	9-23-66	55	9-20-66	151
9-14-66	142	9-23-66	57	9-20-66	155
9-14-66	139	9-23-66	59	9-20-66	157
9-16-66	136	9-23-66	61	9-20-66	159
9-16-66	132	9-23-66	63	8-16-66	160
9-16-66	128	9-24-66	65	8-12-66	11
9-16-66	124	9-24-66	67	8-12-66	6
8-12-66	2	8-12-66	12	8-15-66	16
8-12-66	3	8-15-66	5	8-15-66	7
8-12-66	8	8-15-66	13	8-15-66	9
8-12-66	10	8-15-66	14	8-15-66	4
8-15-66	17	8-16-66	20	8-16-66	24
8-15-66	18	8-16-66	21	8-16-66	26
8-15-66	19	8-16-66	22	8-16-66	15
8-16-66	1	8-16-66	23	8-16-66	25
9-23-66	30				

TENSION

18	9-7-66	54	9-28-66	132	10-3-66
10	9-7-66	58	9-28-66	128	10-3-66
92	9-7-66	62	9-28-66	124	10-3-66
113	9-7-66	66	9-28-66	121	10-3-66
147	9-30-66	70	9-28-66	118	10-4-66
2	9-7-66	74	9-28-66	114	10-4-66
6	9-7-66	78	9-28-66	111	10-4-66
14	9-8-66	82	9-28-66	108	10-4-66
22	9-8-66	86	9-29-66	104	10-4-66
25	9-8-66	89	9-29-66	100	10-4-66
24	9-8-66	95	9-29-66	96	10-4-66
20	9-8-66	98	9-29-66	93	10-4-66
16	9-8-66	102	9-29-66	90	10-4-66
12	9-8-66	106	9-29-66	87	10-4-66
8	9-8-66	110	9-29-66	84	10-5-66
4	9-8-66	116	9-29-66	80	10-5-66
160	9-8-66	119	9-30-66	76	10-5-66
3	9-9-66	122	9-30-66	72	10-5-66
5	9-9-66	126	9-30-66	68	10-5-66
7	9-9-66	130	9-30-66	64	10-5-66
9	9-12-66	134	9-30-66	60	10-5-66
11	9-12-66	138	9-30-66	56	10-5-66
13	9-22-66	141	9-30-66	52	10-5-66
15	9-22-66	144	9-30-66	48	10-5-66
17	9-22-66	150	10-1-66	44	10-14-66 ?
19	9-26-66	153	10-1-66	40	10-14-66
21	9-26-66	156	10-1-66	36	10-6-66
23	9-27-66	158	10-1-66	32	10-6-66
23	9-27-66	1	10-1-66	28	10-6-66
25	9-27-66	154	10-1-66	27	10-6-66
31	9-27-66	152	10-3-66	33	10-14-66
34	9-27-66	148	10-3-66	35	10-14-66
38	9-27-66	145	10-3-66	37	10-14-66
42	9-27-66	142	10-3-66	39	10-14-66
46	9-27-66	139	10-3-66	41	10-14-66
50	9-28-66	136	10-3-66	43	10-14-66
155	10-1-66	75	10-6-66	63	10-8-66
157	10-1-66	77	10-6-66	65	10-8-66
159	10-1-66	61	10-8-66	67	10-8-66
69	10-8-66	71	10-8-66	73	10-8-66
131	10-8-66	133	10-8-66	135	10-8-66
30	10-14-66	[illegible]	[illegible]	49	10-11-66
51	10-11-66	53	10-11-66	55	10-11-66
57	10-11-66	59	10-11-66	97	10-3-66 ?
99	10-13-66	101	10-13-66	103	10-13-66
105	10-13-66	107	10-13-66	109	10-13-66
112	10-13-66	115	10-13-66	117	10-13-66
120	10-13-66	123	10-12-66		

INSTALLATION AND TENSIONING SEQUENCE

INSTALLATION

9-9 28
31
34
38
42
9-9 46
9-12 50
54
58
62
66
70
74
78
82
9-12 86
9-13 90
95
98
102
106
110
116
119
9-13 122
9-14 126
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141
144
9-14 150
9-16 153
156
158
9-16 154
9-14 152
148
145
142
9-14 139
9-16 136
132
128
9-16 124

9-16 121
118
114
111
9-16 108
9-19 104
100
96
93
90
87
84
80
76
72
68 NO
64
60
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52
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44
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36
9/19 32
9/20 29
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39 30
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9/20 45
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9/20 149
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155 — 7 8
157 — 8 67 10/1
9/20 159 — 9
160

TENSION

18
10
22
[illegible]
[illegible] NO
2
6
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22
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9-22 13
9-22 15
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9-26 19
9-26 21
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70 9-28
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98 9/29
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130 9/30
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144
1 [illegible] — 10-1
2 [illegible] —
3 [illegible] —
4 158 —
5 1 —
6 154 —
152
148
145 10/3
142
139
136

132 10/3
128
124
121
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114 10/4
111
108
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RYERSON METALOGICS
JOSEPH T. RYERSON & SON, INC.

POST-TENSIONING FIELD RECORD

BECHTEL CORP. 21T341891. ROCK ANCHORS

Date 10-1

1ST STAGE GROUT AT 1

HOLE NO ~~MEMBER~~ #1 CABLE NUMBER 77 LENGTH 33-10½

NUMBER OF WIRES 90 STRESSING END

RAM AREA OF 500 TON PINE JACK = 129.3 SQUARE INCHES

HT# 88-A673- COIL # 177
80-A-690 " 71

SAMPLE

STRESS CONDITIONS FOR ______ % STRESSING	FORCE KIPS	STRESS KSI	GAUGE PRESSURE PSI
INITIAL PRESTRESSING FORCE			
COMPUTED OVERSTRESSING FORCE			6550
BACKOFF FORCE .7 ULT.			5740
MAXIMUM ALLOWABLE OVERSTRESSING FORCE .8 ULT			6550
COMPUTED ELONGATION WHILE OVERSTRESSING		______ IN.	

ELONGATION WHILE OVERSTRESSING MEASURED AS RAM TRAVEL BETWEEN STARTING GAUGE PRESSURE

OF ______ AND FINAL GAUGE PRESSURE OF ______ = ACTUAL ELONGATION X

$$\left(\frac{\text{FINAL PRESSURE (GAUGE)} - \text{STARTING PRESSURE (GAUGE)}}{\text{FINAL PRESSURE (GAUGE)}}\right) = \underline{\qquad}$$

FIELD RECORD ______ TON ______ JACK

THEORETICAL GAUGE PRESSURE	ACTUAL	RAM POSITION	INCREMENT OF ELONGATION	NET ELONGATION
500	500	1 3/16		
1500		1 5/16		
2500		1 1/2		
3500		1 3/4		
4500		2"		
5500		2 1/4		
6500		2 1/2		
LIFT-OFF	5800			
		TOTAL SHIMS USED → 1 3/2		

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84 BOX B

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HISTORICAL RE-CAP

SUBJECT - ROCK ANCHORS INSTALLATION 10-31-66

PLACING OF ROCK ANCHORS STARTED ON 8-12-66 - VARIOUS TESTS WERE UNDERTAKEN - ALSO INSPECTION - APPROX. 28 ROCK ANCHORS WERE REJECTED - VARIOUS CAUSES WERE - ANCHOR HEADS ON WITH THREADS WRONG WAY - TENDONS TOO LONG, - SOME BUTTON HEADS SEVERELY CRACKED.

AFTER RYERSON SHOP INSPECTION UNDERSTOOD THESE PROBLEMS THERE WERE NO FURTHER DISCREPANCES, OR RELATIVELY FEW

A 50 TON TEST WAS MADE ON HOLE No. 18, TENDON No. 46 THESE TESTS ARE SHOWN IN DETAIL IN THE FILE - THIS TOOK PLACE ON 9-6-66

TENSIONING SEQUENCE WAS AN IMPORTANT ITEM IN THIS PROCEEDURE AND WAS CARRIED OUT ACCORDING TO PREDETERMINED HOLE NUMBERS - ALL TENSIONING WAS COMPLETED WITH A 500 TON JACK - USING TWO TEST GAUGES CALIBRATED IN 1,000 LBS TO 10,000 LBS - RAM AREA 129.3 □"

1ST STAGE GROUT IS CRITICAL AND IT WAS FOUND THAT 4½ GALLONS OF WATER TO A BAG OF CEMENT WITH SPECIAL ADDITIVE GAVE THE DESIRED MIXTURE 4,000# - CUBES PREVIOUSLY ON THE FIRST 20 WERE BREAKING UNDER 4,000

OBSERVATIONS

SCREWING ON THE COUPLING AS WELL AS UNSCREWING WAS A BIG "TIME WASTER". THIS WAS CRUDE IN THE EXTREME AS TWO WORKMAN LABORIOUSLY USED A PIECE OF No. 6 RE-BAR AS A LEVER FOR THIS OPERATION.

CENTERING THE ANCHOR HEAD IN RELATION TO THE TOP PLATE WAS ANOTHER PROBLEM WHICH WAS PARTIALLY SOLVED BY MEASURING AROUND COUPLING AT 4 PLACES FROM EDGE OF TOP PLATE - HERE AGAIN, A LITTLE BIT OF ENGINEERING WITH JACK SCREWS WOULD HAVE HELPED THIS PROBLEM.

PLACING THE SHIMS WAS DIFFICULT - SOME GAPS WERE SHOWING VERTICALLY AS THESE WERE NOT PLACED SNUG TO EACH OTHER - IN HALVES - THE SHIMS THEMSELVES COULD HAVE BEEN DE-BURRED OR TUMBLED AS THESE VERY OFTEN REQUIRED HAMMERING TO LEVEL SHARP EDGES

2ND STAGE GROUT WAS A MESSY PROBLEM ON THE RING, BEAM AND TOP PLATES - MORE TIME AND HOUSE KEEPING COULD HAVE BEEN SAVED, HAD THEY BEEN CLEANED PROPERLY

ROCK ANCHOR INSTALLATION (CONT.D.)

IMEDIATELY.

AFTER 1ST STAGE GROUT IT WAS FOUND THAT SCUM FORMING ON TOP OF ROCK ANCHOR HEAD : IF LEFT FOR A PERIOD OF TIME (SAY TWO WEEKS) SUNK, DOWN THE HOLE TRAPPED IN THE TENDONS, THEN SOLIDIFIED AND SO PREVENTED : PUMP HOSE FROM REACHING BOTTOM OF HOLE FOR 2nd STAGE GROUT.

INITIALLY, ROCK ANCHOR HOLES SHOULD BE DRILLED DEEP ENOUGH TO HOUSE GROUT PIPE PLUS FOUR FEET, ; A FEW ROCK ANCHORS WERE NOT RESTING ON TOP PLATE, THEN WHEN THE COUPLING WAS BEING SCREWED ON THE ANCHOR HAD A TENDENCY TO TURN WITH THE COUPLING

160 ROCK ANCHORS WERE TENSIONED COMPLETE BY 10-18-66. 2nd STAGE GROUT AND CLEAN UP WAS COMPLETE BY 10-31-66

(X) ADDITIVE WAS :-

INTRUSION AID

PREPAKT CONCRETE CO.

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Rock anchors—some design considerations

G.S. Littlejohn (UK)

INTRODUCTION

Although rock anchors have been used successfully for many years in connection with the prestressing of dams, roof strata control and slope stabilisation, there are still many questions concerning their design today that cannot be answered. The use of rock anchors in construction with particular reference to excavation engineering is now increasing at a greater rate than ever before and this combined with the trend towards higher load capacities often associated with poor quality rock has led to a growing need to establish reliable design formulae with realistic safety factors.

When a grouted anchor fails, it must be by one of the following modes:

(a) Failure of the rock mass
(b) Failure of the grout/rock bond
(c) Failure of the grout/steel bond
(d) Failure of the steel tendon

and in order to determine the actual safety factor for the anchor, consideration must be given to all of these aspects.

The purpose of this paper is to describe some current anchor design concepts, and then question the validity of the basic assumptions in order to highlight topics for further discussion and investigation.

OVERALL STABILITY OF THE ANCHOR

The assessment of the overall stability of an anchor is carried out in order to ensure that failure of the rock mass surrounding the anchor does not occur.

Where it is possible to place an anchor in a perfectly homogeneous rock mass this aspect of the design would appear to present little difficulty in practice. However, in many cases heterogeneous rock masses containing joints and fissures of unknown geometry restrict the application of the simple methods described below and necessitate modifications by the experienced rock mechanics engineer using his engineering judgement.

Invariably, an inverted cone of rock is considered to fail in the simple cases but the angle and position of the apex of the cone with respect to the grouted or fixed anchor length are chosen differently by various engineers in different countries (see Table 1).

Table 1.

Geometry of inverted cone		
Included angle	Position of apex	Source
60°	Base of anchor	USA–Hilf [1973]
90°	Base of anchor	USA–White [1973]
90°	Base of anchor	Britain–Banks [1955]
90°	Base of anchor	Britain–Parker [1958]
*60°-90°	Middle of grouted fixed anchor, where load is transferred by bond	Britain–Littlejohn [1972] (see Figure 1)
	Base of anchor where load is transferred by end wedges or plate	
90°	Middle of anchor for bond. Base of anchor for wedges or plate	Germany–Stocker [1973]
60°	Base of Anchor	Canada–Saliman and Schaefer [1968]
90°	Base of Anchor	Canada–Brown [1970]
90°	Top of grouted fixed anchor, or	Australia–Standard CA35 [1973]
60°	Base of Anchor	
90°	Base of Anchor	Czechoslovakia–Hobst [1965]

*60° employed primarily in soft, heavily fissured or weathered rock mass

The uplift capacity is normally equated to the weight of the specified rock cone, and where the ground is saturated and beneath the water-table, the submerged weight of rock is used. If the anchor is inclined then the same geometry is often applied and the effect of groups of anchors involving interaction is to produce a flat vertical plane at the interface of adjoining cones (Figure 2). As the spacing for a single line of anchors reduces further a

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simple continuous wedge failure in the rock is ultimately assumed.

Often no account is taken of the overburden pressure from unconsolidated deposits or the shear strength of the rock at the failure plane. Of the engineers mentioned in Table 1 only Hilf takes direct account of the shear strength by stating that a value of 500 lb/ft^2 (24 kN/m^2) may be allowed for in design. Little data is available on the safety factors employed when analysing the weight of rock in the assumed pull-out zone, but it is known that some designers apply safety factors of 1.6 to 2 while others equate the weight of rock to the required anchor working load and assume that other rock parameters ignored in the calculation e.g. shear strength, will produce a sufficiently large safety factor in the design as a matter of course. The above analyses only apply to anchors at angles below the horizontal and obviously the shear strength of the rock becomes the major factor when dealing with overhead anchors. In roof strata control anchors are generally of low capacity and rock bolts, using quick-setting resins, are often installed on a trial and error basis, this being the quickest and cheapest way to 'stitch' together a newly exposed rockface. The length of rock bolt is often decided on the basis of observed spalling by the mining engineer and in general the larger the excavation the longer the length of bolt. Bolt lengths of 3-6 m are very common but as a further guide Pender et al [1963] suggest $L > 3$ times width of jointed blocks. Thereafter the maximum spacing between bolts is taken as L/2 approximately, in order to provide a continuous zone of compression in the rock. This approach to spacing has been described by Beomonte [1961] Pender et al [1963] and Hilf [1973].

There is a dearth of data on anchor failures in the rock mass but a set of tests which provides some results on the overall stability aspect is presented by Saliman and Schaefer [1960] in which they describe the failure of grouted bars on the Trinity Clear Creek 230 killovolt transmission line. Four tests were carried out on deformed reinforcing bars grouted into 2¾ in. (70 mm) diameter holes to a depth of 5 ft (1.52 m) in a sedimentary rock which was mostly shale. In all cases, failure occurred when a block of grout and rock pulled-out. At failure the propagation of cracking to the rock surface gave an indication of the cone of influence (Figure 3).

Assuming a bulk density of 125 lb/ft^3 (2 Mg/m^3) for the rock analysis of the failure loads suggests that the 90° cone from the middle of the anchor length gives very conservative results with safety factors ranging from about 7.4 to 23.5, while the 90° cone from the base gives safety factors of 0.9 to 2.9.

REMARKS

Bearing in mind the engineer's desire to optimise any design there is little evidence to substantiate the current approaches shown in Table 1. While it is appreciated that several million tons of working anchorage capacity has been provided to date without serious failure, it is considered that much effort should now be expended in the form of field testing in a wide range of rocks to study the shape and position of the rock 'wedges' mobilised at failure. The programme should accommodate single anchors and groups tested over a range of inclinations. Some standardisation on safety factors for temporary and permanent anchors is also desirable together with agreement on what allowances should be made for unconsolidated overburden and the upper layers of weathered rock.

In general it is clear that in order to calculate the anchorage length accurately with a known safety factor, it is necessary to utilise all the tools of rock mechanics, e.g. detailed mapping of joints, assessment of joint filling material properties, thicknesses and dips of bedding planes and other inhomogeneities. This approach is currently the rare exception rather than the rule and discussion should be held on what type of site investigation and field data are required to facilitate rock anchor design.

BOND BETWEEN THE CEMENT GROUT AND ROCK

Basically there are two types of injection rock anchor being constructed today:

(1) Straight shaft
(2) Single or multi underream

The load transfer mechanism for these two categories is completely different. The straight shaft anchor relies mainly on the development of skin friction or shear in the region of the rock/grout interface while the underreamed anchor depends more on the mechanical interlocking of the grout cones and the rock. This section concentrates on the straight shaft anchor.

Estimation of the magnitude and distribution of the bond strength mobilised along the straight shaft rock anchor is without doubt a major problem facing the design engineer. It is current practice to assume an equivalent uniform distribution for bond stress or skin friction along the fixed anchor i.e.

$$L = \frac{P}{\pi \times D \times \sigma_{skin}}$$

where L = fixed anchor length
P = required anchor load
D = effective anchor diameter
σ_{skin} = value of working bond stress

Where shear strength tests are carried out on representative samples of the rock mass, the maximum average working bond stress at the fixed anchor/rock interface should not exceed the minimum shear strength divided by the relevant safety factor (normally not less than 2). According to current usage this approach applies primarily to soft rocks where the uniaxial compressive strength is less than 1000 lbf/in^2 (7 N/mm^2), and the holes have been drilled using a rotary percussive technique. In the absence of shear strength data or field pull-out tests, the ultimate bond stress is often taken as 1/10 of the unconfined compressive strength of massive rocks up to a maximum value of σ_{skin} = 600 lbf/in^2 (4.2 N/mm^2), where the crushing strength of the cement grout is equal to or greater than 6000 lbf/in^2 (42 N/mm^2). Applying an apparent safety

factor of 3, which is conservative bearing in mind the lack of relevant data, the working bond stress is limited to 200 lbf/in² (1.4 N/mm²). A minimum fixed anchor length of 3 m is also generally recommended.

It is however more common to find the magnitude of bond stress simply being assessed by experienced engineers, and the value adopted for working bond stress normally lies in the range 50-200 lbf/in² (0.35-1.4 N/mm²). In this connection Koch of BBR Australia recommends bond stresses for three categories of rock (see Table 2).

Table 2.

Rock type	Bond stress	
	lbf/in²	(N/mm²)
Weak	50 – 100	(0.35 – 0.70)
Medium	100 – 150	(0.70 – 1.05)
Strong	150 – 200	(1.05 – 1.40)

The Australian Code CA35–1973 states that a value of 150 lbf/in² (1.05 N/mm²) has been used in a wide range of igneous and sedimentary rocks, and confirms that site testing has permitted values of up to 300 lbf/in² (2.1 N/mm²). Coates [1970] allows a value of 350 lbf/in² (2.4 N/mm²) with a safety factor of 1.75, for a hard coarse grained sandstone.

Care must be taken however before applying the above values since the degree of weathering of the rock whatever its classification is another major factor which affects not only the value of bond stress at failure but the load-deflection relationship during service or test loading. Figure 4 illustrates the latter effect. These results are for square bars grouted into 2 in. (50 mm) diameter holes 4.75 ft (1.45 m) deep, and tested for use on the Currecanti-Midway transmission line. Good and very poor results were produced by the same rock type, Rhyolite Tuff, in sound and weathered conditions respectively. No data is available on grout or rock strengths but it is significant that the equivalent uniform bond stress at maximum jack capacity is scarcely 0.1 N/mm².

In general, few failures are encountered at the rock anchor interface and new work is based on the successful completion of former projects, i.e. former 'working' skin frictions are re-employed or slightly modified depending on the judgement of the designer. In reality however this assumption of uniform distribution of bond is unlikely to be true except for very soft rocks.

Since there is little information available on field anchors reference must be made to investigations into bond in reinforced concrete. Hawkes and Evans [1951] show that theoretically the distribution of shear stress along the surface of an anchored steel reinforcing rod loaded in tension at its exposed end can be expressed in an exponential form:

$$\frac{S_x}{S_o} = e^{-Ax/d} \quad \text{(Figure 5)}$$

where S_o = shear stress at the top of the anchor
S_x = shear stress at a distance X from the top
d = anchor or rod diameter
A = a constant relating axial stress in the rod to bond stress in the anchorage material.

This theory has been developed by Phillips [1970] where he shows that theoretically

$$\frac{S_x \pi d^2}{P} = Ae^{-Ax/d} \quad \text{(Figure 6)}$$

This type of bond distribution has been verified most recently by Coates and Yu [1970] whose conclusions are sufficiently close to those of Hawkes and Evans to suggest that their approach is applicable to rock anchorages. Coates and Yu used a finite element method to calculate the stress along the anchor in a cylindrical hole in a triaxial stress field. They showed that the stress distribution is dependent upon the ratio of the elastic moduli of the anchor (E_a) and the rock (E_r) [Figure 7]. Comparisons of Figure 6 and 7 show the connection between the two analysis if $\frac{E_a}{E_r}$ is proportional to $\frac{1}{A}$.

From Figure 7 it can be seen that a modular ratio of 10 could be taken as being sufficient to give a reasonably even stress distribution. Considering the elastic modulus of grout, values of 3.04 x 10^{+6} lbf/in² (2.1 x 10^{+4} N/mm²) quoted by Phillips for a neat water/cement ratio of 0.4, and 1.45 x 10^{+6} lbf/in² (1.0 x 10^{+4} N/mm²) given by Boyne [1972] for a 0.35 water cement ratio expansion grout, suggest that before an even stress distribution can be assumed, rocks should have elastic moduli in the range 0.15 – 0.30 x 10^{+6} lbf/in² (0.1 – 0.2 x 10^{+4} N/mm²). Using a statistical relationship derived by Judd and Huber [1966] which relates the compressive strength to the elastic modulus

$$S_c \text{ (compressive strength)} = \frac{E}{350}$$

it may be established that the compressive strength should be 850 lbf/in² (6 N/mm²) or less. For the majority of rock anchors installed to date, normal values of $\frac{E_a}{E_r}$ would be in the range of 0.1 to 1 and hence it is suggested that the bond distribution for these ratios apply to many anchorages in rock. Berardi [1967] has carried out an exhaustive series of tests on this aspect, and some typical results in marly limestone are shown in Figure 8. As expected the main results show that the distribution of stress is most uniform for high values of $\frac{E\ grout}{E\ rock}$, but the distribution varies considerably for low values of this ratio i.e. rocks of high elastic modulus.

Berardi concludes that the portion of the fixed anchor which actually transmits the force is independent of the anchorage length, but dependent on its diameter and the engineering properties of the surrounding rock, especially the modulus of elasticity.

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REMARKS

Since the validity of the uniform distribution of bond, which is most commonly assumed by designers is clearly in question, it is recommended that instrumented anchors should be pulled to failure in a wide range of materials whose engineering properties have been fully classified, in order to ascertain which parameters dictate anchor performance. In this way it should be possible in due course to provide more reliable design criteria.

BOND BETWEEN THE CEMENT GROUT AND CABLE

Little information is readily available on this subject related to rock anchors and the general feeling of engineers is that this part of the design is not critical since the fixed anchor length, necessary to mobilise sufficient resistance at the rock/grout interface, usually allows a large safety factor against failure of the grout/steel bond.

In practice it is common to find anchorage or transmission lengths for bars and wires quoted as some number of diameters since this method ensures a constant value of apparent average bond stress for various diameters. It should be borne in mind however that the transmission length varies with grout strength as well as size and type of tendon, and it is still advisable on ocassions to measure experimentally the transmission length for the known site conditions.

The British Code of Practice CP110 [1972] specifies a minimum anchorage length of 100 diameters for plain wire, where the cube strength of the grout is not less than 5000 lbf/in² (35 N/mm²). Bearing in mind the minimum fixed anchor length of 3 m then the Code is satisfied for bars up to 30 mm diameter. For small diameter strand, recommended transmission lengths are given in Table 3. No allowance is apparently made for groups of strands.

Table 3.

Diameter of Strand		Transmission Length	
in.	(mm)	in.	(mm)
0.37	9.3	8	200
0.50	12.5	13	330
0.70	18.0	20	500

The Australian Code [1973] stipulates a maximum value of 150 lbf/in² (1.05 N/mm²) for the bond stress for a clean wire tendon and 300 lbf/in² (2.10 N/mm²) for a clean strand tendon.

With regard to permissible bond stresses for plain and deformed bars in concrete, Table 4 illustrates the values stipulated by the British Code for different grades of concrete. These values are applied to neat cement grouts on occasions.

Table 4.

Characteristic strength of concrete f_{cu}				
lbf/in² (N/mm²)	2850 (20)	3570 (25)	4280 (30)	5720 (40+)
Maximum bond stress				
Plain bar lbf/in² (N/mm²)	171 (1.2)	200 (1.4)	214 (1.5)	272 (1.9)
Deformed bar lbf/in² (N/mm²)	243 (1.7)	272 (1.9)	314 (2.2)	371 (2.6)

For a group of bars, the effective perimeter of the individual bars is multiplied by the following reduction factors.

No. of bars in group	Reduction factor
2	0.8
3	0.6
4	0.4

REMARKS

While there is an appreciable amount of information available concerning the mechanism of bond transfer in the field of reinforced and prestressed concrete, it is considered that much more study is required in the field or rock anchors with particular regard to load transfer in groups of strand cables and the influence of lateral restraint. The use of spacers and centralisers, leading possibly to decoupling, also warrants investigation.

CABLE DESIGN

A designer usually has accurate information on the ultimate strength of the tendon material which he has chosen and having decided on the anchor working load it is a straightforward procedure to apply the required safety factors and arrive at the cross-sectional area of steel required. Basically there are three types of tendon to choose from, namely: bar, wire and strand and as a result of recent developments in prestressing equipment and general ease of handling, strand is increasing in popularity, although for low capacity anchors of limited length bars are most common.

While the market for temporary anchors is now expanding rapidly throughout the world the same cannot be said for permanent anchors where there is a dearth of published information on long term behaviour and we lack a good understanding of stress/strain distribution around the mechanical or grouted zone resisting pull-out. Until these issues are resolved and in order to maintain a steady, but safe growth in the use of anchorages in soils and soft rocks the writer recommends that all permanent anchors and temporary anchors, where the consequences are severe if failure occurs, should be tested to a least 1.5 times the working load. In Britain since all stress levels and factors of safety must be related to the characteristic strength of the prestressing steel (f_{pu}) as described in CP 110, Part 1.

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1972 the above recommendations may be summarised as follows.

Permanent anchors

(including temporary anchors where failure would be very serious e.g. temporary anchors for main cables of a suspension bridge).

Design force (T_ω)	50% f_{pu}
Test force (T_t)	75% f_{pu}
Factor of safety against breaking the cable (S_b)	2.0
Measured factor of safety (S_m)	1.5

It is noteworthy that this recommendation will not only increase the measured safety factor on each anchor but the lower stress levels in the steel will reduce the bond stress required in the fixed anchor zone which should be of interest to those engineers who have experienced bond failure at the grout/tendon interface.

Temporary anchorages

(consequences not severe if failure occurs e.g. temporary anchors for ground preloading or pipe jacking)

T_ω = 62.5% f_{pu}
T_t = 78% f_{pu}
S_b = 1.6
S_m = 1.25

The importance of safety in ground anchors cannot be over-emphasised as it is the post-tensioning operation which pre-tests the anchor thus ensuring its safety. It is considered that ground conditions are never sufficiently homogeneous or predictable to allow engineers to ignore this cheap insurance.

GENERAL CONCLUSIONS

It is clear from the rapid development of grouted anchors that significant savings are being made on contracts pertaining to a wide range of applications but there is a growing need for investment in the form of instrumentation on new anchor contracts which will allow investigation of such important aspects as:

1. Stress/strain distribution around the fixed anchor
2. Long term behaviour
3. Interaction between anchors and the structure being tied, since this affects stability calculations
4. Load-displacement relationships for fixed anchors in different ground conditions, since these relationships influence choice of safety factor which should be related to permissible movement as well as ultimate load.
5. Ground thresholds where anchors stop being economical through poor load holding capacity or difficulties in construction, or endanger safety through loss of prestress with time.

Only in this way is it believed that permanent anchorages will continue to develop safely and become fully utilised over a wide range of applications.

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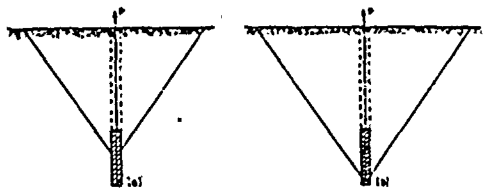


Figure 1.

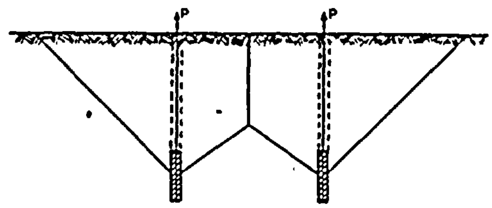


Figure 2.

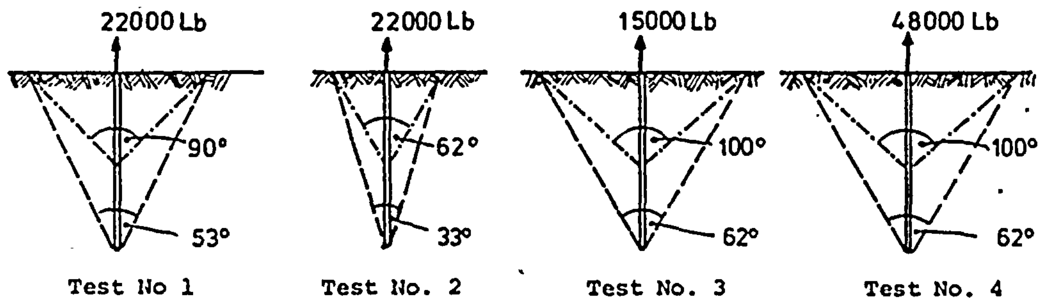


Figure 3. (Based on test results at Trinity Clear Creek).

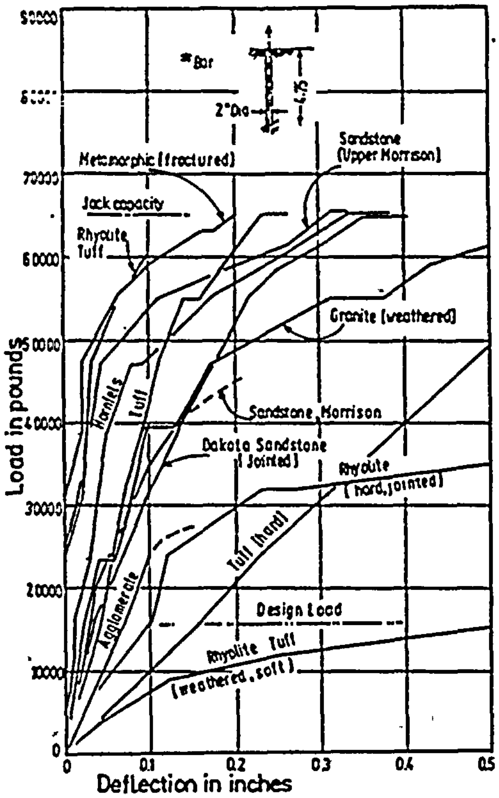


Figure 4. (After Saliman & Schaefer) Currecanti–midway transmission line

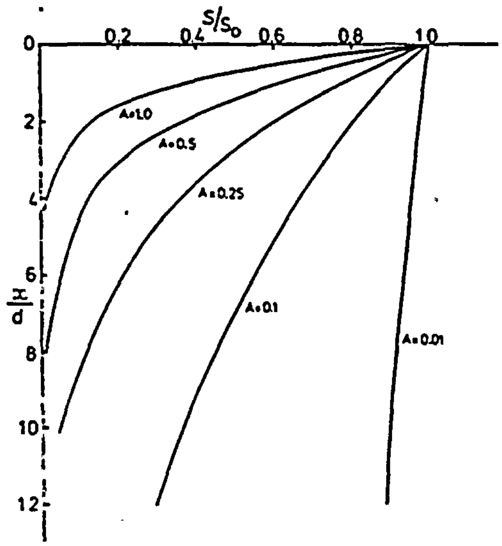


Figure 5. (After Hawkes and Evans) Theoretical stress distribution along an anchor.

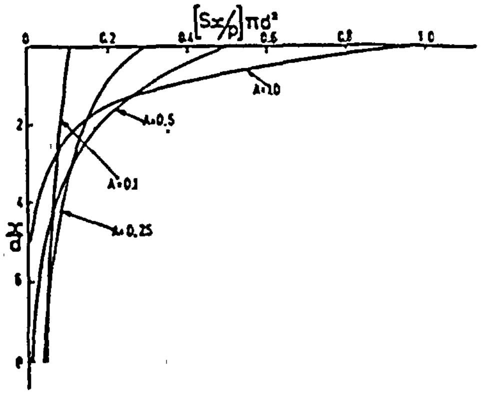


Figure 6. (After Phillips). Load distribution along an anchorage assuming $A\frac{L}{D}$ is large.

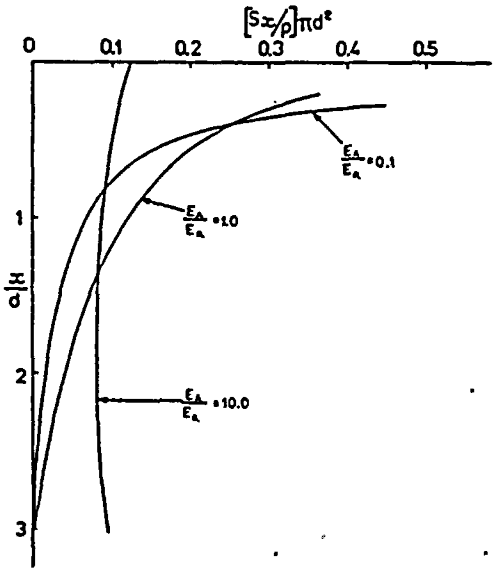
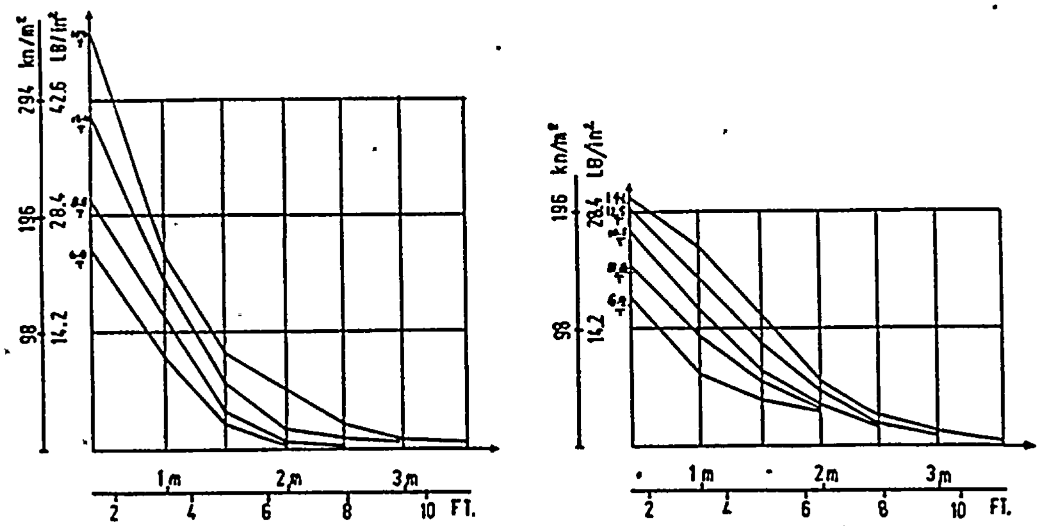


Figure 7. (After Coates and Yu). Load distribution along an anchorage.



<u>Distribution of skin friction along fixed anchor length</u>

Type AC - Diameter = 120 mm; Length = 5.9 m

Type AL - Diameter = 120 mm; Length = 11.0 m

Figure 8. (After Berardi). Distribution of skin friction along fixed anchor length.

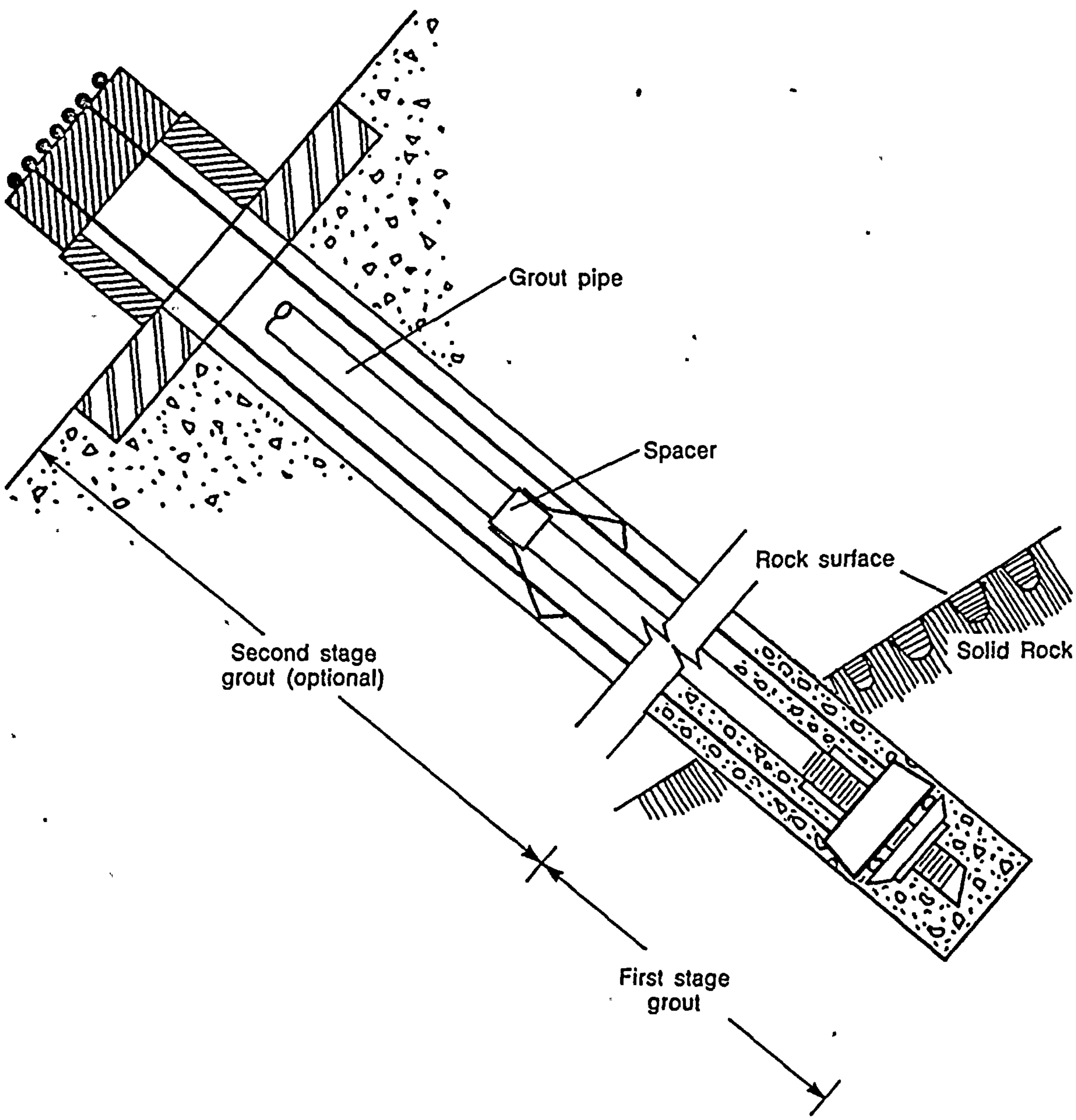
Post-Tensioning Manual

POST-TENSIONING INSTITUTE

POST-TENSIONING INSTITUTE
1701 Lake Avenue, Suite 375, Glenview, Illinois 60025
(312) 729-9660

BBRV WIRE ROCK ANCHORS

Anchor Capacity (No. of wires, max.)		30	46	62	90	116	144	170	208
Bearing Plate Size (inches)	Round (dia.)	10½	13	16	18½	20	22	23½	26
	Square	9¼	11½	14½	16	17½	19½	20½	23
Trumpet O.D. (inches)		4	4½	5	5½	6	6½	7	7½
Stressing Anchor Head Diameter (inches)		4½	5½	6¼	7⅞	8¼	9	9⅜	10⅝
Bore-Hole Diameter (inches)		4	4½	5	5½	6	6½	7	7½
Fixed Rock Anchor Head Diameter (inches)		3¼	3¾	4¼	4¾	5¼	5¾	6¼	6¾



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Chapter 4

Tentative Recommendations for Prestressed Rock and Soil Anchors

4.1 SCOPE

This chapter has been prepared to provide guideance in the application of permanent and temporary prestressed rock and soil anchors utilizing high strength prestressing steel. It represents the present state of the art and outlines what are considered the most practical procedures for installation of prestressed rock and soil anchors.

4.2 DEFINITIONS

Permanent Anchor: Any prestressed rock or soil anchor for permanent use. Generally more than a 3-year service life.

Temporary Anchor: Any prestressed rock or soil anchor for temporary use. Generally less than a 3-year service life.

Downward Sloped Anchor: Any prestressed anchor which is placed at a slope greater than 5° below the horizontal.

Upward Sloped Anchor: Any prestressed anchor which is placed at a slope greater than 5° above the horizontal.

Horizontal Anchor: Any prestressed anchor which is placed at a slope between ±5° with the horizontal.

Anchor Grout: *(Also known as primary injecton)* Portland Cement grout that is injected into the anchor hole to provide anchorage at the non-stressing end of the tendon. In case of a sheathed anchor, also included in the grout between the sheath and the anchor hole. Resins are also used as anchor grout. Their properties are not covered by these tentative recommendations.

Corrosion Protective Filler Injection: *(Also known as secondary injection)* Material that is injected into the anchor hole to cover the stressing length of the prestressed anchor, providing corrosion protection to the high strength steel. This material may be grout or other suitable materials.

Consolidation Grout: Portland cement grout that is injected into the hole prior to inserting the tendon to waterproof or otherwise improve the rock surrounding the hole.

Inserting: The physical placement of the anchor tendon in the prepared hole.

Lift-Off Check: Checking the force in the prestressed anchor at any specified time with the use of a hydraulic jack.

Proof Load: Initial prestressing per anchor, representing the proof loading.

Transfer (lock-off) Load: Prestressing force per anchor after the proof loading has been completed and immediately after the force has been transferred from the jack to the anchorage.

Design Load: Prestressing force per anchor after allowance for time dependent losses.

Tendon: The complete assembly consisting of anchorage and prestressing steel with sheathing when required.

Anchorage: The means by which the prestressing force is permanently transmitted from the prestressing steel to the rock or earth.

Prestressing Steel: That element of a post-tensioning tendon which is elongated and anchored to provide the necessary permanent prestressing force.

Coating: Material used to protect against corrosion and/or lubricate the prestressing steel.

Sheathing: Enclosure around the prestressing steel to avoid temporary or permanent bond between the prestressing steel and the surrounding grout.

Coupling: The means by which the prestressing force may be transmitted from one partial-length prestressing tendon to another.

Sheathed Anchor: An anchor in which the stressing length of the high strength steel is encased in a grout-tight sheath. The annulus between the sheath and the periphery of the drilled hole may be grouted together with the anchor grout.

Un-sheathed Anchor: An anchor in which the stressing length of the high strength steel is not encased in a sheathing.

Coheisve Soils: Soils that exhibit plasticity. Generally defined as composed of material more than half of which is smaller than the No. 200 size sieve.

In order to better define a soil as cohesive or non-cohesive it is necessary to know the percentage of fines and also to know the Atterberg limits of soils containing more than 12 percent fines.

Non Cohesive Soils: Granular material that is generally nonplastic, composed of material more than half of which is larger than the No. 200 size sieve.

4.3 ROCK ANCHORS

4.3.1 Description

A prestressed rock anchor is a high strength steel tendon, fitted with a stressing anchorage at one end and a means permitting force transfer to the grout and rock on the other end. The rock anchor tendon is inserted into a prepared hole of suitable length and diameter, fixed to the rock and prestressed to a specified force. The basic components of prestressed rock anchor tendons are the following: *(see Fig. 4-1).*

1. Prestressing steel which may be a single or a plurality of wires, strands or bars. *(see Guide Specifications for Post-Tensioning Materials, pages 133 to 163.)* The total length of the prestressing tendon is composed of two parts:
   a. Bond length *(socket)*, is the grouted portion of the tendon that transmits the force to the surrounding rock.
   b. Stressing length, which is the part of the tendon free to elongate during stressing.
2. A stressing anchorage is a device which permits the stressing and anchoring of the prestressing steel under load.
3. A fixed anchor is at the opposite end of the tendon than the stressing anchor and is a mechanism which permits the transfer of the induced force to the surrounding grout.
4. Grout and vent pipes and miscellaneous appurtenances required for injecting the anchor grout or corrosion protective filler.

4.3.2 Design Considerations — Rock Anchors

Rock anchors can be installed in downward or upward positions, however, close to horizontal positions are not recommended because of grouting difficulties.

Recommended Bond Stress: The ultimate bond stress values given in the table below are guide values only. Core drilling to explore the rock quality is an absolute necessity, and core testing together with pull-out tests of test rock anchors are strongly recommended to verify the design assumptions prior to installation of production anchors.

The values presented in the table must be used with a Safety Factory which will depend upon the type of application. The following are suggested methods of obtaining safe working loads:

a. Safety factor applied to the ultimate bond stress obtained from either pull-out tests or bond stress table. Safety factor should range from 1.5 to 2.5.

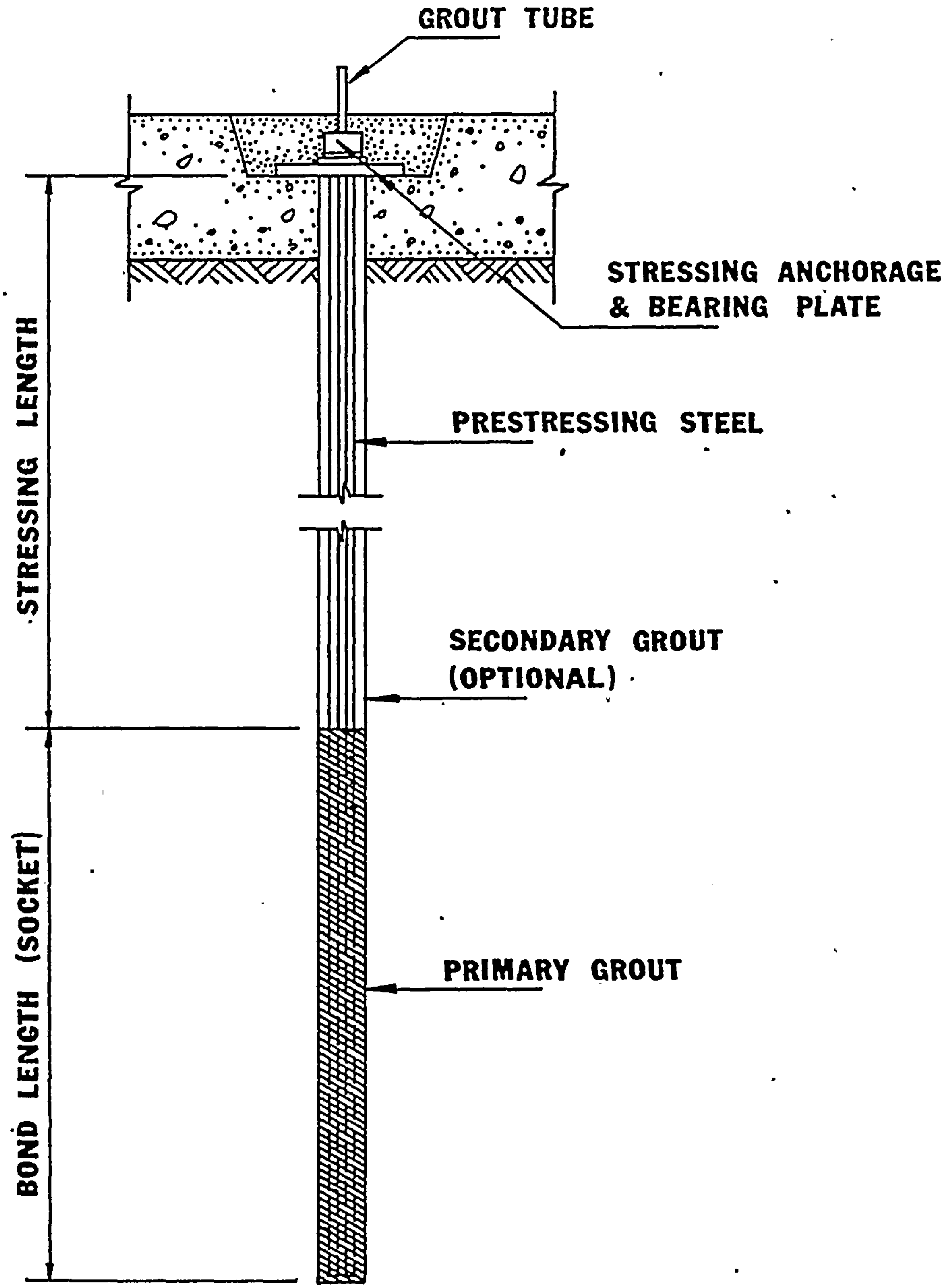

Fig. 4-1 — Rock Anchor

b. Proof loading of every anchor of not less than 115 percent of its transfer *(lock-off)* force. During the proof loading operation, the prestressing force shall not be more than 80 percent of the guaranteed ultimate tensile strength *(GUTS)* of the high strength steel. The duration of the proof loading is to be specified by the Engineer. Transfer *(lock-off)* the prestressing force at a level of between 50 and 70 percent of its guaranteed ultimate tensile strength. The difference between transfer load and design load shall include allowance for time dependent losses.

The duration of the proof loading is usually up to 15 minutes, in which case, the prestressing force is held by the jack. If longer duration is required, it is recommended to transfer the force to the anchorage and remove the jack.

Typical Bond Stresses for Rock Anchors

Type	Ultimate Bond Stresses Between Rock and Anchor-Grout Plug Sound, Non-decayed
Granite & Basalt	250 PSI – 450 PSI
Dolomitic Limestone	200 PSI – 300 PSI
Soft Limestone*	150 PSI – 220 PSI
Slates & Hard Shales	120 PSI – 200 PSI
Soft Shales*	30 PSI – 120 PSI
Sandstone	120 PSI – 250 PSI
Concrete	200 PSI – 400 PSI

**Bond strength must be confirmed by pullout tests which include time creep tests.*

For small load strand anchors (such as single strand) the bond between grout and strand might govern. The bond capacity between grout and strand is about 450 psi.

4.3.3 Drilling

Holes for anchors should be drilled to a diameter, depth, line, and tolerance as specified by the engineer. The hole shall be drilled so that its diameter is not more than 1/8 inch smaller than the specified diameter.

Core drilling, rotary drilling and percussion drilling may be employed as the conditions warrant Core drilling is generally slower and less economical.

Drilling tolerances are controlled by the size of the drill steel, weight of the drill rig, the method of drilling, and the nature of the ground. Holes can be drilled to an angle tolerance of 3 percent of their planned location.

4.3.4 Watertightness

The holes for some or all rock anchors may be tested for watertightness, if specified by the Engineer. When specified, the entire hole shall be tested for watertightness by filling it with water and subjecting it to a pressure of 5 psi. If the leakage rate from the hole over a period of 10 minutes exceeds 0.001 gallons per inch diameter per foot of depth per minute, the hole should be consolidation grouted, redrilled and retested.

Holes are water tested to insure limited grout loss for proper anchoring of the tendon, and to insure corrosion protection by limiting loss of either anchor grout or secondary grout. Consistency of consolidation grout depends on the results of the water test. Should the water test indicate a high volume of leakage in the hole, a stiff consolidation grout should be used, such as, a maximum of five gallons water per sack of

Should the second watertightness test fail, the entire process should be repeated.

Holes adjacent to a hole being tested for watertightness shall be observed during the test so that any inter-hole connection can be more easily detected.

cement. Should the water test indicate a low volume of leakage, a very lean consolidation grout should be used, such as eight gallons of water per sack of cement.

It is normal practice to redrill a consolidation grouted hole after the grout has had 24 hours to set up.

Payment for consolidation grouting, redrilling and testing should be based on unit prices since these quantities are unpredictable. Typical payment units would be: water tests (each); cement (CWT); redrilling (lin. ft.).

4.3.5 Fabrication

4.3.5.1 Materials

Anchor material shall be in accordance with Guide Specification for Post-Tensioning Materials *(see pages 133 to 163).*

Anchor material shall consist of either single or multiple units of the following:

a. Wires conforming to ASTM Designation A421, "Uncoated Stress-Relieved Wire for Prestressed Concrete."

b. Strand conforming to ASTM Designation A416 "Uncoated Seven-Wire Stress Relieved Strand for Prestressed Concrete."

c. High alloy steel bars, either smooth or deformed conforming to ASTM Designation A722 "Uncoated High-Strength Bar for Prestressing Concrete."

Stressing anchorages shall be capable of developing 95 percent of the guaranteed minimum ultimate tensile strength of the anchor material when tested in an unbonded state.

Mill test reports for each heat or lot of prestressing material used to fabricate tendons shall be submitted if required by the Engineer.

4.3.5.2 Fabrication of Anchors

Anchors shall be either shop fabricated or field fabricated in accordance with approved details, using personnel trained and qualified in this type of work.

Anchors shall be free of dirt, detrimental rust or any other deleterious substance.

Anchors shall be handled and protected prior to installation in such a manner as to avoid corrosion and physical damage thereto.

Anchors may be either sheathed or unsheathed.

The sheathing may consist of tubes surrounding individual anchor elements *(bar, wire or*

A light coating of rust on the anchor material is normal and will not affect the ability of the anchor to perform its function. Heavy corrosion or pitting should be cause for rejection of the anchor.

The sheathing material can be either steel, plastic or any other material non-detrimental to the high strength prestressing steel.

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strand) or a single tube surrounding the elements altogether. A seal shall be provided to prevent the entry of grout into the sheath prior to stressing.

4.3.6 Insertion and Anchor Grouting

Anchors shall be placed in accordance with the recommendation of the manufacturer.

Anchors shall be securely fastened in place to prevent any movement during grouting.

Grout tubes and vent networks shall be checked with water or compressed air to insure that they are clear.

Care shall be taken to insure that the bond length of the anchor is centrally located in the hole.

Centering devices are normally provided at about 10 ft. centers throughout the bond length.

If multi-unit tendons are used without a fixed anchorage at the lower end of the tendon, provision should be made for adequate spacing of the tendon elements to achieve proper grout coverage.

Grouting operations shall generally be in accordance with Section 3.2 *(pages 143 to 149.)* and in accordance with the recommendations of the manufacturer.

Primary grout of the proper consistency shall be pumped into the anchor hole through a grout pipe provided for that purpose until the hole is filled to the top of the anchorage zone. The grout shall always be injected at the lowest point of the bond length.

Provisions shall be made for determining the level of the top of the primary grout to assure adequate anchorage.

After grouting, the tendon shall remain undisturbed until the necessary strength has been obtained.

The following data concerning the grouting operation shall be recorded:

Type of Mixer
Water/Cement Ratio
Types of Additives
Grout Pressure
Type of Cement
Strength Test Samples
Volume of first and second stage grout

It should be recognized that water separation or bleed creates a layer of water at the top of any grouting stage. For strand tendons where bleed is more pronounced, bleed water could be over 6 percent of the vertical height of the tendon. Chemical additives are available that will control the bleed. Collodial (high energy) grout mixers will reduce this phenomenon. In the case of two stage grouting, it is normal procedure to fill the void caused by bleed water at the top of the second stage by regrouting after the second stage grout has set.

In the case of sheathed anchors, the first stage grouting covers the full length of the anchor between the sheathing and the periphery of the hole, and may fill the space between the sheathing and tendon throughout the bond length. Second stage grouting may be used to fill the space between the sheathing and the tendon throughout the stressing length or throughout the entire anchor length.

For sheathed anchors, consideration should be given to force transfer through the grout in the annulus around the stressing length.

4.3.7 Stressing

Stressing shall generally be accomplished in accordance with the provisions of Section 6.3.4.

The anchor shall be first stressed to an initial load of about 10 percent of the test load, which is the starting point for elongation measurements.

Stressing is normally carried out seven days after grouting for Type I or Type II cements and three days after grouting for Type III cement. At these times, grout with a water-cement ratio of 0.45 will have a compressive strength of about

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Immediately thereafter, the anchor shall be stressed to the proof load and elongation is to be recorded. The magnitude of the proof load is to be determined by the engineer. If measured and calculated elongations disagree by more than 10 percent, an investigation shall be made to determine the source of the discrepancy.

When the above requirements are met, the anchor force shall be lowered and anchored at the transfer load. This load may be verified by a lift-off test and recorded, if required by the Engineer.

4.3.8 Testing

The stressing anchorages shall be capable of lift-off during the period of installation, in order to check the force.

The lift-off test, if any, is to be specified by the Engineer. Allowances shall be made for time dependent losses when comparing the lift-off force with the previous transfer load.

4.3.9 Corrosion Protection

Prestressed rock anchors shall be protected against corrosion by procedures suitable for the intended service life.

4.3.9.1 Temporary Rock Anchors

Corrosion protection provided for temporary anchors shall be based on the intended service life of the anchor, and on the corrosion potential of the environment in which the anchor is to be installed. For wedge-type post-tensioning systems, protection shall be applied to the anchor head and wedge holes prior to insertion of wedges and stressing of tendons. Corrosion protection of temporary anchors shall be inspected and maintained throughout the service life of the anchor.

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3500 psi.

Movements of the bearing plate in excess of ½ inch shall be taken into consideration in comparing measured and theoretical elongations. For temporary rock anchors, elongation measurements are not usually required.

Usually, the proof load is specified as 115 percent to 150 percent of the transfer load. The proof loading of anchors is part of the stressing operation and occurs just prior to load transfer.

The lift-off, if required, is usually done on a random basis. The Engineer is to determine the percentage of tendons tested. Meaningful lift-offs can be taken as soon as 24 hrs. after the anchor is stressed. It is poor practice to require that the jack be left on an anchor since the jack bleeds off and the results are incorrect.

For most rock anchor applications, the primary time dependent loss is steel relaxation which can be as much as 3 percent of the transfer load in seven days depending on the type of steel. More exact values can be obtained from the rock anchor supplier.

When in rock where there is no apparent danger of corrosive attacks, temporary anchors with a service life up to 3 years are sometimes installed with no corrosion protection along the stressing length. However, normal practice for temporary anchors requires use of a ferrous metal or suitable plastic sheathing covering the stressing length to keep the prestressing steel dry and protect it from contact with the surrounding rock. A watertight seal should be provided between the sheathing and the grout in the bond length on one end and between the sheathing and anchorage device at the other end. The annular space between tendon and sheathing may contain preplaced grease or powder corrosion inhibitors. Asphaltic painting or grease corrosion protection of anchorage hardware is recommended. For wedge-type post-tensioning systems, a small amount of movement or travel of the wedges is required to develop force in the tendon above the transfer load. To develop *the full tendon capacity, the required wedge movement may vary from approximately 1/32 inch to 1/8 inch depending on the wedge type and the transfer load level. Therefore, to assure that the tendons have capacity to sustain unanticipated loads substantially in excess of the transfer load, it is important that corrosion protection of anchorage hardware be provided and maintained.*

Appropriate spacers shall be provided to center the tendon in the hole throughout the bond length to insure adequate cover.

Centering devices are normally provided at about 10 ft. centers throughout the bond length.

4.3.9.2 Permanent Rock Anchors

Permanent rock anchors shall be provided with protective corrosion seals over their entire length.

For tendons utilizing sheathing over the stressing length, the annulus between sheathing and tendon in the stressing length of the tendon shall be protected with a preplaced grease, powder corrosion inhibitor or grout. A grout plug shall be provided to seal the end of the sheathing adjacent to the bond length. Grout shall be applied from the bottom of the anchor hole covering bond length and the annulus between sheathing and rock in the stressing length in one continuous operation.

Permanent rock anchors utilizing a two stage grout system may be fabricated without the use of sheathing above the bond length. Grout shall be injected from the bottom of the anchor to the top of the bond length. Grout quantity shall be continuously monitored. Secondary grouting shall be applied to the stressing length after stressing and any required stress monitoring are complete and accepted.

Special attention shall be given to assure corrosion protection of the tendon at the connection to the anchorage hardware. The anchorage hardware shall be protected by embedment in concrete or other suitable material.

4.4. SOIL ANCHORS

4.4.1 Description

A prestressed soil anchor is a high strength steel tendon, fitted with a stressing anchor at one end and an anchor device permitting force transfer to the soil on the other end. These anchors, which are used in clay, sand or other granular soils, are inserted into a prepared hole or driven into the soil. Concrete is gravity placed to form n anchorage, or grout is injected under pressure to form a bulb of grout to anchor the tendon. Pressure bulb soil anchors are usually equipped with a casing, which is withdrawn during the grouting operation. Subsequent to placement of anchor grout, the soil anchor is stressed and anchored at a specified force.

Soil anchors may be classified as follows depending on their use in cohesive or noncohesive soils.

Soil anchors in noncohesive material are generally pressure grouted *(See Fig. 4-2).* They may be installed by two procedures:

1. Auger drilled - using hollow stem continuous flight augers normally of 6" to 10" diameter, the tendon is placed through the hollow stem of the auger before or after drilling is completed. Concrete or grout is then pumped under pressure through the hollow stem and the auger is withdrawn as the grout fills the hole.
2. Drilled or Driven Casing Pressure Grouted. In this type of anchor a 3" to 6" diameter casing is either drilled or driven into the ground to the final depth. The casing is then cleaned out and the tendon inserted. The anchor is then pressure grouted over the anchoring zone as the casing is withdrawn. Grout pressures used vary from 50 to 200 psi.

A "lost point" on the bottom end of the casing is used in this method. The point remains in the ground during and after casing withdrawal.

Soil anchors in cohesive soils are generally of the following types:

1. Auger Drilled *(See Fig. 4-3)* - using either continuous flight augers or short augers on a Kelly Bar type of machine. These anchors differ from those drilled in cohesionless soil only in the way they are grouted. The auger is withdrawn before gouting, and pressure grouting is not used.
2. Belled Type Anchors *(See Fig. 4-4)* - Drilled either by a Kelly Bar type machine using augers and a standard caisson belling bucket or the drilled casing method which employs a small air or mechanically activated underreamer. The cuttings are removed by air or water flushing. Belled anchors rely on the bearing of the underream cones against the soil for resistance to pullout.

4.4.2 Design Considerations

The design of soil anchors is largely dependent on the soil conditions and upon the type of anchor used. Use of test anchors to determine

For large diameter holes, augered anchor bond stresses in the bond length are normally about 10 psi although there can be a wide varia-

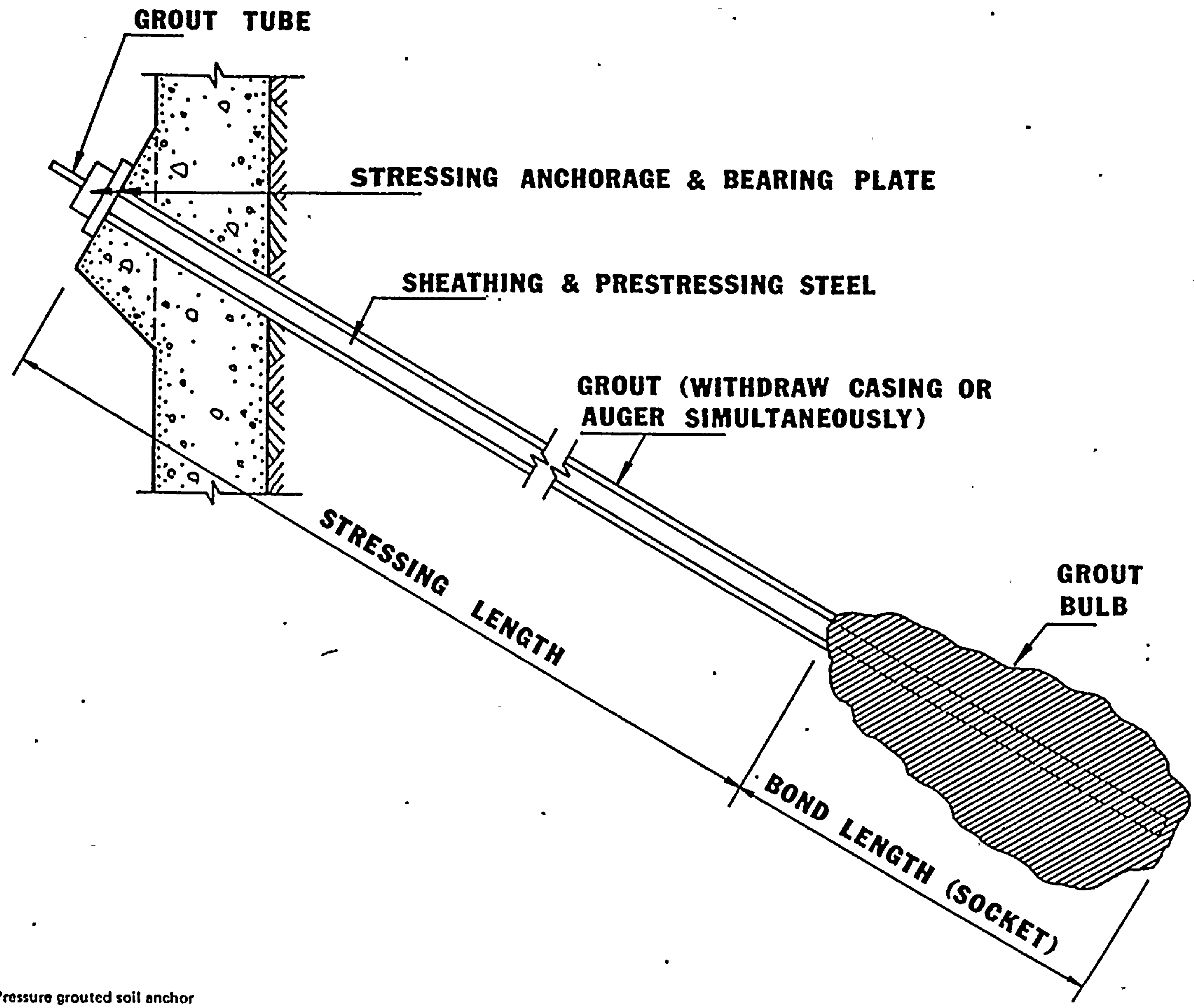

Fig. 4-2 — Pressure grouted soil anchor

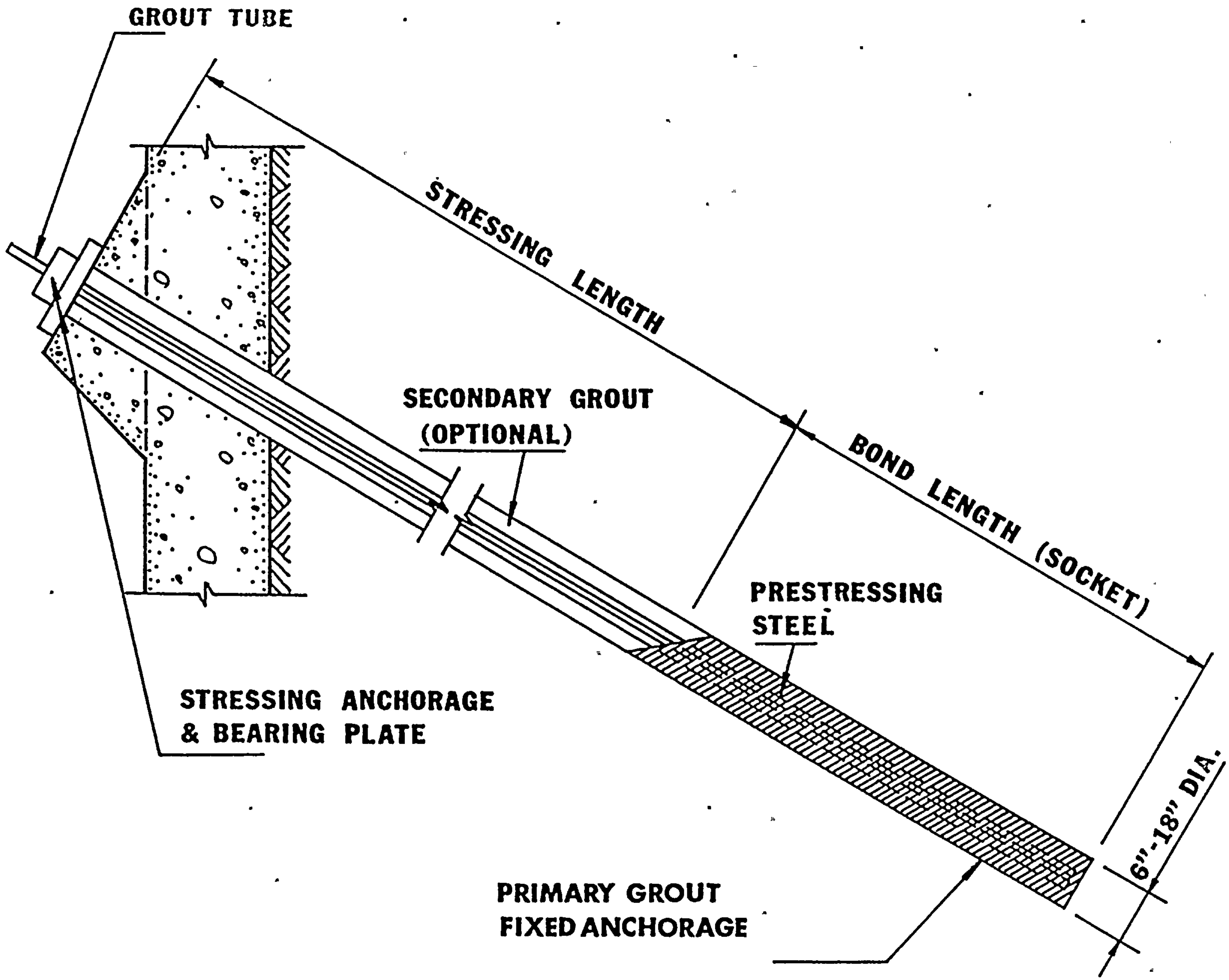

Fig. 4-3 — Augered soil anchor

the necessary bond length is strongly recommended for augered anchors and is essential for pressure bulb type soil anchors.

tion in this figure. It is not practical to give typical bond stress values for pressure bulb type soil anchors. Pressure bulb anchors develop the tendon force partially through bond and partially through bearing of the bulb of the soil. The response of soils to the pressure grouting varies widely, and, for this reason, field anchor tests are necessary to properly design pressure bulb anchors.

Minimum stressing lengths of 20 to 25 ft. are recommended.

The minimum stressing lengths recommended are necessary so that small movements of the stressing anchor will not result in large changes in load.

4.4.3 Drilling

4.4.3.1 Augered holes

Augered holes may vary from 6 inches to 24 inches in diameter and lengths may be as much as 100 feet. Some augers have attachments which permit belling or enlarging the bottom of the hole. More than one bell may be provided in cohesive soils.

Augered holes are the fastest method of drilling a soil anchor.

4.4.3.2 Pressure Grouted Anchors

Pressure grouted anchors are installed by either ramming a casing with a detachable point using an air track, or by augering a small hole with a hollow stem continuous flight auger.

Ramming is usually only employed in fairly loose sands and gravels.

4.4.4 Fabrication

4.4.4.1 Materials

Soil anchor materials shall conform to the requirements of Section 4.3.5.1.

4.4.4.2 Fabrication of Anchors

Anchors shall be either shop fabricated or field fabricated in accordance with approved details, using personnel trained and qualified in this type of work.

Anchors shall be free of dirt, detrimental rust or any other deleterious substance. Anchors shall be handled and protected prior to installation in such a manner as to avoid corrosion and physical damage.

Anchors may be either sheathed or unsheathed.

A light coating of rust on the anchor material is normal and will not affect the ability of the anchor to perform its function. Heavy corrosion or pitting should be cause for rejection of the anchor.

Spacers are normally provided at about 5 ft. centers in the bond length of augered anchors.

The sheathing material can be either steel, plastic or any other material non-detrimental to the prestressing steel.

4.4.5 Insertion and Anchor Grouting

4.4.5.1 Augered or Belled Anchors

Soil anchors are manually inserted in augered

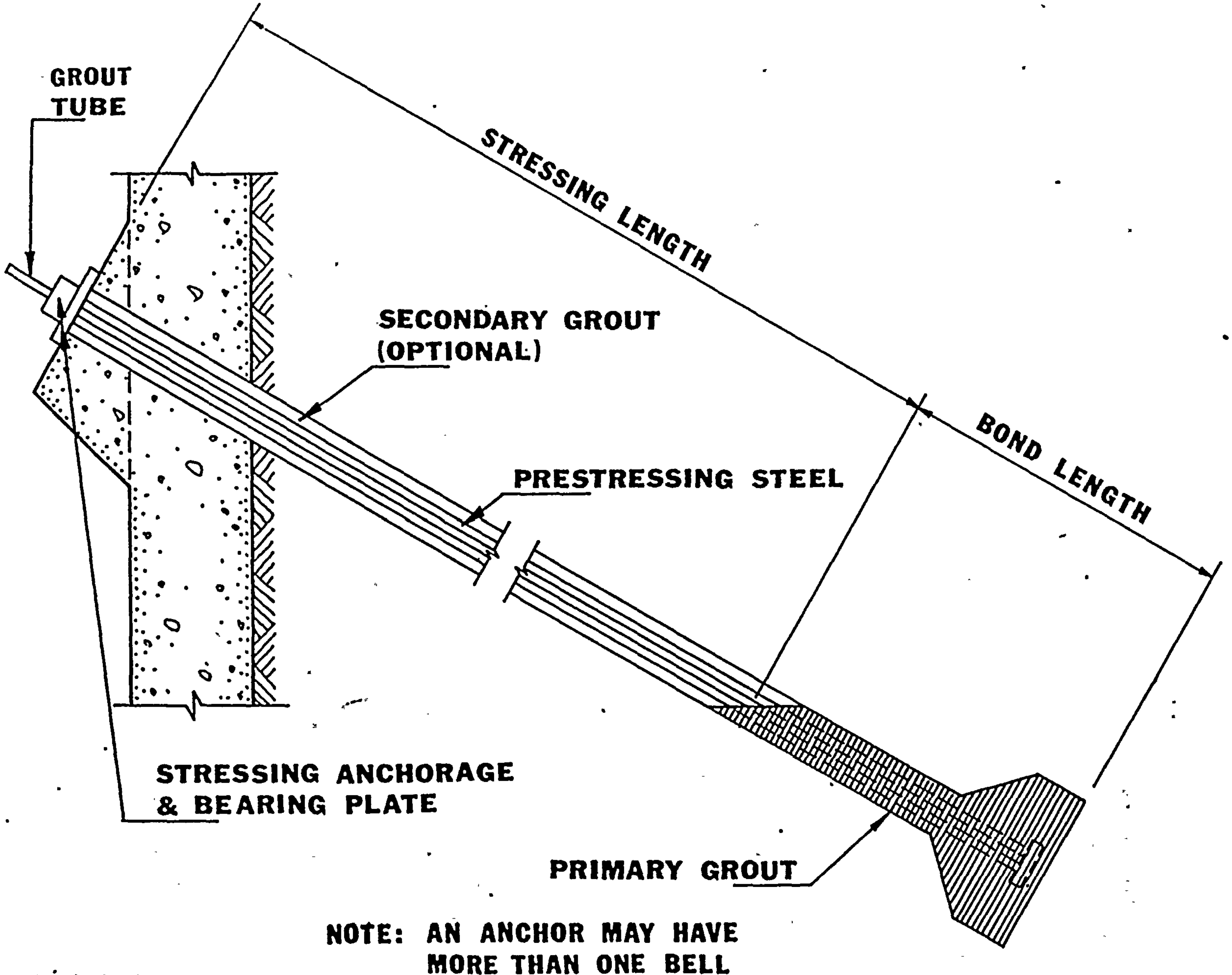

Fig. 4-4 — Belled soil anchor

holes. Concrete or grout is pumped or gravity placed into the bond length of the anchor.

4.4.5.2 Pressure Grouted Anchors

A. Rammed Anchors

The prestressing tendon is inserted in the casing and driven to its final position with the casing, or the tendon may be inserted after the casing is driven. Grout, under pressure, is pumped into the sealed casing as the casing is withdrawn from the hole by means of hydraulic jacks. After the casing has been withdrawn from the bond length, pressure grouting is discontinued and the casing may be withdrawn.

It is common practice to withdraw the casing and continue pumping grout at pressures high enough to result in a grout requirement of one bag of cement per foot of hole. However, the grout requirement depends greatly on the hole diameter, and the permeability and density of the soil.

B. Augered Pressure Anchors

A small diameter continuous flight auger is used to drill the hole. The procedure for installing this type of anchor is exactly the same as the driven anchor described above with the exception that the auger is always completely withdrawn.

C. Upward Sloped Soil Anchors

Pressure type soil anchors may be installed on upward slopes.

4.4.6 Stressing

Stressing shall generally be accomplished in accordance with Section 6.3.4.

Stressing is normally carried out seven days after grouting for Type I or Type II cements, and three days after grouting for Type III cement. At these times, grout with a water-cement ratio of 0.45 will have a compressive strength of about 3500 psi.

Soil anchors are normally stressed to 15 to 50 percent above design load, held at that load for 5 or 10 minutes, and then relaxed and anchored at the design load.

4.4.7 Testing

Soil anchors in cohesive soils normally require more testing than rock anchors since cohesive soils may creep under sustained load. Continuous monitoring systems may be employed when specified by the Engineer.

Lift-off tests are sometimes performed on selected anchors; these may be of 8-hour duration in the case of granular soils, but 24-hour duration may be called for on anchors in cohesive soils.

The average monitoring system consists of a load cell placed behind the stressing anchorage. This load cell has SR4 strain gauges installed on it, and the results can be directly read on a Wheatstone bridge. A separate payment item should be set up for monitoring.

4.4.8 Corrosion Protection

Measures to provide corrosion protection for soil anchors vary depending on whether the anchor is intended for temporary or permanent use. In both cases, protective measures are similar to those for prestressed rock anchors presented in Sections 4.3.9.1 and 4.3.9.2.

[illegible] TR

ERSON CG 312-265-1460 CLG

BECHTEL CORP R M LUKEN 2/28/68
ONTARIO CENTER N Y

POOR ORIGINAL

CC- MESSERS
K T MOMOSE/D K CRONEBERGER
C/O GILBERT ASSOCIATES INC
REDING PA TWX-- 510-651-0420

J O STULL
BECHTEL CORP
ONTARIO CENTER N Y TWX - 710-828-9704

D B TATE
WESTINGHOUSE A PLD
ONTARIO CENTER N Y TWX-- 510-250-2354

E U POWELL
WESTINGHOUSE A P D
PITTSBURGH PA TWX-- 710-797-3658

J W HALLOWELL
WESTINGHOUSE A P D
PITTSBURGH PA TWX-- 710-797-3678

J HOOD
GILBERT ASSOC INC C/O BECHTEL
ONTARIO CENTER N Y TWX 510-250-2306

J GROSCH
G & G MFG CO
7227 WEST WILSON AVE
CHICAGO ILL

E CANABENE
ROCHESTER IRON & METAL CO
P O BOX 565
ROCHESTER N Y

	RT.	INIT.	DATE
Proj. Supt.		[illegible]	[illegible]
Job Supt.	2		
Job Eng.			
Civil Sect.	3		
Mech. Sect.			
Elec. Sect.			
Purchasing			
Adm. Dept.			
Cost Dept.			
Client			
G. Gilbert	1		
FILES [illegible]			

A JOB SITE INVESTIGATION WHICH BEGAN ON FEBRUARY 13 AND WAS CONSUMMATED ON FEBRUARY 27 AND FEBRUARY 28, AS A RESULT OF THE DIFFICULTY IN MAKING UP THE WALL TENDON CONNECTIONS TO THE ROCK ANCHORS, REVEALED A TWO FOLD PROBLEM.

1- CONTROL SYSTEM ALLOWING TOO TIGHT A PITCH DIAMETER IN THE BUSHING I.D.
REMEDIAL ACTIOMN-- RETURN BUSHINGS TO RYERSON STEEL. GAUGE WITH FULL LENGTH GAUGE AND REMACHINE TO ACCEPT FULLENGAGEMENT ON A GO GAUGE WITHIN NO GO GAUGE LIMITS.

2- MINOR DAMAGE TO BOTTOM ANCHOR HEAD THREADS.
REMEDIAL ACTION-- MAKE UP THREAD CHASER AND CLEAN ALL TENDONS NOT INSTALLED PRIOR TO THEIR INSTALLATION IN THE STRUCTURE WITH THIS CHASER. ALL TENDONS NOT NOW INSTALLED WILL HAVE ANCHOR HEADS CHASED TO REMOVE BURRS. FOR THE FIRST LOT OF TENDONS TO BE INSTALLED, A REPRESENTATIVE BUSHING WILL BE SCREWED ONTO THE [illegible] HEAD AND REMOVED BEFORE THE TENDON IS LOWERED INTO ITS

~~WHEN PROGRESS IS DEMONSTRATED TO BE SATISFACTORY, THE BUSHING TRIAL MAY~~
BE ELIMINATED UPON APPROVAL BY BECHTEL.

WORK WILL RESUME ON TENDON MARKS 131 AND 133 WHICH ARE HUNG UP AS SOON AS SCAFFOLDING IS AVAILABLE AND AN A FRAME IS MOUNTED ON THE TOP BASE PLATE.

THE PITTSBURGH TESTING LABORATORY HAS RUN A COMPRESSION TEST ON MONDAY, FEBRUARY 27, ON THE ANCHORAGE COMPONENTS WHICH ARE REPRESENTATIVE OF THE REMEDIAL ACTION BEING TAKEN BY G & G MFG. CO. THIS TEST APPROXIMATES THE MANNER VERY CLOSELY IN WHICH THE LOAD WILL EVENTUALLY BE APPLIED ON THESE PARTS DURING STRESSING. THE FOLLOWING IS A PRELIMINARY REPORT OF THE RESULTS OF THE TEST AS RECEIVED OVER THE PHONE FROM MR. COOPER OF PTL BY FRANK BIALAS.

AFTER LOADING PARTS TO 1,000,700 LBS. AND RELEASING LOAD, THERE WAS A SMALL INITIAL RESISTANCE TO TURNING BY HAND WHICH WAS OVERCOME VERY EASILY AND THE PARTS MOVE EFFORTLESSLY.

AFTER LOADING TO 1,060,000 LBS. AND RELEASING, THE SAME AMOUNT OF RESISTANCE WAS OBSERVED. THIS WAS OVERCOME WITHOUT UNDUE EFFORT.

THE ANCHORAGE COMPONENTS WERE THEN LOADED TO 1,200,000 LBS., THE CAPACITY OF THE MACHINE. AFTER UNLOADING, THE PARTS TURNED EASILY WITHOUT ANY INITIAL RESISTANCE.

THIS TEST PROVED THAT THE THREADS MAY BE SUBJECTED TO A LOAD 20 PERCENT GREATER THAN THE TU. T. S. OF THE TENDON WITHOUT UNDUE ELASTIC DEFORMATION. BASED ON THIS TEST, IT HAS BEEN AGREED THOAT A TOTAL DAMAGE OF 20 PERCENT OF THREADS IN THIS ANCHORAGE COMPONENT IS ACCEPTABLE.

CERTIFICATIONS WILL BE SUPPLIED THAT ALL TENDON ANCHOR THREADS HAVE BEEN SUBJECTED TO INSPECTION AND ARE WITHIN PUBLISHED TOLERANCES. DOCUMENTS WILL SATATE DIMENSIONS OF GO AND NO GO GAUGES TO BE USED ON BUSHINGS AND THAT EACH BUSHING REWORKED HAS BEEN CHECKED.

A DETAILED REPORT EXPOUNDING UPON THE PROBLEM OF ANCHORAGE COMPONENT ENGAGEMENT AND REMEDIAL ACTION TAKEN, THREAD DIMENSIONAL TOLERANCES, CERTIFICATION OF DIMENSIONAL CHECK OF REWORKED BUSHINGS, PHYSICAL TEST REPORT FROM P.T.L. ON TEST PERFORMED, WILL FOLLOW IN 30 DAYS AS AGREED IN THE JOB SITE MEETING WITH BECHTEL ON FEBRUARY 28, 1968.

JOS T RYERSON AND SON INC FRANK J BIALAS- PROJECT ENGR CHGO ILL
EF

END TU
BECHTEL ONTR

POOR ORIGINAL

M LUKEN- BECHTEL CO TWX 510-250-2306
ONTARIO CENTER NY

CC TO K T MOMOSE/D K CRONEBERGER
GILBERT ASSOC INC
READING PA TEX 510-651-0420

D STULL BECHTEL CORP
ONTARIO CENTER NY TWX 710-828-9704

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D D TATE WESTINGHOUSE A P D
ONTARIO CENTER NY TWX 510-250-2354

E U POWELL WESTINGHOUSE A P D
PITTSBURG PA TWX 710-797-3658

RYERSON CHGO- F BIALAS T BROWN W CORSON POST TENS. DEPT.
1-19-68 1035A CST

IT IS AGREED THAT THE REVISED SUGGESTED PROCEDURES FOR INSTALLATION OF WALL TENDONS AS ORALLY TRANSMITTED BY PHONE AND SUBSEQUENTLY WIRED TO THIS OFFICE ON 1/18/68 SHOULD READ AS FOLLOWS--

1. ALL FULL LENGTH WALL TENDONS ARE COILED, DOUBLE BAGGED AND RACKED. THEY WILL BE SHIPPED AND WILL ARRIVE BY TRUCK AND INSERTED INTO TENDON TUBES. IF STORAGE ON THE SITE IS REQUIRED, THE TENDONS WILL BE UNLOADED FROM TRUCKS ONTO RAISED PLATFORMS IN LAYDOWN SPACE AND COVERED FROM THE ELEMENTS BY TARPAULINS. ~~DEMURRAGE WILL BE CHARGED ON A DAILY BASIS AS SUBSTANTIATED BY CARRIER IF TENDONS WILL BE STORED IN TRAILERS.~~

2. AN UNCOILER FRAME WILL BE USED TO UNCOIL TENDONS AND TO PREVENT THE TENDON FROM BEING CONTAMINATED BY DIRT, MUD, WATER, SNOW, OR ANY OTHER FOREIGN MATTER OR OBJECT.

3. A LIFTING COUPLING WILL BE SCREWED ON TOP HEAD OF THE TENDON AND A PROTECTIVE NOSE CONE BE PLACED ON THE BOTTOM OF THE TENDON FOR PROTECTION OF THE THREADS DURING THE UNCOILING OPERATION. A CRANE IS THEN ATTACHED TO THE TENDON LIFTING HEAD AND THE TENDON IS UNCOILED BY LIFTING IT VERTICALLY UNTIL IT IS FULLY SUSPENDED.

4. A ~~CURSORY~~ [illegible] INSPECTION WILL BE MADE ON EACH TENDON BEFORE IT IS LOWERED INTO ITS TUBE. NOTE, A COMPREHENSIVE INSPECTION PROGRAM HAS BEEN ESTABLISHED IN THE MANUFACTURE OF TENDONS TO GUARENTEE THEM TO BE WITHIN PRESCRIBED LIMITS. ~~[illegible]~~

5. THE TENDON IS THEN PLACED IN TENDON TUBE AND LOWERED TO THE ROCK ANCHOR COUPLING WHERE THE ADAPTOR IS ALREADY IN PLACE. GREASE IS PLACED IN THE VOID AT THE ROCK ANCHOR COUPLING PRIOR TO MAKING THE BOTTOM CONNECTION. THE BOTTOM HEAD IS THEN SCREWED INTO THE PLATE AND ADAPTER UNTIL THE THREADS ARE FULL ENGAGED. THE COUPLING ASSEMBLY IS THEN COMPLETELY CHECKED FOR PROPER INSTALLATION BY THE TENDON INSTALLER. *Snugging tendon connected by turn count [illegible]*

JOS T RYERSON & SON INC. FRANK BIALAS CHICAGO ILL

CONSTRUCTION PRO   CTS & SERVICES DIVISION
PHONE: RO 2-2121 • PLANT: 16TH & ROCKWELL, CHICAGO
MAIL ADDRESS: P. O. BOX 8000-A, CHICAGO, ILL. 60680

RYERSON METALOGICS
JOSEPH T. RYERSON & SON, INC.

July 20, 1966

Mr. D. K. Croneberger
Gilbert Association
P.O. Box 1498
525 Lancaster Avenue
Reading, Pennsylvania 10630

Dear Mr. Croneberger:

As Mr. Ted Brown is out of the office this week, I have taken the liberty of reviewing and commenting on drawing B-400-606-1 per your letter of July 13, 1966. Based on my discussions with Mr. Brown, the 2'-4" dimension should be to the top of the concrete, not the bottom of the bearing plate. Also the access cover on the coupling protection "can" should be attached to a turned up or welded on lip (see Sect. AA in red pencil) on the sides and as shown top and bottom.

I have made a copy of half of drawing B-400-606-1 and marked the above in red pencil. I am enclosing a copy of the present drawing of the coupler for your information.

In the design of this coupler, we have used the following values for loads and stressing:

allowable tensile stress in C1018 steel = 26.4 ksi
allowable load on threads in C1018 steel = 1.55 kips on each
inch of thread measured on the O.D.

ultimate strength of 90 wires = 240 ksi = 1060 k
maximum jacking force for 90 wires = 192 ksi = 848 k
maximum transfer force for 90 wires = 168 ksi = 742 k
maximum effective prestress for 90 wires = 144 ksi = 636 k
actual tensile stress on coupling @ 848 k = 22.4 ksi
@ 742 k = 19.5 ksi
actual load on threads @848 k = 1.23 k/in.

Note that this coupler is currently being tested by the Pittsburgh Testing Laboratory.

Very truly yours,

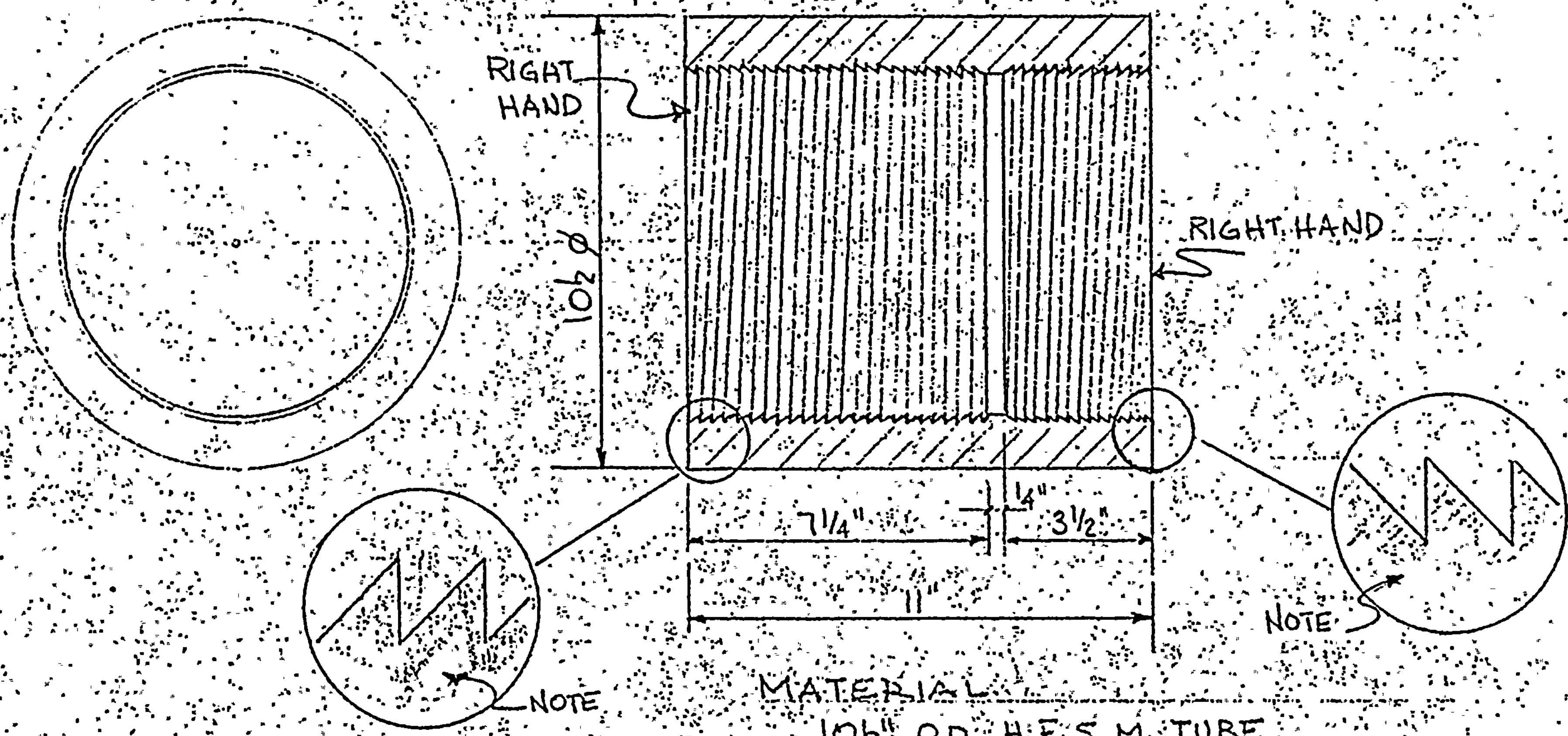
B. Clyde Lathrop

B. Clyde Lathrop
Chief Draftsman
Post-Tensioning

BCL:MM

POOR ORIGINAL

7 1/8 - 8 BUTTRESS THREAD



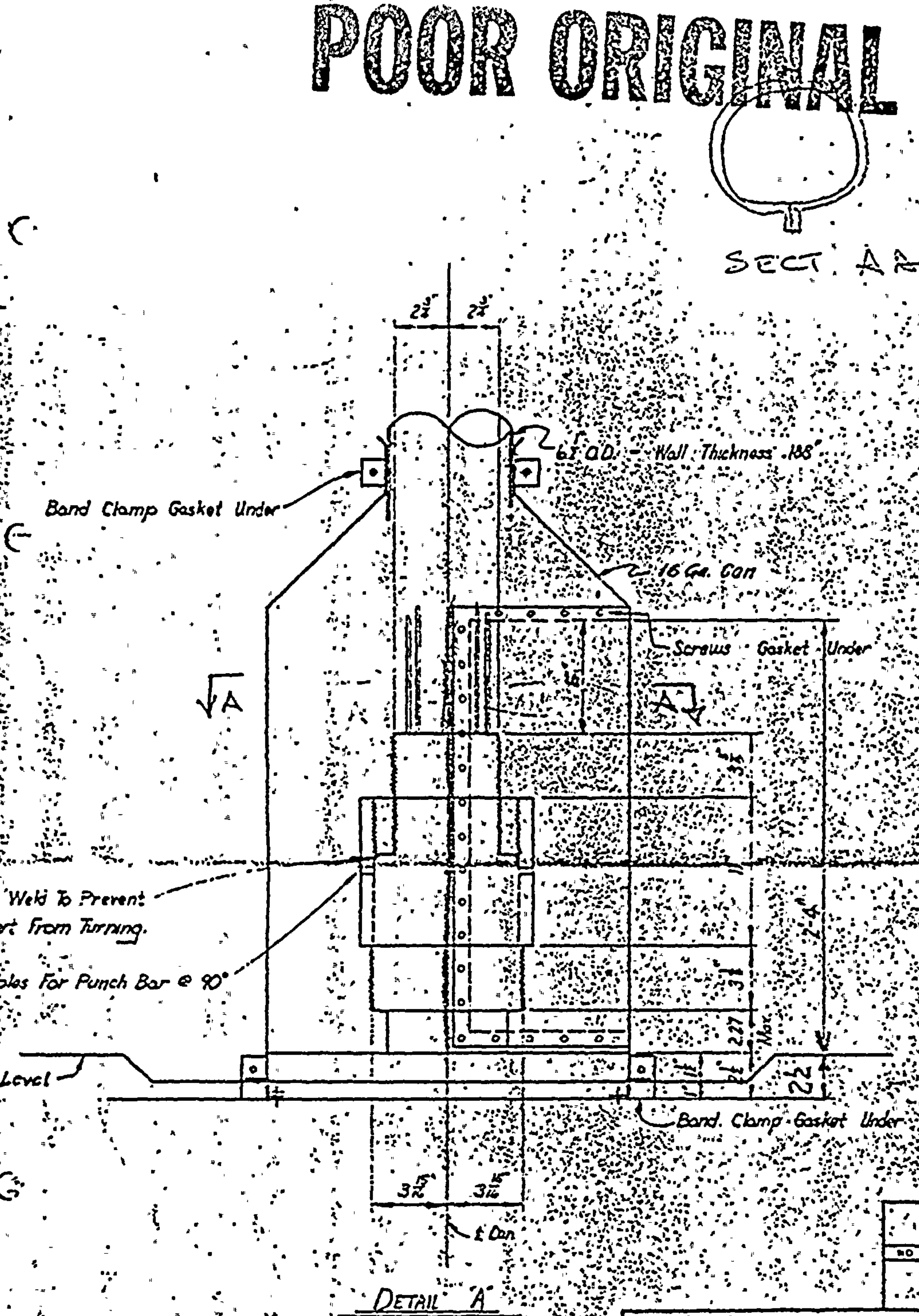
MATERIAL
10 1/2" O.D. H.E.S.M. TUBE
1 1/2" WALL 11" LONG

90 WIRE COUPLER

MADE BY	DATE	RYERSON METALOGICS	CUSTOMER
DER	5/23/66	JOSEPH T. RYERSON & SON, INC.	
[illegible] X7			21T 341891

FORM 780-12[illegible] APR. 63

POOR ORIGINAL



REVISIONS

ROCHESTER GAS & ELECTRIC CORPORATION
ROCHESTER, NEW YORK
BROOKWOOD PLANT UNIT NO. 1

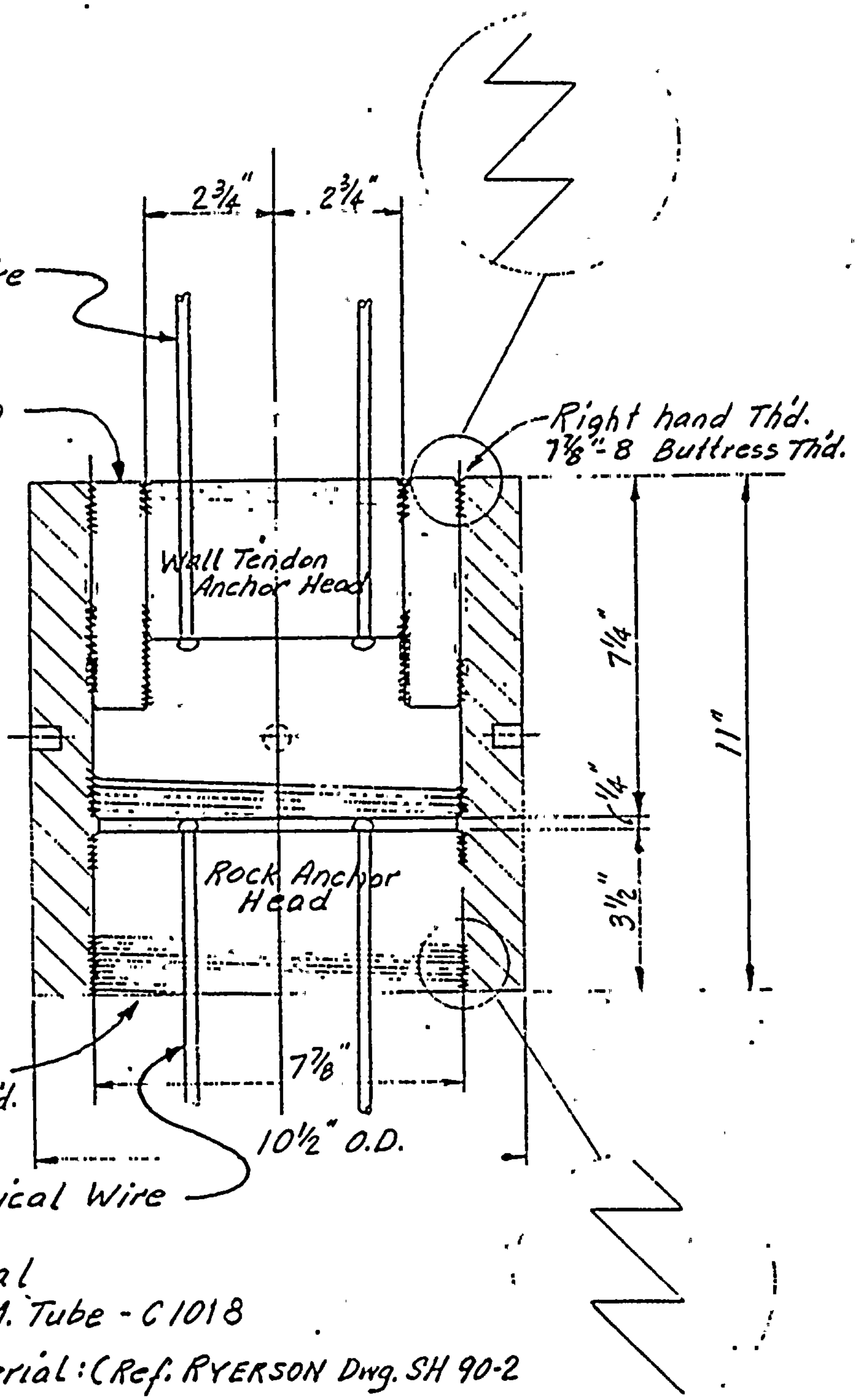
DETAIL FOR TENDON COUPLING

READING, PENNA. GILBERT ASSOCIATES, INC. ENGINEERS AND CONSULTANTS NEW YORK, N.Y.

DRAFTING — MADE: CEW — CHECKED
AS SHOWN 4155 B-400-

1 General Revision

56 For A.E.C. Review



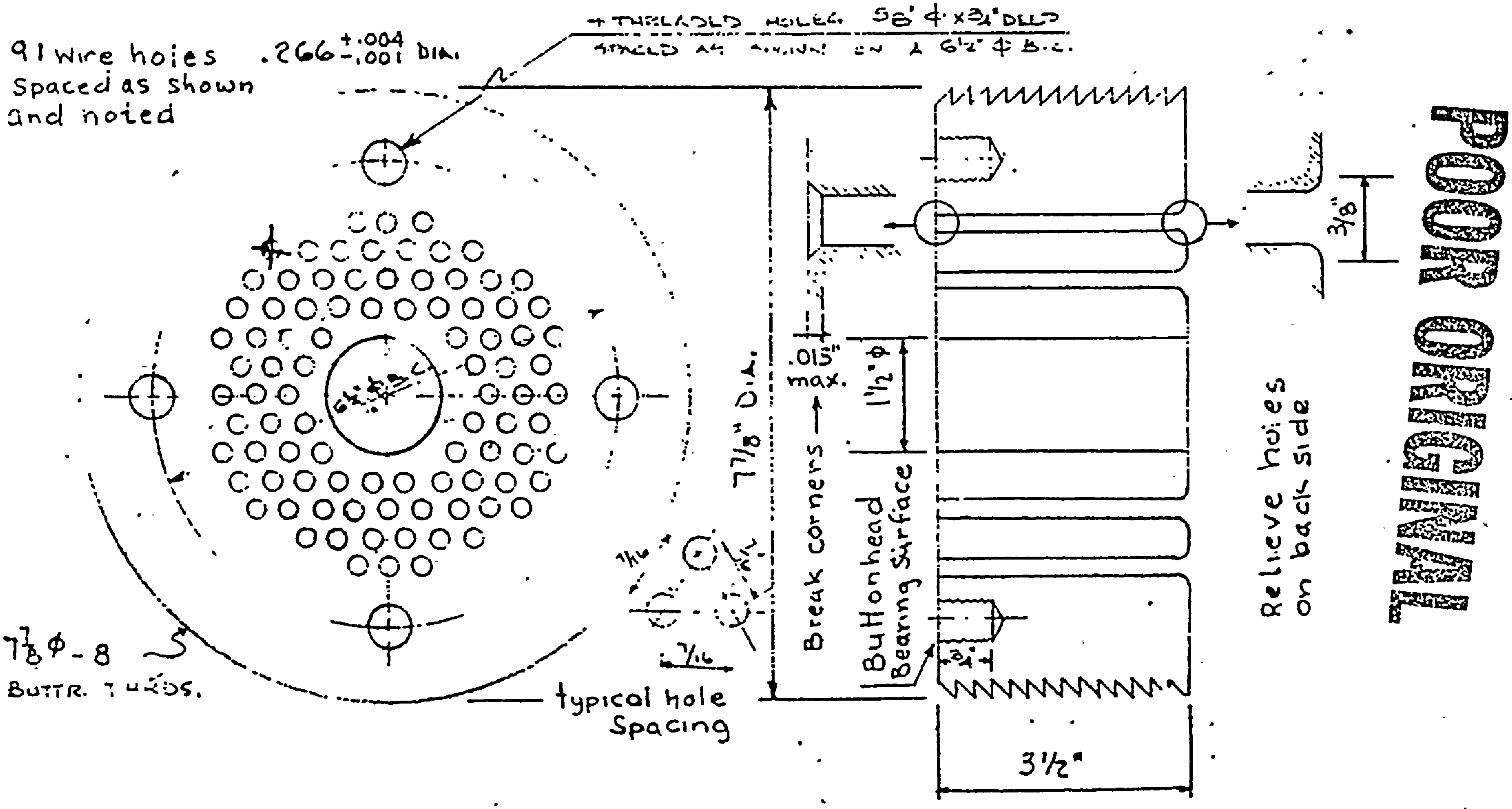
Coupling Material
10½" O.D. H.F.S.M. Tube - C 1018

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

Jacking Force 847^K
Effective Prestress Force 635^K

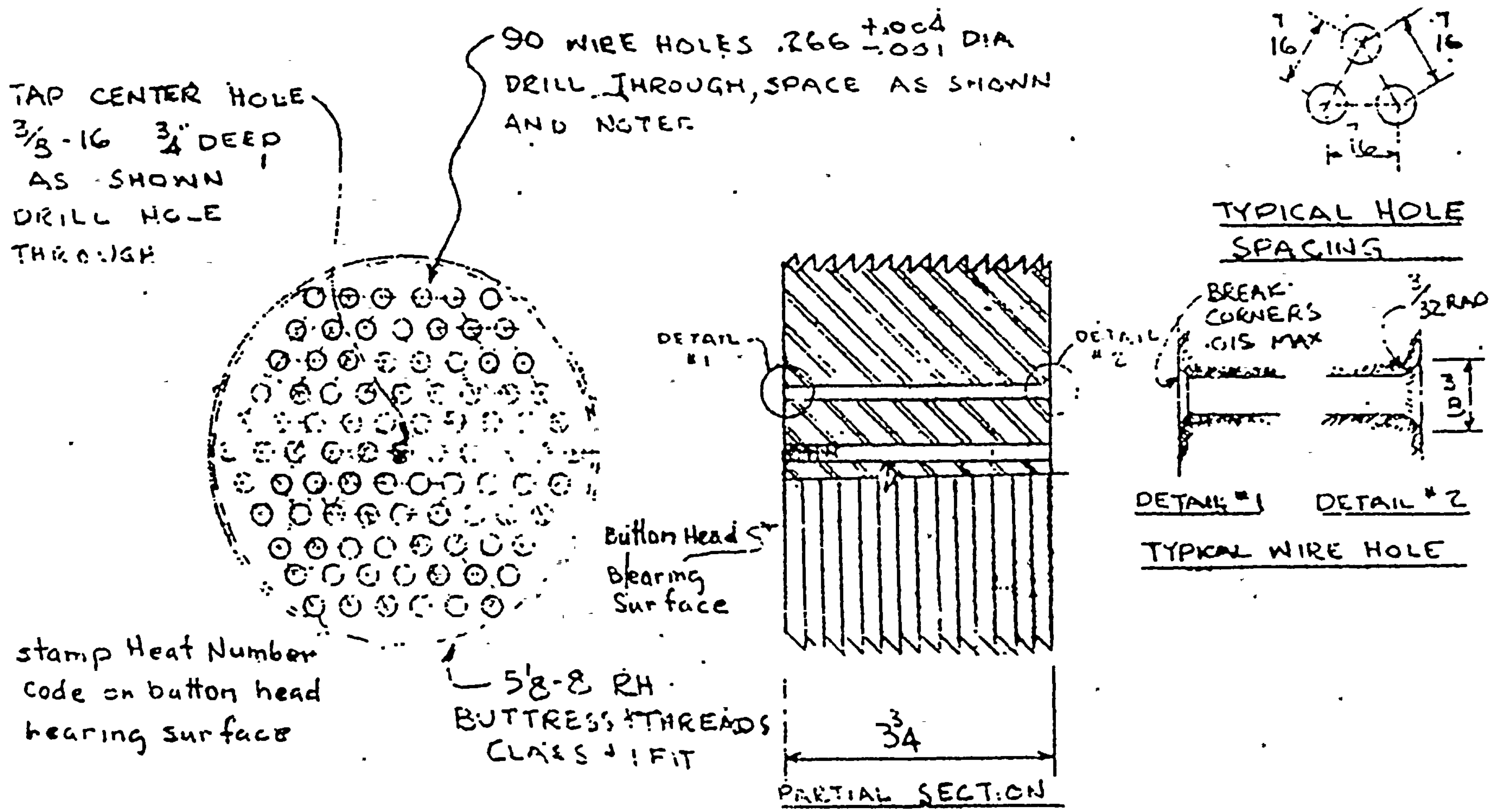
POOR ORIGINAL

TENDON TO ROCK COUPLING
FIG. 5.1.2-12

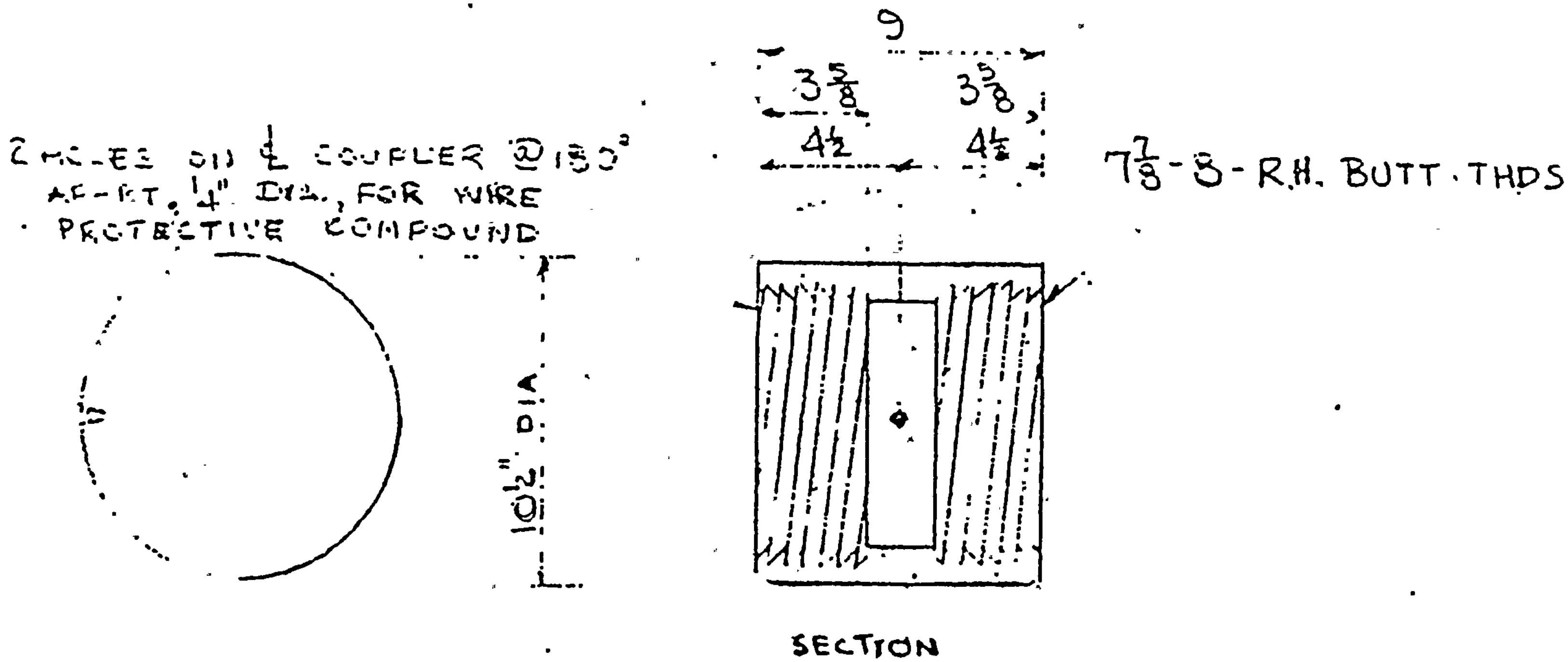


STANDARD 90 WIRE ANCHOR HEAD – ROCK ANCHORS AND TOP END OF WALL TENDONS

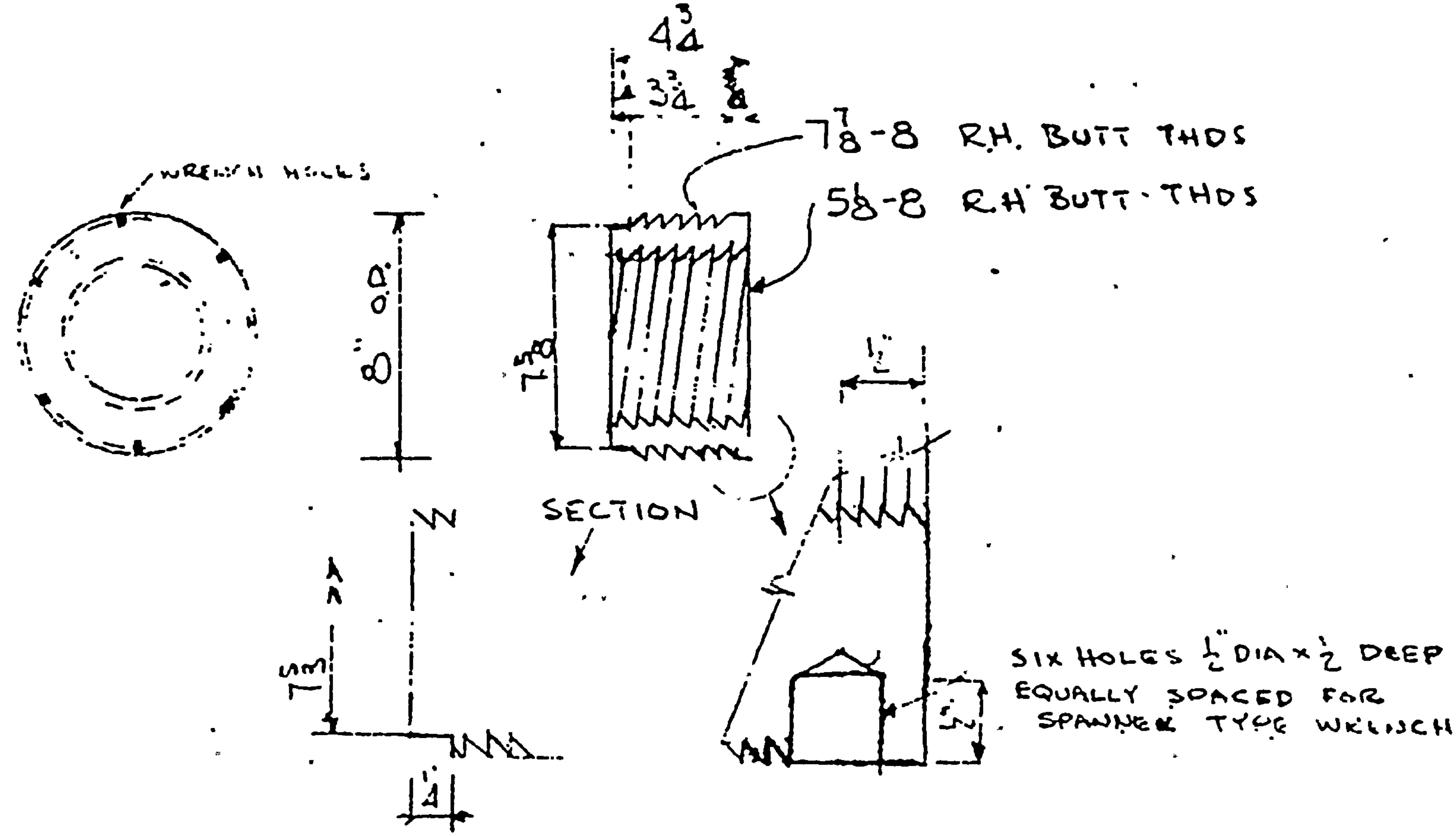
FIG. 5.1.2-13



SMALL, 90 WIRE ANCHOR HEAD - LOWER END OF WALL TENDONS
FIG. 5.1.2-14



COUPLER - ROCK ANCHOR TO LOWER END OF WALL TENDON
FIG. 5.1.2-15



ADAPTOR - SMALL ANCHOR HEAD TO COUPLER
FIG. 5.1.2-16

APPENDIX 5A

PITTSBURGH TESTING LABORATORY
REPORTS

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

AS A MUTUAL PROTECTION TO CLIENTS, THE PUBLIC AND OURSELVES, ALL REPORTS ARE SUBMITTED AS THE CONFIDENTIAL PROPERTY OF CLIENTS, AND AUTHORIZATION FOR PUBLICATION OF STATEMENTS, CONCLUSIONS OR EXTRACTS FROM OR REGARDING OUR REPORTS IS RESERVED PENDING OUR WRITTEN APPROVAL.

LABORATORY No. 652403

March 29, 1967

ORDER No. FG-18619

CLIENT'S No. 217114-3

REPORT

Report of: Compressive Load Tests of 90 Wire Tendon Base Plate Test on Concrete Stand

Report to: Joseph T. Ryerson & Son, Inc.
P. O. Box 8000A
Chicago, Illinois 60680

We were requested to fabricate a concrete base plate in accordance with Ryerson Drawing SPI-1 dated 1/20/67. A concrete mix design, reinforcing bars, base plate and trumpet were submitted for fabrication of the concrete base plate.

The following concrete properties were recorded.

CONCRETE MIX DESIGN PER CU. YD.

Type III Portland Cement	611 lbs.
Dravo Corp. Siliceous Sand ASTM C-33	1240 lbs. S.S.D.
Dravo Corp. Siliceous Gravel 1" Size	1850 lbs. S.S.D.
Water	300 lbs.
Slump	4 inches

COMPRESSIVE STRENGTHS

Date of Testing	Sectional Area Sq. In.	Crushing Load Lbs.	Crushing Strength PSI	Age Days
March 8, 1967	28.27	92,000	3250	2
March 8, 1967	28.27	81,000	2870	2
			3060 Average	
March 9, 1967	28.27	115,000	4070	3
March 9, 1967	28.27	120,000	4240	3
			4150 Average	
March 10, 1967	28.27	124,000	4390	4
March 10, 1967	28.27	121,000	4280	4
			4340 Average	

POOR ORIGINAL

PITTSBURGH TESTING LABORATORY
ESTABLISHED 1881
PITTSBURGH, PA.

LABORATORY No. 652408

CLIENT'S No. 21Y114-3   March 29, 1967   ORDER No. PG-18619

REPORT

When the concrete in the stand had reached the requested strength, the stand was tested by the following method.

A compressive load of 742,000 lbs. was applied in increments of 106,000 lbs., and then released in increments of 106,000 lbs. The gage readings tabulated below were obtained using a deflectometer designed as shown on Page 5 of Ryerson instructions dated 2/2/67.

Cycle One was repeated, recording the same gage readings.

On the third cycle, dial gage readings were recorded only up to 742,000 lbs. The loading continued in 106,000 lbs. increments to 1,200,000 lbs. At 954,000 lbs. hairline cracks appeared on the sides of the stand. There were no other apparent defects at 1,200,000 lbs.

The dial gage instrument was designed so that measurements, either compressive or expansive, were recorded at a specified distance from the center line of the concrete stand or metal base plate.

Gage No.	Location
1	On the concrete 3 inches from edge of base plate.
2	On the base plate 7-1/2 inches from center line of stand.
3	On the base plate 4-3/4 inches from center line of stand.
4	On the base plate 6 inches from center line of stand.
5	On the concrete 1 inch from edge of base plate.

POOR ORIGINAL

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. 652408

March 29, 1967

ORDER No. PG-18619

CLIENT'S No. 21T114-3

REPORT

LOAD DEFORMATION MEASUREMENTS

1st Loading

Load Pounds	Gage #1	Gage #2	Gage #3	Gage #4	Gage #5
0	.000	.000	.000	.000	.000
106,000	-.001	.000	.002	.001	.000
212,000	-.002	.001	.006	.004	-.001
318,000	-.002	.002	.009	.005	-.004
424,000	-.003	.002	.011	.007	-.007
530,000	-.004	.003	.013	.009	-.009
636,000	-.005	.004	.016	.011	-.010
742,000	-.006	.004	.018	.013	-.012
636,000	-.006	.004	.017	.012	-.013
530,000	-.005	.004	.016	.012	-.013
424,000	-.005	.004	.015	.011	-.012
318,000	-.004	.003	.014	.010	-.012
212,000	-.004	.003	.012	.008	-.012
106,000	-.003	.002	.009	.006	-.012
0	.000	.000	.003	.002	-.002

2nd Loading

Load Pounds	Gage #1	Gage #2	Gage #3	Gage #4	Gage #5
0	.000	.000	.000	.000	-.002
106,000	-.002	.001	.004	.003	-.007
212,000	-.003	.002	.004	.004	-.009
318,000	-.004	.003	.009	.006	-.011
424,000	-.005	.003	.010	.007	-.012
530,000	-.005	.003	.012	.008	-.013
636,000	-.006	.004	.013	.010	-.014
742,000	-.006	.004	.015	.011	-.015
636,000	-.006	.004	.014	.010	-.0165
530,000	-.006	.004	.013	.010	-.014
424,000	-.005	.0035	.012	.0085	-.013
318,000	-.005	.003	.011	.0075	-.0125
212,000	-.004	.003	.009	.006	-.0115
106,000	-.003	.002	.005	.004	-.010
0	.000	.000	.000	.000	-.002

POOR ORIGINAL

FM 407 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. 652408

ORDER No. PG-18619

March 29, 1967

CLIENT'S No. 21T114-3

REPORT

LOAD DEFORMATION MEASUREMENTS

3rd Loading

Load Pounds	Gage #1	Gage #2	Gage #3	Gage #4	Gage #5
0	.000	.000	.000	.000	-.002
106,000	-.003	.002	.004	.003	-.009
212,000	-.004	.002	.007	.0045	-.011
318,000	-.004	.003	.009	.006	-.012
424,000	-.005	.003	.011	.007	-.013
530,000	-.006	.0035	.012	.0085	-.014
636,000	-.006	.004	.0135	.010	-.015
742,000	-.007	.004	.015	.011	-.0155

954,000 Hair line cracks visible.

PITTSBURGH TESTING LABORATORY

Earl Gallagher

Earl Gallagher, Manager
Physical Testing Department

cc: 3-Ryerson Steel
1-PTL Chicago

POOR ORIGINAL

VS

BASEPLATE
FOR 90 WIRE TENDON

A. LOADS

Loads developped by the 90 wire Tendon.

Ultimate Strength	1060^{K}
Overstressing Force	848^{K}
Initial Force	742^{K} ← x
Final Force	636^{K}

x Design Force for Baseplate

B. SIZE

ϕ od	18½"	⟶	269 □"
ϕ id	6"	⟶	29 □"

Net Bearing Area 240 □"

Plate thickness 2½"

CUSTOMER Bechtel Company
Palisades 5935-C-5
METALOGICS
RYERSON
JOSEPH T. RYERSON & SON, INC.
MADE BY [illegible] | DATE 2/2/67 | 2
21 PT-34-114

C) BEARING STRESSES

1) Actual

Average 742'000/240 = 3090 psi

2) Allowable for Base-Slab
(use ACI Codes

$f_c' = f_{ci}' = 4000$ psi

$A_b' \rightarrow \phi\ 2'8'' - 803\ \square''$ (Tendonspacing)

$f_{cp} = 0{,}6 \cdot 4000 \sqrt[3]{803/269} = 3450$ psi

> actual O.K.

3) Allowable for Wall and Dome

$f_c' = f_{ci}' = 5000$ psi

A_b' (use minimum 1" clearance around Plates

$\rightarrow \phi\ 20\frac{1}{2}'' = 330\ \square''$

$f_{cp} = 0{,}6 \cdot 5000 \sqrt[3]{330/269} = 3210$ psi

> actual O.K

Conclusion: The Bearing plate size (see B.) is in accordance with the ACI-Code requirement as used on this Project.

CUSTOMER Bechtel Company

Palisades 5935-C-51

METALOGICS

RYERSON JOSEPH T. RYERSON & SON, INC.

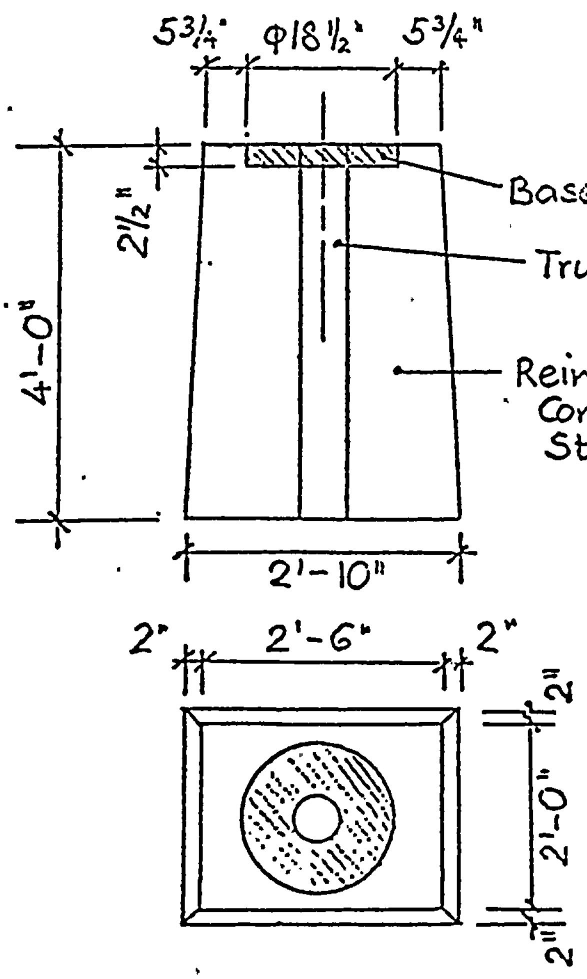
DATE 2/2/67

MADE BY A.W.

B) BASEPLATE TEST

To verify the Adequacy of Plate-thickness and Plate-Material-strength the following Test is proposed.

1) Test Set up
See Ryerson drawing SPT-1 dated 1-20-67.



Elevations

Concrete
$f_{ci}' \geq 4000$ psi

Plan

CUSTOMER Bechtel Company
Palisades 5935-C-51

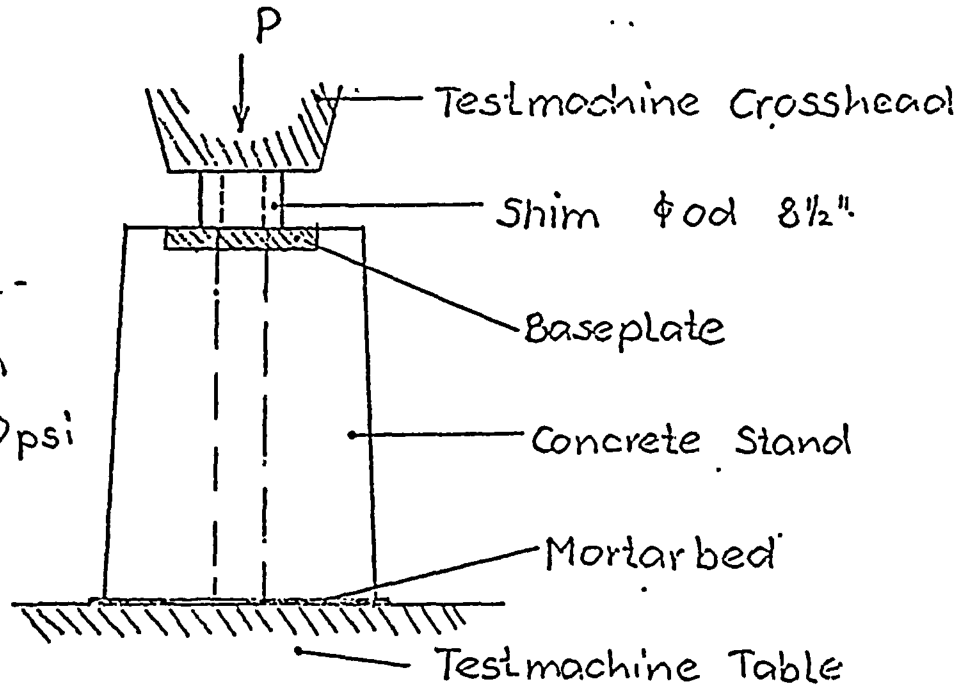
METALOGICS

RYERSON JOSEPH T. RYERSON & SON, INC.

DATE 2/2/65

MADE BY A.W.S

2) Application of Load



Concrete-Strength $f_{ci}' \geqslant 4000$ psi

a) Apply Load in increments of 106^{K} to 742^{K} max.

b) Release Load in increments of 106^{K} to zero.

c) Repeat a) and b)

d) Apply Load in increments of 106^{K} to Failure or Testmachine Capacity

e) Measure Deformations after each Load-increment of a), b) + c). (Set up see 3))

f) Observe Concrete Stand (for Cracks)

CUSTOMER Bechtel Company
Palisades 5935-C-51

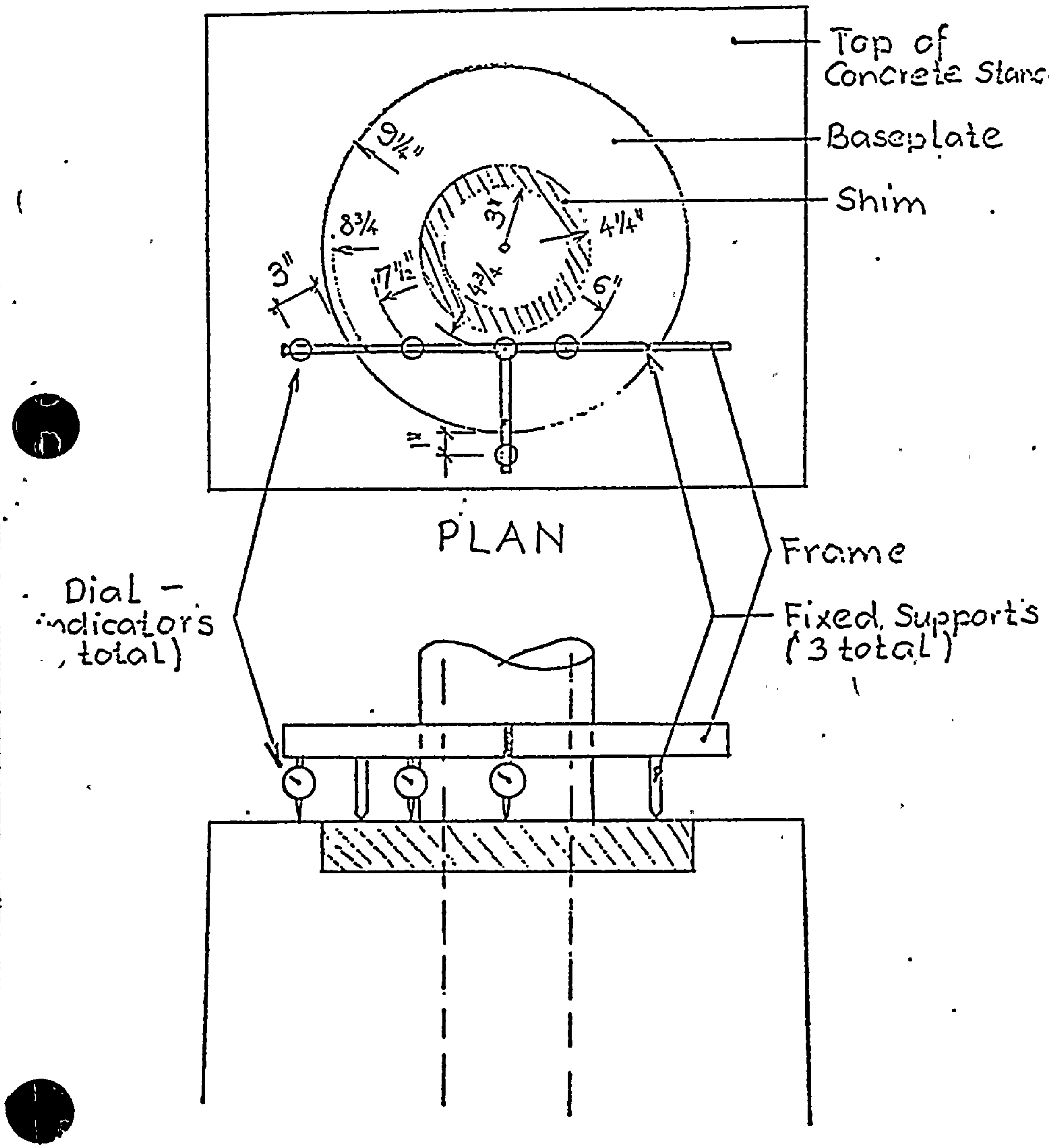
METALOGICS
RYERSON
JOSEPH T. RYERSON & SON, INC.

DATE 2/2/67
MADE BY V/S

POOR ORIGINAL

3) Deformation - Measurements

The instrumentation is shown only to illustrate the required readings.



ELEVATION

CUSTOMER Bechtel Company
Palisades 5935-C-51
RYERSON METALOGICS
JOSEPH T. RYERSON & SON, INC.
MADE BY A.WS DATE 2/2/67

4) Anticipated Test Results

a) Obervation of Concrete Stand

It is anticipated, that the Concrete Stand does not crack (other than Hair Line Cracks) up to the design Load of 742^K. The Hairline cracks to close after removing of the Load. Spalling of the unreinforced (and non structural) Concrete around the Baseplate may occur and is insignificant.

b) Observation of Baseplate

It is anticipatet, that the Plate-Material is not subjected to stresses greater than the Yield strength up to the design Load of 742^K. The deformation measurements should therefore vary Linear with the Load and indicate complete (90%) recovery during unloading. The amount of the deformation measurements to be determined later. (max. Reading < 1/16")

CUSTOMER Bechtel Company

Palisades 5935-C-51

RYERSON METALOGICS

JOSEPH T. RYERSON & SON, INC.

DATE 2/2/67

MADE BY A.Ws

The edge of the Baseplate should stay flush with the edge of the Concrete. Slight seating in is permissible, "curling up" indicates undesirable uneven bearing stress distribution.

5) Concrete Mix.

See attached Letter from Bechtel Corporation to Ryerson dated 1/27/67.

Because of the Specimen Size Limit the max. aggregate to 1½".

Perform the Test if Concrete Test Cylinders indicate a strength greater than 4000 psi

Test Cylinders shall be broken on the same day as the bearing plate test is performed.

CUSTOMER Bechtel Company
Palisades 5935-C-51

RYERSON METALOGICS
JOSEPH T. RYERSON & SON, INC.

MADE BY A.W. | DATE 2/2/67
21PT-34-114

6) Baseplate - Material

See attached Heat Test Report regarding the chemical Composition. (which meets ASTM-A36) The pysical Test-Report of representitiv Samples will follow.

POOR ORIGINAL

CUSTOMER Bechtel Company
Palisades 5935-C-51

METALOGICS

RYERSON
JOSEPH T. RYERSON & SON, INC.

DATE 2/2/67

MADE BY A.Ws.

107 REV.

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. 646099

ORDER No. CH-9583

October 24, 1966

CLIENT'S No. 212341583-18

REPORT

Report of: Compression Tests of 90-Wire Anchor Head Assembly

Report to: Joseph T. Ryerson & Son, Inc.
P. O. Box 8000-A
Chicago, Illinois 60680

We received two (2) 90-wire anchor head assemblies for compression tests in accordance with Drawing 90-PT-1A, 90-PT-2A and addendum dated 10/11/66.

The shims and anchor heads were assembled, loaded for two minutes and disassembled for examination in accordance with the drawings. The following observations were recorded.

ANCHOR HEAD ASSEMBLY 90-PT-1

Load	Remarks
742,000 lbs.	Button headed wires deformed anchor head. The 1/16" and 1/8" shims deformed slightly. Anchor head loosens by hand from adaptor lock nut.
848,000 lbs.	No apparent deformations except as noted above. Anchor head loosens by hand from adaptor lock nut.
956,000 lbs.	No apparent deformations except as noted above. Anchor head loosens by hand from adaptor lock nut.
1,007,000 lbs.	No apparent deformations except as noted above. Anchor head loosens by hand from adaptor lock nut.
1,030,000 lbs.	No apparent deformations except as noted above. Anchor head loosens by hand from adaptor lock nut.
1,200,000 lbs.	Deformations from the shim plates visible on adaptor. Anchor head no longer loosens by hand.

POOR ORIGINAL

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. [illegible]

ORDER No. [illegible]

CLIENT'S No. [illegible]

October 24, 19[illegible]

REPORT

ANCHOR HEAD ASSEMBLY [illegible]-PE-2

Load	Remarks
742,000 lbs.	Button headed wires deformed anchor head. The 1/16" and 1/8" shims deformed slightly.
848,000 lbs.	No apparent deformations except as noted above.
954,000 lbs.	No apparent deformations except as noted above.
1,007,000 lbs.	No apparent deformations except as noted above.
1,060,000 lbs.	No apparent deformations except as noted above.
1,200,000 lbs.	No apparent deformations except as noted above.

PITTSBURGH TESTING LABORATORY

Earl Gallagher

Earl Gallagher, Manager
Physical Testing Department

cc: 3-Joseph T. Ryerson & Son, Inc.
Attn: Mr. Richard H. [illegible]
1-PTL Chicago

POOR ORIGINAL

COMPRESSION TEST PROCEDURE

TEST OF 90 WIRE ANCHOR HEAD ASSEMBLY

SET UP TEST IN MACHINE PER DRAWING 90-PT-1A

APPLY COMPRESSION TO DESIGNATED LOAD (SEE TABLE BELOW)
(THIS IS A STATIC TEST, APPLY + RELEASE LOADS ACCORDINGLY)

HOLD EACH LOAD FOR A PERIOD OF TWO MINUTES

RELEASE LOAD AND DISASSEMBLE

CHECK AND REPORT ON ALL DEFORMATIONS, CRACKS, OR OTHER SIGNS OF FAILURE IN THE ANCHOR HEAD, ADAPTOR LOCK NUT, AND/OR TUBE SHIMS.

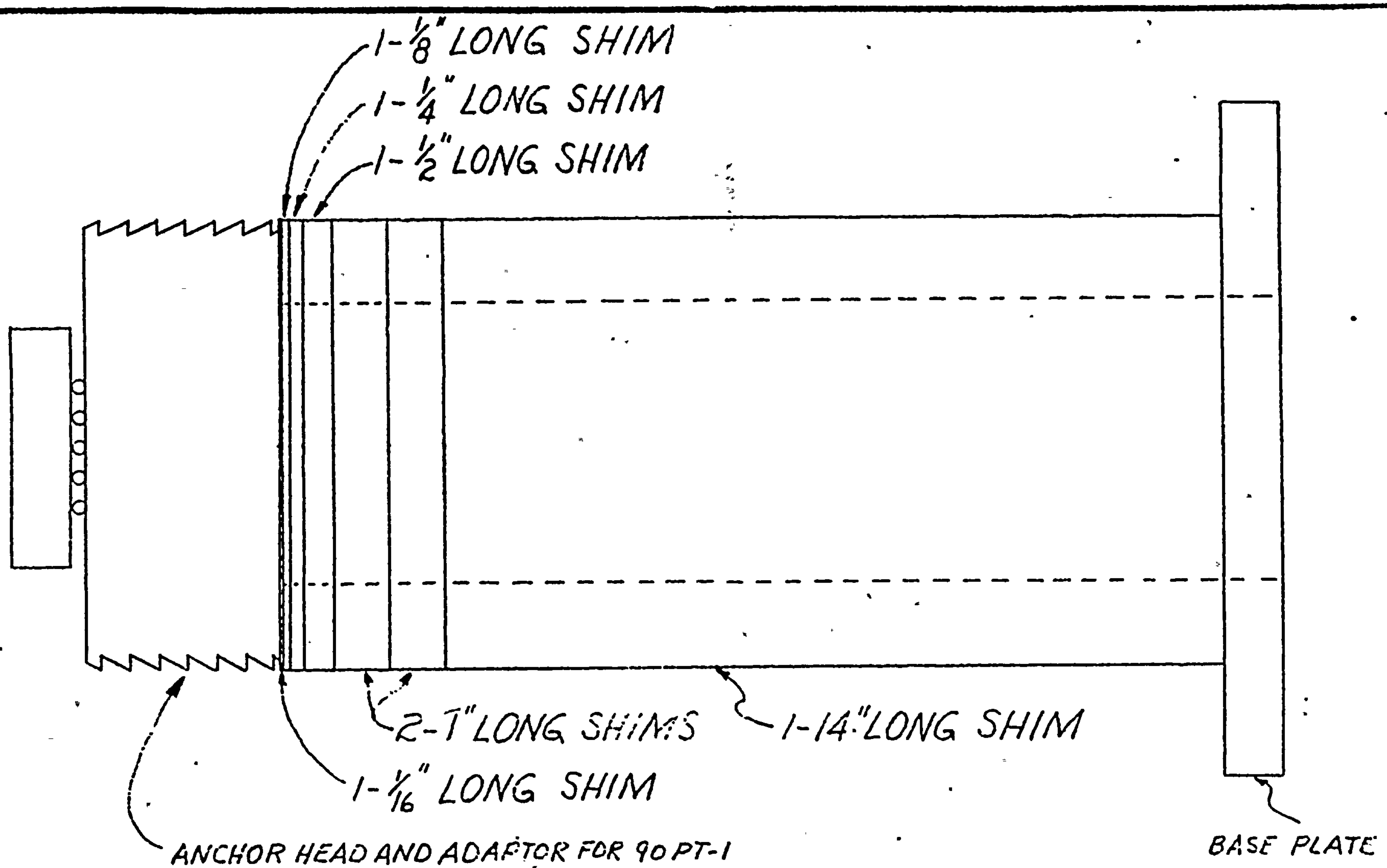
REASSEMBLE AND REPEAT AT NEXT HIGHER LOAD.

LOAD TABLE

742,000 LBS.	1,007,000 LBS.
848,000 "	1,060,000 "
954,000 "	MACHINE MAXIMUM

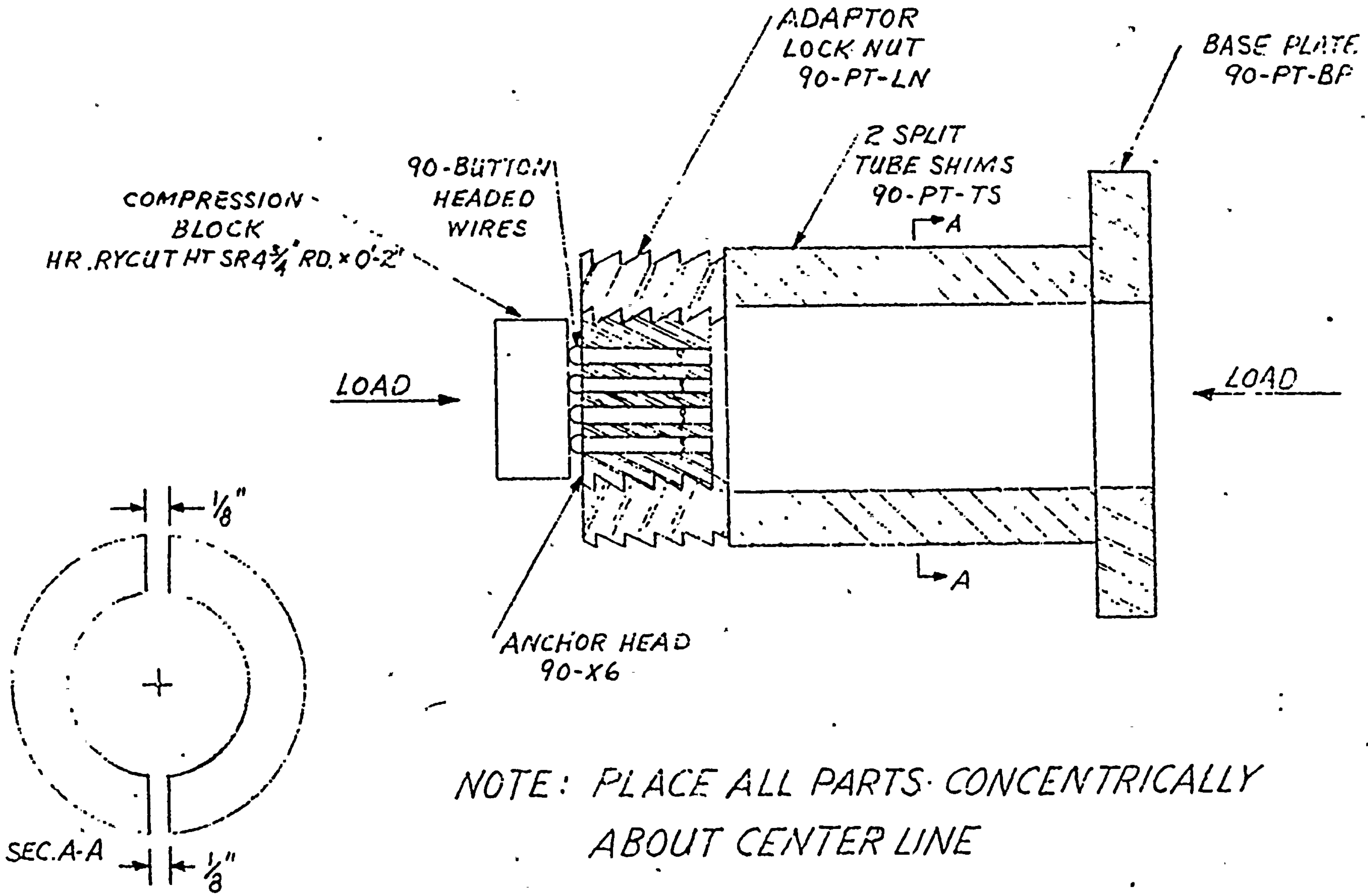
MADE BY	DATE		
RHT	7-25-66	RYERSON METALOGICS JOSEPH T. RYERSON & SON, INC.	CUSTOMER WIRE TEST
90-PT-1			ANCHOR HEAD

5A-16



FOR TESTS 90-PT1 + 90-PT2 NOTE: CHANGE IN SHIMS TO BE APPLIED

MADE BY	DATE	RYERSON METALOGICS JOSEPH T. RYERSON & SON, INC.	CUSTOMER	SHIM REVISION
RHT	10-11-66			
90-PT-1A+2A ADD				



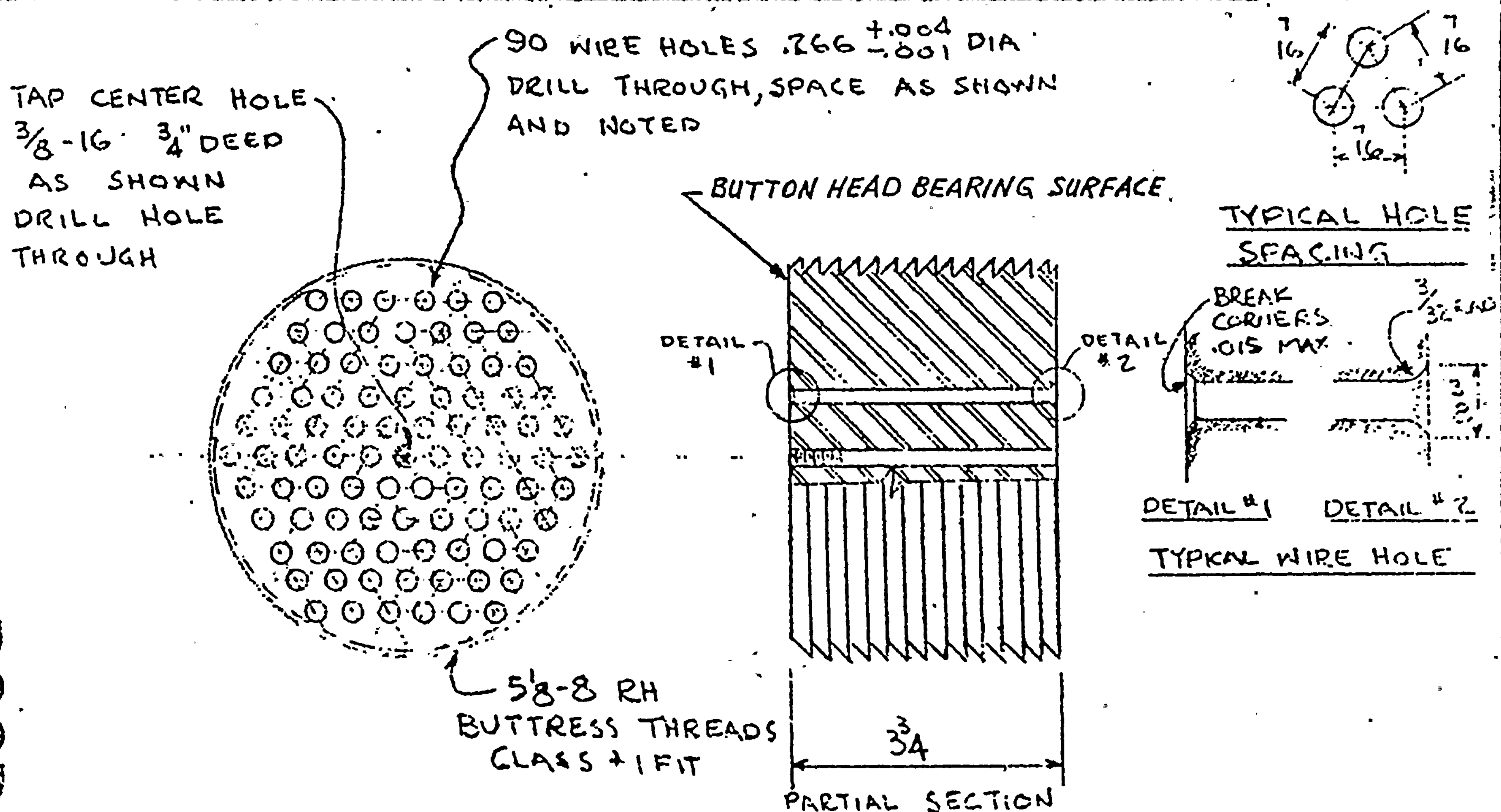
5A-18

POOR ORIGINAL

MADE BY	DATE
RHT	10-8-66
90-PT-1A	

RYERSON
JOSEPH T. RYERSON & SON, INC.
METALOGICS

CUSTOMER



90 WIRE ANCHOR HEAD

MATERIAL-
C1141 5 1/4 DIA H.R. ROUND

HEAT TREAT [redacted]

MADE BY	DATE
CLYDE	6-1-66

90-X6

RYERSON
JOSEPH T. RYERSON & SON INC
METALOGICS

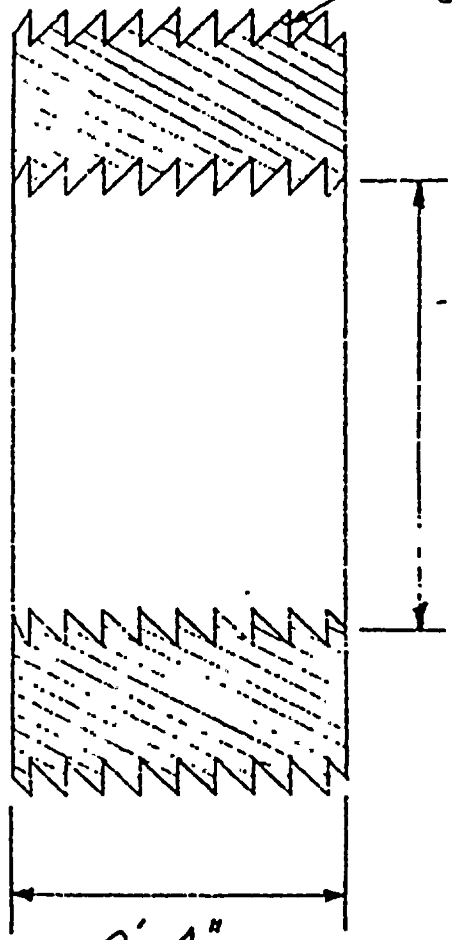
CUSTOMER [redacted]

[redacted]

5A-19

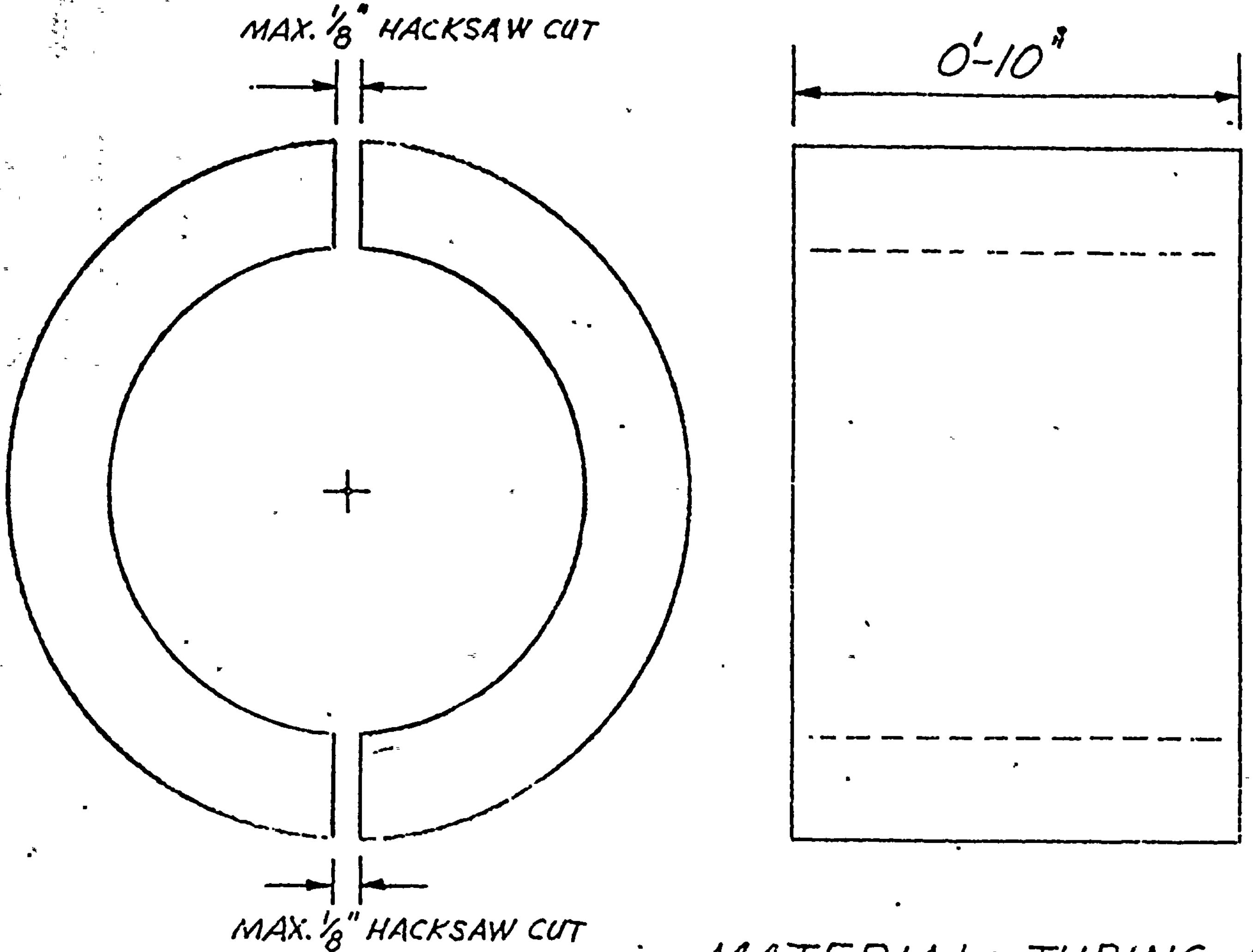
POOR ORIGINAL

5A-20



MATERIAL: HR1141 8" RD. × 0'-4"

MADE BY RHT	DATE 10-8-66	RYERSON METALOGICS — JOSEPH T. RYERSON & SON, INC.
90-PT-LN		CUSTOMER: ADAPTOR LOCK NUT



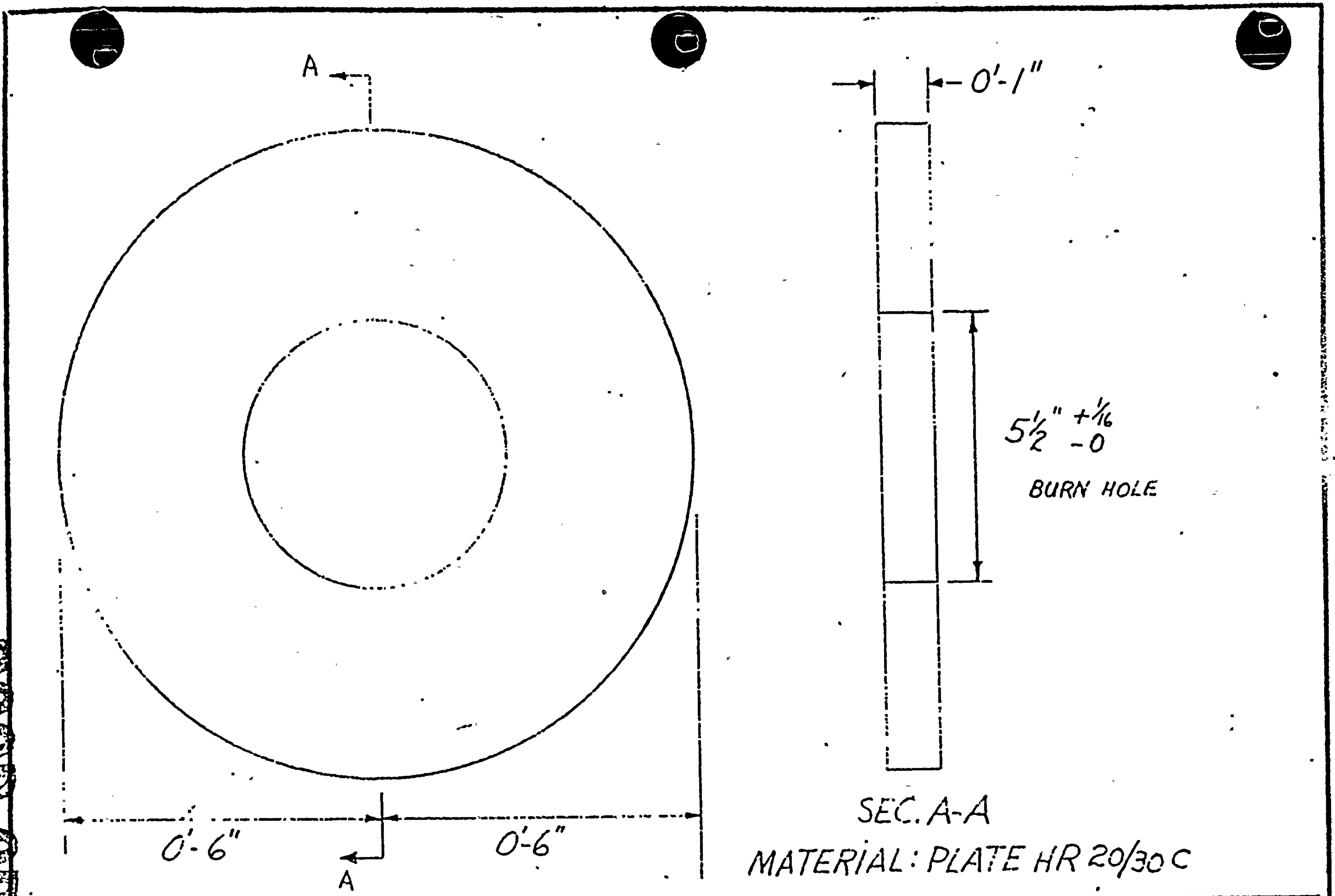
MATERIAL: TUBING HFSM
8" O.D. × 1 1/4" WALL × 0'-10"

MADE BY	DATE
RHT	10-10-66
90-PT-TS	

RYERSON
JOSEPH T. RYERSON & SON, INC.
METALOGICS

CUSTOMER TUBE SHIMS
FOR TEST ONLY

5A-21



MADE BY RHT	DATE 10-?-66	RYERSON METALOGICS JOSEPH T. RYERSON & SON INC
90-PT-BP		CUSTOMER BASE PLATE
		FOR TEST ONLY

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. 642438

August 25, 1966

ORDER No. CH-9583

CLIENT'S No. 21T341891-60

REPORT

Report of: Load Tests of Coupler and Adapter 90-11

Report to: Joseph T. Ryerson & Son, Inc.
P. O. Box 8000-A
Chicago, Illinois, 60680

Attention: Mr. W. A. Corson

We received at our laboratory one bushing measuring 11" long, 7-7/8" x 8 buttress threads on the O. D. and 5-1/8" x 8 buttress threads on the I. D., along with a pulling rod measuring 18" long with 3-3/4" of 5-1/8" x 8 buttress threads. This bushing was to be used in conjunction with the coupling identified in our Laboratory Report No. 640730. The set up was made as shown on Ryerson drawing, that is, the bushing was threaded into the 10-1/2" diameter coupling with a 5-1/4" pull rod on one end and the 8" pull rod on the other end. The assembly was then loaded and tensioned to the required loads, then released and disassembled and the threads checked both inside and outside the bushing for visible defects. It was also checked whether or not the pulling rods turned easily or with difficulty.

The results of these tests are as follows:

Load Lbs.		Remarks
742,000	Rod to adaptor	Hand turn easily.
	Adaptor to coupler	Hand turn easily.
848,000	Rod to adaptor	Hand turn easily.
	Adaptor to coupler	Hand turn easily.
954,000	Rod to adaptor	Hand turn easily.
	Adaptor to coupler	Hand turn easily.

POOR ORIGINAL

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. [illegible]

[illegible] 28, [illegible]

CLIENT'S No. [illegible]

ORDER No. [illegible]

REPORT

Load Lbs.		Remarks
1,007,000	Rod to adaptor	Hand turn easily.
	Adaptor to coupler	Hand turn easily.
1,050,000	Rod to adaptor	Hand turn easily.
	Adaptor to coupler	Hand turn easily.
1,200,000	Rod to adaptor	Hand turn easily.
	Adaptor to coupler	Hand turn easily.

PITTSBURGH TESTING LABORATORY

Earl Gallagher

Earl Gallagher, Manager
Physical Testing Department

cc: 3-Client
Attn: W. A. Corson
1-PTL Chicago

POOR ORIGINAL

TEST PROCEDURE

TEST OF COUPLER 90-X9

SET UP TEST IN MACHINE PER DRAWING

APPLY TENSION TO DESIGNATED LOAD (SEE TABLE BELOW)
(THIS IS A STATIC TEST, APPLY & RELEASE LOADS ACCORDINGLY)

RELEASE LOAD AND DISASSEMBLE

CHECK THREADS AT BOTH END OF COUPLER

CHECK FOR VISIBLE DEFECTS

CHECK FOR TORQUE REQUIRED TO TURN RODS IN COUPLER

MEASUREMENTS ARE NOT NECESSARY, QUALITATIVE REMARKS (TURNS EASILY, DIFFICULT TO TURN, ETC) ARE SUFFICENT

REASSEMBLE AND REPEAT AT NEXT HIGHER LOAD

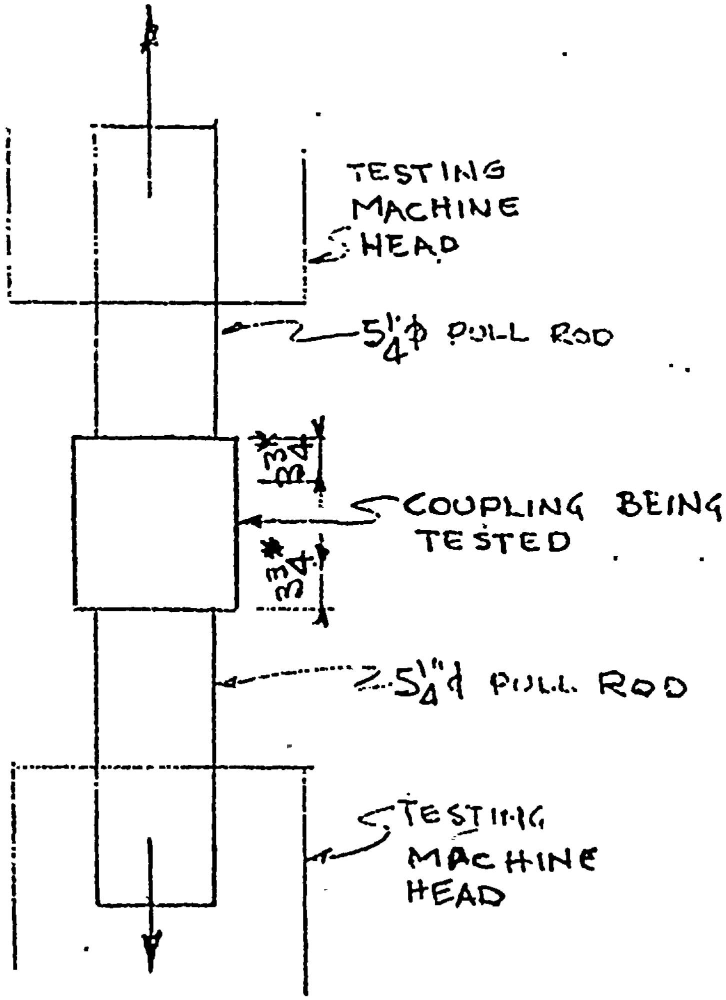
LOAD TABLE

742,000	lbs
848,000	"
954,000	"
1,007,000	"
1,060,000	"
MACHINE MAXIMUM	

MADE BY	DATE		
[illegible]	6-10-[illegible]	RYERSON METALOGICS JOSEPH T. RYERSON & SON, INC.	CUSTOMER 90 WIRE COUPLING TEST

5A-25

POOR ORIGINAL



TEST SET UP

TEST OF COUPLER 90-X9

* THREAD ENGAGEMENT TO BE 3¾" EACH END
HOLD THIS DIMENSION AS CLOSE AS POSSIBLE

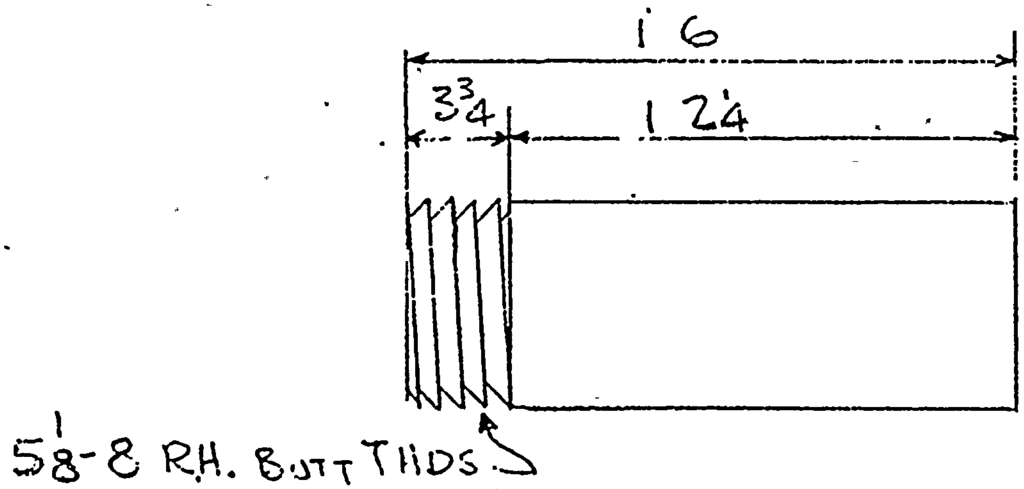
MADE BY	DATE		
	6-10-66	RYERSON JOSEPH T. RYERSON & SON, INC. METALOGICS	CUSTOMER 90 WIRE COUPLING TEST

5A-26

POOR ORIGINAL

5A-27

POOR ORIGINAL



TEST PULL ROD

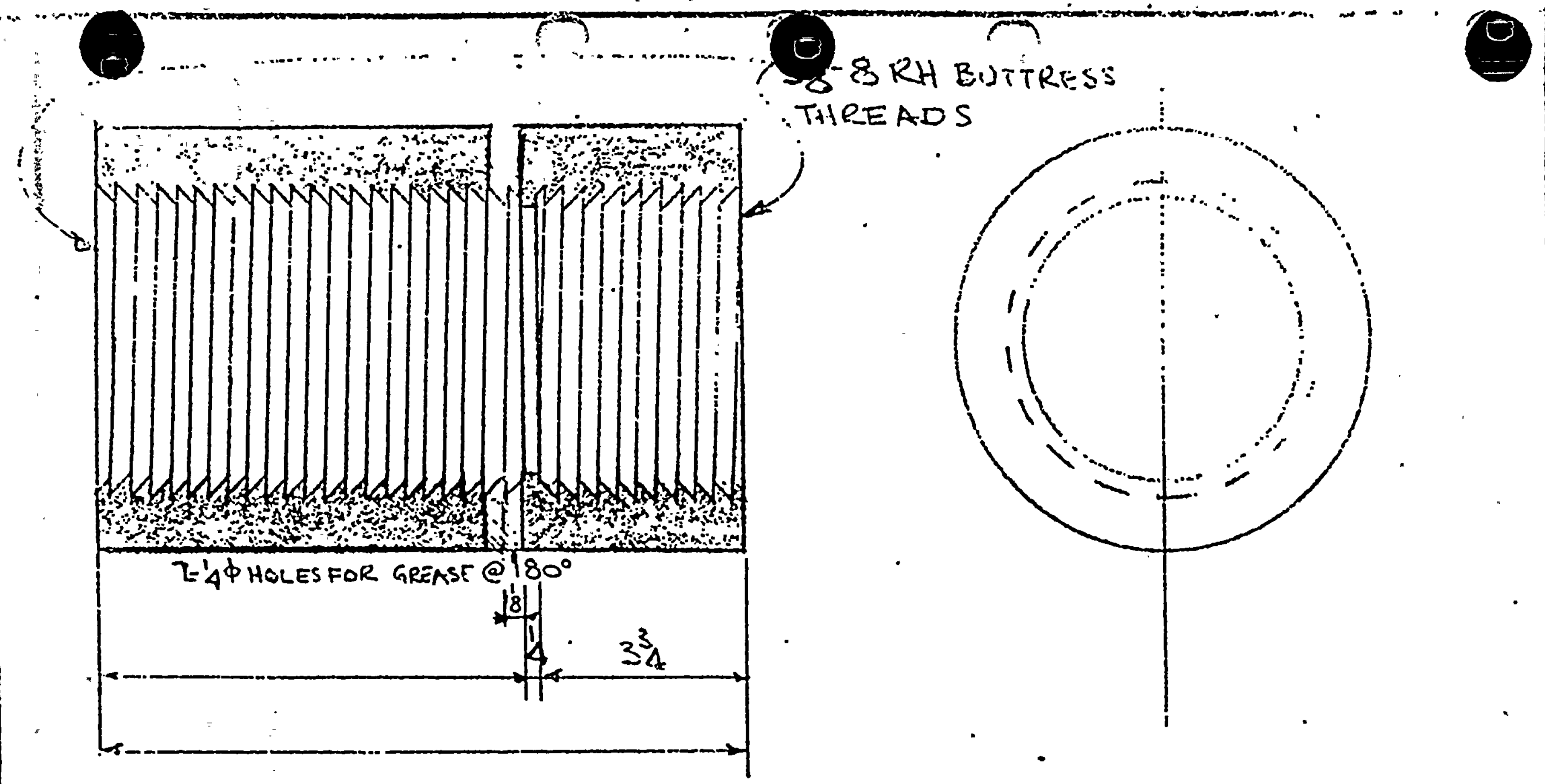
(2 REQ'D)

MATERIAL
1-5 1/4 ROUND X 1'-6
H.R. C1141

HEAT TREAT TO ROCKWELL C36-38

MADE BY	DATE		
[illegible]	6-10-66	RYERSON JOSEPH T. RYERSON & SON, INC. METALOGICS	CUSTOMER 90 WIRE
			COUPLING TEST

5A-28



90 WIRE COUPLING

MATERIAL
C.D.S.M. TUBING 6 1/2 OD X 7/8 WALL
0'-11 1/2 LONG

MADE BY [illegible]	DATE 6-2-66	RYERSON JOSEPH T. RYERSON & SON, INC. METALOGICS	CUSTOMER 90 WIRE
[illegible]-X9			COUPLING TEST

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. 655566

June 5, 1967

ORDER No. PG-18619

CLIENT'S No. Ltr. 3/13/67

REPORT

Report of: 90 Wire Tendon Test

Report to: Joseph T. Ryerson & Son, Inc.
P. O. Box 8000-A
Chicago, Illinois 60680

We received a sample which was identified to us as a 90 wire tendon. We were requested to test the sample in tension measuring elongation over a 120" gage length.

The sample consisted of 90 wires, 1/4" in diameter, with anchor heads on each end. The anchor heads were held on the wire by the wire button heads. The anchor head had external threads which threaded into a coupler. The coupler then threaded onto pull rods, 8" in diameter, which were installed in the upper and lower cross heads of our 1,200,000# testing machine.

An extensometer, modified to give a 120" gage length, was used to record sufficient data to plot the attached curve.

PITTSBURGH TESTING LABORATORY

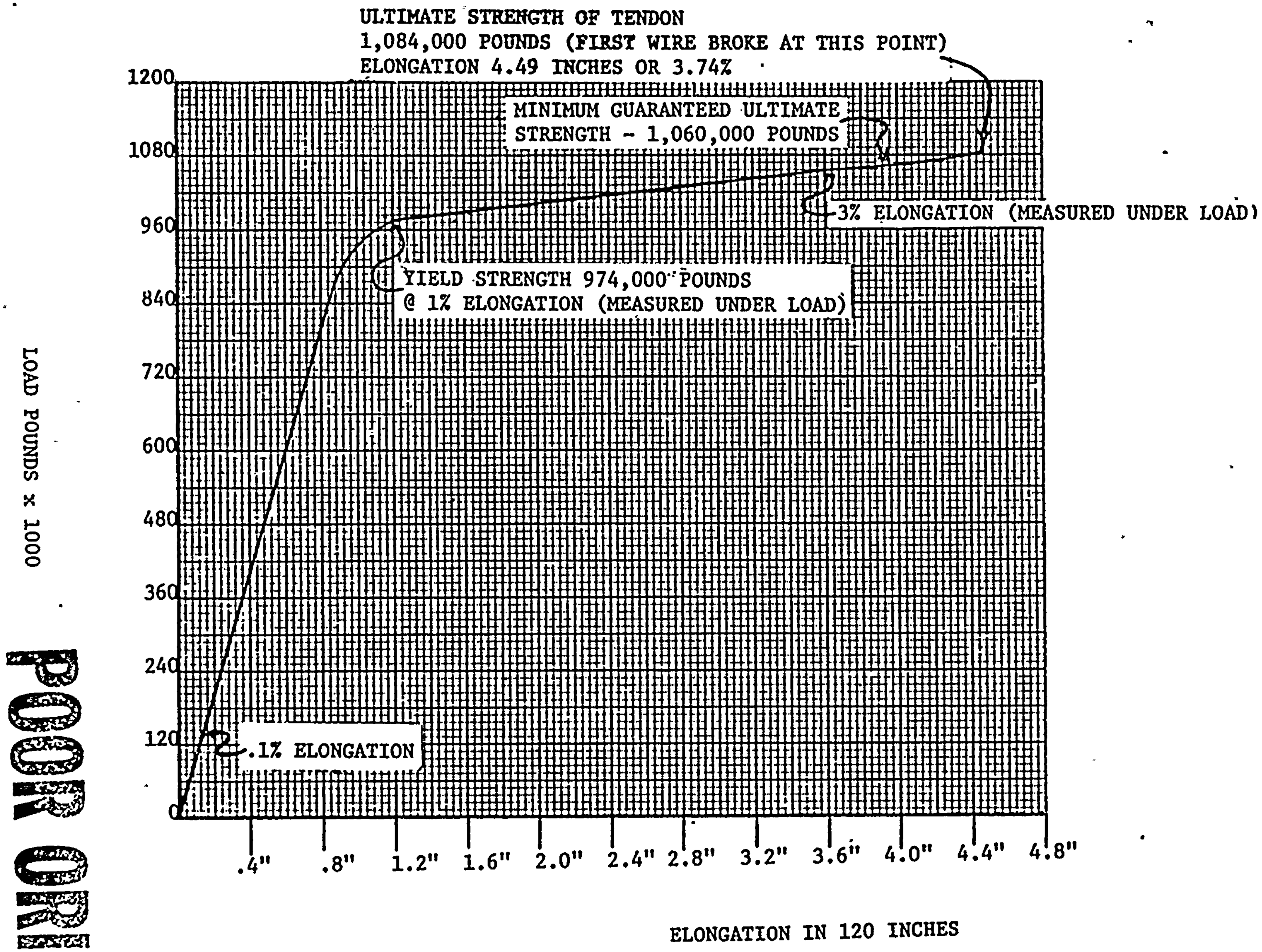
Earl Gallagher

Earl Gallagher, Manager
Physical Testing Department

cc: 3-Client
1-PTL Chicago

POOR ORIGINAL

TENSION TEST OF 90 WIRE TENDON
J.T. RYERSON & SONS, INC. PG-18619
5-26-67 65506



90 WIRE TENDON TEST

The purpose of this Test is to verify, that a Tendon, consisting of 90 wires, the wires anchored at each end in anchorheads by means of Buttonheads, is 90 times as strong as one wire. The Test further allows to measure the Tendon-elongation.

The complete Endanchors have been previously tested beyond the ultimate strength of the Tendon.

(see Ryerson 90-PT-1 dated 7/25/66
90-PT-2 7/25/66

and the corresponding Test report from PTL dated 10/24/66.)

POOR ORIGINAL

5A-31

CUSTOMER Bechtel Company
Palisades 5935-C-51

METALOGICS RYERSON JOSEPH T. RYERSON & SON, INC.

MADE BY A.W.S. DATE 2/3/67
21 PT-34-114

A. LOAD

Min. guarantied ultimate strength of ϕ 1/4" wire (see ASTM - 421)

240'000 psi

Min guarantied ultimate strength of 90 wire Tendon

$90 \cdot 0{,}04909 \cdot 240'000 = \underline{1'060'000^{\#}}$

Min. Yield strength of 90 wire Tendon, measured under Load at 1,0% extension.

$80\% \times$ ult. strength $= 848'000^{\#}$

Anticipated Test Result:
No wirebreak will occur before the Load of 1060^{K} is reached.

B. ELONGATION

Min. Tendon elongation <u>3%</u>
measured under Load
in min. Gauge length of 10 ft
(See PCI, proposed Post-tensioning Material Specifications)

CUSTOMER Bechtel Corp.
Palisades 5935-C-51
RYERSON JOSEPH T. RYERSON & SON, INC. METALOGICS
DATE 2/3/67
BY

The Elongation is to be measured as movement between Anchor-heads.

The Wirelength for the Test tendon is 10'-0" → 120"

The methode of measuring elongation shall be similar to the one specified in ASTM -421.

Initial elongation 0.1 % → 0.12" → 1/8"
Initial Stress 29'000 psi → 128 K

Yield at 1% extension → 1,20" → 1 3/16"
min yield strength 848 K

Min. Elongation 3 % → 3,60" → 3 5/8"
is to be reached before the first wire breaks.

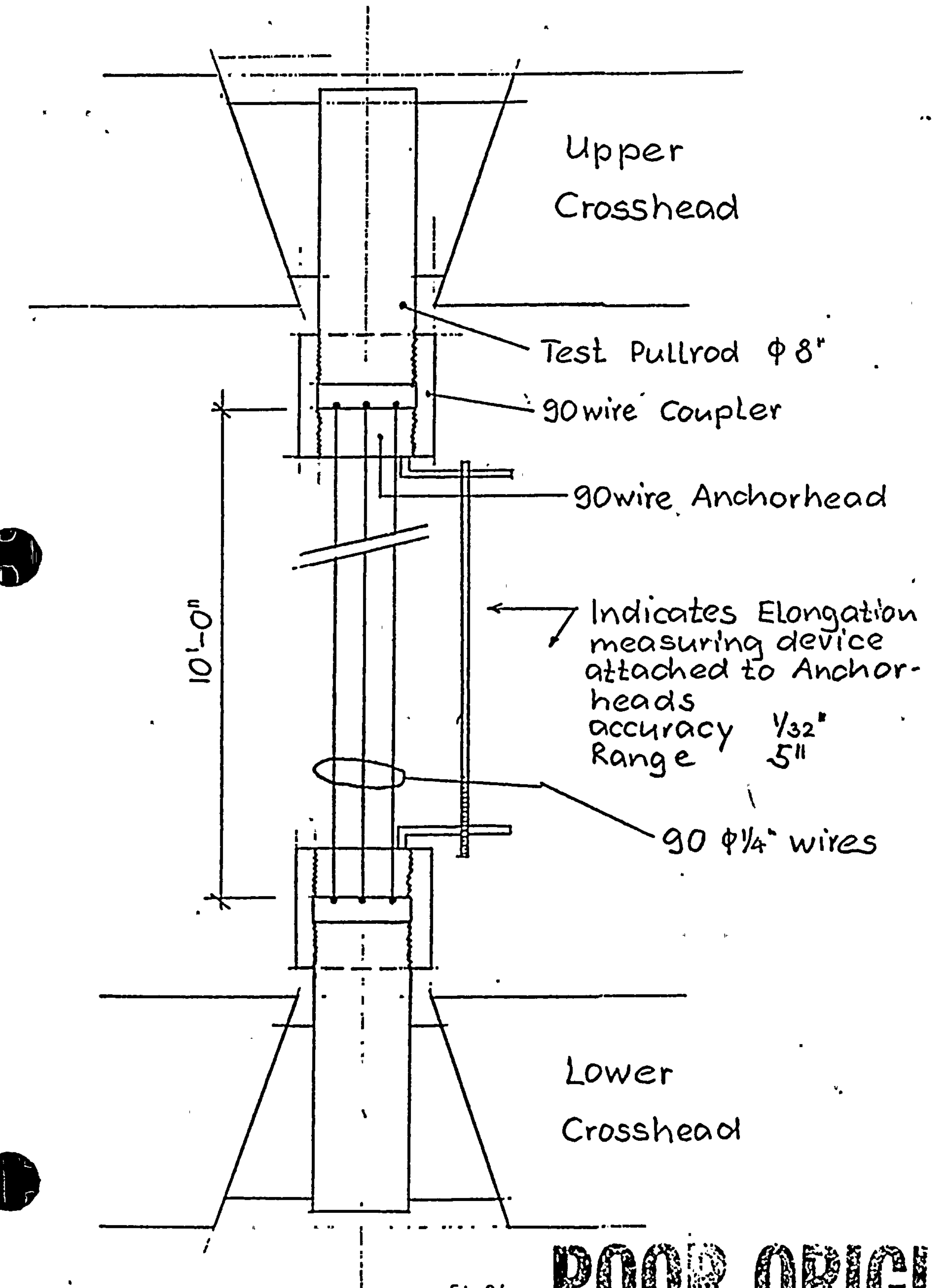
CUSTOMER Bechtel Company
Palisades 5935-C-51

RYERSON METALOGICS
JOSEPH T. RYERSON & SON, INC.

DATE 2/3/67
MADE BY A.W.S.

POOR ORIGINAL

) TEST SETUP



CUSTOMER Bechtel Company
Palisades 5935-C-51

METALOGICS

RYERSON
JOSEPH T. RYERSON & SON, INC.

MADE BY AVS | DATE 2/3/67 | PT-34-114

POOR ORIGINAL

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

LABORATORY No. 640730

August 25, 1966

ORDER No. CH-9583

CLIENTS No. 21T341891-48

REPORT

Report of: Load Tests of 90-X7 Coupler

Report to: Joseph T. Ryerson & Son, Inc.
P. O. Box 8000-A
Chicago, Illinois 60680

Attention: Mr. W. A. Corson

Submitted to our laboratory for load tests was an assembly identified as 90-X7 coupler. We were instructed to set up the coupler assembly as shown on Ryerson drawing that showed the coupler that measured 10-1/2" O.D., 8" long, with a 7-7/8"-8 buttress thread and two 8" diameter pulling rods at either end threaded into the coupler. The thread engagement at each end was 3-1/2". After the assembly was complete, we were to apply designated tension loads and release the loads accordingly. After releasing the loads we were to disassemble the assembly and check threads at both ends of the coupler for visible defects and check whether or not the pulling rods would turn easily or with difficulty from the coupler.

The results of these tests are as follows:

Load Lbs.	Remarks
	Threads lubricated with oil.
142,000	Hand turn top of pulling rod. Hand turn bottom of pulling rod.
848,000	Hand turn top of pulling rod. Hand turn bottom of pulling rod.
954,000	Hand turn top of pulling rod. Hand turn bottom of pulling rod.
1,007,000	Hand turn top of pulling rod. Hand turn bottom of pulling rod. Approximately 3 turns, strap wrenches used from then on. Evidence of thread cutting on rod. Threads on bottom rod dressed with file. Threads lubricated with "Fluoro Glide" dry lubricant.

POOR ORIGINAL

PITTSBURGH TESTING LABORATORY

ESTABLISHED 1881

PITTSBURGH, PA.

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LABORATORY No. 640730

August 25, 1966

ORDER No. CH-9583

CLIENT'S No. 21T341891-48

REPORT

Load Lbs.	Remarks
1,060,000	Hand turn top of pulling rod. Hand turn bottom of pulling rod.
1,200,000	Hand turn top of pulling rod. Hand turn bottom of pulling rod.

PITTSBURGH TESTING LABORATORY

Earl Gallagher

Earl Gallagher, Manager
Physical Testing Department

cc: 3-Client
Attn: Mr. W. A. Corson
1-PTL Chicago