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OVERVIEW OF FOUNDATION INVESTIGATION AND TEST PROGRAM. VOLUME I--ETC(U)
JUL 79 Y LACROIX , J PEREZ , E L FIELDHAMMER DACW43-78-C-0005

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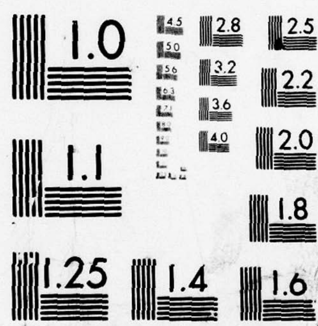

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PHASE IV REPORT

VOLUME I

OVERVIEW OF FOUNDATION INVESTIGATION AND 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

With the assistance of

Booker Associates, Inc.
St Louis, Missouri

15 July 1979

Contract No. DACW43-78-C-0005

Y7C825

PHASE IV REPORT

VOLUME I

6

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Volume I.

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REPORT DOCUMENTATION PAGE

READ INSTRUCTIONS BEFORE COMPLETING FORM

1. REPORT NUMBER	2. GOVT ACCESSION NO. / 3. RECIPIENT'S CATALOG NUMBER
4. TITLE (and Subtitle) FOUNDATION INVESTIGATION AND TEST PROGRAM, LOCKS AND DAM 26, MISSISSIPPI RIVER, ALTON, ILLINOIS Volume I	5. TYPE OF REPORT & PERIOD COVERED Final report
	6. PERFORMING ORG. REPORT NUMBER
7. AUTHOR(s) Yves Lacroix, Jean-Yves Perez, Eugene L. Fieldhammer	8. CONTRACT OR GRANT NUMBER(s) DACW43-78-C-0005 New
9. PERFORMING ORGANIZATION NAME AND ADDRESS Woodward-Clyde Consultants, 11 East Adams Street, Chicago, IL 60603	10. PROGRAM ELEMENT, PROJECT, TASK AREA & WORK UNIT NUMBERS
11. CONTROLLING OFFICE NAME AND ADDRESS U. S. Army Engineer District, St. Louis, 210 North 12th Street, St. Louis, MO 63101	12. REPORT DATE June 1979
	13. NUMBER OF PAGES 37
14. MONITORING AGENCY NAME & ADDRESS(if different from Controlling Office)	15. SECURITY CLASS. (of this report) Unclassified
	15a. DECLASSIFICATION/DOWNGRADING SCHEDULE

16. DISTRIBUTION STATEMENT (of this Report)

Approved for public release; distribution unlimited.

17. DISTRIBUTION STATEMENT (of the abstract entered in Block 20, if different from Report)

18. SUPPLEMENTARY NOTES

- Vol I Overview of Foundation Investigation and Test Program
- Vol II Results and Interpretation of Chemical Grouting Test Program
- 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
- Vol VA Appendices A through E, Results and Interpretation of Rock Anchor Test Program

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

Lock and Dam No. 26, Mississippi River	Timber piles
Chemical grout test	Alluvial sands
Rock anchor test	Benoto method
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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Con 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.

Monolith No.	Timber Pile Configuration	Load* Level	Grouted	Total No. Of Prototype Piles Driven	Measured Displacement in. Lateral	Average Vertical
M1	3 x 4	High	No	13	1.5	1.3
M2	2 x 4	High	No	14	1.9	1.1
M3	2 x 4	High	Yes	8	1.1	0.6
M5	2 x 4	Low	No	7	0.7	0.4
M6	Single	High	No	14	2.0	1.5
M7	Single	High	Yes	8	2.3	1.0

* High-load level was 30 t/pile axially and 6 t/pile laterally; low-load level was 15 t/pile axially and 4 t/pile laterally

The following conclusions were drawn from the test results:

(1) significant cumulative displacements of the monolith (that is, displacements larger than the accuracy of the measurements) were measured when prototype piles were driven at a distance of 50 ft or less from the loaded monoliths;

(2) the cumulative displacement of a given monolith increased with the number of prototype piles driven;

(3) when as many as four piles were successively driven at the same distance from a given monolith, incremental displacement due to each pile generally did not show a stabilizing or decreasing trend;

(4) grouting did not significantly reduce the displacement of the monoliths;

(5) the horizontal displacement of the monoliths at low-load level was about 50 percent less than the displacement at high-load level; the settlement at low-load level was about 30 percent less than at high-load level;

(6) the single timber piles and the pile groups were equally affected by prototype pile driving; and

(7) the vibratory hammer induced much larger displacement of one monolith than the impact hammers (air or diesel); the impact hammers produced similar results.

5.4.3 Monolith Load Testing

Static lateral load at failure in ungrouted soil ranged from 14.3 to 15 t/pile for the single timber pile and the two pile groups tested. It was 14.8 t/pile for the timber pile group and 20 t for the single pile in postgrouted soil.

FOREWORD

"The objective of the investigation and test program is to provide adequate information on present engineering uncertainties concerning alternate rehabilitation schemes in order to assist the Administration in reaching definitive conclusions about their engineering feasibility, reliability, safety, and relative cost"*

A Foundation Investigation and Test Program was designed and implemented starting in November 1977. Although a replacement project for Locks and Dam No. 26 was authorized by Public Law 95-502 on 21 October 1978, the program was allowed to continue at the direction of the Corps of Engineers. The data obtained from the program will not only provide valuable technical information concerning rehabilitation techniques for existing pile-founded locks and dams, but will also provide state-of-the-art foundation design knowledge to the engineering profession.

The results of the Foundation Investigation and Test Program presented in this report confirmed that many of the questions raised as to the feasibility, reliability, safety, and cost of proposed rehabilitation construction techniques were well founded. The effects of such construction techniques were demonstrated.

Adjacent pile driving induced deep-seated, large displacement of loaded test structures. The feasibility of injecting low- and high-strength chemical grouts into the alluvial soil was proven; low-strength grout, however, was not adequate to reduce signficantly the test structure displacement. Soil grouted with high-strength grout was judged too hard to be penetrated by piles driven using conventional pile driving methods. In any case, because the observed test structure displacement induced by adjacent pile driving was deep-seated, soil stabilization by grouting or other means would be required to large depth.

It was possible to drill test piles without objectionable effects on the surrounding ground; production rate, however, was very slow. Installation of test rock anchors induced large ground deformations.

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* *Excerpt from the Memorandum of Understanding Between the Department of the Army and the Department of Transportation on an Engineering Investigation and Field Test Program, Locks and Dam No. 26, Alton, Illinois, 1 July 1977*

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1 INTRODUCTION

1.1 BACKGROUND

The existing Locks and Dam No. 26 on the Mississippi River near Alton, Illinois were constructed between 1934 and 1938. The structures were reported to have deteriorated to a point at which their continued satisfactory performance was questioned. In recent years, a number of repair and rehabilitation schemes have been proposed to extend the life of the structures for fifty years. Several of the proposed schemes involve special overwater construction techniques that had not been previously used under conditions such as exist at Locks and Dam No. 26. In early 1977, the US Department of the Army, Corps of Engineers and the US Department of Transportation determined that a field investigation and test program was necessary to investigate the feasibility, safety, and reliability of the proposed rehabilitation techniques. The purpose of the program was to provide comprehensive technical information that could be used by the Government to evaluate appropriate rehabilitation schemes involving no major unwatering of the existing structures. The plan was to obtain the information by the end of 1978.

A Foundation Investigation and Test Program was designed and implemented starting in November 1977. Although a replacement project for Locks and Dam No. 26 was authorized by Public Law 95-502 on 21 October 1978, the program was allowed to continue at the direction of the Corps of Engineers. The data obtained from the program will not only provide valuable technical information concerning rehabilitation techniques for existing pile-founded locks and dams, but will also provide state-of-the-art foundation design knowledge to the engineering profession.

The foundation investigation and test program described in this report was designed so that the test results and their interpretation would be transferable to Locks and Dam No. 26. However, the application of the findings of the program to engineering the rehabilitation of the existing structures was not part of this study.

1.2 GENERAL SCOPE OF PROGRAM

The foundation investigation and test program addressed two basic avenues of rehabilitation:

(1) improvement of the foundation soil characteristics by chemical grouting; and

(2) improvement of the stability of the existing structures by construction of additional support elements (driven or drilled-in piles and anchors).

Various tests were considered, selected, designed, and implemented. The process by which the individual tests were selected and designed is discussed in this Phase IV, Volume I Report. The test results and conclusions drawn from the results are also summarized in Volume I. The detailed design, performance, and interpretation of the tests is presented in the following other volumes:

Volume II:	Results and Interpretation of Chemical Grouting Test Program
Volume IIA:	Appendices A through G, Results and Interpretation of Chemical Grouting Test Program
Volume III:	Results and Interpretation of Pile Driving Effects Test Program
Volume IIIA:	Appendices H through T, Results and Interpretation of Pile Driving Effects Test Program
Volume IV:	Results and Interpretation of Drilled-In Pile Test Program
Volume IVA:	Appendices A through D, Results and Interpretation of Drilled-In Pile Test Program
Volume V:	Results and Interpretation of Rock Anchor Test Program
Volume VA:	Appendices A through E, Results and Interpretation of Rock Anchor Test Program

1.3 DESIGN AND CONSTRUCTION TEAM

1.3.1 Government Team

Two US Government agencies were responsible for the program: the Department of the Army, Corps of Engineers and the Department of Transportation. The Department of Transportation was represented by a Special Task Force. The Corps of Engineers was represented by the Office of the Chief of Engineers, the Lower Mississippi Valley Division, and the St Louis District. The St Louis District had direct supervision of the program.

1.3.2 Engineering Team

The assignment to formulate, plan, develop, design, supervise, and evaluate the program was given in November 1977 to Woodward-Clyde Consultants (WCC), Chicago, assisted by Booker Associates, Inc, St Louis under contract No. DACW43-78-C-0005. The engineering team was headed by Yves Lacroix (WCC), project director and Jean-Yves Perez (WCC), project manager. Other team members included Eugene L. Fieldhammer (Booker); William L. Durbin (WCC), project coordinator; Ralph A. Fasano (WCC), test engineer and rock anchor tests task leader; Stanley F. Gizienski (WCC), drilled-in pile tests task leader; Jin H. Kim (WCC), chemical grouting tests task leader; D. Michael Holloway (WCC), pile driving effects tests task leader; and Yoshiharu Moriwaki (WCC) and John B. Stevens (WCC), pile driving effects tests assistant task leaders.

WCC and Booker retained a group of consultants consisting of Claude Caron, consulting engineer; Prof M. T. Davisson, University of Illinois; Prof T. William Lambe, Massachusetts Institute of Technology; Prof Lymon C. Reese, University of Texas; Prof M. O'Neill, Univeristy of Houston; Prof Robert J. Hoyle, Washington State University; Robert E. White, consulting engineer; and Robert Y. Busch, consulting engineer.

As the test program progressed, two other organizations were added to the engineering team: Raymond International Builders, Inc, St Louis (drilling for subsurface exploration and installation of ground instrumentation); and Shannon & Wilson, Inc, Seattle (assistance in selection and installation of instrumentation).

1.3.3 Construction Team

The grout injection of the chemical grouting test program was done by a joint venture of Raymond International Builders, Inc, Pennsauken, New Jersey, and Soletanche and Rodio, Inc, McLean, Virginia. The joint venture was a subcontractor to WCC. J. S. Alberici Construction Company, Inc, St Louis constructed and operated the various dewatering systems under contract DACW43-78-C-0094. Luhr Bros, Inc, St Louis, was the contractor for site preparation and chemical grouting test area excavation under contract DACW43-78-C-0093. Construction activities during the pile driving effects, drilled-in pile, and rock anchor test programs were performed by ICOS Corporation of America, New York, under contract DACW43-78-C-0159.

2 OBJECTIVES AND SCOPE OF TESTS

2.1 GENERAL

The objectives of the foundation investigation and test program were initially formulated in Phase I (November and December 1977). As the design of the program evolved in Phase II (January to May 1978), the objectives of the tests were reassessed and the scope of the program was modified to reflect progressive conceptual changes. The changes stemmed from consideration of relative importance of the various proposed tests, relative benefits of the expected test results, present knowledge, budget, and schedule.

The six test programs identified in Phase I were ranked in order of decreasing importance on the basis of the usefulness and interest of the potential test results for selection and engineering of rehabilitation schemes. The objectives of each test program were also ranked. As the design of the tests evolved, certain tests or portions of tests were eliminated either because they were not entirely relevant to the scope of the overall program, or because they were of low priority and could not be accomodated within the fixed budget and schedule of the overall program.

As the field tests progressed, some additional modifications were implemented which resulted in further deletions or increases in testing. The deletions were effected because sufficient information had been obtained from the portions of the tests already completed and it was deemed more beneficial to transfer the unspent effort to other portions of the tests requiring more testing than originally planned. The evolution of the scope of each test program component from initial formulation to actual implementation is discussed in the following sections.

2.2 CHEMICAL GROUTING TEST PROGRAM

The purpose and objectives of the chemical grouting test program remained essentially unchanged from initial formulation. The purpose of the program was to assess the feasibility, applicability, and effectiveness of injecting silicate-based grouts into Mississippi River alluvial sand. Both low-strength and high-strength grouts were used for the tests. The primary intent of low-strength grouts was to decrease potential displacement and rearrangement of sand grains, and, thus, increase the stability of the sand when subjected to vibrations induced by construction activities. The secondary intent was to moderately increase the strength of the sands, which would significantly augment the lateral resistance of piles. The tertiary intent was to increase resistance to erosion and to reduce the permeability of the sand.

The primary intent of high-strength grouts was to increase substantially the bearing capacity and the stability of the sand. The increased bearing capacity had to be permanent. The secondary intent was to increase the resistance to erosion and reduce the permeability of the sand.

The objectives of the chemical grouting test program were:

(1) to investigate the technical feasibility of satisfactorily grouting the sand without inducing objectionable heave, lateral movement, and excess pore pressure;

(2) to compare various grouts and provide a basis for selection of chemical grouts that will produce the desired grouted soil properties;

(3) to compare two common grouting methods, the open-bottom pipe and the sleeve-pipe methods;

(4) to establish an optimum grout-hole spacing by comparing the effects of two spacings, 4.2 ft and 6 ft, in achieving the desired grout penetration and uniformity;

(5) to provide bases for establishing criteria for acceptable and optimum grout quantities, grouting pressures, and optimum and maximum grout flow rates; and

(6) to provide cost elements for future estimating purposes.

The scope of the chemical grouting test program evolved from a dual series of tests (groutability tests and cutoff tests) to a single series of groutability tests. The cutoff tests, which would have entailed constructing two grouted soil rings from ground surface to bedrock, and conducting pumping tests to measure the permeability of the grouted soil, were eliminated early in the design stage. This testing scheme would yield only limited information and would have been excessively expensive. It was felt that permeability tests made in boreholes would be sufficient to evaluate the effects of grouting on soil permeability.

The actual test program involved testing four different grouting methods (open-bottom pipes and three variations of the sleeve-pipe method), eight grout types and two grout hole spacing. A total of 123,500 gal of grout (primarily silicate grout) was injected through 74 grout holes to grout approximately 1440 yd^3 of alluvial sand. The tests are described in detail in Volume II.

2.3 GROUT-PILE PERFORMANCE TEST PROGRAM

The grout-pile performance test program was initially intended to assess the effects of pile driving on the properties of grouted soil through which piles are driven; to assess whether or not piles can be effectively driven through grouted soil without adverse consequences; and to evaluate the effects of grouting before and after pile driving on the axial and lateral load capacity of piles.

Until late in the design stage (March 1978), the proposed scope of the grout-pile performance test was extensive. It involved driving numerous pipe and H piles and two timber piles; grouting two areas with low- and high-strength grouts; dewatering and excavating the test area; load testing several piles both axially (tension) and laterally; and making a long-term plate load test in grouted soil. The program was also to serve as a supplementary test program for additional chemical grouting observations and evaluations.

The usefulness of the proposed grout-pile performance test program was reassessed at the end of the design stage in the light of budgetary constraints. It was realized that most of the initial objectives could be met even if the scope of the program were to be significantly reduced. It was decided to reduce the number of proposed test piles, eliminate the axial (tension) load tests and incorporate the reduced program to the pile driving effects test program, thereby economizing on instrumentation and grouting costs by sharing some of these items between the two test programs.

2.4 PILE DRIVING EFFECTS TEST PROGRAM

The purpose and objectives of the pile driving effects test program remained relatively unchanged from initial formulation. Reduced portions of the grout-pile performance test program were incorporated into this program at the end of the design stage, as discussed in the preceeding section. The purpose of the pile driving effects test program was to provide information regarding the magnitude of the effects of nearby pile driving on vertically and horizontally loaded pile-supported structures. Adjunctly, the test program was also designed to evaluate steel pile performance during driving and lateral loading in grouted and ungrouted soil.

The primary objectives of the test program were:

(1) to assess whether or not test structures founded on vertical timber piles and subjected to vertical and horizontal loads develop permanent displacements in response to nearby pile driving; and

(2) to assess whether or not chemical grouting of the soil surrounding the timber pile foundation of the test structures prevents or mitigates such displacements.

The secondary objectives of the test program were:

(3) to measure the response of the test structures, their timber pile foundations, and the surrounding soil mass to nearby prototype pile driving to help explain the mechanisms governing pile driving effects, and establish relationships between pile driving parameters and observed effects;

(4) to measure ultimate axial and lateral load capacities of single timber piles and timber pile groups in ungrouted and grouted soil; and

(5) to assess the effects of grouting on driving resistance and lateral load capacity of steel piles.

The scope of the pile driving effects test program evolved from construction and testing of two or more timber piles clusters of 9 to 15 piles each, to construction and testing of six test structures (one founded on 12 timber piles, three on 8 timber piles each, and two on single timber piles). Actually, a seventh test structure, founded on 8 timber piles, was designed and partially constructed. Plans for testing this structure were cancelled during field work because the objectives associated with its testing were of lowest priority among the objectives of the pile driving effects tests, and because the testing would have required additional time out of the already extended schedule.

Before the start of the field tests, the scope of the program was slightly expanded to accomodate axial load tests to failure of an 8-timber pile group and of a single timber pile. The additional tests were made at the request of the Department of Transportation for general research purpose.

2.5 DRILLED-IN PILE TEST PROGRAM

The purpose and objectives of the drilled-in pile test program were reduced from the intial formulation. The program was initially intended to assess the feasibility of constructing large-diameter, vertical and batter piles bearing into rock by drilling techniques (as opposed to driving); to measure the load-carrying capacity of such piles; and to assess the effects of pile drilling operations on the surrounding soil mass. Early in the design stage, the proposed load testing of drilled-in piles was cancelled. The concern from which the drilled-in pile tests stemmed was more related to construction feasibility and disturbance caused by construction, than to load-carrying capacity.

The purpose of the drilled-in pile test program, as actually performed, was to assess the feasibility of constructing relatively long, high-capacity batter piles by a drilling method, through submerged alluvial and glacial soil typically found in the Mississippi River Valley.

The objectives of the drilled-in pile test program were to assess:

(1) the feasibility of constructing 20-in.-dia, steel cased piles at a 4 (vert) to 1 (hor) batter through submerged sand and gravel and into rock, using drilling methods;

(2) whether or not pile drilling has an adverse effect on the surrounding soil mass, such as loss of ground and ground deformation;

(3) the feasibility of construction drilled-in piles at angles of batter flatter than 4 to 1; and

(4) to provide cost elements for future estimating purposes.

The third objective was met indirectly; the construction of drilled-in piles at angles of batter flatter than 4 to 1 was not attempted, because field observations during the first portion of the program indicated that it would not be economically feasible to do so. Although it was not a stated objective of the program, the performance of the construction equipment used to install the test drilled-in piles was evaluated and is reported.

2.6 ROCK ANCHOR TEST PROGRAM

The purpose and objectives of the rock anchor test program remained unchanged from initial formulation. 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 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 capacity of 400 k;

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

(4) to provide cost elements for future estimating purposes.

The scope of the rock anchor test program which was to install and load test four anchors and to drill four test drill holes, was slightly reduced during testing. Only three anchors were installed and load tested. The fourth one was cancelled. Sufficient data had been gathered on load performance of anchors to warrant this cancellation. The cancellation permitted proceeding with the drilling effects portion of the tests which was substantially delayed at that time. Although it was not a stated objective of the program, the performance of the construction equipment used to install the test anchors was evaluated and is reported.

2.7 EARTH ANCHOR TEST PROGRAM

The earth anchor test program was cancelled in its entirety at the end of the design stage. The program was initially intended to assess the feasibility of constructing permanent earth anchors to resist cyclic load, and to measure the load-carrying capacity of such anchors.

The earth anchor test program was ranked last on the test priority list. The consensus was that elimination of the test earth anchor program was opportune and would allow better allocation of budget between the remaining tests.

3 APPROACH TO TEST PROGRAM

3.1 PHASES OF WORK

The foundation investigation and test program was undertaken in four phases:

(1) Phase I (November-December 1977) consisted of the formulation of the plan for the overall program;

(2) Phase II (December 1977-May 1978) was the design phase, culminating in the preparation and submission of a detailed plan for the program. Departing from the initial schedule, plans and specifications for test site preparation and dewatering were prepared as part of Phase II work. This had to be done to allow these preliminary activities to be initiated in May 1978 ahead of the general testing contract work for the latter to remain within the proposed schedule;

(3) Phase III (February 1978-June 1978) consisted of the preparation of plans, specifications, and contract documents for the construction of test facilities and all construction associated with the tests. In Phase III, the instrumentation was designed and purchased by Woodward-Clyde Consultants (WCC) on the behalf of the Government. This was done because many instrumentation items required long lead-time for delivery, and because the timber piles instrumentation was expected to require several weeks to accomplish; it was, therefore, impossible to wait until Phase IV and award of a construction contract to initiate these tasks;

(4) Phase IV (April 1978-June 1979) consisted of the actual construction and testing activities, and preparation of this final report. Work for Phase IV was initiated in early April 1978, substantially ahead of the initial schedule, by the chemical grouting test program. The results from this program were needed to select grout mixtures for subsequent use in the other test programs. To accomodate the advanced schedule, the grouting activities were included in WCC contract. WCC, in turn, subcontracted the work to others, with the approval of the Government.

3.2 TEST AREA SELECTION

3.2.1 Criteria and Requirements

The primary criterion for the selection of the test area was similarity of subsurface conditions to those existing at Locks and Dam No. 26. In addition, the following other requirements also influenced the selection:

(1) availability of working area;

(2) accessibility from land and from the river; and

(3) ground surface elevation with respect to river or groundwater surface elevation.

3.2.2 Preliminary Selection

Four candidate locations for the test area were identified by the Government at the start of the project. The preliminary selection of the test area location was made on the basis of the above criterion and requirements. Although no candidate location exactly matched the conditions at Locks and Dam No. 26, Ellis Island was selected at the most desirable location for the test program.

3.2.3 Confirmation of Test Area Location

On the basis of subsurface investigations, Ellis Island was confirmed as the most desirable location for the test area. It was also concluded at that time that all proposed tests could be made on land, as opposed to overwater, provided groundwater level was controlled and maintained close to ground surface for some of the tests. Conducting the tests overwater, as it was initially considered, would complicate the tests and not yield better results. Extrapolation to overwater conditions can be made on the basis of experience of river-oriented contractors in the area.

3.2.4 Test Area Subsurface Conditions

The test area is located on the Mississippi River flood plain. Ground surface at Ellis Island is generally flat, ranging from el 410 to el 420. The subsurface profile in descending order consists of:

(1) flood plain deposits (soft to firm silty clay with varying amount of fine sand and organic material);

(2) recent alluvium (medium dense to dense, fine to medium, poorly graded sand to silty sand with occasional gravel or sandy clay lenses);

(3) alluvial outwash (coarse to fine, poorly graded sand with some silty sand and gravel zones);

(4) Wisconsinan outwash (coarse to medium, poorly graded sand, with traces of silt and gravel);

(5) Illinoian ice contact deposits (coarse to fine, poorly graded sand with numerous boulders, cobbles and gravel, and occasional silty sand zones; and

(6) limestone: St Genevieve Formation.

At some locations, glacial till lenses were found immediately overlying the bedrock. No glacial till was found under Locks and Dam No. 26. The presence of till could affect test results regarding foundation elements installed to rock (drilled-in piles and rock anchors). To avoid this undesirable influence, the test area was located near the downstream end of Ellis Island, where no glacial till was found.

3.2.5 Implications of Test Area Selection

The selected location of the main test area near the downstream end of Ellis Island had several implications. Among them:

(1) a substantial access road was required from Highway 67 to the test area; and

(2) the test area which was at el 410 to el 417 needed to be protected from Mississippi River floods.

The initial intent, and the corresponding initial design, was to ring the test area with a levee having a crest elevation of 430. The access road was also to be constructed on top of an embankment at el 430.

In late April 1978, as plans, specifications and cost estimates for the General Testing Contract were nearing completion, and after bids had been received for the dewatering and site preparation contracts, it became apparent that adjustments in the program would have to be made to maintain costs within budget. As a consequence, the levee and roadway elevation was reduced to el 420. Although this decision reduced the degree of protection against flooding, the projected cost was substantially reduced. At the same time, the scope of the grout-pile performance test program was reduced and the remaining activities for this program were incorporated into the pile driving effects test program (Section 2.4). The earth anchor test program was eliminated entirely (Section 2.7). These adjustments entailed modifications of the main test area configuration.

Another implication of the test area selection impacted the chemical grouting test program. It had been decided to begin the program in early April, ahead of the other tests, and also ahead of the dewatering and site preparation work. The chemical grouting test program could not, therefore, benefit from these constructions. To provide access and flood protection, the tests were conducted near Highway 67, in an area where the natural ground surface was at el 420 or higher. The configuration of the main test area was again adjusted to reflect this change.

3.3 DESIGN APPROACH

The tests were designed following the same general approach. The significant aspects of performance that needed to be measured during the tests to achieve the program objectives were identified and selected at the design stage. The intent was to design the tests so that those aspects of performance that had been selected as significant would be enhanced to facilitate interpretation. These aspects were ranked to allow first, measurement of gross performance, and second, understanding of mechanisms. By gross performance, it is meant the total, unrefined observations and measurements free from any manipulation of data. The gross performance of the tests was predicted. The prediction process was generally as follows:

(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;

(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, the 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.

4 RESULTS OF CHEMICAL GROUTING TEST PROGRAM

4.1 SCOPE OF TEST PROGRAM

The chemical grouting test program involved testing four different grouting methods, eight grout types, and two grout-hole spacings. A total of 123,500 gal of grout was injected through 74 grout holes to grout approximately 1440 yd^3 of alluvial sand. The test area was explored by borings and instrumented before grouting, grouted, reexplored by borings after grouting, dewatered, and excavated to observe results of grouting.

The tests were designed from November 1977 to March 1978. The test instrumentation was developed and installed in March 1978. The field grouting tests were conducted from April to June 1978. The excavation of the tests area was made in August 1978. Several series of laboratory tests were made from March 1978 to January 1979 in support of the field tests.

The sand strata grouted during the test program comprised the upper 20 ft of the recent alluvium deposits of the Mississippi River valley. Initial subsurface conditions at the test area were assessed before grouting by borings, sampling, and in situ testing (standard penetration tests, static cone penetration tests, pressuremeter tests, borehole permeability tests, and shear wave velocity measurements). The alluvial sand is generally fine- to medium-grained and contains on the average no more than 5 percent fines (that is, 5 percent by weight of soil particles passed through a No. 200 US sieve). The coefficient of permeability in the zone that was grouted ranged from 5×10^{-3} cm/s to 3×10^{-2} cm/s. During excavation of the test area, abundant concentrations of carbonaceous material (wood, charcoal, and lignite) ranging in size from silt to large tree trunks, were observed. The alluvial sand was always cross-bedded. Cross-beds were usually gently dipping and were from several inches to a few feet thick.

4.2 GROUTING ACTIVITIES

Grout injection was done by a joint venture of Raymond International Builders, Inc. and Soletanche and Rodio, Inc., subcontractors to Woodward-Clyde Consultants. A modular grouting plant was used to mix and pump silicate and cement-bentonite grouts and monitor grouting parameters (grout pumping pressure and pumping rate, and grout volume). Grout was pumped through open-bottom pipes and through sleeve-pipes. The pumping pressure was kept at less than 1 lb/in^2 per foot of soil above the bottom of open-bottom pipes. Grout was pumped at an average rate of 800 l/hr (3.5 gal/min) through each open-bottom pipe. The average grout take for open-bottom pipe was intended to be 25 percent of the volume of soil to be grouted. Actually, it averaged 25.6 percent.

Three grouting procedures were tested using sleeve-pipes. In one procedure, grout was pumped using a grouting pressure limiting criterion of 1 lb/in^2 per foot of overburden. Grout take was very small for this procedure. In a second procedure, the intended volume of grout was injected in one stage using a pumping rate less than 85 percent of the rate that would induce hydraulic fracturing of the soil. This pumping rate was usually between 300 l/hr and 450 l/hr (1.3 gal/min and 2 gal/min). The average grout take for the second sleeve-pipe procedure was intended to be 45 percent of the volume of soil to be grouted. Actually, it averaged 43.9 percent. In a third procedure, the intended volume of grout was injected in several separate stages, also using a rate of pumping of 300 l/hr to 450 l/hr. The actual grout take for the third procedure was 52.6 percent of the volume of soil to be injected.

4.3 INSTRUMENTATION MONITORING

Ground instruments were installed prior to grouting to monitor the effects of grouting (vertical and horizontal displacements of the soil mass, and porewater pressure). The maximum observed heave was 0.02 ft, except for one zone that heaved 0.048 ft. This localized, larger movement was attributed to injection of cement-bentonite grout. The predicted maximum heave was 0.02 ft. The maximum observed horizontal displacement was 0.018 ft as compared to a predicted value of 0.1 ft. Generally, the horizontal displacements were only slightly greater than the instrumentation accuracy. The maximum observed excess porewater pressure was about 12 lb/in^2 as compared to a predicted maximum change of 30 lb/in^2.

4.4 EVALUATION OF GROUTING RESULTS

Grouting results were evaluated by exploration and testing in boreholes drilled from ground surface, and by excavation, mapping, and in situ testing of the grouted soil. Standard and static cone penetration tests were used to assess the extent of grout penetration. Pressuremeter and borehole permeability tests, and shear wave velocity measurements were used to measure in situ properties of grouted soil. The in situ tests and measurements were found to be effective in assessing grouting results from ground surface.

The test area was dewatered and excavated. The extent of grout penetration was mapped and photographed. Observations indicated that open-bottom pipe grouting method was generally unsuccessful in achieving complete grout penetration. Low-pressure sleeve-pipe grouting was totally ineffective. Single-stage sleeve-pipe grouting yielded better, but not complete grout penetration. Multiple-stage sleeve-pipe grouting was generally very effective in achieving almost complete grout penetration. Six-ft grout-hole spacing was sometimes not adequate, and 4.2-ft spacing was found to achieve the desired grout penetration and uniformity. Cement-bentonite grout did not permeate the alluvial sand. The small quantity of cement-bentonite injected resulted only in creating bulbs along geologic discontinuities. Hydraulic fractures were found in almost all grouted soil masses observed. The majority of the fractures were vertical or coincident to geologic discontinuities. Consistency of grouted soil varied from sandstone-like material for high-silicate-content grouts (55 to 45% silicate) to low-strength, cohesive material for low-silicate-content grouts (28 to 25% silicate).

Index and engineering properties of grouted soil were assessed by in situ and laboratory tests and measurements. Among the in situ testing methods, shear wave velocity measurements appeared to be a most appropriate method to detect both changes in soil properties and extent of grouting. In situ horizontal stresses increased by two to three times due to grouting. In-place dry unit weight of grouted soil averaged 1 to 3 lb/ft^3 more than before grouting. Strength and deformation modulus values were generally increased manyfold by grouting; the actual increases depended on the strength of the grout injected. The strength increase was due only to an increase in cohesion, the angle of internal friction remaining practically unchanged or decreasing slightly. Time dependent properties of grouted soil were tested. Significant creep was observed in some grouted zones, although tendency to creep failure was not generally pronounced. Siroc-type grouts exhibited less creep than grouts containing aluminate reactant. Grouts containing R600 reactant were the most creep-prone. Grouted soil was found to be much less permeable than ungrouted soil. Coefficient of permeability of grouted soil generally ranged from 10^{-4} cm/s to 10^{-6} cm/s. The reduction in permeability did not appear to be a function of grout type, but rather it was found to be a function of grouting method (that is, of grout penetration). Zones showing no decrease in permeability were zones left ungrouted.

4.5 COST

The total cost of grouting (excluding earthwork, dewatering, and engineering) was $565,560. Grouting costs varied widely (from $210 to $618 per cubic yard of soil to be grouted) depending on grout type, grouting method, and grout-hole spacing. When related to grout take, the range of cost variation was almost as wide ($7 to $20 per cubic yard of soil to be grouted and per percent grout take). Cost of open-bottom pipe grouting averaged $294 per cubic yard of soil to be grouted and $10 per cubic yard of soil to be grouted and per percent grout take. Cost of single-stage sleeve-pipe grouting averaged $425 per cubic yard of soil to be grouted and slightly less than $10 per cubic yard of soil to be grouted and per percent grout take. Cost of multiple-stage sleeve-pipe grouting averaged $437 per cubic yard of soil to be grouted and slightly more than $8 per cubic yard of soil to be grouted and per percent grout take.

These costs are not representative of production chemical grouting projects, especially for open-bottom pipe applications, because of the relatively large effect of mobilization and contractor's supervision costs inherent to the test program. It is likely that these costs could be decreased by a factor of two to four and be more representative of production grouting.

5 RESULTS OF PILE DRIVING EFFECTS TEST PROGRAM

5.1 SCOPE OF TEST PROGRAM

The pile driving effects tests were designed to assess the magnitude of permanent displacements of axially and laterally loaded test structures supported on timber friction piles caused by nearby pile driving; assess the efficiency of chemical grout injection into the soil in reducing these displacements; and investigate the mechanisms governing pile driving effects. Adjunctly, the tests involved axial and lateral load tests on single timber piles and timber pile groups in ungrouted and grouted soil. The effects of chemical grouting on steel pile driving resistance and lateral load capacity of single steel piles were also assessed.

The tests were designed from November 1977 to May 1978. The test instrumentation was developed and installed from April to October 1978. The field tests were conducted from August 1978 to March 1979. Six test structures (monoliths) were constructed and tested. The test monoliths were founded on 46 instrumented timber piles having various configurations. Forty-two prototype steel piles were driven near the loaded test monoliths. Six prototype piles were laterally load tested. A total of 65,500 gal of low-strength silicate grout was injected into the soil surrounding two of the test structures. The tests involved a complex instrumentation system for manual and computerized data acquisition.

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 at various times during the tests to detect changes caused by timber pile jetting and driving, protot.pe pile driving, and chemical grouting. The subsurface investigations relied primarily on the use of in situ testing methods (dynamic and static cone penetration, pressuremeter and permeability tests, and cross hole shear wave velocity measurements).

5.2 DESIGN AND CONSTRUCTION OF TEST FACILITIES

5.2.1 Test Program Design

Test Monoliths. The pile driving effects test program was designed such that the test conditions generally modeled the conditions at nearby Locks and Dam No. 26. The test structures represented the structural conditions of the dam to an acceptable degree of similitude; considerations were given to scale, timber pile configuration, construction details, and load levels and history on the existing structures. To some extent, these conditions also model other similar navigation structures on the Mississippi River. The test variables were selected consistent with the objectives of the program stated in Section 2.4.

Six monoliths were constructed; four were founded on 8- to 12-timber pile groups; two on single timber piles. The number, configuration, and other variables of the test monoliths permitted various comparisons and evaluations to be made:

(1) single timber pile vs timber pile groups;

(2) effects of other adjacent timber piles on monolith performance;

(3) behavior of interior vs exterior timber piles;

(4) effects of applied load level on monolith performance; and

(5) effects of grouting on monolith performance.

The performance of the monoliths was predicted using empirical data. Only those aspects of performance which were necessary to meet the objectives of the program were considered. An instrumentation system was designed to be consistent with the ranking of the significant aspects of performance and predicted performance.

Prototype Piles. The prototype piles driven near the loaded monoliths were primarily H piles, a type of pile likely to be used in future construction on navigation structures. A few pipe and sheet piles were also driven for comparison purposes. The primary pile driving hammer used for the tests was a single-acting air hammer imparting a relatively constant impact energy to the piles. Other hammers (diesel and vibratory) were also used.

Chemical Grouting. The effects of grouting on monolith and pile performance were evaluated. A low-strength silicate grout, tested earlier in the chemical grouting test program (Volume II) was selected. The selection was based on the previous test program results. In situ test results and visual observations indicated that piles could be driven through soil injected with this grout; soil injected with all the other grouts tested in this program was considered to be too hard for pile driving by conventional methods.

5.2.2 Construction of Test Facilities

Earthwork and Dewatering. Construction of the test facilities involved extensive site preparation and construction of a dewatering and levee system to protect the site from Mississippi River floods. The dewatering system was used to maintain the groundwater level very close to the ground surface around the test monoliths. By maintaining the soil surrounding the monoliths submerged, the tests did not need to be conducted overwater.

Reaction Structures. The conduct of the tests required design and construction of appropriate reaction structures and systems capable of delivering large axial and lateral loads to the test monoliths. The size and capacity of these reaction structures were important factors in the selection of the scale and configuration of the test monoliths.

Test Monolith Construction. The timber piles under the test monoliths were installed by jetting and driving to a prescribed tip elevation; the installation method was similar to that used during construction of Locks and Dam No. 26. The timber piles were instrumented with inclinometer casings, and multiple levels of strain gages and telltales. Performance of the piles during driving was monitored with a pile driving analyzer. After timber pile installation, the soil properties were reassessed by in situ tests; the soil was generally made denser and stiffer by the timber pile installation.

Chemical Grouting. Chemical grout was injected in the upper 20 ft of soil surrounding two of the test monoliths. Grouting was done through sleeve pipes at 4-ft spacing. The grouting plant used was of the proportional type; the grout components were pumped separately and merged just before entering the grout pipes. Significant grout leakage and small grout takes were experienced under one monolith (M3); the grout appeared to leak from around the timber piles. Grouting was probably not very thorough under the monolith. Elsewhere, grouting was accomplished satisfactorily. After grouting, the soil properties were reassessed by in situ tests; the grouted soil properties were similar to those measured during the earlier chemical grouting test program for the type of grout used (Volume II); that is, an increase of about 20 percent in resistance to dynamic and static penetration, and to expansion of the pressuremeter was noted; somewhat greater increase in shear wave velocity was also observed.

5.3 TESTING ACTIVITIES

5.3.1 Monolith Preloading

Each monolith was first laterally loaded and unloaded for a number of cycles. This preconditioning was done to approximate the effects of load history on actual navigation structures.

5.3.2 Pile Driving Effects Testing

Prototype piles were driven at decreasing distances from a given monolith. Only the monolith being tested was axially and laterally loaded; the other monoliths were unloaded. The monolith displacement and surrounding soil deformation were monitored during prototype pile driving. Performance of the prototype piles during driving was monitored using a pile driving analyzer system. The dynamic responses of the soil mass and test monoliths were also measured. The prototype piles were generally driven 50 ft to 5 ft from the monoliths; for one monolith, at the beginning of the program, prototype piles were driven as far as 200 ft from the monolith.

5.3.3 Monolith Load Testing

Upon completion of the pile driving effects stage of testing, each monolith was load tested to failure, either axially or laterally.

5.3.4 Lateral Load Testing of Prototype Piles

Three instrumented H piles and three instrumented pipe piles were load tested to failure under lateral load. Four piles were driven in ungrouted soil. The soil surrounding two of these four piles was grouted (postgrouted); the soil around the other two piles was left ungrouted. The last two piles were driven through the previously grouted soil (pregrouted).

5.4 SUMMARY OF TEST RESULTS

5.4.1 Monolith Preloading

The predicted displacements of the monoliths were compared to the displacements measured during initial lateral loading. The comparisons show that, generally, the prediction overestimated the lateral displacement of the monoliths by a factor of approximately two. A large part of the overprediction is attributed to the presence of a berm in front of the monoliths; the effects of the berm were not considered in the predictions.

5.4.2 Prototype Pile Driving Effects

Performance of Prototype Piles During Driving. The maximum energy and compression force transmitted from the hammer to the pile butt were predicted at 23 ft-k and 500 k, respectively. Measurements during driving indicated that these values were overestimated; maximum energy was generally between 10 and 17 ft-k, and maximum force was generally between 300 and 400 k. The differences are attributed to hammer assembly inefficiency. The pile driving resistance through grouted soil was approximately twice that in ungrouted soil.

Vibrations Induced by Prototype Pile Driving. The observed displacements of the monoliths during prototype pile driving correlated well with the measured ground vibration characteristics. The cumulative effects of ground vibrations were characterized by the summation with depth of the calculated product of peak particle velocity observed for each foot of prototype pile penetration times the blowcount for that foot (referred to as cumulative peak velocity). Horizontal displacement of the monoliths correlated well with cumulative peak velocity values derived near ground surface; settlement correlated well with cumulative peak velocity values derived at a depth of 50 ft below ground surface.

Detailed examination of the data for monolith M2 indicates that, after each hammer blow, the monolith continued to oscillate horizontally at a frequency close to its natural frequency; however, the amplitude of the horizontal vibratory motion of the monolith was much smaller than that of the ground at shallow depth. The vertical motions of the ground at shallow depth and of the monolith were similar.

Monolith Displacement. The following table summarizes the displacements of the various monoliths during prototype pile driving. The total number, location, and type of piles, and the type of hammer used were different for each monolith; therefore, direct comparison of the values in the table cannot be readily made. The evaluation for each case is discussed in pertinent sections of Volume III.

Monolith No.	Timber Pile Configuration	Load* Level	Grouted	Total No. Of Prototype Piles Driven	Measured Displacement in. Lateral	Measured Displacement in. Average Vertical
M1	3 x 4	High	No	13	1.5	1.3
M2	2 x 4	High	No	14	1.9	1.1
M3	2 x 4	High	Yes	8	1.1	0.6
M5	2 x 4	Low	No	7	0.7	0.4
M6	Single	High	No	14	2.0	1.5
M7	Single	High	Yes	8	2.3	1.0

* High-load level was 30 t/pile axially and 6 t/pile laterally; low-load level was 15 t/pile axially and 4 t/pile laterally

The following conclusions were drawn from the test results:

(1) significant cumulative displacements of the monolith (that is, displacements larger than the accuracy of the measurements) were measured when prototype piles were driven at a distance of 50 ft or less from the loaded monoliths;

(2) the cumulative displacement of a given monolith increased with the number of prototype piles driven;

(3) when as many as four piles were successively driven at the same distance from a given monolith, incremental displacement due to each pile generally did not show a stabilizing or decreasing trend;

(4) grouting did not significantly reduce the displacement of the monoliths;

(5) the horizontal displacement of the monoliths at low-load level was about 50 percent less than the displacement at high-load level; the settlement at low-load level was about 30 percent less than at high-load level;

(6) the single timber piles and the pile groups were equally affected by prototype pile driving; and

(7) the vibratory hammer induced much larger displacement of one monolith than the impact hammers (air or diesel); the impact hammers produced similar results.

5.4.3 Monolith Load Testing

Static lateral load at failure in ungrouted soil ranged from 14.3 to 15 t/pile for the single timber pile and the two pile groups tested. It was 14.8 t/pile for the timber pile group and 20 t for the single pile in postgrouted soil.

The timber pile group in grouted soil exhibited a stiffer response at low load levels than the groups in ungrouted soil. The reverse was found for the single timber piles; however, the load at failure for the single pile was 33 percent larger in grouted soil than in ungrouted soil. The response of the timber piles in grouted soil was affected by the creep characteristics of the grouted soil, and incomplete grouting near the ground surface under the pile groups (monolith M3, Section 5.2.2). The static axial load at failure for the single timber pile in ungrouted soil was 110 t; that for the pile group averaged 90 t/pile.

5.4.4 Lateral Load Testing of Prototype Piles

The results of the lateral load tests on the six prototype piles are given below.

	H Piles		Pipe Piles	
	Load at 0.25 in. Lateral Displacement t	Load at 0.5 in. Lateral Displacement t	Load at 0.25 in. Lateral Displacement t	Load at 0.5 in. Lateral Displacement t
Ungrouted Soil	8.5	15	5.5	10.5
Postgrouted Soil	12	16.5	4.5	8.5
Pregrouted Soil	7	10.5	8	12

The values for the ungrouted cases are in good agreement with the predicted values. The measured loads at the corresponding displacements were considerably smaller than the predicted values for the grouted cases. The differences are attributed to the time-dependent properties of the grouted soil and, to some extent, to the erratic penetration of the grout at shallow depth. The piles in postgrouted soil exhibited larger creep displacement than any other piles. The piles in pregrouted soil exhibited less creep than the piles in postgrouted soil; however, the creep rate in pregrouted soil was much larger than in ungrouted soil.

5.5 INFERRED MECHANISMS

The behavior of monolith M2 was examined in detail in an attempt to explain the mechanisms governing the effects of pile driving on the loaded, pile-founded test structures. Mechanisms inferred from this analysis involve progressive horizontal and vertical deformation of the soil in front of the monolith to a large depth as prototype piles are driven. The front timber piles closest to the driven piles deflect more than the rear piles. The deflection of the front piles is accompanied by a horizontal translation of the pile tips. The loads on the front piles are redistributed deeper along their shaft and to the other piles behind. This mechanism is basically different from the mechanism observed during static lateral load testing, which involves a uniform, shallow pile deflection, and small, shallow soil deformations.

6 RESULTS OF DRILLED-IN PILE TEST PROGRAM

6.1 SCOPE OF TEST PROGRAM

The drilled-in pile tests were designed to assess whether or not drilled-in pile construction has adverse effects on the surrounding soil mass, such as loss of ground and loosening. They were also intended to investigate the feasibility of constructing drilled-in piles at angles of batter flatter than usual (that is, flatter than 4 (vert) to 1 (hor)). The piles had to be drilled through approximately 130 ft of submerged alluvial and glacial soil and socketed into limestone bedrock.

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 January 1979. Three test drilled-in piles at a 4 to 1 batter were installed, and the effects of construction were assessed by various measurements. No pile was installed at a batter flatter than 4 to 1; batters flatter than 4 to 1 were deemed unfeasible on the basis of observations made during construction of the first three test piles.

The subsurface profile at the locations of the test area consisted of approximately 100 ft of sand and gravel of alluvial and glacial origin, overlying limestone bedrock. The sand and gravel were overlain by about 25 ft of cohesive flood plain deposits. The subsurface conditions were investigated at the design stage and again immediately before the tests; the conditions were reassessed after the tests to detect changes caused by construction of the test piles. The subsurface investigations relied primarily on the use of in situ testing methods (dynamic and static cone penetration tests, pressuremeter tests, and density measurements using a nuclear probe).

6.2 TEST PROGRAM DESIGN

6.2.1 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. The major difference was that the cohesive flood plain deposits forming the upper portion of the test area subsurface profile are not present under the existing structures.

6.2.2 Selection of Construction Method and Equipment

The requirements for the test piles were as follows. The piles had to be:

(1) installed by drilling techniques, as opposed to driving techniques, to minimize construction vibrations;

(2) installed through over 100 ft of submerged sand, gravel and occasional cobble zones, and socketed into bedrock;

(3) cased with a 20-in.-dia steel casing; and

(4) at a batter of 4 (vert) to 1 (hor) or flatter.

On the basis of literature review and discussions with specialized contractors and consultants, the Benoto method or some variation of it, was selected. Among available methods, only the Benoto method, which entails advancing a steel casing by applying oscillatory twisting motions and axial thrust to the casing while removing the soil inside the casing with hammergrabs, appeared to be capable of meeting the test requirements with some degree of success.

6.2.3 Effects of Pile Installation

The effects of drilled-in pile installation on the surrounding soil mass were assessed by monitoring ground instrumentation installed before the tests, measuring the quantity of soil excavated from inside the advancing casing, and measuring in situ soil properties after the tests. The ground instrumentation consisted of surface and subsurface settlement monitoring devices (surface reference points, Borros gages, and Sondex rings), and inclinometer casings.

The soil excavated from inside the casing was collected in bins and weighed on scales. On the basis of water content and in-place unit weight measurements, the volume that the excavated soil occupied in the ground was calculated and compared to the theoretical volume of the casing.

The in situ properties of the soil after drilled-in pile installation were inferred from in situ borehole tests; these properties were compared to the initial soil properties.

6.2.4 Angle of Batter

It was planned to attempt installation of up to three test piles at batters of 3 (vert) to 1 (hor) to 2 (vert) to 1 (hor).

6.3 TEST RESULTS

6.3.1 Effects of Pile Installation

Very little ground loss, if any, was experienced during drilling of two test piles at 6 ft centers. The volume of excavated soil exceeded the theoretical volume of the casing only near the bottom of the two piles. Elsewhere, the volume of excavated soil was equal to or less than the theoretical volume. Ground surface settlement was primarily caused by consolidation of the cohesive flood plain deposits under the weight of the fill placed to prepare the test area working platform. In general, the observed ground settlement at depth was small and proportional to the thickness of alluvial and glacial sand underlying the measurement point. The maximum settlement attributed to the installation of the two closely-spaced test piles was 0.26 in., measured at el 395, about 100 ft above bedrock surface.

Horizontal ground deformation was also small, generally less than 0.5 in. and not exceeding 1.5 in. The soil deformation generally did not indicate soil movement toward the piles. Slight reductions in soil properties (density, in situ stresses, and stiffness) were inferred from in situ tests. The reductions, however, were small, compared to the data scatter inherent in the soil; for all practical purposes, it can be concluded that the soil properties did not significantly change due to drilled-in pile installation.

6.3.2 Angle of Batter

On the basis of the very slow rate of progress experienced in installing the three piles drilled at 4 (vert) to 1 (hor) batter, it was concluded that flatter angles of batter would not be economically feasible. Installation of test piles at batters flatter than 4 to 1 was not attempted.

6.3.3 Evaluation of Equipment and Techniques

The Benoto boring machine was generally capable of performing the functions specified for installing piles at 4 to 1 batter. Various modifications to the original equipment had to be implemented to increase the productivity. Despite these modifications, the rate of progress remained very slow: 2.2 ft/hr from 0 to 50 ft depth; 1 ft/hr from 50 ft to 100 ft depth; 0.6 ft/hr below 100 ft depth. The rock socket was readily drilled using a rotary drilling technique and a tricone roller bit.

Water level and depth of soil plug inside the casing were shown to affect the rate of progress to a large extent. The water level inside the casing had to be maintained above the outside groundwater surface to avoid blowouts at the bottom of the casing. As a general rule, it was found that the soil plug inside the casing should not be less than 1 ft to avoid creep of the soil into the casing nor more than 4 ft to be able to advance the casing by oscillation and axial thrust.

6.4 COST INFORMATION

On the basis of the production rates experienced during the tests, corrected for major atypical delays (such as equipment modifications), but uncorrected for testing interruptions, the cost of constructing one drilled-in pile was estimated at $42,500. This estimated cost assumes that equipment and crew requirements would be the same as those experienced during the tests. It is likely that this estimated cost could be decreased by some unknown amount and be more representative of large scale production work.

6.5 SUMMARY OF CONCLUSIONS

In summary:

(1) the Benoto method is feasible to install 20-in.-dia piles at 4 to 1 batter;

(2) the method is extremely slow and does not appear to be economical to install 20-in.-dia piles at 4 to 1 batter;

(3) the method results in very small or no loss of ground provided the water level inside the casing is maintained above groundwater level;

(4) ground deformation associated with drilling is small;

(5) soil properties are not significantly affected by drilling; and

(6) the method is not feasible for angles of batter flatter than 4 to 1 under the conditions prevailing at the test area.

7 RESULTS OF ROCK ANCHOR TEST PROGRAM

7.1 SCOPE OF TEST PROGRAM

The rock anchor tests were designed to assess whether or not rock anchor construction has adverse effects on the surrounding soil mass, such as loss of ground and loosening, and to obtain information concerning the load capacity of rock anchors. The anchors had to be installed at an inclination of 45 degrees through approximately 180 ft of submerged alluvial sand and gravel and grouted into limestone bedrock.

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.

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 sand and gravel were overlain by about 25 ft of cohesive flood plain deposits. 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).

7.2 TEST PROGRAM DESIGN

7.2.1 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.

7.2.2 Selection of Anchor System and Installation Method

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

The Atlas-Copco ODEX 165 overburden method was used to drill the anchor holes. This method was selected at the design stage by comparison of past performance of various drilling 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.

7.2.3 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.

7.2.4 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.

7.3 TEST RESULTS

7.3.1 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.

7.3.2 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 (120 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 (that is, 13 percent of the load after lock-off).

7.3.3 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 7.3.1. 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. The net drilling rate averaged 35 ft per 10-hr shift, excluding atypical delays such as extreme weather, major breakdowns, and interruptions due to testing.

7.4 COST INFORMATION

On the basis of the corrected production rates experienced during the tests (35 ft/10 hr), the cost of installing one rock anchor was estimated at $22,300. This estimated cost assumes that equipment and crew requirements would be the same as those experienced during the tests. It is likely that this estimated cost could be decreased by some unknown amount and be more representative of large scale production work.

7.5 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 is extremely slow due, to a large extent, to the demonstrated need for welding each successive casing section as the casing is 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.