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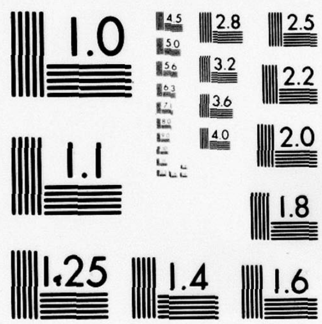

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PHASE IV REPORT

VOLUME V

RESULTS AND INTERPRETATION OF ROCK ANCHOR TEST PROGRAM

EXISTING LOCKS AND DAM NO. 26
MISSISSIPPI RIVER, ALTON, ILLINOIS

Prepared for

United States Army
Corps of Engineers
...Serving the Army
...Serving the Nation

St. Louis District

By

Woodward-Clyde Consultants
Chicago, Illinois

15 July 1979

Contract No. DACW43-78-C-0005

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PHASE IV REPORT

VOLUME V

6

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- Vol IIA Appendices A through G, Results and Interpretation of Chemical Grouting Test Program
- Vol III Results and Interpretation of Pile Driving Effects Test Program
- Vol IIIA Appendices H through T, Results and Interpretation of Pile Driving Effects Test Program
- Vol IV Results and Interpretation of Drilled-In Pile Test Program
- Vol IVA Appendices A through E, Results and Interpretation of Drilled-In Pile Test Program
- Vol V Results and Interpretation of Rock Anchor Test Program
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Drilled-in pile test	Instrumentation of tests
Pile driving effects test	Vibrational effects on structures

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

A series of tests examining various foundation systems and construction techniques were conducted on Ellis Island near Locks and Dam No. 26 in alluvial sand deposits underlain by glacial deposits and limestone. The chemical grout test consisted of grouting the upper 20 feet of the alluvial sand by injecting a number of different silicate and cement-bentonite grout types, while varying the grouting method, hole spacing, and injecting rates. Heave, lateral displacement, and pore pressure were monitored during grout injection. The in situ properties of the sand were measured before and after grouting by standard

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20. penetration tests, static cone penetration tests, pressuremeter tests, bore hole permeability tests, and shear wave velocity tests. Concurrently laboratory tests were conducted to investigate the strength and creep behavior of the grouted sand. After completion of grouting, the site was excavated to examine and evaluate the grouted sand. In the rock anchor test, inclined rock anchors were installed in limestone through 130 feet of alluvial and glacial deposits using a pneumatic down-the-hole hammer with an offset reamer. Load tests were conducted on three instrumentated rock anchors and the feasibility of installation of the rock anchors was determined by evaluating loss of ground during installation, performance of the installation equipment, and rate of installation. The drilled-in pile test consisted of installation of large diameter high capacity pipe piles by the Benoto method. The feasibility of installing these piles was determined by evaluating loss of ground during installation, performance of the Benoto equipment, and rate of installation. In the pile driving effects test, pile founded monoliths were constructed, supported on either one, eight or twelve timber piles jetted and driven in alluvial sand to a depth of 35 feet. After applying lateral and vertical load to the monoliths, steel piles were driven at varying distances from the monoliths while monitoring movement of the monolith and supporting piles; shear, moment, and axial load in the timber piles; and pore pressure, movement, and particle velocity, in the soil. Parameters examined were pile type being driven (sheet, pipe, or H-pile), pile driving hammer (diesel, air-steam, or vibratory), distance of driven piles from monolith, driving of multiple piles at the same distance from the monolith, load level applied to the monolith, and soil properties (grouted and ungrouted). Vertical and lateral load tests were conducted on each pile founded monolith. Tests were also conducted to assess what effect grouted soil has on piles. Piles were driven in both grouted and ungrouted sand to examine driving characteristics and lateral load tests were conducted on H and pipe piles in both grouted and ungrouted sand.

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VOLUME V

RESULTS AND INTERPRETATION OF ROCK ANCHOR TEST PROGRAM

SECTION 0

SUMMARY AND CONCLUSIONS

0 SUMMARY AND CONCLUSIONS

0.1 ROCK ANCHOR TEST PROGRAM

The feasibility of installing inclined anchors through submerged alluvial and glacial soil and into rock was investigated near Locks and Dam No. 26 on the Mississippi River. The tests were designed to assess whether or not rock anchor construction has adverse effects, such as loss of ground and loosening, on the surrounding soil mass, and to obtain information concerning the load capacity of rock anchors.

The tests were designed from November 1977 to May 1978. The test area was prepared from April to September 1978. The test instrumentation was installed in September and October 1978. The field tests were conducted from November 1978 to March 1979. Three inclined test rock anchors were installed and load tested. Four inclined test anchor holes were drilled and the effects of drilling were assessed by various measurements.

0.2 TEST AREA SUBSURFACE CONDITIONS

The test area was located on Ellis Island, about one mile downstream of Locks and Dam No. 26 on the Missouri side of the Mississippi River. The subsurface profile at the location of the test area consisted of approximately 100 ft of sand and gravel of alluvial and glacial origin, overlying the limestone bedrock. The subsurface conditions were investigated at the design stage, and reassessed after the tests to detect changes caused by anchor installation. The subsurface investigations relied primarily on the use of in situ testing methods (dynamic and static cone penetration, density measurements using a nuclear probe, and permeability tests).

0.3 TEST PROGRAM DESIGN

0.3.1 Test Area Selection

The test area was selected from four candidate test sites preselected by the Government. At the location of the selected test area, the subsurface conditions matched best the conditions at Locks and Dam No. 26. To a variable degree, these conditions are also representative of those at other navigation structures on the Mississippi River.

0.3.2 Selection of Anchor System and Installation Method

Anchor System. VSL multiple stranded cable anchors were selected for the tests. This anchor system has been commonly used for long, high-capacity, permanent anchor installations.

Installation Method. The Atlas-Copco ODEX 165 overburden method was used to drill the anchor holes. This method was selected at the design stage on the basis of past performance of various methods in conditions similar to the test conditions. Among all the drilling methods considered, the ODEX 165 appeared to have the best potential for drilling 8-in.-dia cased holes, at an inclination of 45 degrees through over 100 ft of submerged alluvial and glacial sand and gravel. The ODEX system originated in Sweden and has been used in the USA for several years. The ODEX 165 is the largest equipment in the ODEX series and is relatively new; the 165-size had not been used in the USA prior to this test program. The principle of all ODEX equipment is the same: simultaneous advancement of a casing and an eccentric bit by combination of casing driving (top or down-the-hole hammer), bit rotation and air or fluid flushing.

The ODEX 165 provides the required anchor hole diameter. It is operated with a down-the-hole (DTH) hammer. The use of a DTH hammer was deemed necessary to successfully advance the 180-ft-long, inclined casing. Methods using top hammer or rotary methods using drilling fluid were considered, but did not appear to be adequate for the demanding drilling requirements of the tests.

0.3.3 Anchor Load Tests

Three instrumented anchors were fabricated at the site, installed, grouted into rock, and load tested. The anchors were instrumented to monitor response to load application. One of the test anchors was similar to anchors that could be used in actual construction (prototype anchor). The other two test anchors were specially designed to incorporate instrumentation and features facilitating the detection and interpretation of load transfer mechanisms during stressing.

0.3.4 Drilling Effects Test

The drilling effects test consisted of drilling four holes at 9-ft spacing and at an inclination of 45 degrees. As the test drill holes were successively drilled, the effects of drilling on the surrounding soil mass were assessed by various methods. The quantity of cuttings expelled from the drill holes was measured periodically and compared to the theoretical quantity calculated on the basis of the theoretical volume of the hole and the measured in situ unit weight of the soil. The ground deformation caused by drilling was periodically assessed on the basis of ground instrument (vertical and inclined inclinometers, and various types of surface and deep settlement points) measurements. The overall drilling effects were also assessed by measuring changes on in situ soil properties before and after drilling; a static cone penetrometer was used for this purpose.

0.4 TEST RESULTS

0.4.1 Drilling Effects Test

Loss of ground was large during anchor drilling. The quantity of soil expelled from the drill holes was generally 2 to 5 times the theoretical quantity. Locally larger loss of ground was experienced due to drilling difficulties.

Ground deformation resulting from loss of ground was large: maximum observed ground settlement was 0.4 ft at a depth of 80 ft below ground surface. Corresponding maximum ground surface settlement was 0.23 ft. Large lateral soil deformation was also observed.

The ground disturbance resulting from loss of ground was significant. The relative density of the soil surrounding the drill holes was reduced from approximately 70 percent before drilling to as low as 40 percent and locally close to zero after drilling.

0.4.2 Anchor Load Tests

None of the three anchors tested failed. The prototype anchor, having an 18-ft-long anchorage length in rock, sustained a test load of 480 k (20 percent of the design load); at that load, the total tendon elongation was 13.3 in. The two special design anchors, having either a 10- or 15-ft-long anchorage length in rock, sustained a test load of 800 k (twice the design load); at that load, the total tendon elongation for both anchors was 15.4 in. and 16.3 in. respectively. These results are in good agreement with predictions made at the design stage.

The stress distribution in the anchorage zone was complex, involving both compressive and tensile stresses; this is attributed to the shape and configuration of the tendons in the anchorage zone. Generally, the upper portion of the anchorage zone was in compression, the middle portion was in tension, and the bottom portion was relatively unstressed.

Friction losses dissipated at the anchor stressing head and along the free length of the anchor amounted to about 6 percent of the applied load. This means that when a load of 400 k was applied to the anchor head, only 376 k reached the anchorage zone of the anchor.

Lock-off losses or losses experienced during the removal of the jack and seating of the lock-off mechanisms at the end of the anchor stressing amounted to 14 to 20 percent of the design load. This means that although the anchors were stressed to 400 k, the residual load after lock-off was only 346 k to 319 k. These losses are larger than values normally reported by anchor suppliers.

Long-term behavior of the anchors was extrapolated on the basis of a 2.5-month monitoring period. At the rate of loss of load experienced during that period of time, the anchors would be expected to lose about 50 k in 50 years or 13 percent of the load after lock-off.

0.4.3 Assessment of Drilling Method

Various difficulties were experienced with the drilling method. The difficulties were generally the cause of the large loss of ground discussed in Section 0.4.1. Some of the difficulties stemmed from technological problems (incompatibility of tools and casings) and from inexperience of the operators. Other difficulties, however, are inherent to the ODEX 165 system used.

0.5 SUMMARY OF CONCLUSIONS

The following conclusions are based on the results of the rock anchor test program:

(1) the ODEX 165 drilling system is marginally feasible. The tests proved that the system can be used to install 8-in.-dia casing, 180-ft-long, at an inclination of 45 degrees through submerged alluvial and glacial sand and gravel. However, the following results indicate that the system has serious drawbacks:

- (a) large loss of ground (2 to 5 times the theoretical volume of the drill hole) was observed during drilling. Most loss of ground was inherent to the system which consisted of advancing a casing and flushing soil cuttings using a large volume of compressed air;
- (b) the large loss of ground results in large ground movements both at ground surface and at depth. Settlement of several inches was observed; and
- (c) the production rate was extremely slow due, to a large extent, to the demonstrated need for welding each successive casing section as the casing was advanced;

(2) VSL tendons are adequate to provide desired load capacity. They can be installed in the anchor hole with no major difficulty; and

(3) 400-k design capacity anchors can be achieved with as little as 10 to 12 ft of anchorage length in limestone.

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VOLUME V

RESULTS AND INTERPRETATION OF ROCK ANCHOR TEST PROGRAM

SECTION 1

PURPOSE AND OBJECTIVES

1 PURPOSE AND OBJECTIVES

1.1 PURPOSE

The rock anchor tests described in this Volume V were part of an investigation and test program designed to provide comprehensive technical bases for the evaluation of various overwater construction schemes and techniques that could be used adjacent to loaded navigation structures such as Locks and Dam No. 26. One potential reinforcement scheme was to install inclined rock anchors through the foundation soil and into bedrock under the dam piers and the lock walls. The purpose of the rock anchor test program was to assess the feasibility of constructing high-capacity, permanent rock anchors under conditions similar to those present at Locks and Dam No. 26.

The investigation and test program was conducted on Ellis Island, approximately one mile downstream of Locks and Dam No. 26 on the Missouri side of the Mississippi River. In addition to the rock anchor tests, the test program also included an assessment of chemical grouting in alluvial sand (Volume II), an assessment of pile driving effects (Volume III), and an evaluation of the construction feasibility of drilled-in piles (Volume IV). Summaries of conclusions for each of these tests are presented in Volume I.

The overall foundation investigation and test program was performed under contract DACW43-78-C-0005 between the US Department of the Army, Corps of Engineers, St Louis District and Woodward-Clyde Consultants, Chicago, Illinois.

1.2 OBJECTIVES

The objectives of the rock anchor test program were:

(1) to assess the effects of rock anchor drilling on the surrounding soil mass;

(2) to define the physical requirements of permanent rock anchors to achieve a design capaity of 400 k; and

(3) to obtain data that can be extrapolated to predict long-time performance of permanent rock anchors.

1.3 ORGANIZATION OF VOLUME V

The concept of the rock anchor test program, including the design approach, test variables, and expected performance, are discussed in Section 2. The subsurface conditions of the test area are discussed in Section 3. Section 4 presents a description of the anchor load tests. The methods and effects of drilling are described in Section 5.

PHASE IV REPORT

VOLUME V

RESULTS AND INTERPRETATION OF
ROCK ANCHOR TEST PROGRAM

SECTION 2

TEST PROGRAM DESIGN

2 TEST PROGRAM DESIGN

2.1 CONCEPT OF THE TEST PROGRAM

2.1.1 Test Area Selection

At the onset of the project, the Government preselected four candidate test site locations for the entire foundation and test program. On the basis of existing geotechnical and topographical data, it was concluded at the design stage that, although no candidate test site location exactly matched the conditions at Locks and Dam No. 26, Ellis Island was the most desirable location for this investigation and test program. This preliminary test site location was confirmed on the basis of subsurface investigations conducted at Ellis Island during the winter of 1978.

2.1.2 Approach

The rock anchor test program was designed such that the test conditions generally modelled the conditions at nearby Locks and Dam No. 26. The subsurface conditions at the test site approximately matched the conditions of Locks and Dam No. 26 (Section 3). The test anchors were designed and constructed to represent to an acceptable degree rock anchors that could be considered for use at the dam site.

The significant aspects of performance that needed to be measured during the tests to achieve the program objectives were selected at the design stage. These are discussed in Section 2.3. These aspects were ranked to allow first, measurement of gross preformance, and second, understanding of mechanisms. By gross performance, it is meant the total, unrefined observations and measurements, free of any manipulation of the data. The tests were designed to enhance the aspects of performance that had been selected as significant to the interpretation of the results.

Predictions of the outcome of the tests were made by the following process:

(1) assessment of test conditions;

(2) development of a simplified model for these conditions;

(3) selection of mechanisms believed to act in the tests;

(4) selection of a prediction method based on past experience (Section 2.3);

(5) selection of parameters involved in the prediction method and consistent with the model developed in (2);

(6) analysis using selected method and parameters to calculate predictions;

(7) portrayal of the predictions to facilitate comparisons with measured test results; and

(8) comparison between predicted and measured test results to assess reliability of prediction methods and, if necessary, improvement of these methods.

On the basis of the predicted test performance, type, location, and sensitivity of the instrumentation required to measure the significant aspects of performance were selected. Instrumentation measurement schedule during testing followed the priority ranking established for the aspects of performance, that is, first, gross performance measurement and second, mechanisms detection.

2.1.3 General Description

The rock anchor test program consisted of two separate tests: anchor load tests, and drilling effects tests.

Anchor Load Tests. The anchor load tests consisted of fabricating and installing rock anchors using a system commonly available for high-capacity permanent anchors. The anchors were installed at an inclination of 45 degrees through a reaction structure and grouted into rock. The anchors were then loaded and certain response characteristics were measured to obtain information on anchor capacity and long-term performance (Section 4).

Drilling Effects Test. The drilling effects test consisted of drilling holes for prototype anchors. The holes were drilled at an inclination of 45 degrees through the soil overburden and into rock. As drilling progressed, measurements were made to obtain information on the amount of disturbance to the soil mass caused by drilling (Section 5).

Sequence of Activities. The rock anchor test program included the following activities:

(1) initial subsurface investigations (winter 1977-1978);

(2) site preparation (summer 1978);

(3) ground instrumentation and additional subsurface investigations (September-October 1978);

(4) fabrication of test anchors (November-December 1978);

(5) test anchor installation (November 1978-January 1979);

(6) test anchor loading and monitoring (December 1978-March 1979);

(7) drilling of test holes including measurement of ground loss and ground movement (December 1978-March 1979); and

(8) measurement of final soil properties (March 1979).

Configuration of Test Area. The location of the rock anchor tests within the Ellis Island test site is shown in Fig. 2.1. The general configuration of the test area is shown in Fig. 2.2.

The anchor load tests were conducted in an area 250 ft by 125 ft (Fig. 2.2). A reinforced concrete reaction structure, supported on vertical and batter steel H piles, was constructed across the width of the area. A working area 100 ft by 125 ft was prepared forward of the reaction structure for anchor fabrication, drilling and installation activities, and for the load testing facilities. The area above the alignment of the anchors was reserved; no other construction activities, which may have disturbed the load tests, were allowed there.

The drilling effects test was conducted in an area 250 ft by 75 ft (Fig. 2.2). The configuration of the drilling effects test area reflects the probable positioning and inclination of anchors in an actual construction scheme. A 6-in.-thick concrete slab 30 ft by 20 ft in plan was constructed to maintain a clean and level working area for the drilling equipment. Areas 70 ft behind and 25 ft on either side of the drill hole positions were prepared for construction operations. These areas were used for maneuvering the drill rig, storage and handling of drilling tools and equipment, and positioning of auxiliary equipment.

Ground instrumentation for the drilling effects test was installed from the ground surface along and adjacent to the alignment of the drill holes to measure ground movement and disturbance from drilling. An area 150 ft by 75 ft in plan above the alignment of the drill holes was prepared for installation and reading of the instruments. Two inclined inclinometers were also installed adjacent and parallel to the 4-drill hole array. The inclined inclinometers were installed from the concrete working slab. Figure 2.2 also shows the instrumentation area forward of the drill hole positions.

2.2 SELECTION OF VARIABLES

2.2.1 General

During formulation and design of the test program, a wide range of potential test variables and parameters was examined. In identifying the test variables, attention was focused on providing an acceptable degree of similitude between the tests and existing structures. Consideration was given to scale, site conditions, anchor capacity, and construction details. Consistent with the purpose and objectives of the rock anchor test program stated in Section 1, only variables and parameters essential to the satisfactory performance of the program were selected. The test variables are discussed below.

2.2.2 Anchor System

The anchor system chosen for the test program was that manufactured by VSL Corporation. It is a common system consisting of seven-wire, 0.5-in.-dia, stranded steel cables. In the anchorage zone, the cables of the anchor tendons were spread and gathered to form a series of hour-glass shapes. In the free length, the tendons were isolated from the grout. The isolation schemes, developed by VSL, consisted of covering each cable with grease and a plastic sheath at the factory. The anchor load lock-off mechanism of the VSL system consisted of cone-shaped wedges gripping the cables. The wedges were set in compatible seats in the anchor head. Figure 2.3 shows the typical VSL anchor system.

2.2.3 Rock Parameters

The rock parameters selected at the design stage for the anchor load tests were:

(1) rock unconfined compressive strength:
$q_u = 3200$ k/ft^2; and

(2) grout-rock ultimate bond strength (assuming uniform distribution):
$t_{ult} = 50$ k/ft^2.

2.2.4 Anchor Characteristics

The length of the anchorage zone (anchorage length) for the load test anchors was proportioned so that the controlling mode of failure was rupture of the grout-rock bond. The size of the tendons were chosen so as not to overstress the steel during testing. The anchorage lengths were varied to measure grout-rock bond strength. The anchor characteristics were as follows:

Test	Anchor No.	No. of Strands	Hole Dia in.	Free Length ft	Anchorage Length* ft
Design	RD-1	25	6	184.6	15
Design	RD-2	24**	6	178	10
Prototype	RP-1	17	5.5	180	18

* Anchorage length was measured from top of sound rock

** Initial No. of strand was 25; one strand was damaged and broke during extraction of the drill casing

2.2.5 Drilling Method

The drilling method used for installation of the test anchors and assessing drilling effects was the ODEX 165 system, Atlas-Copco. Compared with other available drilling methods, the ODEX system was considered the most appropriate to succeed in the demanding drilling conditions anticipated. Section 5.3 presents a detailed discussion on the ODEX system, its selection, and its performance during the test program. The drilling tools consisted of a 7-5/8-in.-dia casing, a 6-in.-dia bit, and an eccentric reamer that cut a 8-in.-dia hole. The bit and casing were driven by a down-the-hole (DTH) hammer and a rotation motor having a torque of 4400 ft-lb. The minimum compressed air requirements for operation of the hammer were 450 ft^3/min at 150 lb/in^2. The air used to operate the DTH hammer was also used to flush the cuttings from the hole. After the casing was seated in rock, the drill hole was continued by standard rotary-percussion, 5.5- and 6-in.-dia rock bits. Figure 2.4 shows the sequence of drilling with the ODEX system.

2.3 SIGNIFICANT ASPECTS OF PERFORMANCE

2.3.1 General

The tests were designed so that certain aspects of the rock anchor performance and drilling effects could be monitored. The significant aspects of performance which were selected at the design stage and were those considered necessary to meet the objectives of the tests and to extrapolate the results to other conditions. Consistent with the objectives of the rock anchor test program (Section 1.2), the aspects of performance that were of primary importance were those reflecting the gross behavior of the anchor system. The aspects of performance that were of secondary importance were those reflecting the mechanisms governing anchor behavior.

The following aspects of performance were identified for the rock anchor test. They are listed in order of importance and priority.

Priority	Aspect of Performance	Measurable Characteristic
Primary		
(1)	Effects of drilling in the soil overburden	ground movement -quantity of cuttings
(2)	Ultimate anchor capacity	-ultimate load -net anchor movement
(3)	Long-term anchor capacity	-sustained laod
Secondary		
(4)	Bond stress distribution along anchorage zone	-strain within anchorage zone
(5)	Ultimate bond strength	-movement within anchorage zone
(6)	Friction losses	-load and elongation during a load-unload cycle
(7)	Lock-off losses	-load change during lock-off -anchor head movement during reloading

2.3.2 Prediction Method

Effects of Drilling. Several methods are available for advancing a drill hole through overburden soil and into rock, although drilling 180-ft-deep holes, at 45 degrees through submerged sand and gravel is an uncommon achievement. The most common method, including several variations, consists of rotating and pushing a casing through soil and seating it into the upper portion of bedrock. The cuttings are flushed out of the drill hole with water or bentonite drilling fluid. The drilling fluid is pumped down the inside of the casing and returns to the surface around the outside of the casing. Sometimes, the overburden is predrilled with a roller bit and the casing installed after the bit is withdrawn. After the casing is seated into the rock, the rock is then drilled with rock bits attached to drill rods inserted into the casing. The drilling fluid is pumped down through the drill rods and returns to the surface inside the casing.

An overburden drilling method involves advancing a drill bit attached to drill rods together with the casing. The drill bit leads the casing, and with an eccentric reamer, cuts a hole slightly larger than the casing. This allow the casing to pass through obstructions and reduces friction on the outside of the casing. The drilling fluid is pumped down the inside of the drill rods and returns to the surface with the cuttings inside the casing. When the casing has been seated into the upper portion of bedrock, the bit and eccentric reamer are exchanged for a conventional rock bit to advance the hole to the desired depth into rock (Fig. 2.4). Further details of the overburden method and comparison with rotary methods are presented in Section 5.3.

Load Capacity. Rock anchors develop their capacity by frictional resistance to pull-out or bond along the anchorage zone grouted in the rock. It is generally assumed for design purposes that the bond stress is uniformly distributed along the grout-rock interface. Thus, the ultimate capacity of the anchor is a function of the diameter and the length of the grouted anchorage.

Few tests on rock anchors have been carried to grout-rock bond failure. Littlejohn and Bruce (1977) list grout-rock bond stress values that have been used successfully in practice. They note that the ultimate bond strength values are about 10 percent of the unconfined compressive strength of the rock. Coates (1970) suggests this is conservative for low-strength rock and proposes ultimate bond strengths of 20 to 35 percent of the unconfined compressive strength.

For limestone formations similar to those at Locks and Dam No. 26, various empirical values of uniformly distributed ultimate bond strength have been suggested. These suggested values generally range from 30 k/ft^2 (PCI 1974) to 60 k/ft^2 (Losinger and Co 1966), but have been as high as 100 k/ft^2 (Ruttner 1966).

The actual ultimate grout-rock bond strength in the anchorage zone is influenced by several factors, among which are the degree of weathering of the rock, the roughness of the sidewalls of the hole, and the flushing and grouting procedures. Load transfer from the tendons to the grout is influenced by the type of tendon. Most theoretical studies have been made on single smooth bars embedded in grout. Little work has been done on multi-element tendons. The use of spacers and centralizers in the anchorage zone, and tendon geometry further complicate the mechanism of load transfer and bond stress distribution. Most designers specify preconstruction load tests to verify the bond strength values and the factor of safety in their design.

Factors of safety applied to the ultimate bond strength to obtain allowable design values have ranged from two to three in practice. The tendency has been to use a lower factor of safety in sound rock, and a higher factor of safety in fissured and weathered rock.

The diameter of the anchorage zone is usually dictated by the requirements of the steel tendons, considering tendon spacing for transfer of load and grout cover for bond development and corrosion protection. The length of the anchorage required to devleop the anchor capacity is calculated using empirical values of grout-rock bond for the particular rock formation.

The overall factor of safety of a production anchor (which is the smallest factor of safety on any of its components) is dependent on the tendon design because the strength properties of steel tendons are well known and allowable stresses are dictated by code. The properties of grout-rock bond are generally not well known and can be influenced by local conditions. Thus, dimensions of the anchorage for test anchors are proportioned so that the controlling mode of failure is rupture of the grout-rock bond. The size of the tendon is chosen so as not to overstress the steel during stressing.

Long-Term Behavior. The long-term behavior of anchors is primarily concerned with the loss of load over a period of time. Corrosion of the exposed hardware and anchor tendons, which may lead to sudden loss of load of an anchor is beyond the scope of this study. However, means of protecting the anchor components from corrosion have been developed and information on their performance is available from anchor system suppliers. Loss of load on an anchor, other than that due to corrosion, is attributable to relaxation of the steel of the tendons and creep of the rock and grout in the anchorage zone.

Accurate information on the relaxation of high-strength steel bars, wires, and cables commonly used in anchor tendons is available from steel suppliers. For the VSL cable system used in the test program, the loss of load due to relaxation can be as high as 6 percent in 1000 hr after stressing, with 4 percent occurring in the first 100 hr (these values are for tendons stressed to 70 percent of ultimate tensile strength and at 20°C).

Information on the creep in the anchor zone is available through documentation of case histories (Littlejohn and Bruce 1977). However, the amount of creep alone has not been directly measured; rather, total loss of load due to creep, relaxation, and other mechanisms has been monitored over extended periods up to 18 yr. The portion of the loss of load due to creep can be surmised by subtracting the theoretical loss of load due to steel relaxation from the total measured loss of load. However, the difference may represent other mechanisms, such as further seating of the lock-off mechanism, unraveling of multi-element tendons, closing of joints and fissures in the rock, and creep in the concrete of anchor blocks. Most of the contribution from these other mechanisms can be minimized by cycling the anchor load during stressing, or restressing the anchor several days after initial lock off. Nevertheless, the total loss in load, regardless of the cause, is of interest. Of the case histories documented and collected by Littlejohn and Bruce (1977), the majority of the anchors exhibited a loss in load of 5 percent to 10 percent of the load at lock-off, with occasional reports of 12 percent to 18 percent. Most construction codes require locking-off anchors up to 10 percent over the design load as an allowance for relaxation and creep.

2.3.3 Performance Prediction

At the design stage, predictions of the aspects of performance of primary importance were made. Predicted values of the significant characteristics are tabulated below.

Aspect of Performance	Measurable Characteristic	Predicted Value
Effects of drilling	Maximum surface settlement (el 420)	0.3 in.
	Maximum ground movement at el 350	0.3 in.
Ultimate anchor capacity	Maximum sustained load	800 k
Long-term anchor capacity	Loss of load	2 k during 2-month test period

2.4 INSTRUMENTATION

2.4.1 Anchor Load Tests

The characteristics that were measured in the anchor load tests were:

(1) the load applied to the anchor tendon;

(2) the total elongation of the tendon; and

(3) the strain and movement within the anchorage zone (special design anchors only).

The load applied to the anchor was measured by an 800-k capacity, hollow core, cylindrical load cell inserted between the anchor head and the steel bearing plate in the reaction wall. Tendon elongation was measured by extension of the piston of the loading jack and by a reference rod extensometer. Strain and movement within the anchorage zones of the special design anchors were measured directly by embedment strain gages and telltales consisting of a rod extensometer system installed in the anchorage zone of the anchors. Details of the instrumentation and its performance are presented in Section 4.3. Table 2.1 summarizes the instrumentation used in the anchor load tests, the characteristics measured, and the data obtained.

2.4.2 Drilling Effects Test

The characteristics that were measured in the drilling effects test were:

(1) the quantity of cuttings flushed out of the test drill holes; and

(2) ground movement (surface settlement and horizontal and vertical displacement at depth) in the vicinity of the drill holes.

The quantity of cuttings flushed from the drill holes was determined by weight and volume measurements. Details of the measurement technique are presented in Section 5.7.1. Ground movement was measured directly by surface reference points, Borros points, three-dimensional deformation gages, and inclined inclinometers. Details of these instruments and their performance are presented in Section 5.6.1. Table 2.1 summarizes the instrumentation used in the drilling effects test, the characteristics measured, and the data obtained.

TEST	CHARACTERISTICS	MONITORING INSTRUMENTS	MEASURED FIELD DATA	DATA ACQUISITION METHOD	PROCESSED DATA	INTERPRETATION
ANCHOR LOAD TESTS	tendon load	load cell	load	electric readout	tendon load	ultimate anchor capacity; creep and relaxation characteristics
	tendon elongation	telltale and caliper	displacement	manual	net anchor movement	
	strain in anchorage length	strain gages	strain	electric readout	strain distribution within anchorage length	bond stress distribution; grout-rock bond strength
	differential movement in anchorage	telltale and dial gage	displacement	manual		
DRILLING EFFECTS TEST	ground movement	vertical and inclined inclinometers	displacement	electric readout	contour profile of ground movement	acceptable or excessive ground movement
		subsurface settlement points	displacement	optical survey and electric readout		
		surface reference points	displacement	optical survey		
	quantity of material flushed from drill holes	bin	volume	manual	volume of cuttings	acceptable or excessive ground loss
		scale	weight	manual	weight of cuttings	

ROCK ANCHOR TEST PROGRAM

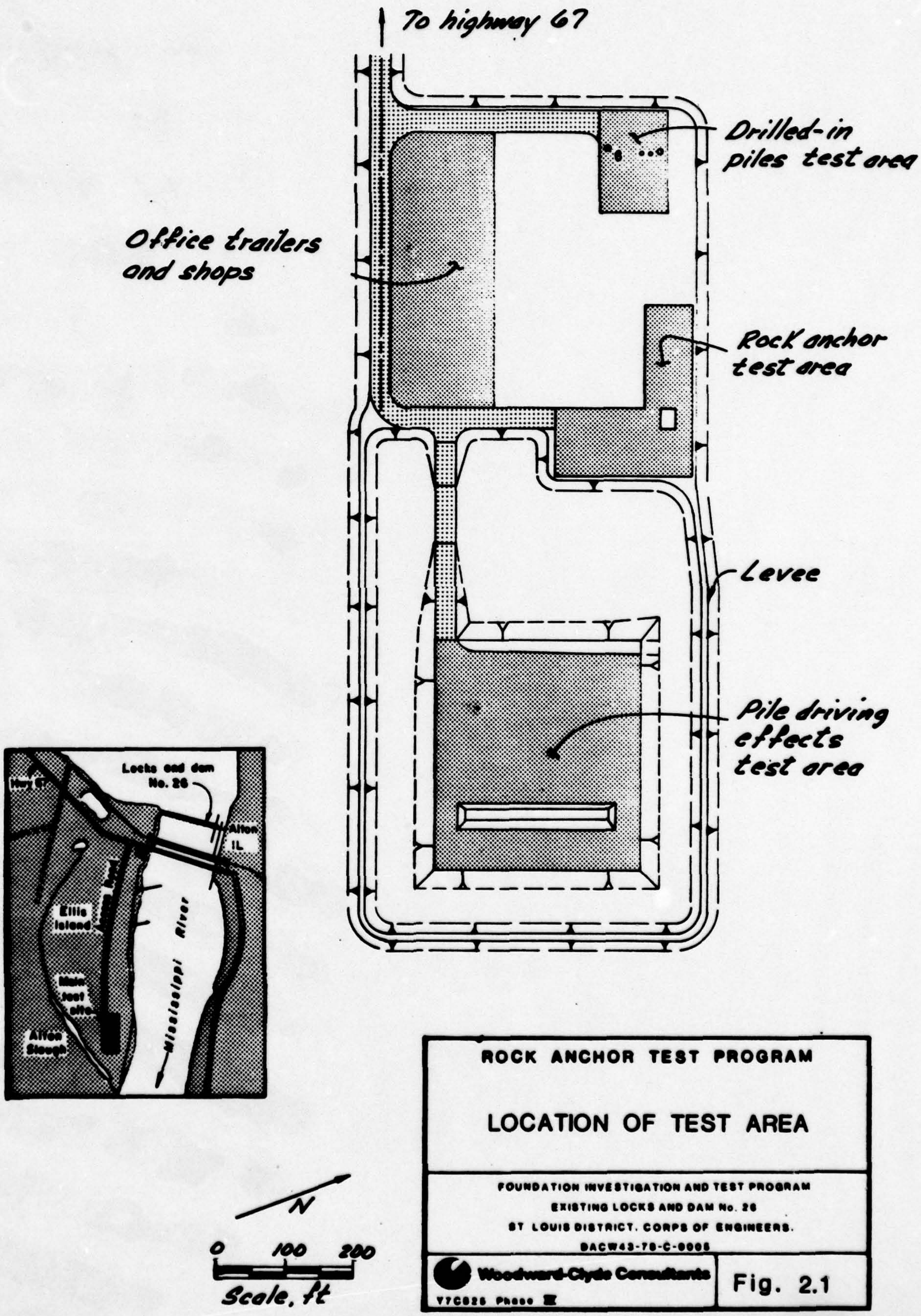
INSTRUMENTATION, CHARACTERISTICS, AND DATA

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0003

Woodward-Clyde Consultants

Table 2.1

77C826 Phase III



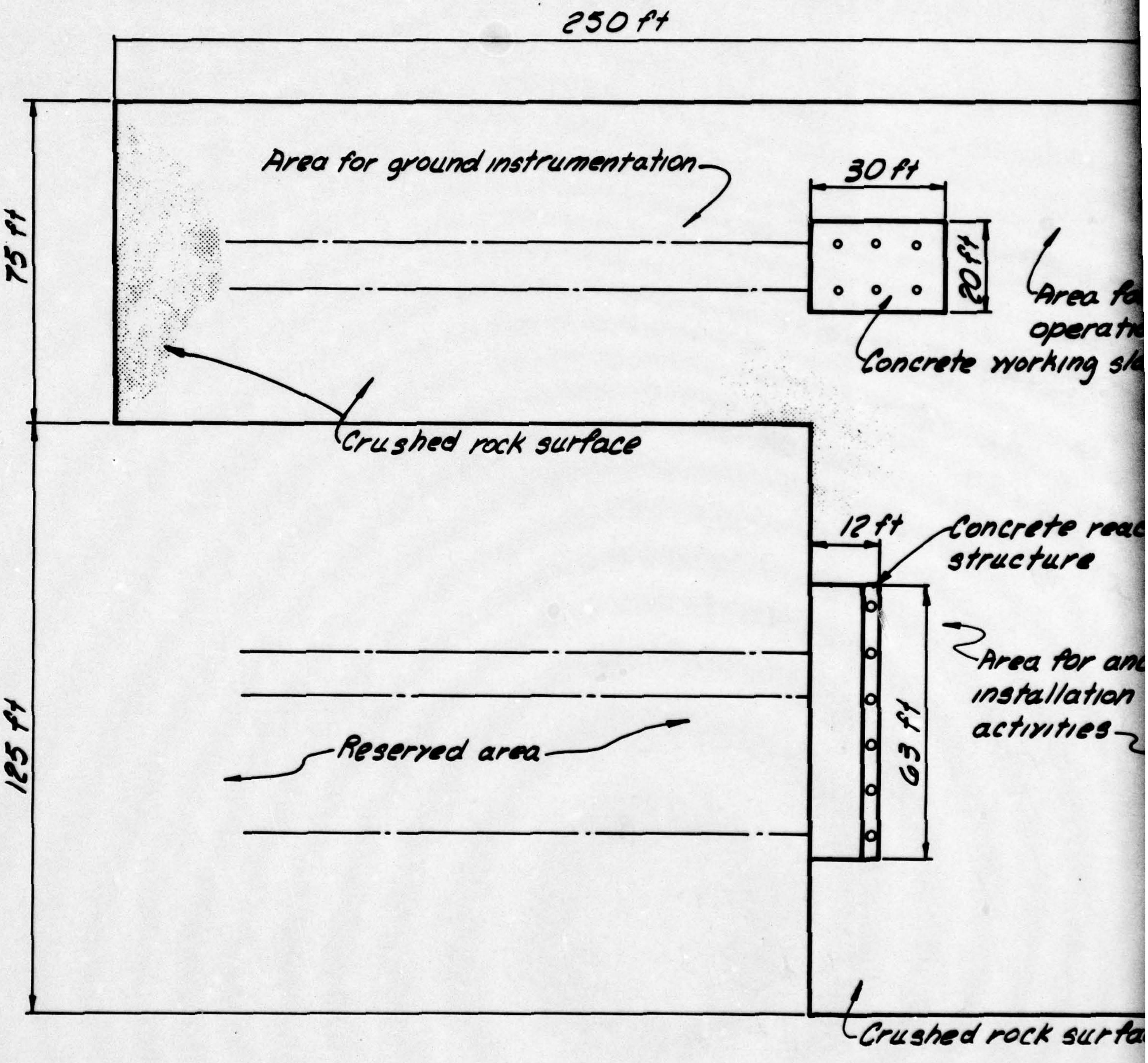
ROCK ANCHOR TEST PROGRAM

LOCATION OF TEST AREA

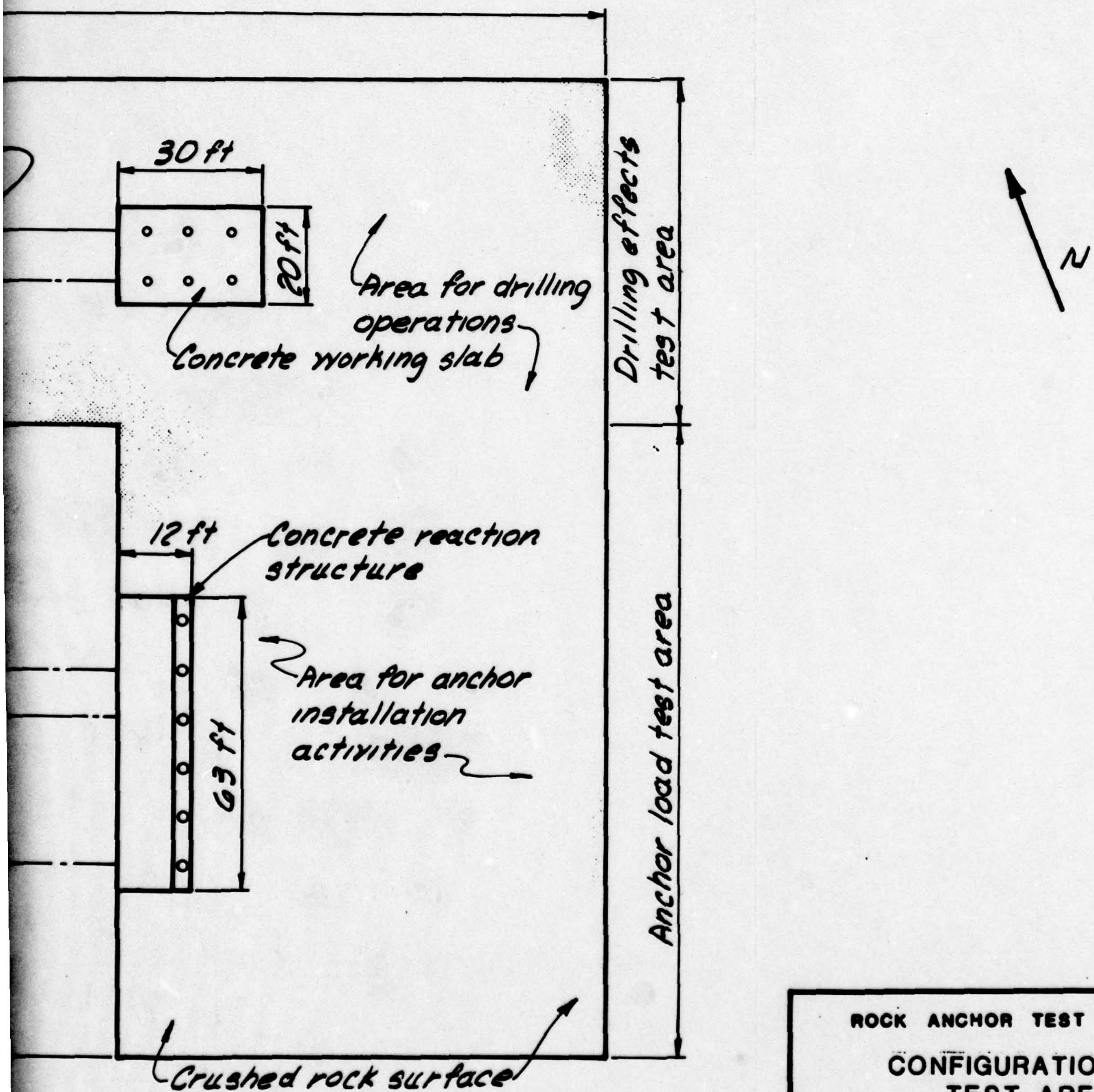
FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

Woodward-Clyde Consultants | Fig. 2.1

Y7C025 Phase III


250 ft
75 ft
Area for ground instrumentation
30 ft
20 ft
Concrete working sla
Area fo
operati
Crushed rock surface
125 ft
12 ft
Concrete reac
structure
Area for anc
installation
activities
63 ft
Reserved area
Crushed rock surfa

1



ROCK ANCHOR TEST PROGRAM

CONFIGURATION OF TEST AREA

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0008

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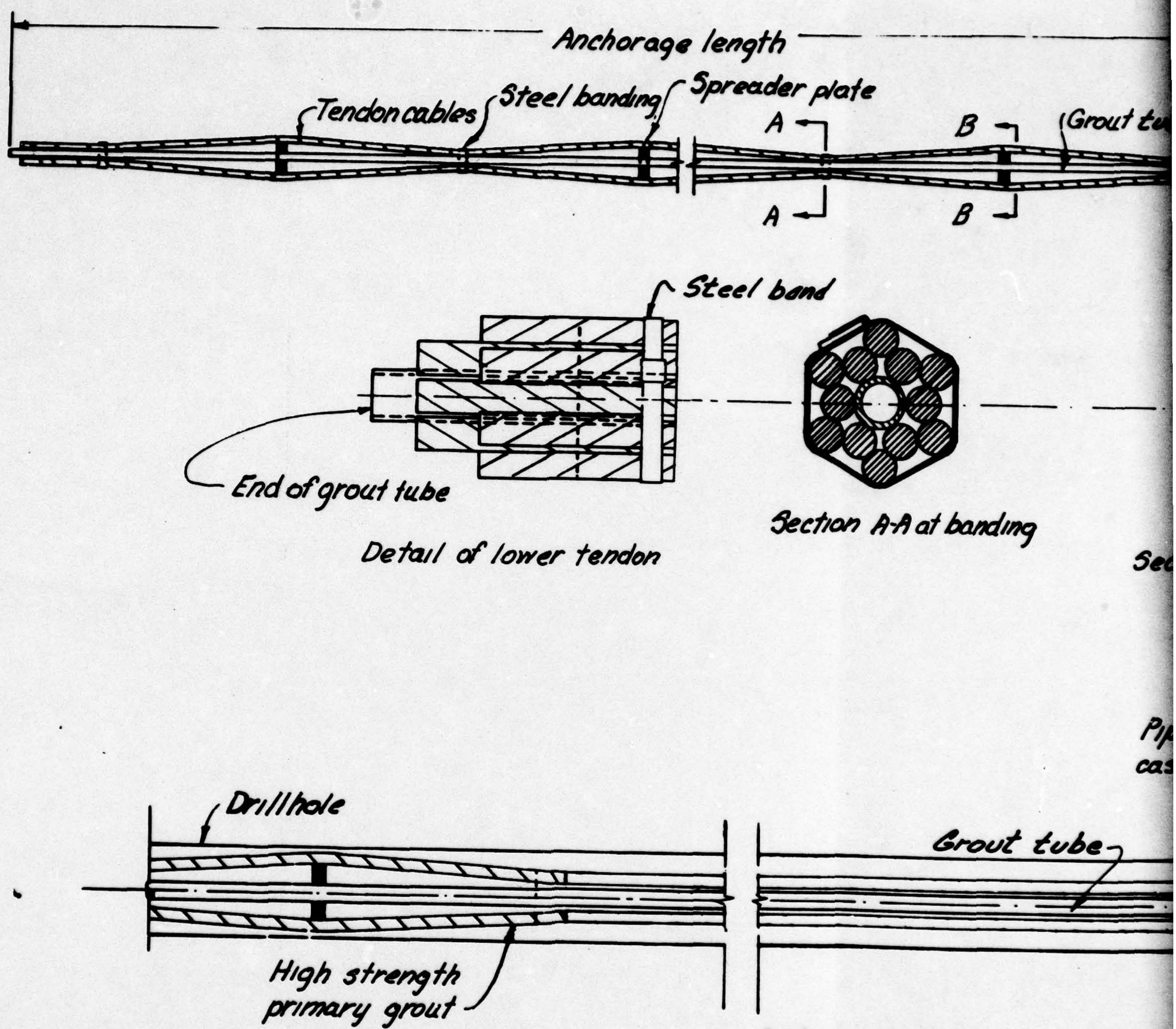
Y7C026 Phase III

Fig. 2.2

2



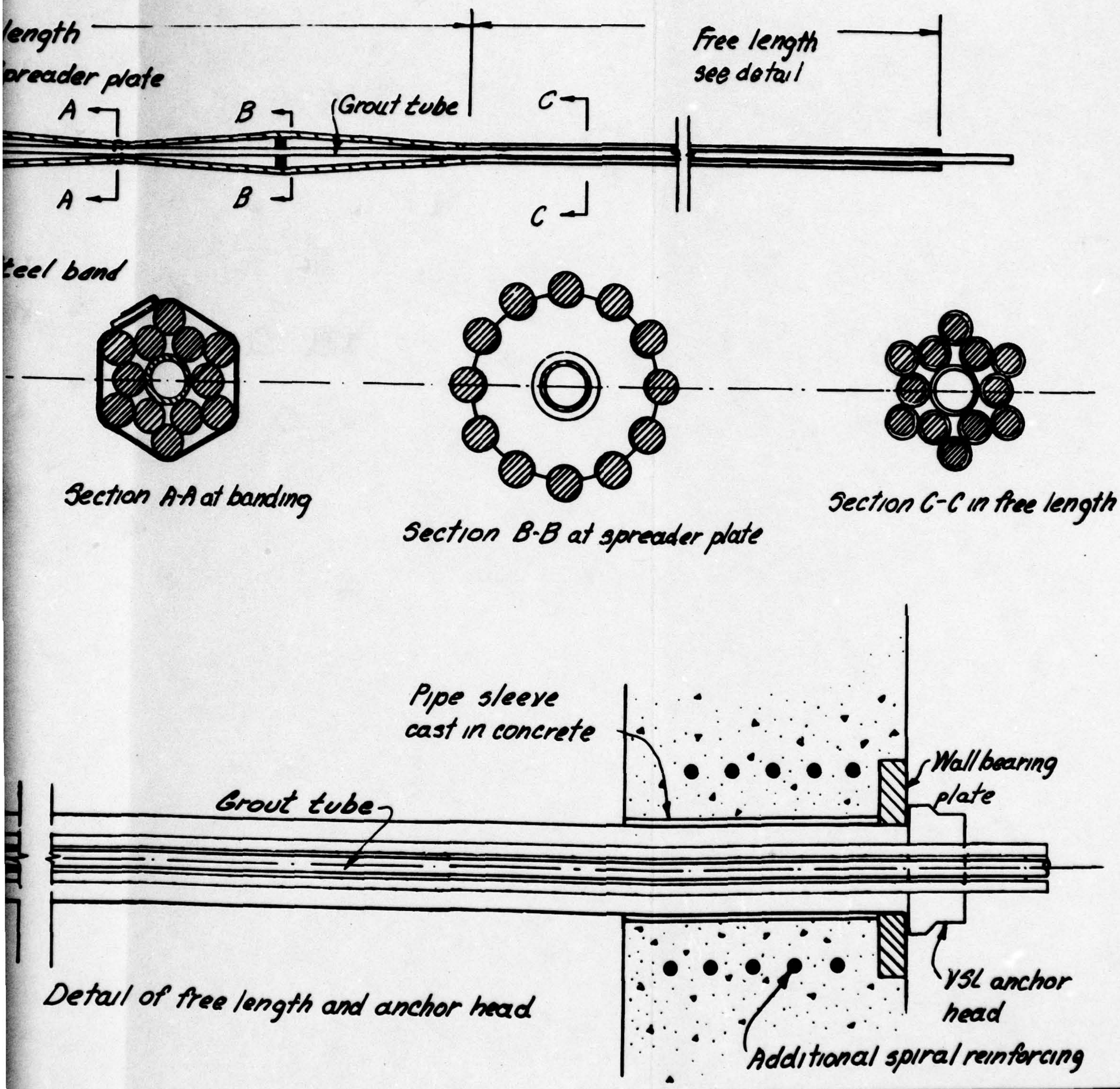
Detail of lower tendon

Section A-A at banding

Detail of free length and ancho

Not to scale

1



to scale

ROCK ANCHOR TEST PROGRAM

TYPICAL VSL ANCHOR SYSTEM

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

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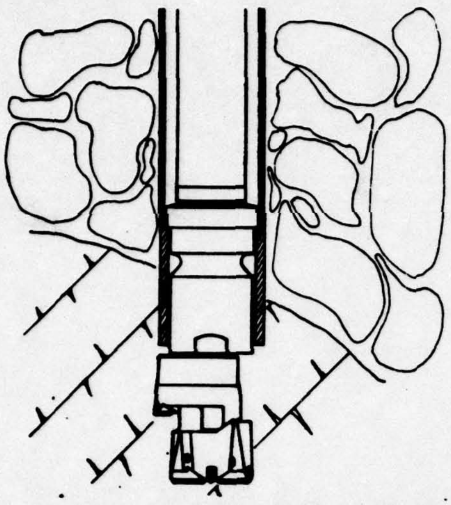
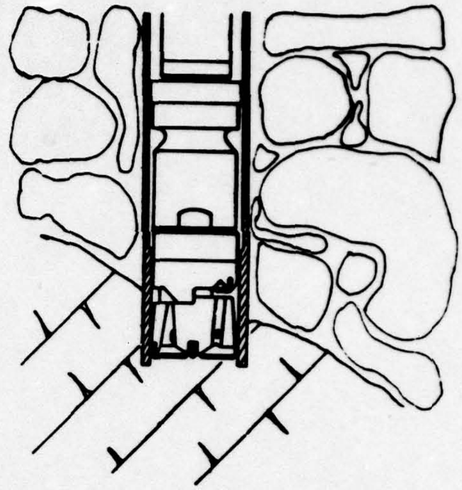
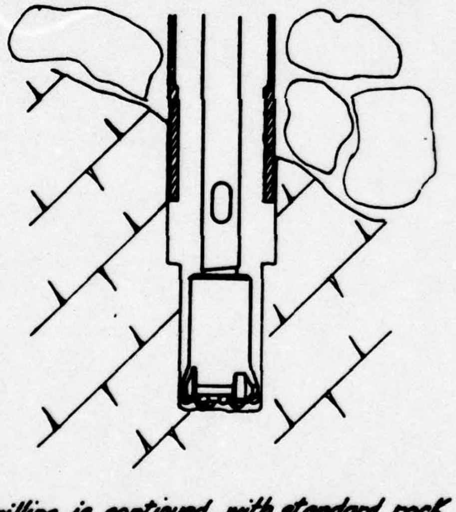
Fig. 2.3



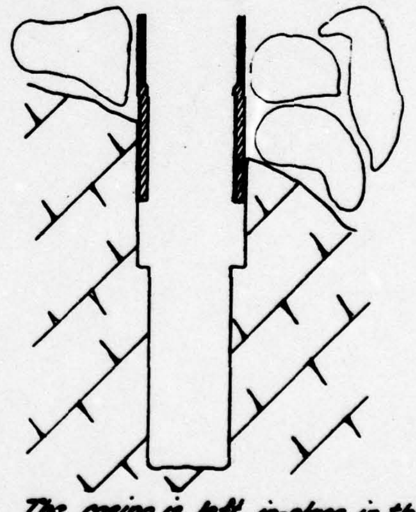
The pilot bit drills at the bottom of the hole. The reamer swings out automatically and reams the hole so that the casing can follow close behind the bit. The cuttings are transported upwards by means of air and water flushing inside the casing.



When drilling is completed, the drill bit is rotated in the other direction, so that the reamer is folded in. With the reamer in this position, the drilling equipment can be withdrawn through the casing



Drilling is continued with standard rock drilling equipment through the casing



The casing is left in-place in the hole to stabilize the walls until the anchor is installed.

ROCK ANCHOR TEST PROGRAM

PRINCIPLES OF ATLAS-COPCO ODEX SYSTEM

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0006

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77C825 Phase III

Fig. 2.4

PHASE IV REPORT

VOLUME V

RESULTS AND INTERPRETATION OF
ROCK ANCHOR TEST PROGRAM

SECTION 3

TEST AREA SUBSURFACE CONDITIONS

3 TEST AREA SUBSURFACE CONDITIONS

3.1 TEST AREA CONDITIONS

The area for the rock anchor test was selected at a location where early subsurface investigations indicated the absence of glacial till between the glacial and alluvial sand and gravel deposits and the underlying limestone bedrock. Investigations in other portions of Ellis Island encountered glacial till. Glacial till was not found under the existing Lock and Dam No. 26 during the subsurface investigations there. Therefore, to realistically model the drilling conditions under the locks and dam, this area, north of the pile driving effects test area (Volume III), was chosen for the rock anchor tests.

3.2 SUBSURFACE INVESTIGATIONS

Subsurface investigations were performed in conjunction with the drilled-in pile tests (Volume IV) to evaluate the initial soil and rock properties to be penetrated during anchor drilling. The investigations included borings, sampling, and in situ and laboratory testing. The locations of the borings are shown in Fig. 3.1. The program in the field included undisturbed Osterberg and Pitcher soil sampling in borings S-16 and D-1; standard penetration testing in borings S-16 and D-1; static cone penetration testing in boring DP-C1; nuclear density measurements in boring DP-D1; constant-head permeability testing in rock in boring S-16; rock coring in borings S-16, D-2, and RA-RC1. The laboratory tests included grain-size analyses on samples from borings S-16 and D-1, and unconfined compression tests on rock cores from borings S-16, D-2, and RA-RC1.

The subsurface stratigraphic profile was developed from the results of visual classification of split-spoon and undisturbed samples, laboratory grain-size analyses, and visual classification of cuttings obtained during drilling for installation of ground instrumentation. A subsurface profile along the axis of the drilling effects test drill holes is shown in Fig. 3.2. The results and location of standard penetration tests in boring S-16, and rock coring in borings S-16 and RA-RC1, are also shown in Fig. 3.2.

3.3 STRATIGRAPHY

3.3.1 General Geology

The test site is located within the Mississippi River flood plain near Alton, Illinois, at the southwestern edge of the central lowland physiographic province. In the vicinity of Locks and Dam No. 26, the flood plain surface is generally flat at el 410 to el 420.

In the vicinity of Locks and Dam No. 26, bedrock rises uniformly from el 270 on the Missouri side to el 330 on the Illinois side. The bedrock is overlain by soil deposits of glacial, alluvial, and colluvial origin. Five major soil strata and one bedrock unit have been identified; various stratigraphic units were inferred within some of the major soil strata. The inferred subsurface profile is shown in Fig. 3.2. The soil strata are, in decending order, flood plain deposits, recent alluvium, alluvial outwash (reworked alluvium), Wisconsinan outwash, and Illinoian ice contact deposits. Occasionally, glacial till pockets are intercalated between the ice contact deposits and bedrock surface. Till was not present at the test area location. The rock units underlying the soil consist of Mississippian limestone of the Meramecian (Valmeyer) Series. The upper formation is the St Genevieve. The following is a description of each soil stratum and rock unit.

3.3.2 Flood Plain Deposits

Flood plain deposits consist primarily of high to low plasticity clay, with varying amounts of silt, fine sand, and organic material. The clay is very soft to firm, and the sand very loose to loose. The source of the flood plain deposits is the active and abandoned channels, back swamps, and flood basin areas of the Mississippi River flood plain. The river materials, mainly silt and clay, are deposited in relatively quiescent waters. New material is carried in during river flood stages, or forms as colluvial deposits.

3.3.3 Recent Alluvium

The recent alluvium originated during aggrading and meandering of the Mississippi River across its flood plain during post-Wisconsinan (Holocene to Recent) time. The recent alluvium is a relatively uniform deposit because of common depositional environment and history, and because of the large scale of the Mississippi-Missouri fluvial system. It is uniform in such characteristics as depositional structures, and abundance and distribution of carbonaceous material. Various minor stratigraphic units were inferred within the recent alluvium; these units range from coarse silt to fine gravel, but are predominantly fine to medium sand. These sediments are characteristically clean, well-sorted (poorly graded) sand composed of at least 70 percent, and frequently more than 80 percent, quartz grains. They also contain abundant concentrations of carbonaceous material including wood, charcoal, and lignite, which range in size from coarse silt to large tree trunks.

3.3.4 Alluvial Outwash

The alluvial outwash consists of coarse- to fine-grained, pooly graded sand, with some silty sand and gravel zones. The alluvial outwash is considered to be an intergrading of recent alluvium and the underlying Wisconsinan outwash. The deposits may have formed contemporaneously with Wisconsin glaciation. The major portion of this alluvial outwash deposit, however, is believed to have formed during the in situ reworking of glacial outwash in post-Wisconsin to Recent time. Variations in stream flow, channel form and width, and obstructions led to renewed scouring of previously deposited sediment (Wisconsinan outwash) and redeposition elsewhere.

3.3.5 Wisconsinan Outwash

The Wisconsinan outwash consists of coarse- to medium-grained, poorly graded sand, silty sand, and gravel. The Wisconsinan outwash was deposited in the Mississippi Valley during the Wisconsin glacial advance into areas west, north and east of the St Louis area. Major streams that carried outwash material included the Illinois and Missouri Rivers, as well as the Mississippi River.

3.3.6 Illinoian Ice Contact Deposits

The Illinoian ice contact deposits consist generally of fine-to coarse-grained, poorly graded sand, with numerous boulder, cobble, gravel, and occasional silty sand zones. The ice contact deposits are generally dense. The ice contact deposits formed immediately adjacent to the glacial ice front, resulting in an extremely variable particle size. The deposits are discontinuous and may not be undisturbed glacial deposits. Large particles from upstream glacial materials, along with alluvium, may have been placed in some areas as channel lag deposits. Till-like material (flow till) resulting from superglacial mud or debris flows are often found in ice contact deposits.

3.3.7 Limestone: St Genevieve Formation

The St Genevieve formation is a light-colored, sandy, oolitic, cross-bedded calcarenite. The upper beds are often separted by thin, shaley limestones and thin shale partings. Exposed on the bluffs north of Alton, portions of the St Genevieve massive cross-bedded oolite are abruptly replaced by thin-bedded shaley or slabby oolitic limestone. Small faults and some solution activity have been noted in this limestone on the bluffs.

3.4 INITIAL SOIL AND ROCK PROPERTIES

3.4.1 Soil Properties

The properties of the various soil units assessed from in situ and laboratory testing included grain-size distribution, static cone penetration resistance, in situ density and relative density. The range of grain-size distribution of the various stratigraphic units identified in Fig. 3.2 are presented in Fig. 3.3. The soil consists primary of a poorly graded fine to medium sand with a trace of silt and trace of gravel grading into a fine to coarse sand. There are local concentrations of gravel, cobbles and boulders.

A static cone penetration resistance profile from boring DP-C1 is presented in Fig. 3.4. Empirical correlations have been developed between cone resistance, relative density, elastic deformation modulus, and angle of internal friction. The cone enables the development of a continuous profile of these properties with depth. A relative density profile derived from cone results is shown in Fig. 3.5; the derivation was done using an empirical correlation developed by Schmertmann (1976). This correlation was developed for normally consolidated, fine to medium, poorly-graded sand, using an electrical cone similar to the cone used in boring DP-C1. The deposits are medium-dense to dense, with an average relative density of 70 percent. The wide variability is due to stratigraphic differences and gravel zones inherent in an alluvial-glacial depositional profile.

In situ total unit weights were based on borehole nuclear density measurements. Natural water content of samples obtained from boring DP-D1 was used to calculate dry unit weights. These results are shown on Fig. 3.6. Total unit weight was also measured directly on undisturbed borehole samples; dry unit weights calculated for these samples are also shown on this figure. From these data, an average dry unit weight of 105 lb/ft^3 was used for ground-loss computations in the drilling effects test.

3.4.2 Rock Properties

Properties of the upper bedrock into which rock sockets were to be drilled for the test anchors were assessed by borehole permeability tests and laboratory unconfined compression tests on rock cores. These test results are summarized in Table 3.1. Borehole constant-head permeability tests in boring S-16 indicated a rather high permeability in the limestone bedrock. Values ranged from 15 to 162 Lugeon Units (LU) with an average of 80 LU. This high permeability is probably due to fractures in the rock.

Unconfined compression tests indicate an unconfined compressive strength of the rock ranging from 15.3 to 22.8 k/in^2 with an average of 18.7 k/in^2 (approximately 1350 t/ft^2). The strain to failure averages 0.3 percent; the secant modulus at 50 percent of the peak stress is 6.5x10^6 lb/in^2.

Values of RQD* measured from rock cored in boring RA-RC1 and S-16 indicate 68 percent above el 290 (fair rock quality), 92 percent from el 290 to el 280 (excellent rock quality), 68 percent from el 280 to el 276 (fair rock quality), and 100 percent below el 276 (excellent rock quality).

---

* *Rock Quality Designation (RQD) is a modified measure of rock recovery from coring. RQD is the ratio of cumulative length of core pieces longer than 4 in. to total length of core run (Deere 1963)*

Boring	Elevation	RQD %	Unconfined Compression Tests: q_u (1) k/in^2	ϵ_f (2) %	E_{s50} (3) $x10^6$ lb/in^2	Permeability (4) LU
D-2	289.9	98.0	17.7	0.32	5.4	
D-2	285.9	98.0	21.7	0.41	6.1	
S-16	288.5	82.7				15.4
S-16	283.2	94.1	17.3	0.33	5.4	58.6
S-16	278.2	68.3				162.5
S-16	273.2	100.0				84.4
S-16	271.2	100.0	15.3	0.21	8.0	
RA-RC1	287.5	68.2	22.8	0.31	8.9	
RA-RC1	282.3	98.3	17.2	0.36	5.2	

Notes:

(1) q_u: *unconfined compressive strength*

(2) ϵ_f: *strain at failure*

(3) E_{s50} : *elastic deformation modulus at 50 percent of failure*

(4) *1 Lugeon Unit in rock is approximately equivalent to a coefficient of permeability of 1×10^{-5} cm/s in soil*

ROCK ANCHOR TEST PROGRAM

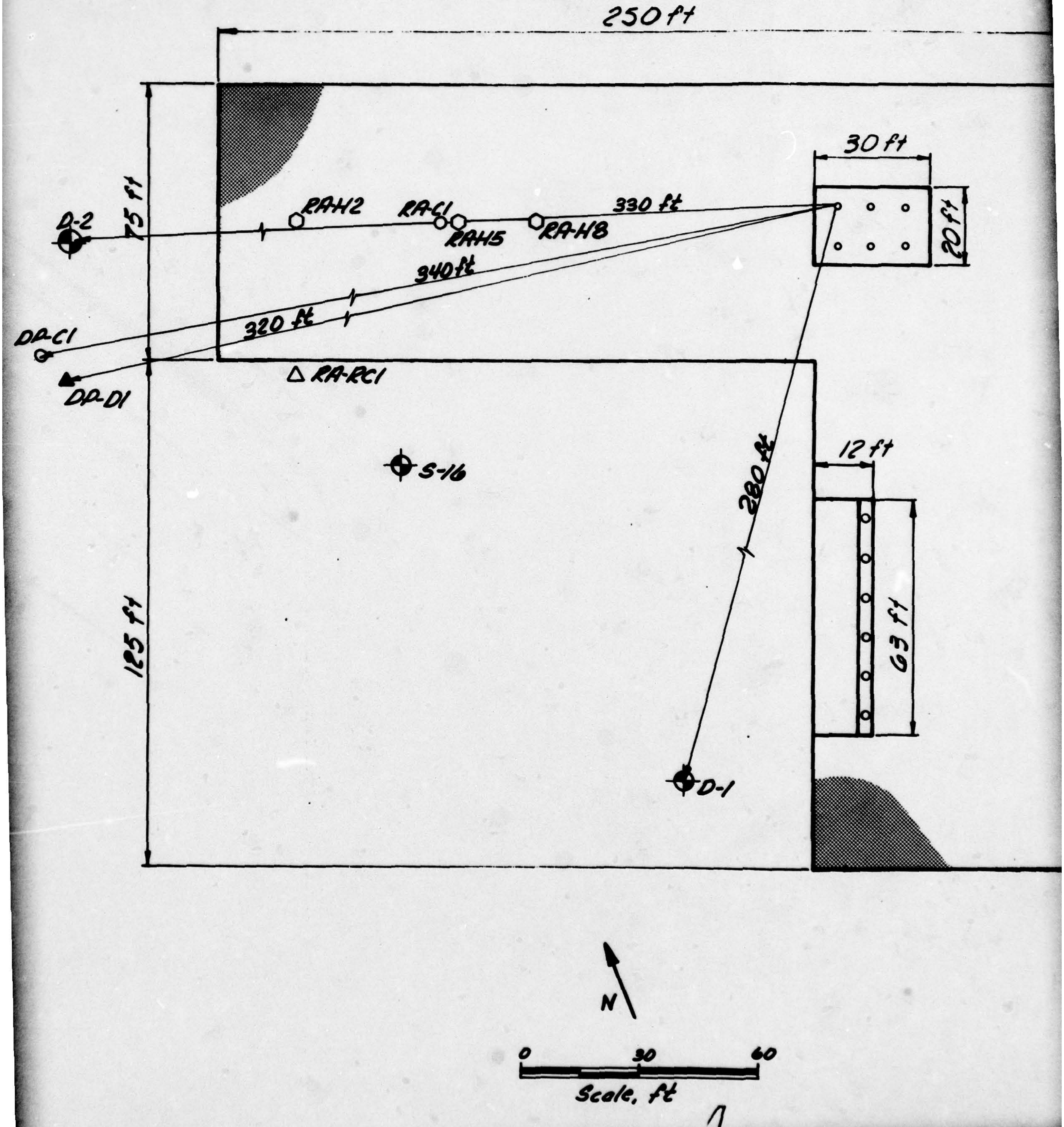
SUMMARY OF ROCK PROPERTIES

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT. CORPS OF ENGINEERS.
DACW43-78-C-0008

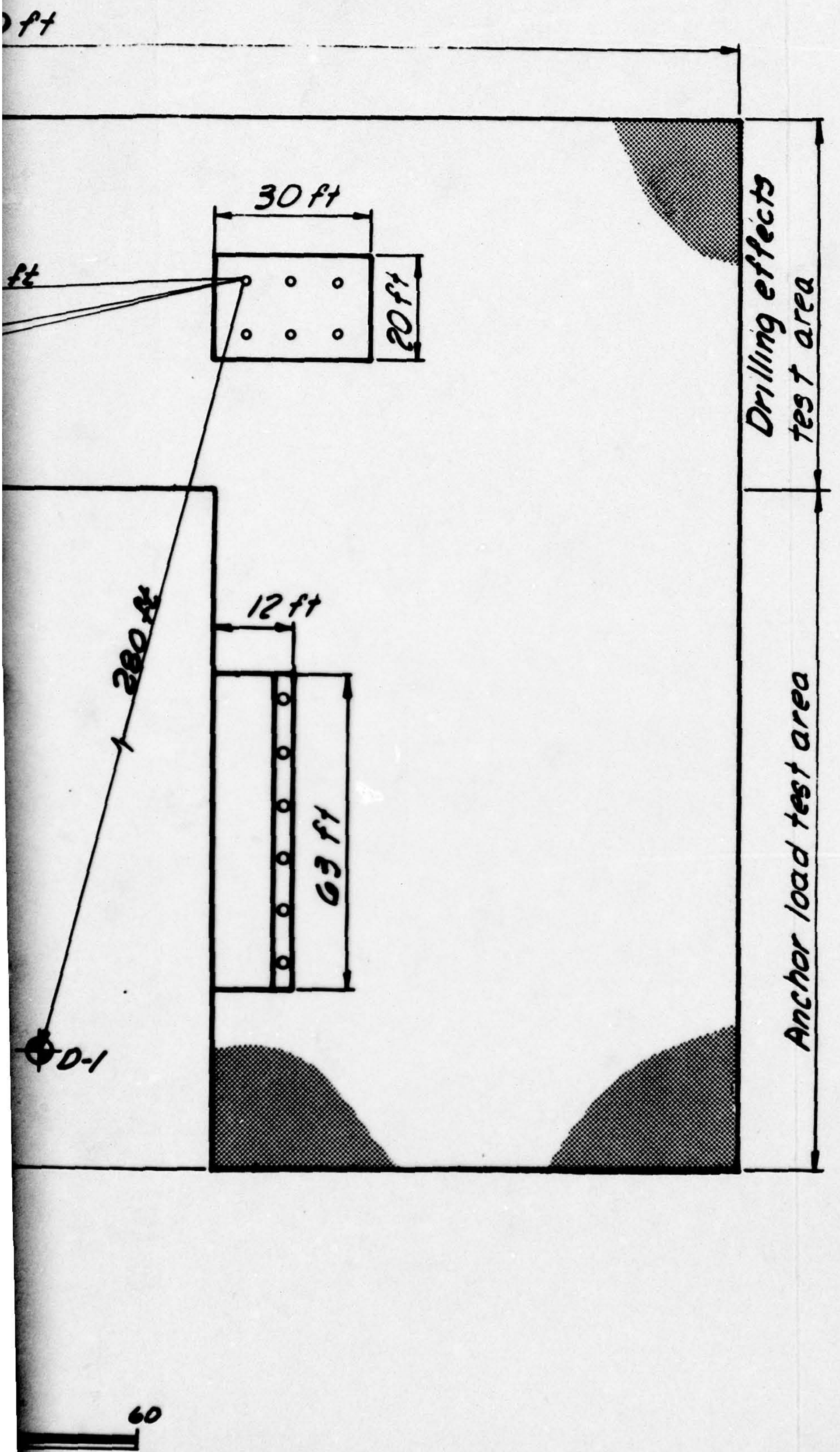
Woodward-Clyde Consultants

Y7C828 Phase III

Table 3.1



250 ft
30 ft
20 ft
75 ft
D-2
RAH2
RA-C1
RAH5
RA-H8
330 ft
340 ft
320 ft
DP-C1
DP-D1
RA-RC1
S-16
12 ft
280 ft
63 ft
125 ft
D-1
N
0
30
60
Scale, ft



Legend

- ▲ Nuclear density boring
- △ Rock core boring
- ○ Instrumentation boring
- ⊕ Boring with undisturbed and disturbed sampling

ROCK ANCHOR TEST PROGRAM

LOCATION PLAN OF SUBSURFACE INVESTIGATION

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0006

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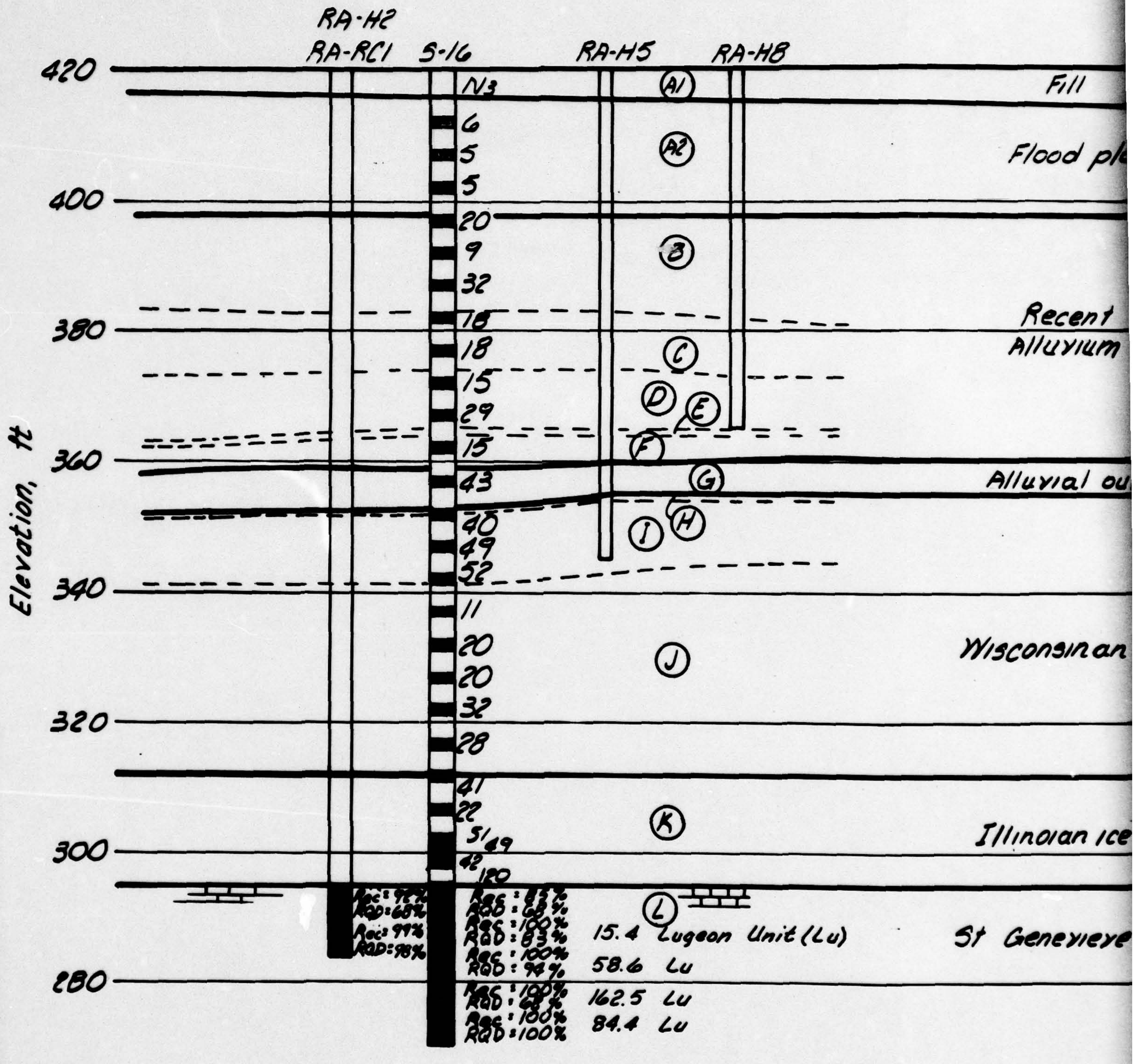
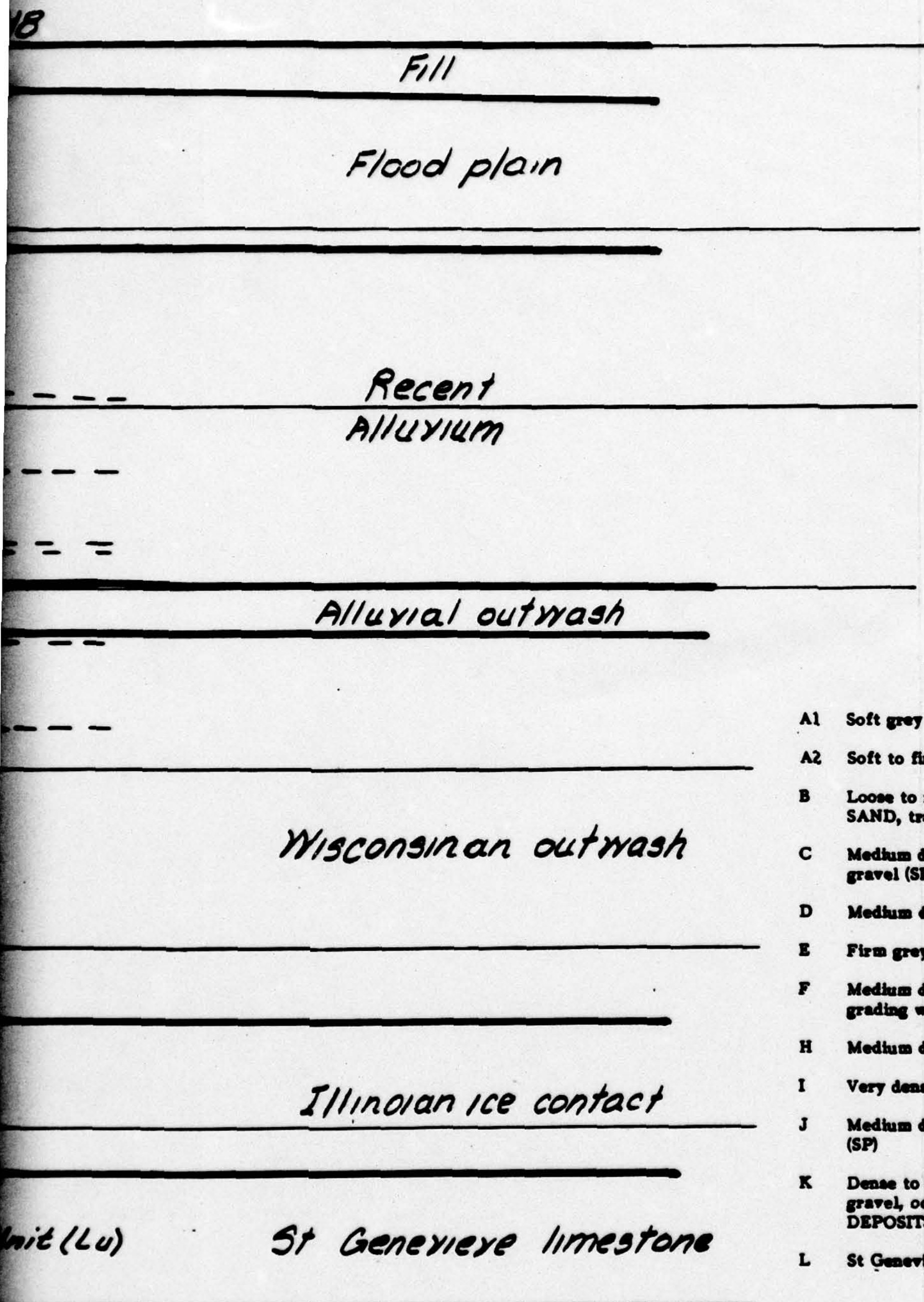
Y7C828 Phase III

Fig 3.1



RA-H2
RA-RC1
S-16
RA-H5
RA-H8
Elevation, ft
420
400
380
360
340
320
300
280
N3
6
5
5
20
9
32
18
18
15
29
15
43
40
40
49
52
11
20
20
32
28
41
22
31
49
42
120
A1
A2
B
C
D
E
F
G
H
I
J
K
L
Fill
Flood pl
Recent
Alluvium
Alluvial ou
Wisconsinan
Illinoian ice
St Genevieve
Rec: 93%
RQD: 68%
Rec: 99%
RQD: 98%
Rec: 83%
RQD: 68%
Rec: 100%
RQD: 83%
Rec: 100%
RQD: 94%
Rec: 100%
RQD: 68%
Rec: 100%
RQD: 100%
15.4 Lugeon Unit (Lu)
58.6 Lu
162.5 Lu
84.4 Lu

1



Subsurface Stratigraphy

A1 Soft grey clayey SILT, trace fine sand (ML-CL), FILL

A2 Soft to firm grey clayey SILT, trace fine sand (ML-CL), FLOODPLAIN

B Loose to medium dense brown to grey fine, fine to medium or fine to coarse SAND, trace silt, trace fine gravel, (SP)

C Medium dense to dense grey fine to medium SAND, trace silt, frace fine gravel (SP)

D Medium dense grey fine to coarse SAND, trace silt, trace fine gravel (SP)

E Firm grey silty CLAY, trace fine sand (CL)

RECENT ALLUVIUM

F Medium dense to dense grey fine to coarse SAND, trace silt, trace fine gravel, grading w/occ shell and lignite fragments (SP), ALLUVIAL OUTWASH

H Medium dense grey-black silty fine SAND (SM)

I Very dense grey fine SAND, trace silt (SP)

J Medium dense to dense grey fine to coarse SAND, trace silt, trace fine gravel (SP)

WISCONSINAN OUTWASH

K Dense to very dense grey fine to coarse SAND, trace silt, trace fine to coarse gravel, occ grading with cobbles and boulders (SP) ILLINOIAN ICE CONTACT DEPOSITS

L St Genevieve Limestone (LS)

ROCK ANCHOR TEST PROGRAM

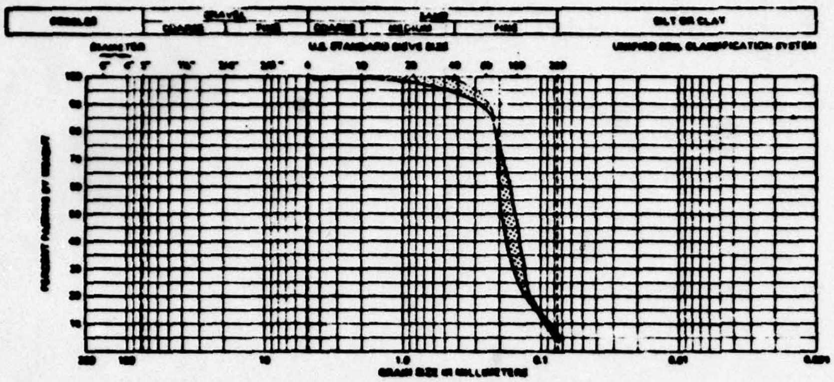
SUBSURFACE PROFILE DRILLING EFFECTS TEST AREA

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

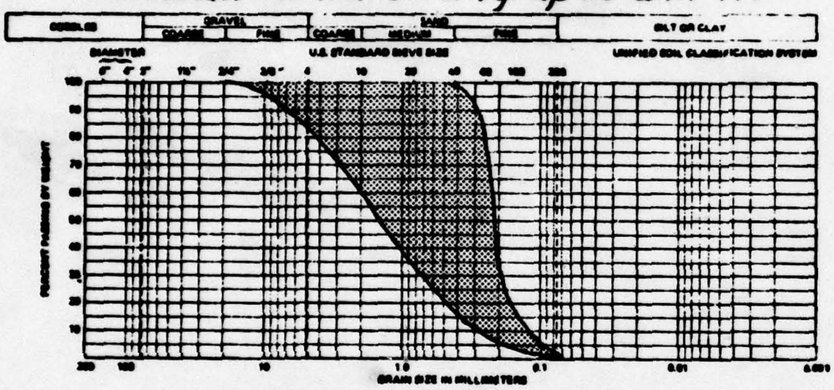
Woodward-Clyde Consultants | Fig 3.2

Y7C826 Phase III

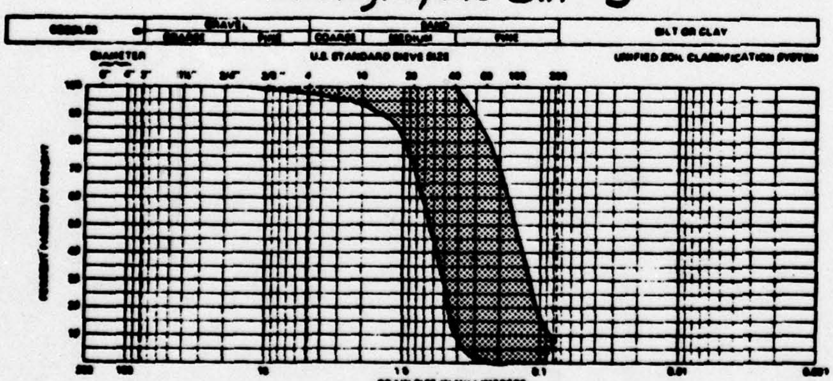
2



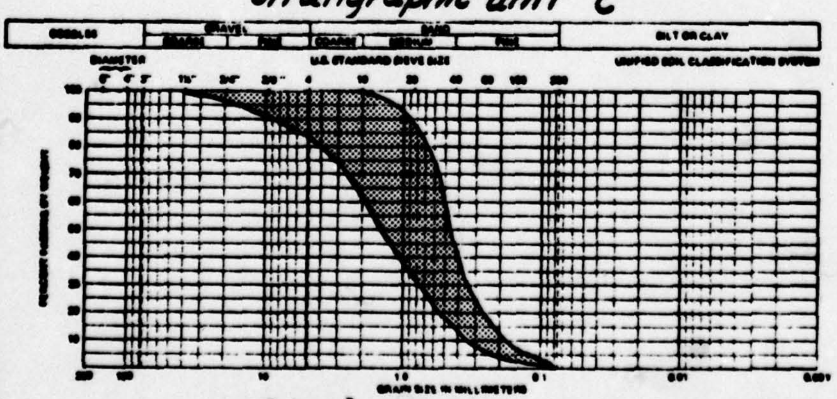
Sandseam within Stratigraphic unit A2



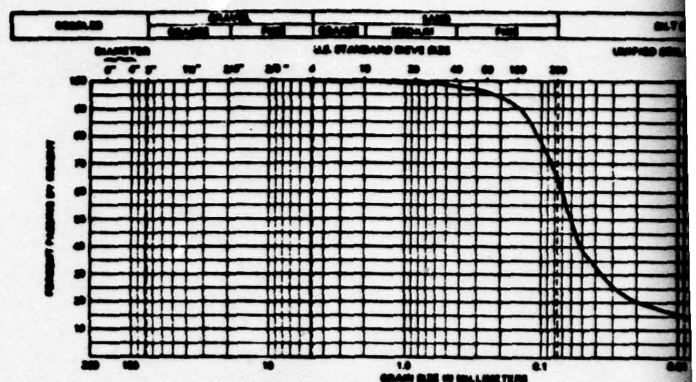
Stratigraphic unit B



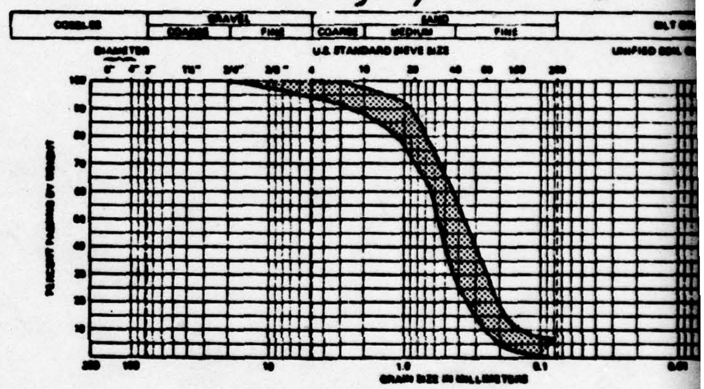
Stratigraphic unit C



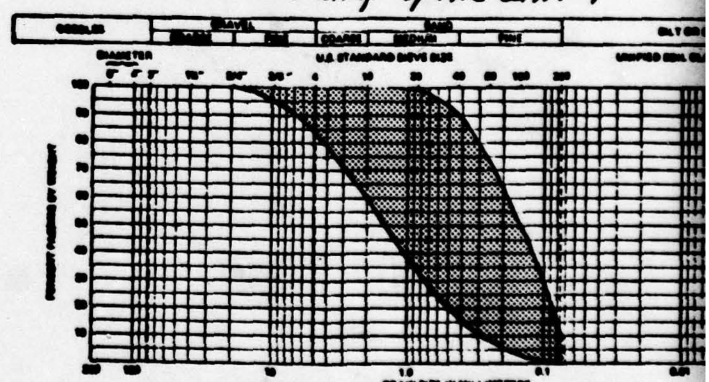
Stratigraphic unit D



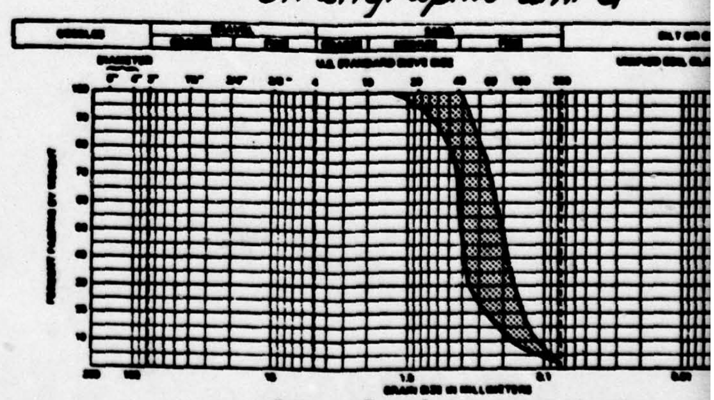
Stratigraphic unit E



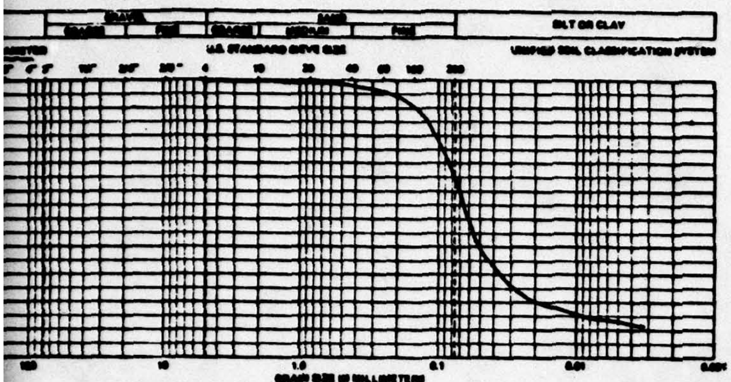
Stratigraphic unit F



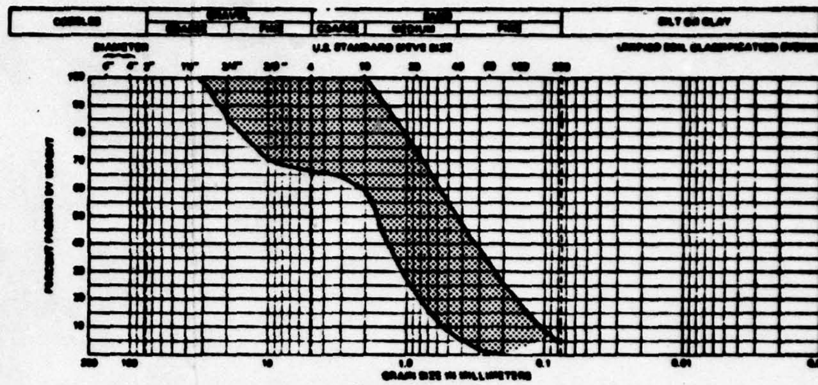
Stratigraphic unit G



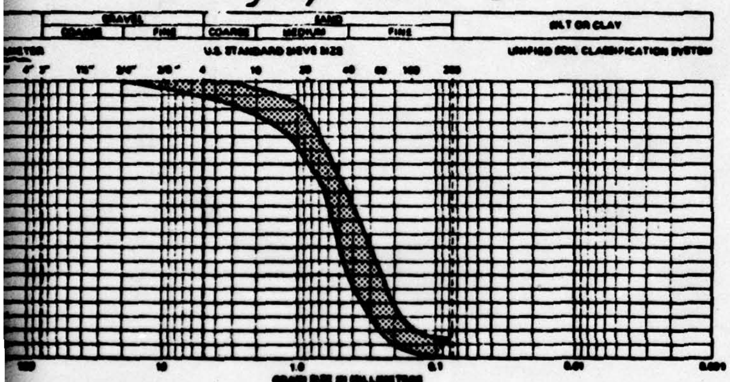
Stratigraphic unit I



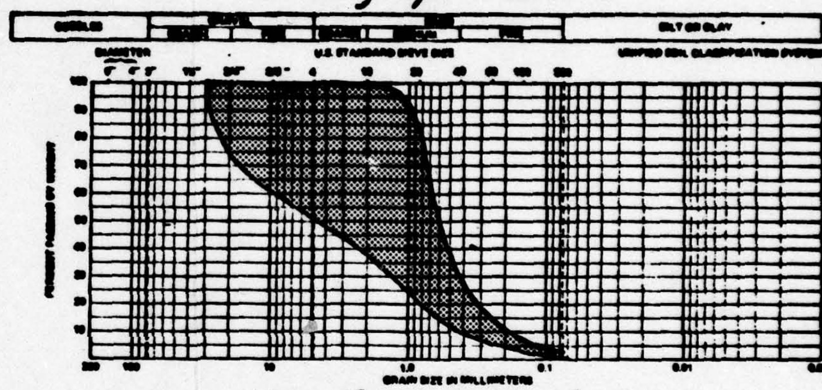
Stratigraphic unit E



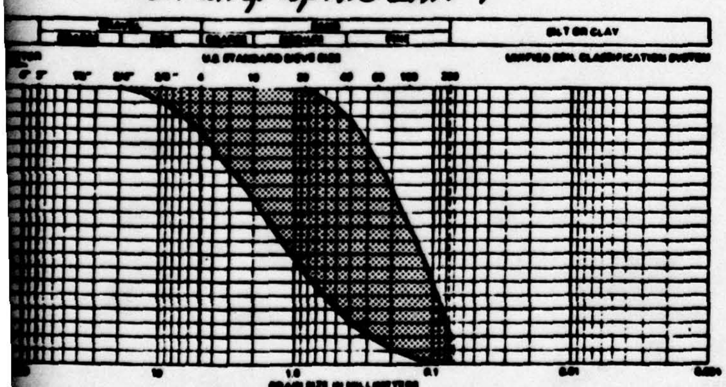
Stratigraphic unit J



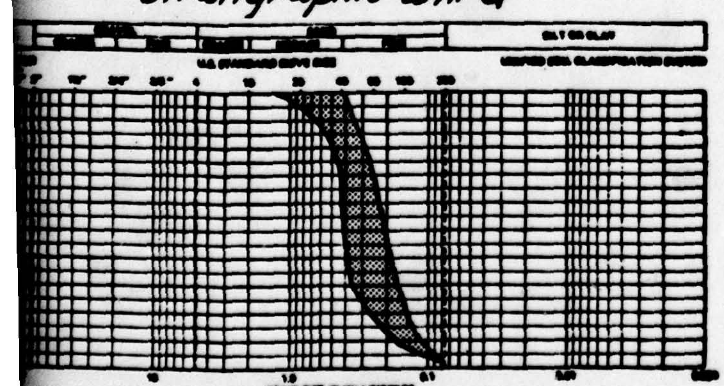
Stratigraphic unit F



Stratigraphic unit K



Stratigraphic unit G



Stratigraphic unit I

Note:
For location and description of stratigraphic units, see fig 3.2

ROCK ANCHOR TEST PROGRAM

GRAIN SIZE DISTRIBUTION OF VARIOUS STRATIGRAPHIC UNITS

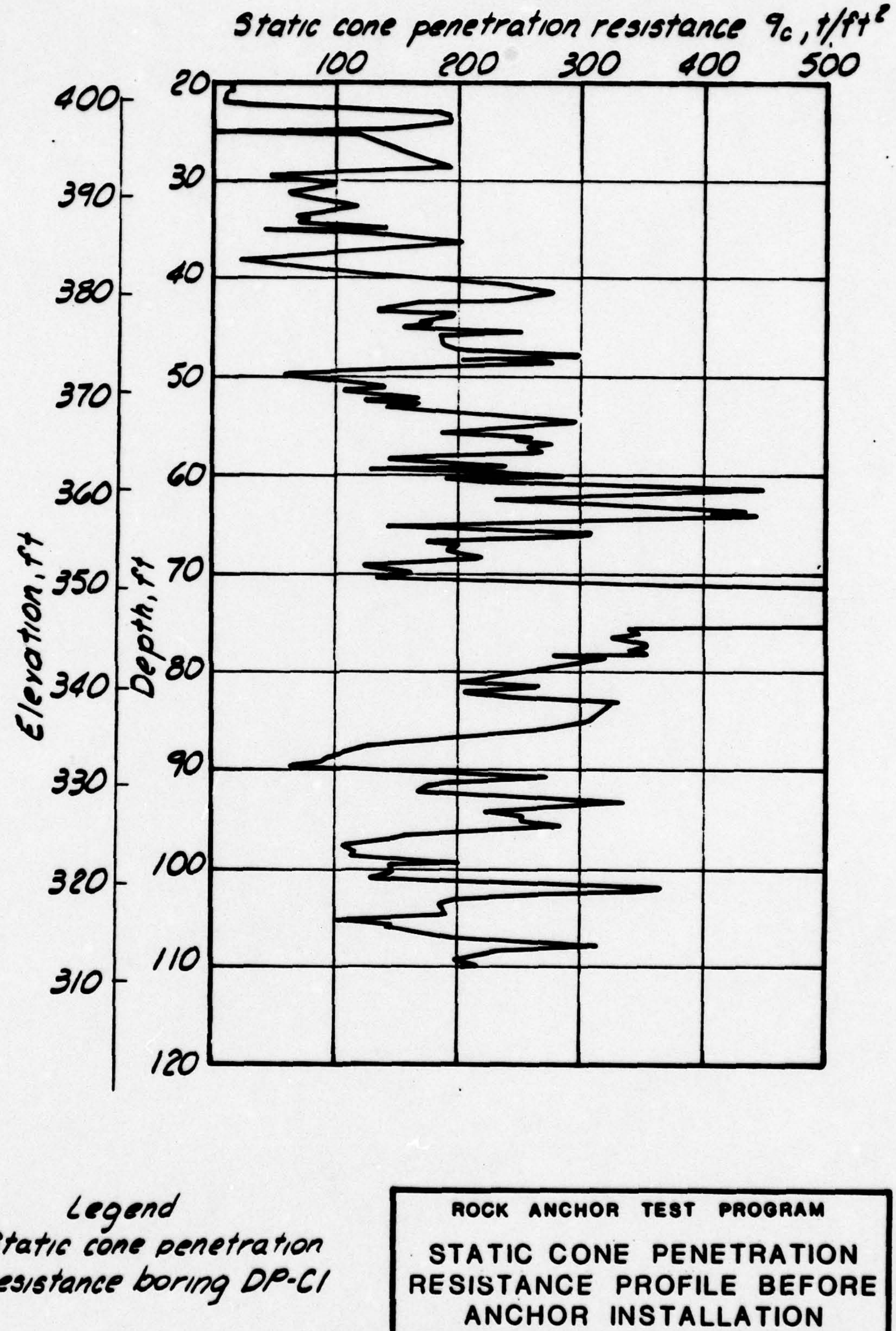
FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0068

Woodward-Clyde Consultants

Y7C825 Phase IX

Fig 3.3

1 2



Legend
— Static cone penetration resistance boring DP-C1

ROCK ANCHOR TEST PROGRAM

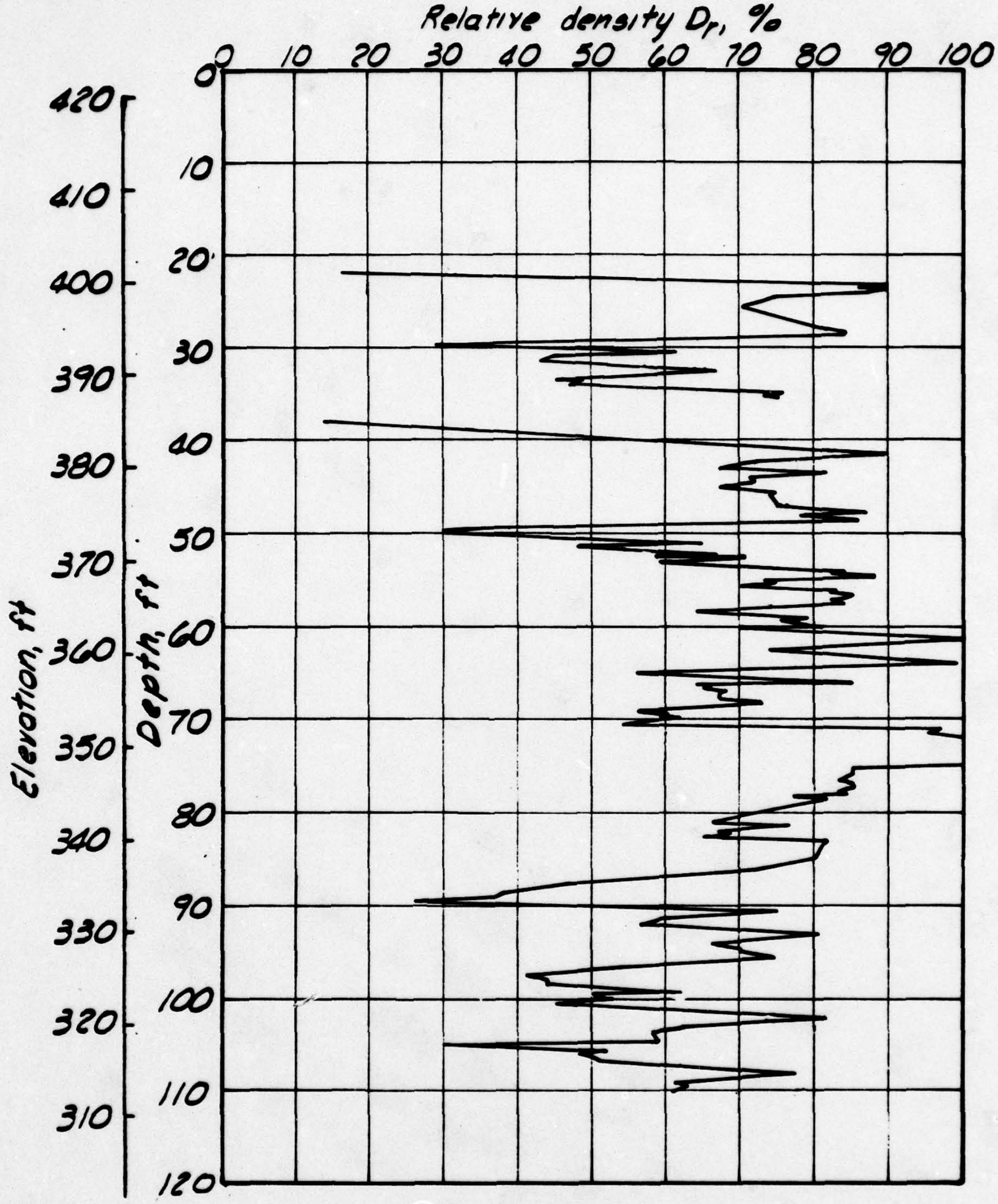
STATIC CONE PENETRATION RESISTANCE PROFILE BEFORE ANCHOR INSTALLATION

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0006

Woodward-Clyde Consultants

V7C026 Phase IX

Fig 3.4



Legend

— Dr derived from static cone penetration test in boring DP-C1, using Schmertmann (1976) Correlation

ROCK ANCHOR TEST PROGRAM

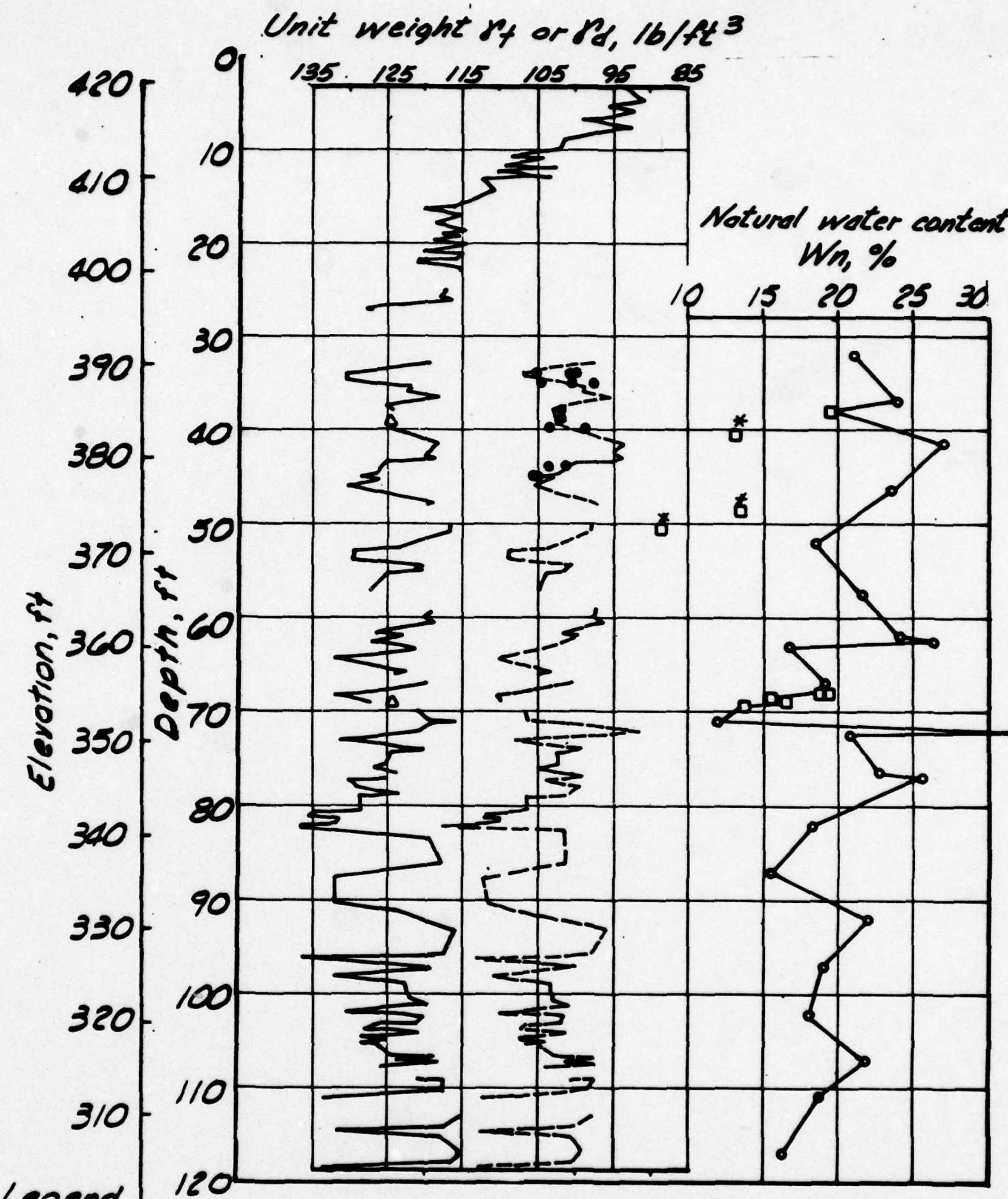
RELATIVE DENSITY PROFILE BEFORE ANCHOR INSTALLATION

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0008

Woodward-Clyde Consultants

Fig. 3.5

Y7C628 Phase III



Legend

Unit weight

— Total-(nuclear density probe)

△ Total-(Hvorslev sample)

--- Dry-(nuclear density probe)

• Dry-(Osterberg sample)

Natural water content

○ Split spoon sample, (disturbed)

□ Hvorslev sample

* Drainage allowed in tube

ROCK ANCHOR TEST PROGRAM

PROFILE AT BORING DP-D1 OF NATURAL DRY UNIT WEIGHT AND WATER CONTENT

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0068

Woodward-Clyde Consultants

77C628 Phase II

Fig. 3.6

PHASE IV REPORT

VOLUME V

RESULTS AND INTERPRETATION OF ROCK ANCHOR TEST PROGRAM

SECTION 4

ANCHOR LOAD TESTS

4 ANCHOR LOAD TESTS

4.1 CONCEPT OF ANCHOR LOAD TESTS

The objective of the anchor load tests was to assess the capacity and performance of 400-k design rock anchors. Three anchors were installed at an inclination of 45 degrees through a pile-supported concrete reaction structure and were grouted into rock (Fig. 4.1). Two types of anchors were tested: a prototype anchor, and special design anchors, each type having a different purpose.

4.1.1 Prototype Anchor

One prototype anchor, RP-1, was fabricated in accordance with the latest VSL scheme to simulate a production anchor which may be used in actual installations. The prototype anchor was installed in a drill hole through the reaction wall and grouted into rock. The anchor was load tested to assess its capacity and long-term performance. The anchor capacity was evaluated on the basis of its stability under proof load and friction losses. Difficulties with an intermediate lock-off scheme prevented lock-off losses from being evaluated on anchor RP-1. Long-term performance was evaluated on the basis of stability of the anchor load after final lock-off.

The anchor was tested by stressing it in load increments up to a proof load of 480 k (20 percent over design load). The proof load was held for an extended period of time to monitor the stability of the load at that level. During stressing, the load on the anchor and elongation of the tendon were measured to assess the load expended due to friction in the system. The anchor was locked-off at the design load of 400 k and the load on the anchor was monitored for two and one-half months.

4.1.2 Special Design Anchors

Two design anchors, RD-1 and RD-2, were specially fabricated to accomodate instrumentation and minimize friction losses to allow detailed load transfer measurements to be made during load testing. Special fabrication entailed installing strain measuring devices within the anchorage zone, and providing extra greasing and sheathing around the tendon bundle in the free length. The anchors were load tested to assess their capacity. The capacity of the anchors was evaluated on the basis of the stress distribution along the anchorage zone and the stability of the anchor load at high load levels. Measurements were also made to assess load losses due to engagement of the lock-off mechanism.

The anchors were tested by stressing them in load increments up to the maximum test load of 800 k (twice the design load). During loading, the load on the anchors, elongation of the tendons, and strain in the anchor zone were measured. From these measurements, an assessment was made of the ultimate capacity of the anchors and the strength of the grout-rock bond.

4.2 SEQUENCE OF ACTIVITIES

Fabrication of the test anchors was started in early November 1978 on two prototype anchors, RP-1 and RP-2. Fabrication work continued on the special design anchors, RD-1 and RD-2, on an intermittent basis due to modifications to some anchor instrumentation and difficulties with drilling. Anchor fabrication was completed by mid-December.

Drilling for test anchor RP-1 was started on 27 November 1978 and the anchor installed on 1 December. Drilling for test anchor RD-1 was started on 5 December, but was interrupted several times due to drilling difficulties. The hole was completed and the anchor installed on 17 January 1979. Drilling for test anchor RD-2 was started immediately after RD-1, and the anchor was installed on 26 January.

At this point, it was decided to delete prototype anchor RP-2 from the testing program. Drilling difficulties had delayed the rock anchor test program schedule substantially, and the drilling effort had to be concentrated on the drilling effects test. Load testing of prototype anchor RP-1 had been completed by this time, and suffificient data gathered such that performance of prototype anchors could adequately be judged based on the results of the one load test.

The load test on anchor RP-1 was performed on 20 and 21 December 1978. Long-term monitoring of the lock-off load on anchor RP-1 lasted approximately 2.5 months until 3 March 1979. The load test on special design anchor RD-1 was started as soon as possible after grouting, with the test being performed from 7 to 12 February. The test apparatus was transferred to special design anchor RD-2 and the load test performed from 14 to 16 February.

4.3 INSTRUMENTATION

Instrumentation used in the anchor load tests consisted of:

(1) load cells to measure anchor load;

(2) rod extensometers to measure tendon elongation and axial displacement at several points within the anchorage zone; and

(3) strain gages to measure strain within the anchorage zone.

4.3.1 Load Cells

Two load cells were used to measure the test load applied to the anchors by the jacking system. The load cells were 800-k capacity, hollow-core, cylindrical load cells, model PC-300M, manufactured by Terrametrics, Inc, Golden, Colorado. A load cell is shown in Fig. 4.2. Load cell reading was done using a Vishay model P-350A strain indicator. A calibration curve was provided by the manufacturer to relate strain indicator meter reading to load. One load cell was used for load testing prototype anchor RP-1 and remained in place after lock-off for long-term monitoring. The other load cell was used for load testing special design anchors RD-1 and RD-2.

The load cells were initially mounted between steel bearing plates beneath the anchor head and jack. This configuration was chosen because it would allow continued measurements of the load on the anchor when the jack is released during resetting of the piston and after lock-off. This load cell arrangement is shown in Fig. 4.6.

During the first load test at RP-1, the test load indicated by the load cell did not coincide with the load indicated by the jack gage. The load cell indicated a lower load, deviating from the jack load by as much as 100 percent. The elastic elongation of the tendon correlated with the jack load. The test was continued assuming the jack load was correct. At the time, it was thought that the calibration of the load cell was in error. Recalibration of the second load cell (the first being locked-off on anchor RP-1) in local laboratories and by Terrametrics were not conclusive in establishing that the original calibration was in error, but did indicate that eccentric loading of the load cell may have contributed to the observed discrepancy.

At the start of the second load test at RD-1, the load indicated by the load cell again was as much as 100 percent lower than that indicated by the jack. During the set up of the test apparatus, care was taken to center the load cell on the tendon extending from the sleeve in the reaction structure. But, because the axis of the setup was at an angle of 45 degrees, initial loading of the anchor caused shifting of the setup. The load test was discontinued and the position of the load cell changed from between the jack and reaction structure to the top of the jack, between the jack piston and anchor stressing head. It was felt that in this position the load cell would not be affected by shifting of the test apparatus during stressing. This load cell arrangement is shown in Fig. 4.7.

The load cell was used in this position for both load tests on RD-1 and RD-2. The load indicated by the load cell was still lower than the jack load, but only by no more than of 22 percent at RD-1 and 30 percent at RD-2. Again the tendon elongation correlated better with the jack load than with the load indicated by the load cell. The tendon bundle was much wider at the top of the jack piston than below at the level of the reaction structure sleeve. Placement of the load cell was no more than 1/8-in. off-center in this position, so position eccentricity was not a major cause of the deviation. Uneven stressing of multi-cable tendons produces a resultant anchor force which drifts eccentrically as the anchor is loaded. This structural eccentricity would be small, probably not more than 0.5 in. in a 7-in.-dia, 25-cable tendon without overstressing some cables.

Small eccentricities are inherent in a field testing situation. The load cells used in the anchor load tests seemed not to be adaptable to these small eccentricities. The load test analyses were based on the load indicated by the jack. During long-term monitoring of the load on anchor RP-1, changes in meter reading on the load cell were correlated with changes in meter reading during load testing to obtain the change in load.

4.3.2 **Rod Extensometers**

Rod extensometers were used to measure two characteristics:

(1) total elongation of the anchor tendons in all the load tests, and

(2) differential elongation of the tendons at various points within the anchorage zone in the load tests on special design anchors.

The rod extensometers consisted of a single telltale rod or a group of telltale rods, fixed at points along the anchorage zone and extending to the surface. Measurements were made of displacements of the ends of the telltale rods, or of relative displacements between a point of interest and a reference telltale rod.

The telltale rods consisted of continuous lengths of 3/8-in.-dia steel rod. The steel rods were isolated from their surroundings by 0.5-in.-dia plastic tubing. The tubing acted as a sheath to prevent friction and deformation of the surroundings from affecting displacement of the telltale rods. A typical telltale rod of a rod extensometer is shown in Fig. 4.3.

Total Tendon Elongation. One telltale rod was installed as a reference for total tendon elongation measurement in all three load test anchors. The reference telltale extended 2.5 to 3 ft below the bottom of the anchorage zone. The reference telltale and its sheathing were bound within the tendon bundle, installed along with the anchor, and grouted into the rock along with the anchorage zone. A length of deformed reinforcing steel was attached to the end of the reference telltale rod to improve its fixity in the grout beyond the anchor anchorage zone.

The reference telltale extended to the surface and was attached to a plastic readout head. The reference telltale and readout head were assumed to be fixed. As the test load was applied to the anchor, measurements were made with a vernier caliper, reading to 0.001 in., between the tendon stressing head and the telltale readout head (Fig. 4.7). Difficulties were encountered which invalidated the assumed fixity of the telltale readout head, as discussed below.

Tendon elongation was also measured using a machinist's rule, reading to 0.001 in., on the stroke of the piston of the stressing jack.

Differential Tendon Elongation Within the Anchorage Zone. In the special design anchors, RD-1 and RD-2, four telltales, in addition to the reference telltale, were installed. The telltales were attached to the spacer plates of the tendon in the anchorage zone (Fig. 4.5). One telltale rod was fixed to each spacer plate. Displacement of the top end of the telltale rod represented displacement of that spacer plate within the anchorage zone. The telltale rods and their sheathing were bound within the tendon bundle, installed with the anchor, and grouted along with the anchorage zone.

The telltales extended to the surface where they were attached to 0.5-in.-dia aluminum rods. The aluminum rods acted as extensions and were inserted into individual bores in the readout head which was attached to the reference telltale (Fig. 4.7). The top of the readout head had a stainless steel plate for use as a reading surface. The plate had small diameter holes which accessed the bores into which the telltale extension rods were inserted.

Measurements were made with a depth dial gage, reading to 0.001 in., from the reading surface to the telltale extension rods. Differential displacement between the telltale rods, relative to the reading surface, was a measure of the differential displacement between the spacer plates in the anchorage zone. The differential displacement was an indication of the tendon strain in the anchorage zone. Questions on the assumed fixity of the readout head did not affect the calculations of differential displacement between the telltale rods, because the same correction was applied to all telltale rods. However, other difficulties were encountered which led to questions concerning the displacement of the telltale rods as in truly reflecting the displacement of the spacer plates, as discussed below.

Assessment of the System. The essence of a rod extensometer system is to consider the telltale rods as rigid and transmiting movements of one end of the rod exactly to the other end of the rod. The telltale rods are supposed to be isolated from movements of their surroundings, frictionless within their sheathing, and free from elastic and thermal elongations and distortion from bending.

Such demanding criteria have been met to various degrees in many field applications, with successful interpretations. Rod extensometers are usually installed in boreholes in rock to monitor movement between points within the rock. Rarely are installations longer than 70 ft. The rod extensometers installed in the anchor load tests were unique in many aspects. In the anchor load tests the rod extensometers were installed in the same drill hole along with numerous other components of the anchor, such as tendon cables, grout tube, anchor hardware, and strain gages. Isolation from these other components was very difficult. The length of the rod extensometers was 180 ft to 200 ft. The slenderness of the telltale rods of that length is such that they cannot be considered rigid. Maintaining the rods frictionless within sheathing of that length was not attainable. Finally, because the rods were bundled within the tendon bundle, freedom from distortion from bending and twisting as the tendon elongated under load was not possible.

These difficulties were demonstrated by the results of the measurements of the movements of the telltale rods, and by observations of erratic jumps of the ends of the rods during load testing. Thus, the interpretation of the data from the rod extensometers was done with much skepticism and used only to define trends and not necessarily magnitudes.

4.3.3 Strain Gages

Four strain gages were used in the anchorage zone of each special design anchor, RD-1 and RD-2, to measure strain within the anchorage zone. The strain gages were embedment type, model CG 129-4, manufactured by Ailtech, City of Industry, California. A typical strain gage is shown in Fig. 4.4.

One strain gage was installed in each section of the anchorage zone, as shown in Fig. 4.5. The gages were mounted on short sections of brass rod to keep them aligned during installation and grouting of the anchor. The brass rods were attached one to each spreader plate of the anchorage zone. The lead wires extended to the surface through a plastic tube acting as a protective sheath. Readout was done with a Vishay model P-350A strain indicator, reading directly in microstrain.

Survival and response of the strain gages was very good. Minor problems were experienced with breaks in the wires during set up of the load test apparatus. After splicing, the gages reacted well throughout the load testing. Some concern of the strain gage response was initially expressed when some gages indicated very small tensile strains or compressive strains. But, in the analysis, this was shown to be due to poor positioning of the gages in relation to the load transfer mechanisms within the anchorage zone itself.

4.4 LOAD TESTING

A typical load test setup is shown in Fig. 4.6. A close-up of the instrumentation readout at the top of the anchor stressing head is shown in Fig. 4.7. The following sections relate the conduct of the load tests on each of the anchors.

4.4.1 Prototype Anchor RP-1

Sequence of Operations. Load testing was performed on prototype anchor RP-1 after the grout attained its design strength as determined by laboratory tests on grout cube samples. The load was generally applied in increments in load-unload cycles to a proof load of 480 k (1.2 times the design load). The load was locked-off at the design load of 400 k. The load at lock-off was monitored for a period of 2.5 months.

The load test was started on 20 December. A seating load of 30 k was applied to the anchor for 2 minutes and released to a nominal zero load of 10 k*. Initial readings were taken of the tendon elongation. The load was then applied in 80-k increments (20 percent of the design load) in load-unload cycles. Loading within each cycle was done in 80-k steps to the peak incremental load of that cycle. Measurements of tendon elongation were made at each step. The peak load of each cycle was held until the elongation readings stabilized, generally within 20 minutes. The anchor was then unloaded to the nominal zero load of 10-k before starting the next cycle.

---

* *The purpose of using a nominal zero load for load-unload cycles rather than true zero is to prevent shifting of the test apparatus upon detensioning of the anchor tendons. A shift of the apparatus would interrupt the continuity of the elongation measurements*

As the test was started, it was noticed that the load indicated by the load cell did not coincide with the load indicated by the jack, as discussed in Section 4.3.1. The test was performed using the load indicated by the jack.

After the proof load of 480 k was reached, the anchor was unloaded in 80-k steps to a load of 80 k and then reloaded in 80-k steps to the design load of 400 k. Elongation measurements were made at each step. This cycle was done to assess friction losses in the anchor system.

At the 400 k load level, the anchor was to be locked-off for measurements of lock-off losses, of creep and relaxation over a 72 period, and to reset the piston of the jack for continued loading to 560 k. The jack used for the tests had a piston stroke of 12 in. For tendon elongations larger than 12 in. the piston had to be reset for continued loading and reset again for final unloading. A lock-off scheme with a modified jack chair and steel shims plates under the anchor head was devised by VSL. As the load on the jack was being released to lock-off the anchor on the shims, the jack chair shifted and some of the shims dislodged. The scheme was not stable enough to attempt locking-off at 400 k. At this point, the test was interrupted and the apparatus dismantled. One tendon cable was broken by a dislodged shim.

At this point, it was decided to delete the last cycle of the test in which the anchor was to be loaded to 560 k (0.8 of the ultimate strength of the tendon cables); additional data on loading the anchors above the design load were to be obtained in the load tests on special design anchors.

The test apparatus was reassembled the following day. The magnitude of the load increments was reduced proportionally to account for the broken cable. The load was applied in 75-k increments to a reduced design load of 376 k. The anchor was locked-off in the normal fashion on the bearing plate.

After final lock-off, frequent measurements of the load were made at increasing time intervals to obtain data on loss of load due to creep and relaxation. At the end of the day, the test apparatus was dismantled. The load cell remained in place between bearing plates under the anchor head. Measurements were made at increasing daily intervals for a period of 2.5 months to obtain data on the long-term loss of load.

Test Results. The results of the load test are presented as load-elongation curves in Fig. 4.8. Each load cycle was not plotted, because the incremental displacements between cycles was small compared to the scale of the curve. The data points plotted in Fig. 4.8 are for the peak load of each cycle up to the proof load of 480 k. The total measured elongation, the elastic elongation of the tendons, and the residual displacement of the anchor are plotted. The elastic elongation is the total elongation minus the residual displacement at the end of each cycle. The theoretical elongation of the free length of the tendons is also shown for reference. Discussions of the test results are given in Section 4.5.

The field data sheets recording the measurements and events of the load test are included in Appendix A, Volume VA.

4.4.2 Special Design Anchor RD-1

Sequence of Operations. After the grout in the anchorage zone attained the design strength as determined from laboratory testing of grout cube samples, the load test on special design anchor RD-1 was performed. The load was generally applied in increments in load-unload cycles to a maximum test load of 800 k (twice the design load).

The load test was started on 2 February. In the early stages of the test, it was noticed that two telltale rods were binding on the anchor head. The test was discontinued and the telltales rearranged. When the test was started a second time, it was noticed that the load indicated by the load cell was deviating from that indicated by the jack, despite efforts during set up of the apparatus to center the load cell carefully. The test was discontinued to change the load cell position, as described in Section 4.3.1.

The test was re-started on 7 February. A seating load was not applied at this time since the anchor had previously been loaded. A nominal zero load of 20 k was applied and initial readings were taken of the tendon elongation, telltales, and strain gages. The load was then applied in 80-k increments in load-unload cycles as described for anchor RP-1. Measurements of tendon elongation were made at each step. At the peak load of each cycle, measurements on the telltales and strain gages were made, in addition to measurement of tendon elongation. To complete each cycle, the anchor was unloaded to the nominal zero load of 20 k.

When the design load of 400 k was reached, the load was held for 16 hr to allow all movements and strains to stabilize for measurements at the design load. The anchor was then loaded in the next cycle of 480 k. After that cycle was completed, the anchor was loaded to be locked-off at the design load for resetting of the jack piston. Some difficulty was encountered when inserting the wedges into the anchor head in the tight space of the standard jack chair. The space was crowded with the 25 cables of the anchor tendon, 5 telltale rods, and wires from the strain gages. Several days delay ensued while the wedging scheme was modified.

The test was continued on 10 February. The anchor was locked-off at 400 k and the jack piston reset. Upon reloading the jack, a lift-off check was made to measure lock-off losses as described in Section 4.5.3. Loading was then continued in 80-k increments in load-unload cycles to a maximum test load of 800 k. Unloading each cycle was only done after resetting to 400 k, because the jack piston was not fully retracted at this load. Measurements of tendon elongation, and on the telltales and strain gages were made as before.

A maximum test load of 800 k was reached without any sign of failure of the anchor. The load at 800 k was held for 10 hr to detect any creep at that load, and to allow stabilization for final readings. The load was then decreased to

400 k and the anchor locked-off to reset the jack piston. A lift-off check was made after resetting the piston, and then the anchor unloaded to zero, completing the test.

Test Results. The results of the load test are presented as load-elongation curves in Fig. 4.9. As with anchor RP-1, each load cycle was not plotted, only the data points for the peak load of each cycle are shown. The test results are discussed in Section 4.5.

Field data sheets recording all the measurements and events of the load test are included in Appendix A, Volume VA.

4.4.3 Special Design Anchor RD-2

Sequence of Operations. The cube samples of the grout in the anchorage zone of anchor RD-2 were inadvertently left out in the cold weather and froze. Based on the results of the laboratory grout strength tests on cube samples from anchors RP-1 and RD-1, it was felt that the grout in anchor RD-2 had attained its design strength in 7 days. The load test at RD-2 was started 19 days after anchor installation and grouting. The load was generally applied in the same manner as for special design anchor RD-1.

The load test was started on 14 February. A seating load of 40 k was applied to the anchor. It was noticed that the rod extensometer readout head was twisting. The anchor was unloaded and the support of the readout head modified.

The load test was restarted on 15 February. A nominal seating load of 20 k was applied and initial readings made of the tendon elongation, telltales, and strain gages. The load was then applied in 80-k increments in load-unload cycles as described for anchor RP-1. The load indicated by the load cell was used in the initial portion of the test, to the 160-k step of the third cycle. At this point, it was realized that the load cell load was deviating further from the jack load than it was in the load test on anchor RD-2. The jack load at this piont measured 250 k. Control of the loading was switched to the load indicated by the jack, and the test continued.

The remainder of the test progressed in the same manner as the load test on anchor RD-1. The design load of 400 k was held for 30 hr. The anchor was locked off for resetting the jack piston at 400 k and a lift-off check made as loading was continued.

The maximum test load of 800 k was reached with no sign of anchor failure. The 800-k load was held for 15 minutes with all readings indicating stable conditions. The anchor was unloaded to 400 k and locked-off to reset the jack piston. After a lift-off check, the anchor was unloaded to zero, completing the test.

Test Results. The results test are presented in Fig. 4.10. Discussions of the results follow in Section 4.5. Field data sheets are included in Appendix A, Volume VA.

4.5 LOAD TEST RESULTS

4.5.1 Anchor Capacity

Ultimate anchor capacity was analyzed by two methods. The first method was based on measured tendon elongation in response to application of load. The second method was based on the ultimate grout-rock bond stress as evidenced by load-strain relationships from instrumentation within the anchorage zone.

First Method. Both special design anchors RD-1 and RD-2, did not show signs of failure during load testing to twice their design capacity. Development of residual anchor displacement as the test load increased is usually one of the indicators of impending failure. Plots of residual anchor displacement for anchors RD-1 and RD-2 are shown in Fig. 4.9 and 4.10, respectively. Both plots show some increasing residual displacement as the test load approached 400 k. The residual displacement stabilized above 400 k for anchor RD-1 and above 480 k for anchor RD-2. At the maximum test load of 800 k, neither anchor exhibited accelerated creep or loss of load during the period of sustained loading (see the field data sheets in Appendix A, Volume VA).

Assuming a uniform distribution of stress along the anchorage zone, an average resistance of 34 k/ft^2 was mobilized in anchor RD-1 and 50.9 k/ft^2 in anchor RD-2. Since the anchors did not fail, the ultimate average bond strength is greater than both these values.

Second Method. Ultimate grout rock bond stress in the anchorage zone was also calculated from measurements made on the rod extensometers and strain gages within the anchorage zone. A discussion of the results and interpretation of the data from these instruments follows in Section 4.5.6. Based on the instrument readings, a disproportionate increase in strain activity was exhibited by anchor RD-1 above 315 k, and by anchor RD-2 above 330 k. Bond stress values at these loads were calculated over the portion of each anchor resisting most of the load (Section 4.5.6). This was interpreted to represent local bond failures. The ultimate local bond strengths thus calculated were 21.3 k/ft^2 in anchor RD-1 and 56.0 k/ft^2 in anchor RD-2.

Comparison With Predictions. Predictions made during formulation of the test program were that anchors RD-1 and RD-2 would sustain a maximum load of 800 k. However, it was expected that the anchors would show signs of failure (accelerating residual anchor movement) as the test load approached 800 k. The ultimate average bond strength value used to make this prediction was 50 k/ft^2 uniformly distributed along the anchorage zone. An ultimate local bond strength value was not predicted.

The conclusions of the load tests concerning anchor capacity are that rock anchors of 400 k design capacity with a factor of safety of 2 are achievable in geologic conditions similar to those at Locks and Dam No. 26, and that current empirical design values for proportioning rock anchors in limestone are conservative.

4.5.2 Friction Losses

During stressing of anchors, a certain portion of the load is expended in overcoming friction among the anchor components. This portion of the load is not transferred to the anchorage zone, and thus is lost in contributing to anchor capacity. Friction losses of a common anchor system were analyzed from measurements made during load testing of anchor RP-1 which was fabricated as a prototype anchor.

Friction losses in the anchor system were analyzed by the cycle method of Fenoux and Portier (1972). By cycling the load on the anchor around the load level of interest, the true load in the tendon and the portion lost to friction can be deduced by examination of the load-elongation curve. Figure 4.11 presents the load-elongation data for one cycle of the load test on anchor RP-1. The cycle plotted is for the 480-k load increment. After that load level was achieved and held for 30 minutes, the load was cycled for lock-off at the design load by unloading to 80 k and reloading to 400 k.

Friction losses in the anchor system were calculated for the 400-k load level and the 480-k load level. During the loading portion of the cycle, friction had to be overcome, so the actual load on the anchor was lower than the applied load by the magnitude of the friction force. The friction force is variable and generally is a function of the applied load and rate of loading. During unloading, the same friction had to be overcome, but in the opposite sense, so the actual load on the anchor was higher than the applied load by the magnitude of the friction force. Thus, the difference between the loading and unloading curves represent twice the friction force at that load level. The line of probable actual load is also shown in Fig. 4.11. The fact that this line, when extended, passed just to the left of the point representing total elongation of the tendon after holding the proof load, indicates that most of the additional elongation exhibited during the hold period was time-dependent release of friction in the anchor system, and not creep of the anchor. The magnitude of the friction force was calculated based on the graphical construction associated with the theory of the cycle method. At the design load level of 400 k, the friction in the anchor system amounted to approximately 24 k, or 6 percent of the applied load. At the proof load level of 480 k, the friction amounted to approximately 28 k, also 6 percent of the applied load.

The plot in Fig. 4.11 also shows that cycling the load reduces the friction in the system. At the first approach to the 320 k load increment in this cycle, the friction losses amounted to approximately 48 k, or 15 percent. But the second approach after cycling reduced that to approximately 21 k or 7 percent. Cycling the load prior to lock-off is commonly recommended for conventional prestressed anchor installations.

No predictions of friction losses were made, although a common value measured in most anchor systems is 10 percent or less.

4.5.3 Lock-Off Losses

The wedges of the VSL anchor system grip the tendon cables by setting in conical seats. In the process, tendon elongation decreases and thus the load on the anchor is reduced. This reduction in load as the wedges are engaged is called lock-off loss.

The best way to measure lock-off losses is with a load cell. Readings of load are made when the wedges are inserted and again when the wedges are fully engaged after release of the stressing jack. In a practical situation, the resultant force of the tendon cables tend to shift as the wedges engage due to uneven stressing and seating. In light of the load cell discrepancies experienced during the testing and the repositioning of the load cell on top of the jack, this technique was not used to measure lock-off losses.

A common construction technique, called a lift-off check, was used to monitor lock-off losses on anchors RD-1 and RD-2. The anchor being tested was slowly reloaded after lock-off was completed. The load on the jack was noted when the lock-off mechanism first started to disengage or when the anchor head first started to lift from the bearing plate. The jack load so noted was the load on the anchor after lock-off, and the difference between it and the proposed lock-off load was the loss. The accuracy of this technique was dependent on the scrutiny of the observer in reacting to the movement of the wedges and anchor head. Upon reloading, the change in elongation of the tendon, as mesured on the jack piston from the lock-off position to the proposed lock-off load was also recorded. The change in load, calculated from the elastic properties of the tendon, was also a measure of the lock-off loss.

Table 4.1 presents the observations and measurements made during the lift-off checks on anchors RD-1 and RD-2, and the calculated lock-off losses. The data shows that the lock-off losses varied from 54 k to 81 k, or 14 percent to 20 percent of the design load of 400 k.

No predictions of lock-off losses were made, although most anchor suppliers report that lock-off losses can be kept to within 5 percent with proper care in the field. A recommended construction technique is to perform several lift-off checks and re-seat the lock-off mechanism at a higher load each time until the loss is at an acceptable value compared to the design load.

4.5.4 Anchor Creep and Relaxation

Use of the terms creep and relaxation has often been arbitrary in the literature. In this analysis, creep is understood to mean movement of the components of an anchor which cause loss of load. Such movement in rock anchors are usually associated with movement within the rock, movement of the anchor block, unwinding of stranded cable tendons, further seating of the lock-off mechanism, and other causes. These movements generally stabilize within a relatively short period of time. Relaxation, on the other hand, is understood to be losing load with no anchor movement, usually associated with high strength steel loss of stress under constant strain. This phenomenon usually continues indefinitely during the life of the anchor, but at a decreasing rate. In common anchor work, both creep and relaxation phenomena tend to occur simultaneously. Distinguishing between them, or trying to measure them separately, to porportionate the cause of loss of load, is practically impossible.

Data on creep was recorded during testing of prototype anchor RP-1. At the peak load of each cycle, the load was held for several minutes until stabilization was indicated or a constant rate of creep was exhibited. The results of these measurements are presented in Fig. 4.12. Creep coefficients in inches and percent of initial stress were calculated for each load level. The creep coefficients were very small for all load levels, being less than 0.5 percent of the initial stress per log cycle of time. The data also indicates stiffening upon increase in load, with values decreasing from 0.35 percent at a load level of 80 k to 0.01 percent at 480 k. The value at the design load of 400 k was 0.06 percent of initial stress. Decreasing creep coefficient with increasing load is beneficial in the interpretation of the performance of the prototype anchor. This indicates mobilization of the rock socket and seating of the system early upon stressing. An anchor approaching failure would exhibit an increasing creep coefficient with increasing load.

No prediction of incremental creep and relaxation was made. The literature relating case histories does not expand on an acceptable value of creep coefficient for rock anchors, except to say that increasing values with load limit the safe capacity of the anchors. A value of acceptable creep coefficient for soil anchors has been tentatively defined in Germany (draft DIN 4125-1974). The code specifies that the safe capacity of a soil anchor shall be, among other criteria, less than $T_k/1.5$, where T_k is the load level at which a creep coefficient of 2 mm (0.079 in.) per log cycle of time is exhibited. This value is much larger than the values calculated from anchor RP-1, as would be expected for rock anchors.

4.5.5 Long-Term Performance

Data for assessment of long-term performance was recorded on prototype anchor RP-1. After the anchor was locked-off at the design load, the load was monitored using the load cell for a period of 2-1/2 months. The load cell was used to monitor the load because the stressing jack was removed and reused for subsequent load tests on anchors RD-1 and RD-2. As explained in Section 4.3.1, changes in meter reading on the load cell during long-term monitoring were correlated with changes in meter reading during load testing to obtain change in load.

The data of change in load with time is presented in Fig. 4.13. Some erratic readings were recorded over the monitoring period, but this has been attributed to temperature effects on the exposed load cell and anchor head. For reference, the air temperature, when available at the time of reading, is shown next to the data point. Generally, warmer temperatures produced higher load readings.

A line representing the mean trend of the readings is drawn through the data points for the portion of the data after one day, which was after the test apparatus was dismantled. The slope of this line was used to extrapolate the loss of load over a 50-year life of a permanent anchor. The rate of loss of load as represented by the trend line is 12 k per log cycle of time. At this rate, the expected loss of load in 50 years (4.26 log cycles after one day) is 51.1 k, or 13.3 percent of the load at lock-off.

For reference, a line representing relaxation of steel is superimposed on the plot. This line is from steel manufacturers for the maximum rate exhibited by stress-relieved stranded cable. The long-term rate indicated is 6.5 k per cycle. It might be assumed that the balance of 5.5 k per cycle measured on anchor RP-1 is due to other creep phenomena, which rate may decrease further in time. However, the separation of load loss due to creep and that due to steel relaxation is at best conjectural.

Predictions of long-term loss of load developed during formulation of the test program were a maximum loss of 2 k over a two-month monitoring period. Anchor RP-1 lost 25 k from the load at lock-off after two months.

As discussed in Section 2.3.2, average long-term losses of 5 percent to 10 percent of the lock-off load have been documented, but this data is from monitoring over much longer periods than was done in the test program. It can be concluded from the test that prototype anchor RP-1 did not perform unlike other actual rock anchor installations.

4.5.6 Stress Distribution Within the Anchorage Zone

Despite difficulties with the instrumentation, trends of stress distribution within the anchorage zones of anchors RD-1 and RD-2 were identified. As discussed in Sections 4.3.2 and 4.3.3, the magnitudes of the movements and strains measured within the anchorage zone were probably not true reflections of the movements and strains at the grout-rock interface. In a complex anchorage, such as the VSL configuration, direct measurements at the wall of the drill hole are needed for analysis of the development of stress distribution resisting the anchor load.

Numerical examples of stress distribution were developed during formulation of the test program. The concept was based on theoretical and laboratory models of bars embedded in grout, and on a field test on a multiple, smooth-sire anchor tendon grouted in rock (Muller 1966). The configuration of the tendon of a VSL anchor is unique. Use of multiple stranded cables make the tendon unlike a multiple, smooth wire tendon. Gathering and spreading the cables to form hourglass shapes make the tendon unlike bars embedded in the center of grout. Thus, the concept of a concentration of stress at the top of the anchorage zone was not valid for the test anchors. The stress was assumed to be tensile and to progress down the anchorage as the load increased and the local bond strength was overcome.

The results of measurements on the strain gages and telltales with the anchorage zones of anchors RD-1 and RD-2 are shown on Fig. 4.14 and 4.15, respectively. The scales of magnitude of strain are distorted to show only trends in strain as discussed above.

One distinct feature stands out in the trend of the strains; that is that the upper portions of the anchorage zone exhibited compression, the center portions tension, and the lower protions exhibited no strain or relatively small magnitudes of strain. This division of the anchorage zones into distinct tension and compression portions may be due to the presence of the spacer plates. Moving with the tendon cables, the plates acted as single embedded anchor plates causing a tension crack immediately behind the plates and compression of the material in front of the plates. Whether compression or tension existed within the grout, the anchor resistance was still developed by shear at the grout-rock interface.

Based on the trends, physical dimensions were assigned to the three portions of the anchorage zones discussed above. The spreader plates and gathered points of the tendon divided the anchorage zones approximately into eighths. In anchor RD-1, approximately the upper 5/8 of the anchorage zone was in compression, resisting the majority of the load. The next 2/8 was in tension, and the remaining 1/8 indicated little strain. In anchor RD-2, approximately the upper 2/8 of the anchorage zone was in compression. The center 3/8 was in tension, resisting most of the load. The lower 3/8 indicated little strain.

As discussed in Section 4.5.1, abrupt changes in strain activity during anchor stressing were used to calculate local bond strength values. Although this method was somewhat conjectural in assigning portions of the anchorage zone to resisting the load, the resulting values are within the range of other empirical values.

Anchor No.	Date	Time	Load k	Elong in.	Event	Losses Elong in.	Losses Load k
RD-1	10 Feb	1345	395	7.46	insert wedges	1.38	70
		1400	-	6.08	locked off		
		1600	-	6.08	jack reset		
		1615	320	-	lift off		75
		1617	395	7.25	re-establish load	1.17	59
	12 Feb	0915	405	8.05	insert wedges		
		-	-	-	jack reset		
		-	330	6.98	lift off		75
		1545	405	8.05	re-establish load	1.07	54
RD-2	16 Feb	1621	390	7.61	insert wedges		
		-	-	-	locked off		
		1658	325	6.60	lift off		65
		1702	380	7.61	try to re-establish load	(1.01)	(51)
		-	390	-	assume load re-established		61
	16 Feb	2215	355	8.02	insert wedges		
		2221	-	6.42	locked off	1.60	81
		2255	-	6.42	jack reset		
		2258	390	7.87			73

Results: Range 54 k to 81 k = 14% to 20% of load at lock-off

ROCK ANCHOR TEST PROGRAM

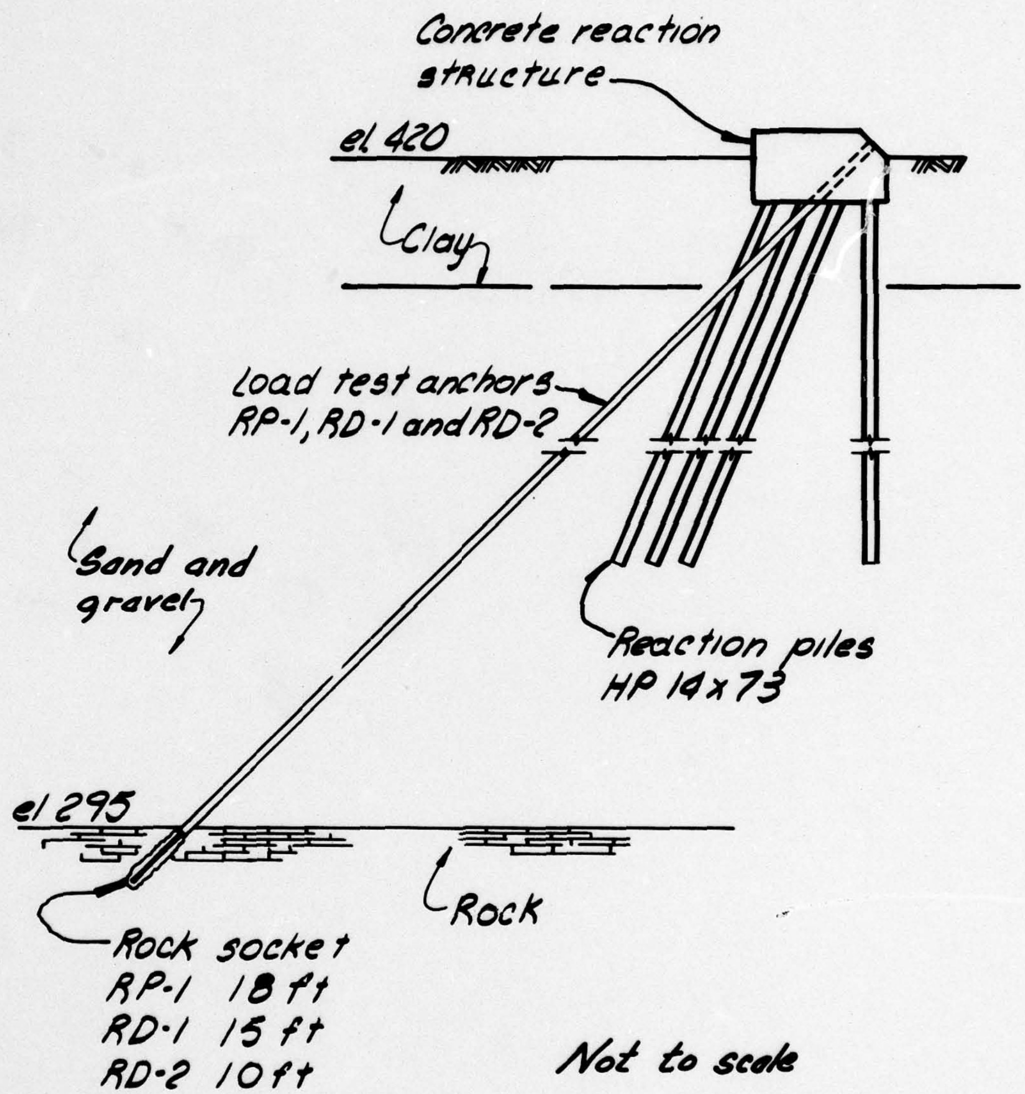
LOCK-OFF LOSSES
ANCHORS RD-1 & RD-2

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

Woodward-Clyde Consultants

77C825 Phase III

Table 4.1



ROCK ANCHOR TEST PROGRAM

SECTION AT
ROCK ANCHOR LOAD TEST

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

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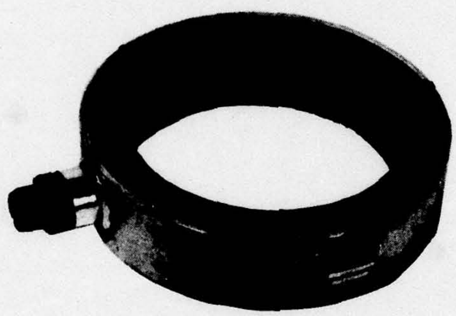
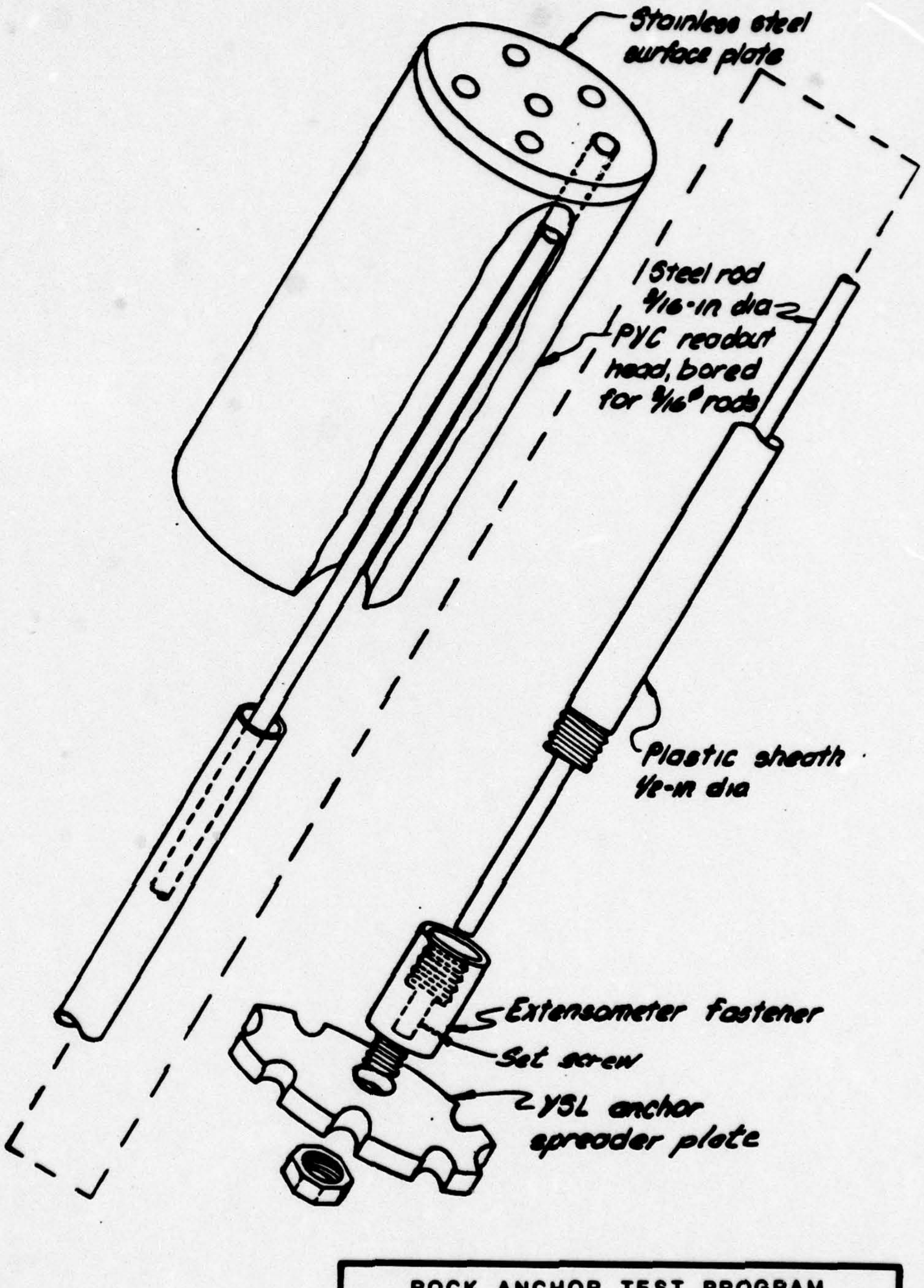
Fig. 4.1



Technical Data
Model PC-300M
Capacity 800 Kips
Sensitivity 0.05%
Outside dia 10-in
Inside dia 7¼-in
Height 6-in.
Weight 20-lb

ROCK ANCHOR TEST PROGRAM
LOAD CELL TERRAMETRICS PC-300M
FOUNDATION INVESTIGATION AND TEST PROGRAM EXISTING LOCKS AND DAM No. 26 ST LOUIS DISTRICT, CORPS OF ENGINEERS. DACW43-78-C-0005
Woodward-Clyde Consultants Y7C825 Phase IX — Fig. 4.2



ROCK ANCHOR TEST PROGRAM

TYPICAL ROD EXTENSOMETER

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0095

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Fig. 4.3

Technical Data
Range ± 20,000 $\mu\epsilon$
Gage factor 2 ± 3%
Length 4-in

ROCK ANCHOR TEST PROGRAM

EMBEDMENT STRAIN GAGE
AILTECH CG-129-4

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-75-C-0005

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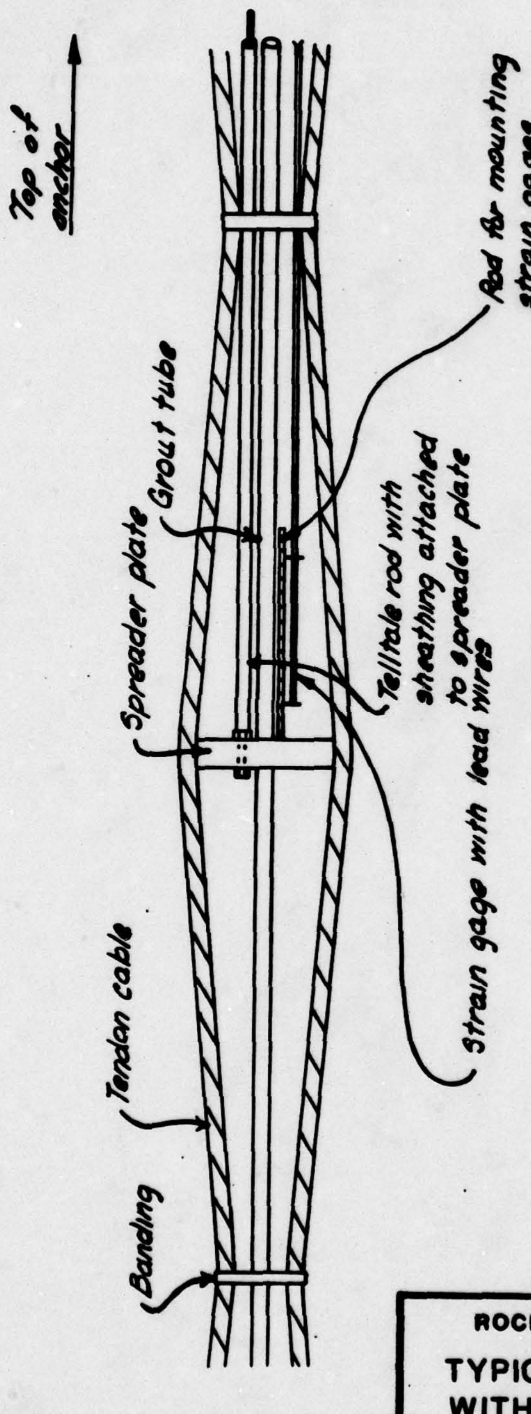
Fig. 4.4



ROCK ANCHOR TEST PROGRAM

TYPICAL INSTRUMENTATION WITHIN ANCHORAGE ZONE

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

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YTC826 Phase III

Fig. 4.5

ROCK ANCHOR TEST PROGRAM

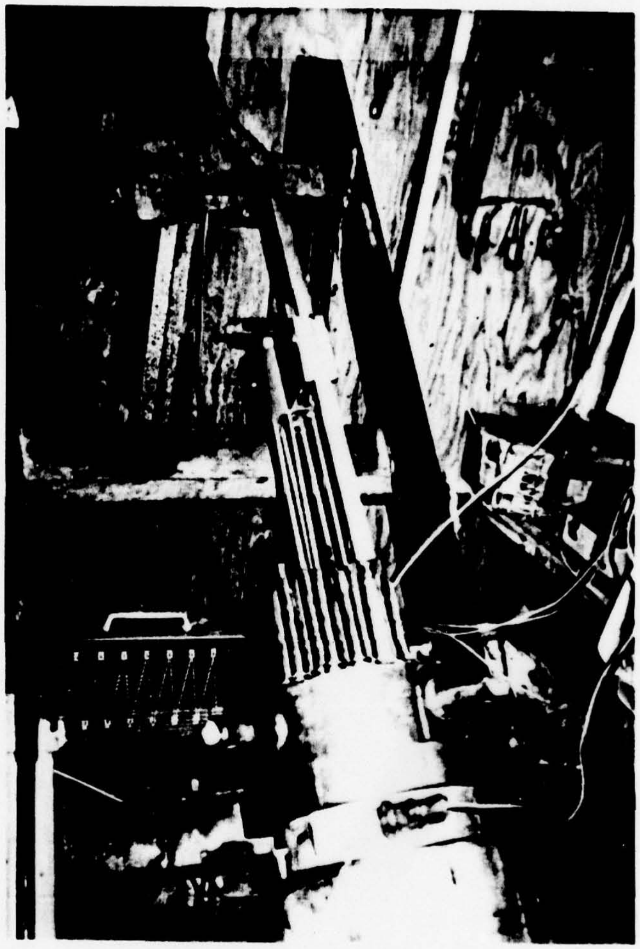
TYPICAL
ANCHOR LOAD TEST SETUP

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST. LOUIS DISTRICT, CORPS OF ENGINEERS
DACW43 78-C-0005

Woodward-Clyde Consultants

Fig. 4.6

Y7C825 Phase IX



ROCK ANCHOR TEST PROGRAM

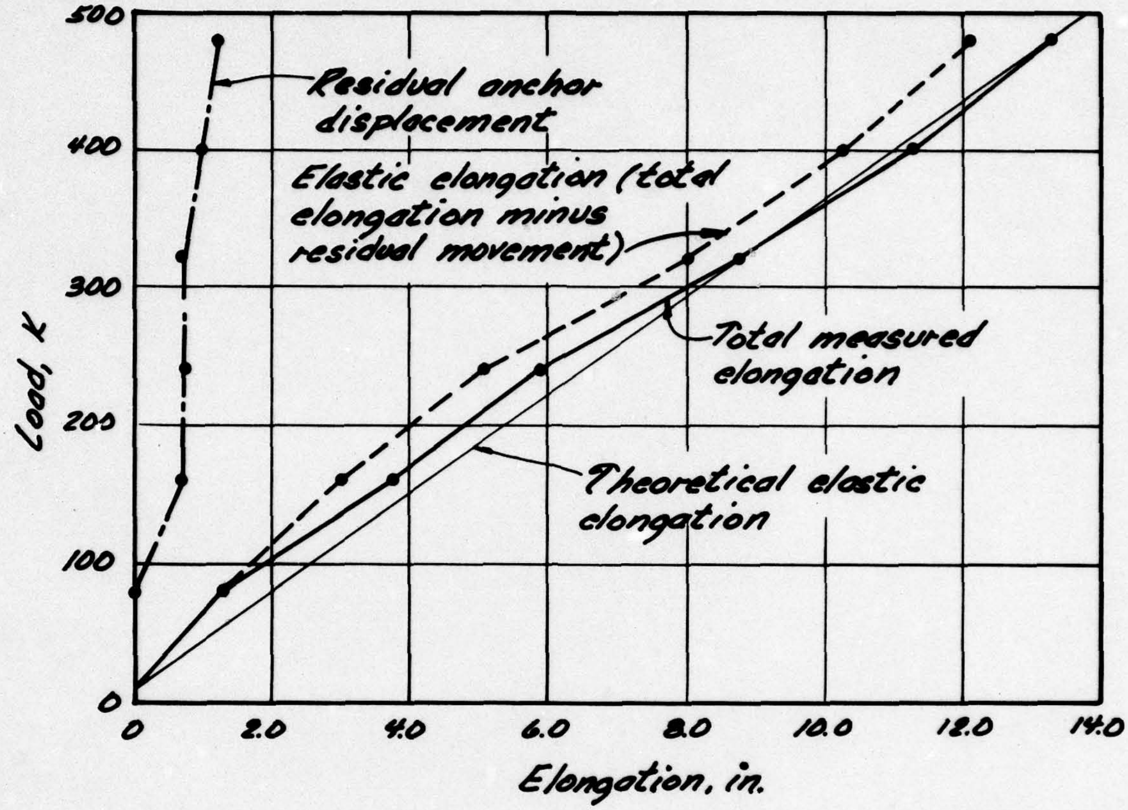
INSTRUMENTATION AT STRESSING HEAD

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS
DACW43-78-C-0005

Woodward-Clyde Consultants

Fig. 4.7

Y7C025 Phase II



Anchor data

No. of strands	17
E of steel, K/in^2	29400
Total length, ft	198.0
Free length, ft	180.0
Anchorage length, ft	18.0
Grout strength lb/in^2 (7 days)	5479

ROCK ANCHOR TEST PROGRAM

LOAD–ELONGATION CURVE ANCHOR RP-1

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS
DACW43-78-C-0095

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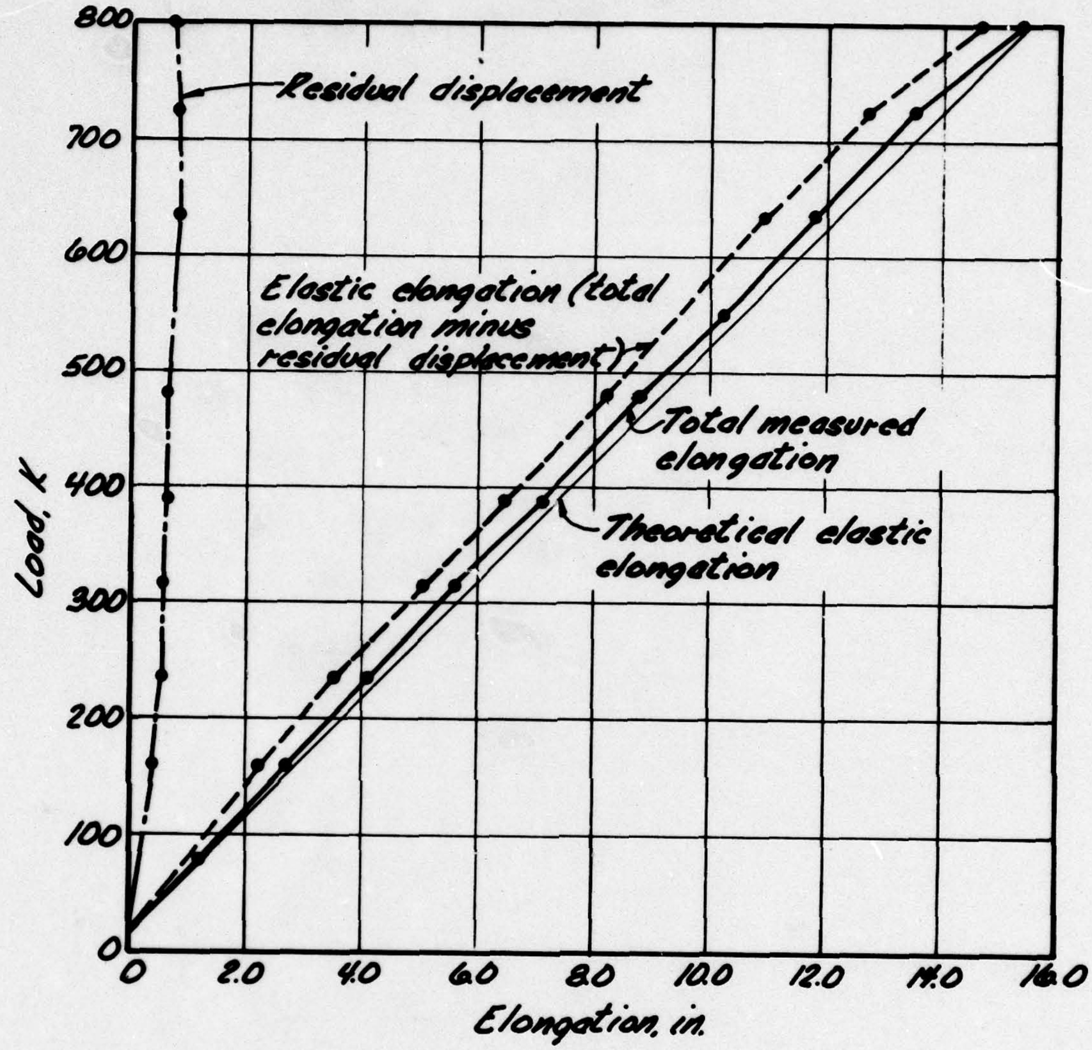
Y7C025 Phase III

Fig 4.8



Anchor data

No. of strands	25
E of steel, $K/in.^2$	29400
Total length, ft	199.6
Free length, ft	184.6
Anchorage length, ft	15.0
Grout strength, $lb/in.^2$ (7 days)	6292

ROCK ANCHOR TEST PROGRAM

LOAD–ELONGATION CURVE
ANCHOR RD-1

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS
DACW43-78-C-0005

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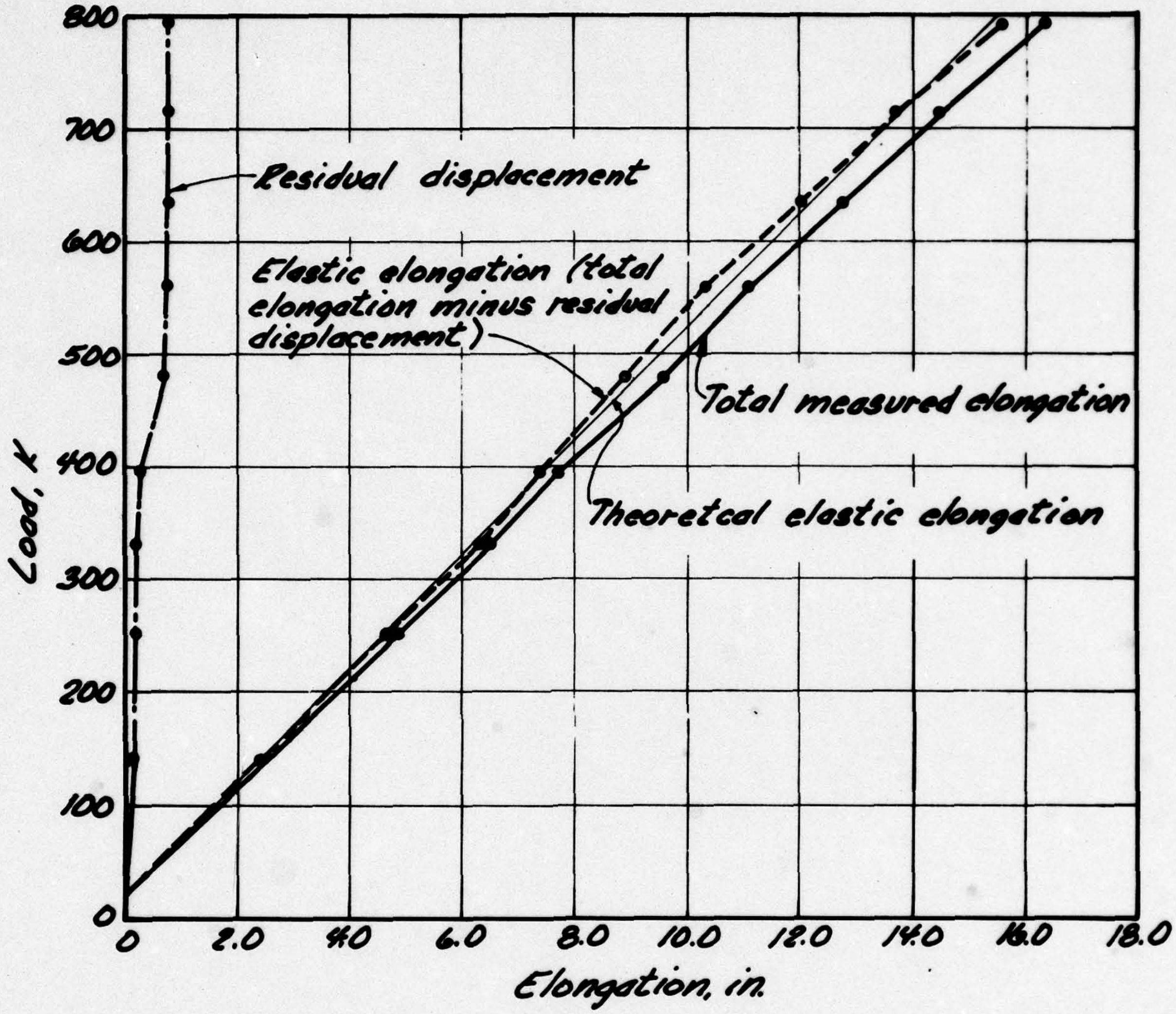
Y7C026 Phase II

Fig 4.9



Anchor data

No. of strands	24
E of steel, K/in²	29400
Total length, ft	188.0
Free length, ft	178.0
Anchorage length, ft	10.0
Grout strength, lb/in²	*

* Cubes were inadvertently frozen in the cold weather

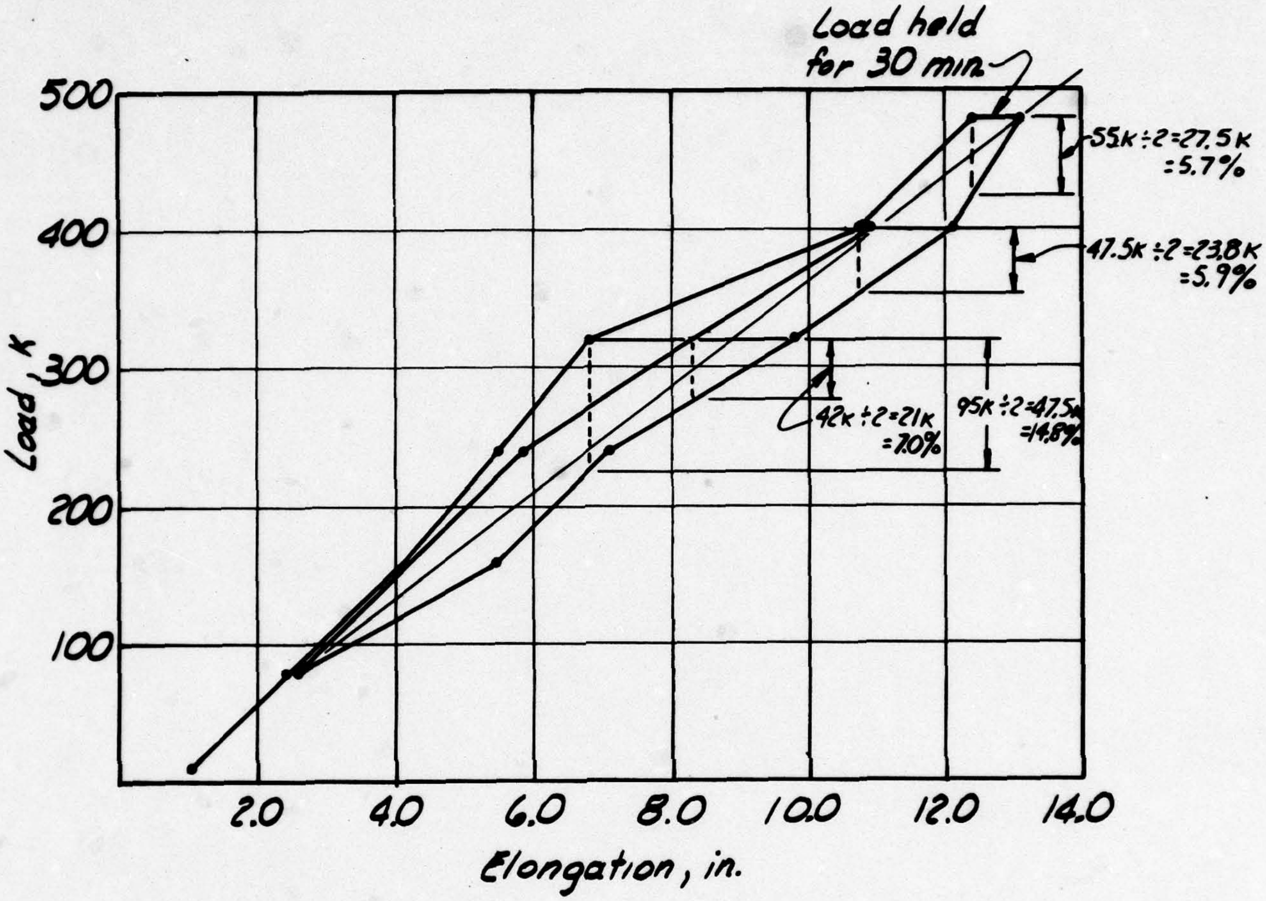
ROCK ANCHOR TEST PROGRAM

LOAD—ELONGATION CURVE
ANCHOR RD-2

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

Woodward-Clyde Consultants
Y7C525 Phase IX

Fig 4.10



Notes
(1) The test cycle shown is for the 480-k load increment
(2) Friction losses were analyzed by the cycle method of Fenoux and Portier (1972)

ROCK ANCHOR TEST PROGRAM

DETERMINATION OF FRICTION LOSSES ANCHOR RP-1

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST. LOUIS DISTRICT, CORPS OF ENGINEERS
DACW43-76-C-0005

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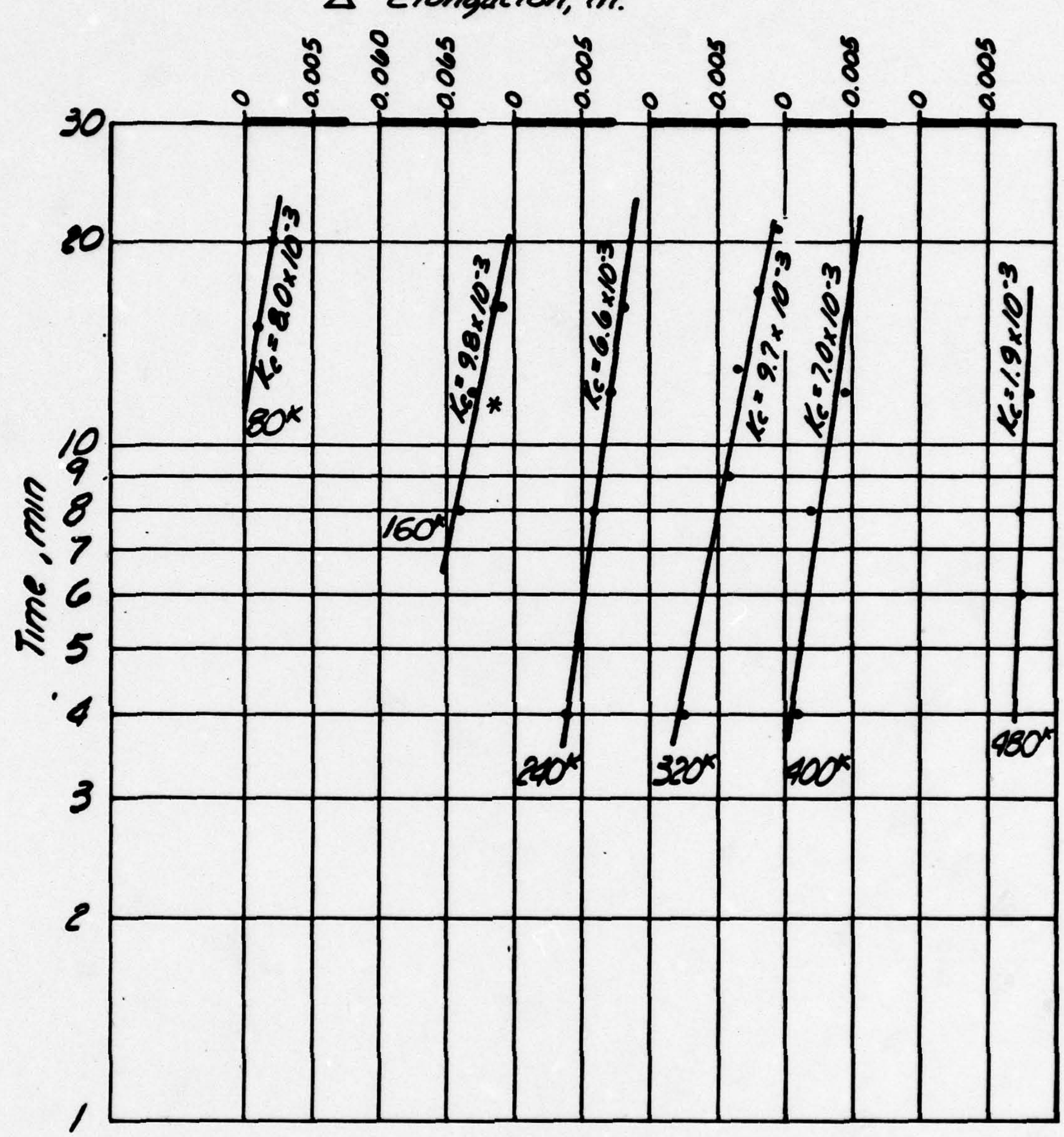
Fig. 4.11

Y7C023 Phase II



Creep coeficient

$$K_c = \frac{\Delta_2 - \Delta_1}{\log \frac{t_2}{t_1}}$$

* Data from caliper readings.

ROCK ANCHOR TEST PROGRAM

ANCHOR CREEP AT SELECTED LOAD LEVELS ANCHOR RP-1

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS
DACW43-78-C-0005

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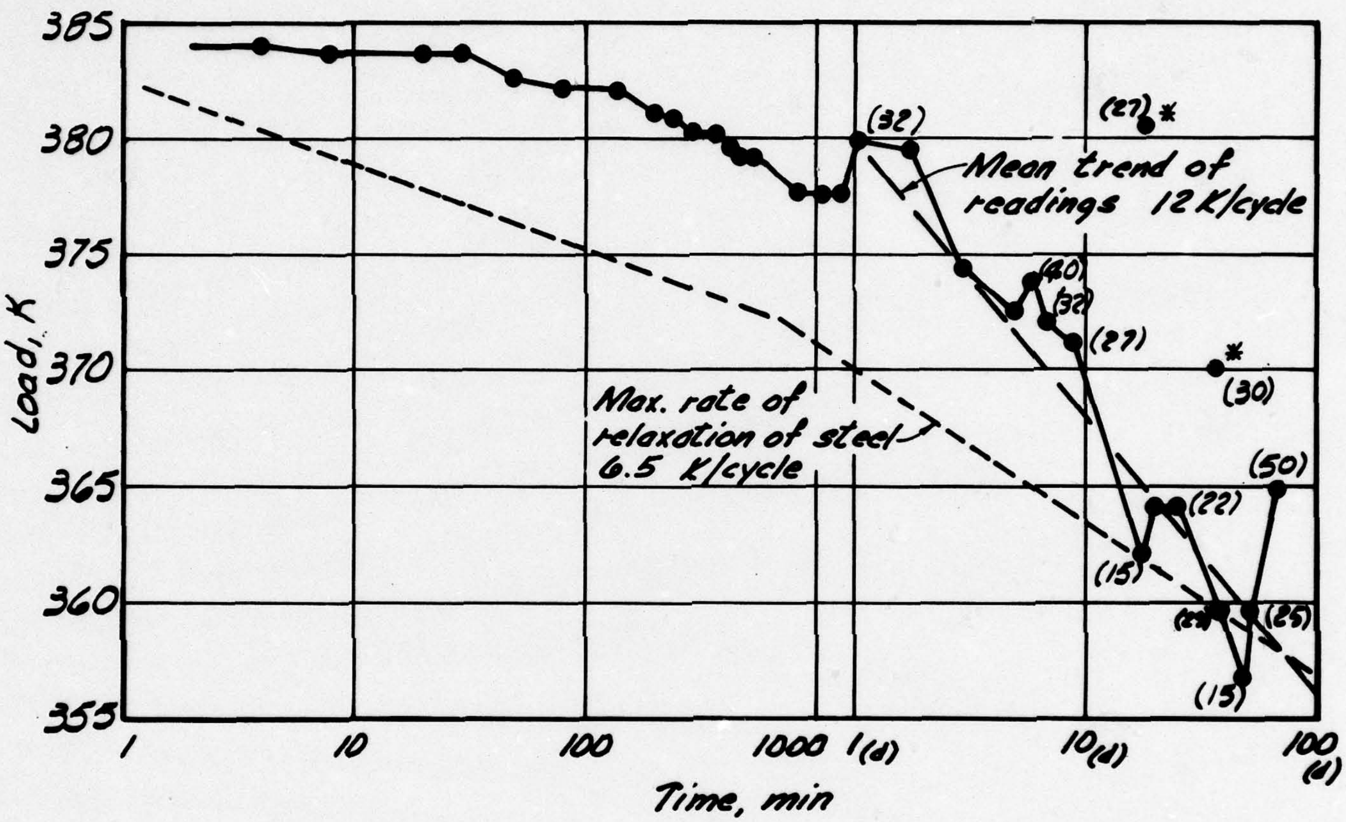
Y7C025 Phase IX

Fig. 4.12



Legend

●* Data point not used in mean trend

(30) Air temperature at time of reading, °F

(d) Days

ROCK ANCHOR TEST PROGRAM

LONG-TERM MONITORING
ANCHOR RP-1

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

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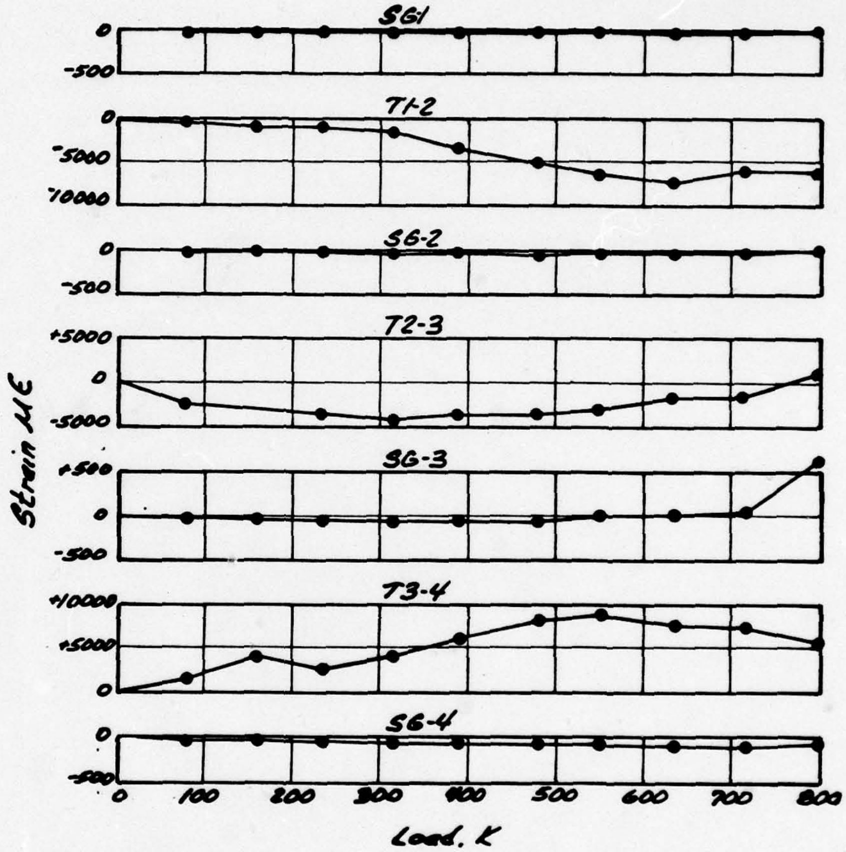
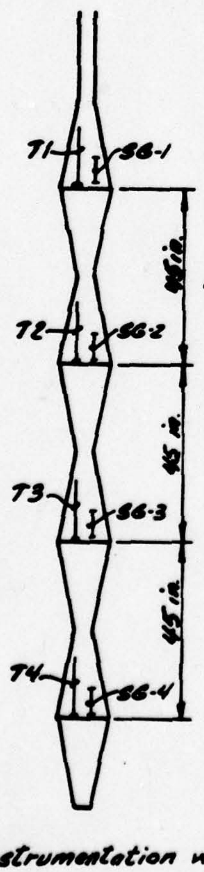
Y7C825 Phase IX

Fig. 4.13





Instrumentation within anchorage zone RD-1

Notes:

(1) SG-1 indicates strain gage No. 1; reads directly in microstrain, $\mu\epsilon$

(2) T-1 indicates telltale No. 1; reads in inches displacement

(3) T1-2 indicates differential displacement between T-1 and T-2 over distance between T-1 and T-2; expressed in microstrain, $\mu\epsilon$

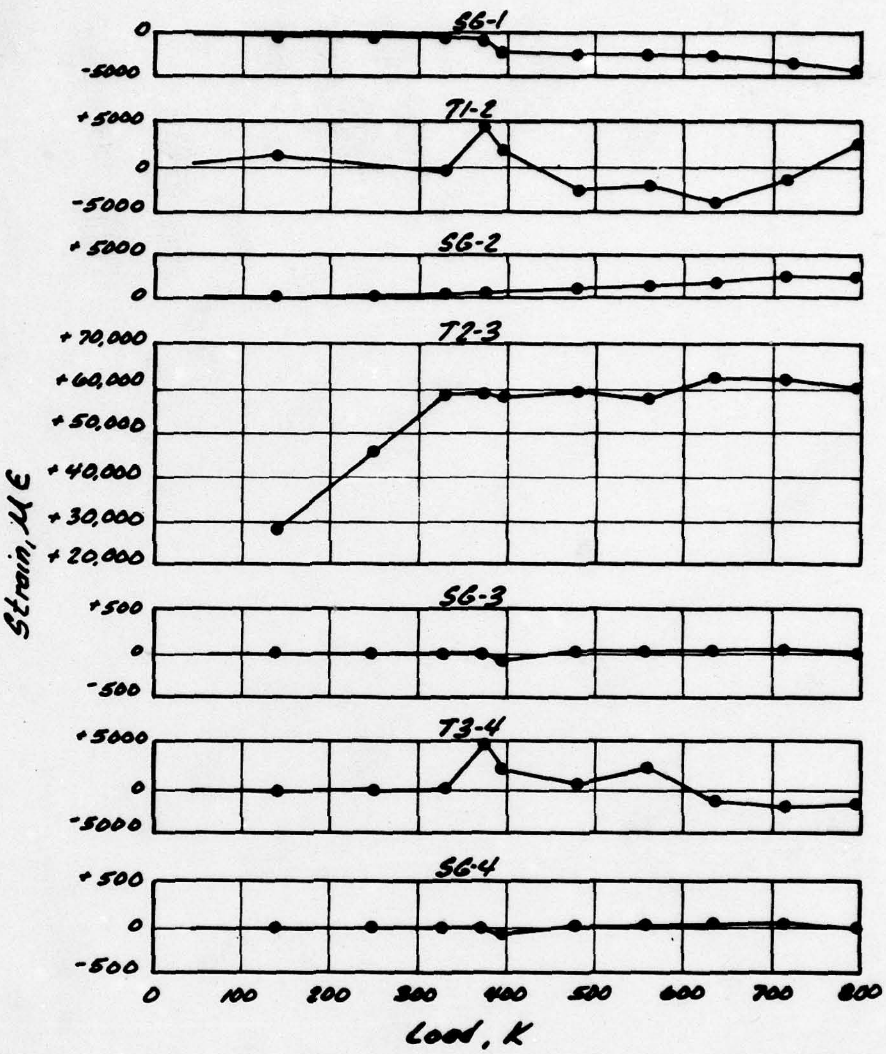
ROCK ANCHOR TEST PROGRAM

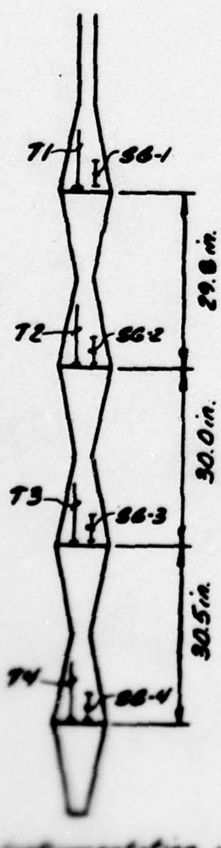
STRAIN MEASUREMENTS WITHIN ANCHORAGE ZONE ANCHOR RD-1

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0008

Woodward-Clyde Consultants
Y7C825 Phase IX

Fig. 4.14





Instrumentation within anchorage zone RD-2

Notes:

(1) SG-1 indicates strain gage No. 1; reads directly in microstrain, με

(2) T-1 indicates telltale No. 1; reads in inches displacement

(3) T1-2 indicates differential displacement between T-1 and T-2 over distance between T-1 and T-2; expressed in microstrain, με

ROCK ANCHOR TEST PROGRAM

STRAIN MEASUREMENTS WITHIN ANCHORAGE ZONE ANCHOR RD-2

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0000

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Y7C825 Phase IX

Fig. 4.15

PHASE IV REPORT

VOLUME V

RESULTS AND INTERPRETATION OF
ROCK ANCHOR TEST PROGRAM

SECTION 5

DRILLING EFFECTS TEST

5 DRILLING EFFECTS TEST

5.1 CONCEPT OF DRILLING EFFECTS TEST

The objectives of the drilling effects test was to assess the feasibility of drilling a group of anchor holes to rock, and to assess the ground disturbance caused by the drilling on the surrounding soil mass.

Four anchor holes were drilled at an inclination of 45 degrees through soil overburden and into rock (Fig. 5.1). The holes were drilled in a square array, 9-ft center-to-center to simulate positioning of anchors in an actual construction scheme. As drilling progressed, measurements were made of the amount of cuttings expelled from the drill holes and on ground instrumentation to assess the disturbance to the ground from drilling. Soil properties were also measured in situ before and after drilling.

5.2 SEQUENCE OF ACTIVITIES

Prior to drilling, ground instrumentation along the alignment of the test drill holes was installed. Installation started on 20 September 1978 and was completed with initial readings taken on 12 October. Inclined inclinometers within the drill hole array were installed in February 1979 in test drill holes. Borings were made in February and September 1978 in the vicinity of the test area to recover soil and rock samples for laboratory testing and to measure initial soil properties in situ.

Drilling of the test drill holes was started on 28 December. The late start was due in part to the unavailability of drill rigs engaged in other activities elsewhere at the test site and in part to difficulties using the drilling tools.

Drilling was started at drill hole RA-I1 using the ODEX 115 tools. The smaller diameter tools were used because the drill hole for inclined inclinometer RA-I1 was not originally part of the drilling effects test program. The ODEX 165 tools were first used in drill hole RA-I1 on 6 February 1979, and then were used in all subsequent drill holes. Test drilling progressed regularly from then on, to drill holes RA-I2, RE-1, and RE-2. Test drilling was completed at RE-2 on 3 March. Drill holes RE-3 and RE-4 were deleted from the test program because sufficient information had been obtained with the first four drill holes.

Ground loss measurements were made at periodic depth intervals during drilling of the test drill holes. The measurements were started on 15 February in RA-I2 and continued for all the remaining drill holes. Measurements of ground movements were made regularly from the start of drilling in drill hole RA-I1, with the final set of measurements made on 5 March. Final soil properties were measured in situ in borehole RA-C1 on 10 March 1979.

5.3 ODEX DRILLING SYSTEM

5.3.1 Concept of the System

The ODEX system of overburden drilling advances a casing together with a drill bit for the entire depth of the hole. By using an eccentric bit to cut an oversize hole and underream the casing, the casing continuously follows the bit as the drill hole is advanced. This has several advantages:

(1) collapse of the soil into the hole is prevented by continuously casing the hole, minimizing ground loss;

(2) by cutting an oversize hole, the casing passes through obstructions, such as boulders, allowing the drill hole to continue as a cased hole of the same size below the obstruction;

(3) the friction on the casing is decreased substantially by cutting an oversize hole, allowing greater drilling depths with less machine power; and

(4) the casing is advanced well into the top of rock creating a better seal in highly weathered or fractured rock for continued drilling of the hole into rock.

When rock is reached, the ODEX tools are exchanged for standard rock bits for continued drilling into rock. Figure 2.4, Section 2, shows the concept of the ODEX system.

The ODEX system is available in four sizes, ODEX 76 for 3-in.-dia (76 mm) holes in rock, ODEX 115 for 4.5-in.-dia (115 mm) holes in rock, ODEX 127 for 5-in.-dia (127 mm) holes in rock, and ODEX 165 for 6.5-in.-dia (165 mm) holes in rock. The ODEX 165 size tools were used in the test program. Figure 5.2 shows the components and dimensions of the standard ODEX 165 tools.

The ODEX 115 and 165 are particularly designed for use with a down-the-hole (DTH) hammer, whereas the ODEX 76 and 127 use top hammers. A DTH hammer delivers drilling energy directly to the bit at the bottom of the drill hole. This eliminates any loss of drilling energy through the drill rods and makes deeper drilling through hard formations easier. Coupled with the ODEX tools, the DTH hammer also imparts an impact driving force directly to the casing shoe for easier advancement of the casing through obstructions and against soil friction.

The DTH hammer is operated with compressed air. The air is expelled through the bit and is then used to flush the cuttings from the hole up inside the casing. A foaming agent and water can be added to the air to assist carrying the cuttings to the surface.

5.3.2 Selection of the System

During the design phase of the test program, several drilling methods, both common and special, were considered for use in installing the test anchors and for assessment of drilling effects. The methods considered included common rotary systems vs special underream options, cased vs bentonite-stabilized drill holes, and top drive vs down-the-hole hammers.

The ODEX system was selected for use in the test program considering the past performance of the various available drilling methods and the demanding drilling conditions which were anticipated in anchor installations. Because potential anchor drilling would be near existing structures, disturbance to the surrounding soil mass and existing foundations from excessive ground loss was of concern. Drilling through obstructions such as existing foundations and boulders in alluvial and glacial soils would be required. Because anchors at inclinations were required, the inclined drill holes must be cased to keep them from collapsing before installation of the anchors. Drilling through large thickness of overburden to reach rock for anchoring, must overcome high friction along the casing. The ODEX system has been designed for these conditions, and has performed well in similar conditions over the years since its development.

Basic rotary methods were not considered suitable in these conditions. Rotary methods often cause excessive loss of ground and are not adapted to pass through obstructions in soil, such as boulders. Predrilling is not suitable for inclined holes because it is difficult to keep the inclined hole from collapsing before the casing is inserted, even if the predrilled hole is stabilized with bentonite drilling fluid. These methods are also limited to holes of shallow depth (usually 70 ft maximum) due to the friction buildup from the overburden on the outside of the casing. Excessive strain on the casing results, and drilling motors capable of very high torque are required. Therefore, rotary methods were not recommended for use in the test program.

The ODEX 165 size tools were used in the test program because the special test anchors required a 6-in.-dia (152 mm) hole in the rock. The ODEX 165 suited this purpose as it can drill up to a 6.5 in.-dia hole in rock. The prototype anchors required a 5-in.-dia (127 mm) hole in the rock. The ODEX 127 could have been used here. But, rather than mobilize two sets of drill tools for the test program, the larger size was chosen; it can also drill with a 5-in.-dia rock bit. The questions about drilling with air below the water table was considered secondary to using the DTH hammer for drilling long lengths and through bouldery material.

The ODEX 165 is in the class of large drilling tools. The system is adaptable to common heavy construction drill rigs. The drill rig used at the site was a Driltech model D-40K manufactured by Driltech, Inc, Alachua, Florida. The rig is described as a truck-mounted, high-pressure air, well drilling machine. The rotary motor had an available torque of 4400 ft-lb, and a maximum speed of 100 rpm with an infinitely variable speed control. The feed system was capable of applying a maximum downpressure force of 40,000 lb. The compressor for the DTH hammer was a Sullair 750 ft^3/min, 250 lb/in^2 unit mounted on the drill rig.

5.3.3 Assessment of the System

Experience with the ODEX 165 system in drilling seven anchor holes in the test program, demonstrated that several items required attention: drilling with air, compatibility of drill tools and casing, and casing strength.

Drilling with Air. The ODEX 165 system is made for use with a DTH hammer operated by high pressure air. The disadvantage of using air to drill below the water table is that the drill hole is always open under negative hydrostatic head which creates a constant potential for water and soil to run into the casing. The tendency is that excessive amounts of material collect in the casing between drilling runs when drilling is stopped to add sections of drill rod and casing. The material collected inside the casing around the drill holes tends to form a plug. In clays, or sands and gravels with some cohesive binder (glacial till, for example), very little material may collect in the casing. In well-graded coarse sands and gravels, small controllable amounts of material may collect. However, in the medium-fine to fine sands at the site, excessive amounts of sand constantly ran into the casing both during drilling and between drilling runs.

In the soils at the site, drilling and flushing with water or bentonite fluid would have minimized the collection of sand and plugging in the casing. But then, a DTH hammer could not be used. An air hammer mounted at the top with the rotary motor would be required. The ODEX 76 and 127 systems have such an arrangement. However, with a top hammer impact, energy at the bit and casing shoe would be lost through long lengths of drill rod and casing.

Other rotary underream systems using water or bentonite drilling fluid exist, but they generally entail withdrawing the pilot bit to insert the reamer. Reaming the drill hole is usually carried far ahead of the casing through an obstruction. The casing is then pushed after reaming is completed. The ODEX system is unique in that it advances the casing continuously together with the rotation of the pilot bit and reamer, and close behind the pilot bit and reamer. This tends to increase production and theoretically provide better control of ground loss.

Compatibility of Drill Tools and Casing. The ODEX tools are manufactured in Sweden and are, thus, of metric size. The impact energy of the DTH hammer is transferred from the hammer to the casing at the contact between the collar of the guide and shoulder of the casing shoe. The tolerances in the sizing of the bit guide and casing shoe must be very close for the pieces to be compatible. The US-size casing nearest to the metric casing normally used in Europe with the ODEX 165 is not within these tolerances. Compatibility between the tools and casing can be obtained by either importing metric casing or modifying the tools (Section 5.3.4). Table 5.1 shows a comparison between metric- and US-size casing and metric- and US-modified-size ODEX guide.

The other ODEX sizes, 76, 115, and 127, do not have a compatibility problem. The US-sized casings nearest the metric casing used within these ODEX sizes are within the needed tolerances. These ODEX sizes have been used successfully in this country for several years.

Casing Strength. In the clean granular soils at the site, the annular space created around the casing by the reamer probably collapsed rather quickly. While the casing was in motion, the friction from the sand and gravel on the casing did not fully develop, because the particles which collapsed into the annular space were also in motion and remained in a loose state. This is unlike a displacement pile which compresses and densifies the surrounding soil, building friction as it is driven. When advancement of the casing stopped, the particles probably settled around the casing, allowing full friction to develop. Upon resumption of drilling, frictional rsistance along the entire length of the casing had to be overcome.

The first type of casing used in the test program was threaded flush-joint casing, 7-5/8-in.-od by 7-in.-id, of grade API J-55. This steel grade is better than standard size UW drill casing which is API H-40. The casing proved to be too weak at the threads and broke several times during drilling. The casing was changed to welded joint casing of the same size but of better steel grade, API K-55. The casing steel itself withstood the hard drilling conditions, but several breaks were experienced at the joints due to poor quality field welds. The weather conditions under which the field welds were made probably contributed to the poor quality. After several welds had broken, x-ray inspection of the welds was done on a periodic basis.

The casing strength problems experienced at the site are not unique to the ODEX system. All drilling systems using casing would encounter high friction, although systems which ream the hole larger than the casing would be expected to decrease the friction somewhat.

5.3.4 Modifications to the System

Two major modifications to the ODEX 165 system were made to adapt the tools to the available steel casing and to the soil conditions at the site.

Casing Size. As mentioned above, the ODEX tools are of metric size. To import metric casing for the test program, or for most US construction projects, is not economically feasible. Thus, the dimensions of the ODEX tools had to be modified to fit the nearest US-size casing.

Figure 5.2 shows the size of the standard ODEX 165 bit guide as manufactured in Sweden. Modification consisted of reducing the diameter of the guide to fit a US casing shoe. This was done by turning down the steel of the guide collar and shaft on a lathe. After turning down, the steel of the guide had to be recase-hardened to reestablish its wear-resistant properties against sand abrasion and impact at the collar. Figure 5.3 shows the modified ODEX guide after re-sizing. Recase-hardening was not done for the first few modified tools received at the site. These tools wore out very quickly. Later modified tools were recase-hardened and performed much better.

Soil Conditions. As mentioned above, using air to flush the cuttings from the hole causes extra material from around the drill hole to be drawn into the casing. In the saturated cohesionless soil at the site, this resulted in excessive amounts of sand constantly running into the casing. The ODEX guide was provided with ports in the collar to allow the cuttings to pass from the bit area into the casing and out to the surface. The standard size ports constituted 2.3 in^2 of passageway, which is 7 percent of the inside area of the casing shoe. This standard port area proved to be too small to effectively handle the large quantities of sand being drawn into the casing at the site. Several modifications to the guide were made before an effective configuration for adequate flushing was achieved. Figure 5.4 shows the modified ODEX 165 guide with enlarged port channels cut into the guide. The modified ports constituted 5.3 in^2 of passageway, which is 17 percent of the inside area of the casing shoe. The enlarged ports, however, had the disadvantage of allowing more material to be flushed, increasing the amount of ground loss.

5.4 DRILLING PRODUCTION

5.4.1 General

During the course of drilling for the load test anchors and for the test drill holes, drilling operations were inspected and observations recorded. The purpose of closely monitoring the drilling operations was to gain information on the expediency of the ODEX method for constructing inclined, deep anchors of large diameter.

Drill rigs were assigned to the rock anchor test program for 97 calendar days. One drill rig was available from 27 November 1978 to 3 March 1979. A second drill rig was available from 17 January to 27 January 1979. During this time, there were extended idle periods due to major breakdown of the equipment or technical drilling problems. The two drill rigs were in operation for both the load test anchors and test drill holes was a total of 83 rig-days. The total time expended was corrected to account for the frequent minor occurrences during which the operations were delayed. The occurrences considered were, for example, those directly related to testing, waiting for materials or auxiliary equipment, unfamiliarity of labor and operators with procedures, and extreme weather conditions. The net productive time spent in drilling holes, installing anchors, and pulling casing, was 637 hr. Operations associated with drilling of the soil and rock accounted for 482 hr. The net drilling footage rate for a ten-hour shift averaged 35 ft/10 hr, with a range of 49 ft/10 hr in drill hole RE-2 to 20 ft/10 hr in drill hole RA-12. Table 5.2 presents the times and rates expended for each load test anchor and test drill hole. Field drilling logs are included in Appendix B, Volume VA.

This production rate is considered slow if compared to the rate usually attained in common construction projects, such as installing anchors for earth retaining structures. In these common projects, one complete anchor installation, 4-in.- to 6-in.-dia and 70-ft long, per drill rig per shift is expected in a variety of materials. The dimensions of the test drill holes (8-in.-dia, 176-ft-long) were, however, significantly larger than common applications. Thus, such a comparison would not be valid. Few case histories are available where comparable drilling was done, but the consensus is that one similar production anchor could be completed under similar conditions in two 10-hr shifts.

5.4.2 Advancing the Drill Hole in Soil

Extensive difficulties were encountered when advancing the drill hole through the overburden soil. Two factors primarily contributed to the difficulties:

(1) fine granular soil running into the casing due to drilling deep below the water table; and

(2) frictional resistance from the sand and gravel on the casing.

Problems generally developed between drilling runs while the next sections of casing and drill rod were added. The problems manifested themselves at the start of the next drilling run.

Excessive quantity of sand running inside the casing tended to cause plugs, preventing the DTH hammer from working and the drill rods from rotating. When this occurred, the remedy generally was to blow large volumes of high-pressure air, sometimes with foam lubricant, to break the plug and flush the casing. At times, this remedy did not work and it was necessary to pull the drill tools and casing out of the hole. To prevent a plug from forming when the drill rig was idle, it became necessary to blow air continuously through the system, flushing the casing of any sand running in. These steps taken to break plugs, or prevent plugs from forming, at times drew additional soil into the casing, aggravating the amount of ground loss.

Due to collapsing of the annular space created by the reamer around the casing, frictional resistance along the casing had to be overcome upon resumption of drilling. Initial operation of the DTH hammer required imparting high impact energy to the casing to overcome the friction and start the casing moving again. The threaded casing proved to be of inadequate thickness at the threads and broke several times. With the welded-joint casing, welds of poor quality tended to break under this high initial force.

Both of these difficulties which were experienced in the test program can be addressed to improve overall production of installing anchors. Excessive running sand might be alleviated by using bentonite drilling fluid as mentioned in Section 5.3.3. Some lubrication along the outside of the casing to minimize friction would also be provided by the use of bentonite drilling fluid. The use of drilling fluid, however, would preclude the use of a DTH hammer. The opinion of an Atlas Copco representative who came to the site was that the use of a DTH hammer was a requisite to drilling the 180-ft-long inclined anchor hole. Casing of adequate thickness and strength, and welds of good quality, are a matter of construction quality control.

Considering these problems which slowed drilling production substantially, the rates realized in drilling the soil with the ODEX 165 tools averaged 1.6 ft/min, with a range of 0.6 ft/min in drill hole RA-I2 to 4.3 ft/min in drill hole RE-1. As noted in Table 5.2, there was a substantial increase in soil drilling rate for the last two drill holes RE-1 and RE-2 over previous drill holes. At this point, the drill rig operators became less concerned that quicker rates of drilling would cause excessive blockages at the bit due to improper flushing. It is also interesting to point out that, as presented in Section 5.7, the amount of ground loss in drill holes RE-1 and RE-2 was less than that measured in drill hole RA-I2.

5.4.3 Advancing the Drill Hole in Rock

As shown in Fig. 2.4, after the casing was seated into the top of rock, the drill hole was continued with common rock drilling tools. In the limestone at the site, a 6-in.-dia tungsten-carbide cross-bit was used.

No problems were encountered in advancing the drill holes into rock. The rock socket was cut evenly and at a steady rate. This can be attributed to the use of the DTH hammer which provides the drilling energy directly at the bit. Use of a top hammer and end rotary motor with long lengths of drill rod tends to cause the bit to bind and drilling to be less even.

The rate of drilling observed in rock was considered normal for cutting limestone by percussion methods. The drilling rate averaged 0.4 ft/min, with a range of 0.1 ft/min in the first drill hole RP-1 to 0.7 ft/min in drill hole RA-I2.

The most time consuming activity associated with drilling the rock was extracting the ODEX tools from the drill hole and lowering and raising the rock drilling tools. Generally, a total of two shifts per drill hole were consumed by these operations. In the last drill hole, RE-2, the rock socket was cut with the ODEX tools to see whether the time-consuming operation of changing the drill tools could be eliminated. The drilling rate realized with the ODEX tools was comparable to that of the standard rock drilling tools, 0.4 ft/min. However, the length cut into rock was only 5 ft, because the contractor was concerned about binding the casing into the rock and not being able to retrieve it.

5.4.4 Pulling the Casing

Some difficulty was encountered in pulling the casing from the drill holes. The cause of the difficulty was high frictional resistance of the long length of casing in the sand and gravel. The drill rig did not have sufficient power alone in its pull chain to break the friction initially, despite the fact that the rig is considered to be in the class of large machines. Assistance in pulling the casing was provided by the use of the DTH hammer. The hammer was mounted at the top of the casing and operated as the pull chain was engaged. Vibratory impact from the hammer on the casing assisted in overcoming the friction and keping the casing in motion. Use of the hammer was continued until a sufficient length of casing was extracted such that the drill rig could pull the casing unassisted. Generally, one and one-half to two shifts were needed to pull the casing from each drill hole.

5.5 ANCHOR INSTALLATION

As originally planned, monitoring installation of the anchors was to be done as part of the drilling effects test. A prototype anchor was fabricated to be used as a dummy installation in each drill hole. However, this portion of the test program was deleted because these activities were observed and documented in installing the load test anchors. The following comments are based on observations made during installation of prototype anchor RP-1 and special anchors RD-1 and RD-2.

On most construction projects, anchor tendons are delivered to the job site pre-fabricated and in coils. On projects where many long anchors are to be used, it may be advantageous to fabricate the tendons on site, having the tendon cables delivered pre-cut in rolls. This was done at the test site. A long work bench was set up on which the cables were laid-out, the hardware, grout tube, and instrumentation attached, and the tendons bound for handling. No difficulties were encountered in fabricating the anchor tendons in this manner.

The tendons for the load test anchors were about 200 ft long. Handling the long tendons for insertion into the drill holes proved to be quite cumbersome. The operation required a team of five ironworkers, one light crane, and one heavy crane. One heavy crane with the longest boom available could not lift the tendons high enough for insertion into the drill holes directly. It was necessary to loop the tendons through a choker on the heavy crane and maneuver them with the light crane, while the ironworkers led the tendons into the drill holes. The operation lasted 3 hr to 6 hr per tendon.

On a regular construction project with such long anchor tendons, lifting and handling the tendons singly by other means, say with a helicopter, for direct insertion into the drill holes would be more expedient.

After insertion of a tendon, the drill hole was flushed with water through the grout tube. Return flow of the flushing water was not obtained. Permeability tests conducted during the site investigation phases showed rock permeabilities averaging 80 LU (Section 3). This range usually indicates the necessity for pregrouting the rock prior to injecting the anchor grout. For timeliness, injecting the anchor grout was done without pregrouting. A larger volume of grout was pumped compared with the volume of the rock socket, but return grout flow was obtained. The grout tube was was pulled up along the tendon about 50 ft and casing extraction was started. As each section of casing was removed, secondary grout was pumped until return flow was obtained. Then, the grout tube was pulled up another 25 ft. This sequence was continued until all the casing was extracted from the hole.

5.6 GROUND DEFORMATION

5.6.1 Instrumentation

Horizontal and vertical ground deformation was measured to assess the potential disturbance to existing structures from drilling to install a group of anchors. Prior to drilling the test drill holes, instrumentation was installed in the ground and on the surface to detect ground deformations due to drilling. The instrumentation included surface reference points, Borros points, and 3-D deformation gages. Inclined inclinometers were also installed. These inclinometers were placed in test drill holes as the holes were completed. Location of the instrumentation in plan and in section is shown in Fig. 5.5 and 5.6, respectively.

Thirty-six surface reference points were installed at the ground surface around the test area and over the alignments of the test drill holes. The surface reference points consisted of steel rods cast in cement grout in shallow, 4-in.-dia holes in the ground surface. Settlement of the surface reference points relative to a benchmark established in rock was measured by optical survey using a precision self-leveling level and a level rod with target vernier reading to 0.001 ft.

Eighteen Borros points were installed below the ground surface between the two lines of test drill holes. The Borros points were installed in vertical boreholes. The Borros tips consisted of three prong points which were expanded in the bottom of the boreholes by mechanical (driving) or hydraulic methods. Inner rods extended from the tips to the ground surface through protective outer pipes. As settlement occurred, the tips moved with the soil, and the inner rods transmitted the tip displacement to the surface. Settlement of the top of the inner rods relative to the benchmark were measured by optical survey with a precision self-leveling and a level rod with target vernier reading to 0.001 ft.

Three 3-D deformation gages were installed in boreholes below the ground surface between the two lines of test drill holes. The 3-D deformation gages were a combination of an inclinometer casing and Sondex rings.

Horizontal displacement of the soil mass along the profile of the boreholes was measured using a Digitilt inclinometer (Slope Indicator Co) in special grooved casings installed in the boreholes. An inclinometer probe measures incremental inclination of the casing. Incremental horizontal displacement of the casing is calculated from the incremental inclination. The total displacement of the top of the casing with respect to the bottom is computed by summing the incremental displacements from the bottom to the top of the casing. Repeating such measurements periodically provides data on magnitude and direction of deflection of casing with depth and, thus, deformation of the surrounding soil mass.

Settlement within the soil mass was measured using a Sondex settlement probe (Slope Indicator Co). The probe was lowered into the inclinometer casing which was fitted with plastic sleeves with metallic rings mounted at selected elevations. The rings were friction fitted to the casing allowing them to move vertically with the soil surrounding the casing. Electrical circuitry within the probe locates the metal rings and a survey tape attached to the probe measures their depth relative to the top of the casing. A series of readings allow settlement of the soil to be calculated.

Three inclined inclinometer casings were also installed in completed test drill holes RA-I1, RA-I2, and RE-1. These inclinometers were used to measure horizontal movements from drilling subsequent test drill holes. The operation and readout of the inclined inclinometers was the same as that for the vertical inclinometers at the 3-D deformation gages, except that a probe modified to read at 45 degrees was used.

5.6.2 Results

Ground deformation, at the surface, and at depth was calculated and plotted upon completion of each test drill hole. Figures 5.7 through 5.10 present the settlement of the ground as contours of equal settlement after each event. Data from the surface reference points, Borros points, and Sondex were used in making these plots. Field data sheets of the measurements on the ground instrumentation are included in Appendix C, Volume VA. Data from the inclinometers was used to evaluate horizontal displacement of the ground at depth.

Maximum resultant horizontal displacement after each event are presented in Fig. 5.11. Profiles of horizontal deflection for each inclinometer are presented in Appendix D, Volume VA.

The results of the measurements show large ground deformation developing during drilling of the first test drill hole, RA-I1. Settlements in the order of 0.15 ft above the alignment of the drill hole and horizontal displacements of 0.5 in. towards the drill hole were measured after test drill hole RA-I1 was completed. This is probably due to the fact that two unsuccessful attempts were made at the location of RA-I1 before a third attempt was successful.

After drilling each subsequent test drill hole, the settlement continued to increase up to a maximum value of 0.40 ft. The seat of the settlement seemed to originate at approximately el 340, 10 ft to 15 ft above the alignment of the test drill holes. This correlates well with the ground loss measurements which recorded large quantities of material flushed from the drill holes between el 350 and el 340 (Section 5.7). The tendency of the ground subsidence seemed to propagate upwards and slightly to the south towards the top of the drill holes, paralleling the alignment of the drill holes.

Predictions made during formulation of the test program expected a maximum settlement of 0.3 in., and a trend of contours paralleling the drill holes, decreasing with distance above the drill holes. The magnitude of the settlement, however, exceeds the predictions almost tenfold. This is understandable when considering the fact that the amount of ground loss also was far in excess of that predicted (see Section 5.7). The trend of the contours of measured settlement generally follow the predictions.

After drilling each subsequent test drill hole, the horizontal deformation of the soil generally continued to increase towards the east in the direction of RA-I1 and RE-1, but at a decreasing rate. The inclinometer in 3-D gage RA-3D3 did show some movement towards the west in the direction of RA-I2 and RE-2 after drilling these test drill holes, but the movement was small (0.05 in. and 0.10 in.) when compared to the easterly movements after drilling RA-I1 and RE-1. No predictions of horizontal ground deformation were made.

5.7 GROUND LOSS

5.7.1 Measurement Technique

The quantity of soil expelled from the drill holes was measured and compared with the theoretical quantity to assess the degree of ground loss due to drilling.

As drilling was in process, the cuttings were flushed from the drill hole through a discharge hose attached to the top of the casing. The discharge hose was directed into a steel bin to contain the cuttings. Small holes in the bottom of the bin allowed any water carried with the cuttings to drain. Periodically during drilling, usually at 25-ft intervals, drilling was stopped to add sections of drill rod and casing. At these intervals, measurements of the quantity of cuttings expelled from the drill holes during the previous run were made. Two methods of measuring were used, depending on the relative amount of material collected. When the amount of material was small and easily handled, the material was scooped from the bin and its weight and water content determined. Using soil weight-volume relationships, the volume of soil normalized to a mean in situ dry unit weight (105 lb/ft^3) was calculated. When the amount of material was too large to be easily weighed, its volume was measured directly in the bin and equated to the volume at the in situ unit weight. Laboratory tests give values of 95 lb/ft^3 to 112 lb/ft^3 for the minimum and maximum dry unit weights of the sands and gravels at the site. Thus, the direct volume measurements might be in error by up to 10 percent. But, considering the excessive volume percentage over the theoretical volume when the amount of material collected in the bin was large, this degree of accuracy was acceptable. Field data of the cuttings measurements are presented in Appendix E, Volume VA.

5.7.2 Results

The quantity of cuttings were measured in three test drill holes, RA-12, RE-1, and RE-2. The data shows that the amount of material flushed from the drill holes during drilling is far in excess of the theoretical volume of the drill holes. The degree of ground loss (actual volume to theoretical volume) generally was in the range of 2 to 5 times the theoretical volume of the hole. A low value of 0.25 times the theoretical volume was measured in RE-1 in the first drill run below the clay of the flood plain deposit, from el 398 to el 389. A high value of 8.7 times the theoretical volume was measured in RE-2 in the last drill run above top of rock, from el 300 to el 295. Plots of degree of ground loss vs depth for the three test drill holes are presented in Fig. 5.12 through 5.14. The data shows a triangular trend of increasing ground loss with depth, which would be expected under increasing hydrostatic head. All drill holes experienced a locally greater degree of ground loss approximately between el 350 and el 340. Drill hole RA-12 experienced an extreme amount of cuttings flushed during an interruption in drilling at el 355. Drilling difficulties at that elevation probably contributed to the high degree of ground loss measured during that run. These elevations generally coincide with a zone of very fine to silty fine sand encountered in the borings. This material can be expected to run more profusely into the casing requiring excessive flushing to keep the casing clear for continued drilling. In the two drill holes, RE-1 and RE-2, larger degree of ground loss was experienced from el 302 to el 295 when drilling through the bouldery zone just above top of rock than was experienced from the other soil strata.

Predictions of the amount of ground loss were not made during formulation of the test program; however, numerical examples used to predict magnitude of settlement assumed a value of degree of ground loss of 1.2 times the size of the reamed hole. The measured ground loss exceeded this value on an average by a factor of 2 to 4.

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RESULTS AND INTERPRETATION OF ROCK ANCHOR TEST PROGRAM. EXISTIN--ETC(U)
JUL 79 J PEREZ, R A FASANO DACW43-78-C-0005

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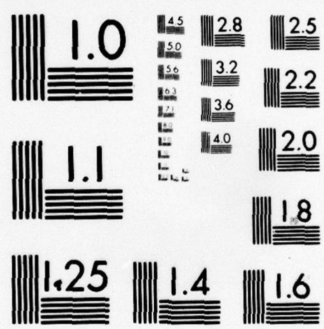


MICROCOPY RESOLUTION TEST CHART

5.8 GROUND DISTURBANCE

Ground disturbance due to drilling was assessed by comparing soil properties before and after drilling the test drill holes. The soil property compared was in situ relative density derived from static cone penetration tests in boreholes. A test section between el 375 and el 330 was chosen for the study because cone data above el 375 were too erratic and no cone data below el 330 were obtained after drilling.

The discussion in Section 3, and particularly Fig. 3.5 and 3.6, give original soil properties at the site, and mean values of cone resistance and relative density in the test section. The original relative density of the sands and gravels between these elevations generally ranged from 60 percent to 80 percent.

After the four test drill holes were completed, a cone penetration sounding, RA-C1 was made at the location shown in Fig. 5.5. The sounding was discontinued at el 330 because the sand was too loose to maintain an open hole despite the use of heavy drilling fluid. The alignment of the test drill holes passed that location at el 328. Results of the sounding show a marked decrease in cone resistance just above the alignment of the test drill holes, with less influence with distance above the drill holes. Figure 5.10 shows mean values of cone resistance before and after test drilling, and a plot of their ratio. The relative density values after test drilling derived from the cone resistance range from 60 percent between el 375 and el 360 to 40 percent and close to zero between el 360 and el 330.

No predictions of change in soil properties were previously made, but the significant effect on the in situ density of the soil correlates well with the ground loss and movement measurements which indicate large volumes of material expelled from the drill holes and collapsing of the soil structure.

	Dimensions: Metric Size Equiv in.	Dimensions: US Size in.
Casing outside diameter	7-1/2 min 7-11/16 max	7-5/8
Casing inside diameter	7-1/16 min 7-1/8 max	7
ODEX Guide outside diameter	6-15/16	6-27/32 (after modification)

ROCK ANCHOR TEST PROGRAM

COMPARATIVE DIMENSIONS OF ODEX TOOLS

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-76-C-0005

Woodward-Clyde Consultants

V7C025 Phase II

Table 5.1

Drill Hole	Total Days	Net Production hr	Net Drilling hr	Net Footage in 10 hr	Soil Drilling Rate ft/min	Rock Drilling Rate ft/min
RP-1	6	68	52	39	0.6	0.1
RD-1	6	91	60	34	0.7	0.4
RD-2	10	85	53	36	0.8	0.5
RA-I1						
(with ODEX 115)	38	66	66	(20)[1]	(1.2)[1]	-[2]
(with ODEX 165)	7	76	43	41	0.7	?[2]
RA-I2	7.5	115	99	20	0.6	0.7
RE-1	4.5	84	72	29	4.3	0.4
RE-2	4	52	37	49	3.7	0.4[3]
Totals and Averages	83	637	482	35 (average)	1.6 (average)	0.4 (average)

Notes:

(1) *Rates using ODEX 115 in RA-I1 were not included in the averages*

(2) *Drilling of the rock socket in hole RA-I1 was not monitored*

(3) *The rock socket of RE-2 was drilled with the ODEX tools. The rock sockets of the other drill holes were drilled with a standard rock bit*

ROCK ANCHOR TEST PROGRAM

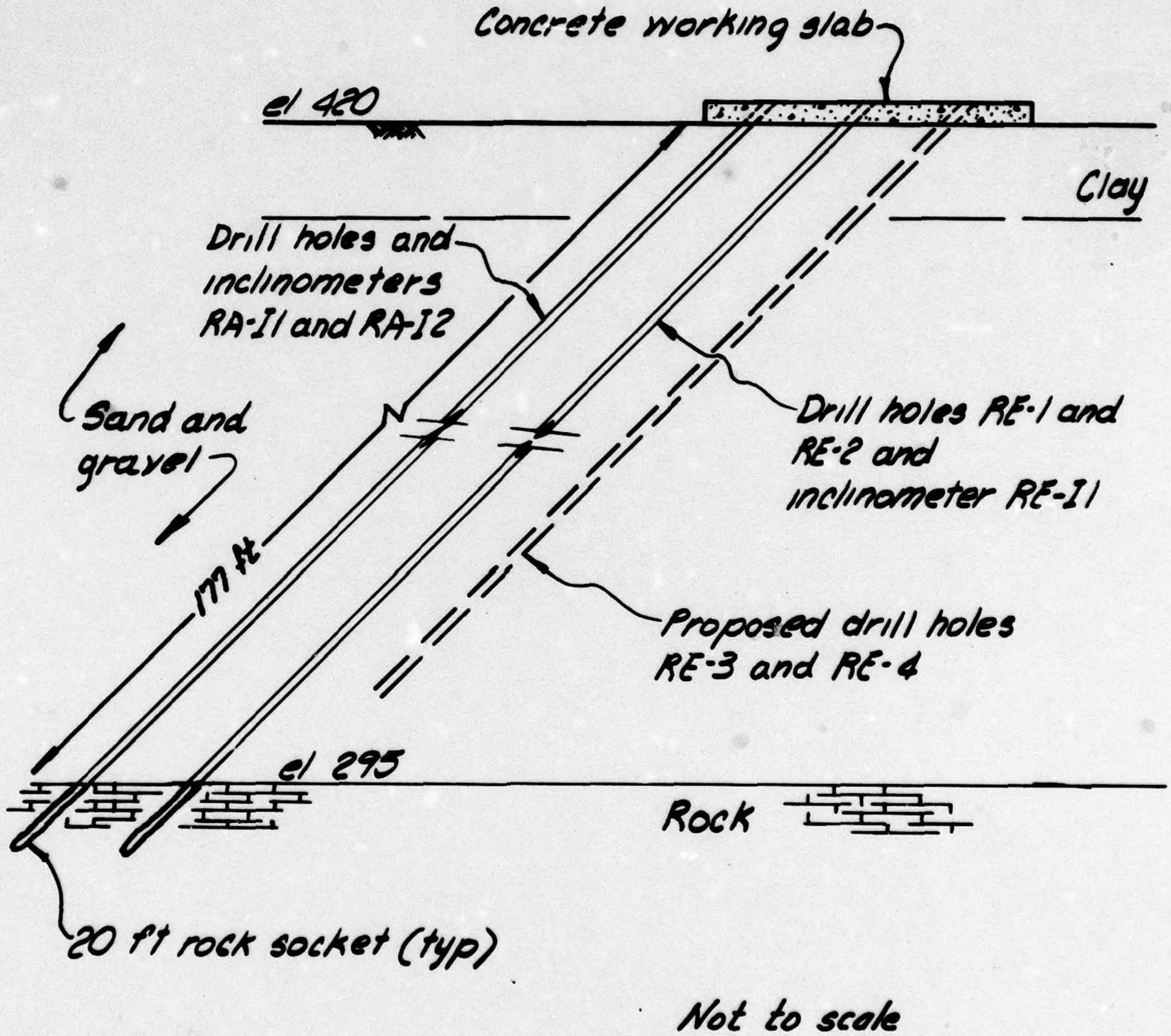
DRILLING PRODUCTION

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

Woodward-Clyde Consultants

Y7C826 Phase III

Table 5.2



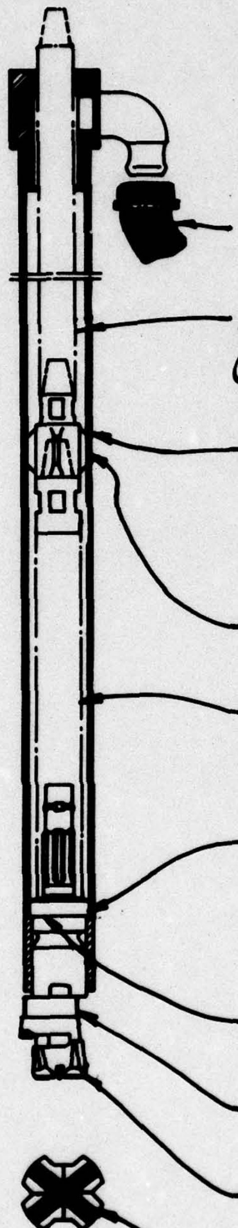
ROCK ANCHOR TEST PROGRAM

SECTION AT
DRILLING EFFECTS TEST

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-[illegible]

Woodward-Clyde Consultants
Y7C628 Phase II

Fig. 5.1



Component		Dimensions m.m.	Dimensions in.
Discharge hose	ID	100	4
Drill rod (API 3½ reg)	OD	114	4½
	L	3050	120
Casing	ID	180	7 1/16
	OD	194	7 5/8
	L	3050	120
Wingcoupling	OD	178	7
DTH Hammer (COP 6)	OD	136	5 3/8
	L	1245	49
Casing Shoe	ID	168	6 5/8
	OD	194	7 5/8
	L	280	11
Guide	OD	176	6 15/16
Reamer	OD	212	8 3/8
Pilotbit	OD	152	6
Rockbit	OD	165	6½

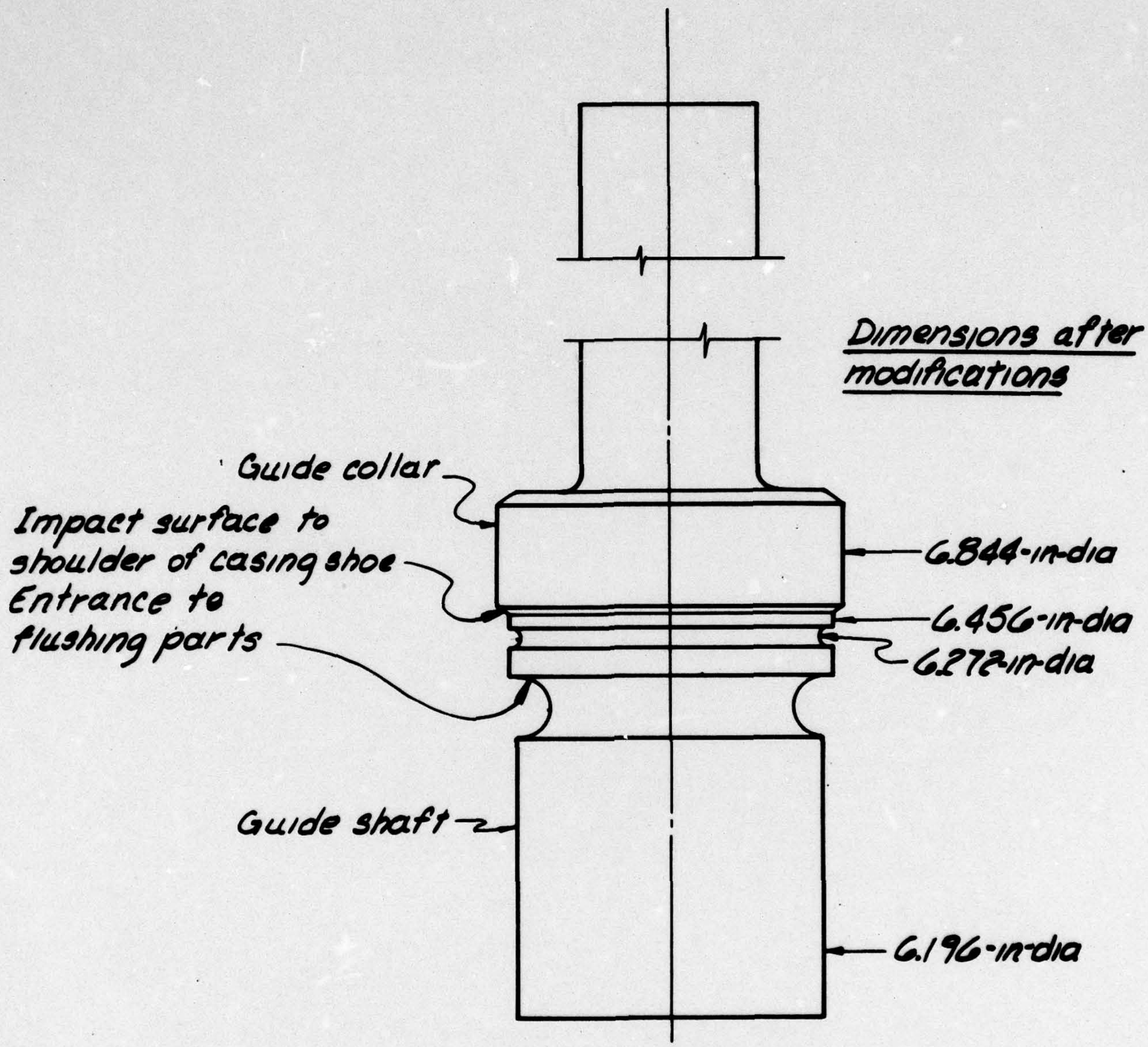
ROCK ANCHOR TEST PROGRAM

DIMENSIONS AND DETAILS
STANDARD ODEX 165

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

Woodward-Clyde Consultants
V7C825 Phase II

Fig. 5.2



Note:
All surfaces are re-case hardened after modification

ROCK ANCHOR TEST PROGRAM

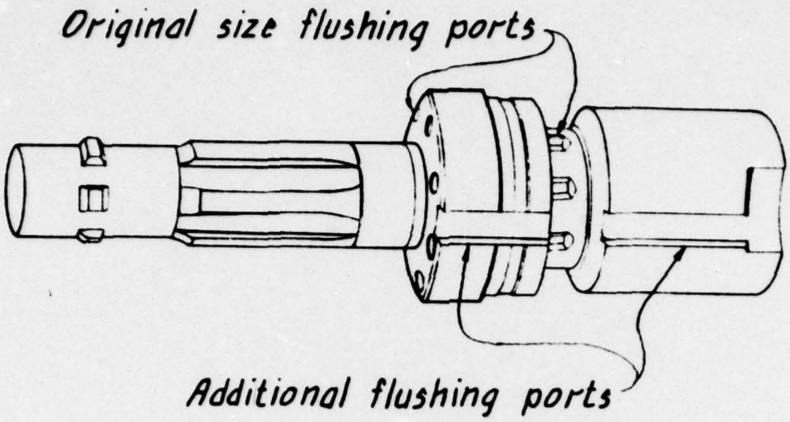
MODIFIED ODEX 165 GUIDE FOR US SIZE CASING

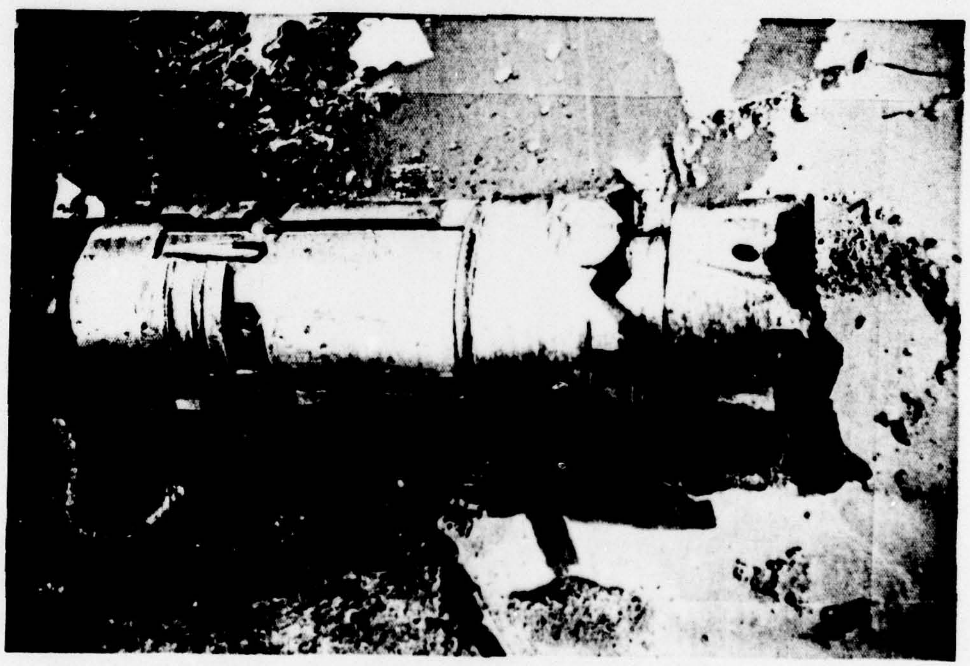
FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0002

Woodward-Clyde Consultants

Fig.5.3

YTC826 Phase III





ROCK ANCHOR TEST PROGRAM

MODIFIED ODEX 165 GUIDE FLUSHING PORTS

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

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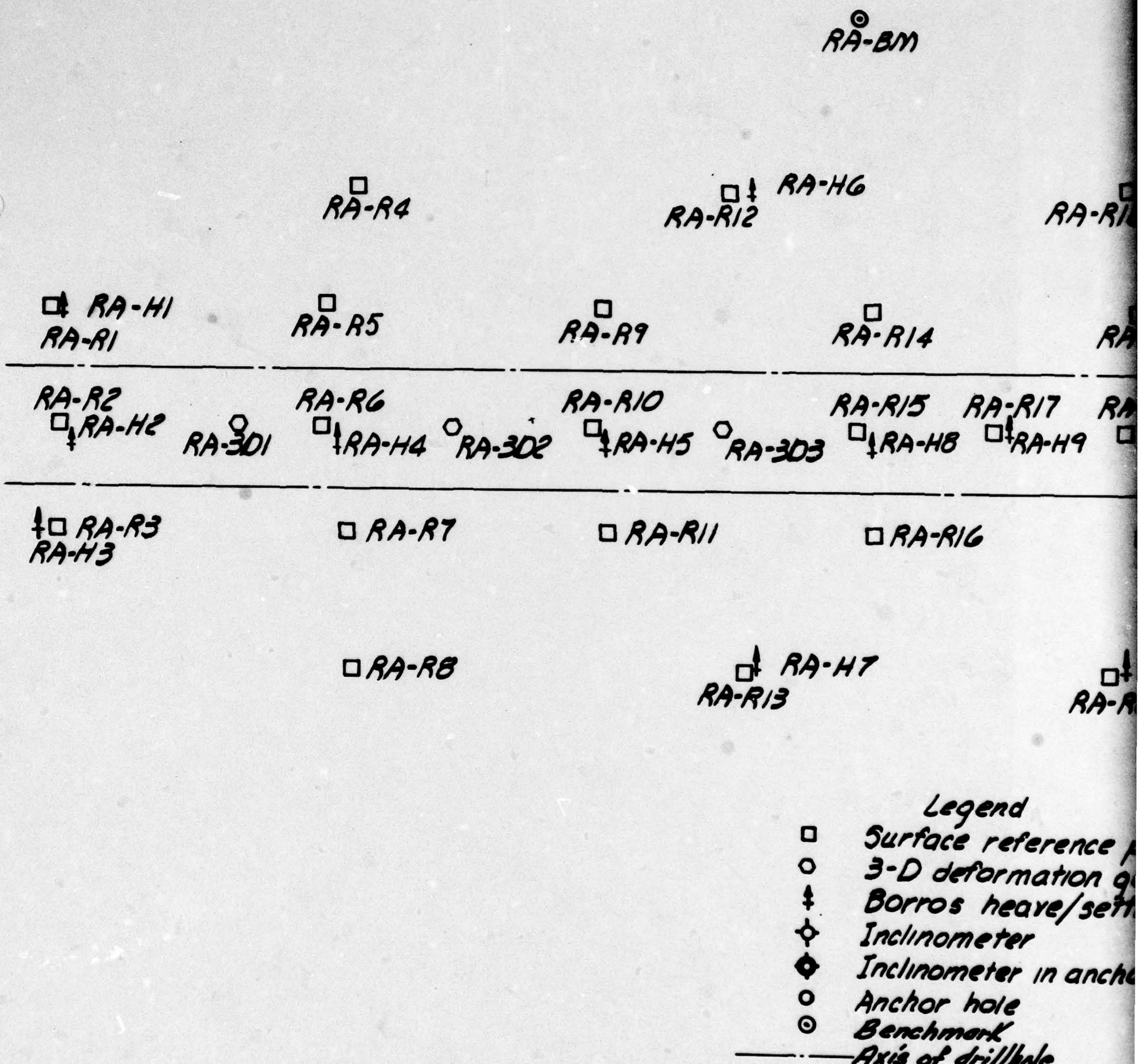
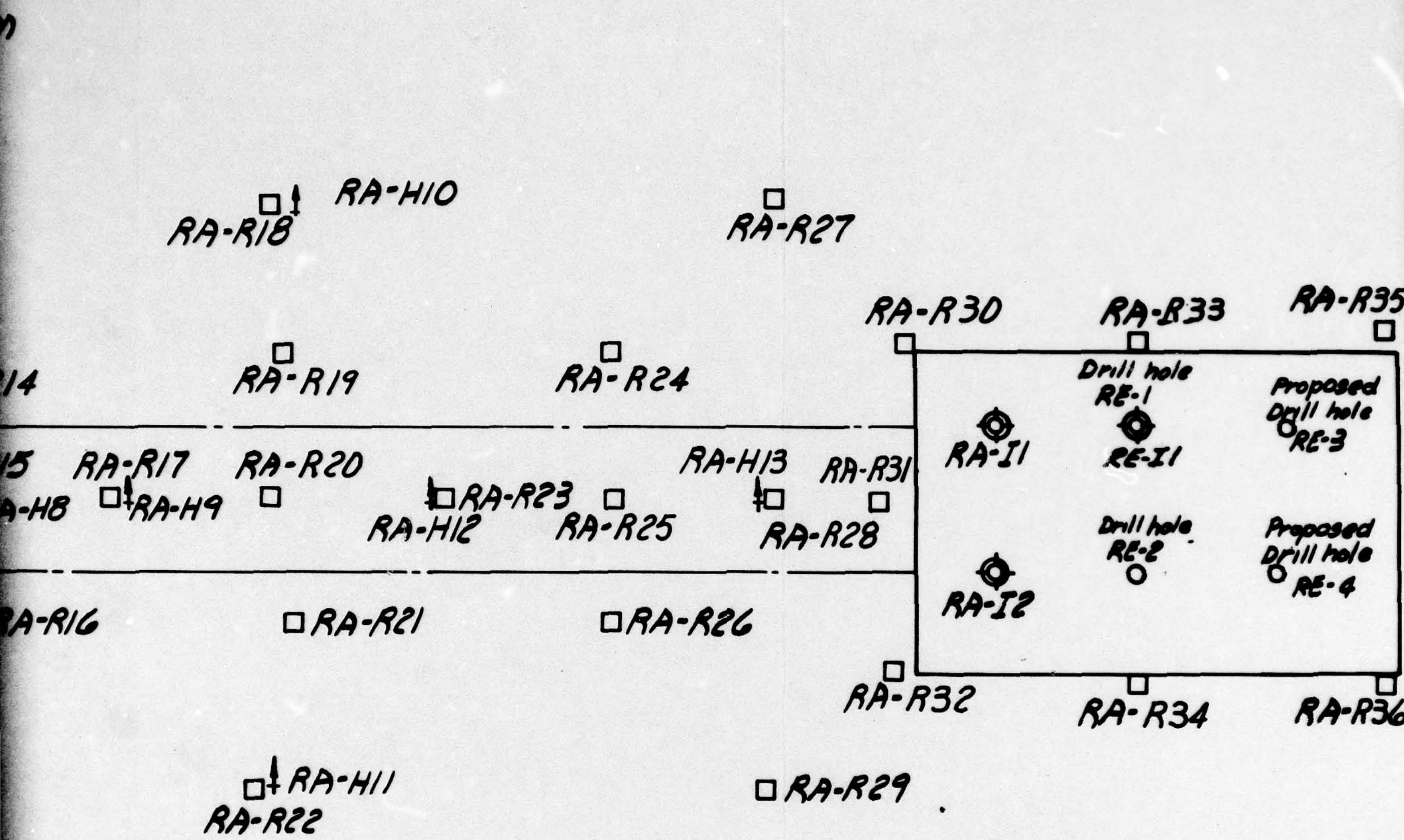
Y7C825 Phase IX

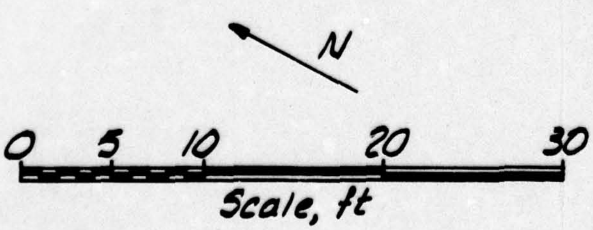
Fig. 5.4


RA-BM
RA-R4
RA-R12
RA-H6
RA-R1
RA-H1
RA-R1
RA-R5
RA-R9
RA-R14
RA
RA-R2
RA-H2
RA-3D1
RA-R6
RA-H4
RA-3D2
RA-R10
RA-H5
RA-3D3
RA-R15
RA-H8
RA-R17
RA-H9
RA
RA-R3
RA-H3
RA-R7
RA-R11
RA-R16
RA-R8
RA-H7
RA-R13
RA-R
Legend
Surface reference
3-D deformation g
Borros heave/sett
Inclinometer
Inclinometer in anch
Anchor hole
Benchmark
Axis of drillhole

1



Legend
rface reference point
D deformation gage
rros heave/settlement point
linometer
linometer in anchor hole
hor hole
chmark
n of drillhole



ROCK ANCHOR TEST PROGRAM

LAYOUT
OF GROUND INSTRUMENTATION

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT. CORPS OF ENGINEERS.
DACW43-78-C-0000

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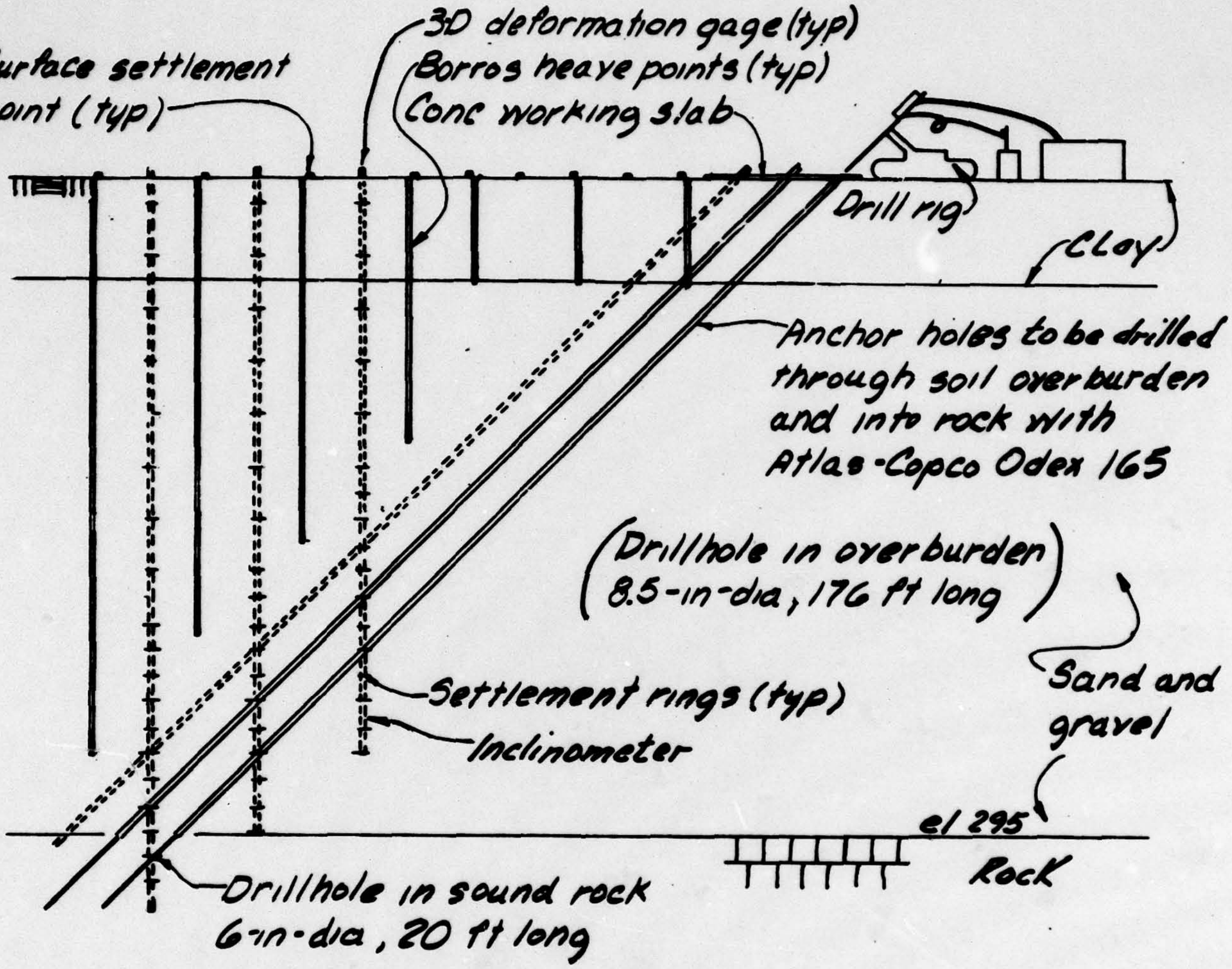
Y7C026 Phase III

Fig. 5.5

2



Note:
Inclinometer installed in completed anchor holes RA-I1, RA-I2, and RE-1

ROCK ANCHOR TEST PROGRAM

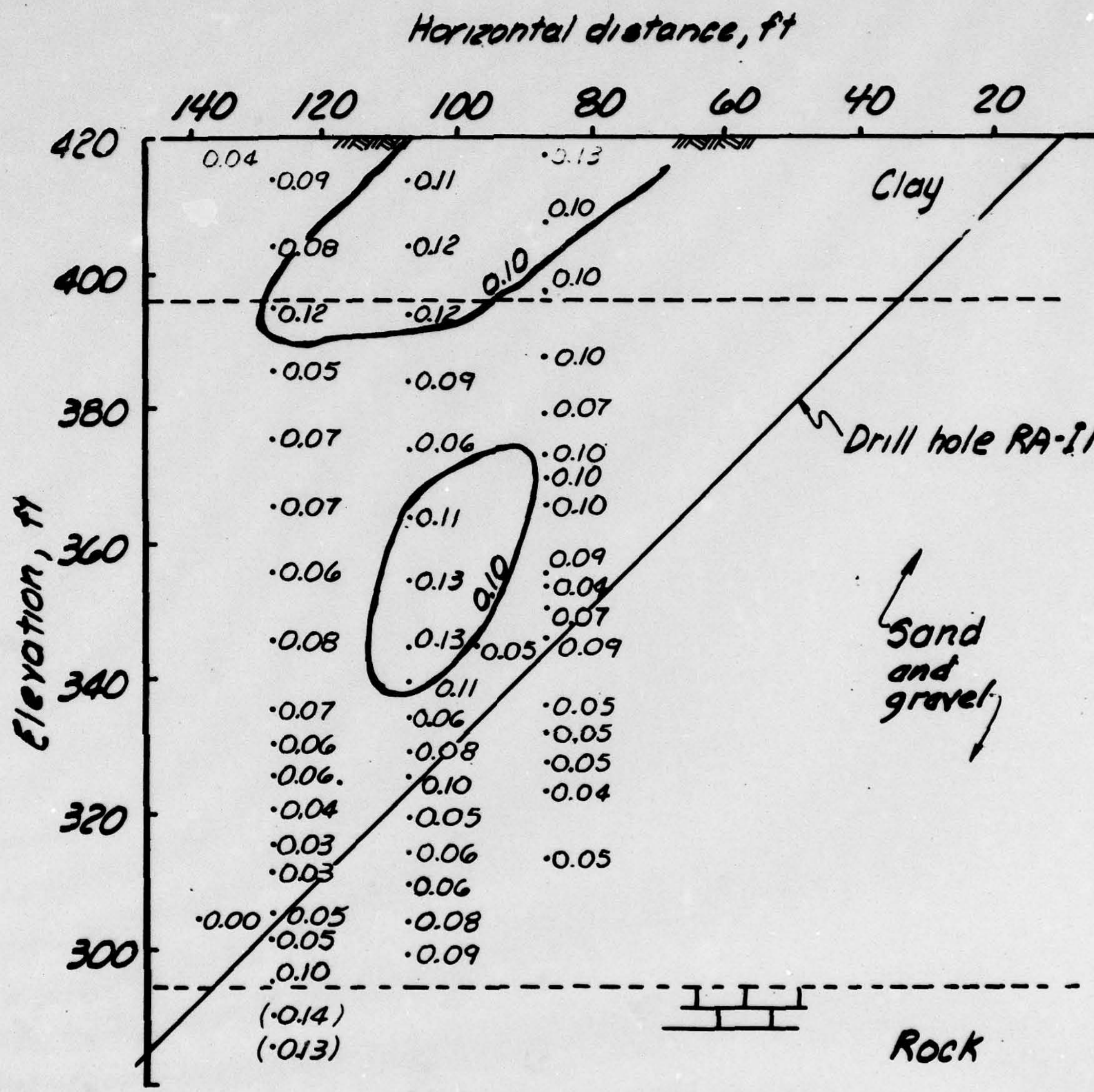
SECTION OF GROUND INSTRUMENTATION

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

Woodward-Clyde Consultants

Y7C825 Phase IX

Fig. 5.6



Note:

(1) Settlement values are in, feet

(2) Settlement data from surface reference points, borros points and sondex

ROCK ANCHOR TEST PROGRAM

GROUND SETTLEMENT
AFTER DRILLING RA-11

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-72-C-0007

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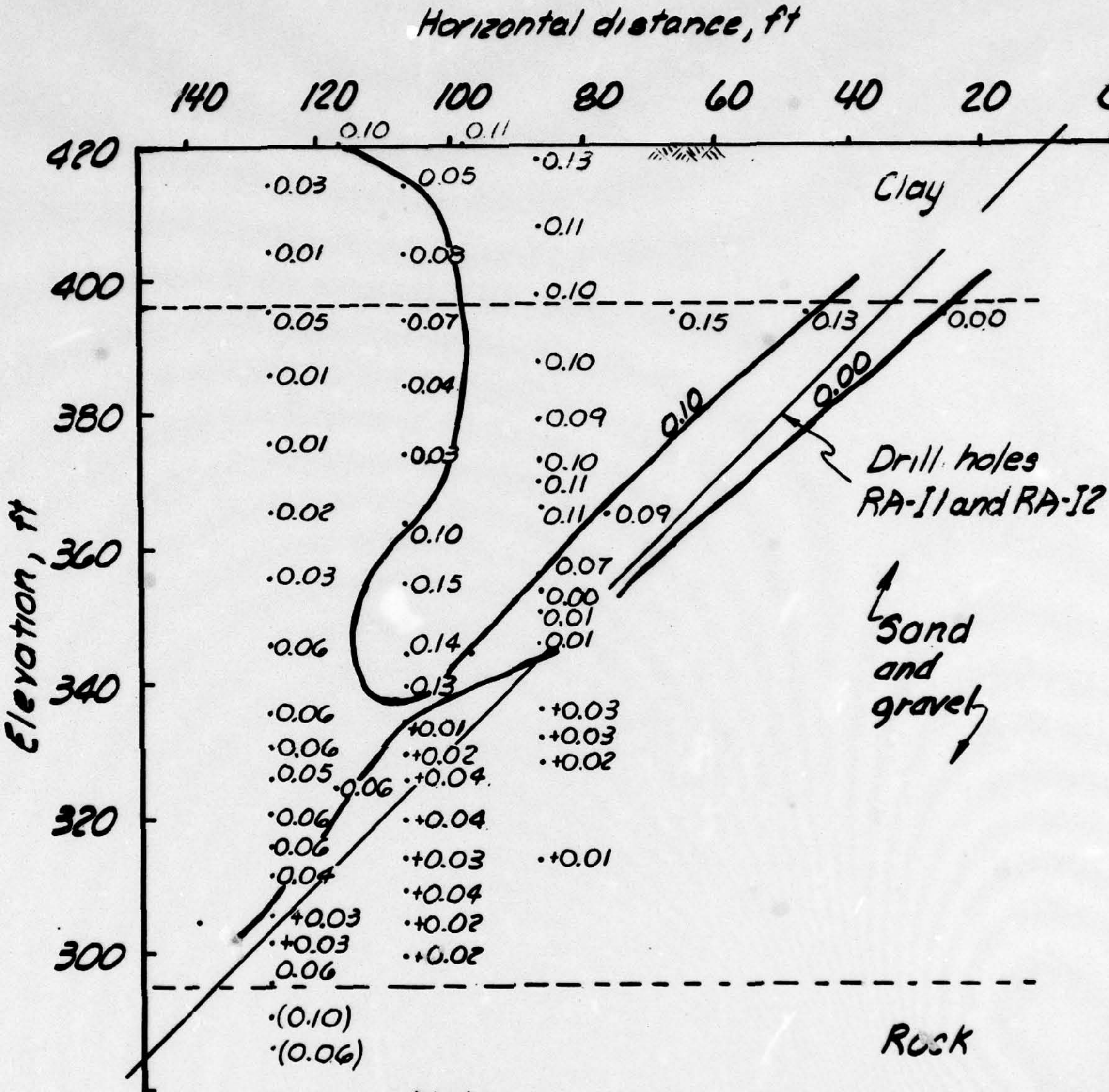
V: C026 Phase II

Fig. 5.7



Note:

(1) Settlement values are in feet

(2) Settlement data from surface reference points, borros points and sondex

ROCK ANCHOR TEST PROGRAM

GROUND SETTLEMENT AFTER DRILLING RA-12

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0008

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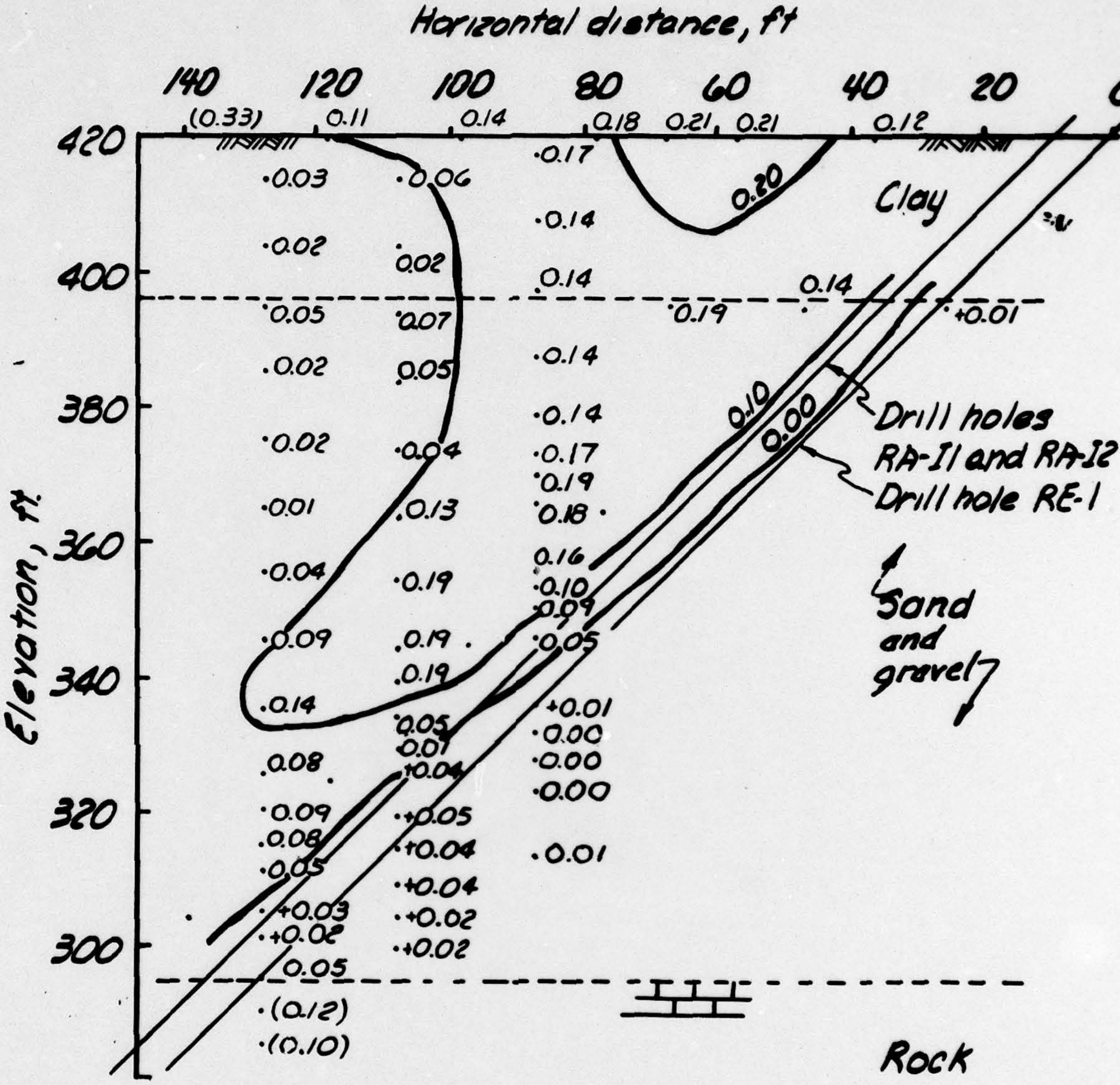
Fig.5.8

Y7C826 Phase IX



Note:
(1) Settlement values are in, feet
(2) Settlement data from surface reference points, borros points and sondex

ROCK ANCHOR TEST PROGRAM

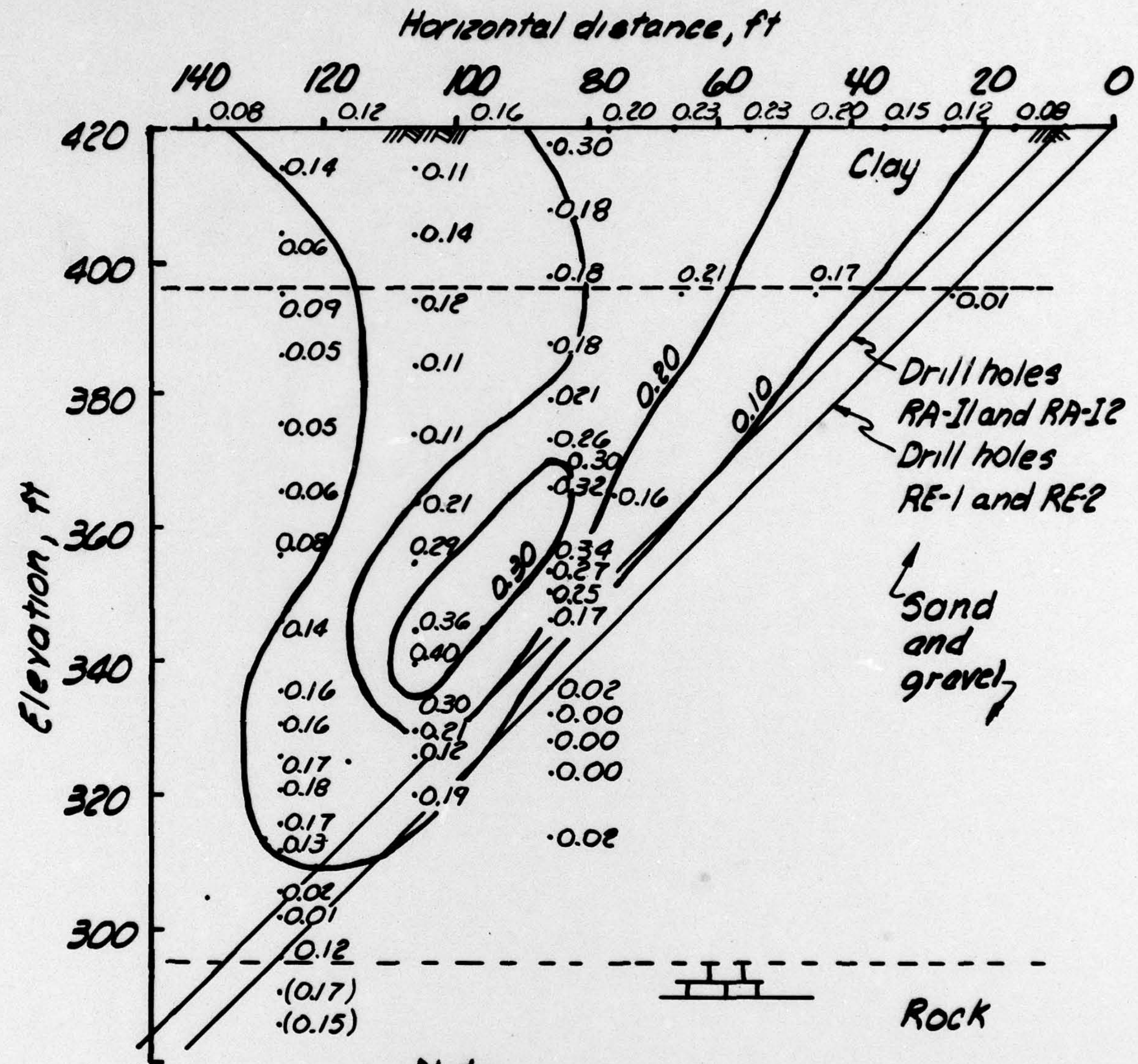
GROUND SETTLEMENT AFTER DRILLING RE-1

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0086

Woodward-Clyde Consultants

Y7C828 Phase III

Fig. 5.9



Note:

(1) Settlement values are in feet

(2) Settlement data from surface reference points, borros points and sondex

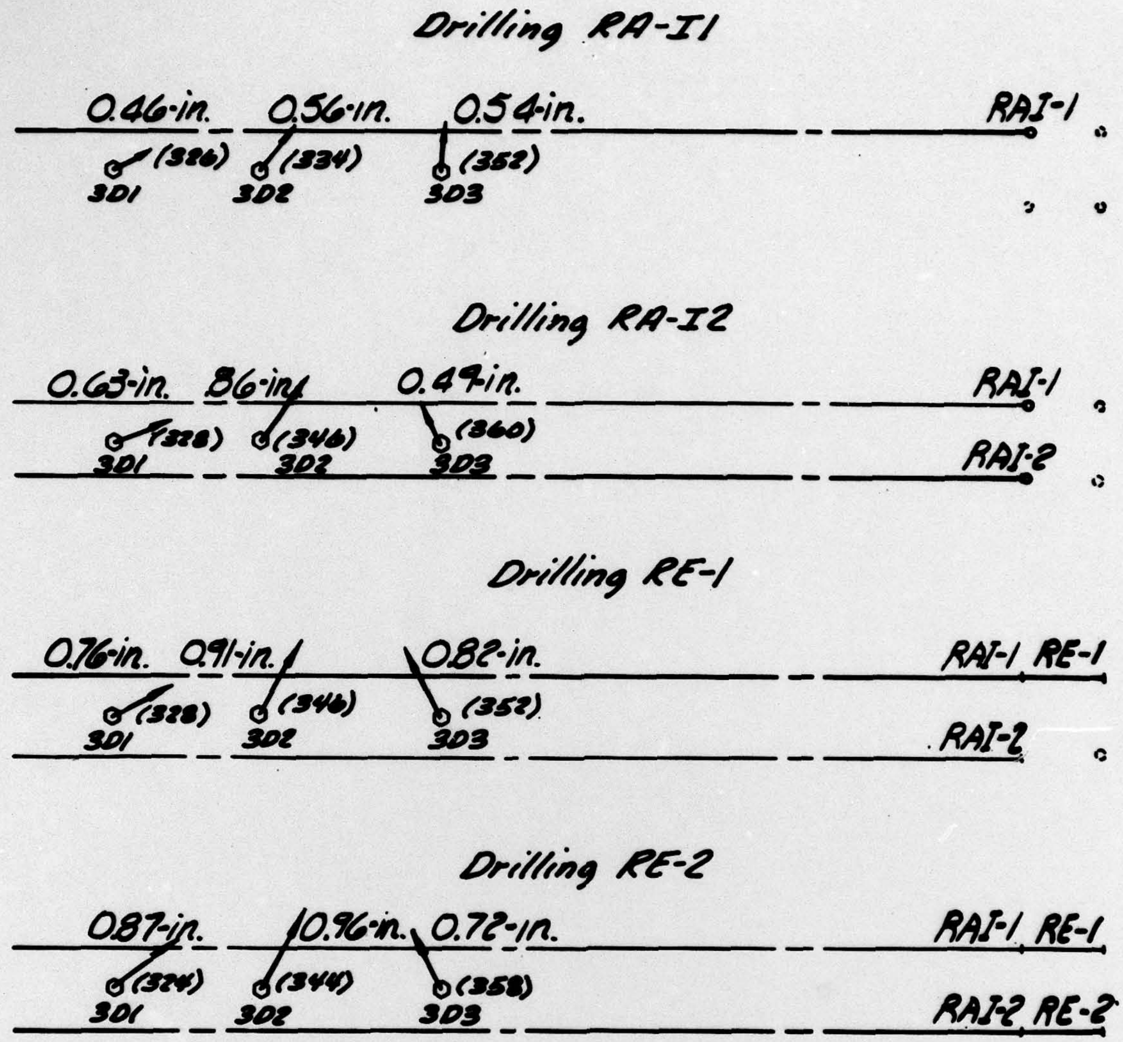
ROCK ANCHOR TEST PROGRAM

GROUND SETTLEMENT AFTER DRILLING RE-2 (FINAL)

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0066

Woodward-Clyde Consultants
Y7C026 Phase III

Fig. 5.10



Note
Max resultant displacement measured at the end of a specific anchor installation and are shown as cumulative values.

Legend
o Inclinometer location
\ Direction of max resultant displacement
0.87-in Magnitude of maximum displacement
(320) Elevation of maximum displacement
-·- Axis of drilling

ROCK ANCHOR TEST PROGRAM

MAXIMUM HORIZONTAL GROUND DISPLACEMENT DURING DRILLING EFFECTS TEST

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST. LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0066

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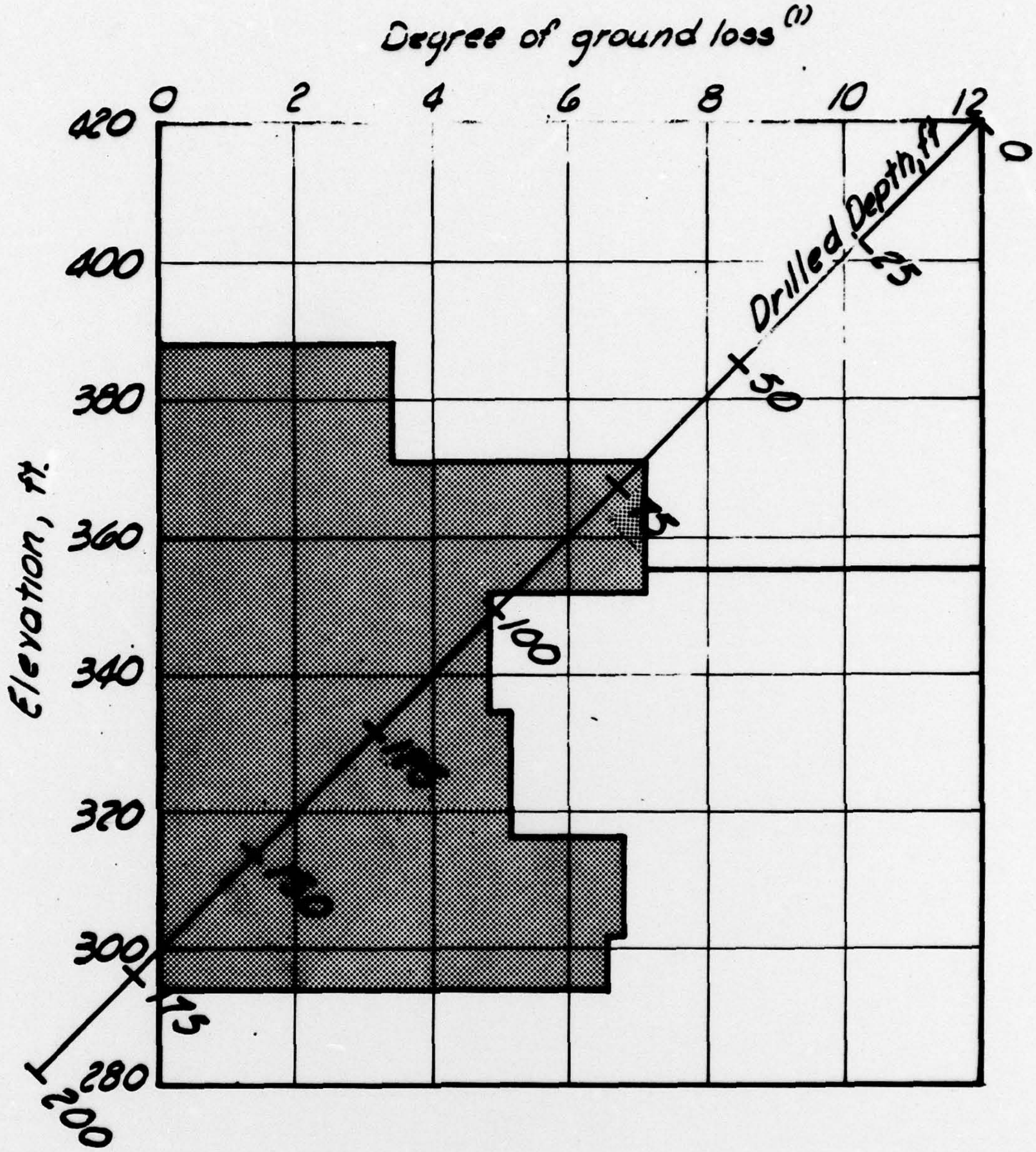
Y7C026 Phase II

Fig.5.11



Note:
(1) Degree of ground loss = $\frac{\text{Measured volume of cuttings expelled from drill hole}}{\text{Theoretical volume of drill hole}}$

ROCK ANCHOR TEST PROGRAM

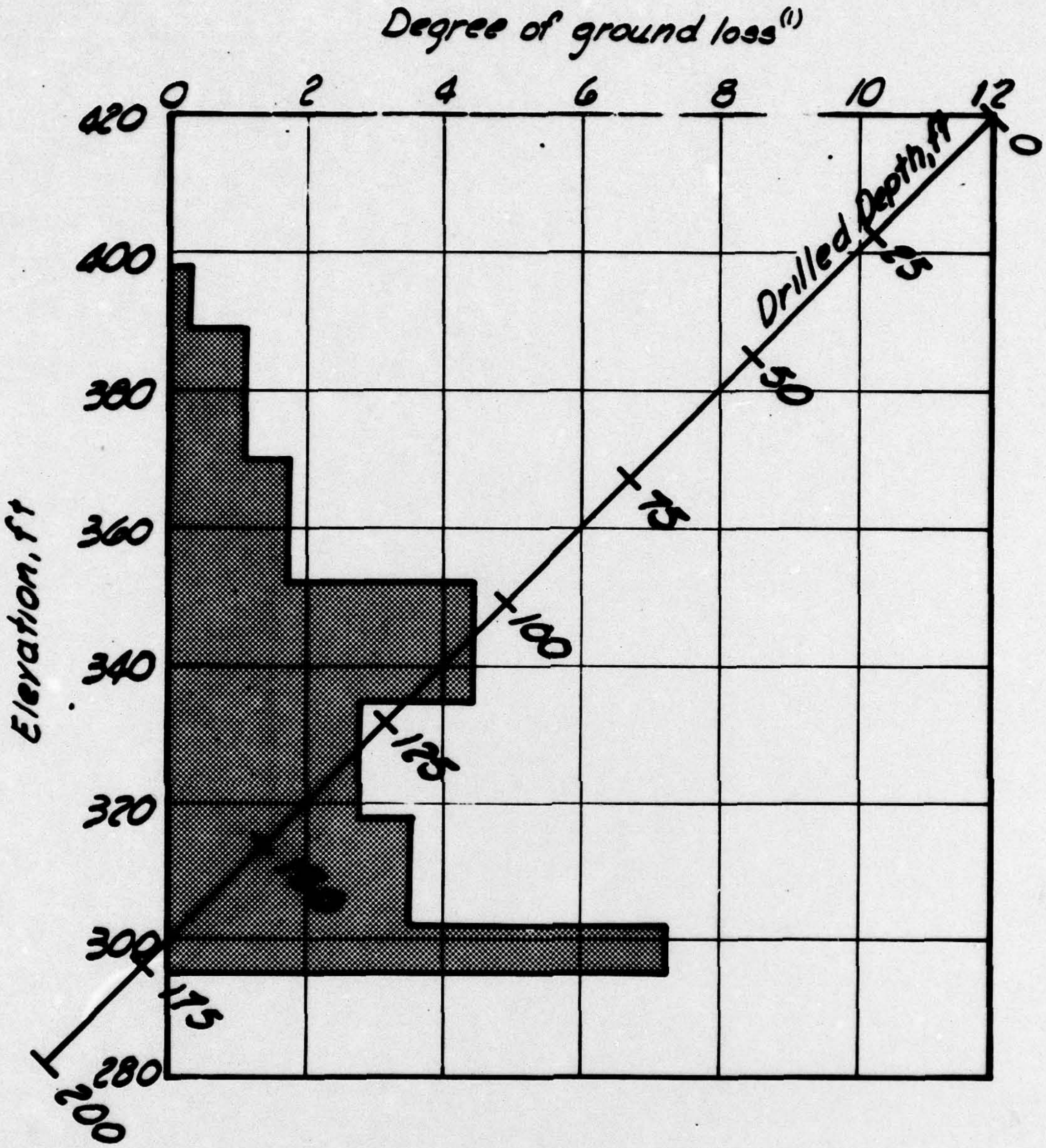
LOSS OF GROUND
DURING DRILLING RA-12

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0006

Woodward-Clyde Consultants

Fig.5.12

Y7C026 Phase II



Note:
(1) Degree of ground loss = $\frac{\text{Measured volume of cuttings expelled from drill hole}}{\text{Theoretical volume of drill hole}}$

ROCK ANCHOR TEST PROGRAM

LOSS OF GROUND
DURING DRILLING RE-1

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0066

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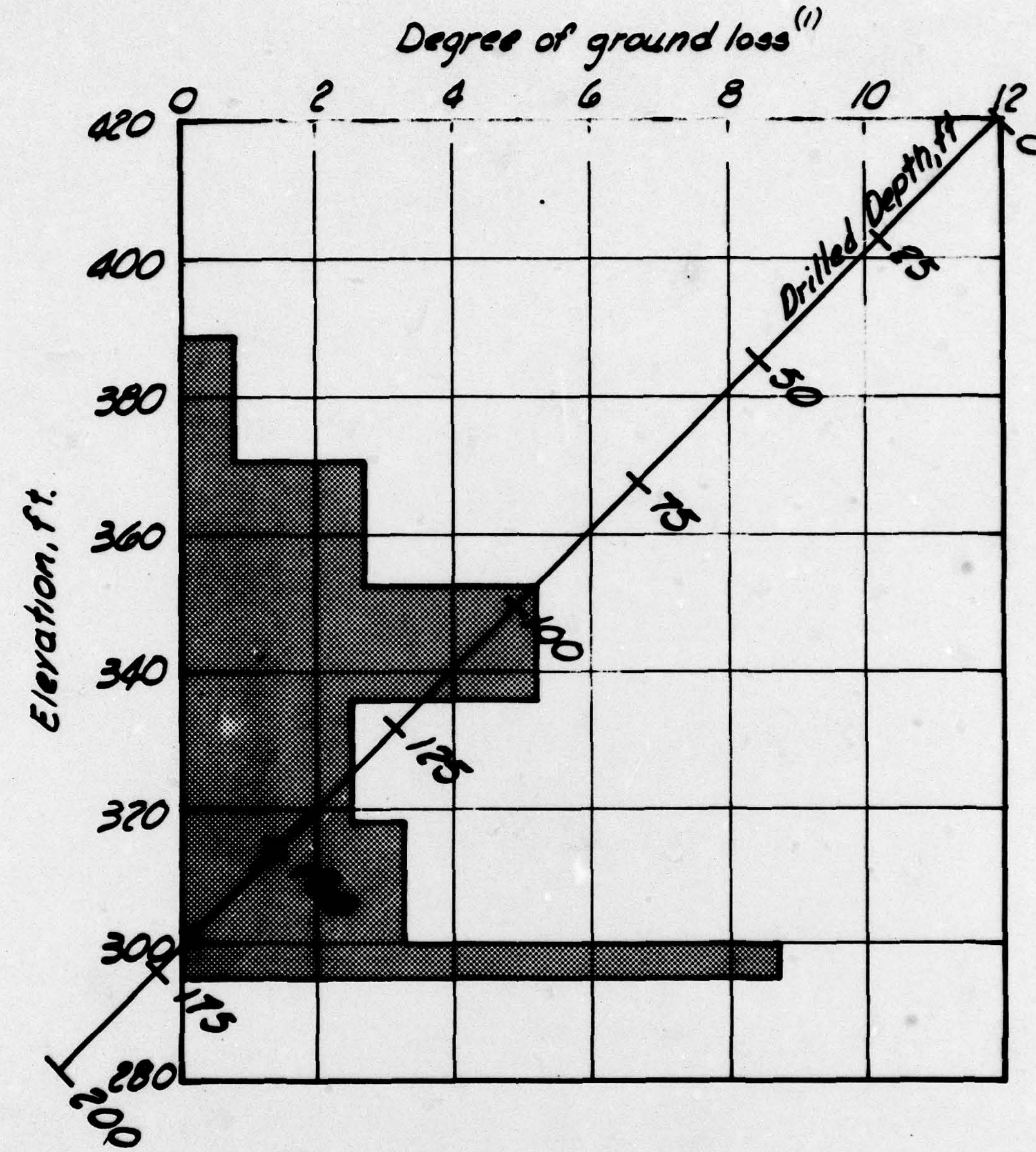
Fig. 5.12



Note:
(1) Degree of ground loss = $\frac{\text{Measured volume of cuttings expelled from drill hole}}{\text{Theoretical volume of drill hole}}$

ROCK ANCHOR TEST PROGRAM

LOSS OF GROUND DURING DRILLING RE-2

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0000

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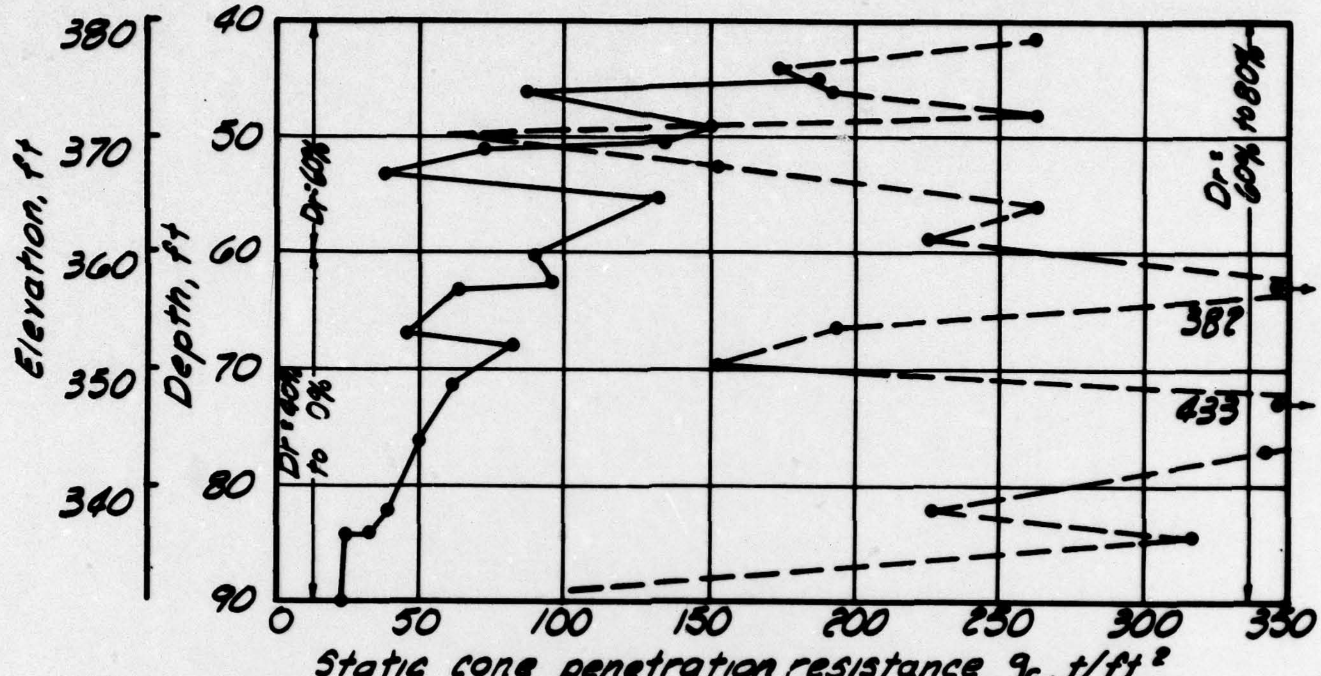
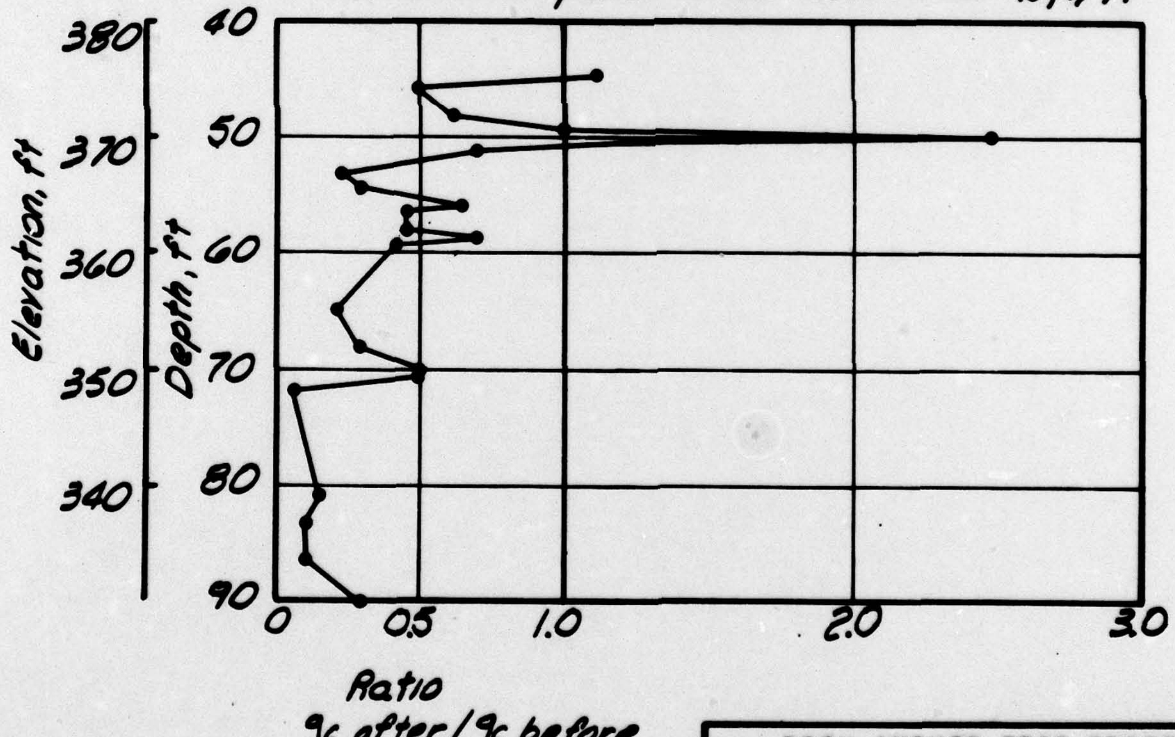
Fig. 5.14

V78088 Phase III





Legend
Before anchor installation
--- Boring DP-C1
After anchor installation
— Boring RA-C1

ROCK ANCHOR TEST PROGRAM

COMPARISON BETWEEN STATIC CONE PENETRATION RESISTANCE BEFORE AND AFTER ANCHOR INSTALLATION

FOUNDATION INVESTIGATION AND TEST PROGRAM
EXISTING LOCKS AND DAM No. 26
ST LOUIS DISTRICT, CORPS OF ENGINEERS.
DACW43-78-C-0005

Woodward-Clyde Consultants

VYC
Fig. 5.15

PHASE IV REPORT

VOLUME V

RESULTS AND INTERPRETATION OF
ROCK ANCHOR TEST PROGRAM

SECTION 6

COST INFORMATION

6 COST INFORMATION

6.1 GENERAL

This section presents an analysis of the cost of construction of rock anchors by the methods used in the test program. The costs presented are based on unit rates for materials, equipment, and labor for the St Louis area.

6.2 PRODUCTION

In the test program, an average net drilling rate of 35 ft per 10 hr shift was attained. At this rate, the average time for drilling one anchor, 200 ft long, is 6 shifts. The total average time for completion of one anchor, including all activities, is as follows:

Activity	Time
Drilling	6.0 shifts
Installation	0.5
Grouting and pulling casing	1.5
Stressing	0.5
Move to next hole	0.5
Total production rate per anchor	9.0 shifts

The construction schedule used in this cost analyses is one 10-hr shift per day and 22 working days per month.

6.3 EQUIPMENT AND LABOR

The following equipment was used for construction of the anchors in the test program. Listed with equipment are monthly rental trades and the corresponding shift rates.

Equipment	Rate Per Month	Rate Per Shift
Drill rig, Driltech D-40K	$ 16,000	$ 727
Hydraulic crane, 15-t	2,400	109
Crawler crane, 60-t	5,000	227
Welder, 250 amp	580	26
Grout mixer, 60 gal/min	920	42
Stressing jack, 250-t	800	36

The construction trades which participated in the anchor construction, and their respective rates, are as follows:

Labor	Rate Per Hour	Rate Per Shift
Operator	$ 18.30	$ 183
Oiler	16.00	160
Laborer	15.70	157
Welder	21.70	217
Ironworker	21.80	218

The above equipment and labor rates include 30 percent for contractor's overhead and profit.

6.4 MATERIALS

The 400-k design capacity anchor tendons contained 17 VSL cables. The price for one anchor, 200 ft long, including hardware, is $2,800 delivered to the site.

The quantity of cement grout in the entire length of the anchor is 75 ft^3. Using a high-strength grout with a water-cement ratio of 0.5 and appropriate additives, the price for grout materials is $350 per anchor.

The casing can be reused for an average of 5 drill holes before discarding. At this usage rate, the casing price is $590 per anchor.

The total cost of materials for one anchor is as follows:

Component	Cost
Anchor tendon	$ 2,800
Cement grout	350
Casing	590
Cost per anchor	$ 3,740

6.5 COST ANALYSIS

The cost for construction of the rock anchors is based on the production rates experienced during the testing program, the equipment and labor needed to perform the activities, and the materials expended. The total cost is calculated in the following sections for each construction activity. The cost is normalized for construction of one anchor.

6.5.1 Drilling Anchor Hole

The time required for one anchor is 6 shifts.

Item	Quantity	Rate Per Shift	Cost	
Equipment				
drill rig	1	$ 727	$4,362	
hydraulic crane	1	109	654	
welder	1	26	156	
	Equipment cost per anchor		$5,172	
Labor				
operator	2	$ 183	$2,196	
oiler	1	160	960	
laborer	3	157	2,826	
welder	1	217	1,302	
	Labor cost per anchor		$7,284	
Materials			None	
TOTAL DRILLING COST PER ANCHOR				$ 12,456

6.5.2 Installing Tendon

The time required for one anchor is 0.5 shift.

Item	Quantity	Rate Per Shift	Cost	
Equipment				
hydraulic crane	1	$ 109	$ 55	
crawler crane	1	227	114	
	Equipment cost per anchor		$ 169	
Labor				
operator	2	$ 183	$ 183	
oiler	1	160	80	
ironworker	5	218	545	
	Labor cost per anchor		$ 808	
Material				
anchor tendon	1	$2,800	$2,800	
TOTAL INSTALLATION COST PER ANCHOR				$ 3,777

6.5.3 Grouting and Pulling Casing

The time required for one anchor is 1.5 shifts.

Item	Quantity	Rate Per Shift	Cost
Equipment			
drill rig	1	$ 727	$1,091
hydraulic crane	1	109	164
welder	1	26	39
grout mixer	1	42	63
	Equipment cost per anchor		$1,357
Labor			
operator	2	$ 183	$ 549
oiler	1	160	240
laborer	6	157	1,413
welder	1	217	326
	Labor cost per anchor		$2,528
Material			
cement grout			$ 350
casing			590
	Material cost per anchor		$ 940

TOTAL GROUTING AND PULLING CASING COST PER ANCHOR — $ 4,825

6.5.4 Stressing Anchor

The time required for one anchor is 0.5 shift.

Item	Quantity	Rate Per Shift	Cost
Equipment			
stressing jack	1	$ 36	$ 18
Labor			
operator	1	$ 183	$ 92
ironworker	2	218	218
	Labor cost per anchor		310
Material			None

TOTAL STRESSING COST PER ANCHOR — $ 328

6.5.5 Moving to Next Hole

The time required for one anchor is 0.5 shift.

Item	Quantity	Rate Per Shift	Cost	
Equipment				
drill rig	1	$ 727	$ 364	
hydraulic crane	1	109	55	
	Equipment cost per anchor		419	
Labor				
operator	2	$ 183	$ 183	
oiler	1	160	80	
laborer	3	157	236	
	Labor cost per anchor		$ 499	
Material			None	
TOTAL MOVING COST PER ANCHOR				$ 918

6.5.6 Cost Summary

The total cost for construction of one anchor is as follows:

Activity	Cost
Drilling anchor hole	$12,456
Installing tendon	3,777
Grouting and pulling casing	4,825
Stressing anchor	328
Moving to next hole	918
GRAND TOTAL PER ANCHOR	$22,304

This cost reflects the conditions experienced during the test program. Although production rates were modified for the cost analysis to account for atypical delays inherent in the tests, the total cost per anchor may still represent an upperbound value. It is likely that this cost could be decreased by some unknown amount and be more representative of large scale production work.

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VOLUME V

RESULTS AND INTERPRETATION OF ROCK ANCHOR TEST PROGRAM

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