AD-A160 524 CLARENCE CANNON DAM AND MARK TWAIN LAKE FOUNDATION AND EMBANKMENT COMPLET..(U) ARMY ENGINEER DISTRICT ST LOUIS MO DEC 84 1/5

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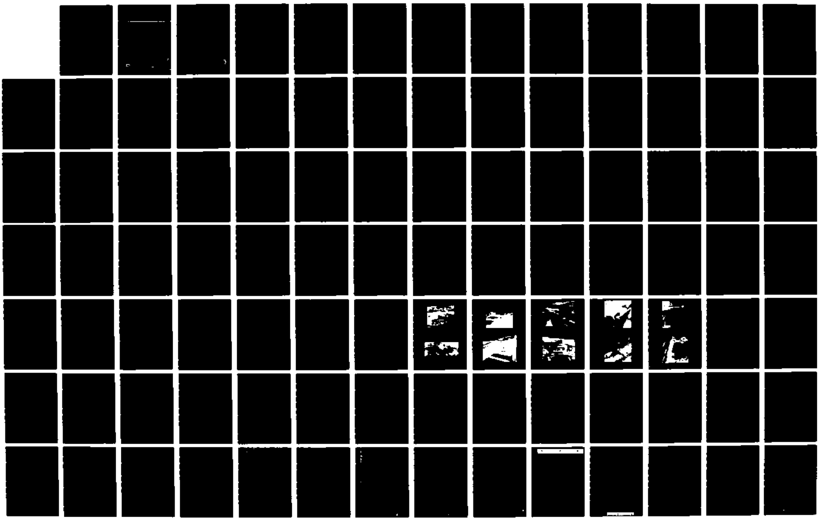

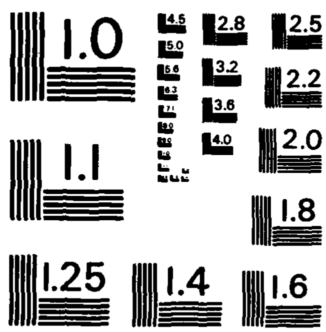

MICROCOPY RESOLUTION TEST CHART
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(6)

CLARENCE CANNON DAM
& MARK TWAIN LAKE

AD-A160 524

FOUNDATION AND EMBANKMENT COMPLETION REPORT

PART II MAIN DAM

PHASE II CONSTRUCTION AND RELATED CONTRACTS

VOLUME I — NARRATIVE

SECTIONS 1 THRU 9

US Army Corps
of Engineers
St. Louis District

DECEMBER 1984

85 10 16 141

CLARENCE CANNON DAM AND RESERVOIR

SALT RIVER, MISSOURI

CENTER, MISSOURI

FOUNDATION AND EMBANKMENT COMPLETION REPORT

PART II MAIN DAM

PHASE II CONSTRUCTION AND RELATED CONTRACTS

VOLUMES I and II - NARRATIVE

Department of the Army
Corps of Engineers, St. Louis District
210 Tucker Boulevard, North
St. Louis, Missouri 63101

FOUNDATION AND EMBANKMENT COMPLETION REPORT
CLARENCE CANNON DAM AND RESERVOIR

PART II MAIN DAM

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SECTION 1

INTRODUCTION

A. General Information

The Clarence Cannon Dam and Reservoir was authorized by the Flood Control Act of 23 October 1962 and was designed for the following purposes: flood control, electric power, water supply, recreation, fish and wildlife conservation, and incidental nagivation (Mississippi River). An afterbay for pumpback hydropower generation is formed by a re-regulation dam located 9.5 miles downstream of the main dam, an integral part of the project design.

The Clarence Cannon Project is located principally in Monroe and Ralls Counties in northeastern Missouri approximately 25 miles west of Hannibal and 125 miles northwest of St. Louis, Missouri. The main dam site is on the Salt River 63 miles upstream from its confluence with the Mississippi River near Louisiana, Missouri, with a re-regulation dam 9.5 miles downstream of the main dam. Road access to the Project is provided by U.S. Route 24 on the north and west, Missouri Route 154 on the south, and Missouri Route 19 on the east (refer Drawing No. 1/2 for Project location and Plate No. 1 for a list of the pertinent Project data).

B. Main Dam Description

1. Concrete Structure

(a) General

The concrete unit is comprised of a powerhouse, gated spillway, and non-overflow sections, 845 feet in length. A rolled earth fill water temperature control weir to control temperature of water releases downstream is located 400 feet upstream of the centerline of the dam. An upper inspection gallery at approximate El. 635 feet NGVD traverses the non-overflow monoliths from D-1/2 to the powerhouse and D-11/12 to D-16. A lower gallery, which runs the length of the concrete structure, serves as a drainage and inspection gallery and was used as access for foundation grouting. Access to both galleries is provided through the stairway at roadway level to Monolith D-11/12 and through the powerhouse. Missouri Route J is routed over the dam, powerhouse and spillway (refer Drawing No. 3/2 for general plan).

(b) Spillway

The gated spillway, a concrete gravity type structure 230 feet in length, is founded on the Louisiana Limestone at approximate El. 465 feet NGVD. There are four 50-foot wide tainter gate bays and an ogee-shaped overflow section with a crest elevation of 600 feet NGVD. The spillway is spanned by a 65-foot wide roadway, walkway-overlook, and liftway bridge with crown elevation of 653 feet NGVD. A concrete stilling basin with two rows of 12-foot by 13-foot baffle piers and a 5-foot vertical end sill dissipates the water energy and velocity from the spillway. The stilling basin is 176.75 feet in length with a floor elevation of 508 feet NGVD. An exit channel with bottom elevation from 513 feet NGVD to 515 feet NGVD and width of 384 feet tapering to 220 feet extends from the end sill to the river channel 3,000 feet away. This channel is protected by 1-foot thick concrete paving 340 feet wide to a distance approximately 450 feet downstream of the end sill and 27-inch and 20-inch riprap on plastic filter cloth over the remainder of the channel to a distance 2,700 feet downstream of dam centerline. The stilling basin slab is anchored to the Hannibal Shale with No. 18 reinforcing steel anchor bars. Concrete retaining walls are provided on the right of the spillway and the left side of the powerhouse tailrace with a splitter wall between the two. Top of the retaining wall is at El. 572.5 feet NGVD.

(c) Non-overflow Monoliths

The non-overflow monoliths, reinforced concrete gravity structures, are comprised of three monoliths (D-1/2, D-3/4 and D-5/6), between the powerhouse and the embankment, and six monoliths (D-11/12 through D-17), between the spillway into the right embankment. Monoliths D-1/2 through D-5/6 and D-11/12 are founded on the Louisana Limestone at approximate El. 465 feet NGVD, whereas the non-overflow sections located in the right abutment are founded at various elevation with the Burlington/ Chouteau Limestones and the Hannibal Shale. The 43-foot roadway, designed as an integral part of each monolith, has a crown elevation of 653 feet NGVD.

(d) Powerhouse

The powerhouse is a concrete gravity structure 210 feet 9 inches in length located adjacent to the left side of the spillway and is founded on the Louisana Limestone at approximate El. 465 feet NGVD. It has four 18-foot wide by 48-foot high water intake passages to provide operating power to the 27,000 KW conventional and the 31,000 KW reversible generators. Six draft tube water passages, each 18-foot wide by 48-foot high, wide discharge water releases into the tailrace. The tailrace slopes upward from EL 583 feet NGVD to intersect the exit channel at El. 585 feet NGVD 160 feet farther downstream. The tailrace foundation is protected by a 1-foot thick reinforced slab with drain holes on 10-foot centers. The powerhouse structure is topped by a 43-foot wide walkway and roadway with crown elevation of 653 feet NGVD.

(e) Water Temperature Control Weir

To eliminate cold water releases detrimental to downstream fish habitat, a fixed crest compacted earth fill water temperature control weir has been constructed 400 feet upstream of the centerline of the main dam concrete structure. The earth fill structure is approximately 600 feet in length with a crest elevation of 580 feet NGVD. It is 30-foot wide at the crest with 1V:3H upstream and downstream side slopes. Scour protection has been provided upstream of the reversible power unit to prevent damage to the earth structure during pumpback operations. A 60-foot wide line of 5,000-pound capstone on 12-inch riprap bedding extends from the reversible power unit intake structure to approximately 30 feet upstream of the crest of the water temperature control weir. An area on either side of the line of the capstone, ranging from 3 feet of the slope toe to 30 feet at the crest, has been protected with 27-inch riprap on filter cloth.

2. Earthen Embankment

(a) General

The rolled earthen embankment has a crest length of approximately 1,100 feet and rises about 110 feet above the valley floor. Crown width at crest elevation 654 feet NGVD is 30 feet. Upstream slopes of the embankment are 1V:3.5H between the crest and approximate El. 605 feet NGVD and 1V:10H from that point to El. 585 feet NGVD. A relatively flat stability berm extends upstream from the embankment cross section across the first-stage diversion channel. Downstream slopes are 1V:3H from the crest to El. 611 feet NGVD, 1V:6H to El. 562 feet NGVD, and 1V:3H from that point to natural ground. Missouri Route J is routed across the crest of the dam. The embankment contains approximately 3 million cubic yards of compacted earth fill.

(b) Seepage Control

Internal seepage control is provided by a 10-foot thick vertical chimney drain with its upstream face at the embankment centerline beginning at El. 640 feet NGVD and intersecting a horizontal filter blanket at El. 545 feet NGVD. The 3-foot thick filter blanket extends downstream from the centerline approximately 450 feet to a toe drain system. Underseepage is controlled by a trapezoidal shaped cutoff trench whose base width varies from 60 feet to 65 feet (Buried Channel Area, approximate Station 12+00 to Station 14+00). The trench was excavated through the alluvial valley materials to either limestone or shale and then was backfilled with compacted clay. Transition from the impervious compacted fill in the cutoff trench to the alluvial/colluvial foundation materials is accomplished with a filter blanket placed upstream and downstream of the cutoff trench. The Phase 1 contract drawings indicated that the filter zone varies from 10 feet at Station 12+00, to a zero thickness at Station 11+00, and 10 feet thick at Station 14+00 to a zero thickness at Station 15+00. From approximate Station 15+80 to the left abutment, the sands/gravels do not exist due to natural or construction processes. Within this reach the entire embankment lies directly on Hannibal Shale. Control of possible seepage and/or abutment drainage at the left abutment/embankment contact is provided by the placement of 10 foot thick filter blanket material downstream of the dam centerline.

(c) Slope Protection

The upstream slope of the main dam embankment has been protected with riprap. On the 1V:10H slope between El. 585 feet NGVD and El. 605 feet NGVD, 16-inch riprap has been placed on a 6-inch bedding. Between El. 605 feet NGVD and crown elevation 654 feet NGVD, 22-inch riprap is placed on

9-inch bidding. The downstream slope is protected against erosion with grass cover. The rock fill end cone requires no slop protection.

C. Purpose and Scope of Report

Engineering Regulations Nos. 1110-1-1801 (15 December 1981) and 1110-2-1901 (31 December 1981) from the Office of the Chief of Engineers, outlines the needs for and scope of the as-built foundation and embankment reports, and authorizes their preparation for major or unique construction projects. The following narratives were prepared from data collected during construction and are intended to provide a complete and accurate record of foundation, embankment and concrete details so that a reference will be readily available in the future. This Report will be the basis of analysis for maintenance work and any future problems threatening the integrity of the main dam.

This Report discusses the site geology, methods used in excavation, procedures and extent of foundation treatment and condition of the final foundation, methods used for embankment and concrete placement, types of instrumentation and their installation procedures, limits of foundation grouting and various grouting techniques, exploration programs prior to and during construction, and lastly, a description and method of treatment for the various types of construction problems.

This is the final foundation report by the engineering regulation for the Clarence Cannon Dam and Mark Twain Lake. The initial foundation report for the main dam titled Clarence Cannon and Reservoir, Part 1 Main Dam, Phase I Construction, covered Phase I contract work (contract DACW43-71-C-0063) by Clarkson Construction Co. of Kansas City, Mo. This report was submitted shortly after the completion of construction in August, 1972. The second foundation report titled Re-Regulation Dam, Part III, covered foundation and embankment construction (contract DACW43-76-C-0101) by Rosiek Construction Co., Inc. of Morrilton, Arkansas. The report was submitted in April. 1981.

D. Contractors and Contract Supervision

The second phase of construction was advertised under Invitation No. DACW43-73-B-0016 and was awarded under Contract No. DACW43-73-C-0134 to Massman Construction Co. of Kansas City, Missouri, on 17 March 1973. Phase II commenced on 26 April 1973. and all field construction was essentially completed by August 1984.

Listed below are the major contractors referenced in this Report and their principal features of work during Phase II construction.

Contractor	Principal Features of Work
Massman Construction Co. Kansas City, Missouri (Prime Contractor)	All concrete, instrumentation and foundation treatment for the concrete structure
Luhr Bros., Inc. Columbia, Illinois (Subcontractor)	(a) Overburden and rock excavation for the concrete structure and earthen embankment, and rock bolt installation (b) Embankment instrumentation and foundation treatment (c) Main dam embankment and riprap placement
Continental Drilling Co. Madera, California (Subcontractor)	(a) Foundation drilling and grouting for the right abutment and Monoliths D-16 and D-17 (b) Lower gallery drains
Stang Hydronics Tulsa, Oklahoma (Subcontractor)	Unwatering operations
Western Waterproofing Construction Co. Kansas City, Missouri (Subcontractor)	Shotcreting operations
Inland-Ryerson Co. Melrose Park, Illinois (Subcontractor)	Post-Tensioning operations
Continental Drilling Co. Madera, California (Prime Contractor, Uplands Exploratory Drilling, Contract No. DACW43-78-C-0049)	Exploratory program for the left and right abutments
Boyles Bros. Drilling Co. Woods Cross, Utah (Prime Contractor, Main Dam Abutment and Uplands Grouting, Contract No. DACW43-79-C-0107)	(a) Foundation drilling and grouting operations for the right and left abutments (b) Foundation drilling and grouting in the lower gallery of the concrete structure (c) Lower gallery drains

Inspection of all contracts was performed by the Department of the Army, Corps of Engineers personnel assigned to the Project.

E. Chronology

26 April 1973	Day one of contract
30 April 1973	First equipment arrived at job site
18 May 1973	Commenced clearing operations on road to Soils Laboratory
21 May 1973	Commenced clearing operations on Saddle Dam
22 May 1973	Commenced clearing operations in Borrow Area No. 4
29 May 1973	Commenced stripping operations in diversion channel
30 May 1973	Removal of Phase I cofferdam
1 June 1973	Commenced excavating operations in diversion channel
14 June 1973	Commenced clearing operations in Borrow Area No. 1
22 June 1973	Commenced uncovering Phase I settlement gages
25 June 1973	Commenced draining operations in Borrow Area No. 2
28 June 1973	Completed uncovering settlement gages
29 June 1973	Commenced and completed clearing operations for Triangulation Stations Nos. TS-02 and TS-03
7 July 1973	Commenced drilling operations for permanent Bench Mark No. BM-01
9 July 1973	Commenced clearing operations for right abutment
10 July 1973	Commenced pile driving operations for the construction bridge over the diversion channel and clearing operations for the downstream exit channel
12 July 1973	Commenced drilling operations for Triangulation Station No. TS-03
19 July 1973	Commenced stripping operations for Borrow Area No. 2, commenced drilling operations for Triangulation Station No. TS-02 and completed driving piling for the construction bridge
28 July 1973	Commenced concrete placement for the construction bridge

6 August 1973	Commenced overburden excavation on right abutment
15 August 1973	Commenced rock drilling on right abutment
16 August 1973	Detonated first blast on right abutment
6 September 1973	Completed installation of permanent bench marks and Triangulation Stations Nos. TS-02 and TS-03
9 October 1973	Commenced rock bolt testing program
22 October 1973	Commenced fill placement in downstream temporary cofferdam
27 October 1973	Diversion of river from right abutment area to the diversion channel
4 November 1973	Commenced fill placement in Step I channel plug and upstream temporary cofferdam
9 November 1973	Completed rock bolt testing in limestone
10 November 1973	Completed Step I channel plug
29 November 1973	Commenced driving H-piling for batch plant
30 November 1973	Commenced shotcrete operations for right abutment 1V:1H slope
4 December 1973	Construction bridge washed out by high water and heavy drift
17 December 1973	Commenced construction of temporary diversion crossing and installation of deep well dewatering system near the downstream cofferdam
16 January 1974	Completed temporary diversion crossing and temporary cofferdam
5 February 1974	Completed driving H-piling for batch plant
6 February 1974	Commenced erection of batch plant and driving H-piling for second construction bridge across diversion channel
15 February 1974	Commenced pumping of deep well dewatering system
28 February 1974	Commenced left abutment excavation; commenced raising the cofferdam by 2 feet

2 April 1974	Completed second construction bridge
5 April 1974	Commenced installing well points for upstream dewatering system
15 April 1974	Commenced pumping of upstream well point system
25 April 1974	Commenced shale excavation, Station 7+00 to Station 10+00
7 May 1974	Commenced rock bolt testing program in shale
13 May 1973	Completed rock bolt tests in shale
3 June 1974	Commenced placing protective slab on El. 485 berm in shale excavation
18 June 1974	Commenced sawing shale faces on right abutment
9 July 1974	Removed upstream well point dewatering system
26 July 1974	Commenced construction of Saddle Dam
1 August 1974	Commenced excavation for downstream channel widening
14 August 1974	Commenced excavation of Louisiana Limestone
10 October 1974	Completed Saddle Dam
15 November 1974	Commenced drilling powerhouse post-tensioning borings
19 November 1974	Commenced placing protective slab in stilling basin wall
29 November 1974	Completed batch plant construction; commenced performance tests
9 December 1974	Placed SP 6-1; first concrete from batch plant
13 December 1974	Completed drilling operations for powerhouse post-tensioning borings. Started placing protective slab for Monolith D-13
18 January 1975	Placed first concrete for spillway Monolith D-7
22 February 1975	Fire damaged batch plant, ice plant and quality control laboratory

1 August 1975	Placed protective concrete in Monolith D-11/12 from reconstructed batch plant
28 August 1975	Completed rock excavation for main dam
22 January 1976	Mobilization and exploratory drilling on left abutment
4 March 1976	Completed overburden excavation for main dam structure
5 August 1976	Commenced drilling operations for stilling basin anchors
7 August 1976	Commenced installation of stilling basin anchors
2 September 1976	Completed drilling operations for stilling basin anchors
3 September 1976	Completed installation of stilling basin anchors
24 September 1976	Commenced placing riprap in exit channel
20 October 1976	Commenced placing capstones on water temperature control weir
5 November 1976	Commenced placing pervious fill behind tailrace wall
18 November 1976	Commenced placing upstream end cone rock fill
8 February 1977	Commenced rock excavation on left abutment
15 March 1977	Commenced placing exit channel pavement slabs
13 April 1977	Discovery of cavities in left abutment
15 July 1977	Commenced stressing tainter gate anchorage and commenced contact grouting on right abutment
23 August 1977	Commenced drilling and grouting operations for right abutment
10 October 1977	Commenced placing tailrace slabs
13 October 1977	Commenced lower gallery drain drilling
21 October 1977	Commenced setting tainter gates
21 November 1977	Completed placing fill behind stilling basin wall

8-9 March 1978	OCE/LMVD Geotechnical Conference; Left Abutment Treatment
3 April 1978	Uplands Exploratory Drilling contract awarded
19 June 1978	Completed drilling and grouting operations for right abutment
July 1978	Completed lower gallery drain drilling
24 July 1978	Completed left abutment cutoff wall inspection by OCE, LMVD and SLD personnel
8 January 1979	Commenced installation of plumb line in powerhouse
26 February 1979	Placed rock fill in upstream end cone
18 April 1979	Drilled dewatering wells in diversion channel
25 April 1979	Installed dewatering piezometers upstream
26 April 1979	Installed dewatering piezometers in downstream bank of temporary bridge
27 April 1979	Set upstream deep well pumps
30 April 1979	Placed 900-pound riprap in pilot channel, and hauled sands and gravels for third-stage cofferdam construction
2 May 1979	Installed joint movement plugs in pedestrian walkway and commenced placement of filter cloth on exit channel slope
7 May 1979	Commenced excavating expanded Borrow Area No. 2 for impervious fill
1 June 1979	Commenced rock excavation from roadway (S-JB north)
4 June 1979	Raised downstream pilot channel from El. 546 feet NGVD to El. 550 feet NGVD
5 June 1979	Commenced trench excavation for installation of toe drain
13 June 1979	Commenced work on inside berm of pilot channel
16 July 1979	Commenced drilling production holes (Road D), continued common excavation of exit channel and continued riprap placement in exit channel
18 July 1979	Commenced upstream temporary berm construction
23 July 1979	River diverted from diversion channel through sluices

3 August 1979	Cleaned left abutment cutoff trench prior to presplitting operations
9 August 1979	Completed installation of Open System Piezometers (PCA and PCS) for right abutment and concrete structure
10 August 1979	Placed rock fill on downstream end cone
18 August 1979	Commenced fill placement for main dam embankment
6 September 1979	Cleanup of diversion channel prior to embankment placement
11 September 1979	Commenced foundation preparation of Hannibal Shale in diversion channel
15 October 1979	Commenced drilling consolidation grout holes in left abutment
30 October 1979	Last day of foundation preparation for Hannibal Shale
6 November 1979	Commenced drilling left abutment curtain grout holes
19 November 1979	Commenced grouting operations in lower gallery
3 March 1980	Lower gallery inspection; all drains and grouting operations complete
11 March 1980	Commenced grouting operations on right abutment
9 April 1980	Commenced drilling Exporatory Holes Nos. 1, 2, 3 and 4
18 April 1980	Grouted Exploratory Holes Nos. 1, 2, 3 and 4 on left abutment
21 April 1980	Commenced removing frost damaged embankment material
5 May 1980	Completed removal of frost damaged embankment material
9 May 1980	Labor strike started
29 June 1980	Labor strike ended
30 June 1980	First day of fill placement
24 October 1980	Completed drilling and grouting operations on left abutment
31 October 1980	Last day of foundation preparation on left abutment for construction season
22 November 1980	End of fill placement for construction season

8 April 1981	Commenced removing 1980-1981 frost damaged material
30 April 1981	Completed removing 1980-1981 frost damaged material
18 May 1981	Periods of heavy rain and high river stages--all work ceased
25 May 1981	Contract work resumed
16 July 1981	All work complete for grouting contract
23 July 1981	Periods of heavy rain and flood conditions resulting in overtopping of cofferdam
10 August 1981	Work resumed
10 August thru 25 November 1981	Embankment restoration and minor foundation treatment along left abutment
13-18 August 1981	Cleanup of upstream area prior to placement of embankment and commenced construction of Embankment Protection
22 October 1981	Completed construction of upstream Embankment Protection to El. 592.4 feet NGVD
31 October 1981	Shutdown embankment operations for the winter
14 April 1982	Commenced removing 1981-1982 frost damaged material and embankment placement operations
4-10 June 1982	Periods of heavy rains and high river stages resulting in accumulation of debris behind the dam
17 November 1982	Embankment operations shutdown for winter
2-8 December 1982	Periods of heavy rains and high river stages resulting in accumulation of debris behind the dam
1-6 April 1983	Periods of heavy rains and high river stages resulting in accumulation of debris behind the dam
10 May 1983	First day of embankment placement for season
15 August 1983	Commenced preliminary cleanup and repair work on interior of deversion sluices
20 August 1983	Completed foundation preparation on left abutment

23-24 August 1983	Commenced and completed installation of three steel bulkheads for closure of the three diversion sluices
30 August 1983	Completed closure of the water temperature control weir
2 September 1983	Completed installation of the capstones on the water temperature control weir
8 September 1983	Completed sand chimney to El. 640 feet NGVD
23 September 1983	Completed main dam embankment to El. 654 feet NGVD (topout)
30 September 1983	Commenced intrusion grouting of first preplaced aggregate zone in diversion sluices (Sluice D9)
5 October 1983	Completed placement of 900-pound riprap on upstream slope of main dam embankment
8 October 1983	Completed seeding Borrow Areas Nos. 1 and 4
21 October 1983	Completed installation of epoxy grouted stainless steel waterstop in diversion sluices
9 November 1983	Completed intrusion grouting of preplaced aggregate zones in diversion sluices
21 November 1983	Commenced placement of pumpcrete in optional zones of diversion sluices
22 November 1983	Completed asphaltic surfacing on S-JB across main dam embankment
23 November 1983	Opened S-JB across main dam to traffic
30 November 1983	Completed planting tress and shrubs in Borrow Areas Nos. 1 and 4
8 December 1983	Completed mass filling of diversion sluices with pumpcrete
5-6 January 1984	Commenced and completed shrinkage grouting of top of diversion sluices
18 January 1984	First water releases from testing of powerhouse turbines
25 January 1984	Completed placement of 400-pound riprap on slope of downstream exit channel
August 1984	Field construction activities essentially complete with only minor amounts of seeding remaining.

PERTINENT DATA SUMMARY

Project Purposes	Flood control, power, water supply, recreation, fish and wildlife conservation, incidental navigation (Mississippi River)

Stream flow under natural conditions
(at dam site) (period 1925 to 1965, inclusive)

Average daily flow	1,450 cfs
Maximum flow	67,900 cfs
Minimum flow	0 cfs

Items	Units	Initial	After 100-year Sedimentation (1)*
Main dam site (river mile)		63.0	
Re-regulation dam site (river mile)		53.5	
Drainage area (above main dam)	sq.mi.	2,318	
Drainage area (between dams)		29	
Inactive storage pool			
Top elevation	ft. NGVD	567.2	567.2
Top area	acres	5,900	4,400
Storage	ac.ft.	87,000	44,300
Storage (runoff)	inches	0.70	0.36
Joint-use pool			
Top elevation	ft. NGVD	606.0	606.0
Top area	acres	18,600	17,400

*Numerals in parenthesis refer to like-numbered notes at the end of the data summary

Item	Units	Initial	After 100-year Sedimentation (1)*
Storage	ac.ft.	457,000	399,500
Storage (runoff)	inches	3.70	3.23
Regulated outflow			
Main dam			
Maximum	cfs	12,207 (2)	
Minimum	cfs	11,759 (2)	
Re-regulation dam			
Maximum	cfs	3,296 (3)	
Minimum	cfs	50 (4)	
Flood control pool (lower zone)			
Top elevation	ft. NGVD	624.8	624.8
Top area	acres	28,600	28,500
Storage	ac.ft.	442,000	426,200
Storage (runoff)	inches	3.58	3.45
Regulated outflow			
Mississippi River not in flood			
Main dam			
Maximum	cfs	12,000	
Minimum	cfs	11,973 (5)	
Re-regulation dam			
Maximum	cfs	12,000	
Minimum	cfs	551	
Mississippi River in flood			
Main dam			
Maximum	cfs	12,000	
Minimum	cfs	9,884 (6)	
Re-Regulation dam			
Maximum	cfs	3,230	
Minimum	cfs	453	

Item	Units	Initial	After 100-year Sedimentation (1)*
Flood control pool (upper zone)			
Top elevation	ft. NGVD	638.0	638.0
Top area	acres	38,400	38,400
Storage	ac.ft.	442,000	440,900
Storage (runoff)	inches	3.58	3.54
Regulated outflow			
Mississippi River not in flood			
Maximum	cfs	12,000	
Minimum	cfs	12,000	
Mississippi River in flood			
Maximum	cfs	9,900	
Minimum	cfs	9,000	
Induced surcharge pool			
Top elevation	ft. NGVD	642.0	642.0
Top area	acres	42,200	42,200
Storage	ac.ft.	161,000	161,000
Storage (runoff)	inches	1.30	1.30
Maximum outflow	cfs	217,000	
Surcharge pool (total)			
Top elevation	ft. NGVD	648.0	648.0
Top area	acres	48,700	48,700
Storage	ac.ft.	433,800	433,800
Storage (runoff)	inches	3.51	3.51
Maximum outflow	cfs	267,500	
Freeboard			
Top elevation	ft. NGVD	653.0	653.0
Top area	acres	53,200	53,200

Item	Units	Initial	After 100-year Sedimentation (1)*
Storage	ac.ft.	257,120	257,120
Storage (runoff)	inches	2.08	2.08
Height	feet	5.00	5.00
Standard project flood			
Peak flow (dam site)	cfs	210,560	
Peak inflow (reservoir)	cfs	295,000	
Maximum outflow	cfs	12,000	
Design storm	inches	12.19	
Runoff (total)	inches	7.93	
Runoff (total)	ac.ft.	980,362	
Runoff (above 12,000 cfs)	inches	7.00	
Runoff (above 12,000 cfs)	ac.ft.	865,400	
Spillway design flood			
Peak inflow	cfs	476,200	
Peak outflow	cfs	276,500	
Design storm	inches	21.79	
Runoff	inches	15.71	
Runoff	ac.ft.	1,942,180	
Main Dam			
Top elevation	ft. NGVD	653.0	
Height above streambed	feet	138	
Length of crest	feet	1,940	
Spillway (gate controlled)			
Crest elevation	ft. NGVD	600.0	
Crest width (gross)	feet	230	
Number of gates	each	4	
Gate width	feet	50	
Gate height	feet	39	

Item	Units	Initial	After 100-year Sedimentation (1)*
Tailwater elevations			
12,000 cfs	ft. NGVD	530.0 (7)	
276,000 cfs	ft. NGVD	577.0	
Re-regulation dam			
Top elevation	ft. NGVD	537.0	
Height above streambed	feet	40	
Length of crest	feet	1,550	
Spillway (gate controlled)			
Crest elevation	ft. NGVD	499.0	
Crest width (gross)	feet	68	
Number of gates	each	2	
Gate width	feet	30	
Gate height	feet	31	
Sluice			
Diameter	inches	36	
Tailwater elevation			
12,000 cfs	ft. NGVD	517.0	
50 cfs	ft. NGVD	500.0	
Power			
Installation (nameplate)	KW	58,020	
Number of units	each	2	
Conventional	KW	27,000	
Reversible	KW	31,020	
Turbine releases			
At design head (75 feet) and rated capacity			

Item	Units	Initial		After 100-year Sedimentation (1)*
Conventional	cfs	5,545		
Reversible	cfs	5,625		
Maximum release	cfs	12,207		
Gross head	feet	81	(8)	
Average net head	feet	75.4	(9)	
Design head	feet	75.0	(10)	
Minimum net head	feet	61.7	(11)	
Drawdown	feet	13.3	(12)	
Prime capacity	KW	6,127	(13)	
Prime energy	KWH	53,673,600	(14)	
Dependable capacity	KW	54,000	(15)	
Average annual firm energy	KWH	47,337,223	(16)	
Average annual total energy	KWH	84,197,000	(17)	
Average annual energy required for pumping	KWH	28,461,000	(17)	

NOTES:

(1) One hundred-year sediment volume, 115,000 acre feet, distributed below El. 627.4 feet NGVD, highest pool elevation attained in period of record routing.

(2) Required instantaneous release through turbines for firm power.

(3) Mean daily re-regulated power release.

(4) Downstream release on pumpback days and on weekends.

(5) Firm powerwhile at pool El. 606.0 feet NGVD.

(6) Firm power while at pool Elevation 624.8 feet NGVD.

(7) Based on a continuous flow through the re-regulation pool.

(8) Pool El. 606.0 feet NGVD, tailwater El. 525.0 feet NGVD.

(9) Pool El. 601.4 feet NGVD, tailwater El. 525.0 feet NGVD, 1-foot head loss.

(10) Pool El. 601.0 feet NGVD, tailwater El. 525.0 feet NGVD, 1-foot head loss.

(11) Pool El. 592.7 feet NGVD (minimum elevation for generator), tailwater El. 530.0 feet NGVD (elevation near end of generating period), 1-foot head loss.

(12) Pool El. 606.0 feet NGVD to minimum for generation El. 592.7 feet NGVD.

(13) Power available on a continuous basis during the year of minimum energy generated.

(14) Total energy generated during year of most adverse generating conditions utilizing pumpback.

(15) The load carrying ability of system under most adverse conditions and minimum pool elevation of 592.7 feet NGVD.

(16) Energy generated from joint-use pool only, no surplus energy included.

(17) Based on period of record routing 1925 through 1965--41 years.

SECTION 2

FOUNDATION INVESTIGATIONS

A. Investigations Prior to Construction

Over a period of years from 1964 to 1970, the area was surficially mapped and researched in detail. A number of traverses was run with a portable seismic unit and portable resistivity unit by SLD Geology personnel. A more extensive geophysical survey was accomplished in December 1966 to determine the configuration and extent of the buried channel. A total of 376 machine borings was drilled for construction of the main dam covering both Phase I and Phase II; 37 borings were drilled in Borrow Area No. 1, 26 in Borrow Area No. 2, 14 in Borrow Area No. 3, 30 in uplands Borrow Area No. 4, 6 in alternate Borrow Area No. 5, 4 in the saddle dam, 25 in the left (north) abutment, 69 in the right (south) abutment, and 165 in the general flood plain area. In addition to these borings, a total of 6 well point piezometers was installed to monitor the groundwater table in the general flood plain; 4 were installed in the flood plain located downstream of the main dam embankment and 2 were located in the downstream Borrow Area No. 1. Four machine-dug test pits were excavated in the uplands Borrow Area No. 4. The purpose of the borings and test pits were:

1. Delineate soil types, obtain samples for testing and determine depositional patterns.

2. Obtain rock samples for testing, determination of stratigraphy, determination of aberrants, solutional activity, weathering, jointing and fracturing.

The difficulties encountered in these exploratory borings occurred principally in penetrating the large boulders nested in the bottom of the buried channel and in penetrating the Mississippian Residual Chert-Pennsylvanian clay shale complex which overlies the Burlington Formation (refer Drawings Nos. 5/2 thru 55/2 for the as-drilled boring locations and foundation descriptions).

B. Investigations During Construction

Comprehensive investigations were performed when a significant foundation problem was encountered during the construction of a particular feature of work. The foundation for Monolith D-14 and the left abutment embankment contact area are two of the more noteable areas which required further examination. The scope of work for each investigation is addressed in the appropriate sections of this Report.

SECTION 3

GEOLOGY

A. Physiography

The reservoir site is situated in the Dissected Till Plains Section of the Central Lowland Physiographic Province. It is a glaciated, maturely eroded plain with only occasional remnants of the original glacial features preserved.

B. Topography

Imperfect, slightly developed Karst topography is exhibited in isolated sections. Sinks, bedrock springs and caverns are fairly common. Springs have a flow of 1 cubic foot per second or less and emerge near the valley floor at the Hannibal Shale-Chouteau Limestone contact. Sinks and caverns are closely related and most of the caves are merely the widened bottoms of sinks. There are no known caverns having horizontal lengths exceeding 100 feet. Upland divides are commonly broad, flat and concordant and have been cleared of timber for agricultural purposes. Drainage ravines are steeply incised and retain their forest cover. Vertical and near-vertical rock bluffs are common along the Salt River and its tributaries.

C. Description of Overburden

1. Residuum

Upland soils are generally a mixture of residuum from Pennsylvanian sediments and thin glacial deposits with no evidence of loessial contributions. As the valleys are approached from the flat to gently sloping upland areas, the Pennsylvanian sediments thin and the soils are influenced primarily by the underlying Mississippian rocks. At the dam site, a thin topsoil layer of silts and clays overlies a considerable thickness of residual chert which lies upon and grades into the Burlington Limestone Formation. This residual chert varies in thickness from some 20 feet at the edge of the bluffs to 40 feet in the near-bluff and uplands. It may be generally divided into three distinct, but transitional layers. The topmost layer consists of a heavy, red clay containing chert fragments ranging from sand size to cobbles. The middle layer is predominately chert, in fragments up to boulder size, in a matrix of red clay and mostly multi-colored Pennsylvanian clay-shales. The lower layer, or mixed zone, consists of limestone and chert boulders and rock stringers in a matrix of the clay-shales. Both the limestone and chert boulders and fragments exhibit a wide range of weathering with some ledges remaining attached to the bedrock.

2. <u>Talus</u>

Rock fragments, blocks and boulders dislodged from the limestone bluffs rest against the Hannibal and Chouteau Formations up to an average elevation of 560.0 feet NGVD. These rock fragments are encased, very lightly, in a matrix of clayey slope wash and rest upon the alluvial deposits or weathered shale in the valley floor.

3. <u>Glacial Features</u>

Glaciation of the Nebraskan and Kansas periods was responsible for the deposition of till, sands and gravels, boulder zones, as well as, sculpturing much of the bedrock topography at the dam site. The only occurrence of glacial material at the dam site has been found in the boulder zone of the buried channel. Glacial features which are significant to the design of the dam are the till-filled saddle, Borrow Areas Nos. 4 and 5, and the buried channel. Just to the north of the main dam site, a preglacial valley of the Salt River was blocked by the ice and subsequently filled with a considerable thickness of till which surpassed the heights of the former valley walls. Due to a slight amount of erosion, this saddle was the site of a small dam raised to conform in elevation to the top of the main dam. Borings drilled in the saddle have encountered up to 130 feet of bluish gumbo and clayey-gravelly till overlying some 18 inches of glacio-alluvial sand and gravel. The deepest bedrock encountered beneath the saddle was the Hannibal Shale at approximate El. 502.0 feet NGVD. The upland till deposit between the dam site and saddle has been explored and designated as Borrow Area No. 4. The upstream flank of the till-filled saddle has been designated as Borrow Area No. 5. Explorations disclosed a deep depression in the rock surface of the Salt River flood plain delineating a deepened channel which is the result of glacial or post-glacial scour. Based on the boring data, the buried channel appeared to have a normal gradient with no indications of potholes, barriers or side valley extensions. A rather large pothole was encountered at the northwest, upstream corner of the core trench. Incised in the main valley bedrock, some 50 feet, the old channel walls appeared to slope gently on either side, but when exposed were nearly vertical in the core trench area. The channel was cut entirely through the Hannibal Formation and bottoms in the Louisiana Formation. From approximately El. 480.0 feet NGVD to El. 460.0 feet NGVD, there was a heavy concentration of very large, nested boulders and slump blocks from all of the rock formations present except the Louisiana.

4. Valley Soils

Outside the limits of the buried channel are found approximately 30 feet to 40 feet of alluvial and colluvial deposits. The colluvial deposits are located adjacent to the base of the valley walls. The alluvial deposits of the flood plain exhibit the typical characteristic discontinuities and complex stratification normally associated with this type of deposit. The upper, major stratum, some 10 feet to 35 feet thick, consists of impervious brown to gray, silty and sandy, low clays (CL) with occasional pockets and lenses of brown silt and sand. The impervious stratum is found both upstream and downstream from the dam site. Only in the existing river channel is the stratum absent. Downstream of the dam axis an area of ridge and swale topography is encountered near the junction of the present river channel and the left (north) valley wall. Within this area, the sand and silt content of the upper impervious stratum shows a marked increase over upper stratum soils located outside this area. The lower stratum of the alluvial valley soils varies in thickness from 0 foot to 20 feet and consists of brown and gray, sandy gravels with occasional layers of silty and clayey sands. The lower deposits of sands and gravels are absent in the area adjacent to the valley walls. In the present river channel, the depth of the alluvial deposit is approximately 15 feet thick and consists of sand and sandy gravel. The upper, major stratum materials constitute the usable soils in Borrow Areas Nos. 1, 2 and 3.

D. Bedrock Stratigraphy

Rocks underlying the project area in northeast Missouri include representative formations varying in age from Middle Ordovician to Pennsylvanian. Ordovician Formations are exposed on the lower reaches of the Salt River and are overlain by Silurian or Devonian Formations and at the dam site by unassigned Mississippian-Devonian rocks. Mississippian Formations crop out extensively in the reservoir area and at the dam site. The Residual Chert, sinks filled with Pennsylvanian sediments and Pennsylvanian outliars of clay, shale, sandstone and coal are found on the uplands. The stratigraphic succession at the dam site from youngest to oldest is: Residual Chert (containing Pennsylvanian sediments); Burlington Limestone, Chouteau Limestone and Hannibal Shale (all Mississippian); Louisiana Limestone, Saverton Mudstone and Grassy Creek Shale (unassigned Devonian-Mississippian) and the Callaway Limestone (Devonian). It was first thought that the Saverton and Grassy Creek Formations were absent in the foundation for the concrete structure; however, during excavation, these formations were encountered at depths of 2 feet to 3 feet beneath the Hannibal/Louisiana contact. The presence of these formations was verified by a team of State geologists. Although the concrete structure is actually founded upon the Callaway, the documents for this Report were not changed in order to maintain continuity between the past contract correspondence and the contract documents. In addition, the lithologic descriptions and engineering properties presented in this Report are not affected by the misclassification.

E. Bedrock Structure

In Pike and Lincoln Counties, lying just southeast of the project area, a major upfold designated as the Lincoln Fold, is the major structural feature in the vicinity. An extension of this fold with the axis aligned northwest-southeast passes the dam site area some three miles to the east and controls the attitude of the sedimentary rocks underlying the dam site. Since the anticline is plunging to the north, the effects on the rocks at the site are slight and the bedding can be considered as horizontal for all practical purposes. No evidence of faulting has been discovered. Joint patterns are well defined in the Burlington and Chouteau Formations and relatively defined in the Louisiana. From a borescope survey performed in the right (south) abutment, it was determined that the Hannibal Shale contained no joints or fractures where confined by the full thickness of both overlying limestones. In the valley, however, relief jointing was very evident.

F. Bedrock Weathering

The depth and degree of weathering in the upper limestones is slight because of the relatively impervious cover. Where exposed as vertical cliffs, the degree and depth of weathering is upward of 20 feet, particularly along open exposed joints. The depth of weathering of the Hannibal Shale varies from zero up in the abutments to 25 feet to 30 feet beneath the talus slopes to between 3 feet and 10 feet in the valley. The depth of weathering was very slight in the Louisiana Limestone exposed in the bottom of the buried channel.

G. Leaching and/or Solution Activity

Springs are abundant in the reservoir area. All springs located have been flowing from the Burlington or Chouteau Formation from El. 550.0 feet NGVD to El. 590.0 feet NGVD. Most are principally "wet-weather" type springs and issue from fissure type openings. There are a number of caves and sink holes in the reservoir area, but those located should have no affect on the reservoir. Pressure test water losses in borings indicate the Burlington and Chouteau Formations are solutioned along joints and seams from approximately El. 555.0 feet NGVD to El. 640.0 feet NGVD.

H. Ground Water

Ground water in the vicinity of the reservoir is obtained from wells and springs. Shallow sources are glacial till and alluvium in the Salt River and its tributaries. Deep sources of ground water are the Burlington and Chouteau Formations, Kinderhookian Limestones, Kinderhookian or Devonian silty shales and the St. Peter Sandstone. The majority of wells drilled on farms in the area produce from the Burlington-Chouteau. There appears to be three levels of water in the vicinity, one near the surface in till, approximately 80 feet below the surface and approximately 120 feet below the surface. The quality of the ground water in the vicinity varies greatly in mineralization. Many wells produce water too heavily mineralized for human consumption. This was the case with the three wells drilled in the upland for this contract (DACW43-71-C-0063).

I. Engineering Characteristics of the Overburden Materials

Based on considerable testing and a conservative selection, the design strengths to be used were: "Q" Test: $\phi = 0$, $\underline{C} = 1.15$ TSF; "R" Test: $\phi = 15°$, $\underline{C} = 0.30$ TSF; "S" Test: $\phi = 28°$, $\underline{C} = 0.0$ TSF. Stability analysis was performed on two basic sections which were considered to be typical of the entire embankment. The slopes were initially checked by the wedge method and were confirmed by the arc method. As a supplement to the major studies, three special studies involving the end of construction case were made to determine the effects of the following features on the overall stability of the structures: the initiation of dual embankment construction stages; the reduced shear strength assigned to the Hannibal Shale and the use of material from the construction of relocated S-JB on the downstream berm.

Settlement studies were made for three typical embankment sections to determine the required overbuild. Sections were located, based upon soil stratification, embankment height and rock elevation. Consideration of the most feasible sequence of borrowing and the available quantity of borrow from each source resulted in the use of the consolidation test data on representative valley borrow soils for that portion of the embankment located below El. 573.0 NGVD and upland borrow testing for that portion located above El. 573.0 NGVD in determining embankment settlement. Based on these analyses, a maximum overbuild of 12 inches will be provided at the centerline and along the upper embankment slopes.

J. Engineering Characteristics of the Bedrock Materials

The laboratory testing of the foundation rocks consisted of: 9-Poisson's Ratio; 8-Single-plane Repetitive Direct Shear; 85-Unconfined Compression; 47-Modulus of Elasticity; 121-Single Direct Shear; 13-Triaxial Compression; 15-Brazilian Tensile and 20-Double Direct Shear Tests. The design values assigned to the Burlington and Chouteau Formations from their portion of the above tests were:

Property	Burlington Formation	Chouteau Formation
Unit Weight	155 pcf	155 pcf
Modulus of Elasticity	6.6×10^6 psi	3.6×10^6 psi
Unconfined Compressive Strength	9,500 psi	9,500 psi
Shear Strength:		
Peak	C = 16.x TSF, Ø = 45°	C = 16.2 TSF, Ø = 45°
Residual	C = 3.0 TSF, Ø = 45°	C = 3.0 TSF, Ø = 45°
Concrete-Rock	C = 16.2 TSF, Ø = 45°	C = 16.2 TSF, Ø = 45°

The design values for the Hannibal Shale are:

Unit Weight	150 pcf
Modulus of Elasticity:	
Abutment	200,000 psi
Valley	27,000 psi
Unconfined Compressive Strength:	
Abutment	1,250 psi
Valley	140 psi
Shear Strength- Valley:	
Peak Undrained	C = 4.5 TSF, Ø = 26°
Residual Undrained	C = 1.6 TSF, Ø = 20°
Peak Drained	C = 1.4 TSF, Ø = 19°
Residual Drained	C = 0.0 TSF, Ø = 19°
Concrete-Shale	C = 0.0 TSF, Ø = 19°
Anchor Grout-Shale	C = 2.25 TSF

The design values for the Louisiana Limestone are:

Unit Weight	150 pcf
Unconfined Compressive Strength	9,500 psi
Modulus of Elasticity	3.0×10^{6} psi
Shear Strength:	
Peak	C = 16.2 TSF, Ø = 45°
Concrete-Rock	C = 16.2 TSF, Ø = 45°
Residual	C = 3.0 TSF, Ø = 45°

Although the Burlington and Chouteau Limestones differ considerably in many of their geologic properties, their engineering behavior is sufficiently similar to warrant combined discussion. Both limestonesare sufficiently strong so as to present no problems, in bearing capacity or sliding stability, to monoliths founded upon them. A sufficient number of tests was performed to indicate the argillaceous or shalely zones in the Chouteau do not constitute planes of significant weakness within the formation. Joints and fractures constitute the principal zones of weakness in these formations. In addition to the conservative low design strength of the Hannibal Shale, its air-sensitivity and the existence of soft zones within it have exercised profound influence on the design of the dam. The air-sensitivity has been found to vary widely within the dam site area. Three distinct zones of soft shale can be detected. The first occurs at the top of rock and is synonymous with the weathered shale zone in the valley. In the very near abutments softening occurs at the Chouteau-Hannibal contact. The third zone occurs in the lowest 10 feet of the shale out in the valley and is apparently a reflection of original lithology, subsequent weathering and water migration within the upper 5 feet of the Louisiana Limestone. Laboratory testing has indicated that the Louisiana Formation is a competent foundation member and that the numerous shale partings being extremely irregular do not constitute any real plane of weakness within the formation. However, near the top of the formation, a persistent zone of shale or shalely limestone exists directly below the top bed of the Louisiana. As previously noted, this shale seam was later identified during construction as the Saverton/Grassy Creek Formation.

SECTION 4

UNWATERING

A. Introduction

The contract unwatering specification was a performance specification which required the Contractor to design and furnish an unwatering system capable of allowing all work to be performed in the dry. The specification criteria required that all surface water and seepage be controlled and that the groundwater be lowered a minimum of 5 feet below excavated slopes and foundation surfaces. The Contractor used the required channel plugs, cofferdams, upstream cutoff trench, ditches, sumps, deep wells and assorted pumps during excavation (1973-1977) of the concrete structure and for main dam fill placement (1978-1983). (Refer Drawing Nos. 57/2, 58/2 and 59/2 for the cofferdam sequence, Drawing No. 60/2 for details of the unwatering system and Aerial Photograph No. 195/2 for an overall view of the unwatering operation).

B. Concrete Structure

1. Water Temperature Control Weir

On 22 October 1973, the Contractor commenced construction of the upstream river channel plug. The plug extended from the right abutment to approximate dam axis Station 3+60, where it tied into the second-stage cofferdam (refer Drawing No. 60/2 for exact location and detail). Construction of the river plug consisted of excavating river channel and flood plain deposits to the Hannibal Shale, and the placement of semi-compacted impervious material by scrapers to El. 540± feet NGVD. A CAT D8 dozer established the 1V:3H side slopes; all construction was completed on 10 November 1973. The river channel downstream of the channel plug was then unwatered by a diesel-driven 6-inch pump which discharged into the diversion channel.

Excavation of the upstream cutoff trench, which would lie directly underneath the water temperature control weir, followed immediately. The cutoff trench was constructed along the upstream face of the structural excavation. The trench which had a 10 foot base width and extended through the natural sands and gravels to the top of rock, had the primary purpose of providing a seepage cutoff during construction of the concrete monoliths. The trench originates at approximate Station 11+25, extends upstream from the dam centerline for approximately 400 feet and then turns south until it intersections the right abutment. Refer to Section 5 for construction and equipment details.

In order to dewater the cutoff trench, Luhr Bros., Inc., installed a well point system designed by Stang Hydronics of Tulsa, Oklahoma. The system consisted of 87 well points installed on approximate 6-foot centers at Elevation 510± feet NGVD on the upstream side of the trench. The well points connected to a 6-inch diameter header line at Elevation 520± feet NGVD which ran to a 1,200 gpm Stang pump. On 15 April 1974, the upstream well point system began operating with all water being discharged to the diversion channel. The unwatering system was used as needed until 9 July 1974 when it was disassembled since fill elevation in the cutoff trench had reached a sufficient height above the water table (refer Drawing No. 60/2 for location and dimensions of the upstream plug and cutoff trench). Surface runoff was controlled by a 1 foot deep interceptor ditch, adjacent to and a maximum of 5 feet from the upstream side of the cutoff trench. All water was pumped from the ditch using a 3 inch trash pump. After the clay core in the trench was started, the interceptor ditch was backfilled with sand and compacted.

2. Deep Wells

Two 24-inch diameter, gravel-packed, deep wells were constructed by Luhr Bros., Inc. to control groundwater from the downstream aquifer. Each well consisted of 30 feet of 16-inch diameter perforated pipe and a 16-inch diameter riser pipe. An International diesel-driven 475 gpm. 8-inch diameter, submersible pump was installed in each well which discharged water into the diversion channel. The first deep well (W-1) was installed on 17 December 1973, at Station 10+58, Offset 724 feet downstream, bottom Elevation 460± feet NGVD. A second well (W-2) was installed on 4 January 1974, at Station 8+99, Offset 861.3 feet downstream, bottom Elevation 460± feet NGVD. The wells were located within the confines of the buried channel to maximize drawdown. A third deep well was started, but abóndoned when problems arose with the drilling. Sand infiltration tests were performed immediately after installation and every six months thereafter. On 15 February 1974, pumping of the deep well unwatering system began and continued on an "as-needed" basis. The quantity of effluent was measured twice daily and recorded. Upon completion, Well W-2 was grouted using a non-shrink grout and Well W-1 was kept open for use by the Contractor as a water well for embankment operations.

Nine temporary well point piezometers were installed to monitor the two deep wells (refer Drawing No. 60/2 for locations). The piezometers were constructed of a plastic screen attached to a 1 and ½-inch galvanized riser pipe. The piezometers were tested by performing a falling head test. Piezometer levels were read and recorded twice daily by the Contractor.

When monitoring was no longer required, Piezometers Nos. S-1, S-2, S-3, S-4 and B-1 were pulled. Piezometers Nos. D-1_A, D-2, D-3 and C-D_4 were back-filled with non-shrink grout. An analysis of the piezometer readings indicates that the buried channel acted as a very large sump into which water from the sands/gravels outside the buried channel flowed into when it was pumped by a deep well. Once the water level in the buried channel was drawn down it had a very slow recharge. Consequently, as long as drawdown was maintained below the top edge of the buried channel there appeared to be little direct relationship between the pumping rate and water level in the overburden piezometers (S-1 thru S-4).

3. Cofferdams

Construction of the second-stage cofferdam began on 22 October 1074, and continued intermittently until 16 January 1975. The cofferdam was constructed to Elevation 551± feet NGVD with a 30-foot wide top and sides having a 1V:3H slope. The Contractor was directed to raise the cofferdam 2 feet to Elevation 553 feet NGVD to provide additional protection. The second-stage cofferdam was divided into three parts. The downstream segment was constructed of material excavated from the right abutment and exit channel. It extended from the right abutment to approximate dam axis Station 13+50, Offset 200 feet downstream, where it tied into a combination of permanent and temporary foundation fill. This fill (second segment) extended approximately 300 feet upstream (to Station 8+35, Offset 100 Upstream), and was composed of approximately 10 feet of permanent fill (second-stage cofferdam fill) and approximately five feet of temporary fill for Massman Construction Co.'s concrete batch plant and associated buildings. The temporary fill was removed and either placed in random fill or was wasted. From the upstream end of the foundation fill to a point where the second-stage cofferdam intersected the Water Temperature Control Weir (approximate Station 10+100, Offset 400 feet upstream), the cofferdam was built as part of the permanent embankment. From the Water Temperature Control Weir to where it merged into the upstream river channel plug, the second-stage cofferdam was constructed of semi-compacted impervious materials. This last segment of the second-stage cofferdam served as a temporary cofferdam (approximately 1 year) until the Water Temperature Control Weir could be constructed to Elevation 551 feet NGVD (Refer Drawing No. 60/2 for locations and details).

4. Surface Water Control

Surface water was controlled by the use of sumps, ditches, sandbags, assorted-sized pumps and collector pipes. Sumps were constructed in several areas inside the cofferdam, with three being semi-permanent. The main sump was approximately 200 feet long, 50 feet wide, with the center at Station 7+50+, Offset 700 feet downstream. Excavation of the mainsump began in June 1974, and continued intermittently until Elevation 505± feet NGVD was reached. The 730 gpm sump pump was equipped with an electrically operated float-controlled switch. The main sump acted as a collection point for all surface water collected from ditches and the sumps located in the stilling basin and tailrace. In addition, a diesel-driven 6-inch pump was used to supplement float-control pumping during periods of heavy rain.

Surface water which collected in low areas upstream of the batch plant was drained into the diversion channel through a 12-inch pipe. The gravity flow pipe was equipped with a flap gate which prevented reverse flow during periods of high water in the diversion channel.

Surface water and seepage from the main dam excavation were discharged upstream and downstream from a complex unwatering system. Along the excavation perimeter of the main dam excavation, (Monoliths D-1/2 thru D-11/12), a 10-foot wide berm was excavated at Elevation 485± feet NGVD with a flat-bottom paved ditch being incorporated into the berm to control surface runoff. The collected runoff was discharged to the main sump area by a 1,650 gpm sump pump located on the north edge of the tailrace wall. Two flat-bottom ditches were excavated downstream between the stilling basin wall and the splitter wall. The collected surface water in the ditches flowed into the shotcreted swales which drained into the sump at the downstream end of the splitter wall or a sump at the downstream end of the tailrace wall. The water in either case was discharged by a 6-inch pump into the main sump area. After concrete placement of the monoliths began, a temporary sump was constructed approximately 90 feet upstream of the centerline near Station 5+00. It discharged into an interceptor ditch which was excavated upstream of dam axis along the right abutment at Elevation 522± feet NGVD. The ditch intercepted runoff from the bluff below the south overlook and carried it to the diversion channel upstream. It should be noted that until excavation of the monoliths was completed, some sumps and ditches were shifted to allow construction of required haul roads.

C. Embankment

1. Cofferdam

The purpose of the third-stage cofferdam was to protect the main dam embankment from a possible 10-year flood during its early construction. Construction began in April 1978, utilizing scrapers and dozers, and ended late in the construction season of 1979. The cofferdam was constructed from Borrow Area No. 2 material (Refer Drawing No. 4/2 and 58/2 for detailes).

Construction of the third-stage cofferdam from the left abutment (approximate dam axis Station 18+36±, Offset 820 feet upstream), to dam axis Station 13+10±, Offset 955 feet upstream, consisted of semi-compacted impervious clay. From the latter station to the Water Temperature Control Weir (approximate dam axis Station 9+00±, Offset 386 feet upstream) the cofferrdam had a semi-pervious core covered by an impervious clay blanket. Generally, the top and upstream slopes had a minimum 10-foot impervious clay blanket and the downstream slop had a minimum 5-foot impervious clay blanket. The upstream slope of the entire third-stage cofferdam was covered with thirty six inches of limestone revetment. The limestone had been excavated from the left abutment and stockpiled for such use. By Modification No. P00140 the Contractor was directed to raise the cofferdam 4 feet to Elevation 581± feet NGVD due to high water in May 1981. This was constructed with semi-compacted impervious material from the Highway S-JB north excavation stockpile. By Modification No. P00145 the Contractor was directed to raise the cofferdam an additional 4 feet to Elevation 585 feet NGVD due to the July 1981 flood. The material came from the above source. Following the July 1981 flood, the third-stage cofferdam, within the limits of the "notched area", was rebuilt to Elevation 580± feet NGVD. The rest of the cofferdam was degraded to Elevation 580± feet NGVD by dozers pushing the material onto the slopes. The upstream portion of the third-stage cofferdam, from left abutment to approximate dam axis station 13+10, was degraded to Elevation 550± feet NGVD or original ground surface, whichever came first, during the 1983 construction was required by the specifications. The excavated material was placed in the upstream berm.

2. Channel Plugs

The Contractor constructed two permanent channel plugs and one temporary channel plug. The upstream temporary plug was installed in the diversion channel to help redivert the river through the three sluices within the conrete structure. On 20 July 1979, work began on the permanent upstream plug. It extended from approximately 800 feet upstream, dam axis Station 7+40±, to 1,040 feet upstream, dam axis Station 8+90, and was constructed of semi-compacted impervious material from the Step 1 channel plug, associated ramp and Borrow Area No. 2 stockpile. A trench on the downstream toe of the plug was excavated and subsequently backfilled with semi-compacted impervious material. Work on the permanent downstream channel plug commenced on 15 July 1979. It was built with 1V:3H slopes to EL. 550± feet NGVD with a 20-foot wide top and located approximately 650 feet downstream of dam axis. Earthen material used came from Borrow Area No. 2 stockpile and the Water Temperature Control Weir diversion notch. The upstream slope was later incorporated into the embankment impervious zone while the downstream slope merged into the downstream ranaom fill material (Refer Drawing No. 58/2 for general location and detail on channel plug construction).

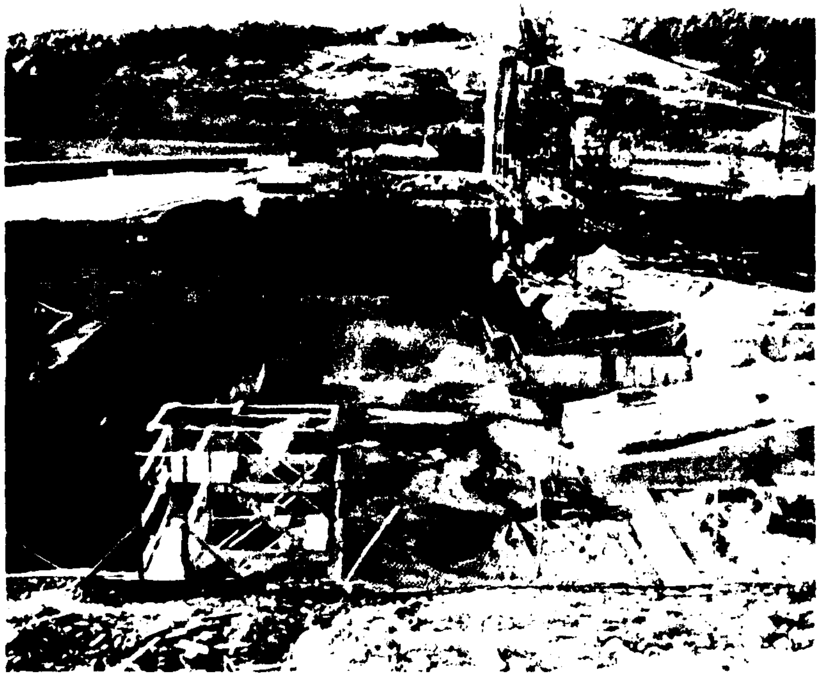
3. Deep Wells and Piezometers

Two 24-inch diameter, gravel-packed, deep wells were installed to Elevation 480± feet NGVD by Luhr Bros., Inc. to control possible seepage from the sands/gravels beneath the Phase I fill upstream of the auxiliary cutoff trench, the upstream channel plug and third stage cofferdam. Each well consisted of a perforated 16-inch diameter pipe and a 16-inch diameter riser pipe. A diesel-driven 8-inch International

475 gpm submersible pump was installed in each well. Then discharge was pumped over to the upstream third-stage cofferdam. One deep well was installed in March 1974 at Station 14+50, Offset 700± feet upstream. A second deep well was installed in April 1979 at Station 15+10±, Offset 660 feet upstream. The deep well system began pumping on 27 July 1979 and continued on an "as needed" basis until November 1979 when it was backfilled with concrete. A Failing drill rig was used to install three temporary well point piezometers to monitor the deep wells (Refer Drawing No. 60/2 for the locations, piezometer tip elevations and installation dates). Immediately after installation, they were tested using a falling head test. Readings were taken twice daily and recorded. They indicated that the water table was held to a minimum of 5 feet below the working surface as per specifications. After use, Piezometers Nos. SE-1 and SE-2 were pulled and Piezometer No. SW-1 was backfilled using non-shrink grout.

4. Surface Water Control

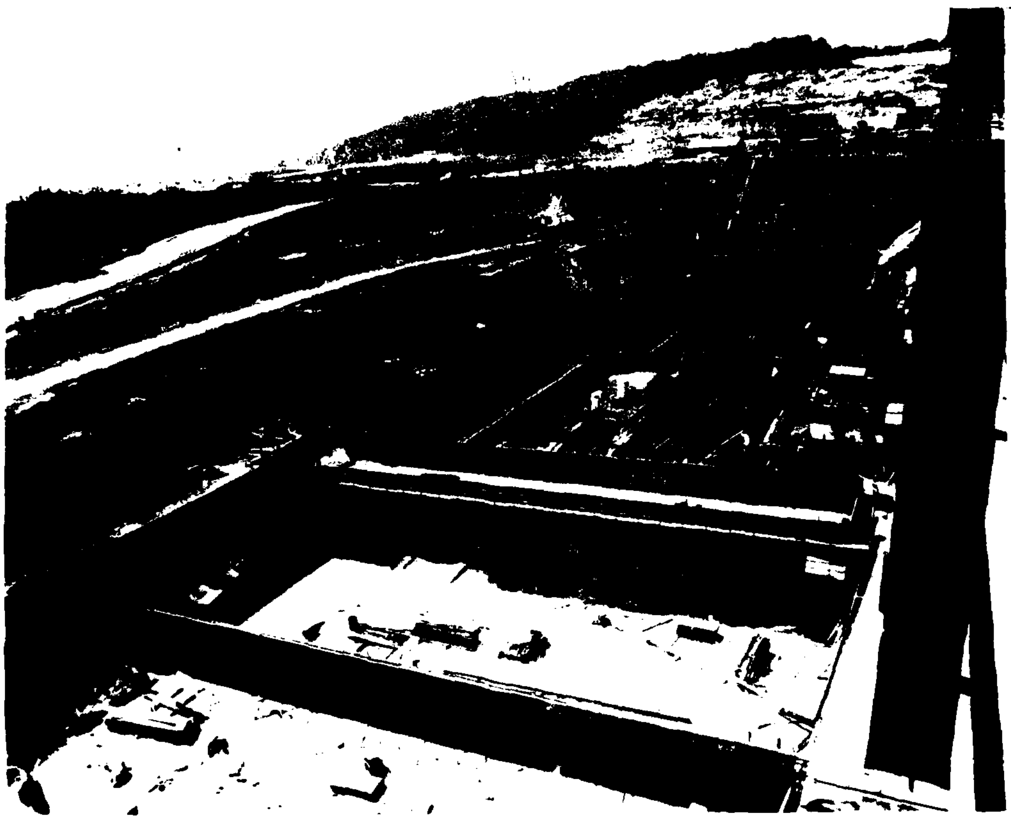
Surface water within the diversion channel during mucking and embankment operations was controlled by the intermittent use of various sized pumps. ditches, trenches and sumps. Removal of the channel water resulting from the construction of the upstream and downstream channel plugs was accomplished by directing the channel water through a temporary notch in the downstream channel plug and by discharging water upstream over the third-stage cofferdam with a 6 inch pump. The remaining areas of ponded water were removed by pumping with a 3 inch trash pump and ditching. During embankment placement, the fill was sloped so that the surface water either drained downstream onto the random fill or upstream into a temporary sump. The upstream sump was located directly upstream of the berm enlargement and directly downstream of the third-stage cofferdam adjacent to the left abutment. A CAT D4 tractor with a 6-inch water pump or a diesel-driven engine with a 8-inch Marlow Pump was used on an "as needed" basis discharging water over the third-stage cofferdam until the cofferdam was degraded in 1983.



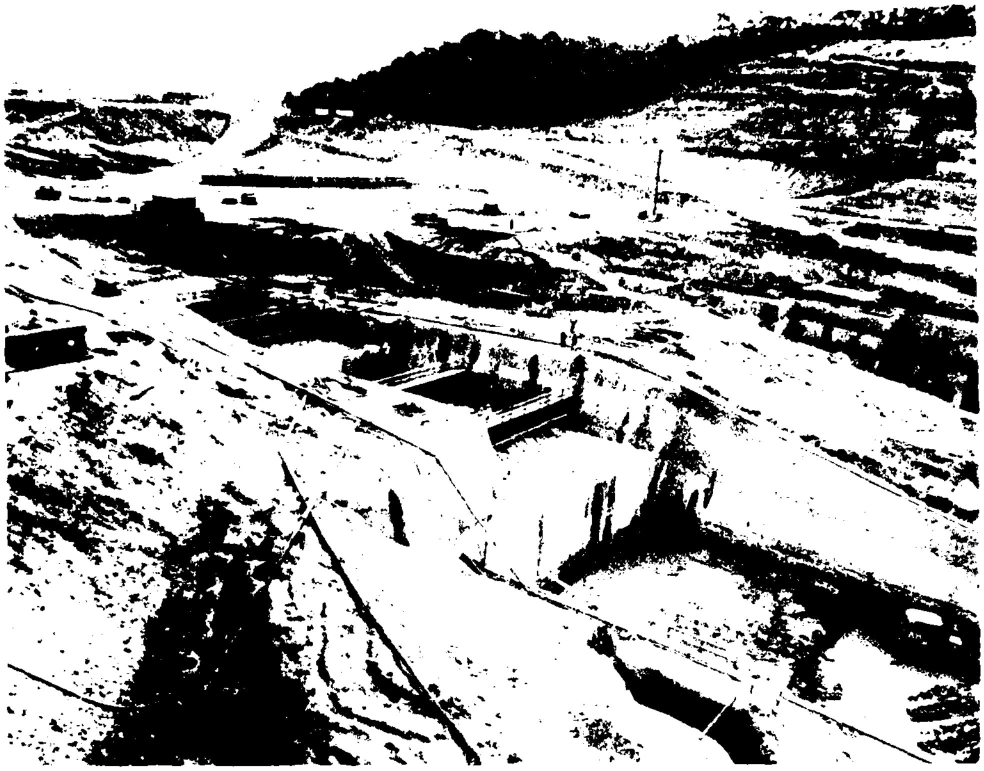
Photograph A. Overall view looking upstream. Note second stage cofferdam in foreground and upstream channel plug in background. Main sump lies directly upstream of backside of cofferdam.

Photograph B. Overall view of concrete structure. Note 4851 berm at base of exposed shale. Ponded water on floor of structure was removed using small portable trash pumps and pumped to main sump.

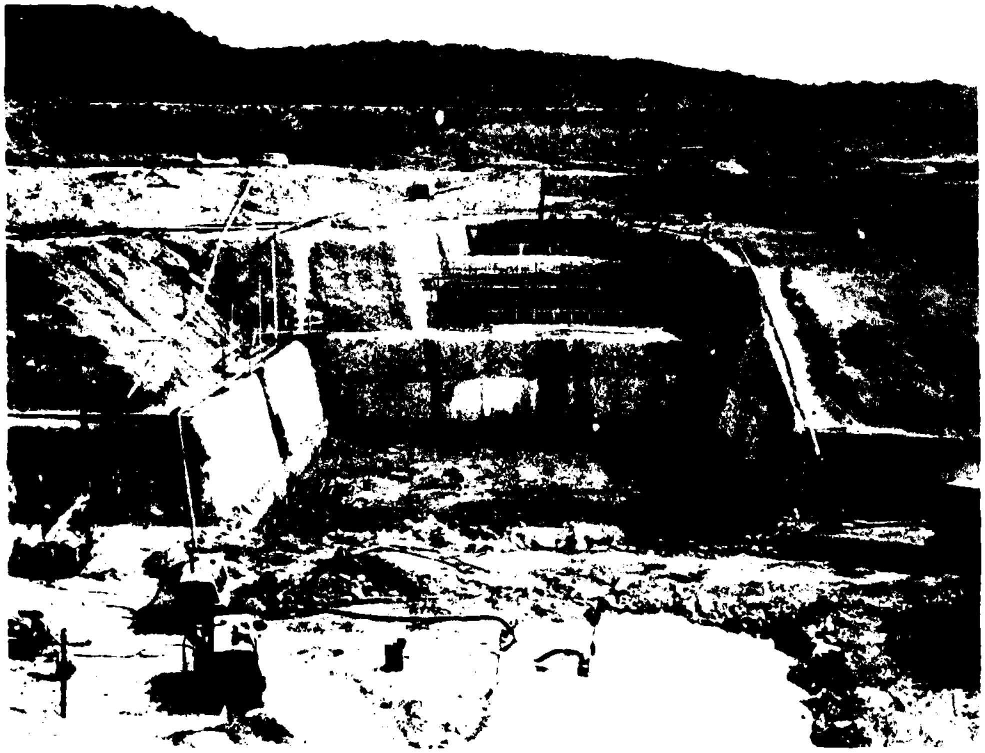
Photograph C. Overall view of dewatering operations in concrete structure. Note interceptor ditch which lies above 4851 berm on upstream and north sides of excavation.



Photograph D. Overall view of dewatering operations in concrete structure. Note header lines used to carry water back to main sump. Lines extend from base of excavation over downstream walls of excavation.

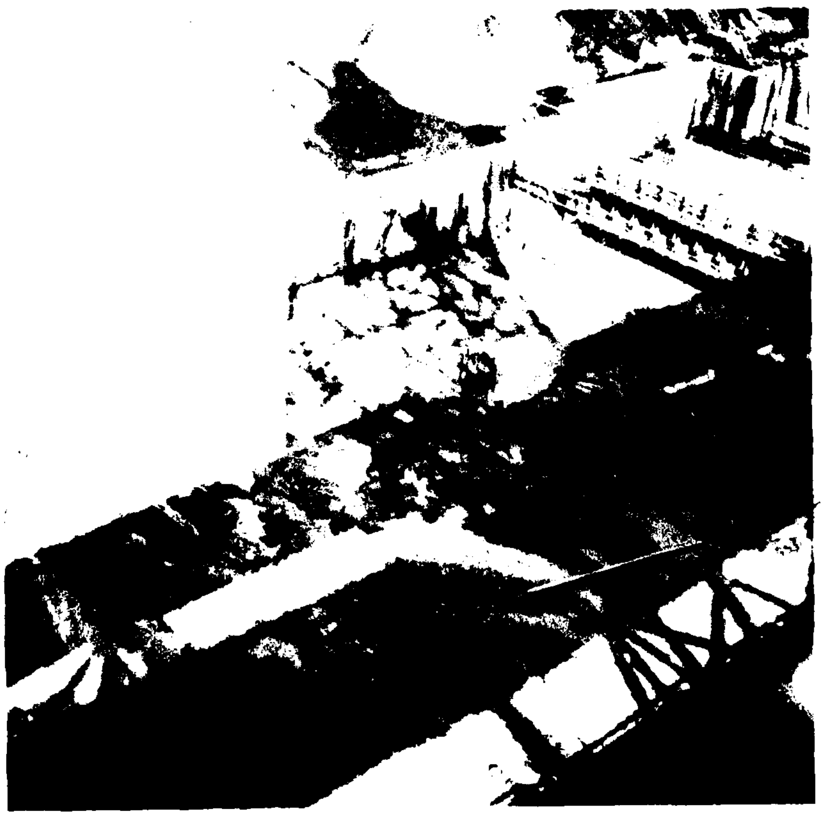


Photograph E. Picture showing dewatering operations. Note header lines used to carry water to main sump. Ponded water on floor of tailrace wall foundation was removed using small portable trash pumps.

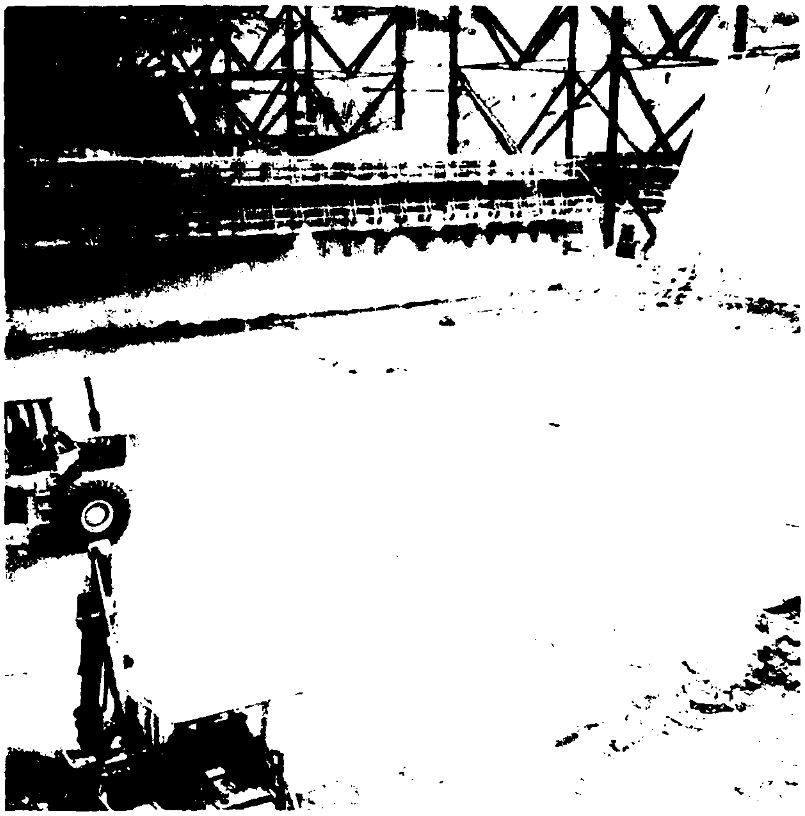


Photograph F. Picture showing dewatering operations in concrete structure. Note large dewatering pump in foreground. Second stage cofferdam in background.

Photograph G. Pumping out ponded water from foundation of TA-1 center section. This area was used as an intermittent sump.



Photograph H. Pumping out ponded from foundations D8-D11. Note 485' berm in upper part of picture.



Photograph I. Ponded water on foundation Monoliths D8-D12. Note header lines extending down to floor of excavation.

Photograph J. Picture shows interceptor ditch along south side of Stilling Basin foundation catching surface water.

SECTION 5

FOUNDATION EXCAVATION

A. Excavation for Concrete Structure and Right Abutment

1. Overburden Excavation (Clays, Sands and Gravels)

Stripping in the area of the structure excavation consisted of removing 340 cubic yards of vegetation in an area from Station 8+00± to Station 11+00±. Stripped material was unsuitable for top dressing and was wasted in Borrow Area No. 3 waste area. Stripping was performed in November 1983. Equipment used included CAT 631 scrapers, CAT 637 scrapers and a CAT D8 dozer.

Excavation of valley overburden started in November 1973 and continued intermittently through March 1976. Valley overburden was comprised of two strata. The upper stratum from El. 540± feet NGVD to El. 520± feet NGVD consisted of silty clays (CL) and sandy clays (CL). The lower stratum from El. 520± feet NGVD to El. 510± feet NGVD consisted of gravelly fine to coarse sands and sandy gravels with occasional layers of silty and clayey sands or gravels. In addition to natural deposits described, the Phase I cofferdam overlaid an area of the clays. Valley overburden excavation was in an area from Station 3+50 to Station 11+00.

Overburden excavation on the right abutment, between Stations 0+50 and 3+50, started in August 1973 and was completed in April 1974. Almost all of this material was unsuitable for use in the embankment and was wasted.

Total overburden excavated was approximately 309,300 cubic yards, described as follows: 81,000 cubic yards (including 16,000 cubic yards of Phase I cofferdam) were clays suitable for use in permanent embankments and were either placed in stockpiles or cofferdams then later used in embankments; 1,600 cubic yards of sands and gravels were stockpiled then later used for structural backfill; 8,700 cubic yards of sands and gravels were hauled to the water temperature control weir and placed between the gabions; and the remaining 218,000 cubic yards of overburden were disposed of in depleted Borrow Area No. 3.

Generally, the following equipment was used (in varying degrees and combinations) for excavation of overburden clays:

Two (2) CAT D8 Dozers
Two (2) CAT D9 Dozers
Two (2) CAT 637 Scrapers
Four (4) CAT 631 Scrapers
One (1) CAT 16F Motorgrader

In some areas, the valley clays that were unstable due to their high moisture content and those clays located in the lower portion (1 foot to 2 feet) of the upper stratum which overlaid silty or clayey sands were excavated by draglines and bottom dumps.

For the most part, the sands and gravels were excavated using the following equipment:

One (1) Bucyrus Erie 38B Dragline
One (1) Bucyrus Erie 88B Dragline
Two (2) Euclid 23TDT Bottom Dumps
Five (5) CAT 630B Bottom Dumps
One (1) CAT 977 Front-end Loader
One (1) CAT 988 Front-end Loader
Two (2) Terex 50-ton End Dump Trucks
Two (2) Euclid FD97 End Dump Trucks
Two (2) CAT D8 Dozers

In some areas where sands and gravels were free of silt and clay and relatively dry, the equipment used for clay excavation was also used for excavation of sands and gravels (refer Drawing No. 61/2 for overburden excavation plan).

2. R. '- Excavation

(a) Introduction

Rock excavation for the concrete structure and the right abutment began on 15 August 1973, and was completed on 28 August 1975. The excavation consisted of the satisfactory removal and disposal of limestone or shale within the limits shown on Drawings Nos. 62/2, 63/2 and 64/2. The scope of work was divided into three areas: (1) excavation of the 1V:1H slope on the right abutment, (2) excavation of the right non-overflow monoliths and (3) excavation of the left non-overflow and spillway monoliths and related structures. The Contractor (Luhr Bros., Inc.) generally utilized a two-shift operation to perform the required excavation which included drilling, blasting, ripping, sawing, rock removal, installation of rock bolts/rock anchors, installation of weep drains and the placement of shotcrete and concrete. The principal equipment used consisted of the following:

CAT D5, D6, D8 and D9 Dozers
CAT D7 Dozer with Blade
CAT 631 and 637 Scrapers
Euclid TDT Bottom Dumps
CAT 630B Bottom Dumps
CAT 950, 977, 980 and 988 End Loaders
Terex Bottom End Dumps
Terex 28-ton End Dumps
Euclid FD 97 End Dumps
Joy 10-R-U Coal Saw
Gardner-Denver ATD3100A Drills
Gardner-Denver 600 cfm, 750 cfm and 900 cfm Compressors

Gardner-Denver AT3700 Drills

Sullair 1,200 cfm Air Compressor

CAT 255 Excavator

Rippers for D8 and D9 Tractors

Terex 50-ton End Dumps

Case 580 Backhoe with End Loader

Case 1470 4-wheel Drive Tractor

CAT 16F Motorgrader

Case 1737 Tractor with End Loader

True Gun-All 2-batch Concrete Mixer

Sigunit Mixer

Cyclone Dry Mix Pump

Joy 105 Compressor

Case 680C Backhoe with End Loader

Case Hydraulic Hoe Ram

Assorted small hand tools, including several jack hammers, jigger drills, picks, shovels and blow pipes.

(b) Right Non-Overflow Area

(1) Limestone Excavation

Rock excavation of the right non-overflow area commenced in August 1973 with the Contractor excavating to the limits and grades shown on Drawing No. 62/2. Excavation began with Monolith D-17 and continued successively until Monolith D-13 was completed. The excavation was divided into two areas: (1) limestone excavation (Monoliths D-17 thru D-15) and (2) shale excavation (Monoliths D-15 thru D-13). The Contractor began the limestone excavation starting with Monolith D-17 utilizing primary blasting methods until El. 652 feet NGVD was reached. This was allowed since the upper limits of the Burlington Limestone were higher than shown on the contract drawings. Upon reaching El. 652 feet NGVD, the Contractor followed the specified sequence for presplitting in advance of primary blasting.

Presplitting operations in the limestone began on 29 August 1973 and were completed on 3 May 1974. Since the specifications stated a maximum depth of 20 feet for presplitting operations, the location of the initial presplit line for each monolith was determined by the number of 20-foot increments (including 1-foot offsets for successive lifts) required to reach grade. Generally, the depth and alignment of the presplit borings were controlled by use of a string line. The spacing of the 3-inch diameter borings was 2 feet with additional presplit holes at the corners of the monolith. The inside corners had three holes on 6-inch centers with two additional holes on 18-inch centers, whereas the outside corners had eight holes at 6-inch centers. These additional holes at the corners were to serve as guide holes with only the even-number holes being lightly loaded.

The types of explosives for presplitting consisted of cartridges of Red Arrow (2-inch by 8-inch) and Trimtex (7/8-inch by 24-inch). These explosives are classified as semi-gelatin dynamites and are manufactured by DuPont. The properties of these blasting agents are listed below.

Properties	Red Arrow Dynamite	Trimtex Dynamite
Weight Strength	70%	65%
Velocity	13,200 ft/sec.	13,200 ft/sec.
Stick Count	44	100
Specific Gravity	1.3	1.3

The Contractor changed his loading procedure as he progressed down the abutment. In monolith D-17, the presplit holes along the upstream and downstream walls were loaded with a full stick of Red Arrow (bottom) and 8-inch cartridges of Trimtex on 12-inch centers. The back wall was loaded with a half stick of Red Arrow and 12-inch cartridges of Trimtex on 2-foot centers. The initial shot for Monolith D-16 was loaded with a half stick of Red Arrow and 12-inch cartridges of Trimtex on 12-inch centers. The remainder of the presplit shots for Monoliths D-16, -15 and -14 utilized 12-inch cartridges of Trimtex only with the spacing being changed to 15 inches for the last shot in Monolith D-16 and 18-inch centers for Monoliths D-15 and -14. The presplit shot utilized stemming (limestone chips) of the entire boring with the collar (top 3 feet) of hole being unloaded. The charge was detonated with instantaneous blasting caps attached to the required length of 50 grain primacord (refer Plate No. 1 for a typical presplit shot).

The exception to the loading patterns described above was the guide holes at the corners of the monolith (D-15 and -14). These borings (even-numbered) received approximately one-fourth of the above charge and the cartridge spacing was increased to 21-inch centers. In addition, when voids were encountered within the presplit borings, that portion of the boring was not loaded.

The Contractor started drilling for primary blasting in August 1973 with the last production shot being detonated on 4 June 1974. As required by the specifications, no primary blasting was permitted within a horizontal distance of 50 feet from a required presplit or line-drilled face until such face had been presplit or line drilled. In addition, lift thickness was limited to a maximum of 10 feet in the upper lifts and to a minimum of 2 feet immediately above the monolithic foundation. The 3-inch diameter borings were drilled vertically. Generally, the production shot drilled pattern consisted of square, rectangular and staggered patterns which varied from 2 feet by 2 feet to 6 feet by 9 feet. The delay patterns were generally arranged in a vee shaped configuration. The Contractor utilized the MS Delay Series (DuPont) ranging from instantaneous (0) to 350 milliseconds (#11).

The types of explosives utilized for primary blasting consisted of cartridges (2-inch by 8-inch) of Red Arrow and 50-pound bags of ANFO-P. The Red Arrow is classified as a semi-gelatin dynamite, whereas ANFO-P is an ammonium nitrate. The specific gravity of ANFO-P is 0.85 g/cc with a loading density of 2.6 lb./ft. for a 3-inch diameter hole. The loading procedure of the holes was as follows:

Bottom - 2-inch x 8-inch cartridge of Red Arrow; 1.25-inch x 8-inch Red Arrow (Monolith D-16, 2-foot foundation lift)

Column Charge - ANFO (holes greater than 5 feet)

Collar - crushed stone chips (stemming ranged from a minimum of 1.3 feet to a maximum of 3.0 feet)

Generally, the only exception to the above loading procedure was if the Contractor encountered a void or mud seam in a boring then that hole or portion of that hole was not loaded (refer Plate No. 2 for a typical shot plan.

The Contractor performed the following operations after the excavation of each production lift (typically 10-foot) until the approximate floor elevation of each monolith was reached.

1) Scaling
2) Installation of rock bolt, rock anchors and weep pipes (not required in Monolith D-17)
3) Shotcrete placement (not required in Monolith D-17)

Generally, the Contractor would utilize his night shift to scale (picks, shovels and compressed air) loose rock fragments/slabs from the monolithic walls. The rock bolt and rock anchor program is discussed in Section 7. The 2-inch diameter weep holes were drilled on 6-foot centers and to a depth of 10 feet. These drains were lined with a perforated PVC pipe and filled with gravel. The shotcrete (wet mix) was applied using a high velocity system (True Gun All Machine) in Monolith D-16 through D-13 with the following design mix:

1) 1 part (cubic foot) Atlas (low alkalic) Type 1 cement

2) 2.3 parts (cubic feet) 3/8-inch crushed rock (Central Stone of Huntington, Missouri)

3) 3.3 parts (cubic feet) washed sand (Missouri Gravel Co. of LaGrange, Missouri)

4) 3 gallons water

5) 1:10 Accelerator/water (Sika-set from Sika Chemical Corporation)

In order to provide reinforcement for the shotcrete, the Contractor first installed steel wire mesh approximately 3 inches from the rock face. The thickness of each application was approximately 3 inches with an allowance for a short setup period between each application (refer Drawings Nos. 62/2 and 64/2 for locations and details).

The final steps in the excavation sequence included foundation treatment of the monolithic floor, placement of a 6-inch protective concrete slab, placement of a 2.5-foot thick reinforced concrete slab and the placement of a 2.0-foot thick reinforced concrete foundation wall.

Section 6 deals with foundation treatment and Drawings Nos. 64/2, 65/2 and 66/2 depict which monoliths received the various types of concrete slabs and/or walls. Generally, the concrete floor slab was tensioned with rock bolts ranging in length from 20 feet to 35 feet. The rock bolts in the high-angle rock walls (15 feet to 40 feet in length) were tensioned prior to the placement of the reinforced foundation walls and then retensioned (rock bolt extension and pipe sleeve) after the placement of the reinforced foundation wall. The bolts for the foundation slab were not tensioned until the concrete had been in place for five days. The bolts for the foundation wall were not tensioned until the concrete had been in place for three days. It should be noted that these items of work had to be completed prior to beginning excavation for the next lower monolith.

Disposal of the shot rock or rock from the scaling operations from each monolith occurred by pushing the material over the abutment face and then by transporting the material to the appropriate disposal area. In many cases, this material was used as temporary fill for the haul roads and the batch plant mound.

(2) Shale Excavation

Shale excavation of the right non-overflow area included Monoliths D-15, -14 and -13. Foundation excavation began on 18 June 1974 and was completed on 20 December 1974. Due to slope stability considerations, blasting (presplitting or primary) was not allowed within the confines of these monoliths and for a distance of 75 feet upstream and downstream.

The Contractor followed the practice of excavating the shale by a combination of sawing and ripping to the lines and grades shown on Drawing No. 62/2. Sawing in 5-foot vertical increments was used to establish the monolith limits, whereas the interior shale was excavated by ripping with a dozer working normal to the dam axis. A string line was used to control the alignment of the saw blade. Each 5-foot saw cut was back-filled with shale cuttings to prevent dessication of the face. Ripping was continued to within 6 inches of final grade, at which time Massman Construction Co. excavated the remaining shale with a backhoe. The excavated material was pushed down the slope and wasted.

After excavation of each 5-foot lift, the Contractor (Luhr Bros., Inc.) was required to scale the vertical faces, apply a bituminous coating, install rock bolts and drains, and then apply 6 inches of shotcrete. Scaling operations were accomplished by small hand tools followed by compressed air. The final surfaces were then covered with a bituminous coating within one hour as per the specifications. The purpose of the coating was to keep the shale from losing moisture during reinforcement of the slopes. Generally, the rock bolts and drains were installed simultaneously with the installation of reinforced steel wire mesh. Shotcrete was then applied in the previously described method within 15 days. After application of the shotcrete, excavation of the next 5-foot vertical lift commenced with the above sequence being repeated until final grade was reached.

The final steps in the excavation sequence included foundation treatment of the monolithic floor, placement of a 6-inch protective concrete slab, placement of a 2.5-foot thick reinforced concrete floor slab and placement of a 2.0-foot thick reinforced concrete foundation wall. Section 6 deals with foundation treatment and Drawings Nos. 64/2, 65/2 and 66/2 depict the construction details for the concrete foundation slabs and foundation walls. Generally, the concrete floor slab was tensioned with rock bolts ranging in length from 20 feet to 35 feet. The rock bolts in the high-angle rock walls (15 feet to 40 feet in length) were tensioned prior to the placement of the reinforced foundation walls and then retensioned (rock bolt extension and pipe sleeve) after the wall placement. The bolts for the foundation slab were not tensioned until the concrete had been in place for five days, whereas the rock bolts for the foundation wall were not tensioned until the concrete had been in place for three days. The purpose of the reinforced foundation walls was to protect and reinforce the high-angle shale slopes against rebound, expansion of existing joints and the creation of new joints.

The excavation, placement of concrete and all applications of bituminous coatings and shotcrete in Monolith D-13 were conducted under a temporary shelter due to inclement weather and low temperatures. The shelter consisted of a reinforced polyethylene covering that extended from the top of the wall to the base. A forced-air heater was provided in order to maintain the internal temperature above freezing since shotcrete could not be applied in freezing weather.

(c) Right Abutment 1V:1H Slope

(1) Limestone Excavation

Rock excavation of the 1V:1H slope commenced in August 1973 and was completed in November 1974. The limits of excavation are shown on Drawing No. 62/2. Rock excavation of the 1V:1H slope was divided into two major areas: (1) limestone (Burlington and Chouteau) excavation and (2) shale (Hannibal) excavation.

Presplitting operations began in August 1973 and were completed by May 1974. The Contractor utilized drilling and blasting techniques similar to those used in the right non-overflow area. The 3-inch diameter borings were drilled at an inclination varying from 50° to 53° and normally to a maximum depth of 20 feet. Presplit hole spacing was generally 4 feet, although 2-foot and 3-foot hole spacings were utilized in those areas where the abutment configuration changed direction. The types of explosives were the same as those used in the right non-overflow area. The loading procedure consisted of the placement of a half cartridge of 2-inch by 8-inch Red Arrow followed by 7/8-inch by 12-inch cartridges of Trimtex on 12-inch centers. The depth (3 feet to 6 feet) to the uppermost cartridge of Trimtex varied depending on hole depth (refer Plate No. 3 for a typical presplit shot plan).

Primary blasting within the confines of the 1V:1H slope was conducted by utilizing the same diameter borings (3-inch), the same type of blasting agents (Red Arrow and ANFO) and the same loading procedure as discussed in the previous narrative for the right non-overflow area. The major difference from the previously discussed shot plans was the type of delay pattern and the production shot pattern. In this area, the Contractor principally used a 6-foot by 9-foot drill pattern; however, square patterns (6-foot by 6-foot and 7-foot by 7-foot) were used for those shots which had considerable variation in depth. Generally, an echelon delay pattern was used since the Contractor had the advantage of a natural free face for each of the production shots (refer Plate No. 3 for a typical production shot plan).

The shot rock from each lift was dozed over the slope and either wasted or used in temporary construction such as haul roads, diversion crossings and overflow weir. After each lift, typically 10-foot, the Contractor would scale the abutment face, install rock bolts and drains, and apply the 6-inch layer of shotcrete (refer Drawings Nos. 65/2 and 66/2 for the shotcrete limits).

Due to Modification No. P00014, the Contractor was required to perform additional rock and dental excavation, and to remove loose rock that had fallen into the open joints or crevices. The areas of additional work were collectively called by the modification as the five "redesigned areas". Areas 1 and 5 required additional rock excavation, whereas Areas 2 through 4 required dental treatment. Excavation in Area 1 consisted of the removal of a rock nose between the limits shown on Plate No. 4. Excavation in Area 5 consisted of the establishment of 10-foot wide benches in 10-foot vertical increments between the limits shown on Plate No. 5. Rock reinforcement and protection were not required in these areas. Areas 2 through 4 (Plates Nos. 4 and 5) received the following types of treatment.

1) Perform precise excavation in voids, joints and rock surface
2) Place shotcrete or concrete filler backfill
3) Apply shotcrete
4) Install weep holes

During excavation of the upstream 1V:1H slope, a fill sink was encountered from 45± feet upstream to 125± feet upstream and extended from El. 640± feet NGVD to El. 625± feet NGVD. The corrective measures consisted of excavating a 20-foot wide bench at approximate El. 640 feet NGVD and the installation of numerous rock bolts around the perimeter of the sink. None of the bolts were stressed since this could have led to further instability. While installing the rock bolts below El. 625 feet NGVD in the sink area, a number of voids were encountered, consequently, Continental Drilling Company was mobilized to grout these voids with a sanded grout.

(2) Shale Excavation

Excavation of the Hannibal Shale began in April 1974 and was completed by 25 October 1974 with the last shotcrete being applied on 6 November 1975. For the most part, shale excavation was performed by ripping and sawing (5-foot increments) to the lines and grades shown on Drawing No. 62/2. The exception to these methods occurred on the upstream 1V:1H slope which was line drilled due to the radius of curvature of the slope being too short for the saw to operate. After excavation of each lift, the Contractor would scale the newly exposed face, perform dental treatment (joint preparation), apply a bituminous coating, install rock bolts/rock anchors and then apply the shotcrete. The method of shotcrete application was the same as previously discussed for the right non-overflow area (refer Drawings Nos. 65/2 and 66/2 for the general limit of shale excavation).

d. Concrete Structure

(1) Shale Excavation

This section deals with rock excavation for the left non-overflow monoliths (D-1 thru D-16), the spillway monoliths (D-7 thru D-12), the powerhouse, tailrace and tailrace wall, splitter wall and, lastly, the stilling basin and stilling basin wall (refer Drawings Nos. 62/2, 63/2 and 64/2 for locations and excavation details). Foundation excavation began in April 1974 and was completed in August 1975. The rock excavation consisted of two parts: (1) shale excavation and (2) limestone excavation.

Shale excavation was performed by utilizing three excavation methods; ripping, sawing and line drilling. The Contractor began excavation of the

1V:1H slope from El. 510± feet NGVD to El. 485± feet NGVD along the upstream perimeter of Monoliths D-12 thru D-1 to the tailrace wall (refer Drawing No. 62/2). A series of inclined guide holes (45°) was drilled on 6-foot centers along the perimeter from the top of the Hannibal Shale (El. 510± feet NGVD) to El. 485± feet NGVD. The purpose of these borings was to establish the limits of the exterior shale slope during ripping operations. Coincident excavation in the interior of the monoliths was also being performed by ripping to El. 485± feet NGVD. Generally, all ripping operations adjacent to the shale slope were performed perpendicular to the dam axis in order to minimize any potential damage due to the existing joints and their orientation. When El. 485± feet NGVD was reached from the upstream perimeter to the tailrace wall, a 10-foot wide berm was constructed to collect surface water and seepage from the shale.

The Contractor followed the practice of excavating the shale within the interior of Monoliths D-1 through D-12 (including powerhouse) by sawing (5-foot increments), line drilling and ripping from El. 485 feet NGVD to 470 feet NGVD. Sawing or line drilling was used to establish the monolithic limits whereas the interior shale was excavated by ripping with a dozer. For the most part, the upstream walls of Monoliths D-1 through D-12 and the powerhouse were line drilled. The exception being some corners which required sawing. The downstream walls of Monoliths D-1 through D-6 and the powerhouse were sawed, whereas the downstream walls of Monoliths D-7 through D-12 were line drilled.

Line drilling consisted of drilling 3-inch diameter dry holes from El. 485± feet NGVD to El. 470± feet NGVD. As per the specifications, these holes were spaced so there was no more than 2 inches of rock between adjacent holes. At distances of 2 feet, the borings were drilled to El. 465± feet NGVD and backfilled with 5 feet of sand. These holes were later used as 5-foot presplit borings within the Louisiana Limestone. After the line drilling was completed, the Contractor ripped the interior shale in 5-foot vertical increments down to the top of the limestone. In those areas which required sawing, the Contractor followed the same procedures as discussed in the right non-overflow area.

Upon exposure of each 5-foot vertical face, the Contractor scaled all loose rock and installed the required rock reinforcement and protection (including shotcreting) in the same manner as discussed in the right non-overflow area. This procedure was repeated until the Louisiana Limestone (El. 470± feet NGVD) was reached.

In addition to the above excavation, simultaneous excavation of the stilling basin, stilling basin wall, splitter wall, tailrace and tailrace wall was being performed. The Contractor followed the same procedures for shale excavation in these areas as previously discussed. Luhr Bros., Inc., was responsible for excavating the shale to within 6 inches of final grade and Massman Construction Co. was responsible for the remaining shale excavation during foundation treatment operations (refer Section 6, Foundation Treatment). As per the specifications, the Contractor only installed rock anchors into the foundation walls for the stilling basin, splitter and tailrace wall structures.

(2) Limestone Excavation

Rock excavation of the Louisiana Limestone (Monoliths D-1 thru D-12, SP1-3, TA1-3 and powerhouse) was performed utilizing drilling and blasting techniques similar to those used in the right non-overflow area (refer Drawings Nos. 62/2, 63/2 and 64/2 for locations and details). The Contractor commenced presplitting operations in August 1974 and fired the last shot in June 1975. Primary blasting began in September 1974 with the last shot being detonated in June 1975.

For the purpose of this Report, the narratives dealing with blasting/excavation are divided into three subareas based upon the final floor elevation. The first area includes Monoliths D-1 through D-12, powerhouse, Monoliths TA2 and TA3 and Monolith SP1-2; the second area includes the powerhouse keyway for the draft tube and sump; and the last area includes Monolith TA1.

In the first area, the 3-inch diameter presplit borings were generally drilled from El. 470 feet NGVD to El. 465 feet NGVD on 2-foot centers except at the monolithic corners where the presplit borings (guide holes) were drilled on 6-inch centers. The types of explosives used were the same as those discussed in the right non-overflow area. Normal loading of the presplit holes were as follows:

Bottom - 12-inch Trimtex plus one-half stick of 1 and 1/8-inch by 8-inch 40% special gelatin

Column - Crushed stone stemming 6 inches to 8 inches above shale seam

Caprock - 8-inch Trimtex with at least 12 inches to 14 inches of stemming to top (12-inch Trimtex on west splitter wall and TA2 and TA3 walls with at least 12 inches of stemming to the top)

The shale seam referred to was a continuous 6-inch to 10-inch thick shale seam throughout the Louisiana Limestone normally at a depth of 1-foot to 3-feet below the Hannibal Shale/Louisiana Limestone contact.

The term caprock refers to that rock which was above the shale seam. The exception to the above loading procedure was the east and south splitter walls which were loaded as follows: one-half stick of 40% special gelatin, 8-inch stemming (shale seam), 12-inch Trimtex, 18-inch stemming, 8-inch Trimtex and 10-inch stemming (refer Plate No. 6 for typical presplit shot plan).

During primary blasting, the Contractor used either a 2-inch or 3-inch diameter production hole with the majority of the holes being drilled vertically. The principal exception was the production holes in Monolith D-11/12 which were drilled at a 10° inclination from vertical. The orientation of these borings was normal to the adjacent walls. The production lift thicknesses within the limits of Monoliths D-11/12 and SP1-3 were generally 3 feet for the initial lift and 2 feet for the second lift. In Monoliths TA2 and TA3, the production lift thickness was 5 feet. The Contractor only used square production hole patterns which varied principally from 3-foot by 3-foot to 2-foot by 2-foot configuration. In an effort to increase the efficiency of his blasting program, the Contractor used a variety of delay patterns such as the vee shaped delay pattern, the modified vee pattern and the echelon delay pattern. For example, in Monoliths D-1 through D-6, the Contractor used the vee delay pattern for the initial production shot and echelon pattern for subsequent production. In contract to the production shots for the right non-overflow area where the Contractor used electric blasting caps (MS), these shots were delayed with MS Connectors attached to the primacord.

Generally, the production shots witnin the limits of the spillway monoliths were loaded with a single cartridge of 1.25-inch by 8-inch Red Arrow.

In contrast to the six production lifts for the sump, the keyway was shot in two production lifts. The initial lift was from El. 470 feet NGVD to El. 460 feet NGVD, whereas the second lift was from El. 460 feet NGVD to El. 458.8 feet NGVD. The Contractor used 3-inch diameter borings loaded with one stick of 2-inch by 8-inch Red Arrow (bottom) except for those two rows of borings next to the east and west presplit lines which were loaded with one cartridge of 1.25-inch by 8-inch Red Arrow. In the second lift, the Contractor used 2-inch diameter holes loaded with one-half cartridge of 1 and 1/8-inch by 8-inch 40% special gelatin. In both lifts, a square production boring arrangement (first lift 5-foot by 5-foot, second lift 2-foot by 2-foot) was used and delayed with MS connectors in a vee shaped configuration.

In the final area (Monolith TA1) the perimeter was presplit in a single increment (10-foot to 11-foot). The presplit hole spacing, alignment, boring diameter and explosive type were the same as those used in the above-mentioned areas (refer Plate No. 9).

The production lift thicknesses varied from a maximum of 5 feet in the upper lift (El. 470 feet NGVD to El. 465 feet NGVD) to a minimum of 2 feet immediately above the floor. The square production hole patterns ranged from 2.5-foot by 2.5-foot to 5-foot by 5-foot and were delayed either in a vee delay pattern or "sinking shot" pattern with millisecond delay. Normal loading of production borings was similar to the powerhouse keyway for the draft tube and sump (refer Plate No. 10 for typical shot plan and blasting computations).

It should be noted that, in all three areas, the Contractor experienced some problems with "pulling" the rock in the monolithic corners. If the corner did not properly break, then the Contractor first tried to excavate the rock with heavy equipment. If this procedure failed, then the rock would be redrilled with small diameter slanted borings and shot. Most often, these borings received one-half cartridge of 1 and 1/8-inch by 8-inch 40% special gelatin.

The Contractor began initial limestone excavation in the keyway for the draft tube and sump. He then excavated toward the tailrace and spillway wall areas. Further excavation proceeded toward Monolith D-1 and sidewalls with successive excavation of the monoliths toward Monolith D-12. All of the excavated limestone was subsequently wasted.

PROJECT — CLARENCE CANNON DAM & RESERVOIR, ST. LOUIS DIST. | COMPUTED BY | DATE

SUBMITTED BY: DATE:

SCALE = 1" = 10'

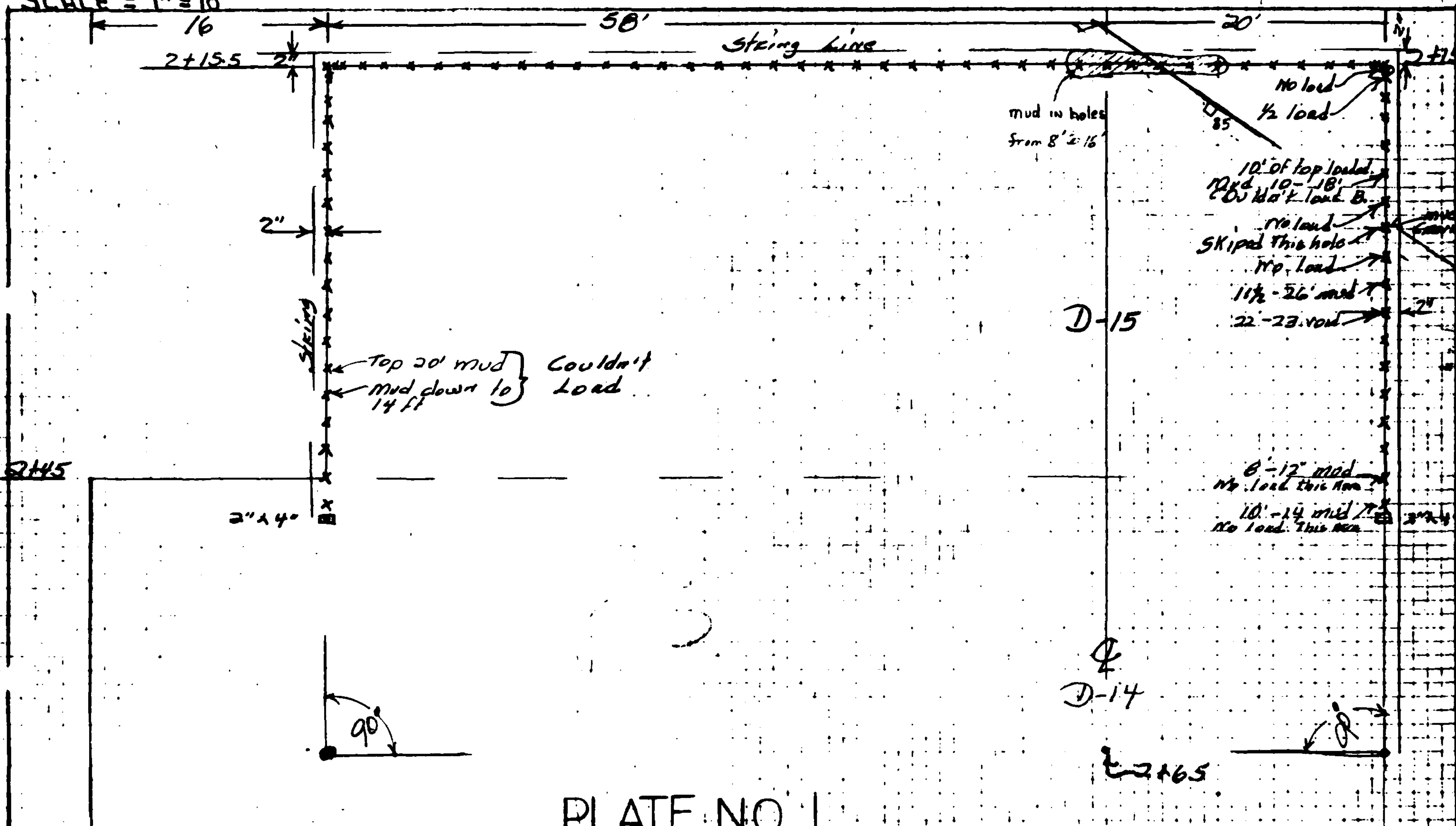


PLATE NO. 1

ROCK TYPE	
HOW DRILLED	PR J 123
HOLE DIAMETER : in	3 inch
HOLE SPACING : in	2' C.C. Except Corners
EXPLOSIVE	Trimlex + Red Arrow
STEMMING	3 foot
DEPTH of HOLE	28' ±
FIRING DATA	Inst @ Bottom

PRODUCTION

Back Wall D-15 loaded 12" stick and 18" Space, Except holes 6" from Corners ½ load = 6" Trimlex 21" Space.
U.S. wall and D.S. wall loaded the same.

CONTROLLED

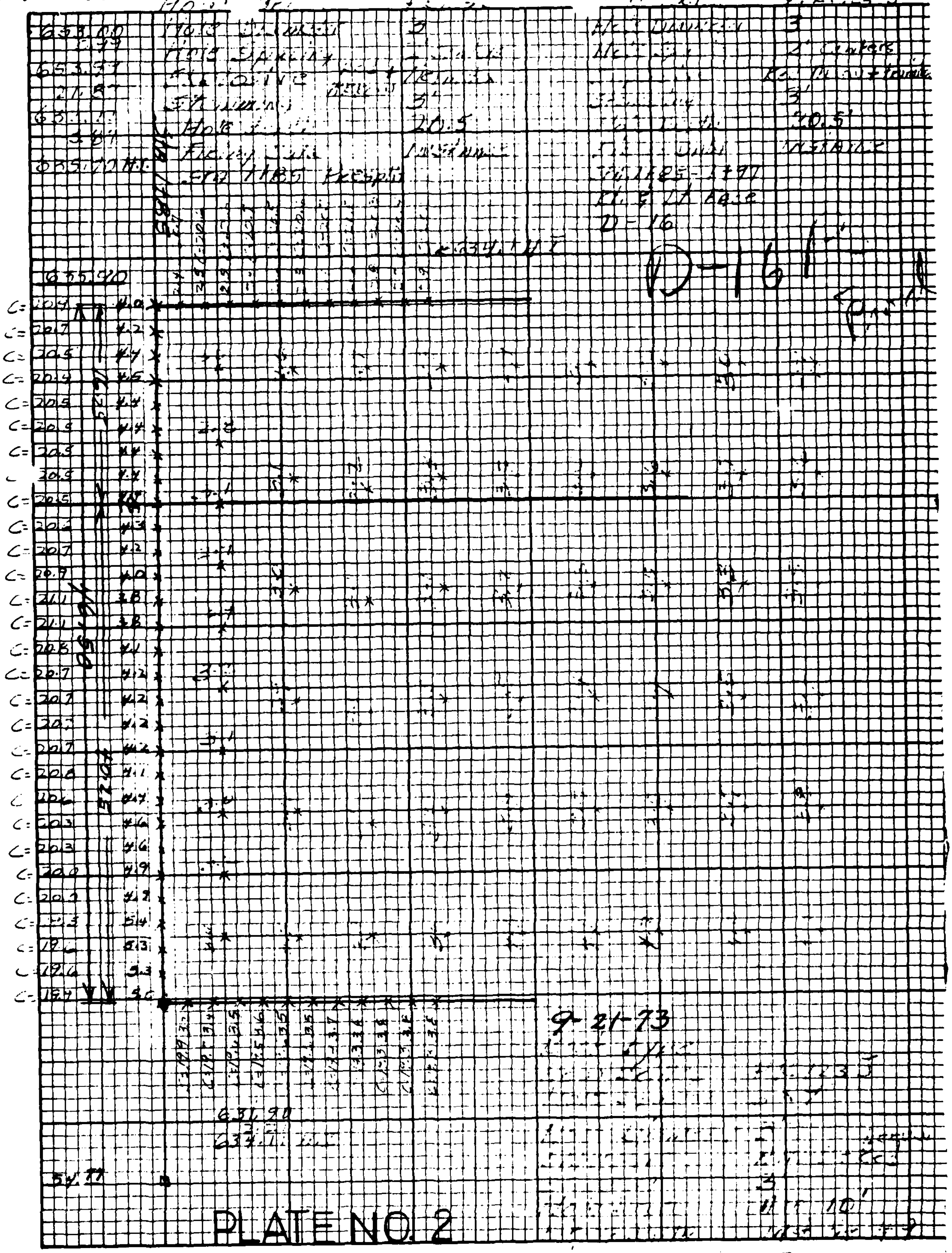
STA. 2+15 STA. 2+47
OFFSET 20' U.S. 58' U.S.
Elev. 580 — 552 ±

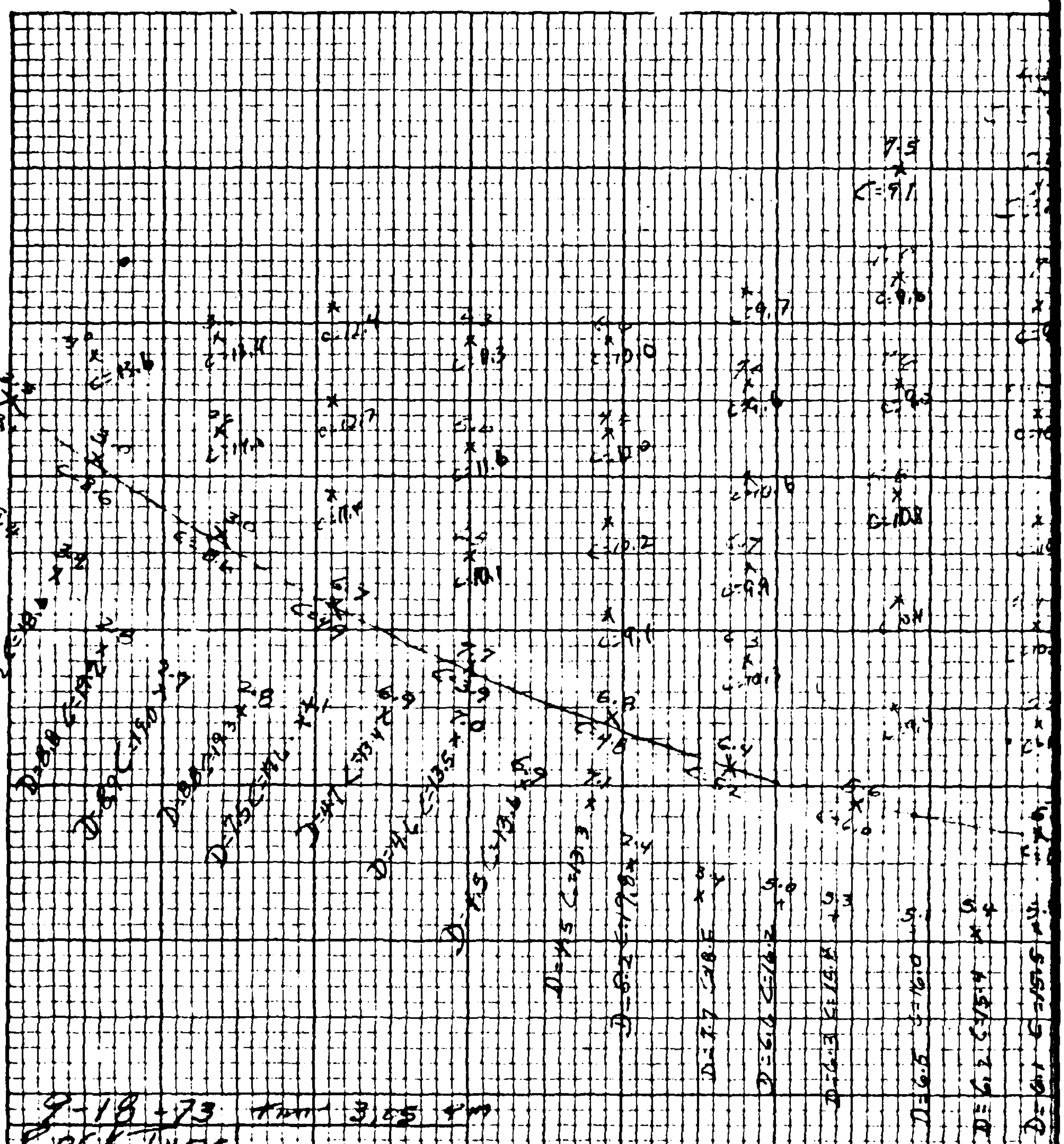
PLATE NO. 1

Scale 1" = 10'

9-19-73 Rock Type

9-20-73





9-18-73	Time 3:05 PM
Rock Type	
Hole Drilled	PD 123 J
Hole Spacing	6 x 7' (4' Center 1 row)
Hole Diameter	3"
Explosive	AMFO + Red Arrow (Primer & Red Arrow) 1-1 Stage
Stemming	3 - 5'
Depth of Hole	Av 10
Firing Data	Inst. thru #9
Sta. 17+75 to 24+35	
18' to 140 Left	

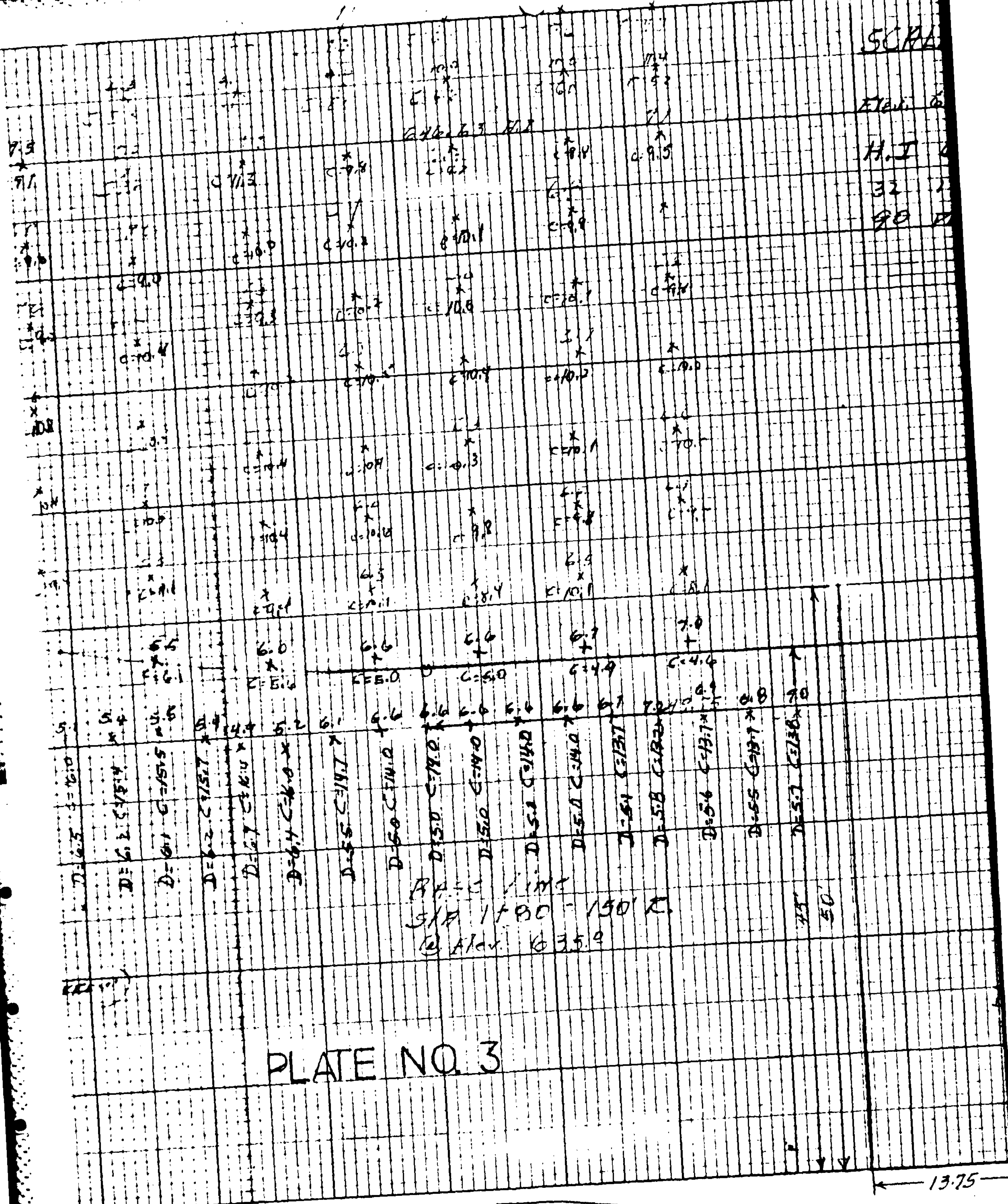
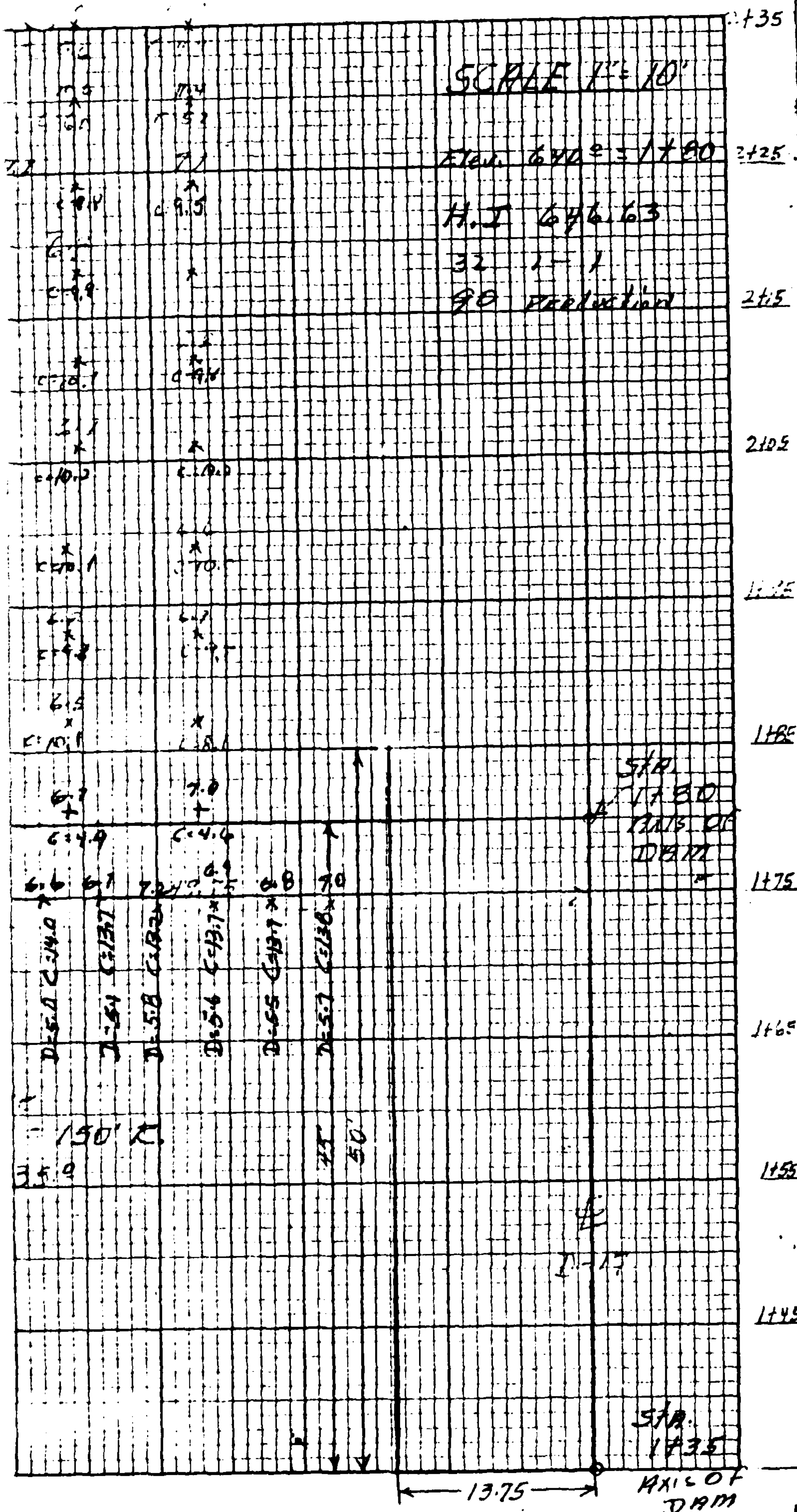
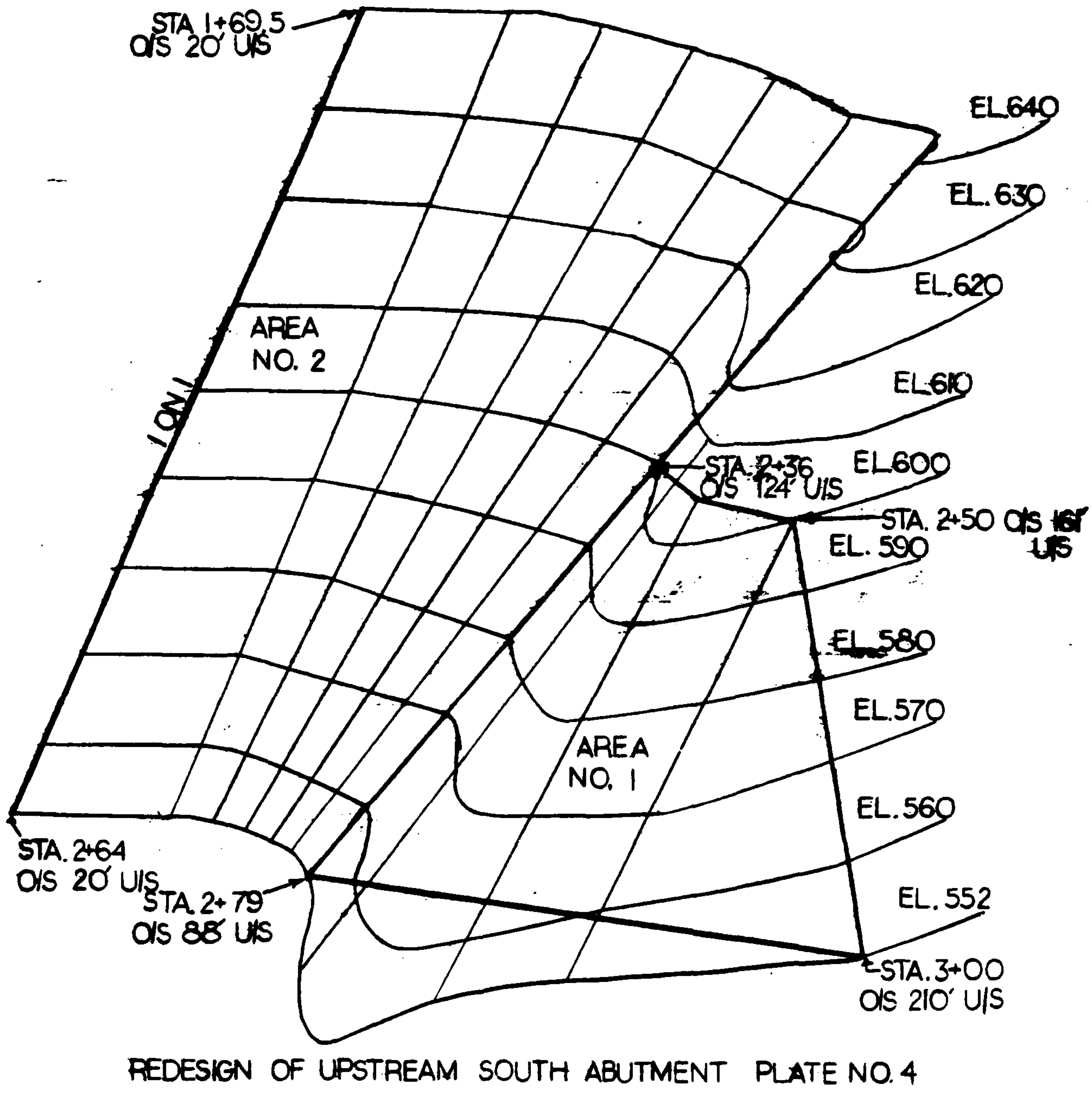


PLATE NO. 3


SCALE 1" = 10'
H.I 646.63
STA. 1+80 AXIS OF DAM
STA. 1+35
AXIS OF DAM
13.75
150' R.
50'
1+75
1+65
1+55
1+45



REDESIGN OF UPSTREAM SOUTH ABUTMENT PLATE NO. 4

REDESIGN OF DOWNSTREAM SOUTH ABUTMENT

PLATE NO.5

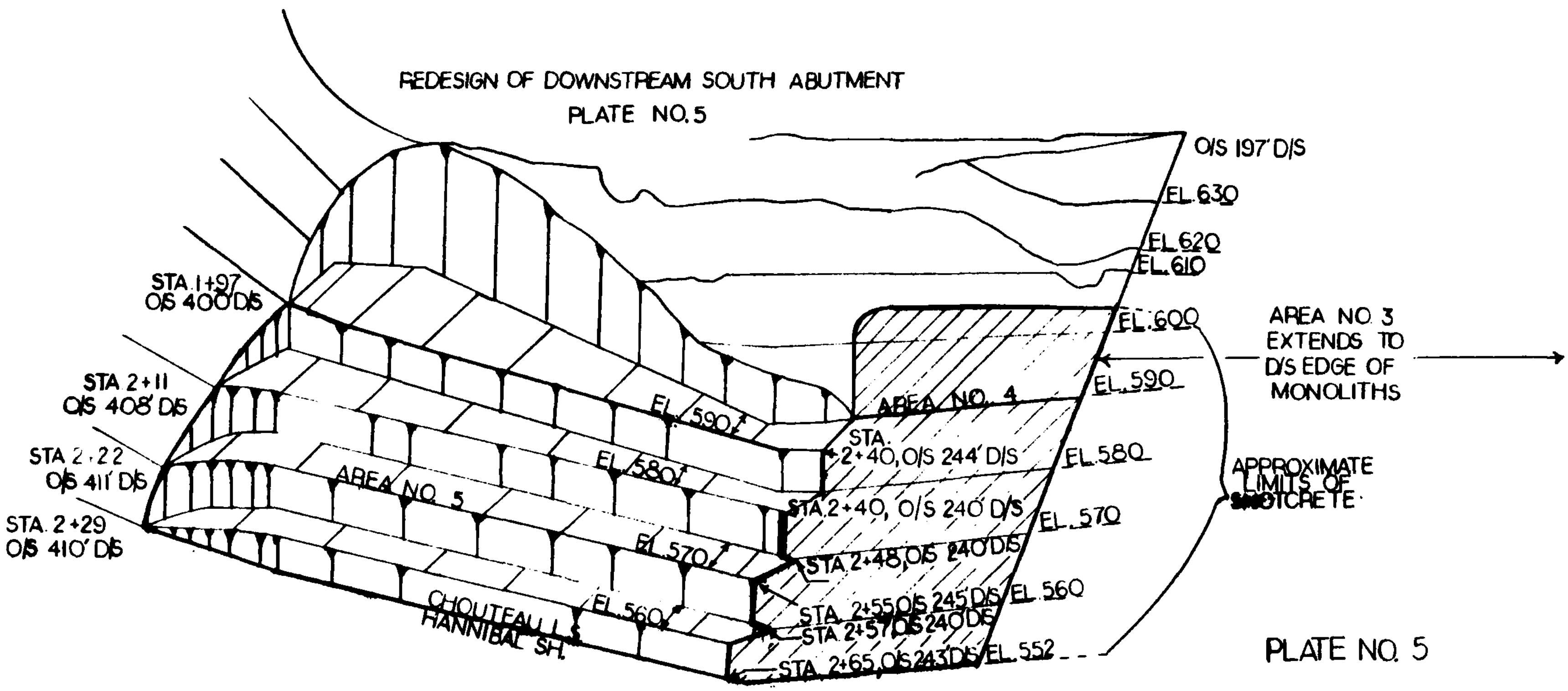


PLATE NO. 5

PROJECT CLARENCE CANNON DAM & RESERVOIR ST. LOUIS DIST.

SUBMITTED: DATE COMPUTED DATE

SP 1-3

PRESPLIT

ROCK TYPE LOUISIANA
HOW DRILLED PR J 123
HOLE DIAMETER 3 INCH
HOLE SPACING 2' Centers
EXPLOSIVE Trojan Special Gelatin 40%
STEMMING Top - 2 Holes
HOLE DEPTH 5 feet ±
FIRING DATA Inst

PLATE NO. 6

SHEET NO. 1

STA 5+48 STA. —
OFFSET 122 - 198
ELEV. 470 - 465

1

South Wall
SP-1-3

A — Z　　　　A1 - A13

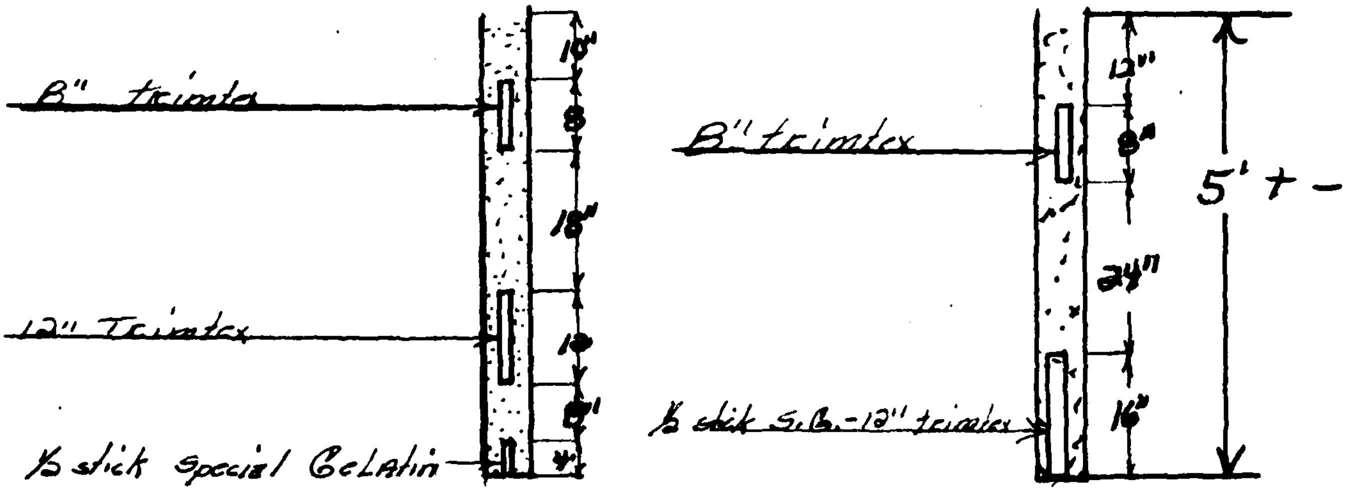


PLATE NO. 6
SHEET NO. 2

PROJECT CLARENCE

D 1

SUBMITTED BY:

DATA 9-18-74

SCALE 0 20 40

STA 8+65 STA 9+20

OFFSET 20 D.S. OFFSET 62 D.S.

ELEV 470 ELEV 467

NOTE: ALL OF SHOT NO. 1 Loaded with one stick 1¼ x 8" @ Bottom of Hole

ROCK TYPE LOUISIANA

HOLE DRILLED PR J123

HOLE DIAMETER 3 INCH

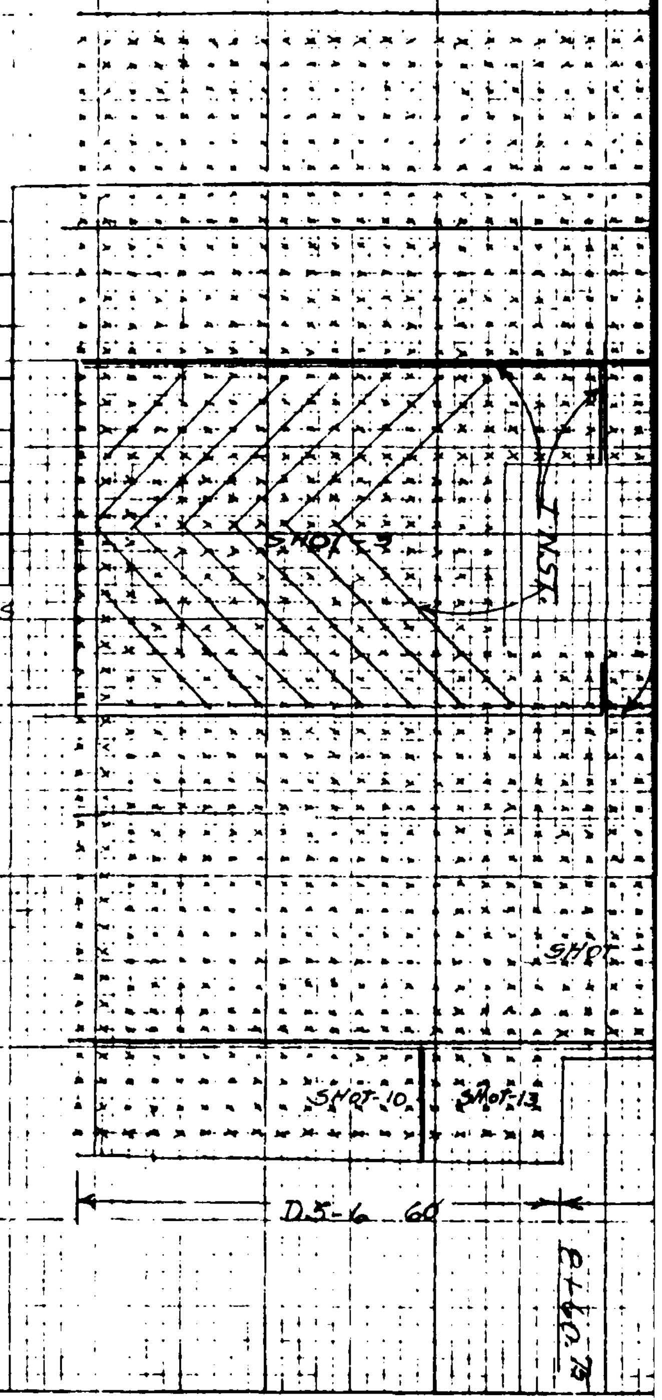
HOLE SPACING 3' X 3'

EXPLOSIVE RED ARROW

STEMMING TOP TO BOTTOM

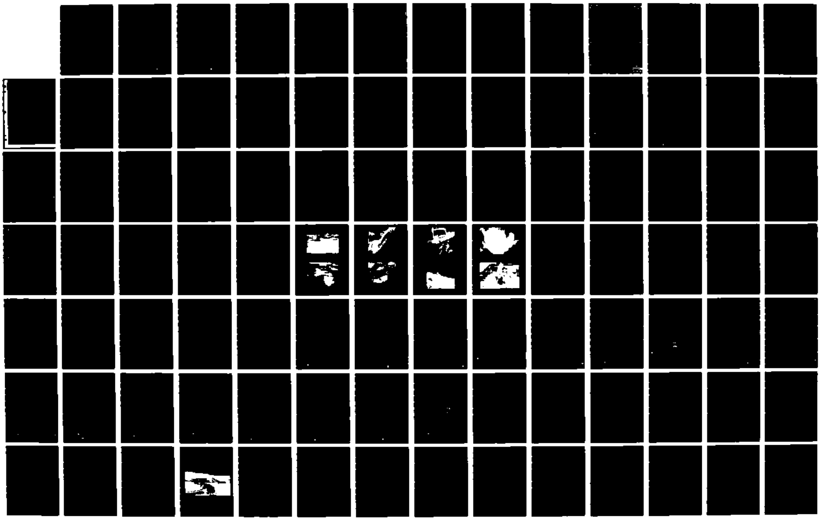
HOLE DEPTH 3 FOOT

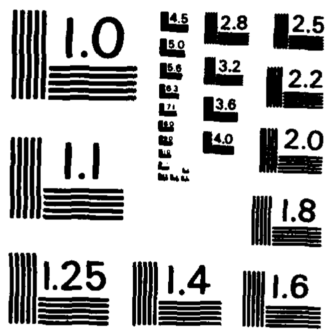
FIRING DATA INST. PRIMACORD CAPS



AD-A160 524 CLARENCE CANNON DAM AND MARK TWAIN LAKE FOUNDATION AND EMBANKMENT COMPLET..(U) ARMY ENGINEER DISTRICT ST LOUIS MO DEC 84 2/5

UNCLASSIFIED F/G 13/13 NL





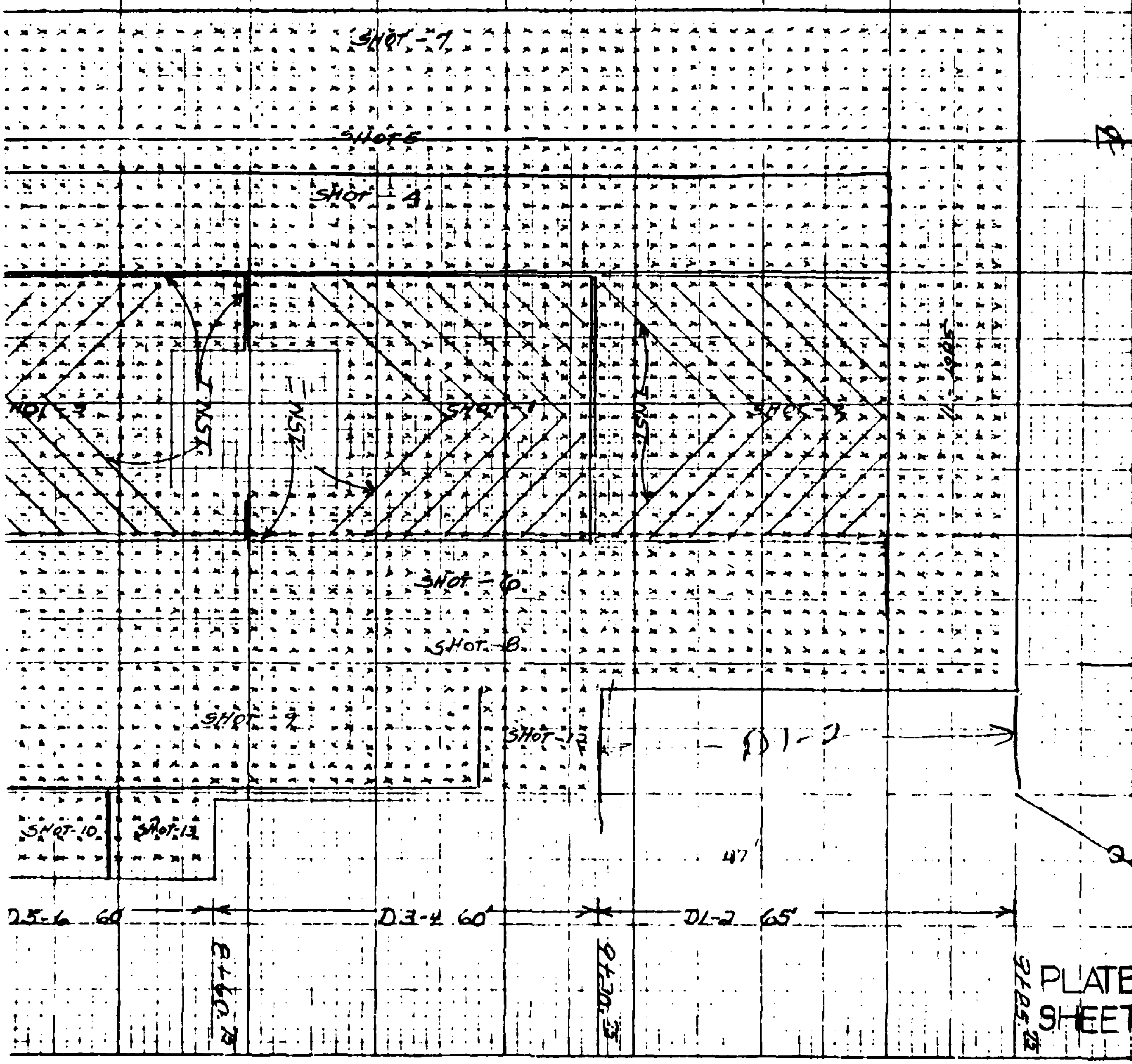
MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

CLARENCE CANNON DAM & RESERVOIR — ST. LOUIS D

D 1-6

COMPUTED DATA

PLATE NO. 7
SHEET NO. 1



RESERVOIR ST LOUIS DIST 9/18/74

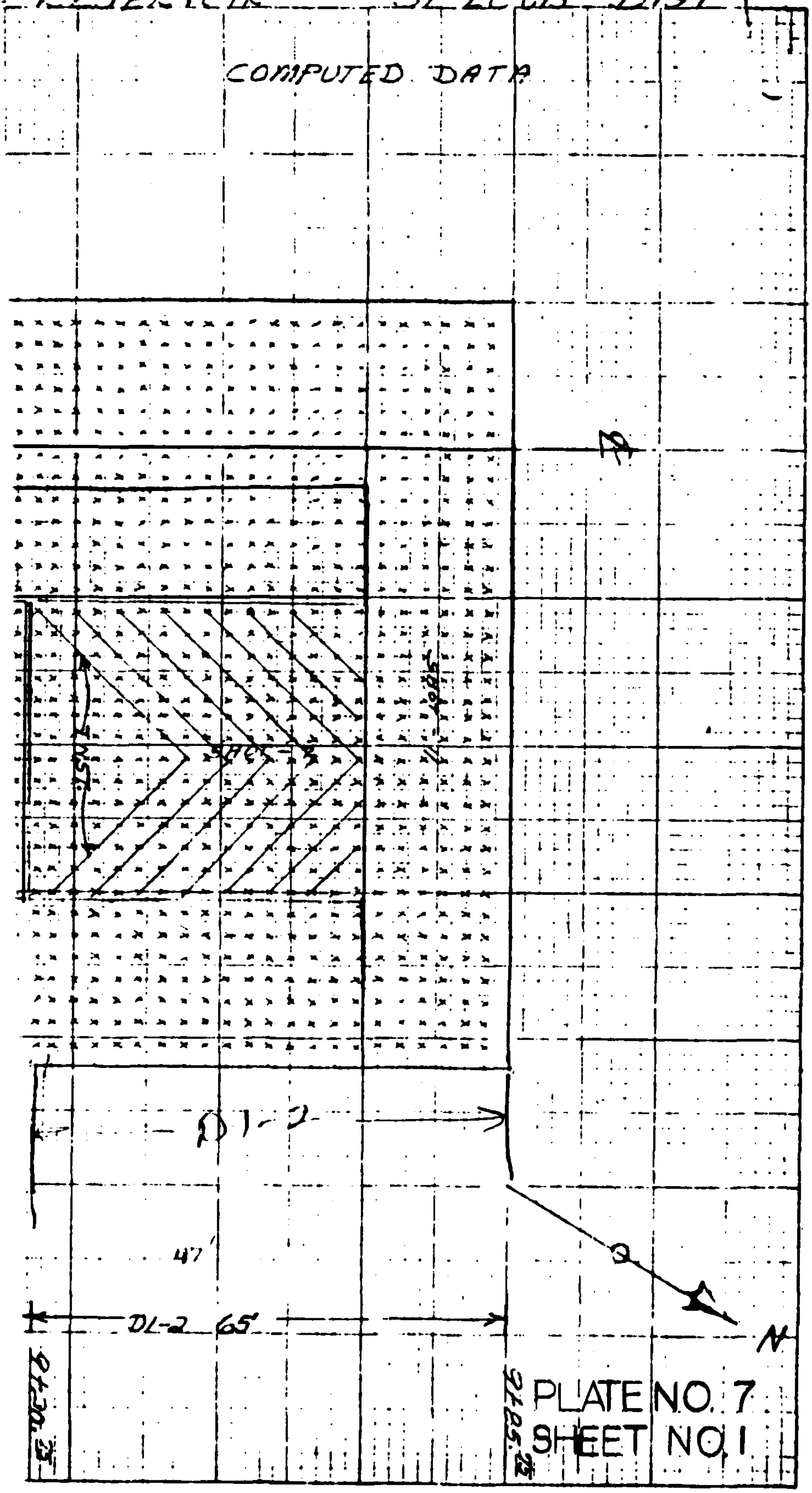


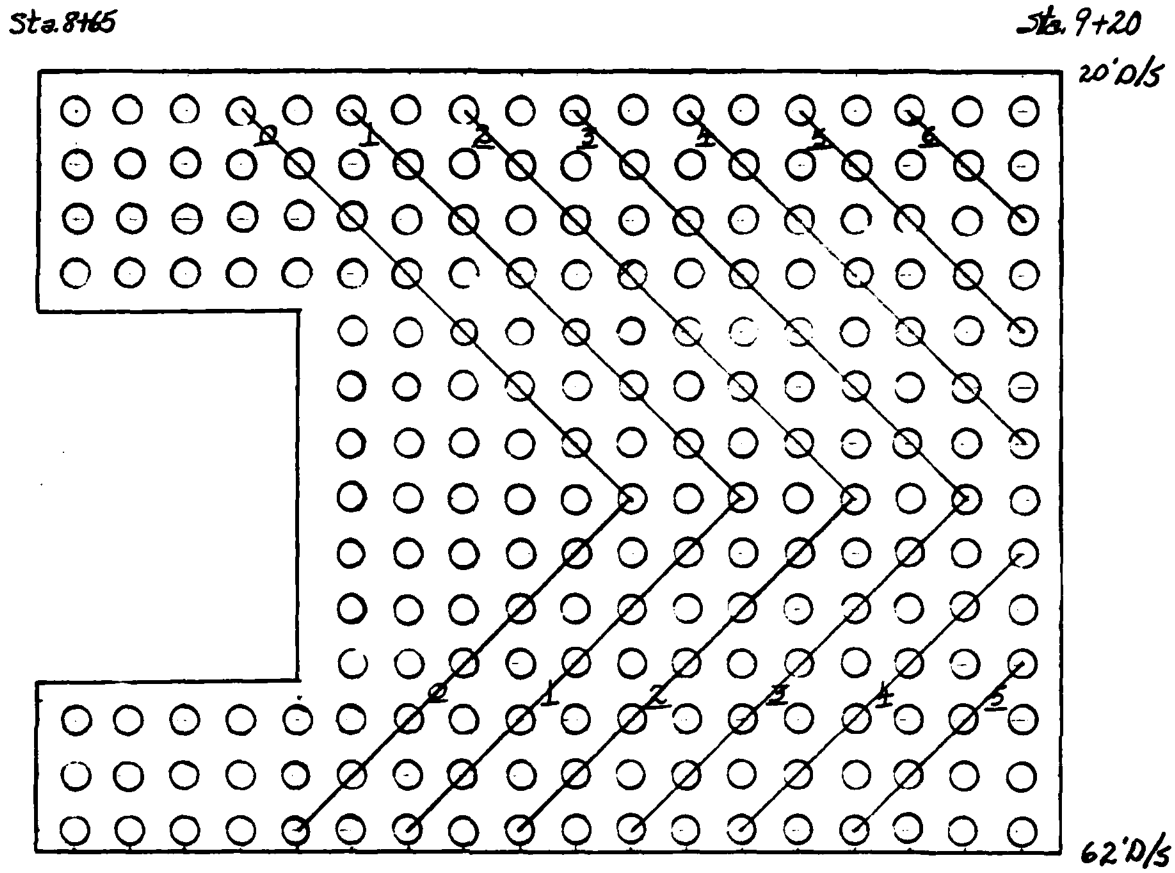
PLATE NO. 7
SHEET NO. 1

PROJECT Clarence Cannon Dam and Reservoir | Page 1 of 2 | COMPUTED BY [illegible] | DATE 15 Jan. 75.

SUBJECT Blasting Computations of Louisiana Limestone | CHECKED BY | DATE

Sta. 8+65 % 20' to 62' D/S. ⟶ Sta. 9+20 % 20' to 62' D/S.

Elev. 470' – 467' Shot – 9/18/74



Loading Pattern

Hole Diameter: 3 inch Hole Spacing: 3' X 3' square

Stemming: 28" – chat Delays: Instant thru #6 @ 5ms each

Load: one stick per hole of 1¼" X 8" Red Arrow

Total Holes: 217 Total Footage: 651 ft

Depth of Holes: 3 ft Area Shot: 1,990 ft^2

Volume Shot: 5,970 ft^3 Total Red Arrow: 98.64 lbs

221 yd^3

462.8 Tons

PLATE NO. 7

SHEET NO. 2

LMV FORM 1 AUG 68 107a COMPUTATION SHEET

PROJECT		COMPUTED BY	DATE
Clarence Cannon Dam and Reservoir	Page 2 of 2		
SUBJECT		CHECKED BY	DATE
Blasting Computations of Louisiana Limestone			

Sta. 8+65 ⁰/s 20' to 62' D/S. → Sta. 9+20 ⁰/s 20' to 62' D/S.
Elev. 470'-467 Shot - 9/18/74

Blasting Agent Properties

Red Arrow Dynamite

Weight Strength: 70% Velocity: 13,200 ft/sec.

Stick Count: 110 Sp. G. - 1.3

Computations

Loading Density:

$$d_e = 48 D_e^2 / SC = \frac{TW}{Nd}$$

$$= \frac{98.64}{651} = \underline{0.15 \text{ lb/ft}}$$

Powder Factor:

$$PF = W/EN = 462.8 / (.454)(217) = \underline{4.70 \text{ tons/lb}}$$

$$= 27 EN / W_y L = \frac{(27)(.454)(217)}{5,970} = \underline{0.44 \text{ lbs/yd}^3}$$

Mass Density:

$$\rho_e = 5.71 \, d_e / D_e^2 = (5.71)(0.15)/1.56 = \underline{0.55 \text{ lb-sec}^2/\text{ft}^4}$$

Average Velocity: 13,200 ft/sec

Accoustical Impedance:

$$Z_e = \rho_e V_e = (0.55)(13,200) = \underline{7260 \text{ lb-sec/ft}^3}$$

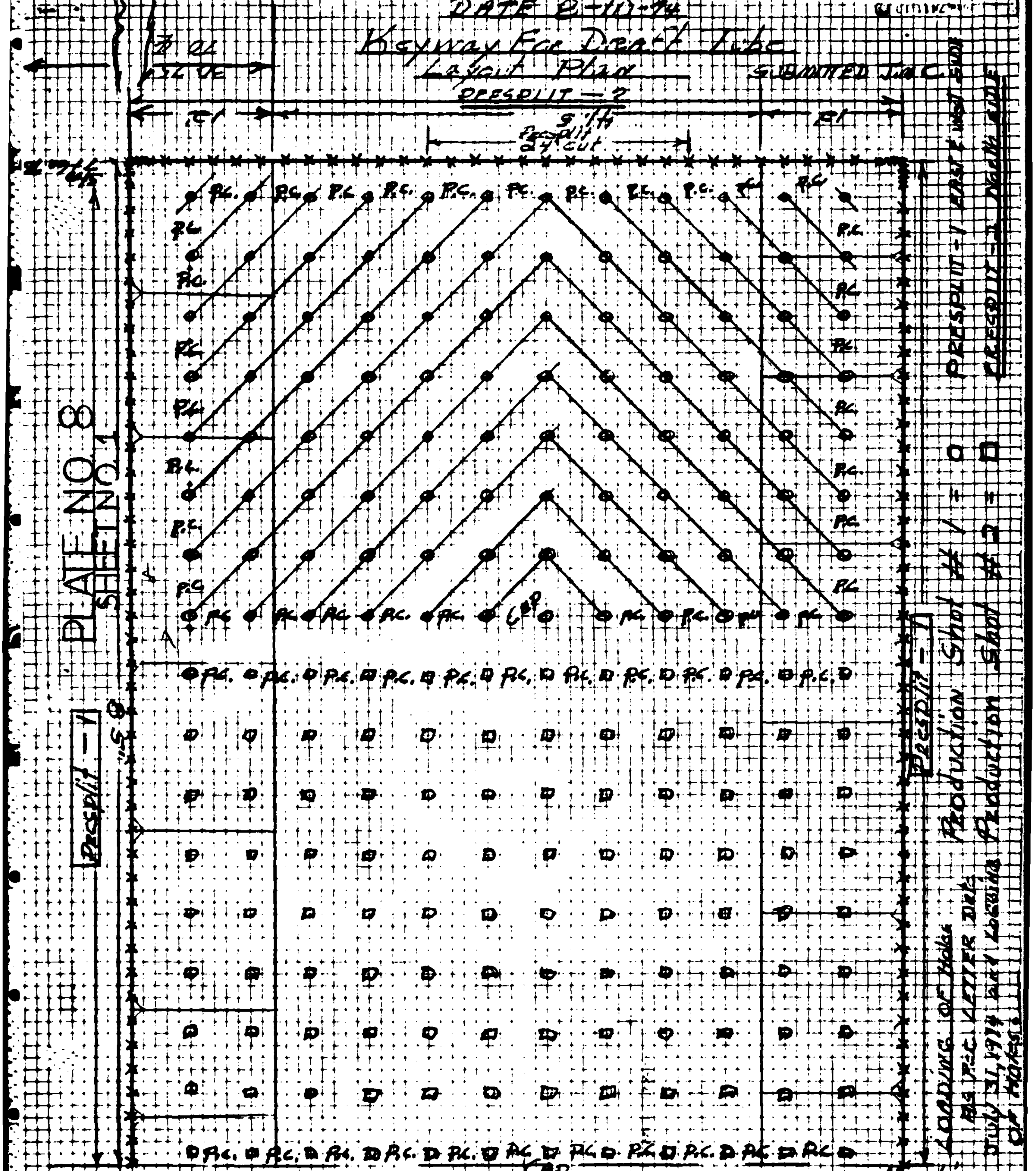
$$Z_e \neq Z_R$$

$$7,260 \neq 49,904 \longrightarrow \underline{1:6.9}$$

PLATE NO. 7

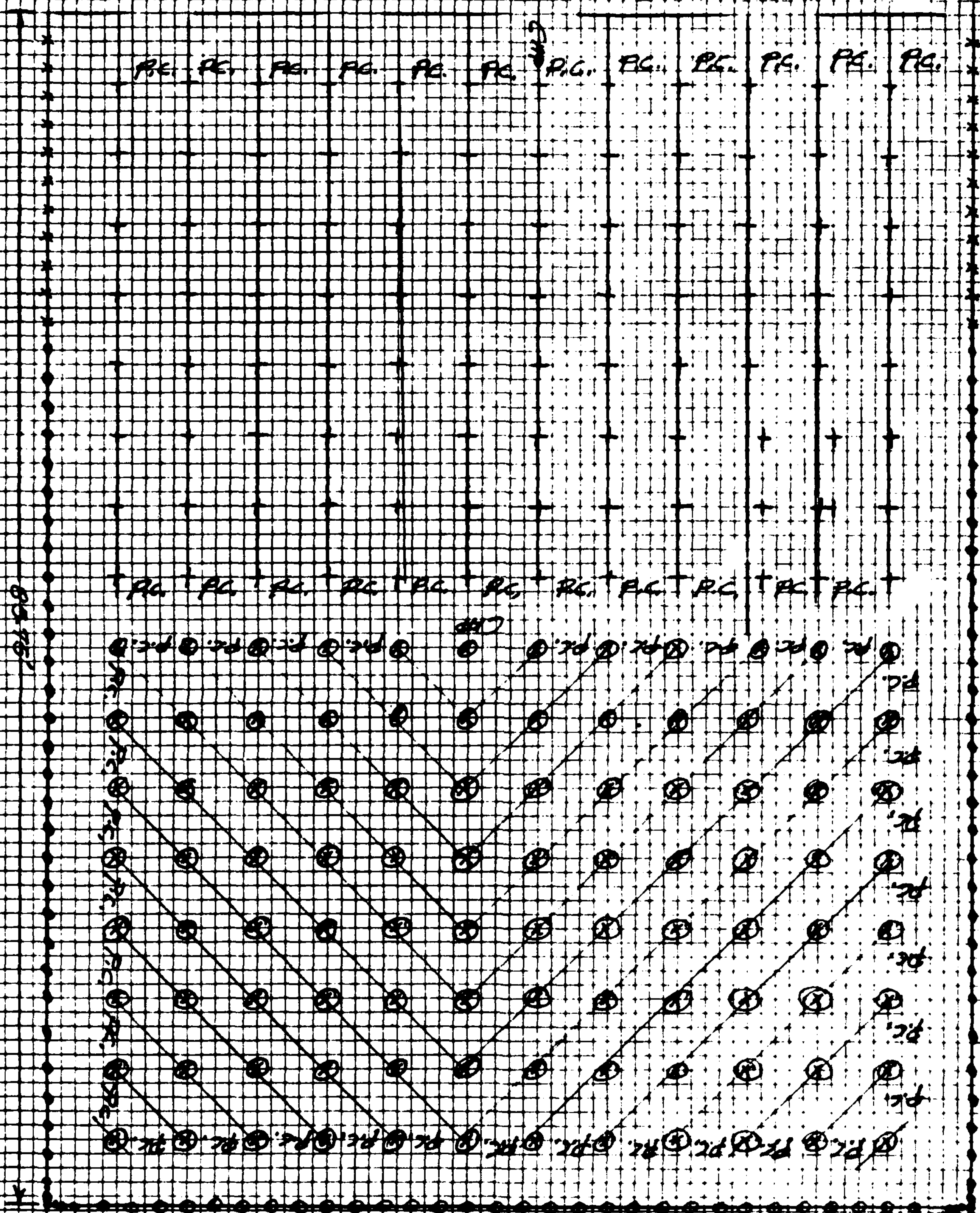
SHEET NO. 3

LMV FORM 1 AUG 68 107a COMPUTATION SHEET



Rock Type	Louisiana Limestone	Explosive	Trimtex & Red Arrow	Location
How Drilled	P.E. J 123	Stemming	To Top	Sta. 7+60-6+45
Hole Diameter	3 Inch	Hole Depth	Presplitting 10' Production 5'	185-845
Hole Spacing	Presplit: 2' Centers	Firing Data	Prim. Conn.	Downstream
[illegible]	[illegible]			Elev. 470-460

SHEET # 2



X PRESPLIT SHOT # 1

● PRESPLIT SHOT # 3

○ PRESPLIT SHOT # 4

+ PRODUCTION SHOT # 5

⊗ PRODUCTION SHOT # 4

Location
Sta. 6+45 - 5+96
@ 18.5 - 84.5
Downstream

Elev. 470-460

PLATE
NO. 8
SHEET NO. 2

8-14-1994

Production Shot No. 1 of
Keyway Demit Tube
Elev. #70 to #65 ±

Note:
All holes loaded with one
stick of 5/8 x 8" E-1 Hercul
at bottom of hole, except
2 rows of holes next to
East and west presplit
lines. These holes loaded
with one stick of 1¼"
by 8" E-1 Hercul

PLATE NO. 8
SHEET NO. 3

Jimmie Canada
S. Luhr Bros. Inc.

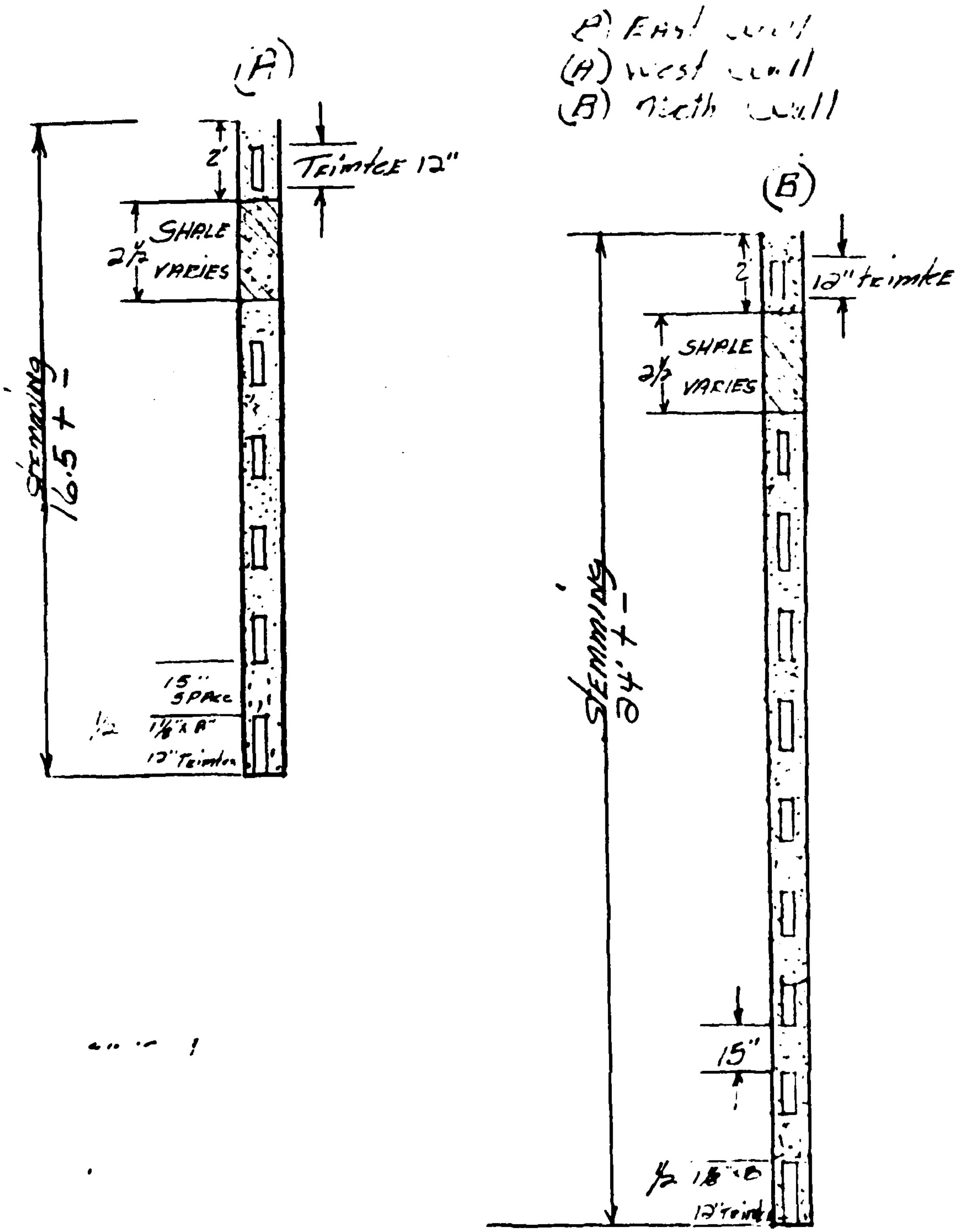


PLATE NO 8

SHEET NO. 4

Submitted by: Jim [illegible] Tailrace 1-3

PROJECT - Clarence Cannon Dam

Date [illegible]

PLATE NO. 9

SHEET NO. 1

Location

[illegible]

[illegible] 11.5' [illegible] 203

Elev. 470-460

TAILRACE 1-3

PRESPLIT

SCALE 1/4" = 6"

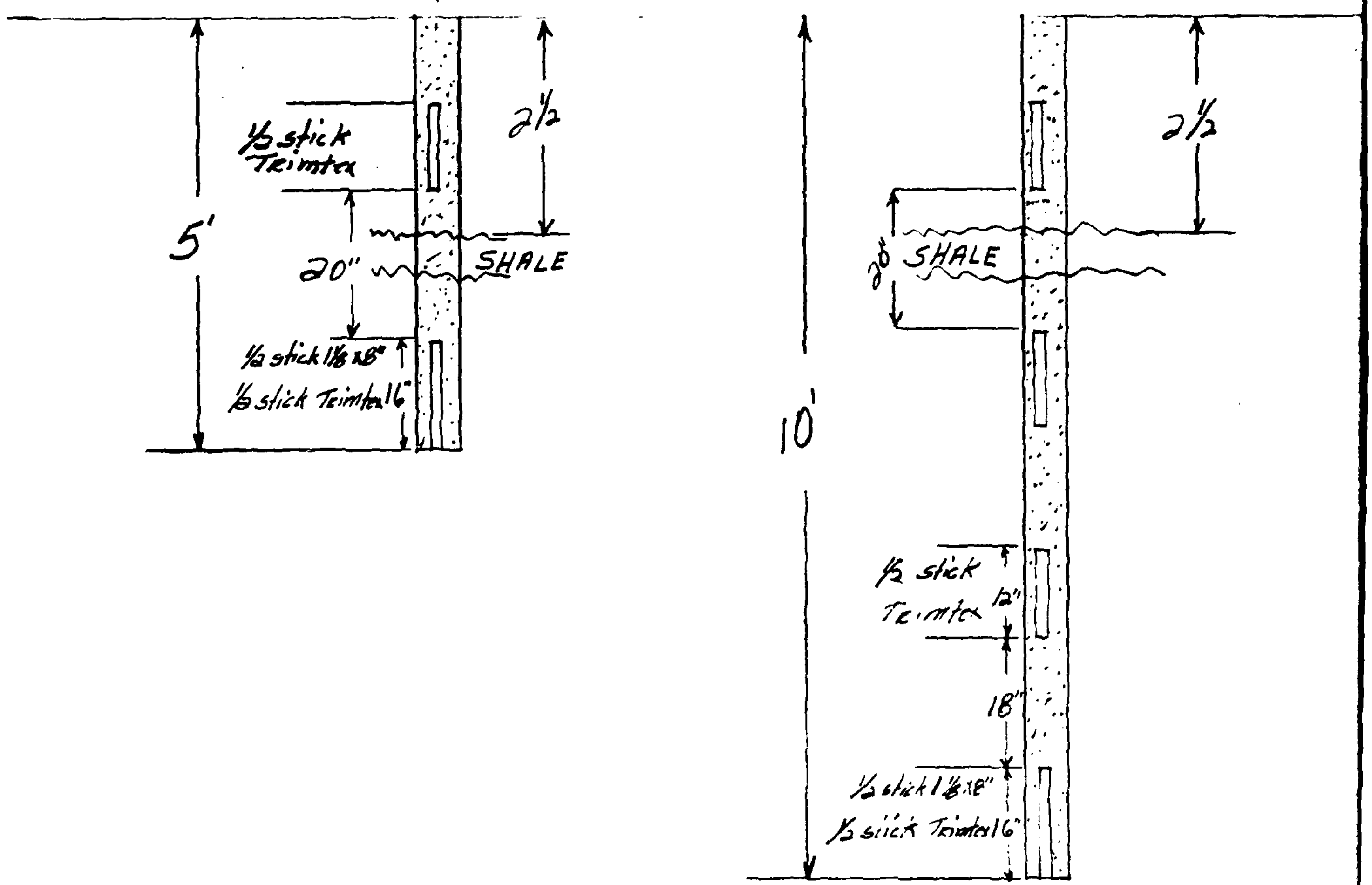


PLATE NO. 9
SHEET NO. 2

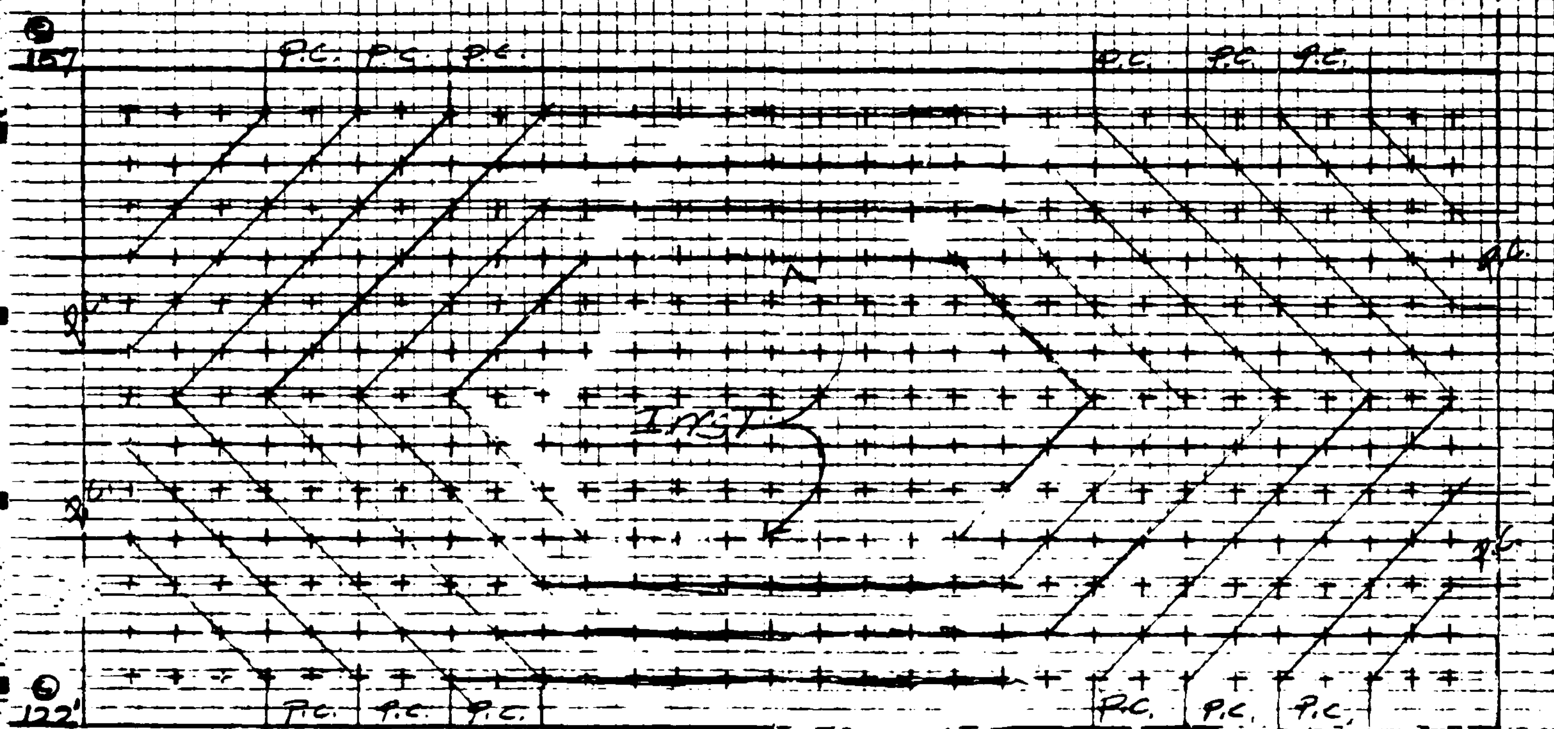
SUBMITTED BY: Jim Canada TAILRACE 1-3 DATE: 9-4-1979

212' PROJECT – CLARENCE CANNON DAM

TR-3

PLATE NO. 10
SHEET NO. 1

TR-2



ROCK TYPE Louisiana
HOW DRILLED P.R.J. 1-2-3
HOLE DIAMETER 3 Inch
HOLE SPACING 2.5 x 2.5

EXPLOSIVE H.D. SPECIAL Gelatin
STEMMING Top to Bottom
HOLE DEPTH 3 feet
FIRING DATA Inst.
Primacord
Caps

LOCATION
STA 1+5 STA 2
212'
Elev 4--4--

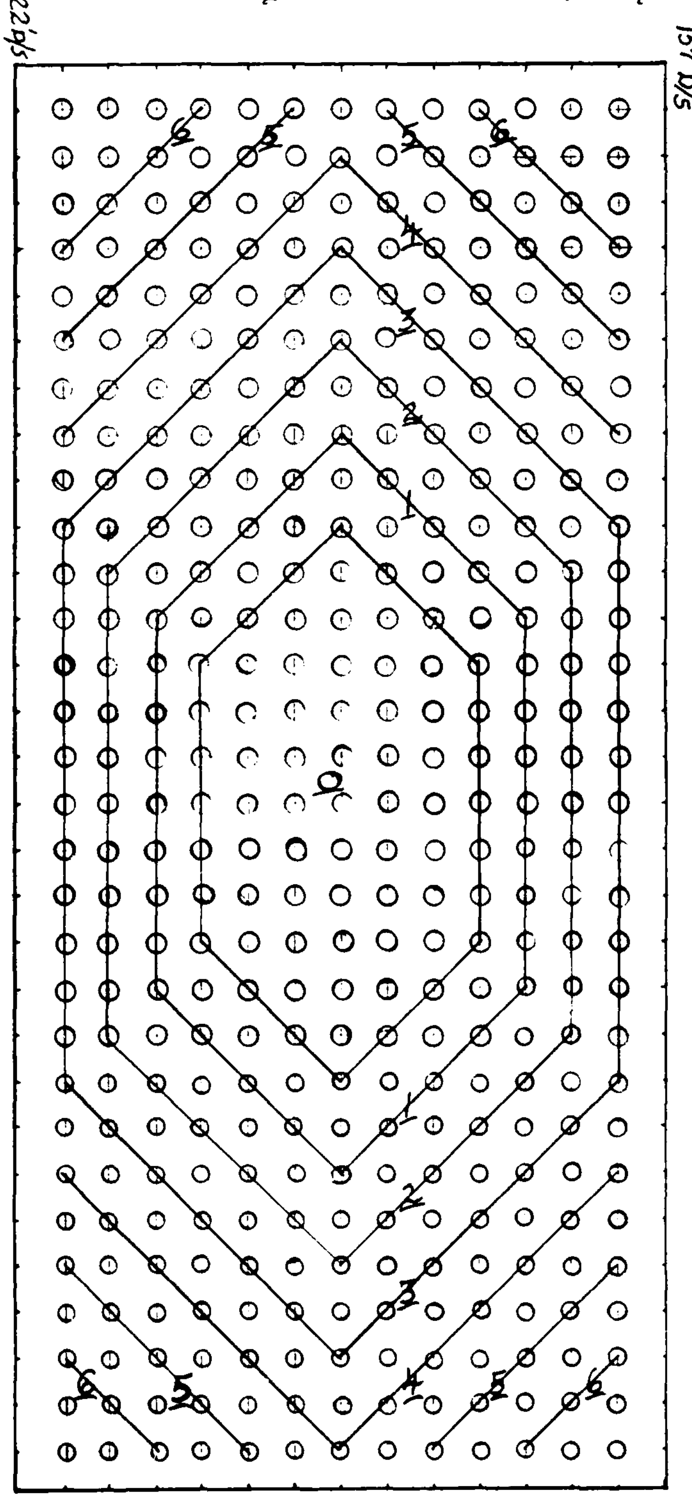
PROJECT		COMPUTED BY	DATE
Clarence Cannon Dam and Reservoir	Page 1 of 3	RH	11/5/74
SUBJECT		CHECKED BY	DATE
Production Blasting – Tailrace Wall – TA-1			

Sta. 7+23 to Sta. 8+00.75 %s 122' D/S to 157' D/S Elev. 465' to 462'

Date Shot – 9/4/74

Delays – 0 thru 6 @ 5ms/delay

Sta. 8+00.75



Sta. 7+23

PLATE NO. 10
SHEET NO. 2

PLATE NO. 10
SHEET NO. 3

Loading Pattern

Hole Diameter : 3" Hole Spacing : 2.5' X 2.5' square pattern

Stemming : 2'4" with clean chips Delays : 0 Thru 6 @ 5 ms/delay

Vertical Holes : 1 stick of 1 1/8" X 8" 40% special gelatin per hole

Total No. Holes : 390 Total Footage : 1170'

Average Depth : 3' Area Shot : 2695 ft^2

Volume Material Shot : 8085 ft^3 — 299.44 yd^3 — 626.59 tons

Total Dynamite : 171.6 lbs

Blasting Agent Properties

Special Gelatin - 40%

Weight Strength — 40% Stick Count — 114

Velocity — 14,400 ft/sec. Sp. G. — 1.24

Computations

Loading Density:

$$d_e = 48 D_e^2 / SC = TW_e / Nd = 171.6 / 1170$$

$$= \underline{0.147 \text{ lb/ft}}$$

Powder Factor:

$$\overline{PF} = W / EN = (626.59) / (.147)(1170) = \underline{3.64 \text{ ton/lb}}$$

$$= 27EN / WyL = (27)(.147)(1170) / 3000 = \underline{0.57 \text{ lbs/yd}^3}$$

Mass Density:

$$\rho_e = 5.71 d_e / D_e^2 = (5.71)(0.147) / (1.125)^2 = \underline{0.67 \text{ lb-sec}^2/\text{ft}^4}$$

B. Excavation for Embankments

1. Saddle Dam

Excavation for the saddle dam consisted of stripping 12 inches of topsoil and excavating an inspection trench. The stripped material was stockpiled and later used for the top dressing on side slopes and 3-foot road shoulders on the saddle dam.

The contract specified an inspection trench 5 feet deep with 1V:1H slopes and 8-foot flat bottom; however, the Contractor elected to excavate the trench with a 12-foot± flat bottom in order to employ normal scraper operation. The inspection trench was excavated from saddle dam Station 3+40 to Station 10+80 and Station 16+10 to Station 24+80. The natural ground line between Station 10+80 and Station 16+10 was higher than planned top elevation of the saddle dam and was left in place. The material excavated was found to be suitable clays (CL and CH) and was utilized in backfilling the trench.

Stripping, inspection trench excavation and backfill were peformed in July and August 1974 using the following equipment:

Two (2) CAT D8 Dozers
Two (2) CAT 631 Scrapers
Two (2) CAT 637 Scrapers
One (1) CAT 825B Roller with Caron Wheels

Drawing No. 114/2 depicts profile and typical sections of the saddle dam.

2. Water Temperature Control Weir

Foundation excavation for the water temperature control weir consisted of removing muck from the original river channel and banks, removing that part of the Phase I cofferdam within the limits of the weir, excavating to suitable foundation material at the contact area on the right abutment and excavating a cutoff trench.

The centerline of the cutoff trench is 400 feet upstream and parallel to the dam axis, and extends from the right abutment at Station 2+00± to Station 8+35± where it ties into the auxiliary cutoff constructed during the Phase I contract. The trench was excavated from El. 540± feet NGVD at the north end, El. 518± feet NGVD in the river channel, down to sound shale at El. 510± feet NGVD. Contract drawings specified 1V:2H slopes and 10-foot wide bottom; however, due to the width of equipment used, the Contractor was allowed to excavate the trench with slightly steeper slopes and 12-foot wide bottom.

Excavated clays suitable for impervious fill were limited to an area from Station 6+00± to the north end at Station 8+35± and down to the top of sands and gravels at El. 523± feet NGVD.

Approximately 20,800 cubic yards of clays were excavated from this area of which 17,000± cubic yards were natural deposit valley clays and 3,800± cubic yards were from the Phase I cofferdam. Approximately 13,300 cubic yards were placed directly into permanent fill. The remainder was placed into temporary cofferdams and Contractors' work areas then later placed into permanent fill. Valley clays were sandy clay (CL) and silty clay (CL). Phase I cofferdam material was clay (CL and CH).

Total other material excavated was approximately 29,500 cubic yards generally comprised of sands and gravels, and muck and shale (all of which was wasted in depleted Borrow Area No. 3). Plans and sections of the water temperature control weir are shown on Drawing No. 110/2.

Excavation started in November 1973 and was completed in June 1974 using the following equipment:

Two (2) CAT 631 Scrapers
Two (2) CAT 637 Scrapers
Two (2) D8 Dozers
One (1) CAT D9 Dozer
One (1) CAT 16F Motorgrader
One (1) Bucyrus Erie 38B Dragline
Three (3) CAT 630B Bottom Dumps

3. Main Dam

(a) Phase I Fill

The Contractor was required by the specifications to remove at least the upper 18 inches of Phase I fill and an additional depth as determined by the Contracting Officer. The specifications also required the removal of Phase I material along the slopes of the diversion channel unsuitable for embankment foundation as the result of erosion and freeze-thaw.

Approximately 26,600 cubic yards of clays were removed from the Phase I fill from a depth of 18 inches to 4 feet. An additional 20,000± cubic yards were removed from the slopes of the diversion channel. The clays were excavated on an "as needed" bases just prior to placement of Phase II embankment. The first excavation of this material was performed in November 1973 and was completed in July 1980. Diversion channel slope excavation started in August 1979 and was completed in July 1980.

Approximately 1,500 cubic yards of the above-described clays contained vegetation and were wasted in depleted Borrow Area No. 3. The remainder was either stockpiled and later placed in permanent fill or hauled directly to permanent fill.

In an area along the downstream edge of Station 10+40± to Station 12+75± (the south bank of the diversion channel), a wedge of Phase I fill consisting of approximately 1,500 cubic yards was removed as a result of Modification No. P00101 which directed the Contractor to extend the compacted fill downstream. This excavation consisted of backsloping the downstream edge of Phase I fill in order to tie the extended fill into Phase I fill. Excavation was performed in October and November 1980.

The Phase II Contractor erected his concrete plant on H-piling driven through Phase I fill. After the concrete plant was removed, the Contractor was unable to pull the piling. The Contractor was then directed by Modification No. P00124 to excavate the area of the piling from the top of Phase I fill at El. 545± feet NGVD to El. 530 feet NGVD and at this point cut the piling. As specified by the modification, the excavation was backfilled with compacted impervious material with the moisture content within a range of optimum -1% to optimum +1%. The excavation was in an area from 100± feet to 200± feet downstream of the centerline and from Station 11+25± to Station 12+40±. Side slopes were 2H:1V. Approximately 14,000 cubic yards were excavated using a CAT backhoe, a CAT dozer and CAT 637 scrapers. Excavation started in May 1980 and backfill was completed in July 1980.

During construction of the Phase I fill, evaluation of testing from undisturbed record block samples indicated thatpockets of softer material at a water content in excess of that specified were being encountered. While these pockets tended to be of very limited extent within the block sample, shear testing yielded strengths lower than those used in design.

To more fully evaluate the extent of this program, a full-scale evaluation program was initiated. This program consisted of additional record block samples, undisturbed soil sampling with a drill rig, field in-situ strength tests, increased quality assurance testing, laboratory testing and office studies including stabilities.

It was concluded that the Phase I fill as placed was acceptable and would perform successfully. While softer and wetter pockets of fill with "Q" shear strengths lower than design were found, there was no evidence that these pockets were continuous in either a lateral or vertical direction nor was there any evidence to indicate that the weaker strengths associated with these softer materials were present in sufficient numbers to cause a reduction in selected design strength. However, to provide for future modifications should such be necessary to the embankment section, several changes were made to the section as originally set forth in the specifications. An area of natural foundation clays located at the upstream and downstream toes of the section was removed and replaced with impervious compacted fill. The substitution consisted of a fully compacted impervious fill for a semi-compacted random fill along the downstream toe of the dam and the filling of the second-stage diversion channel upstream of the embankment to natural ground elevation with semi-compacted fill.

The results of this program, including all field test and exploration, laboratory testing, engineering evaluation and design studies, conclusions and recommendations, have been presented in the Report entitled "Clarence Cannon Reservoir, Salt River, Missouri--Main Dam Embankment--Special Study Phase I Fill, 1977 (Test Revised 15 October 1978)".

(b) Diversion Channel

Excavation of the diversion channel allowed the Salt River to be diverted from its natural flow path (near the right abutment) over to the left abutment. The diversion allowed the subsequent excavation and construction of the upstream gabions, water temperature control weir, concrete structure and downstream exit channel. The river was diverted on 27 October 1973 and rediverted back through the concrete structure on 24 July 1979.

The first section of the diversion channel from Station 0+00 to Station 19+00± (channel stationing) was excavated by the Phase I Contractor. The remaining section from Station 19+00± to Station 38+50± (channel stationing) was excavated under the Phase II Contract. The topsoil within the diversion trench limits was stripped in late May and early June 1973, and excavation was started in June 1973 and completed in October 1973. The bottom width of the channel was 110 feet, with 1V:3H side slopes and a bottom elevation of 515 feet NGVD. Excavation was started at approximately 75 feet from the river channel and proceeded downstream to the Phase I channel. The upstream 75-foot plug was removed as the last order of work for the diversion channel prior to diversion. Quantities and disposition of materials excavated from the Phase II diversion channel were as follows: approximately 13,000 cubic yards of stripping were stockpiled for later use; approximately 207,000 cubic yards of clays suitable for impervious fills were excavated, of which 182,000± cubic yards were stockpiled for later use in the embankments, and the remainder placed directly on the embankments; approximately 93,000 cubic yards of materials unsuitable for the permanent embankments were excavated, all of which were wasted in depleted Borrow Area No. 3. A major part of clays excavated was extremely wet, some of which were spread in thin layers over Borrow Area No. 2 as they were excavated in an effort to partially dry them. Excavation after diversion of the river through the sluices consisted only of removal of materials, as necessary, for embankment foundation (refer Drawing No. 4/2 for alignment of the diversion channel).

Equipment utilized for excavation of the diversion channel included the following:

Two (2) CAT 637 Scrapers
Four (4) CAT 631 Scrapers
Two (2) Euclid 23TDT Bottom Dumps
Five (5) CAT 630B Bottom Dumps
One (1) Bucyrus Erie 38B Dragline
One (1) Bucyrus Eris 88B Dragline
One (1) CAT D6 Dozer
Four (4) CAT D8 Dozers
Two (2) CAT D9 Dozers

(c) Left Abutment Excavation, Rock

(1) Excavation for Cutoff Trench and Embankment Contact Area

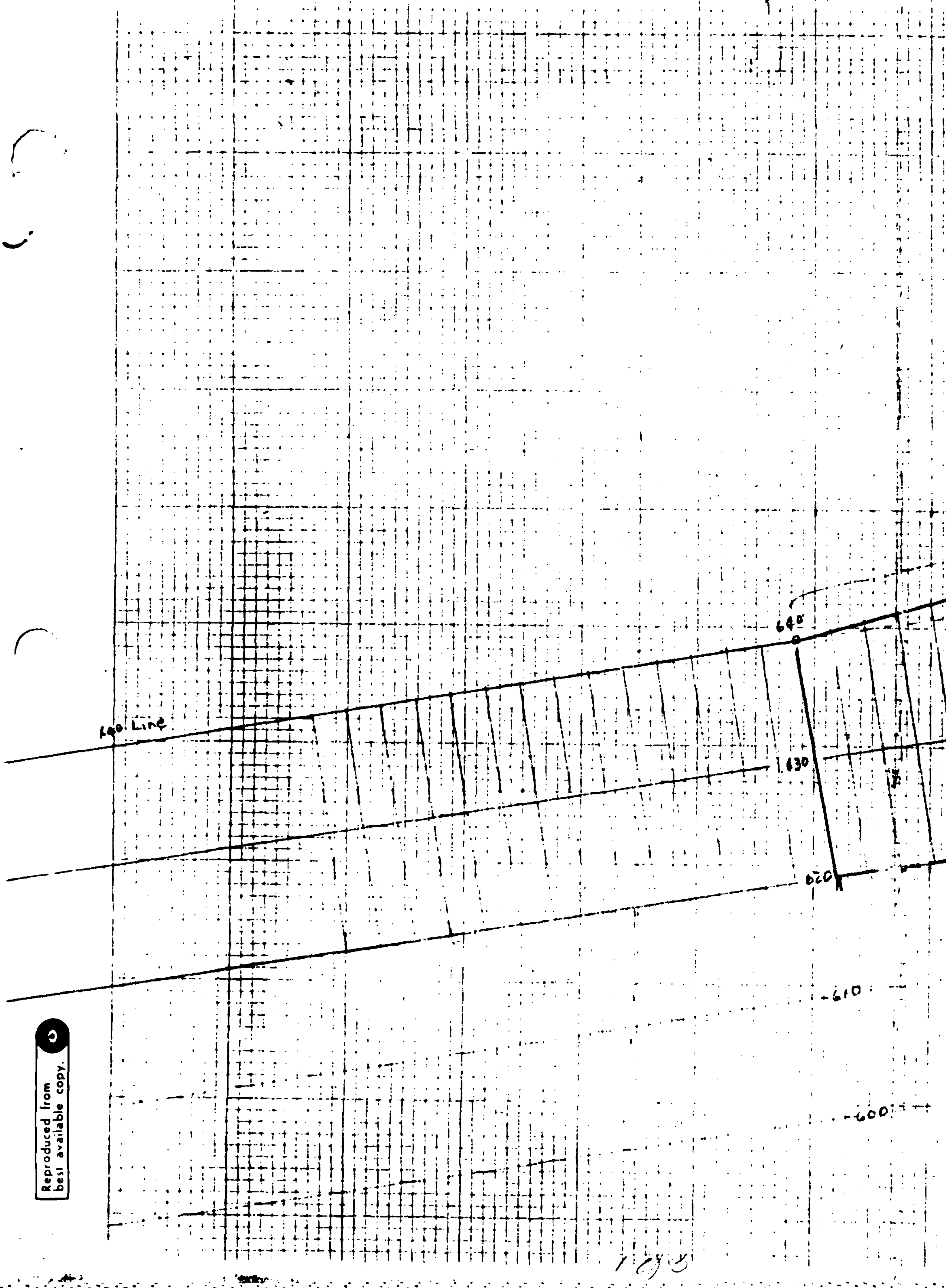
Contractual work for the left abutment excavation began on 8 February 1977 and lasted until the early part of September 1977. The scope of work entailed the presplitting of the cutoff trench walls and embankment contact area to a 1V:1H slope, primary blasting and actual rock removal. The scope of work was enlarged by modification to include cavity cleanout and air track drilling for the 7-foot by 7-foot cavity detection program. Generally, the Contractor utilized a two-shift operation (21 March thru 30 April, and 14 July thru 24 August 1977) for the majority of the abutment excavation and cavity cleanout. An Operating Engineers' strike interrupted all project work from 1 May thru 5 July 1977. The principal equipment utilized during the left abutment excavation and cavity cleanout consisted of the following:

Four (4) 3700 Gardner-Denver Air Tracks
One (1) Gardner-Denver 750 cfm Air Compressor
One (1) Gardner-Denver 900 cfm Air Compressor
One (1) Sullair 1050 cfm Compressor
Four (4) Allmand Maxi-lite Light Plants
One (1) Case Tractor King
One (1) D8H CAT Dozer
One (1) 580 Case Backhoe
One (1) 245 CAT Backhoe
One (1) Bucyrus Erie Crane
Two (2) Terex 50T End Dumps
One (1) Euclid 97FD End Dump
One (1) Diamond T Fuel Truck
One (1) Grove Hydrocrane
Blasting Mats
Two (2) Walkie Talkies
Jigger Drills, Jack Hammers, Shovels and Picks

Massman Construction Co.'s subcontractor, Luhr Bros., Inc., commenced drilling 3-inch diameter presplit holes with two 3700 Gardner-Denver Air Tracks along the perimeter of the left abutment cutoff trench on 8 February 1977 and finished at the end of February 1977. On 25 February 1977, the Contractor started presplitting operations from the embankment contact area and finished on 16 August 1977. Generally, the depth and alignment of the boring were controlled by a string line. The spacing of the presplit boring was 3 feet. Surface exposure of the Burlington Limestone and the specifications requirement for a 10-foot cut along the axis of the cutoff trench allowed the Contractor to drill the presplit holes to the full depth of the required cut. Based upon the presplit hole depth specification requirement (maximum depth 20 feet), the Contractor utilized a series of 1-foot offsets to obtain the required cut along the face of the embankment contact area to El. 553± feet NGVD (Chouteau Limestone-Hannibal Shale Formation contact). Visual observation of the presplit rock faces generally revealed competent surface for embankment placement.

The type of explosive for presplitting consisted of a continuous 1-inch diameter column of Tovex T-1 manufactured by DuPont. The explosive was classified as a water gel with a loading density of 0.25 pound/foot. The loading procedure consisted of lowering the column of Tovex T-1 to the bottom of the hole to establish the necessary length, then withdrawn by the length of unloaded hole (3 feet) and cut. The water get was then primed by either an electric blasting cap or 50-grain detonation cord that was placed in a slit approximately 1 foot from the top. The explosive was then lowered to the bottom of the presplit hole. Generally, the top 3 feet of the presplit hole were stemmed with crushed stone. When detonation cord was used, the shot was initiated by two No. 3 electric blasting caps attached to the detonation cord. Blasting began on 11 February 1977 and continued on a continual basis until 20 August 1977 (refer Plate No. 11 for a typical shot plan).

The Contractor started drilling for primary blasting in the cutoff trench on 16 February 1977 with two 3700 Gardner-Denver Air Tracks and detonated the last production shot on 11 March 1977. On 7 March 1977, the Contractor started drilling for production shots along the embankment contact area and fired his last production shot on 20 August 1977. Primary blasting was not permitted as per the specifications within a horizontal distance of 50 feet from a required presplit or line-drilled face until such face had been presplit or line drilled. In addition, the lift thickness was restricted to a maximum of 10 feet.



640 Line
640'
630
620
610
600

U/S Slope

Pre-split Shot No. 5

Date : 25 Feb. 1977

How Drilled : 3700 Air Track

Hole Diam : 3"

Hole Depth : Variable to Elev. 620

Hole Spacing : 36"

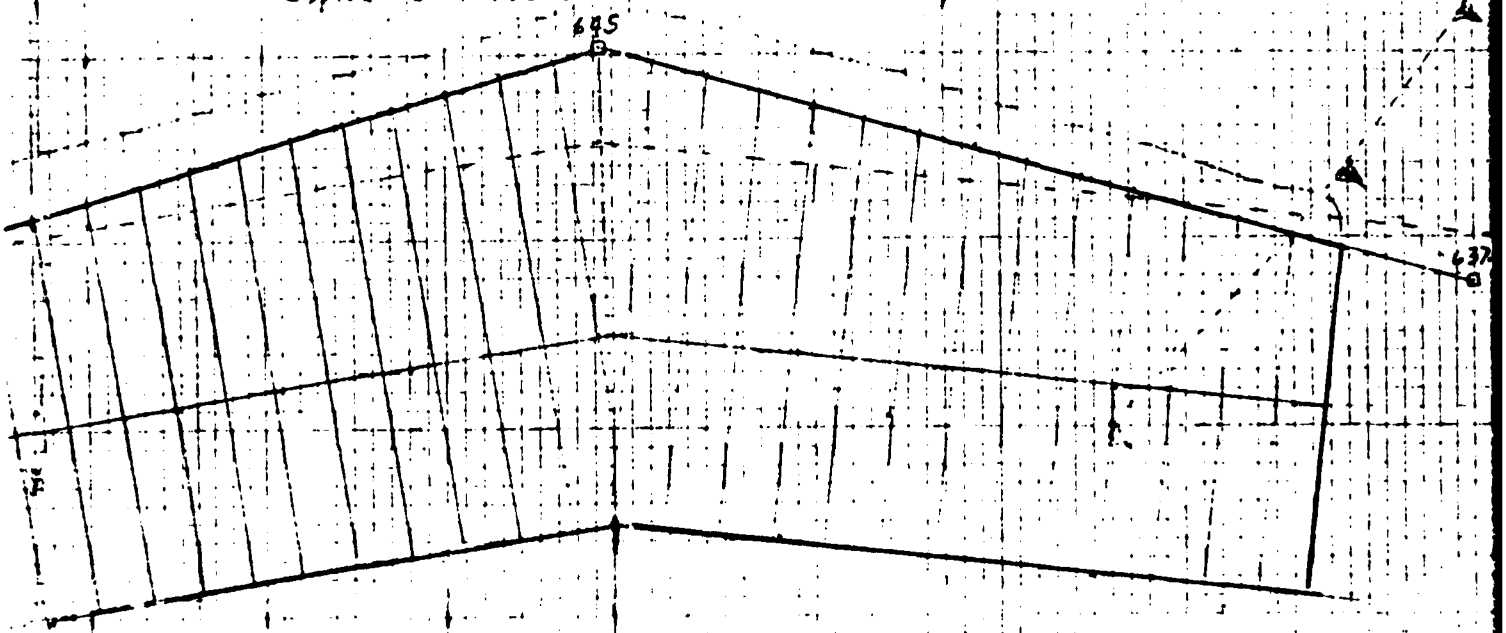
Stemming : Crushed Rock in Top 3' of hole

Explosive : Tovex T-1 initiated w/Elec. Caps and E-Cord

1312.5# Tovex - T1

2E0 L.F. E-Cord

2 - Electric Caps No. 3



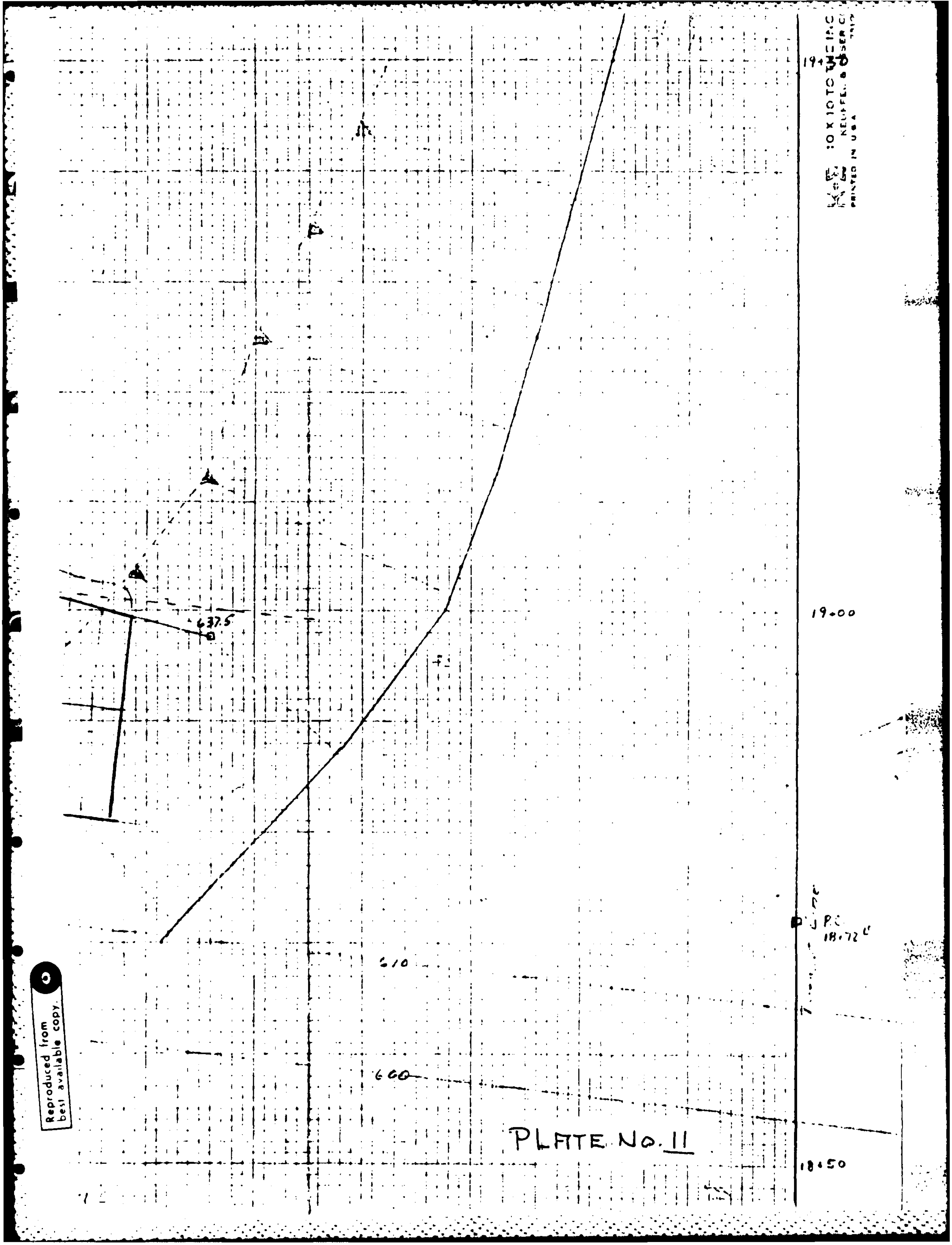


PLATE NO. 11

SHOT No. 5
Date: 22 Feb. 1977
How Drilled: 3700 Air Tracks
Hole Diam: 3"
Hole Depth: 10' Approx.
Hole Spacing: 6 x 8
Stemming: Crushed Stone
Explosive: Primed w/ Detaprime and loaded w/Anfo to within 6'
Firing Data: As Shown

SHOT No. 4
Date: 21 Feb. 1977
How Drilled: 3700 Air Tracks
Hole Diam: 3"
Hole Depth: 11' Approx.
Hole Spacing: 6 x 8
Stemming: Crushed Stone
Explosive: Primed w/ Detaprime and loaded w/ Anfo to within 6'
Firing Data: As Shown

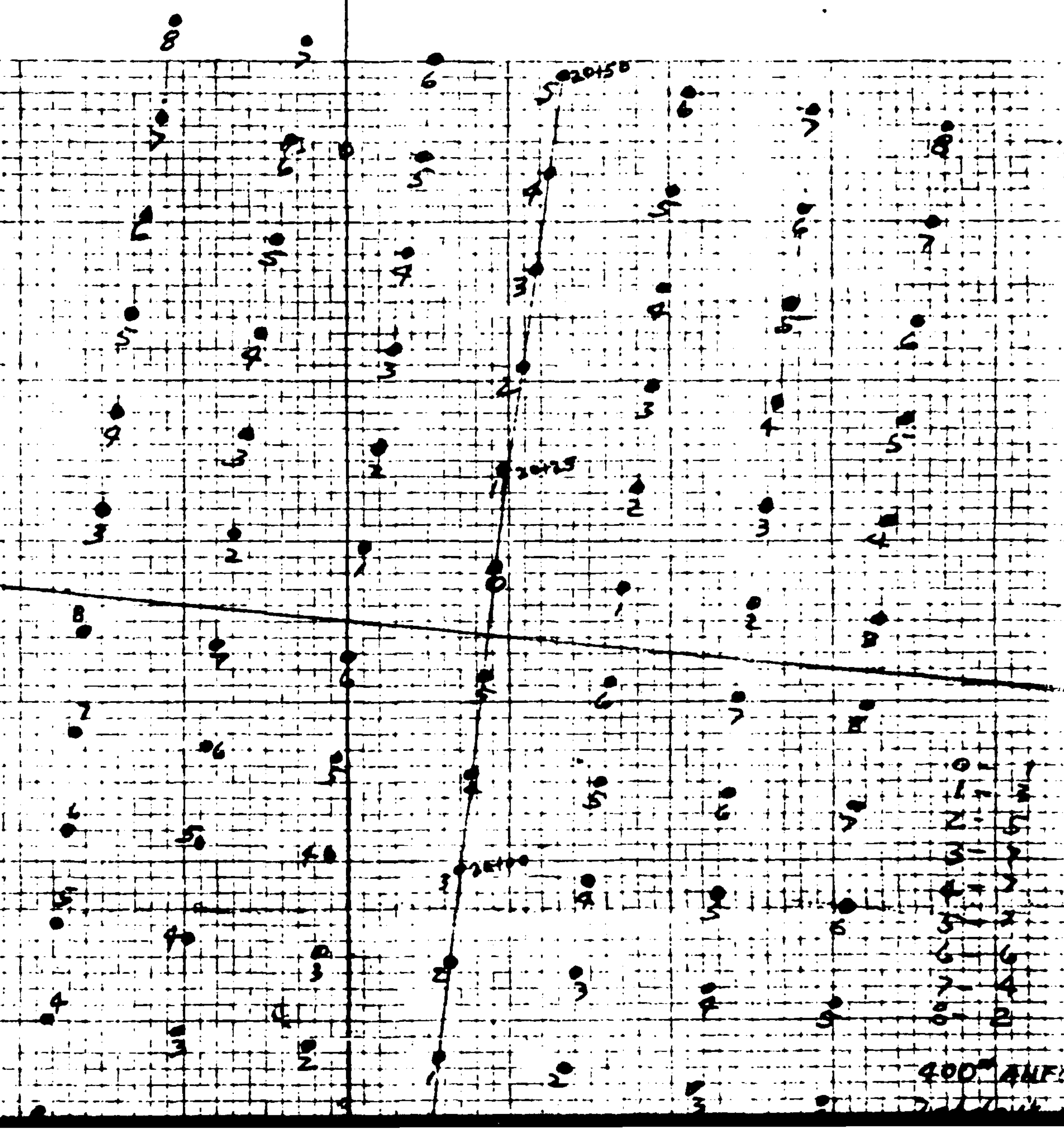
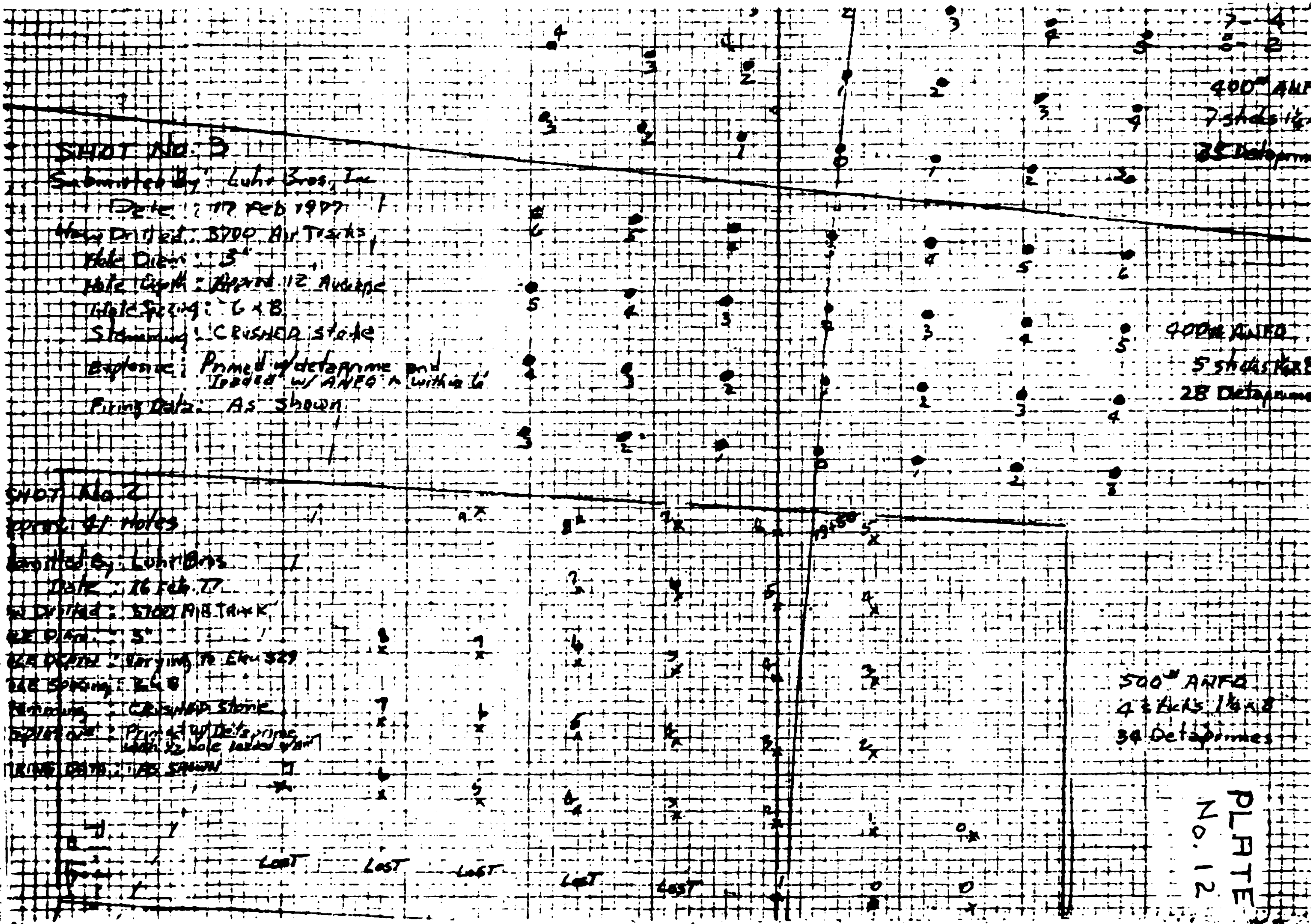


PLATE No. 12


SHOT No. 3
Submitted by: Luhr Bros, Inc
Date: 17 Feb 1977
Holes Drilled: 3700 Air Tracks
Hole Diam: 3"
Hole Depth: 12 Average
Hole Spacing: 6 x 8
Stemming: CRUSHED stone
Explosive: Primed w/ detaprime and loaded w/ ANFO within 6'
Firing Data: As shown
400# ANF
7 sticks
28 Detaprime
400# ANFO
5 sticks
28 Detaprime
SHOT No. 2
Hole Diam: 5"
Hole Depth: Varying to Elev 329
Hole Spacing: 8 x 8
Stemming: CRUSHED Stone
Firing Data: AS SHOWN
LOST
500# ANFO
34 Detaprimes

The production boring diameter was 3 inches and the holes were drilled vertically. Generally, the production shot drilled pattern principally consisted of square patterns which varied from 6 feet by 8 feet, 6 feet by 6 feet or 5 feet by 7 feet. The delay patterns were generally arranged in a vee cut or in an echelon arrangement. The patterns utilized (millidet) delay periods ranging from instantaneous to 250 milliseconds. The blasting caps were primed with Detaprime primers.

The types of explosives utilized for primary blasting consisted of cartridges (1.25-inch by 8-inch) of Tovex 210 and 50-pound bags of ANFO-P manufactured by DuPont. The Tovex 210 is classified as a water gel, whereas the ANFO is an ammonium nitrate. The dry holes were loaded with ANFO to within 6 feet of the hole collar and then stemmed with crushed stone. The exceptions to the above loading procedure would be production holes located in fractured areas, wet areas or if the depth of a production hole was less than 6 feet the hole was loaded with one or two cartridges of Tovex 210. The powder factor for primary blasting averaged one pound per cubic yard (refer Plate No. 12 for a typical production shot plan).

Due to the proximity of the Corps Soils and Geology Lab, the Contractor's conveyor system and the second-stage access, the Contractor utilized rubber blasting mats to cover the majority of the production shots.

In order to ensure the safety of the Contractor's labor force, after each step of primary blasting, the drill crews would scale the newly exposed 1V:1H rock slopes with picks, shovels and compressed air for the removal of all loose rock fragments and fractured rock. The Contractor followed a practice of utilizing his night shift for the majority of all scaling operations in order to ensure a continuous drill-blast cycle.

In order to provide equipment access for disposal of the newly shot rock to depleted portions of Borrow Areas Nos. 2 and 3, the Contractor first dozed shot rock from the cutoff trench and the initial shot rock over the abutment face and onto the north bank of the diversion channel. This procedure allowed the Contractor to establish a berm of sufficient width for equipment access. After the establishment of the berm, the Contractor transported the remaining shot rock directly to the waste area. Approximately 110,000 yards of rock were excavated along the abutment face, 43,000± yards of rock were used for the upstream revetment on the third-stage cofferdam and the remainder wasted in depleted Borrow Areas Nos. 2 and 3.

After the excavation of the last limestone lift, the Contractor placed multi-colored sand bags along the newly exposed Chouteau Limestone-Hannibal Shale contact. Protection of the formation contact was required to prevent deterioration due to the highly air sensitive nature of the Hannibal Shale.

During the period of the left abutment excavation, the discovery of extensive solution features that could jeopardize the integrity of the structure dictated a comprehensive invesgitaiton and re-evaluation of the original design criteria by SLD. The findings of the investigations were presented as a report entitled "Report on Positive Cutoff Treatment, Cannon Dam Left Abutment" during the Geotechnical Conference on 7 and 8 September 1977. Due to its comprehensive nature, Appendix A (dealing with exploration and geology) follows this paragraph. The second part of the Report dealing with treatment will be presented in the Left Abutment Modification narrative. Drawing No. 137/2 shows the locations of the cavity detection borings and the locations of the various cavities.

APPENDIX A

REPORT ON POSITIVE CUTOFF TREATMENT
CANNON DAM LEFT ABUTMENT
7, 8 September 1977
EXPLORATION AND GEOLOGY

2-01 EXPLORATION.

Excavation of the left abutment cutoff trench began on 28 February 1977, exposing the upstream end of the first major solution cavity at Station 19+40. Exposure continued, and this cavity was inspected by SLD and LMVD personnel on 28 March 1977. At this time, the cavity was recognized as a significant foundation problem, and a meeting with OCE was scheduled.

When the conference was held on 12 April 1977, Cavities Nos. 2 and 3 at Station 19+75 and approximate Station 18+65, 200 feet upstream, had also been exposed. The decision was then made to cleanout the cavities to their full extent and to sound the cutoff trench using air track drills. The drilling was started on 25 April 1977 upstation of Cavity No. 1 and proceeded to the end of the cutoff trench. Initially, 4-inch vertical rock bit holes were pneumatically drilled to El. 610 feet NGVD approximately 40 feet deep in a 7-foot by 7-foot staggered pattern. At Station 20+10 and thereafter, the pattern holes were drilled at 30° from vertical to increase coverage. When voids were encountered, supplemental holes were drilled at varying angles and spacings to delineate their extent. Vertical holes encountering voids were caliper logged.

Cavity No. 4 was encountered by the pattern drilling at approximate Station 20+40 in the cutoff trench. Shortly thereafter, all work at Cannon Dam was halted by a strike which lasted from 1 May to 6 July 1977.

On 7 July 1977, approximately 5 feet of rock were removed by blasting from the upstream embankment contact area, exposing Cavity No. 5. The pattern drilling continued in this area and a line of holes 60-foot deep at Station 19+25 was drilled in order to intersect a possible upstream extension of Cavity No. 1.

Pattern drilling in the upstream embankment contact area was finished on 15 July 1977 and began in the downstream embankment contact area at that time. A highly solutioned zone was detected downstream of Cavity No. 2 and delineated by vertical holes.

LMVD and SLD met at Cannon Dam on 21 July 1977 and inspection revealed caverns opening downstream of both Cavities Nos. 1 and 2. It was decided that cleanout of Cavity No. 1 should continue and that pattern drilling in the downstation cutoff trench and on the 1V:1H slope within the cutoff trench area should be expedited. Therefore, the pattern drilling was done concurrently in the upstream embankment contact area, downstation cutoff trench and 1V:1H cutoff trench area, and drilling rigs were deployed according to the availability and accessibility of cleaned rock surfaces. Holes on the 1V:1H slope were drilled normal to the slope, 20 feet deep. During this time, a void system (Cavity No. 6) was detected in the cutoff trench at approximate Station 19+00, extending downstream at least 100 feet.

On 10 and 11 August 1977, an impulse radar survey of the left abutment was initiated but was not completed. The highly irregular surface of the abutment, coupled with intense reflections from metal construction equipment and attenuation due to the signal of residual clay, rendered the method infeasible.

On 12 August 1977, six rock samples were taken from the left abutment to be tested for relative solubility rates and insoluble residues. The results of the solubility test are listed on Sheet Nos. 10 thru 12 of this Appendix.

By 15 August 1977, the basic pattern drilling of the cutoff trench, embankment contact area and 1V:1H cutoff trench area was complete. During the following week, additional rock bit borings were done in the major cavity areas, and NX core borings were drilled from the bottom of Cavity No. 1. All drilling was completed on 20 August 1977.

2-02 CAVITIES.

a. Cavity No. 1 occurs along a joint striking N78°E, crossing the dam axis at Station 19+35. The joint has a near vertical dip and its width varies vertically and laterally from 30 feet to less than 1 inch. It is open to a depth of 4 feet below grade (Elevation 630 feet NGVD) under the upstream 1V:1H slope, and has been penetrated to 25 feet at 15 feet upstream. The pipes downstream of the centerline were originally filled with residual clay and chert which have been excavated to a depth of 40 feet. Dye was introduced into the pipes and was observed to reappear at the shale/limestone contact along the Salt River 400 feet upstream of the centerline.

To facilitate cleanout, the partition between the two pipes was blasted out. When the excavation reached Elevation 584 feet NGVD, a room was found 5-foot wide, extending 6 feet downstream and terminating ina solutioned joint, with a clay floor at Elevation 580 feet NGVD.

On 14 and 15 July 1977, a line of angle holes 30° from vertical was drilled at Station 19+25 upstream of the cutoff trench in order to intersect any extension of Cavity No. 1 at approximate El. 615 feet NGVD. The drilling indicates that the joint associated with Cavity No. 1 does extend upstream at least 40 feet and is solutioned in some areas to widths of 15 feet and clay filled. On 15 through 23 August 1977, NX core holes were drilled from the bottom of the downstream pipe into the Hannibal Shale and it was found that the joint at El. 584.5 feet NGVD extended downward at least 19.3 feet to approximate El. 565 feet NGVD.

b. Cavity No. 2 was discovered as the cutoff trench was brought to grade on about 15 April 1977. At this time, most of the cavity was bridged by a chert layer at approximate El. 635 feet NGVD. On 7 July 1977, the chert bridge was removed by blasting and it was apparent that an opening of considerable depth extended downstream.

Pattern drilling has shown a complex void system about 60 feet downstream of Cavity No. 2 extending approximately 150 feet downstream of the centerline. This data indicates an irregular branching solutioned joint originating in the cutoff trench and underlying the downstream embankment contact area. Branches of this cavity have been encountered by drilling as wide as 10 feet at elevations ranging from 635 feet NGVD to 580 feet NGVD, both open and clay filled. Drilling on 20 August 1977 verified clay depths in excess of 580 feet at Station 19+75.

c. Cavity No. 3 consists of two solution pipes each approximately 4 feet in diameter along a highly solutioned joint trending approximately N84°E with a near vertical dip. The upper portions of the two pipes were exposed on or about 15 April 1977 as the upstream 1:1 excavation was initiated. At this time, both of the pipes were filled with residual clay and chert. The solutioned zone of this joint does not appear to extend below El. 600 feet NGVD; however, no drilling has been done in this area.

d. Cavity No. 4 is a void system with no surface expression indicated by the cutoff trench pattern drilling. The first holes penetrating this cavity were drilled on 29 April 1977 and several holes were drilled thereafter to delineate the zone. The borings show that this cavity consists of several branching, sinuous passages; the maximum thickness encountered was 5 feet and the maximum height was 15 feet. The voids are predominately clay filled and occur from near surface (El. 645± feet NGVD) to El. 604 feet NGVD. Dye was introduced into borings penetrating Cavity No. 4 and was observed to reappear in a spring near the Chouteau Limestone/Hannibal Shale contact at approximate Station 24+00, 800 feet upstream.

e. Cavity No. 5 was exposed on 7 July 1977 when the upstream embankment contact area was excavated by blasting to El. 640 feet NGVD. The cavity has an irregular shape, approximately 15 feet by 25 feet, and is clay filled. Pattern drilling in the area indicates that the cavity extends vertically at least 60 feet, and open and clay-filled voids up to 10 feet in width were found by angle holes and pulse radar as far as 30 feet northeast of Cavity No. 5.

f. The void system comprising Cavity No. 6 was first penetrated on 2 August 1977 by the downstation cutoff trench pattern drilling. At Station 18+95, a void was encountered near the surface and was jack hammered open. Drill data shows voids as wide as 10 feet from El. 620 feet NGVD to El. 558 feet NGVD, extending 120 feet downstream.

2-03 GEOLOGY.

As thus far determined, solution activity on the left abutment primarily affects the rocks of the Chouteau and Lower Burlington Limestones with some activity also in the Upper Burlington. The Upper Burlington is a cherty limestone showing numerous small unconformities and a significant basal unconformity. Being the uppermost rock exposed, the zone of general weathering is primarily confined to this member. Although solution openings do exist along joints and in the basal portion (in associated with the major cavities), the general weathering is more significant. Solutioning shows the influence of local chert layers resulting in hourglass cross sections. Overlying the upper Burlington are various associations of residuum, Pennsylvanian deposits and glacial materials.

The Lower Burlington is a relatively massive, medium to coarse crystalline limestone, while the Chouteau is medium to thinly bedded, argillaceous limestone having thin shale partings. Solution activity has occurred along the joints and locally large pipes have been developed. The pipes show the majority of their development within the Lower Burlington (20-foot thick) and the upper portion of the Chouteau. Toward the lower part of the Chouteau, some enlargement is noticed along the horizontal, while the pipes appear to die out. No large horizontal solution passages have been detected as of this time from the drilling program.

The pipes were noted to be roughly circular but coalesce into assorted shapes. Fluting along the sides is typical of subsurface vadose or near phreatic solutioning. The uppermost part of the large pipes occasionally extended a few feet into the Lower Burlington but shows a lesser amount of solution development. Connecting joints may be extremely narrow and, unlike the pipes, are generally open. The controlling joint set runs N80°±E and is generally vertical.

In addition to the major pipes, general solutioning along joints and within the Upper Burlington has occurred. This solution activity appears to be much younger than the solutioning which created the pipes and under conditions more similar to the present.

Underlying the soluble limestones is the Hannibal Shale which is more properly called an air slaking siltstone. This formation generally forms the base of solution activity since it acts as an impermeable barrier to the migration of water.

2-04 JOINTS.

Excavation and surface cleanoff of the left abutment has exposed numerous joints comprising one major set and two minor sets plus several random minor joints. To date, 30 significant joints have been mapped by the Cannon Resident Office and St. Louis District Office on the left abutment embankment contact area and cutoff trench excavation.

The dominate joint direction is basically east-west with 57 percent of the joints mapped falling into this zone (Zone 1). This information corresponds with both regional jointing as mapped in DM 4A, Site Geology of Clarence Cannon Dam (June 1966) and of the detailed mapping of the joints on the right abutment and main dam excavations.

The primary zone of joints falls between N80°W and N80°E with six of the seven solution cavities having joints falling within this zone. The seventh major joint falls into a second zone, N60°E to N75°E, which comprised 17 percent of the joints mapped.

A third minor joint zone extending from N50°E to N60°E was mapped comprising only 13 percent of the total joints mapped. The strike of 93 percent of the joints falls within 40° of east-west.

The formation of solution cavities shows a high degree of control by the joint systems with the widening of joints resulting in the formation of solution pipes. This condition can be particularly noted when the intersection of two or more joints occurs.

The dominate dip angle of the joints is vertical with nearly all joints of Zone 1 falling in this category.

JOINT LIST
LEFT ABUTMENT
CANNON DAM

----------Joint----------			-----Location-----		
1.	EW	Vert.	20+44	35' US	
2.	EW	Vert.	18+90	140' US	*
3.	EW	Vert.	18+75	129' DS	
4.	EW	Vert.	19+20	35' US	
5.	N87°W	Vert.	18+96	38' US	
6.	N87°W	85°N	19+80	30' US	
7.	N87°E	78°N	18+65	110' US	
8.	N86°E	Vert.	18+60	110' US	
9.	N85°W	87°N	19+60	℄	
10.	N84°E	Vert.	19+40	℄	*
11.	N87°E	Vert.	18+50	190' US	*
12.	N83°E	Vert.	18+63	40' US	
13.	N84°E	Vert.	18+70	125' US	
14.	N83°E	Vert.	18+65	75' US	
15.	N80°W	Vert.	20+50	℄	*
16.	N85°E	Vert.	18+50	200' DS	*
17.	N84°E	Vert.	19+78	32' DS	*
18.	N75°E	Vert.	18+64	140' DS	
19.	N60°E	Vert.	19+99	35' DS	
20.	N75°E	Vert.	18+60	140' DS	
21.	N72°W	Vert.	20+15	36' DS	
22.	N72°W	Vert.	20+13	36' DS	
23.	N60°E	Vert.	18+90	75' DS	
24.	N69°E	Vert.	18+72	120' DS	*
25.	N55°E	Vert.	19+12	28' US	
26.	N50°E	80°S	19+10	30' US	
27.	N57°E	Vert.	18+96	20' US	
28.	N50°E	Vert.	18+95	20' US	
29.	N23°E	Vert.	18+90	130' US	
30.	N19°W	Vert.	18+50	190' US	

*Indicates major joints

SOLUBILITY RATES AND INSOLUBLE RESIDUE DATA

The attached curves are derived from periodic weighing of 6 pairs of 1/2"x2"x2" tiles, each simultaneously immersed in 3 liters of 3% solution of hydrochloric acid. After three hours, the acid was changed, the samples were completely dissolved, and the insoluble residue was weighed.

SAMPLE LIST

#1 Sta 19+60, 25' d.s., el. 630±

Upper Burlington Formation - Brown, very fine crystalline silty limestone.

#2 Sta 19+40 on centerline, el. 618±

Upper Burlington Formation - Very light brown - pink coarse to medium crystalline slightly silty limestone.

#3 Sta 19+30 on centerline, el. 615±

Lower Burlington Formation - Gray coarse to medium crystalline, fossiliferous, stylolitic limestone.

#4 Sta 18+75, 120' U.S., el. 605±

Lower Burlington Formation - Gray coarse to medium crystalline, fossiliferous limestone.

#5 Sta 18+65, 200' U.S., el. 580±

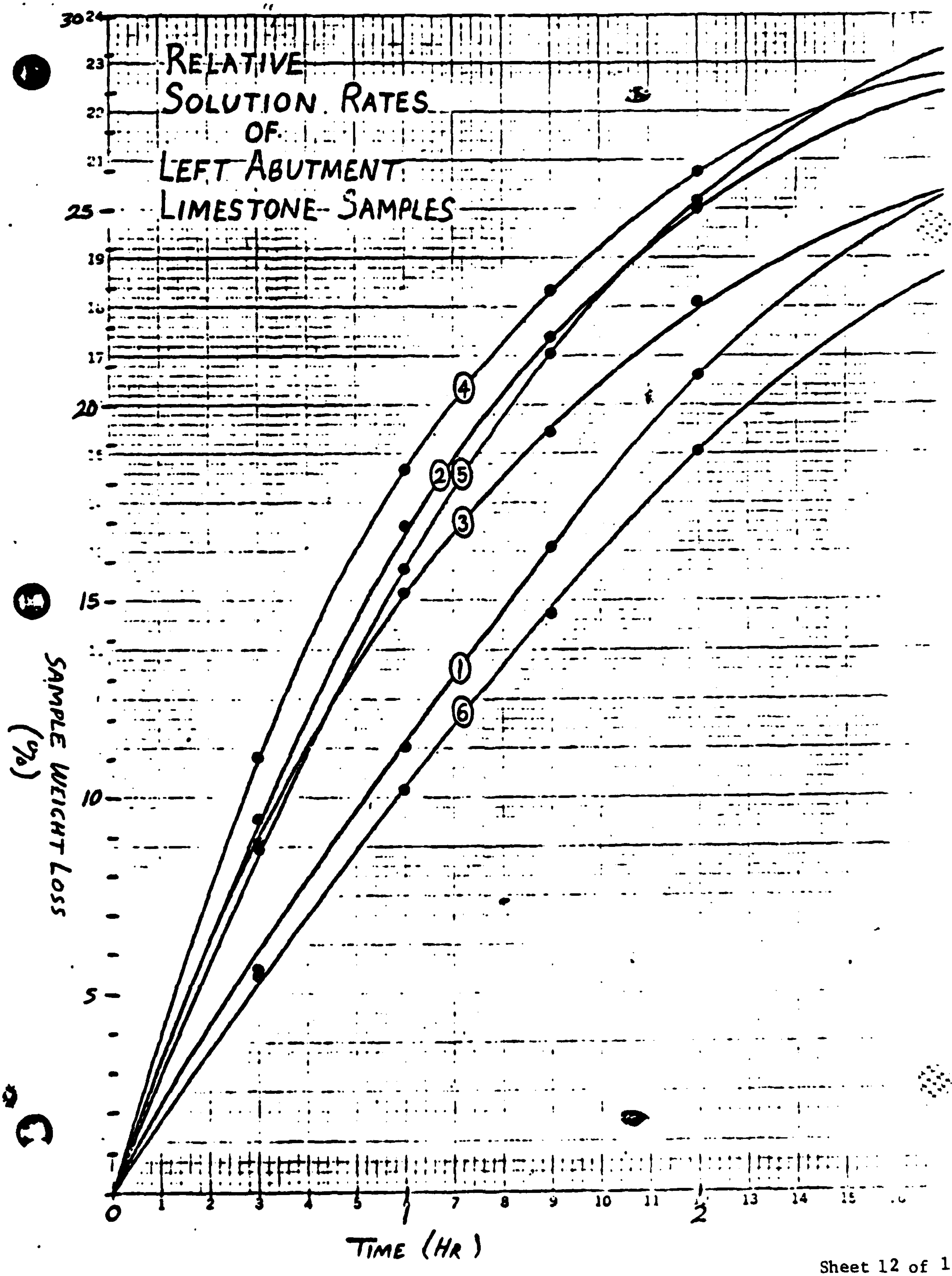
Upper Chouteau Formation - Gray fine crystalline to sublithographic argillaceous limestone.

#6 Sta 18+30, 125' d.s., el. 560±

Lower Chouteau Formation - Gray mottled brown fine crystalline to sublithographic silty, argillaceous limestone.

INSOLUBLE RESIDUES

Sample #1	10.1%
#2	2.9
#3	1.2
#4	1.2
#5	7.4
#6	31.0



RELATIVE SOLUTION RATES OF LEFT ABUTMENT LIMESTONE SAMPLES
SAMPLE WEIGHT LOSS (%)
TIME (HR)
0
1
2
5
10
15
20
25
30
4
2
5
3
1
6

Boring Totals

Cutoff Trench		
Upstation	=	186
Downstation	=	119
Downstream	=	234
Upstream	=	96
1 on 1	=	187
TOTAL:		822 Borings

(2) Left Abutment Modification

[a] Re-evaluation of Cutoff Trench

The findings of the cavity detection program and exposure of extensive solution features within the left abutment cutoff trench and embankment contact area during excavation dictated that an elaborate plan of treatment be formulated by the design elements. A comprehensive review of various plans of treatment was undertaken during the 7 and 8 September 1977 Geotechnical Conference and the scope of work was finalized into Modification No. P00085.

The modification included the following abutment work:

1. Redesign and re-excavation of the left abutment cutoff trench and the development of a 200-foot concrete vee-shaped cutoff wall from Station 18+00 to Station 20+00±.

2. Cleanout and the placement of concrete backfill in the cutoff wall and cavities.

3. Treatment of the downstream ravine.

4. Development and implementation of new drilling and grouting specifications.

5. Placement of concrete fillets within the left abutment embankment contact area.

The modification work began in mid-December 1977 and was essentially complete by mid-November 1978. A short description of each phase of work is presented below.

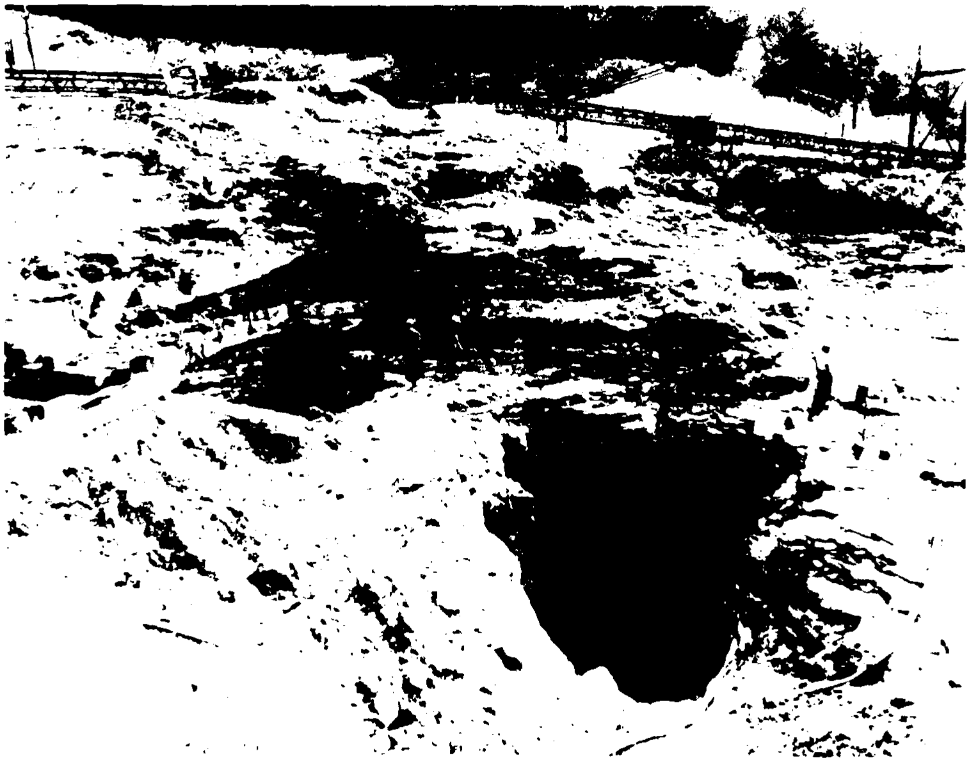
The first order of work performed by [illegible] Bros., Inc., was the re-excavation of the left abutment cutoff trench to lines and grades shown on Drawing No. 120/2. The original contract specifications concerning presplitting, primary blasting and scaling, as outlined earlier in the narrative, were followed. Drilling for presplitting began on 13 December 1977 and the last production shot was detonated on 21 March 1978. The only exception was the rock excavation in the rear of the cutoff trench beyond Station 20+60 due to the presence of Massman Construction Co.'s conveyor. The remaining portion of the cutoff trench was presplit from 9 through 31 August 1979 and shot to grade by primary blasting on 4 September 1979. Generally, the Contractor (Luhr Bros., Inc.) utilized a single-shift operation for the majority of the cutoff trench re-excavation and started two-shift operations on 13 March 1978 for cutoff trench scaling and ravine work. Generally, the same equipment as described in Part 1 was utilized for the re-excavation of the left abutment.

The types of explosives, the loading procedure and the delay patterns for presplitting and primary blasting were essentially the same as utilized during the construction season of 1977 for the left abutment excavation (refer Plates Nos. 13 and 14 for a typical presplit and production shot plan). The scaling procedures were similar to the earlier operation with the exception that the top edge of the presplit was cleaned back 5 feet with a CAT 245 backhoe and safety mesh (chain-link fabric) was anchored from the top of the cutoff trench to 3 feet below the chert horizon. The safety mesh extended from Station 20+16 to the upstream nose of the cutoff trench in order to protect the Contractor's labor force during excavation and concrete backfill of the cutoff wall.

The shot rock was either pushed over the face of the cutoff trench and hauled with Terex end dumps to waste areas in depleted Borrow Areas Nos. 2 and 3 or used to fill the cavities for access during the drill-blast cycle.

The second order of work performed by Luhr Bros., Inc., in the cutoff trench was the rock excavation and cavity cleanout for the placement of a concrete wall from Station 18+00± to Station 20+00±. The modification required that a 200-foot vee-shaped slot be excavated in the lower Burlington and Chouteau Formations by presplitting and primary blasting in 20-foot lifts from El. 610 feet NGVD to El. 550 feet NGVD. The design width of the slot was 14 feet from El. 610 feet NGVD to El. 590 feet NGVD, 12 feet from El. 590 feet NGVD to El. 570 feet NGVD and 10 feet from El. 570 feet NGVD to El. 550 feet NGVD. The width requirements were later modified to 16-foot, 14-foot and 10-foot in order to accommodate the use of the Contractor's air tracks.

Left Abutment Excavation

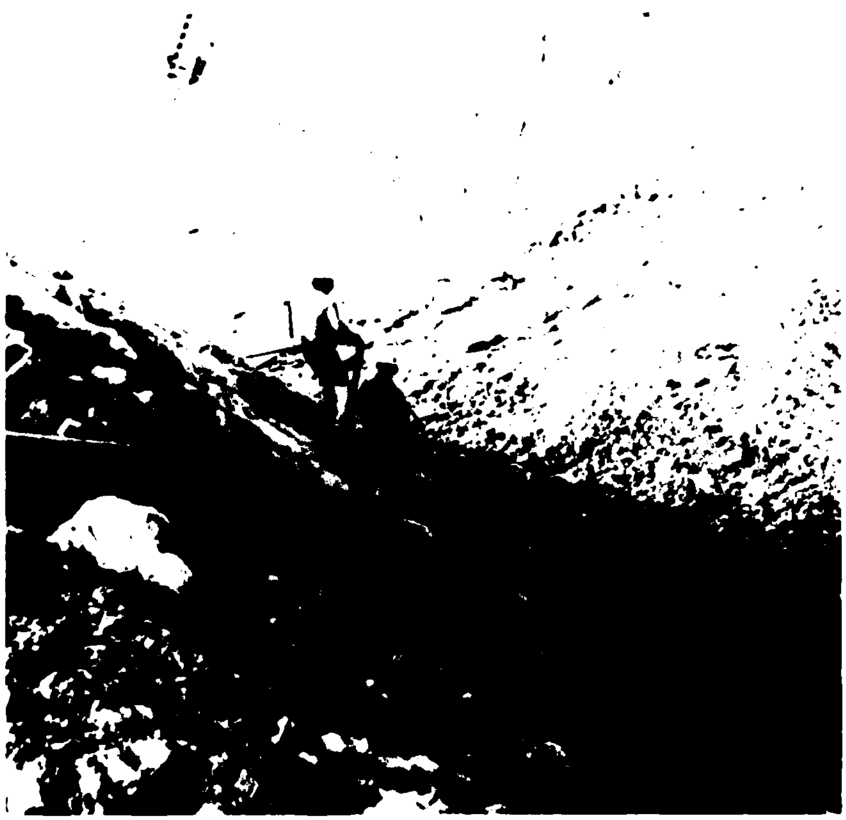


Left Abutment Cutoff Trench, Exposure of Cavities Nos. 1 and 2

Drilling presplit holes along the downstream slope of the cutoff trench.

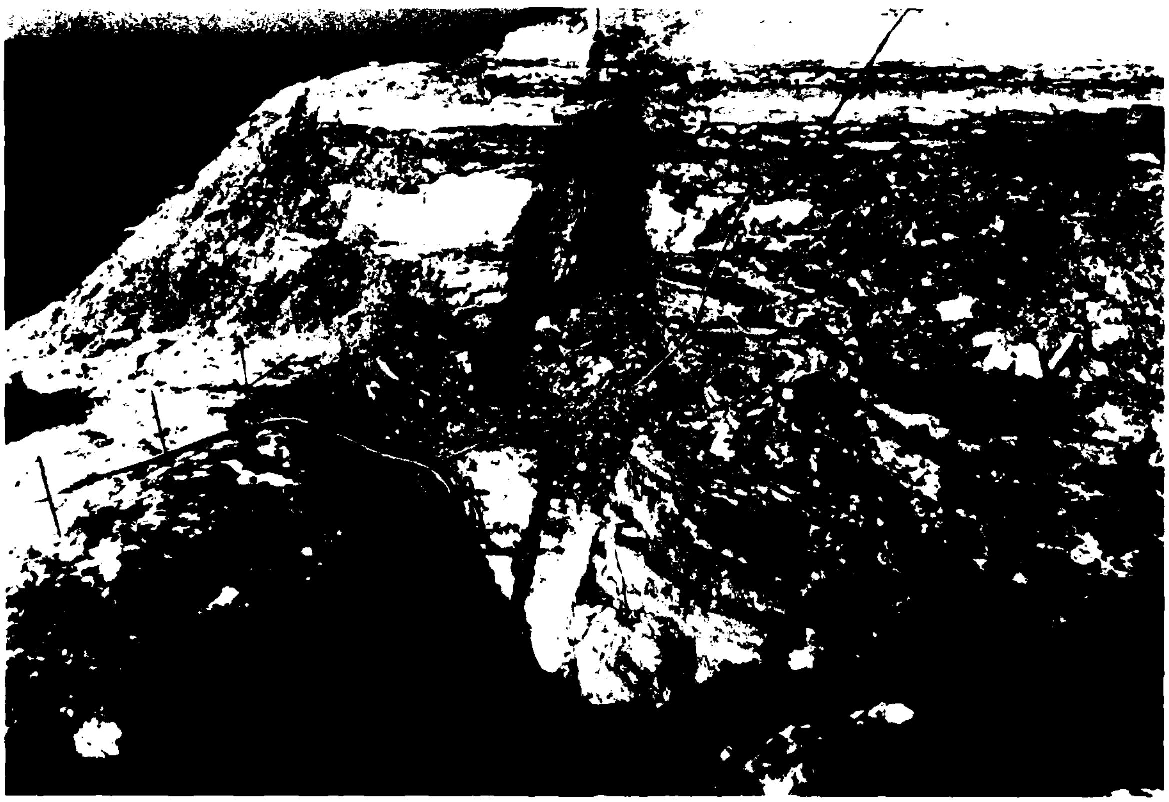
Drilling presplit holes along the north slope of the cutoff trench.

CAT 245 backhoe excavating shot rock from back (north) slope of the cutoff trench.



Laborers cleaning downstream slope of the cutoff trench with compressed air.

Cavity No. 1 delineating degree of cleanout.



Cavity No. 1

In order to construct the positive cutoff wall according to guidelines presented and approved during the 7 and 8 September 1977 Geotechnical Conference and the SLD/LMVD inspection on 1 June 1978, the original contract specifications were modified in the following manner:

1. Blasting

a. Contractor shall submit primary and production shot plans a minimum of 8 hours prior to commencing drilling and the plan shall not be submitted until the completion of the previous shot.

b. Required shot plan parameters: Plan layout depicting Burden/Spacing Ratio and delay patterns; section view depicting the loading and stemming of a typical hole, hole diameter, angle, type of explosives, charge per foot and pounds of explosives per cubic yard of rock.

c. Presplitting: 3-inch diameter hole on 18-inch centers with every other hole loaded and a maximum hole depth of 20 feet.

d. Primary blasting: 2-inch diameter hole, 20-foot lift, a maximum total burden not greater than shot width and a powder factor no greater than 1.1 pounds per cubic yard. These parameters are dependent upon predicting a vertical free face and essentially horizontal throw.

e. The last lift shall have a maximum lift thickness of 5-foot with the last 1 and 1/2 feet of shale above final grade being excavated by means other than blasting.

2. Concrete Backfill for Cutoff Wall and Cavities

a. Areas directed by the Contracting Officer will require high-pressure air/water jets to remove clay coatings. The amount of cleanout in each feature will be established in the field and will depend on the type and condition of material in the feature, together with its location, upstream and downstream extent, size and accessibility of the area.

b. Top of Cavity No. 5 will be excavated for placement of a reinforced concrete cap.

c. Cavities with a dimension greater than 10 feet as measured along the cutoff wall excavation may require a vertical mat of steel.

d. The tops of all significant cavities or other joints and fractures will be widened to form a vee-shaped opening with side slopes not less than 30° from the vertical.

e. Where the cutoff wall excavation intersects a cavity, a form shall be erected along the neat lines, except as indicated below, and that cavity shall be filled with concrete either by bucket or by pumping to the top of the cavity. The concrete backfill in the cavity shall precede the concrete within the cutoff wall; however, in no case shall the differential between the concrete backfill of the cavity exceed the concrete within the cutoff wall by more than 30 feet.

f. Cavities smaller than 2 feet measured horizontal along the face of the cutoff wall or cavities having a relatively shallow depth measured normal to the cutoff wall regardless of horizontal width will generally not require forming.

3. Rock Reinforcement

Rock bolts and chain-link fabric will be installed on the vertical faces within the cutoff wall excavation. Chain-link fabric shall be removed before concrete placement. In order to ensure stability of the rock face, it will be necessary to install rock bolts from 5 feet to 20 feet in length.

4. Rock Protection

Horizontal final shale surfaces in the cutoff wall excavation shall be protected within one hour by concrete. Vertical final shale surfaces in the cutoff wall excavation will be protected by bituminous spray.

5. Treatment of Cavities

a. Cavity No. 1, Downstream - Cleanout rubble to extent previously cleaned out, expose rock; upstream - cleanout rubble, clean to firm natural cavity fill and cease. Perform standard dental treatment on joint extensions.

b. Cavity No. 2 - Cleanout rubble to firm natural cavity, fill and cease. Perform standard dental treatment on joint extension.

c. Cavity No. 6 - Cleanout rubble to firm natural cavity, fill and cease.

d. Cavity No. 3 - Remove all cavity fill to a depth of approximately 5 feet and backfill with concrete.

e. Cavity No. 5 - The concrete cap previously required is deleted. The cavity shall be backfilled with material from the surrounding area as directed in the field.

On 10 February 1978, SLD personnel and their blasting consultant from the U.S. Bureau of Mines met with the contractors and their DuPont blasting consultant to discuss and finalize the details for the initial primary blasting plan. All details of the shot plan, such as type and diameter of explosives, lift thickness, burden/spacing dimensions, loading density and time involved in submittal and approval of the shot plans were reviewed.

In addition, the details concerning cleaning and scaling, rock bolting and installation of safety curtain and anchors were outlined for a 20-foot lift. This was the first of a series of meetings between Contractor and Government personnel due to the exacting blast designs and the degree of fracturing that occurred in the side wall during primary bla_ 'ng for the first two lifts.

Drilling the presplit in the cutoff wall began on 5 April 1978 and the last production shot to El. 553± feet NGVD was detonated on 29 July 1978. Generally, the Contractor utilized a two-shift operation for the cutoff wall excavation. The Contractor followed the practice of scaling with jigger drills, installing clain-link fabric and rock anchors, rock bolting and cavity cleanout, and washing after the removal of each 20-foot lift. In addition, the newly exposed vertical rock wall was geologically mapped and photographed in conjunction with the above operations. The status of each cavity concerning the final degree of cleanout can be found in Drawings Nos. 134/2 and 135/2 and photographs on Pages 201, 204 and 205 for concrete placements within the cutoff wall.

The types of explosive and the loading procedure for presplitting were essentially the same as utilized during the construction season of 1977 for the left abutment excavation. Specification criteria of 18-inch presplit hole spacing was changed to 36 inches at the left abutment conference attended by SLD/LMVD/OCE personnel on 3 April 1978. The change was based upon the results of the presplit test panel located on the upstream slope in the cutoff trench (refer Plate No. 15 for a typical presplit shot plan for the last lift from El. 570 feet NGVD to El. 550 feet NGVD).

The type of explosive utilized for primary blasting in the cutoff wall consisted of 1.25-inch by 16-inch and 1.5-inch by 16-inch water gel cartridges of Tovex 220 and 210 manufactured by DuPont. The l^ad'ng density of the cartridges is 0.806 and 1.17 pounds per 16-inch stick, respectively. The smaller cartridges were utilized due to the specification requirement of a 2-inch diameter production hole and to eliminate the possibility of the cartridges "hanging up" in the shot hole. The production hole pattern was a square arrangement, whereas the delay pattern was generally vee shaped. The delays ranged from instantaneous to 250 milliseconds. The production hole depth for the first two lifts (El. 610 feet NGVD to El. 570 feet NGVD) was generally 20 feet except on the abutment face where the depths varied from 4.5 feet to 10 feet. The final lift was drilled to the Chouteau/Hannibal contact (El. 553± feet NGVD) and the hole was stemmed back with crushed stone cips for 1 foot. The collar of the hole was stemmed with crushed stone chips and varied in depth from 4 feet to 4.5 feet. In addition, crushed stone was used as decking in production holes which crossed joints and cavity fill material. The range of powder factors used in the three lifts was 0.875 pound per cubic yard (El. 610 feet NGVD to El. 590 feet NGVD), 1.13 pounds per cubic yard to 0.92 pound per cubic yard (El. 590 feet NGVD to El. 570 feet NGVD) and 0.98 pound per cubic yard (El. 570 feet NGVD to El. 553± feet NGVD). The shot rock was excavated by the CAT 245 backhoe and the CAT 977 end loader for the first lift and by the CAT 977 end loader for the remaining lifts (refer Plates Nos. 16 thru 23 for typical shot plans for each lift).

The specification requirement of drilling to El. 555 feet NGVD was amended to El. 553 feet NGVD in order to prevent the bottom lift from terminating within the basal massive bed of the Chouteau Limestone. This change facilitated rock excavation and prevented subsequent production shot to breakup the remaining 2 feet of limestone.

The variance noted in the above shot plans resulted from extensive fracturing in the cutoff walls and the increased confinement as the shots preceded upstation. The degree of fracturing for cutoff wall excavation is illustrated in Photographs on Pages 198, 199 and 200.

The remaining 2.5 feet and 3 feet of fractured Hannibal Shale were excavated to El. 550 feet NGVD principally by a CAT 977 end loader. The newly exposed shale walls were scaled and sprayed with bituminous coating. The shale walls were "touched up" with bituminous coatings, as necessary, until the completion of the first lift.

PRESPLIT SHOT NO. 6

How Drilled: Gardner-Denver 3700 Air Track

Hole Depth: Variable from original ground to El. 635 feet NGVD

Hole Diameter: 3 inches

Hole Spacing: 3-foot center-to-center at 45° upstream, 37° downstream
Holes with less than 6-foot burden will require 18-inch centers with every other hole loaded

Explosive: Tovex, T-1, 1.25 pounds per foot

Stemming: Top 5-foot of hole stemmed with crushed stone chips

Firing Data: No. 3 electric caps on each end of trunk line

Pre-split Shot No. 10

How Drilled : 3700 Air Track
Hole Depth : Variable from Original Ground to Elev. 635
Hole Diameter : 3"
Hole Spacing : 3 ft. Center to Center @ 45° U/S 37° D/S
Holes with less than 6' Burden will require 18" centers with every other hole loaded.
Explosive : Tovex T-1 1/4 lb./ft.
Stemming : Top 5 ft. of hole stemmed with crushed stone.
Firing Data : No. 3 Electric Caps on each end of Trunk line.

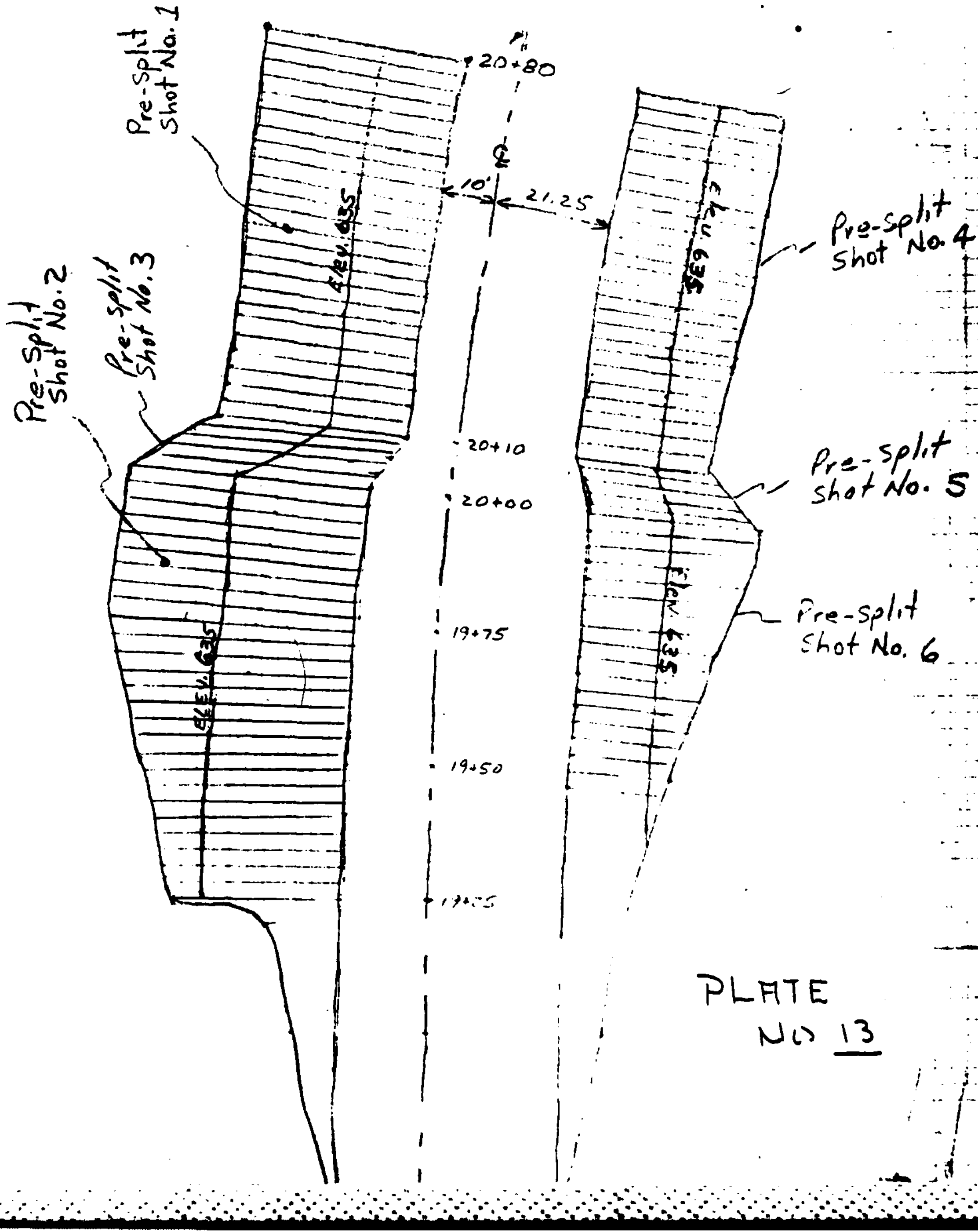


PLATE
NO 13

PRODUCTION SHOT NO. 5

How Drilled:	Gardner-Denver 3700 Air Track
Hole Depth:	Approximately 7.5 feet
Hole Spacing:	5 x 6 spacing
Hole Diameter:	3 inches
Firing Data:	As shown
Explosive:	ANFO with Detaprime
Stemming	Crushed stone chips

Maintain 4-foot stemming in holes. For holes 4.5 feet deep or less, use 6-inch ANFO with remainder stemming.

ANFO 600 pound

114 Caps

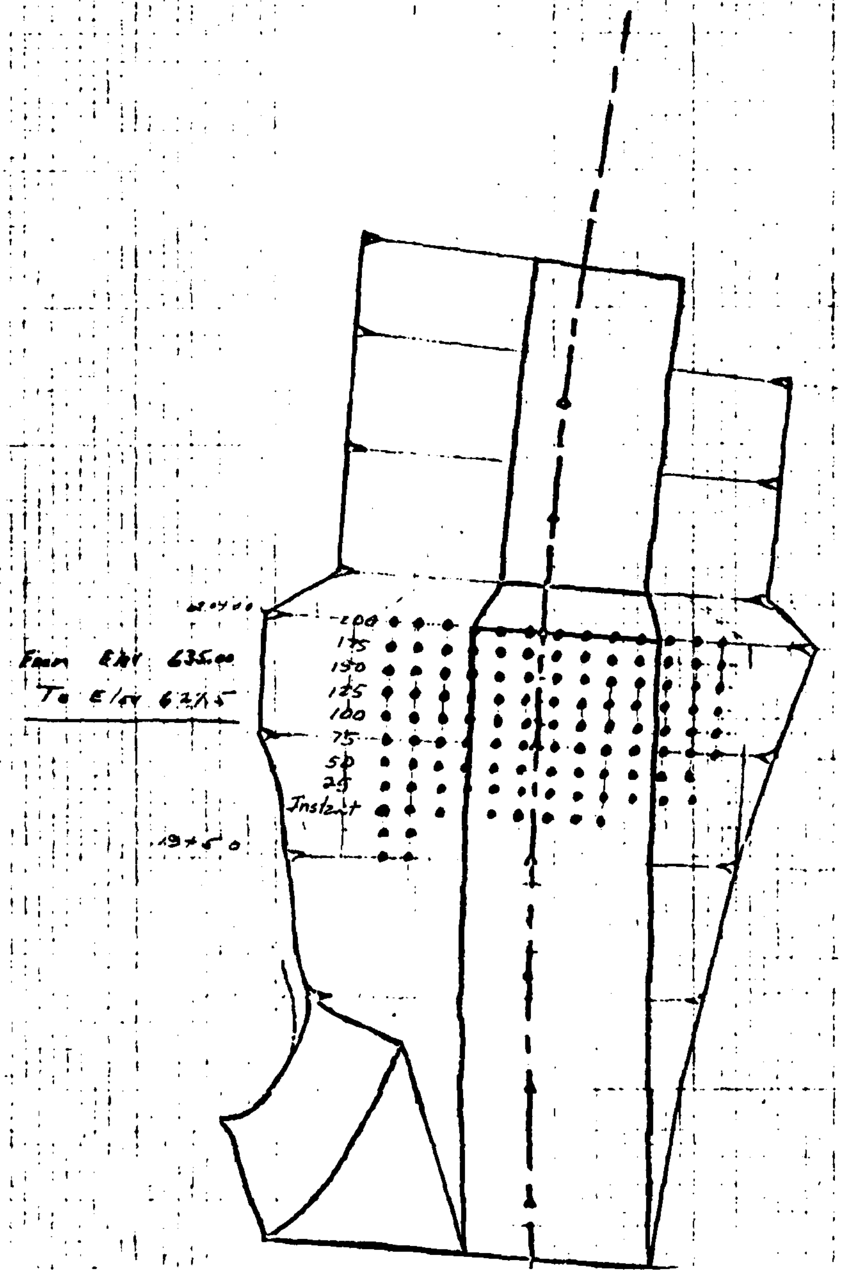
114 Detaprime

1 Roll Shot Wire

114 Holes Drilled and Shot
1 March 1978

Pr. uction Shot No. 5
How Drilled : 3700 Air Track
Hole Depth : Approx. 7.5 ft.
Hole Spacing : 5x6 Spacing
Hole Diameter : 3"
Firing Data : As Shown
Explosive : AnFo with Detaprime

stemming : Crushed Stone



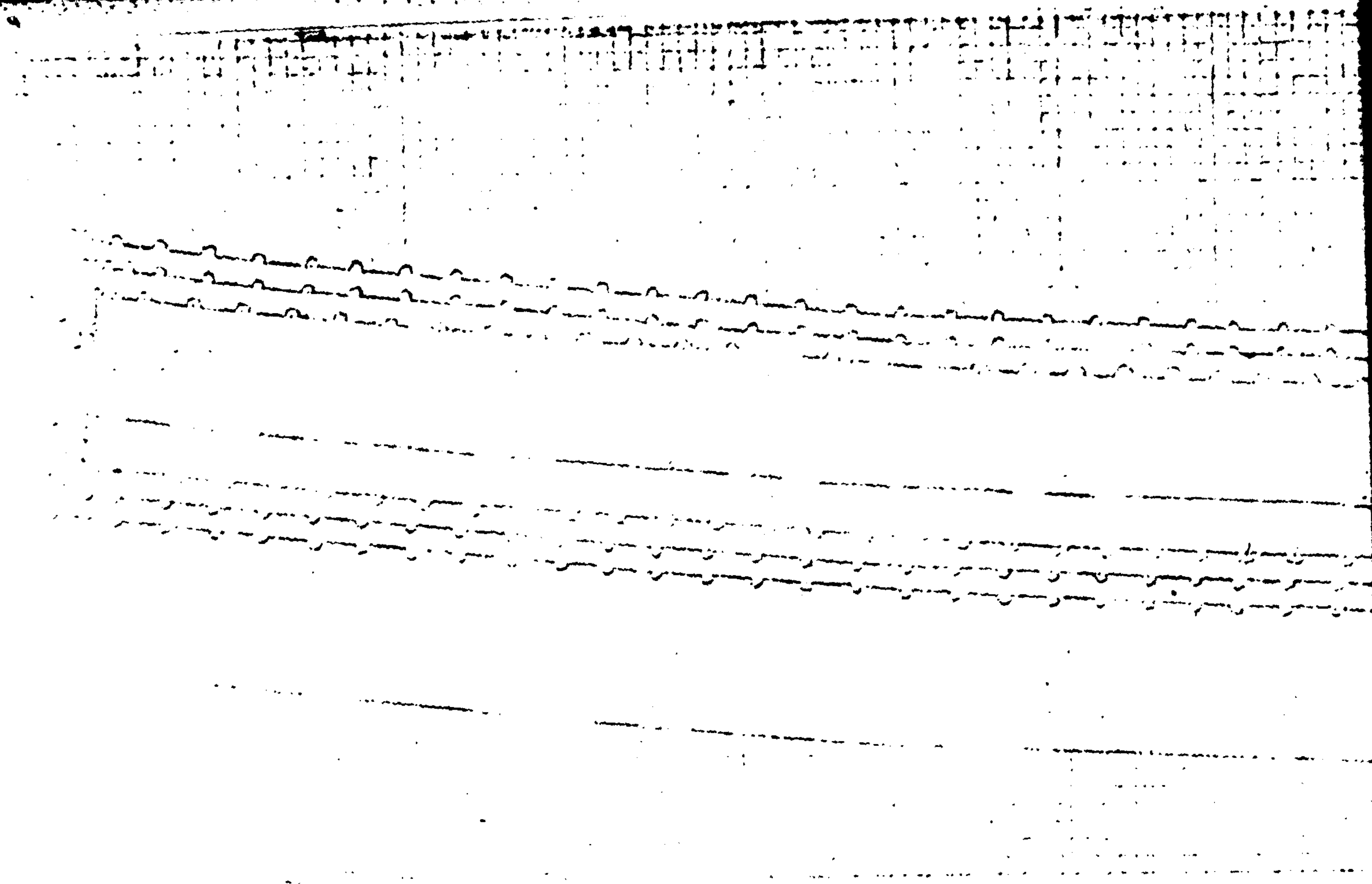
Maintain 4 ft stemming in holes. For holes 4.5 ft deep or Less use 6" AnFo with remainder stemming.

Approved 1 Mar 78
JHT

ANFO 600 LBS
114 Caps
114 Detn Prime
1 Rol Shotwire

114 Holes Drilled
AND Shot 1 March 78

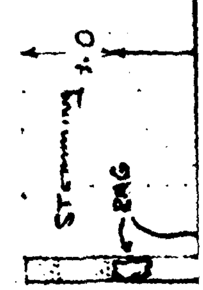
PLATE
NO 14



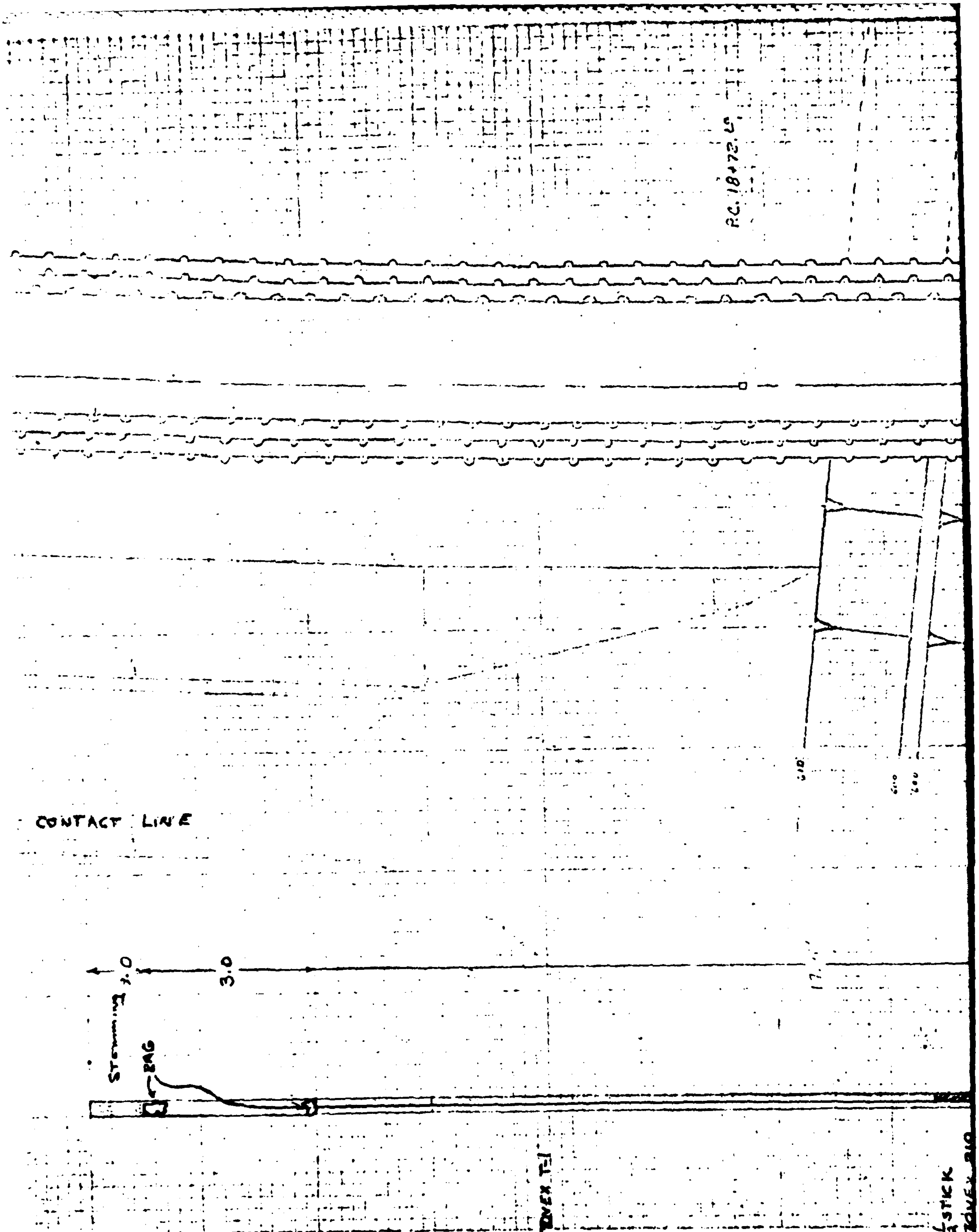
PRESPLIT SHOT No. 9 D/S WALL

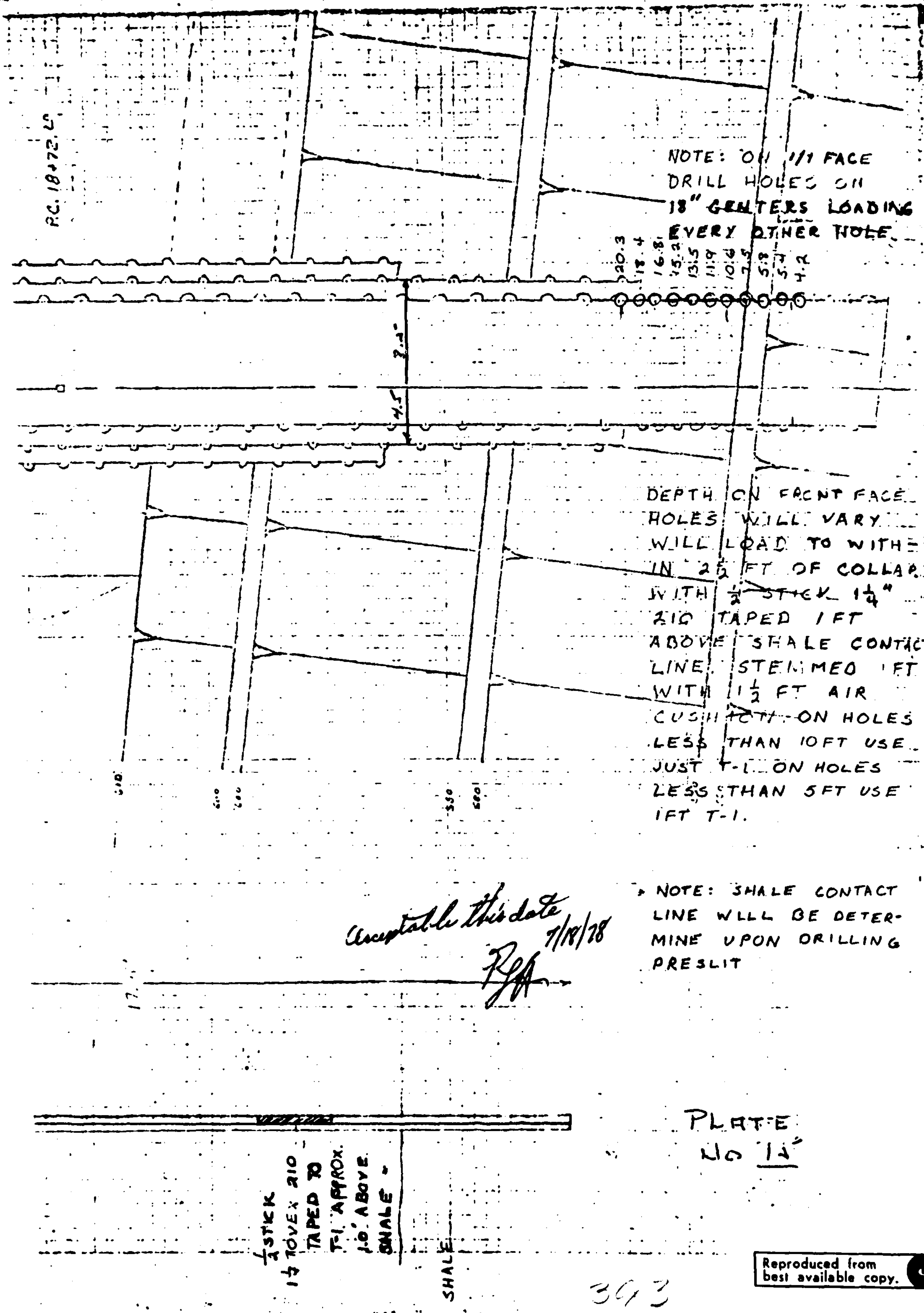
HOW DRILLED	:	3700 AIR TRACK
HOLE DEPTH	:	20 FT
HOLE SPACING	:	3.0 C.C EXCEPT ON FRONT FACE 1.5 C.C
HOLE DIAMETER	:	3"
STEMMING	:	CRUSHED STONE
EXPLOSIVE	:	TOVEX T-1 WITH 1 STICK 1 FT ABOVE SHALE CONTACT LINE

NOTE: W/S + D/S PRESPIT LINES TO BE DRILLED SIMULTANEOUSLY. SHOOTING OF LINES MAY REQUIRE ADJUSTMENT AS FIELD AND WEATHER CONDITIONS DEMAND



ELEV - 570 → 550


P.C. 18+72.0
CONTACT LINE
STEMMING
1.0
3.0
BAG
INDEX T-1
1/2 STICK



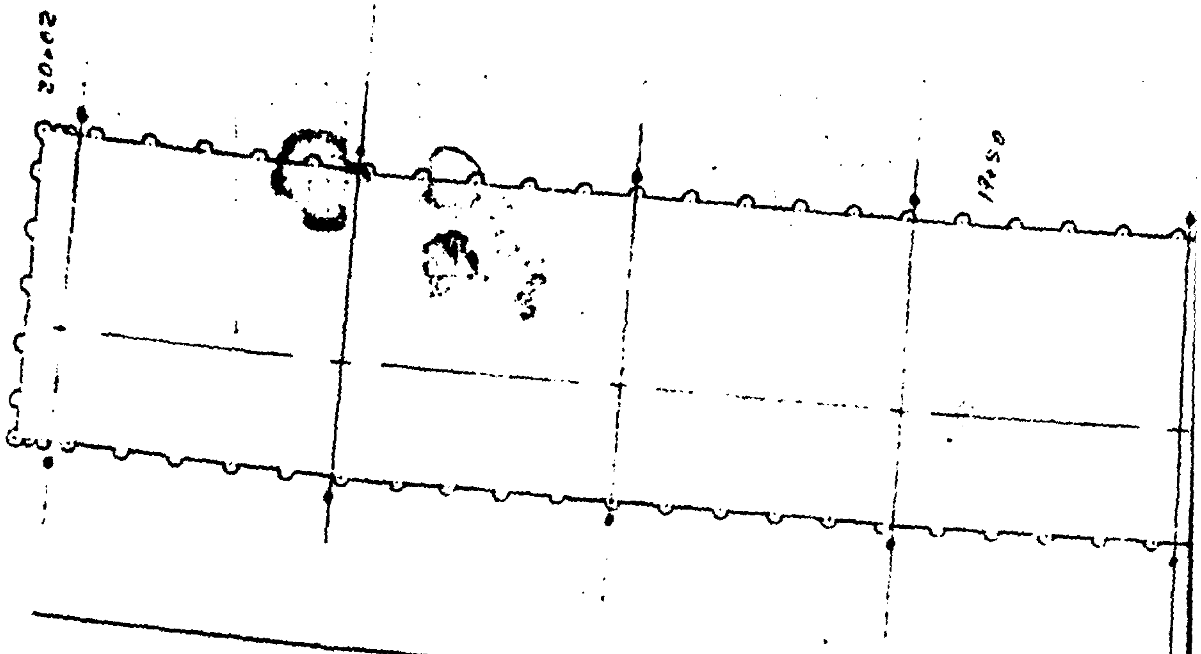
NOTE: ON 1/1 FACE DRILL HOLES ON 18" ~~CENTERS~~ LOADING EVERY OTHER HOLE.

DEPTH ON FRONT FACE HOLES WILL VARY WILL LOAD TO WITHIN 2 1/2 FT OF COLLAR WITH 1/2 STICK 1 1/4" 210 TAPED 1 FT ABOVE SHALE CONTACT LINE. STEMMED 1 FT WITH 1 1/2 FT AIR CUSHION. ON HOLES LESS THAN 10 FT USE JUST T-1. ON HOLES LESS THAN 5 FT USE 1 FT T-1.

NOTE: SHALE CONTACT LINE WILL BE DETERMINE UPON DRILLING PRESLIT

Acceptable this date 7/18/78
RJA

PLATE No 12



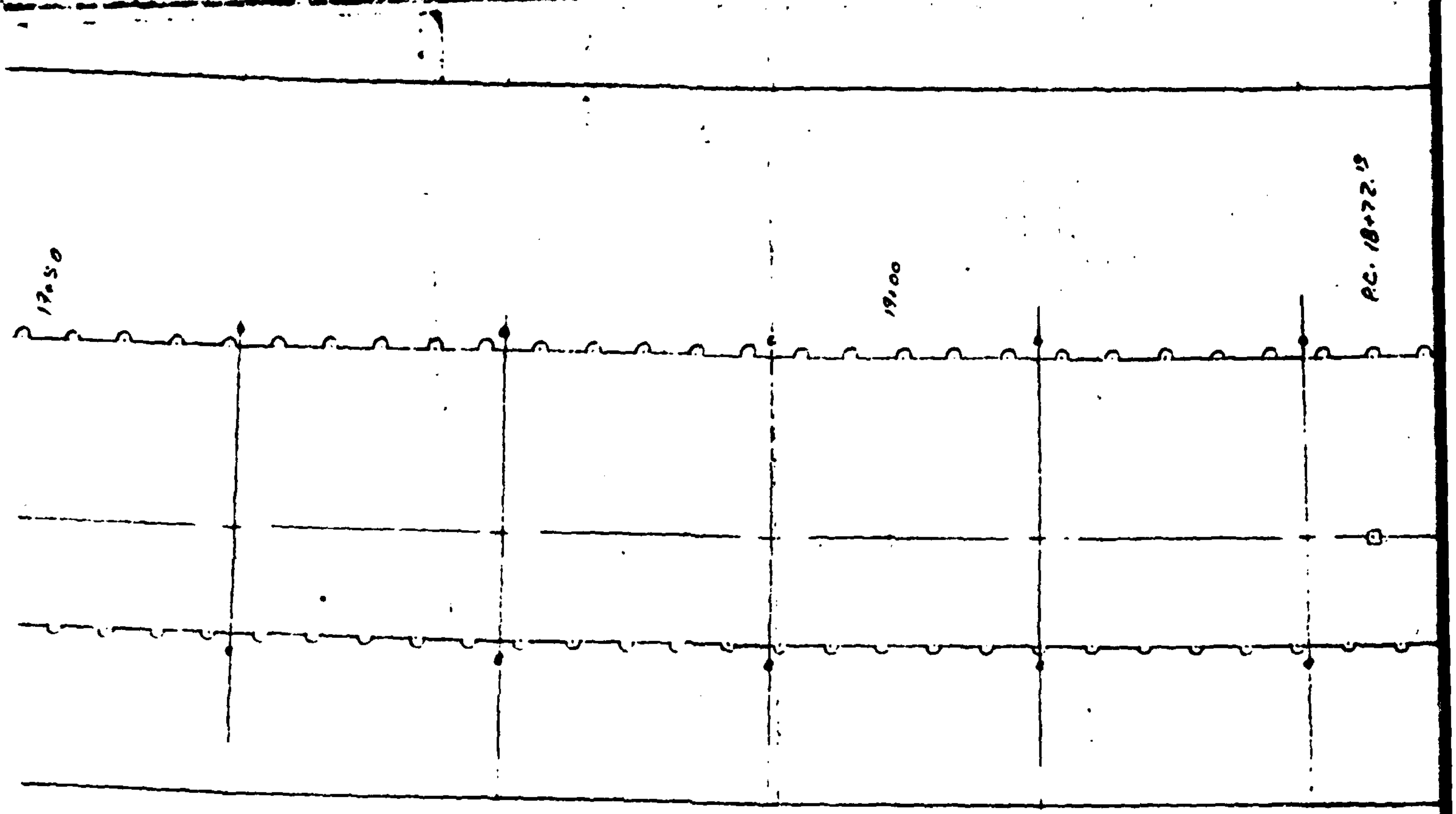
Production Shot No. 1

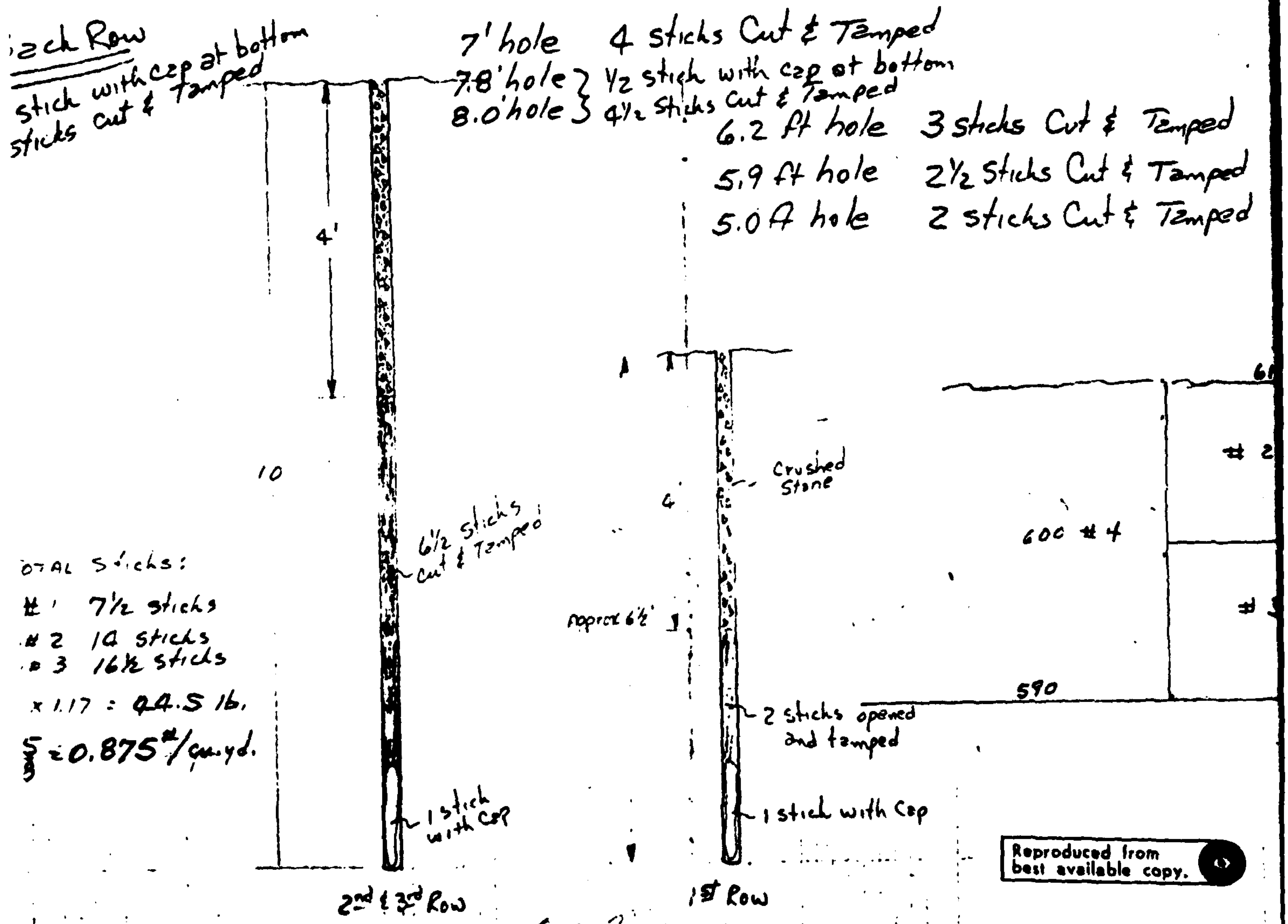
How Drilled	:	3700 Air Tracks
Hole Depth	:	6.5 to 10' Approx.
Hole Diameter	:	2-Inch
Hole Spacing	:	5 ft. × 5 ft
Explosive	:	Tovex 210 - 1½" × 16"
Density 210	:	1.15 = 1.17 lb./16" Stick
Stemming	:	Crushed Stone
Firing Data	:	As Shown
Total Burden	:	15.5 Ft.
Total Spacing	:	16.0 Ft.
Total Depth	:	See Sketch
Total Cubic Yards	:	~~Approx 66 cu.yds~~ ≈ Approx 50.8
Allow. Load Factor	:	0.95 lb/cu.yd.
Allow. Explosive	:	~~62.0 lbs.~~ 48.26 lbs.
Number of Holes	:	9 Holes
Explosive Per Hole	:	See Sketch
No. 16" Sticks/Hole	:	See Sketch
Actual Load Factor	:	0.875#/cu.yd.

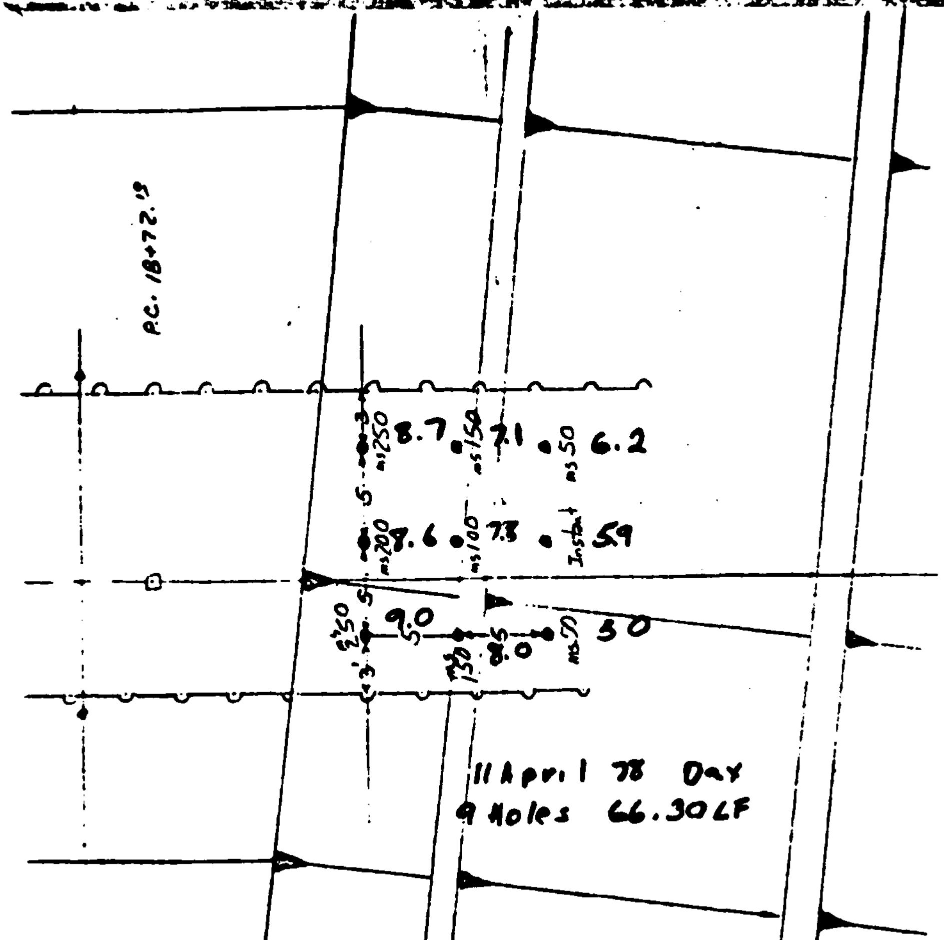
Back Row
½ stick with cap
5 sticks cut & ta

TOTAL sticks:
Row #1 7½ stick
Row #2 14 stick
Row #3 16½ stic
38 × 1.17 = 44.5
$\frac{44.5}{50.8}$ = 0.875#/c

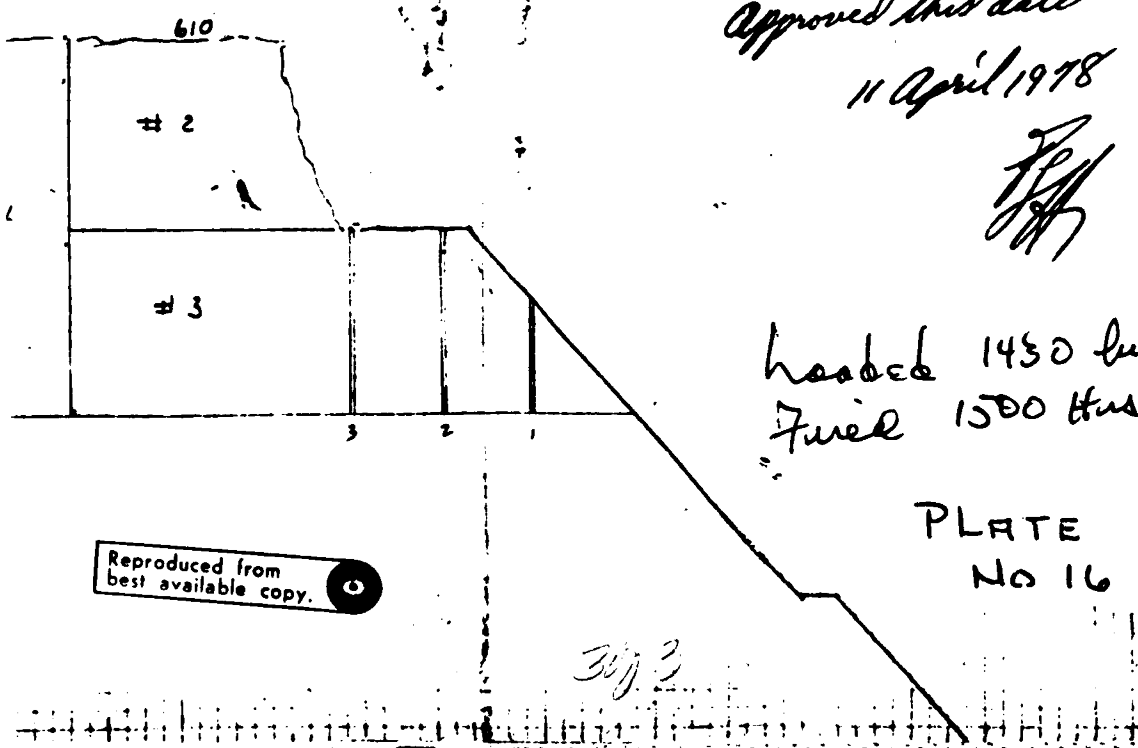
Submitted By	:	Luhr Bros., Inc.
Date Submitted	:	6 April 1978 9:40 a.m.
Date Approved	:	11 April 1978 8:00
Date Drilled	:	11 April 1978 11:18
Date	:	


17+50
19+00
P.C. 18+72.9


...ach Row
...stick with cap at bottom
...sticks cut & tamped
7' hole 4 sticks Cut & Tamped
7.8' hole
8.0' hole
1/2 stick with cap at bottom
4 1/2 sticks cut & tamped
6.2 ft hole 3 sticks Cut & Tamped
5.9 ft hole 2 1/2 Sticks Cut & Tamped
5.0 ft hole 2 sticks Cut & Tamped
4'
10
6 1/2 sticks Cut & Tamped
1 stick with cap
2nd & 3rd Row
4
Crushed Stone
approx 6 1/2'
2 sticks opened and tamped
1 stick with cap
1st Row
...otal Sticks:
#1 7 1/2 sticks
#2 14 sticks
#3 16 1/2 sticks
x 1.17 = 44.5 lb.
= 0.875 #/cu.yd.
600 #4
590
#2
#3



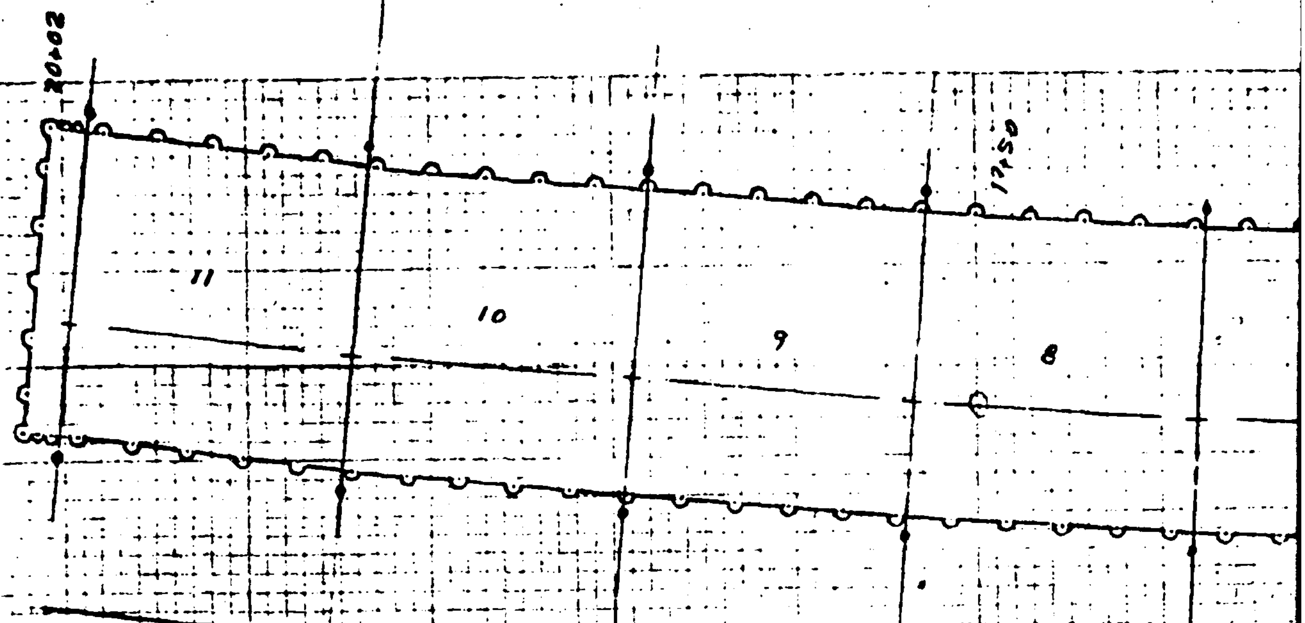
: Tamped
& Tamped
& Tamped



Approved this date
11 April 1978

Loaded 1430 hrs 11 April 78
Fired 1500 Hrs 11 April 78

PLATE
No 16

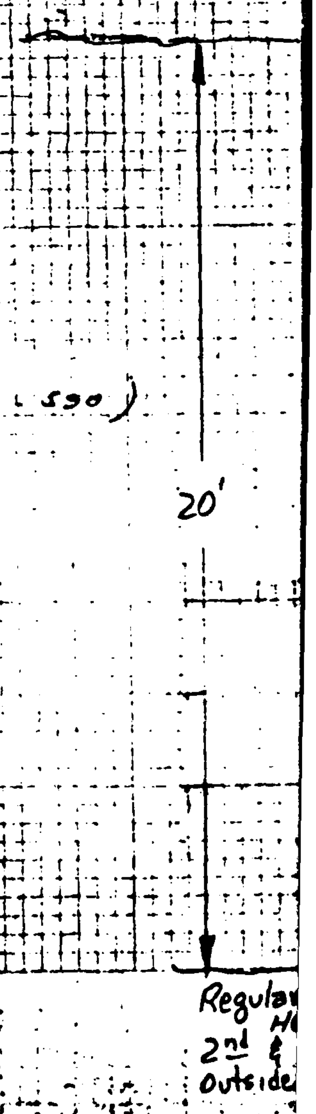


Production Shot No. 4

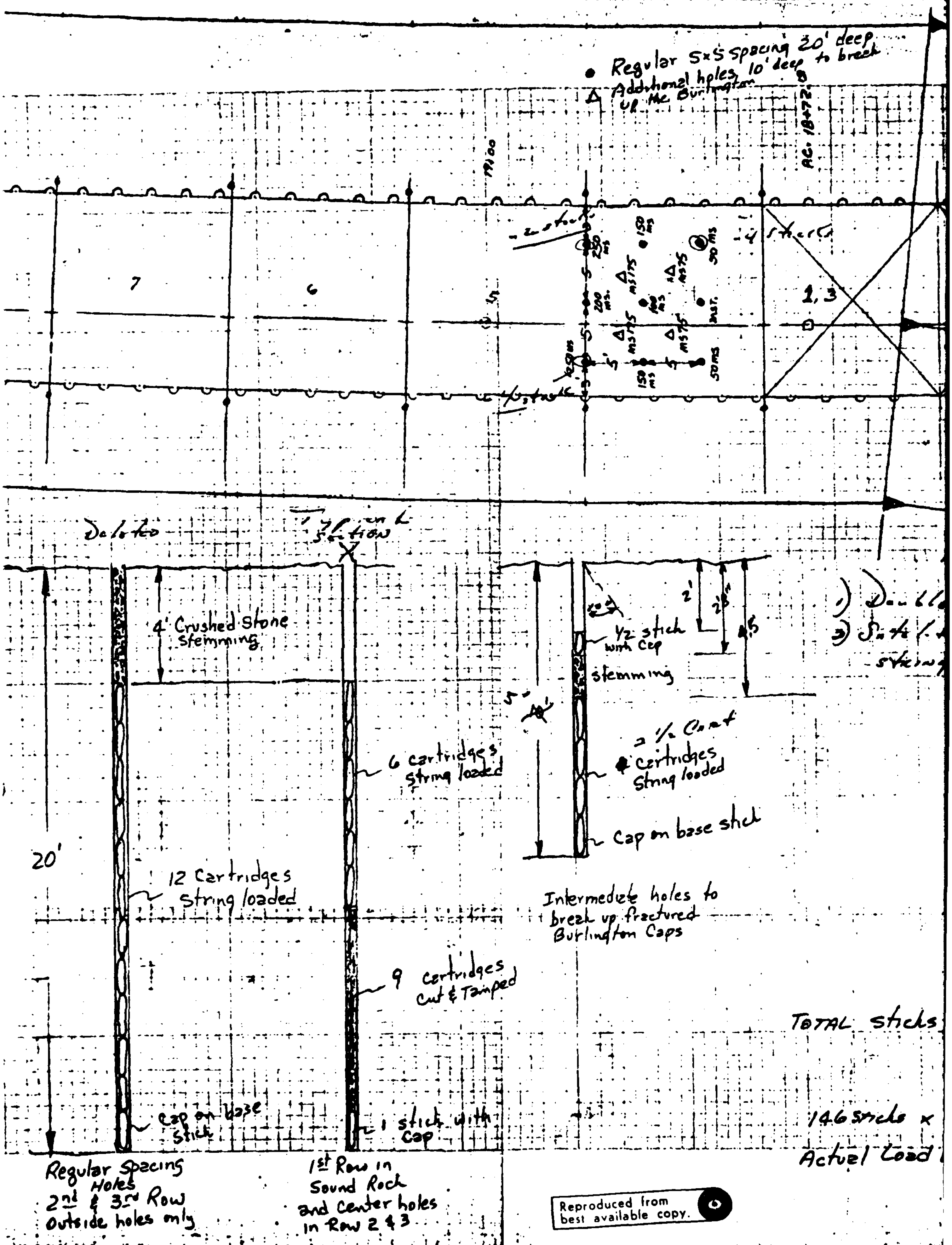
How Drilled : 3700 Air Tracks
Hole Depth : 20 Feet (All Holes Vertical)
Hole Diameter : 2-Inch
Hole Spacing : 5 ft. x 5 ft.
Explosive : Tovex 210 - 1½" x 16"
Density 210 : 1.15 ± 1.17 lb/16" Stick
Stemming : Crushed Stone
Firing Data : As Shown

Total Burden : 15.5 Ft.
Total Spacing : 16.0 Ft.
Total Depth : ~~[illegible]~~ 19.6 ft Avg. (the elev 610 - 590)
Total Cubic Yards : ~~[illegible] Cu.Yds.~~ 180.03 yds.
Allow. Load Factor : 0.95 lb/cu.yd.
Allow. Explosive : ~~[illegible]~~ 171.03 lbs
Number of Holes : 9 Holes
Explosive Per Hole : See Sketch
No. 16" Sticks/Hole : See Sketch
Actual Load Factor : 0.949 lb/cu.yd.

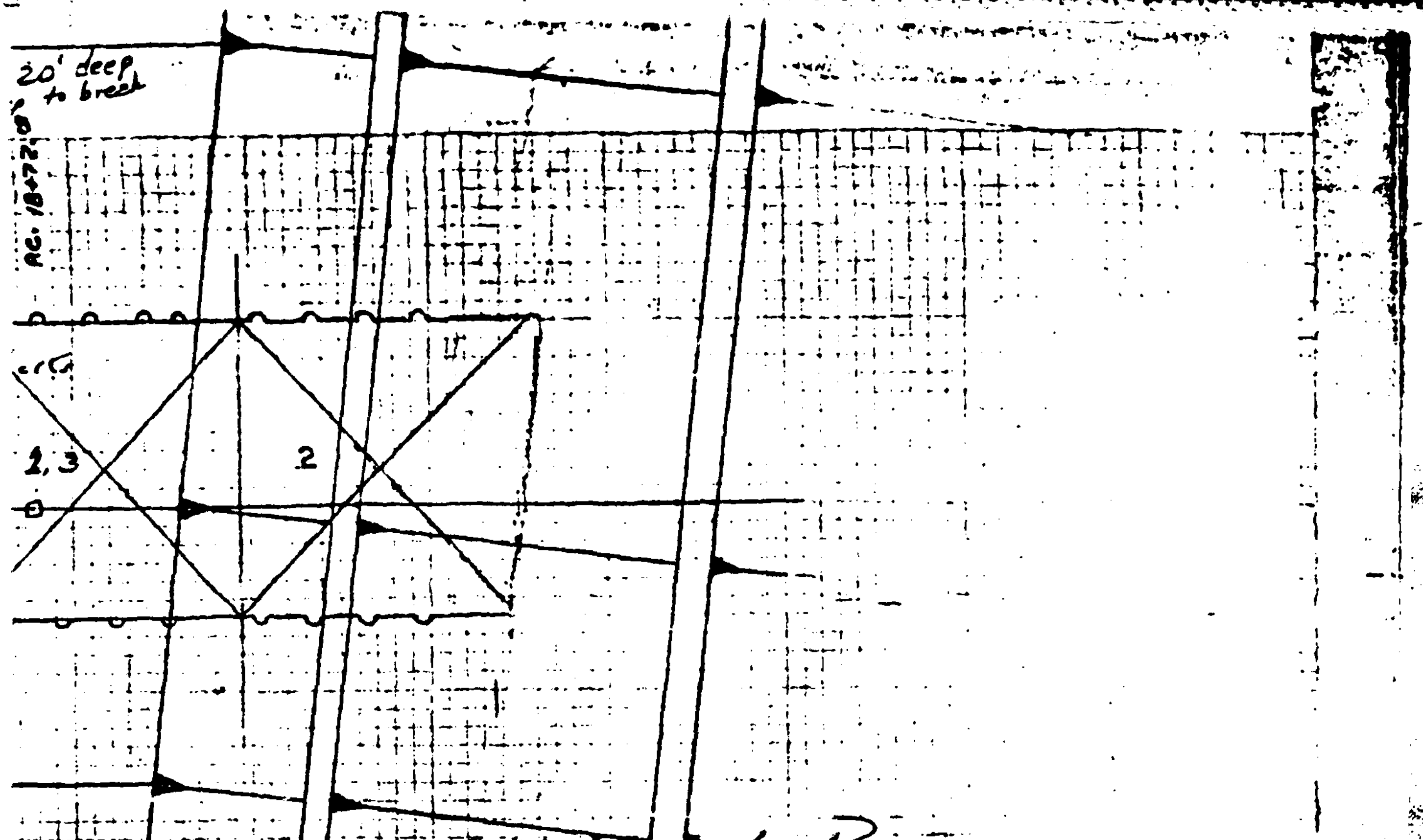
Submitted By : Luke Bros. Inc.
Date Submitted :
Date Approved :
Date Drilled :
Date Loaded :



...E INCH
...ER CO.



Regular 5x5 Spacing 20' deep
Additional holes 10' deep to break up the Burlington
7
6
5
1, 3
Deleted
Typical Section
4' Crushed Stone Stemming
6 cartridges String loaded
12 Cartridges String loaded
20'
9 cartridges Cut & Tamped
cap on base stick
1 stick with cap
Regular Spacing Holes 2nd & 3rd Row Outside holes only
1st Row in Sound Rock and Center holes in Row 2 & 3
2'
½ stick with cap
stemming
5'
4 cartridges String loaded
Cap on base stick
Intermediate holes to break up fractured Burlington Caps
TOTAL sticks
146 sticks x
Actual Load



Approved, Subject to Below Revisions

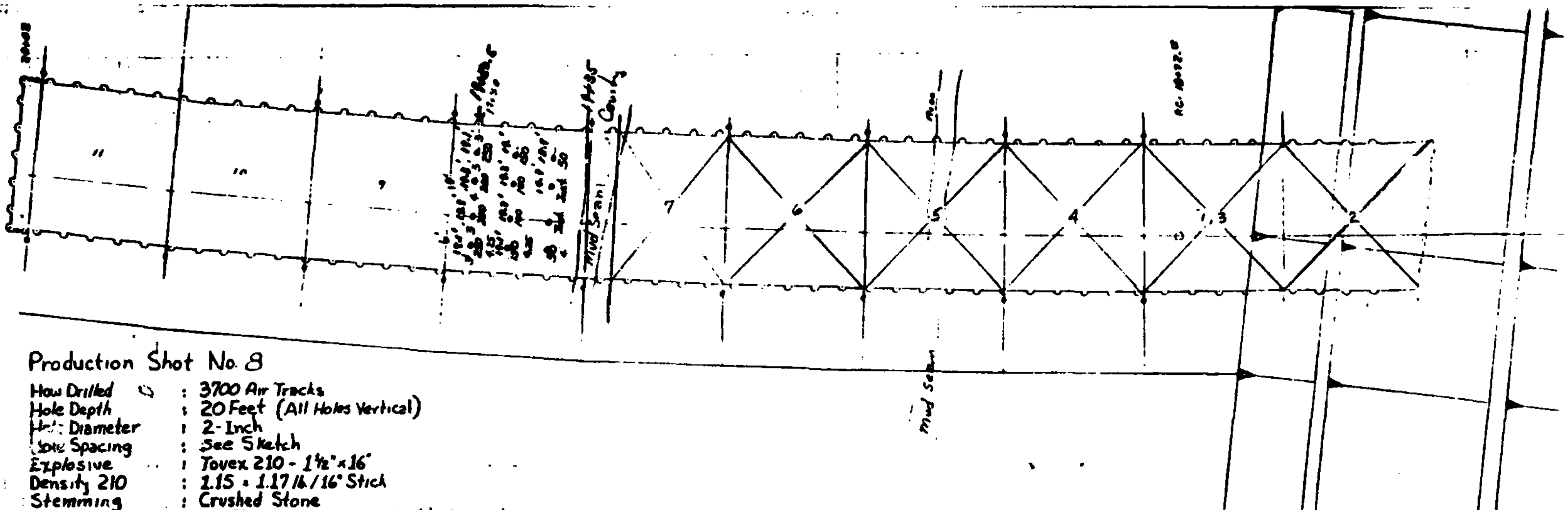
1) Double Cy Boring
2) Satellite holes will be 5' @ 10' w/ 2½ cartridges string loaded

TOTAL sticks 12 × 4 = 48
16 × 5 = 80
4.5 × 4 = 18
146

146 sticks × 1.17#/stick = 170.82

Actual Load Factor = 0.949

PLATE
No II

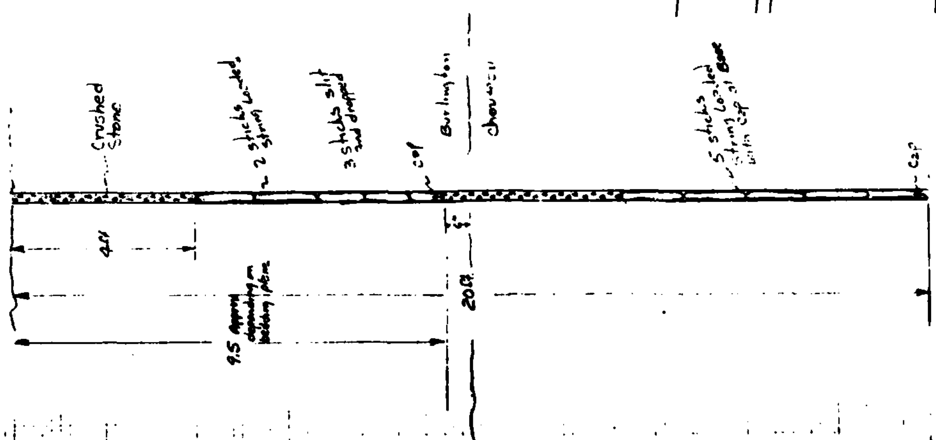


Production Shot No. 8

How Drilled	: 3700 Air Tracks
Hole Depth	: 20 Feet (All Holes Vertical)
Hole Diameter	: 2-Inch
Hole Spacing	: See Sketch
Explosive	: Tovex 210 - 1½" x 16"
Density 210	: 1.15 + 1.17 lb/16" Stick
Stemming	: Crushed Stone
Firing Data	: As Shown ; All Holes Double Primed

Total Burden	: 12.5 Ft.
Total Spacing	: 16.0 Ft.
Total Depth	: 20.0 Ft.
Total Cubic Yards	: 148.15 Cu.Yds
Allow. Load Factor	: 0.95 lb/cu.yd.
Allow. Explosive	: 140.74 lbs
Number of Holes	: 12 Holes
Explosive Per Hole	: lbs/Hole
No. 16" Sticks/Hole	: 10.0 Sticks
Actual Load Factor	: 0.948 lb/cu.yd.

Submitted By	: Luhr Bros., Inc.
Date Submitted	: 18 April 1978 at 0800 hr
Date Approved	: 18 April 1978 at 0900 hr
Date Drilled	:
Date Loaded	:

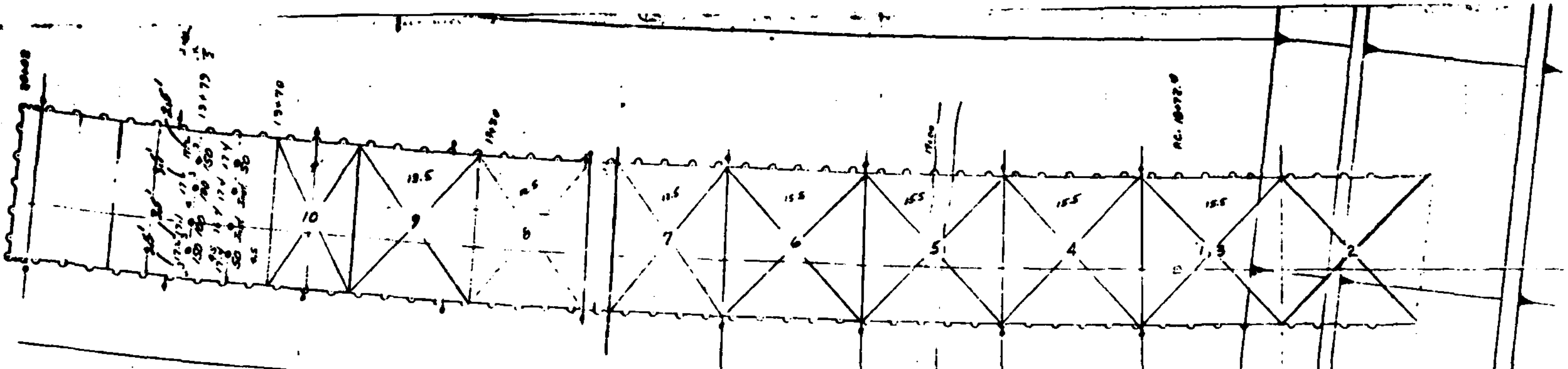


Drilled, loaded 47 ... 18 April 78
Fired at 1640
All holes loaded

Approved as shown
18 April 1978

PLATE
No 18

[redacted]
best available copy.



How Drilled	: 3700 Air Tracks
Hole Depth	: 20 Feet (All Holes Vertical)
Hole Diameter	: 2-Inch
Hole Spacing	: See Sketch
Explosive	: Tovex 210 - 1½" x 16"
Density 210	: 1.15 = 1.163 lb/16" Stick [illegible] = 1.16277#/stick
Stemming	: Crushed Stone
Firing Data	: As Shown ; All Holes Double Primed

Total Burden	: 9.0 Ft.
Total Spacing	: 16.0 Ft.
Total Depth	: 20.0 Ft.
Total Cubic Yards	: 106.67 Cu. Yds
Allow. Load Factor	: 0.78 lb/cu.yd. (Changed by Hood [illegible])
Allow. Explosive	: 83.20 lbs
Number of Holes	: 8 Holes
No. 16" Sticks/Hole	: 9.0 Sticks/Hole
Explosive Per Hole	: 10.46#/Hole
Actual Load Factor	: 0.785 lb/cu.yd.

Changed again from 0.87 to 0.78 on 19 April

ELIMINATED 3rd Row 20 April 78

Submitted By	: Luhr Bros., Inc.
Date Submitted	: 21 April 1978 1000 hrs
Date Approved	: 21 April 1978 1030 hrs
Date Drilled	:
Date Loaded	:

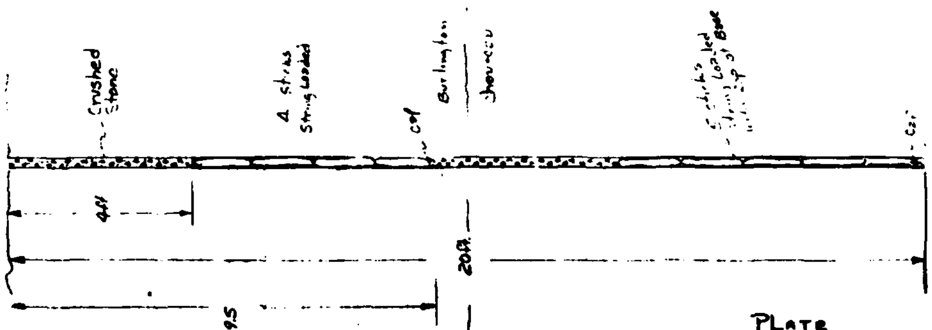


PLATE No 19

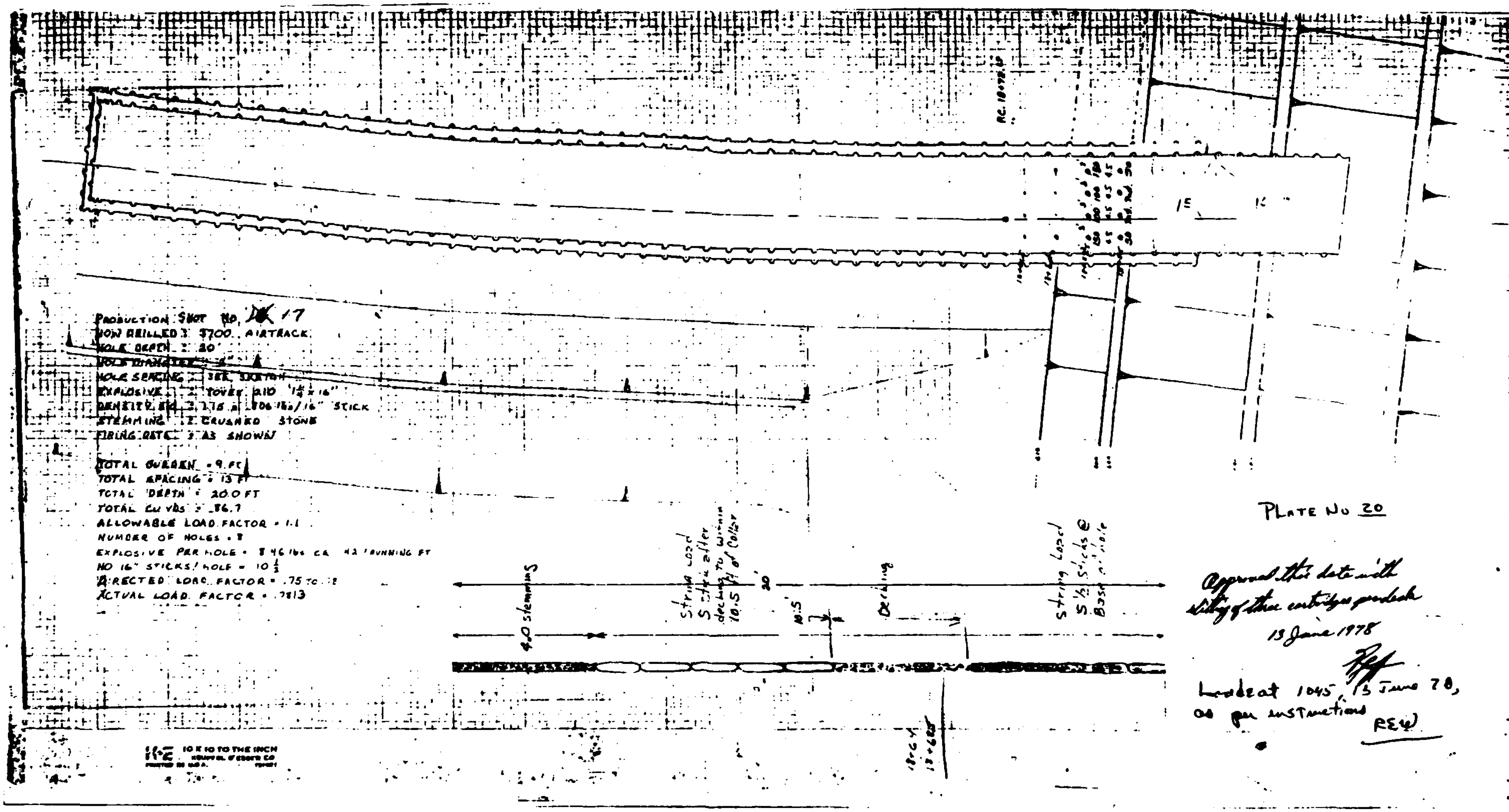
4/21/78 16 00

Note: [illegible] with 10 sticks [illegible]

[illegible]

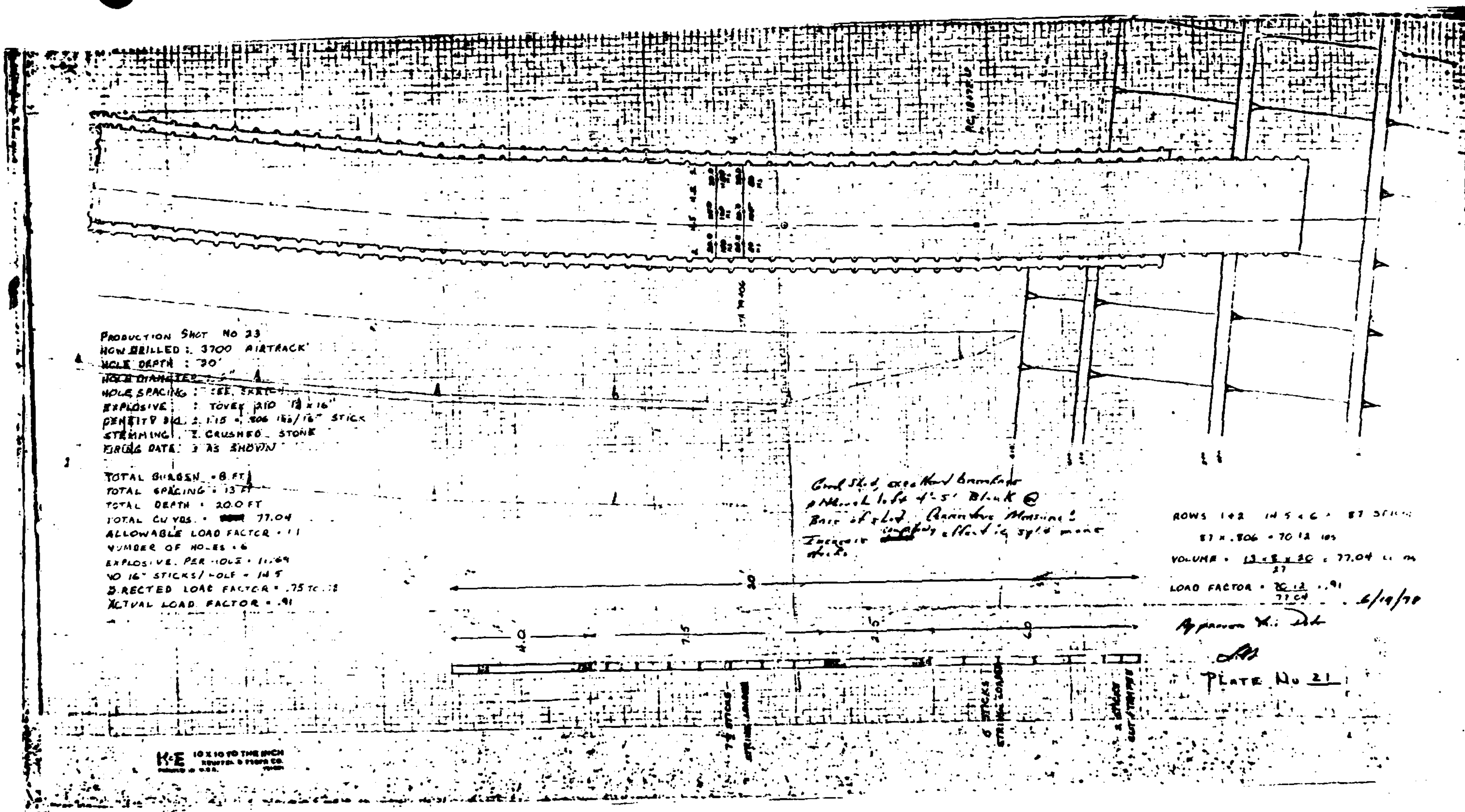
Bottom 2 sticks in outside rows to be split one side.

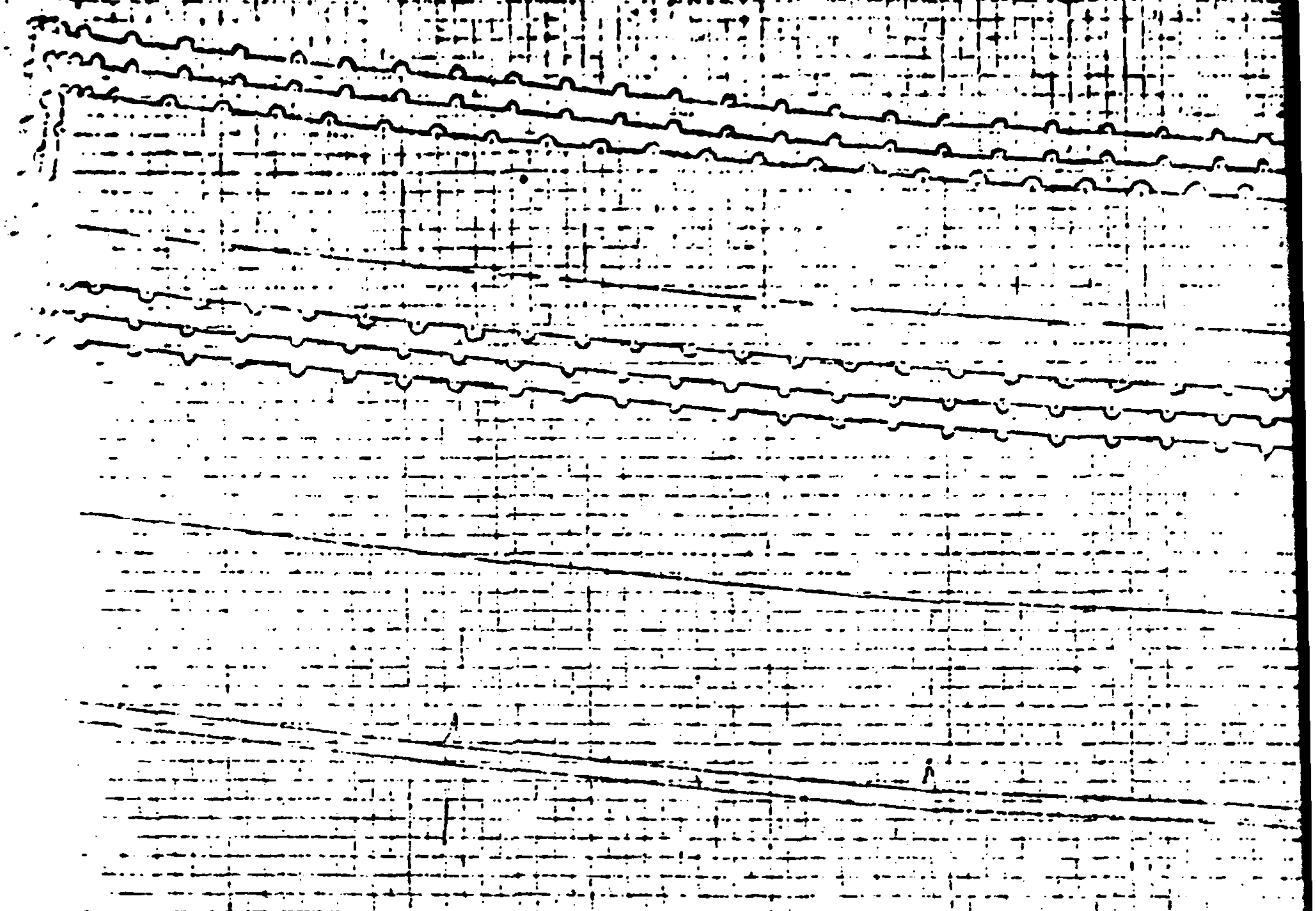
PLATE No 20



Approval this date with the understanding that cartridges...
13 June 1978

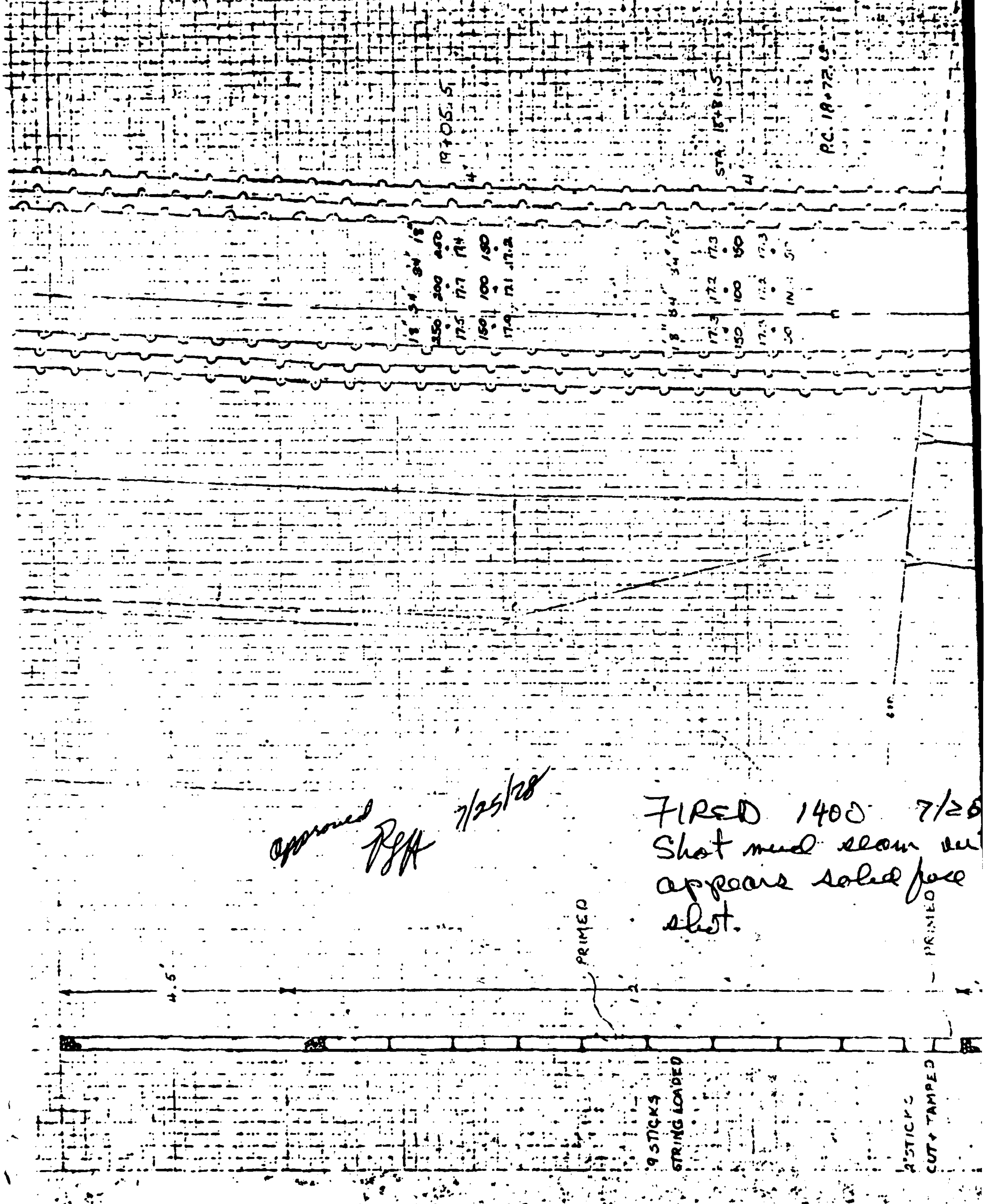
Loaded at 1045, 13 June 78, as per instructions
REW


PRODUCTION SHOT NO 23
HOW DRILLED : 3700 AIRTRACK
HOLE DEPTH : 20'
HOLE SPACING : SEE SKETCH
EXPLOSIVE : TOVEX 210 1½ x 16"
DENSITY : 1.15 = .806 lbs/16" STICK
STEMMING : CRUSHED STONE
FIRING DATE : AS SHOWN
TOTAL BURDEN = 8 FT
TOTAL SPACING = 13 FT
TOTAL DEPTH = 20.0 FT
TOTAL CU YDS. = [redacted] 77.04
ALLOWABLE LOAD FACTOR = 1
NUMBER OF HOLES = 6
EXPLOSIVE PER HOLE = 11.69
NO 16" STICKS/HOLE = 14.5
DIRECTED LOAD FACTOR = .75 TO .8
ACTUAL LOAD FACTOR = .91
Good Shot, excellent breakage
ROWS 1+2 14.5 x 6 = 87 STICKS
87 x .806 = 70.12 lbs
VOLUME = 13 x 8 x 20 / 27 = 77.04 cu yds
LOAD FACTOR = 70.12 / 77.04 = .91
6/19/78
PLATE No 21
6 STICKS
K&E 10 X 10 TO THE INCH



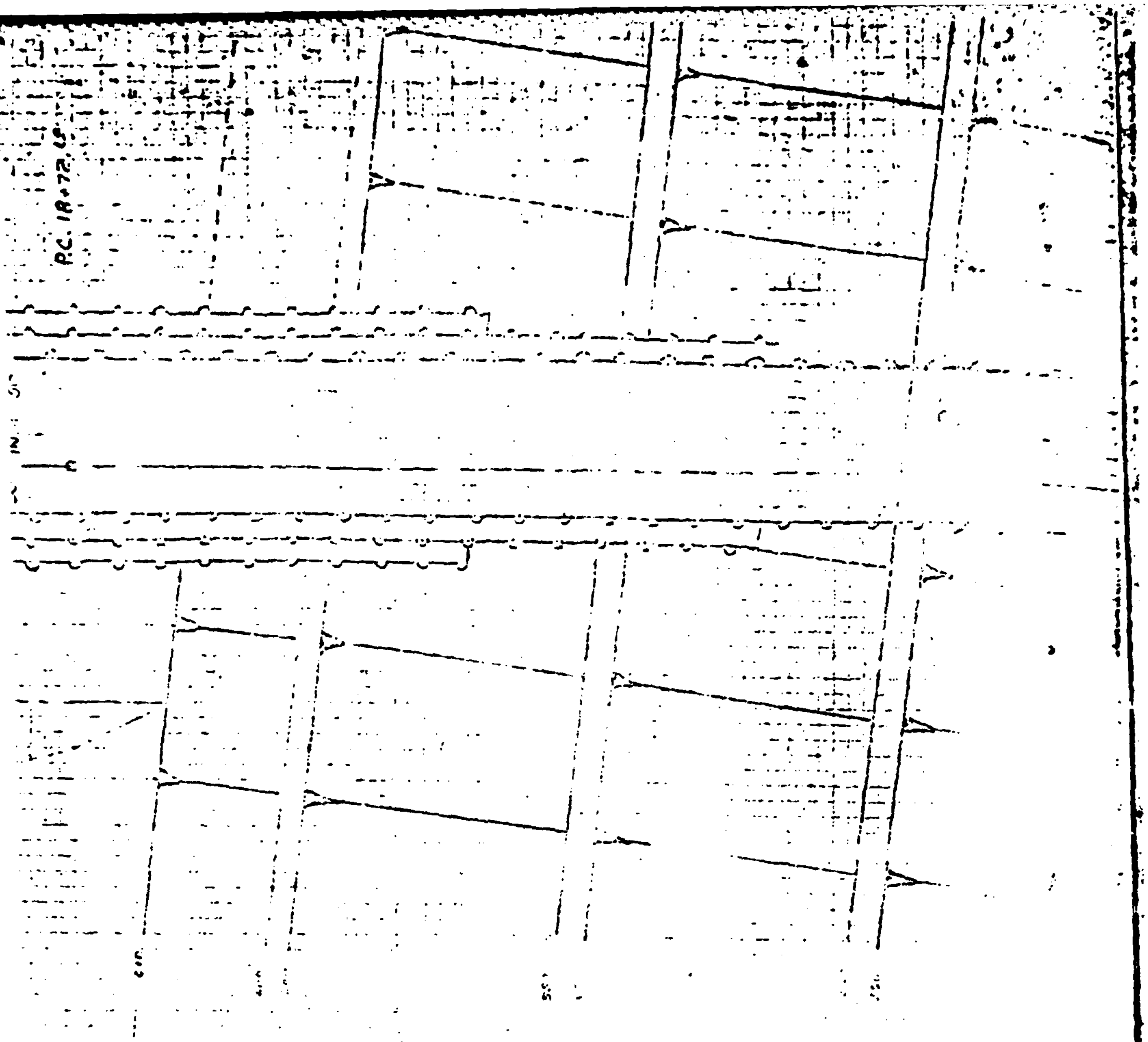
PRODUCTION SHOT NO. 45
HOLE DRILLED : 3700 AIRTRACK
HOLE DEPTH : 17.5
HOLE DIAMETER : 2"
HOLE SPACING : SEE SKETCH
EXPLOSIVE : 1¼" x 16" TOVEX 220
DENSITY : 1.20 .769 lbs/STICK
FIRING DATE : AS SHOWN

BURDEN = 8 FT
SPACING = 10 FT
DEPTH = 17.5
CU YDS = 51.85
ALLOWABLE LOAD FACTOR = 1.1
NUMBER OF HOLES = 6
EXPLOSIVE PER HOLE = 8.459 lbs
NO 16" STICKS/HOLE = 11
DIRECTED LOAD FACTOR = 1.00 TO .85
ACTUAL LOAD FACTOR = .978

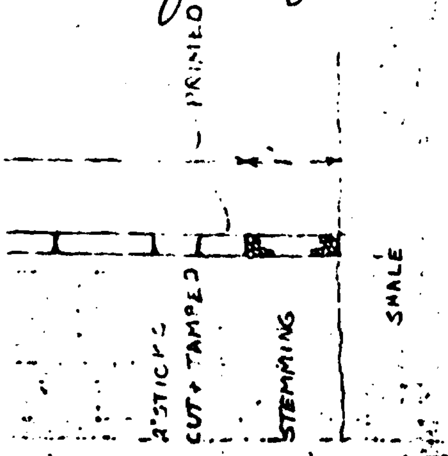


approved RJH 7/25/78

FIRED 1400 7/26
Shot mud seam out
appears solid face
shot.



'400 7/25/76
2 seam cut -
solid face for next



CU YDS

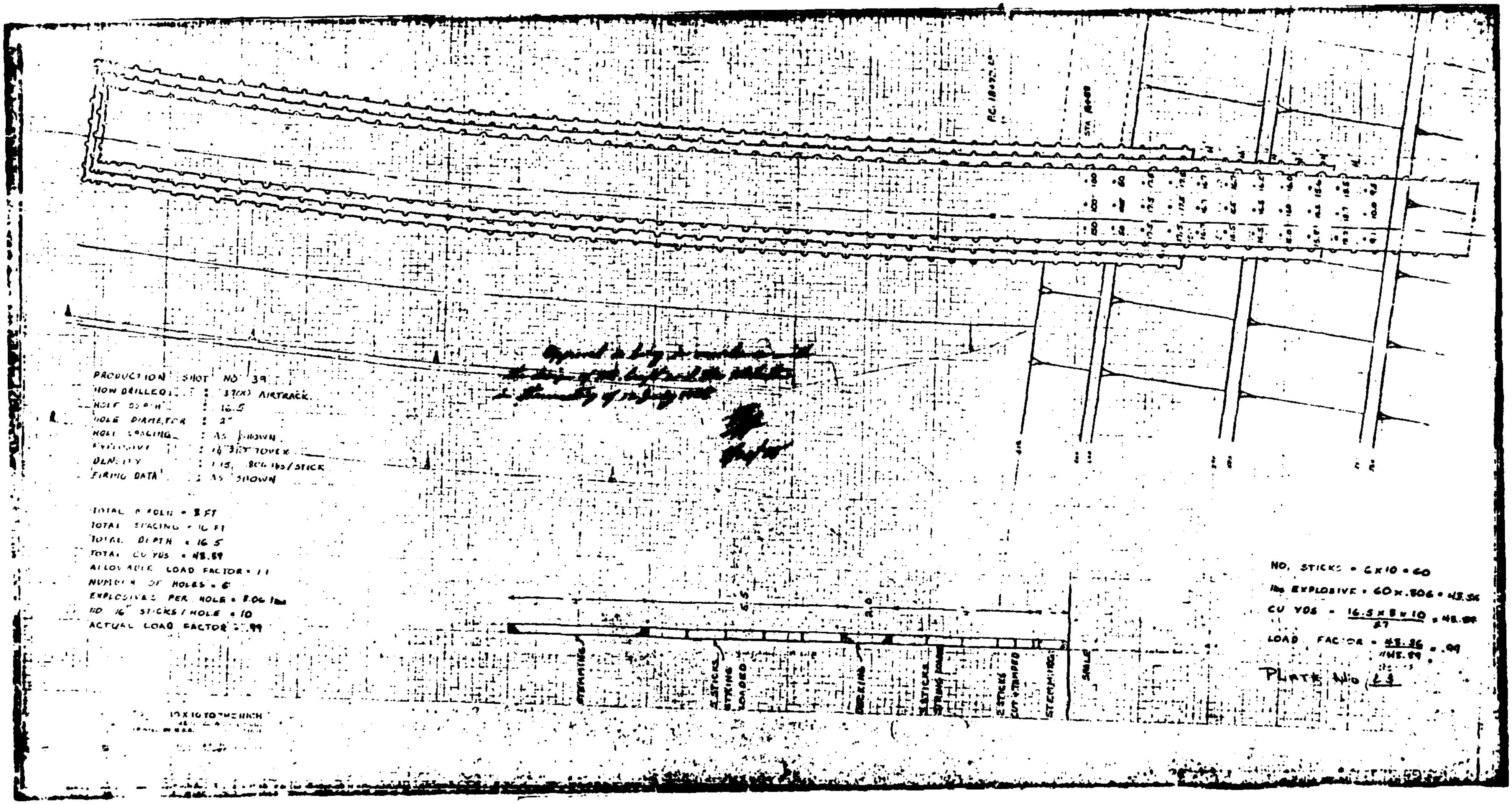
$\frac{17.5 \times 8 \times 10}{27} = 51.85$

NO STICKS = 6 x 11 = 66

lbs EXPLOSIVE = 66 x .769 = 50.5

LOAD FACTOR = 0.978

PLATE No 22


PRODUCTION SHOT NO 39
HOW DRILLED : 3700 AIRTRACK.
HOLE DEPTH : 16.5
HOLE DIAMETER : 2"
HOLE SPACING : AS SHOWN
FIRING DATA : AS SHOWN
TOTAL SPACING = 10 FT
TOTAL DEPTH = 16.5
ALLOWABLE LOAD FACTOR = 1.1
NUMBER OF HOLES = 6
NO 16" STICKS / HOLE = 10
ACTUAL LOAD FACTOR = .99
NO. STICKS = 6 x 10 = 60
CU YDS
LOAD FACTOR
STEMMING
STICKS
STRING LOADED
PLATE No

[b] Excavation and Backfill of Cutoff Wall and Cavities

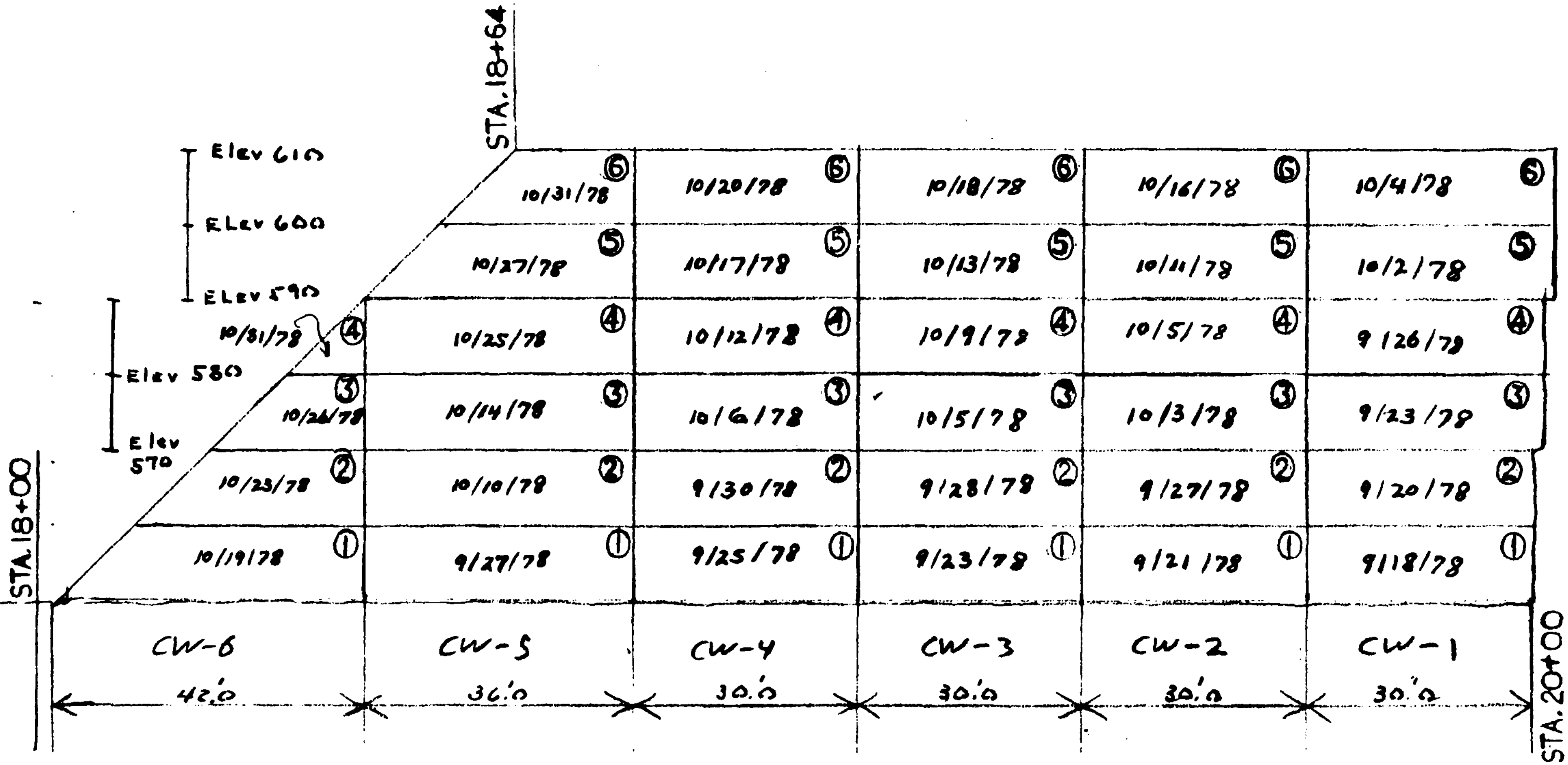
Concrete backfill for the left abutment cutoff wall and Cavities Nos. 1, 2 and 6 began on 18 September 1978 and was completed on 31 October 1978 by Massman Construction Co. The Contractor generally utilized single 10-hour shifts for forming and concrete placements. Principal equipment used for this modification work was:

Two (2) C.C.C. Trucks (Crane Carrier Corporation)
One (1) 9260 American Crane
Concrete Buckets (1, 2 and 4 cubic yard buckets)
Massman Construction Co.'s Batch Plant and Support Equipment (conveyor)
One (1) Hydrobroom
One (1) 60T Northwest Crane
One (1) Gardner-Denver 600 cfm Air Compressor
Concrete Vibrators (4-inch and 6-inch)
One (1) Grove Crane

A total of 4,309 cubic yards of 6-inch nominal size aggregate concrete was placed in 10-foot lifts within the cutoff wall. Generally, concrete backfill (6-inch aggregate) of the above cavities preceded the adjacent cutoff wall placement by only a single 10-foot lift. A continuous 12-inch wide PVD waterstop located 2 feet downstream of the dam axis was placed in all vertical cutoff wall construction joints. No internal reinforcing was used for either cavity or cutoff wall backfill except for selected placement of vertical mats of No. 9 reinforcing steel at the cutoff wall-cavity (less than 10-foot wide) interface (refer Plate No. 24 which shows dates, designations and dimensions of each cutoff wall concrete placement). The locations of the vertical mats of reinforcing steel are shown in the cutoff wall photographs for concrete backfill (Photographs on Pages 229, 232, 250, 265 and 266). The range of dates of concrete backfill for each of the above cavities is outlined on Table No. 1.

[c] Treatment of Downstream Ravine

Extensive foundation treatment of the highly weathered limestone in the ravine located downstream of the future cutoff wall was done concurrently with the cutoff wall excavation due to the modification (Modification No. P00085) restriction on blasting adjacent to the future grout curtains (200-foot limitation) and the concrete backfill (50-foot limitation) of the cutoff wall. Previously, the work would have been performed as the embankment fill was being placed.



CUTOFF WALL
SEQUENCE & DESIGNATION
OF CONCRETE PLACEMENTS

LEGEND

CW-1 ① DENOTES CONCRETE POUR NUMBER

PLATE
No 24

CONCRETE BACKFILL FOR CAVITIES

Cavity Designation	Approximate Date of Cavity	Elevation of Cavity Pour
No. 1, Upstream	4 October 1978	570-580
	5 October 1978	580-590
	9 October 1978	590-600
	12 October 1978	600-610
	20 October 1978	610-620
	24 October 1978	620-630
	25 October 1978	630-635
No. 1, Downstream	9 October 1978	590-600
	11 October 1978	600-610
	20 October 1978	610-620
No. 2, Downstream	28 September 1978	590-600
	29 September 1978	600-610
	30 September 1978	610-620
	5 October 1978	620-630
	9 October 1978	630-635
No. 6, Upstream	13-14 October 1978	590-600
	17 October 1978	610-613
	30 October 1978	613-618
	4 November 1978	618-620

TABLE No. 1

Based on the embankment design, the ravine foundation surface would be covered by a sand drainage blanket. The contract specifications for the placement of the embankment in the contact area dictated that all large rock overhangs and protrusions be eliminated by blasting (presplitting or line drilling) or by filleting so that the resulting surfaces would be suitable for compaction of embankment material and that no remaining vertical surfaces would be more than 5-feet in height. Since this area was characterized by numerous ledges and overhangs, preparation of the slopes by blasting was necessary (refer photographs on Page 285 for generalized field condition prior to excavation).

The sequence, degree and limits of ravine preparation were discussed and finalized during the Geotechnical Conference on 8 and 9 March 1978. The foundation treatment consisted of the following.

1. The four ledges shown in photographs on Page 285 were eliminated by blasting. The rock was removed on a 1V:1H presplit plane from the toe of the ledge to daylight on the upper horizontal surface. A smooth presplit face was not required since the face would be covered by pervious material.

2. Dental excavation and backfill of the weathered joints and bedding planes exposed by the removal of the ledges were accomplished immediately after presplitting. Filled joints and bedding planes were excavated as deeply as possible, washed and backfilled with mortar or lean concrete. The purpose being to stabilize any remaining natural fill to prevent further weathering of exposed surfaces before fill placement.

3. Preliminary dental backfill in the downstream ravine was exempt from blasting proximity restrictions on the modification specifications. Damaged or loosened dental backfill from blasting was replaced or repaired during final foundation cleanup and preparation directly preceding the fill.

4. The limits of treatment downstream extended from the dam axis to the toe of the embankment. The area upstream of the dam axis was treated as the embankment was placed.

The ravine work was conducted by Luhr Bros., Inc., on a two-shift basis starting on 14 March 1978 and was essentially completed on 22 June 1978. The work involved presplitting of four ledges, scaling, dental excavation of the exposed clay seams on the presplit face, washing, tuckpointing with mortar grout and the placement of dental concrete backfill (refer photographs on Pages 287 thru 290 for typical treatment surfaces). The locations of the four ledges are shown on Drawing No. 128/2. Upon completion of the ravine work, a portion of the Contractor's labor force continued to perform dental excavation of the clay-filled joints along the entire embankment contact area until mid-August 1978. In addition, Cavity No. 3 was cleaned out to the elevation shown on Drawings Nos. 126/2 and 131/2 in order for Massman Construction Co. to be able to form and place the concrete backfill in conjunction with their fillet work. During this period, geological mapping along the abutment face was performed. The results of the field mapping are shown on Drawings Nos. 126/2 through 129/2.

[d] Placement of Concrete Fillets Within the Embankment Contact Area

In addition to the discovery of extensive solution chimneys within the left abutment cutoff trench and embankment contact area, numerous clay-filled and open major joint systems oriented east to west along the entire abutment face were exposed during the left abutment excavation in 1977. The development and concentration of these joints were especially predominate in the lower portion of the Chouteau Formation. Due to the orientation and nature of the major and minor joint systems (refer to the earlier SLD Report entitled "Report on Positive Cutoff Treatment Cannon Dam, Left Abutment") and the inherent fracturing from the blasting operation, large detached limestone blocks at the Chouteau/Hannibal Shale contact were removed by a CAT 245 backhoe during the 1978 construction season. Their removal generated vertical rock faces well in excess of the 5-foot contract specification limitation.

In order to obtain an overall 1V:1H rock slope for embankment placement, the concrete fillets were placed according to the guidelines presented in the SLD/LMVD Field Conference on 1 June 1978. The guidelines are as follows:

1. 1V:1H Slope Downstream of Pseudo-Core (area of final foundation treatment).

a. At the limestone-shale contact there will be a concrete fillet from 5-foot to 10-foot high.

b. The concrete fillet will be vertical and not on a batter.

c. The fillet should be approximately one-half the height of the vertical rock face.

d. There will be no length restrictions on fillets.

e. Above the limestone-shale contact there will be minor fillets (5 feet or less) and dental work.

2. Pseudo-Core (area of final foundation treatment)

a. At the limestone-shale contact, fillets will be a maximum of 5 feet in height.

b. The fillets will be vertical and not on a batter.

c. The inside corners will have a 1-foot subfillet on a batter--hand formed--no bond necessary.

d. There will be no length restrictions on these fillets.

e. Intersection of 5-foot fillets or vertical rock faces with the formed 1V:1H cutoff wall will have battered fillets in these corners.

3. 1V:1H Slope Upstream of Pseudo-Core (area of final foundation treatment)

a. At the limestone-shale contact, there will be a concrete fill 5-foot to 7-foot high.

b. Concrete fillets will be vertical and not on a batter.

c. The inside corners will have a 1-foot subfillet on a batter--hand formed--no bond necessary.

d. There will be no length restrictions on these fillets.

The above fillet guidelines were generally followed for the left abutment with the only change being the concrete fillet batter requirements. On 6 August 1980 during a site inspection, OCE/LMVD personnel requested that all future fillets for the upstream embankment face be placed with a batter ranging from 1/2V:1H to 1V:1H. The basal concrete fillets were formed and placed from late July 1978 until 20 September 1978. Generally, 1-inch or 1 and 1/2-inch top size aggregate concrete was used for protective slabs and 6-inch top size aggregate concrete was used for the basal fillets.

The scope of work in Modification No. P00085 dealing with the additional drilling and grouting was eliminated due to difficulties in negotiation with the drilling subcontractor. The Contracting Officer decided to seek competitive bids for the additional foundation drilling and grouting work in order to obtain a fair price and to expedite settlement of Modification No. P00085. The work was awarded by competitive bid to Boyles Bros. Drilling Company of Woods Cross, Utah, in the fall of 1979. The detail of the program will be discussed in Section 12.

The narratives dealing with foundation treatment for the basal and cutoff wall concrete fillets are found in Section 6, Foundation Treatment, Part B, Embankments.

Diversion channel excavation and temporary construction bridge as viewed from north abutment.

SECTION 6

FOUNDATION TREATMENT

A. Concrete Structure

1. Introduction

Foundation treatment for the concrete structure, powerhouse and downstream walls began in April 1974 and was completed by August 1975. Foundation treatment for the stilling basin and tailrace concrete slabs was completed on 1 June 1976 and 17 November 1977, respectively. Foundation treatment encompassed foundation preparation (removal of incompetent foundation rock), and dental treatment (joint preparation and backfill). Treatment was performed by Massman Construction Co. utilizing a 6-man to 8-man crew and, depending upon the time of year, an 8-hour to 10-hour shift. The Contractor utilized the following equipment for treatment and removal:

680C Case Backhoe
580B Case Backhoe
Gardner-Denver 900 Compressor
Gardner-Denver 600 Compressor
966C CAT End Loader
American 9260 Crane
Grove RT 605 Hydrocrane
Washington Revolving Crane
2.5-Ton GMC Service Truck
Terex End Dumps
Hydrobroom
Assorted small hand tools, including several air hammers, air chisels, rock hammers, blow pipes and spades.

This Report is subdivided into the following five sections: (1) Non-overflow Monoliths (D-1 thru D-6 and D-13 thru D-17); (2) Overflow Monoliths (D-7 thru D-12); (3) Powerhouse Foundation Sections; (4) Downstream Walls (Stilling Basin, Splitter and Tailrace) and (5) Stilling Basin and Tailrace.

All photographs pertinent to Section 6 are shown in Volume 3, Section 2, with all pertinent drawings for items referenced in Paragraph A are found in Volume 4 (Drawings Nos. 67/2 thru 86/2).

In order to gain an insight of the various factors affecting foundation treatment for the monoliths founded on the Louisiana Limestone, a "Memo to File" (edited) written by the Resident Geologist is presented as an Appendix at the end of this section.

As shown on the geologic cross sections (Volume 4 Drawing No. 56/2), treatment within the limits of the concrete structure encompassed the following four geologic formations: Burlington Limestone, Chouteau Limestone, Hannibal Shale and Louisiana Limestone.

Foundation preparation consisted of the removal of all weathered, fractures, drummy sounding rock and the backfill of overdrilled blast holes. Dental treatment consisted of the veeing and backfilling of all joints and blast fractures. Foundation treatment was performed under the direction of a Government geologist. When the Government geologist was satisfied with the degree and extent of treatment, the final surface was geologically mapped and photographed. The final limestone foundation surfaces were left open up to 15 days before shotcrete and/or concrete placement, whereas the final shale foundation was covered with a bituminous protective covering or concrete within one hour to prevent dessication. The specifications allowed up to 15 days for concrete or shotcrete placement on the final shale surface after bituminous application. The exception to this usage of bituminous covering occurred on the tailrace and exit channel pavement foundations. These two foundations were covered with 6 inches of sand by a Gradall within one hour of exposure.

2. Non-Overflow Monoliths (D-1 thru D-6 and D-13 thru D-17)

(a) Monolith D-1/D-2

Foundation treatment began in November 1972 with the removal (tractor-mounted hydraulic ram) of a large slab of rock which had been displaced laterally by blasting operations. In addition, considerable rock removal was required in the corners since the rock failed to "pull" during blasting operation. Removal of this rock resulted in a relatively flat foundation surface with minor ridges and domes, the whole of which dipped toward the northwest. The foundation floor in the northwestern portion of the monolith consisted of a triangular-shaped bench which was approximately 2 feet in elevation above the lowest portion of the foundation floor. The length of the north-south leg of this bench was approximately 30 feet and approximately 45 feet for the east-west leg with the hypotenuse being slightly concave toward the northwest corner. This bench was comprised of two sandy limestone beds of approximately equal thickness while the remaining portion of the foundation floor consisted of thinly bedded, relatively pure limestones and argillaceous limestones. In addition, the foundation contained a series of low mounds and shallow depressions, some of which were elongated east-west. Their height or depth varied between 0.1 foot and 0.3 foot with diameters or widths of approximately 5 feet, while the lengths of the elongated mounds were up to 15 feet. Other foundation features included the presence of asymetric ripple marks, crinoid stems and fossil casts.

Due to changes in the Contractor's plans, foundation treatment was discontinued until the second whirley crane was operational. Work resumed on 9 January 1975 and continued until 13 January 1975 when the work area was covered with plastic and then heated. During the period from 13 January to 18 January 1975, the work crews were shifted to foundation work in Monolith D-7. On 19 January 1975, the protective covering for Monoliths D-1/D-2 collapsed due to the heavy snowfall. Treatment resumed on 22 January 1975 with final approval of the foundation for concrete placement being given on the night shift of 29 January 1975.

During each period of foundation treatment, considerable work was required largely due to the fact that the area was subjected to freeze-thaw cycles on numerous occasions. In addition, because of the confined work area and the contours of the foundation, ponded water was a continual problem with complete unwatering occurring only for concrete placement.

The worse area of overdrilling during blasting was located about 15 feet west of the northeast corner. It is most probable that this was done during redrilling and shooting of this area after it failed to pull on the initial shots. Further, this was another area where water continually accumulated and froze, and where workmen could not see what they were doing through the water/ice.

(b) Monolith D-3/D-4

This foundation was initially exposed to El. 465± feet NGVD in the late fall of 1974. Some minor excavation was performed to bring the floor to grade and to remove the fractured rock resulting from blasting. After that time, the monolith foundation laid exposed until August 1975. Throughout this period, the foundation was almost continually covered with water and ice during the winter months.

When foundation treatment resumed on 9 August 1975, the entire foundation area was unsound to a depth of approximately 6 inches. Visual inspection revealed the absence of natural jointing; however, two major blast fractures were noted. The first fracture was located in the northeast corner while the second fracture was located in the southwest corner of the monolith; generally, these cracks corresponded to the shot pattern.

Upon removal of the 6 inches of incompetent rock, a very thinly bedded, shaly limestone was encountered which tended to loosen or "pop-up". Underlying this material was a thicker (1 foot±) sandy limestone which was more competent and remained sound. In many cases, the thin bedded material had to be removed to the thicker sandy unit. The monoliths were worked intermittently until 28 August 1975, when the foundation was approved. Concrete was placed on 29 August 1975.

(c) Monolith D-5/D-6

The foundation for these monoliths was worked intermittently during November 1974 but was left exposed until the first part of December 1974. In the second week of December 1974, final foundation cleanup began in earnest. Approval was given for concrete placement on 16 December 1974; however, this approval was by area since foundation treatment was being performed on the western portion.

Bedding within the foundation was almost horizontal with a very slight dip to the northwest. The eastern two-thirds of the foundation floor, from approximately 15 feet downstream to 113 feet downstream, were composed of a bedded limestone, while the western one-third was composed of a sandy limestone. Unlike the other monolithic foundations where the bedded limestone occurred, this area contained only a few problem areas within the limits of the mounds and depressions. Generally, the beds in this area were thicker than elsewhere and were on the order of 4 inches to 6 inches thick. The only reason the sandy limestone was exposed was due to over-excavation which exceeded 1 foot below grade, and even at this depth, the blast holes were still visible.

In addition to the overdrilled blast holes, there was a lack of proper drainage. Water entered the monoliths from the powerhouse areas from the El. 485 feet NGVD bench from precipitation, weeps and an exploratory hole in the northeastern corner. Consequently, this water collected in the low areas where it was exposed to numerous periods of freezing; thus, subjecting many different beds to a wedging action.

(d) Monolith D-13

Foundation treatment began in September 1974 as the monolithic walls were exposed. Preparation of the foundation floor began on 13 December 1974 and was completed by 17 December 1974. The shale foundation was a sound, competent rock with few joints. Due to the presence of large joints in Monolith D-14, the floor bolts were installed and stressed prior to removal of the last two feet of shale above the final foundation. Final approval was given by area; therefore, concrete was placed in one section of the monolith while foundation treatment was being performed in another section. The first placement of protective concrete was on 13 December 1974 with the first regular lift of concrete being placed on 7 January 1975.

(e) Monolith D-14

On 23 September 1974, the foundation floor of Monolith D-14 was exposed. Two sets of parallel joints trending from upstream to downstream, normal to dam axis, were noted with the northwest upstream portion of the foundation floor being highly fractured. The joints were such that they posed a possible threat to the integrity of the structure. Consequently, a stop order was issued by the Contracting Officer. The design elements decided to have the Contractor install and stress the floor bolts for the purpose of closing the joints prior to protective slab placement. This was done but the joints did not close. Considerable dental excavation was performed in the upstream fractured area of the foundation. All joints were veed out and subsequently backfilled with concrete. Six additional extensometers and two load cells were installed to measure possible movement. The extensometers were installed in the back wall of Monolith D-13 and the upstream and downstream walls of Monolith D-13/D-14, whereas the load cells were installed in the foundation floor. Foundation treatment was completed by 30 September 1974 with the 6-inch protective concrete slabs being placed from 2 October 1974 to 4 October 1974.

(f) Monolith D-15

The walls of Monolith D-15 extended through the Chouteau Formation with the floor being founded on the Hannibal Shale. Several vertical joints were encountered in the upper limestone walls. Numerous upstream/downstream trending joints occurred in the shale floor; however, none required special treatment. Foundation preparation began in May 1974 and continued intermittently as the walls and floor were exposed until the concrete protective slab was placed on 2 July 1974.

(g) Monolith D-16

The foundation floor for Monolith D-16 rests on the upper Chouteau Formation, whereas the monolithic walls extended into the Burlington Formation. Foundation treatment began on 2 April 1974 and was concluded by 19 April 1974. Dental excavation was performed on a vertical, clay-filled joint (up to 2 feet wide in the east corner), striking due east, and the two less pronounced joints branching from it. The joints were excavated for a distance of 1 foot on either side and for a depth of 1 foot. Mud was cleaned from the joints by hand approximately 6 inches below the joint excavation and then washed with high-pressure air and water. A considerable amount of rock was removed from the foundation floor which resulted in a 1-foot to 3-foot protective concrete slab being placed in order to bring the floor back to grade.

(h) Monolith D-17

This monolith is founded in the Upper Burlington Formation. The south end of the excavation extended into a sinkhole with the walls and floor being composed of fractured limestone and chert with residual Pennsylvanian clay fill. It was necessary to excavate a large amount of this material in order to ensure satisfactory conditions.

The monolith was brought to grade in September 1973 with no further work being performed until July 1975 when foundation preparation began. Due to the long period of exposure, it was necessary to remove several feet of rock from the south and north edges, and approximately 6 inches from the remainder of the foundation floor to reach sound rock. A small concrete lift was then placed to fill the depressions at the south end and two days later on 15 October 1975 the first concrete lift was placed.

3. Overflow Monoliths (D-7 thru D-12)

(a) Monolith D-7

Monolith D-7 was excavated to within 6 inches of final grade on 24 December 1974. No further work was performed until 15 January 1975 when the Contractor began foundation treatment. The foundation surface was approved on the night of 17 January 1975 with concrete being placed the following day.

The major foundation feature within Monolith D-7 was the amount of relief which was present after final cleanup. During preliminary treatment, a ledge was generated while working the foundation surface from the west-northwest to the south-southeast. The bedding dipped north-northeast, and thus, as the work proceeded in a southeasterly direction, a ledge of 1 foot to 2.5 feet high was generated with the greatest relief near the southeast corner of the draft tube keyway. The bedding sequence (ledge face) consisted of a cycle of thin limestone beds (each bed 2 inches to 3 inches) overlain by beds of sandy limestones (approximately 1-foot thick). The lower sequence of thin beds comprised the western half of the final monolithic foundation, the surface of which was planar and dipped easterly. The first sandy limestone which formed the initial major ledge extended from the southern limits of the monolith (30± feet downstream) for a distance of 14 feet in a northerly direction and then turned back to the east. Once the ledge turned easterly, it increased in elevation. The upper sandy limestone bed in the northeastern corner of the monolith was the only rock at the specified grade elevation of 465 feet NGVD.

The development of the first ledge resulted from blasting operations within the southern limits of the draft tube keyway since this area was reshot.

(b) <u>Monoliths D-8 thru D-12</u>

Prior to foundation work, the preliminary foundation surface contained a number of small ledges (1±) oriented from east to west. This surface varied from El. 465 feet NGVD to El. 464 feet NGVD with higher areas existing along the eastern edges, especially in the southeastern corner. The only area lower than El. 464 feet NGVD was in Monolith D-8 which was adjacent to and centered on Monolith D-7.

The foundations for Monoliths D-8 thru D-12 were worked sequentially from south to north. This method resulted in working the rock in a down dip direction which had its good points as well as bad. On the negative side, it tended to create constant problems with ponded water in the working area and probably resulted in the removal of additional rock. However, it reduced the possibility of having to remove parts of the protective concrete slabs which might have been necessitated if work on an adjacent area had loosened a bed of rock underlying the protective concrete.

Treatment and subsequent exposure of the total final foundation surface with depth revealed the following lithologies:

(1) A light colored, fine grained, sandy limestone with the upper bedding plane being marked by a thin argillaceous laminea upon which all of Monolith D-11/D-12 were founded.

(2) A sequence of medium grey, argillaceous, very thin bedded limestones with discontinuous sandstone lenses. The sandstone lenses were typically cross bedded as shown by their presence in the Monolith D-12 foundation wall.

(3) Overlying the argillaceous unit was another sandy unit consisting basically of three distinct beds. The upper and lower beds were approximately 5 inches to 7 inches thick while the center bed varied

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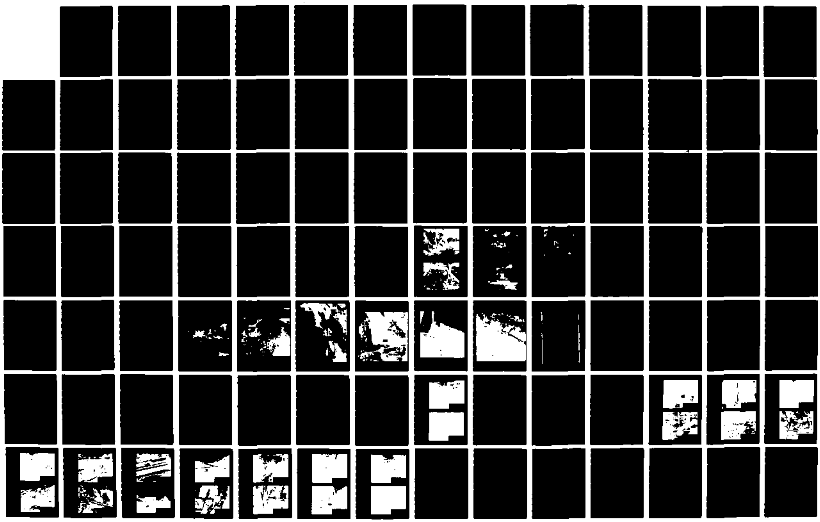

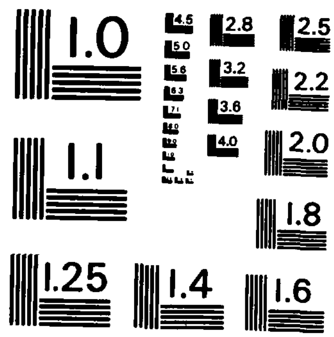

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from 10 inches to 18 inches thick. These beds were uniformly light colored, fine grained, weakly cemented, sandstones with the central bed being characterized by small, irregular features. This unit formed the bulk of the foundation for Monoliths D-10 and D-9.

(4) A unit of argillaceous, medium to dark grey, mottled dolomites, the bedding of which varied in thickness laterally, but ranged from 3 inches to 8 inches as exposed. This unit formed part of the foundation for Monolith D-7/D-8.

(5) The last unit was composed of generally light grey, fine grained sandy dolomites and limestones which varied in thicknesses from 3 inches to 5 inches. This unit formed the eastern part of the foundation floor Monolith D-7/D-8. In general, the sandstones were very similar in appearance and nature. They tended to thicken where they overlaid troughs in the argillaceous rocks and were generally thinner up dip and to the west. The bedding planes were generally finer grained and darker in coloration (refer Drawings Nos. 73/2 and 73A/2 for geologic cross section and details).

The argillaceous limestones and dolomites were generally medium to dark grey overall, but normally consisted of lighter and darker laminations. These laminea varied from even and parallel to discontinuous and contorted (referred to elsewhere as mottled). Bedding was generally thin, ranging from a fraction of an inch to 10 inches; the majority of the beds were 2 inches to 4 inches thick. The bedding surfaces were invariably dark colored and sometimes separated by shaly, dolomitic, argillaceous material which was quite susceptible to deterioration due to water, ice or excessive heat. Some of the final surfaces were even and smooth, but most surfaces had numerous sedimentary features (the most common were rod-like casts 2 inches to 3 inches long and about 0.5 inch in diameter) which generated "bumpy" surfaces. Absorption of water ranged from 3.2% to 5.8% relative to dry weight. Only one specimen was tested for swelling and it expanded 0.017 inch/foot perpendicular to the bedding. Specimens of this category of rock tend to deteriorate rapidly when alternated between soaking and drying conditions due to separation between laminea.

On 13 June 1975, foundation treatment began within the general confines of Monolith D-11/D-12 with a special emphasis on the 2-foot wall foundation. The Contractor decided to place protective concrete on all foundations below planned grade which would receive steel reinforcement. Since the plans called for steel reinforcement in the area from Station 3+00 to Station 3+85, 34 feet downstream to 111 feet downstream, this area was worked in conjunction with the wall foundation and was broken up into four placements lettered A through D, each of which measured about 40-foot square. The first of these, Placement A, which was located in the southeastern corner, was signed off on 30 June 1975. Placements B, C and D were generally ready by the time Placement A was completed, but they were left for awhile so that the carpenters and steel workers could work on the forms for the 2-foot wall placement. The rock removed from these areas and, in fact, all of Monoliths D-11 and D-12, had been very thin bedded (1± inch) which made it susceptible to loosening. However, it was found that the two or three layers upon which Placements A through D were founded were durable and could be left alone for a long period without adverse affect. After minor work, Placements B and C were placed on 11 July 1975 followed by Placement D on 15 July 1975.

During the first week of July 1975, the Contractor expressed concern as to how far the same bedding plane for Monoliths D-11/D-12 foundation would be used as the foundation for Monolith D-10 since a high ledge (3-foot to 4-foot) had been created in this monolith without any apparent improvement. The Contractor was directed to clean off the area from Monolith D-1 through Monolith D-10, and on 9 July 1975, the District and Division Geologists inspected the exposed foundation surfaces. They concurred with the method and extent of treatment. Work continued on the Monolith D-10 area and consisted mainly of rock removal with a hoe ram. The amount of work in this area tapered off until 23 July 1975 when all rock work ceased for a period of five days. By this time, it had been decided by the Contractor to cover all foundations with concrete from the on-site batch plant. Expecting to be able to make concrete by 1 August 1975, work on the foundations in Monoliths D-11, D-12 (western part) and D-10 resumed on 28 July 1975. These areas, subject to a time limitation, were signed off on 30 July 1975, but due to difficulties, only the southern 40 feet of foundation floor in Monolith D-11/D-12 were covered. The ledge in Monolith D-10 was ignored while foundation work was shifted to Monoliths D-8 and D-9 from 31 July 1975 until 2 August 1975.

On 2 August 1975, treatment resumed on the ledge in Monolith D-10 primarily in the downstream area where both vertical and horizontal preexisting cracks had separated significantly. The ledge was taken back beyond the vertical cracks and otherwise excavated until a satisfactory degree of soundness was achieved. Concrete placement for Monolith D-10 took place on 4 August 1975 followed by concrete placements for Monoliths D-9 and D-11/D-12 (remaining portion) during the remainder of that week.

Work in Monolith D-8 was intermittent from the end of July 1975 to the early part of August 1975 with the Contractor assigning only a few men during those shifts when the area was worked.

During this period, the foundation in Monolith D-8 was checked daily by Corps' representatives and appeared to be fairly competent. This is plausible since the overlying sandstone proved to be non-expansive, but highly absorptive, while the underlying rock was very expansive. This is assumed to have put the upper rocks under considerable stress.

On 11 August 1975, new tension cracks were noted on the upstream end of the foundation floor in Monolith D-8. An exploratory pit was directed and, during its excavation, the foundation bowed and ruptured along the northwest-southeast trend. This rupture extended through the sandstone and into the underlying thinly bedded unit. The hoe ram was brought back into the area and started breaking and removing the affected rock.

During the morning of 13 August 1975, a meeting was held between the Corps' project personnel and the Contractor's superintendent regarding the possibility of placing the protective concrete slab in Monolith D-8 that night. It was deemed feasible, but not by the start of the second shift. The area was ready by 0400 hours and concrete was placed on 14 August 1975.

4. Powerhouse Foundation

The powerhouse foundation consisted of several foundation sections which were designated individually by letter. Those foundation sections within the erection bay were designated with the letter "E", those within the pump back unit with the letter "P", and those within the kaplan unit area with the letter "K". For purposes of this report, the narratives dealing with foundation treatment within the powerhouse limits are divided into the following three areas: (1) powerhouse keyway (foundation sections P-1a, P-1c, K-1a and K-1b); (2) upstream foundation sections; and (3) downstream foundation sections. The location of individual sections are shown on Plate No. 1.

(a) Powerhouse Keyway

The keyway foundation consisted of four sections (P-1a, P-1c, K-1a and K-1b) which lie between dam axis Station 5+90, 30.75 feet downstream to 70.25 feet downstream, and dam axis Station 7+60.75, 30.75 feet downstream to 70.25 feet downstream. The foundations consisted of fine grain argillaceous, thin bedded limestone. The dip of the bedding was in a northerly direction and the rock was absorptive.

In general, the final foundation surfaces were planar with one or two bedding planes forming the floor for each of the four concrete placements which covered the keyway. Due to the dip of the rock, the final grade was much lower in the northern end than in the southern end. The southern end was also the only area where sandy rock was encountered. Cracks were essentially absent except in the southern end (close to design grade) and the northern end (where the presplit shot was drilled and loaded too deep).

Initial removal of loose rock in the keyway was performed intermittently from the end of January 1975 through 22 February 1975. During the latter part of this period, construction of a protective shelter over the southern two-thirds of the keyway was nearly completed when the batch plant fire occurred on 22 February 1975 terminating this work. Subsequent foundation treatment did not resume until the last part of May 1975.

(1) Section P-1a

Foundation treatment started on 22 May 1975 and was completed by 28 May 1975. Visual inspection of the foundation surface during preliminary work showed many fractures associated with blast hole patterns, but these died out with depth. Generally, the final foundation followed a single bedding plane with the exception of treatment in a single small dome area where a test excavation was made into the structure which revealed a small sandy lense. The results of the test excavation indicated further treatment could cause damage to the surrounding competent rock while only marginally improving the foundation.

(2) Section P-1c

This section was founded on the same limestone bed as P-1a and is unique in that it had no significant features at all. Foundation treatment began on 29 May 1975 and continued until the area was covered with protective concrete on 3 June 1975.

(3) Section K-1a

Final treatment of this foundation section began in conjunction with work in Section P-1c on 2 June 1975 with concrete being placed on 5 June 1975. The Contractor was able to "step up" to the next bedding plane in this section since the foundation was free of defects.

(4) Section K-1b

Foundation treatment for this section involved extensive excavation of a sandy/argillaceous limestone by the hoe ram. The final foundation (first above the white porous sandstone encountered in the sump) was signed off on the night of 9 June 1975 and was covered with concrete on 10 June 1975.

(b) Upstream Foundation Sections

(1) Section P-2a

Final work on this foundation section began in conjunction with foundation treatment operations in P-2b and K-2b on 25 April 1975. Final foundation approval was given on 1 May 1975 with the protective concrete being placed on 2 May 1975. There were no significant features or problems encountered during final treatment.

(2) Section K-2a

The foundation for this section was plagued with both natural and man-made problems from the time it was blasted until the protective slab was placed. Initial production blasts apparently rifled so the area required reshooting. The holes drilled for the reshooting were smaller diameter holes and were placed between the initial 3-inch holes. The result was a floor showing the two well developed patterns of blast induced fractures connecting the holes. In addition, there was a poor degree of control on the depth of the holes and the placement of the charges as shown by the floor being approximately 1 foot below grade and yet many of the holes were still visible (mostly the 3-inch diameter holes). The area was worked intermittently between 13 January 1975 and 29 January 1975 with the majority of the work being accomplished during the period of 17 February 1975 through 20 February 1975. During this period (17 thru 20 February 1975), the protective plastic covering collapsed twice and the foundation was allowed to be exposed to freezing. Since the El. 485 foot NGVD bench was allowed to drain on this foundation and because the foundation had many small and large depressions, this freezing had a profound, adverse affect on the competency of the rock. The end result was that each time the area was worked, a large amount of rock removal was required. Protective concrete was placed in four placements over a period of time from 20 February 1975 through 16 June 1975.

A major foundation feature was a deep excavation in the southern part of this section which was bounded by a 1-foot by 2-foot ledge composed of poorly bedded sandy limestone except for the top 3 inches of the ledge. This upper bedding plane formed the final floor along the western edge of the section. The majority of the floor was founded in the lower sandy limestone material which was shattered by blasting.

(3) Section K-2b

The first protective concrete slab placed after the batch plant fire (22 February 1975) was for foundation section K-2b on 30 April 1975. Due to its size and location between two monoliths having protective concrete, treatment was rather easy and was done on an intermittent basis from 23 April 1975 through 29 April 1975 when it was signed off. The major problem with this monolith was the presence of a drain from the El. 485 foot NGVD bench above the western extension which kept the surface wet and caused significant ponding in the eastern portion.

The rock type was basically sandy limestone and, except in the lowest areas, lacking any continuous bedding plane. One major crack ran east-west along the northern portion, but was of little consequence. Generally, the entire surface required some degree of treatment.

(4) Section E-2a

The foundation for this section was composed almost entirely of poorly bedded sandy limestones/calcareous sandstones, except for minor remnants of the thin bedded argillaceous limestone. The major feature of the foundation was a joint set consisting of two prominent joints traversing the entire east-west dimension of the foundation section. This joint set aligned with the major joint running through the north wall of the sump and also aligned with a row of blast holes. The final surface of Section

E-2a was rough and undulatory due to lack of bedding planes and blast shatter affects. Most of the areas removed were of limited extent, many being created and enclosed by jack hammer distress marks. Shatter and propagation cracks tied into joints in some areas and propagation cracks between holes were common. The foundation was covered with a plastic enclosure after the first of the year, but was exposed to many periods of freeze-thaw cycles prior to that time. After its enclosure, it was heated sporadically which contributed to the thawing of the foundation and then refreezing when heat was again removed. The freeze-thaw action was aggrevated by water being allowed to pour onto the foundation from the El. 485 foot NGVD bench above. The plastic structure repeatedly collapsed due to snows during January 1975 and February 1975. The structure was again heated during the period 15 February and 16 February 1975 just prior to placing the protective concrete on 18 February 1975.

(5) Section P-2c

Foundation treatment for Section P-2c principally occurred from 8 May 1975 to 15 May 1975 with the protective concrete being placed the following day. The final planar surface consisted of a series of beds ranging in thickness from 0.2 foot to 0.5 foot which dipped in a northerly direction (the rock which was removed eventually created a ledge of considerable thickness by the time "stepping up" was achieved in foundation section K-2c).

The foundation contained three major cracks or sets of cracks which had a minor influence on the extent of treatment since all were tight and did not require dental work. The treatment of the blast holes was limited in scope due to the amount of excavation in excess of the design grade. Initially, the uppermost foundation layer was a thick sandstone (refer treatment narrative for foundation section K-2c) in which both directions of the blast patterns and most of the holes could be seen. Once this thick layer was removed during common excavation, only the major east-west fractures remained. Few other problems arose since this rock surface was easily worked with jack hammers once the sandy cap rock was removed. The remainder of the rock removed during final preparation was thin bedded with well defined, continuous bedding planes. These layers had a fine grained, dark, bedding plane with the majority of each bed being light colored and arenaceous.

(6) Section P-2e

Final treatment of this foundation section began in conjunction with treatment operations for Sections Nos. P-2c and E-2b on 5 May 1975. The section was signed off on 13 May 1975 with protective concrete being placed on 14 May 1975. There were no significant features or problems encountered during final preparation since the majority of drummy sounding rock had been excavated along its eastern downstream permieter. Due to this, the Contractor began the protective concrete placement on the western (upstream) portion and placed toward the eastern end.

(7) Section K-2c

The protective slab for foundation section K-2c was placed on 21 May 1975 and was one of the most difficult foundation sections to prepare. The main difficulty was a high ledge of rock which had first been encountered in foundation sections K-2a and P-2e. Due to the northerly dip of the beds, the ledge had grown progressively higher as it was removed entirely from Section P-2e. On 16 May 1975, the ledge was located in the southern end of the section and formed a triangle with the southern and eastern section limits. At this time, the upper surface was deemed drummy in the western part of the section and the bottom of the ledge was "making water". Additionally, near the northeastern corner, there was a deep crack running N32°W which was rapidly draining any water available to it. The decision was made to remove this area and try to save the eastern portion which was still sound. This work was successful with very little extra rock having to be removed due to the excavation of the main mass. When this excavation was completed, it was apparent that the blast-induced fractures had completely penetrated the ledge rock and, with one minor exception, had failed to cross the bedding plane between the dissimilar rock types. The work in the upper surface was completed with relative ease since it was spotty and invariably bounded by blast fractures which eliminated much guesswork for the laborers.

This left the northern or upper part of the foundation in sandy limestone with the lower floor on a thin bedded argillaceous limestone. Only two fractures penetrated from the upper into the lower unit, both of which were oriented east-west and were reduced to barely visible hairline fractures in the lower floor.

(8) <u>Section K-2d</u>

This foundation section was worked, mostly in conjunction with other powerhouse foundation sections, from 7 May 1975 thru 12 May 1975. The majority of the rock was a sandy limestone with few exceptions. Blast fractures and blast holes were of local importance with one major fracture running roughly east-west in the northern one-third of the section.

In the northeastern corner, a large block of rock had to be removed and this work continued even while concrete placement was underway. The block daylighted into the keyway and was founded on argillaceous limestone. Although water was a constant problem, due in part to the choppy nature of the floor, excavation problems were minimal.

(c) Downstream Foundation Sections

(1) Section E-2b

Although foundation treatment had been performed on this area in 1974, it was of an areal nature and specific removal toward a final foundation did not begin until about 1 May 1975. At this time, Section E-2b was bounded on the north by several lifts of concrete (Monolith D-5/D-6) and on the east and west by protective slabs of Sections E-2a, E-1c and E-2c. Intensive treatment was performed from 1 May 1975 through 6 May 1975 with protective concrete being placed on 7 May 1975.

The northern portion (Station 7+90 to Station 8+12) consisted of a flat surface having a dark, fine grained bedding surface with the rock being quite competent. The rest of the foundation was composed of the "sandy limestone" which was thickly bedded. Locally discontinuous and irregular fine grained laminea formed domes or depressions on a small scale.

Prior to final treatment, there were two areas below grade; one was adjacent to the sump and which was created during the sump excavation, while the second area was next to Section E-2a. The low area running along Section E-2a was rectangular with its long axis running N35°W. Both the eastern edge (in E-2b) and western edge (in E-2a) showed blast holes along the peripheries. This condition, along with holes within the depression, indicated it to be an area inadvertently loaded too deeply.

(2) Section E-2c

The foundation of Section E-2c was worked approximately four times, including preliminary work done during the latter part of 1974. Loose rock was removed and steel mats were placed on 27 January 1975, but the foundation was left unprotected. On 28 January 1975, the Contractor asked for a determination of the soundness of the rock and was informed that there were significant areas of unsound rock which would have to be removed. After receiving this information, the Contractor elected to finish placing the steel mats which quite effectively reduced the access to the foundation to 1-foot grids between the bars. On 3 February 1975, foundation work again started (under a protective plastic shelter) and continued until 6 February 1975 when the Contractor decided to cease treatment operations due to gradation problems in the batch plant and the resultant lack of concrete. This area was heated and protected from most of the snow which fell from 6 February 1975 through 12 February 1975. The Contractor resumed work on 12 February 1975 with final approval being given at the end of the second shift and concrete being placed on 13 February 1975.

Most of the rock in this area consisted of a thinly bedded limestone with the beds varying in thickness from 1 inch to 2 inches. These beds covered a thick bed of sandy limestone. The final surface was generally flat except for an area in the center where some of the thin beds, just above the sandy limestone, were in low domes and basins. Another sedimentary feature noted was the presence of ripple marks in the uppermost layers, especially in the northern quarter, which was initially sound.

The major amount of foundation treatment occurred in those areas were numerous blast holes were present. Generally, these areas were worked back to a fracture which ran east-west between the northern one-third and the central one-third of the foundation section. This fracture extended from the eastern section limits at a point 9 feet from the northeast corner to a point approximately 22 feet west and 8 feet from the northern section limits where it died out. This fracture was in line with a row of blast holes and had a few holes located in its boundary at both ends. The surfaces of the fracture were fresh and, in general, typical of a blast-induced fracture. About 4 feet from the eastern section limits, a deep blast hole was noted with extensive radial cracking and, at this point, the fracture showed its greatest relief. This was the only area where the rock was broken to the north of the fracture.

(3) Section P-2j

Initial foundation treatment was performed during the period from late 1974 to early 1975. Intensive foundation preparation commenced on 3 June 1975 with the protective concrete slab being placed on 11 June 1975. It was considered an "easy" foundation to prepare due to the presence of large slabs in the southern end and a hard, thick, sandy bed in the northern end. The only problem area seemed to be its western boundary with the keyway where large blocks of rock bounded by parallel fractures were present.

The southern portion required the removal of a couple of argillaceous layers approximately 6 inches each. These layers were worked back until they intersected a long blast fracture. Additional rock was removed from the southeastern corner where it joined Monolith SP-1 since the bedding planes had been exposed to ponded water.

The biggest problem in the northern portion was the lack of a bedding plane to work to and the presence of numerous blast holes and fractures. Some problem areas were generated by jack hammers loosening sound rock. In general, the drummy areas were spotty, limited in size and normally associated with blast holes.

(4) Section P-2m

Foundation section P-2m contained the entire spectrum of lithologies present in the Louisiana Limestone from the brittle undulating argillaceous limestone to the lighter colored, moderately to poorly cemented sandstone. Jointing was not a problem since no natural joints were detected and generally relatively few propagation blast fractures were present.

The foundation floor of Section P-2m was originally exposed in October 1974. At this time, the Contractor discovered the rock was above maximum grade elevation and worked the floor down to grade. At the conclusion of this excavation, the area was left exposed to freeze-thaw cycles until it was protected with plastic cover and heated after the steel mats were placed. Between 1 February 1975 and 14 February 1975, heat had been discontinued in Section P-2m over one weekend and the floor again allowed to freeze. The freeze-thaw cycles greatly contributed to the volume of rock removed. As in Section E-2c, the Contractor elected to perform final foundation preparation after the placement of the lower reinforcing steel mats (8-inch grids), consequently, this practice increased the treatment duration. Protective concrete was placed on 14 February 1975.

(5) Section K-2g

Foundation section K-2g was exposed throughout the winter of 1974-1975. Foundation treatment started in late spring 1975 with 6-inch protective concrete placed on 2 June 1975. Treatment involved the removal of approximately 1 foot of drummy sounding rock. It was discovered shortly after the commencement of treatment operations that the southern one-fifth of the foundation floor was approximately 4 inches above grade. This area was subsequently taken down to grade but the operation was restricted by the presence of reinforcing steel protruding from foundation section P-2m. As stated earlier, the resultant foundation floor was normally good after the removal of the initial 1-foot of rock; however, the exceptions are noted below. The foundation floor was a combination of thin, discontinuous, laminated beds of limestone separated by shale/siltstone partings and more sandy limestone with no apparent bedding being encountered during excavation. Where the thin sandy unit surfaced, hammer marks were numerous due to the lack of weak bedding planes to break to; however, they had little affect on the soundness of the floor. Two extensive blast fractures were noted for a distance of 5 feet to 7 feet off the southern extent of Section K-2g. Some rock was lost in the southwest corner of the foundation apparently due to these fractures. Otherwise, the rock was tight on either side of the cracks but some dental work was required where they had opened. The other area of rock loss due to blasting was in the northeast section corner which intersected with Monolith TA-1. A large apparent joint was noted running east-west through this foundation. Its origin as being blast induced was not supported by observation. The joint was observed to be between two presplit holes in the east wall and the draft tube keyway. The joint was 0.5 foot south of a natural joint in the cap rock (foundation wall) which dies out in a shale seam. It is felt that the joint in the K-2g foundation was an extension of the joint in the above cap rock.

(6) Section K-2h

The foundation for this section was exposed through the winter of 1974-1975 with no protection from the weather. When foundation treatment began on 26 May 1975, approximately 1 foot of extremely drummy sounding rock existed over the entire floor. After this rock was removed, a sound foundation was obtained. The drummy rock consisted of a very thin silty limestone (occasionally shaly) varying in thickness of 1 inch to 2 inches with the remainder of the unit being a sandy limestone (homogenous). The foundation was approved on the night shift of 29 May 1975 with the protective concrete being placed on 30 May 1975.

(7) Section E-2d

This foundation was worked slightly in 1974 and some preparations were made in February 1975, but when the batch plant fire occurred on 22 February 1975, all work ceased. Due to the fire, placement sequences were drastically altered to make allowance for the loss of site concrete and the Contractor directed his work crews to the upstream powerhouse area. Work resumed in Section E-2d on 17 May 1975 and continued until 22 May 1975 when approval was given for concrete placement which took place on 23 May 1975.

When foundation treatment started on 17 May 1975, there were numerous large areas where the beds (0.1 foot to 1 foot) of rock were extremely drummy. Although underlain by a thick sandy unit, the working surface was within the thinly bedded and undulating rock. It all areas except the northern 9± feet, and the southwestern corner, excavation to sound rock resulted in the foundation being formed by a single undulating bedding plane. The northern foundation area was founded on two bedding planes which overlaid the main surface and was similar in nature but more planar. A small triangular excavation in the southwestern corner carried through the main layer into the sandy limestone exposed in the sump.

Fractures were significant within this foundation section, especially along the eastern edge formed by the deeper excavation of Monolith TA-1. The numerous blast fractures which passed from Monolith TA-1 foundation wall into the foundation of Section E-2d and the lower elevation of the floor of Monolith TA-1 resulted in the necessity of removing rather large blocks of rock from the edge. It should be noted that the final blast in Monolith TA-1 was loaded with twice the within accepted powder level; however, in view of the nature of its foundation and those adjacent to Monolith TA-1, it can only be concluded that most, if not all, of the problems during excavation were a direct result of the overloading.

5. Downstream Walls

(a) Stilling Basin Wall

(1) Monoliths SB-1 through SB-4

Monoliths SB-1 through SB-4 are founded on the Hannibal Shale at approximate El. 487.5 feet NGVD. Initial excavation was to within 1 foot of final foundation grade in October 1974. The Contractor started foundation treatment operations in April 1975 and placed protective concrete in Monoliths SB-1 through SB-4 from 10 April 1975 through 21 April 1975. As the foundation shale surfaces were exposed, they were treated, protected and covered immediately. Extensive foundation treatment was performed under and around the shotcrete protection, especially in the keyway.

(2) Monoliths SB-5 through SB-7

Foundation excavation for these monoliths began on 9 October 1974 with foundation treatment beginning in early November 1974 and the placement of all protective concrete being completed by 25 November 1974. Due to the expediency of excavation, the rock surfaces were not subjected to any undue hardships; hence, no major treatment or problems resulted.

(b) Splitter Wall

(1) Monolith B-7

Due to the fact Monolith B-7 is located directly in front of the splitter wall (from Station 5+60 to Station 5+90, Offset 110 feet to 125 feet downstream), the foundation treatment performed on this monolith will be discussed in this section.

The foundation of Monolith B-7 was shot to grade during the latter part of September 1974 and the first part of October 1974. It was left with a high relief bench in its northwest corner which consisted of a sandy limestone with no prominent bedding. The bench had a thickness of approximately 1.5 feet and topped out at El. 465 feet NGVD. Only a small amount of unsound rock had to be removed from the perimeter of this bench. The remainder of the floor was composed of thin bedded argillaceous limestone and broke evenly along bedding planes. Generally, the dip of the beds was very gentle toward the northwest corner. The lowest elevation encountered was El. 463.2± feet NGVD in the extreme northeast corner, whereas the remaining major part of the floor was at an average elevation of approximately El. 464 feet NGVD. Overdrilled blast holes were noted in the lower, southeast section of Monolith B-7 and along its eastern boundary, but blast shatter and propagation fractures were not present even though the holes ranged up to 0.6 foot in depth. The floor was exposed to freeze-thaw action but this had little affect in the area. As a whole, the foundation was quite sound with only a few minor areas of drummy rock being removed. The foundation of Monolith B-7 was approved on 7 January 1975 with the protective concrete being placed the same day.

(2) Monoliths SP-1 and SP-2

The foundation for these two monoliths were worked on numerous occasions in the latter part of 1974 and the beginning of 1975 with the majority of work being performed through reinforcing steel mats. Initial work had left a foundation surface composed primarily of two large bedding planes which dipped in a northerly direction. However, after a number of freeze-thaw cycles, it became apparent that the upper layers of rock had become loosened. Additional cycles of intermittent treatment exposed more beds and, thus, the Contractor decided to remove the reinforcing steel prior to the final treatment in early June 1975 with the foundation being signed off on 16 June 1975, but was not covered with protective concrete due to a lack of sufficient concrete. The foundations were flooded by rain the same night (16 June 1975). Cleanup and labor problems prevented protective concrete placement until 20 June 1975 and 24 June 1975. During this time, the foundation was relatively untouched and well maintained. The only adverse affects noted were the slight loosening of a 1-inch thick layer in an area having approximately a 2.5-foot diameter and characterized with jack hammer pits.

Final exposure revealed that the foundation was composed of 4-inch to 8-inch beds of sandy limestone having well defined bedding planes marked by dark argillaceous material. The northern end consisted of thicker bedded sandy limestone while only in the southern end near the wall were there any thin layers of siltstone. Consequently, the overall foundation surface was planar with a uniform dip toward the north. In general, working practices were satisfactory and, at least during the final treatment, little damage was done to the rock.

(3) Monolith SP-3

The Louisiana Limestone in this monolith was exposed at approximate El. 470 feet NGVD on 19 August 1974. The rock was subsequently drilled and shot to grade (or below) in numerous separate blasts from the period 19 August 1974 to the latter part of October 1974. Preliminary cleanup of the foundation was performed on 31 October 1974 with minor amounts of unsound rock being removed through the steel reinforcing mats on 8 January 1975 and 9 January 1975. The foundation was approved on 10 January 1975 with protective concrete being placed the same day.

The foundation surface of Monolith SP-3 was largely composed of the sandy, porous, coarse-grained limestone with a bed of the more coarse-grained, laminated limestone occupying the northern one-fourth to one-third of the floor. A number of beds was observed in the foundation floor which dipped very gently to the west-northwest. A few overdrilled blast holes were noted in the northern quadrant of the foundation, but these holes did not damage the foundation.

No major problems were observed in the floor. One major joint or possible blast propagation fracture was observed. All things considered, the foundation of Monolith SP-3 at the time of protective concrete placement was sound and in excellent condition for receiving concrete.

(4) Monoliths SP-4 through SP-6

Excavation of these foundations began on 1 July 1974 and continued intermittently through July 1974 and August 1974. The Contractor performed final foundation treatment and placed protective concrete as the foundations were exposed and approved. Very little treatment was required on the monolithic shale foundations with no major problems being encountered.

Final approval for approximately half of the foundation for Monolith SP-6 was given on 30 July 1974 with the protective concrete being placed on 31 July 1974. The remaining portion of the foundation for Monolith SP-6 was approved on 1 August 1974 with the protective concrete being placed the same day. The greater part of the foundation for Monolith SP-5 was signed off on 6 August 1974 along with placement of the protective concrete. The remaining foundation was approved concurrently with the placement of protective concrete on 8 August 1974. The majority of the foundation for Monolith SP-4 was approved on 16 August 1974 with protective concrete being placed the same day. The remaining portion was approved and protective concrete placed on 19 August 1974.

(c) Tailrace Wall

(1) Monolith TA-1

The initial limestone foundation for Monolith TA-1 was in poor condition. This was due to three factors: (1) the foundation was exposed from September 1974 through May 1975 when foundation treatment began; (2) the foundation was continually used as sump and (3) overloading of blast holes. The combination of these factors required considerable foundation treatment. The area of principal concern was in the center two-thirds of the foundation where 2 feet to 3 feet of rock excavation occurred due to blasting. There were also several joints which discharged groundwater. These were subsequently sealed. Final approval for approximately two-thirds of the foundation was given on 27 June 1975 with protective concrete placed the same day. The remaining portion of the foundation was approved and convered with concrete on 10 July 1975.

(2) Monoliths TA-2 and TA-3

Foundation preparation for Monoliths TA-2 and TA-3 began on 29 October 1974 and continued through 5 November 1974 when the foundation was approved. Protective concrete was placed the following day, 6 November 1974.

The majority of the rock was thin bedded limestones having little or no dip. The only area where the thicker bedded sandy limestone was encountered was in an area of overexcavation in the northeastern corner. This area extended from dam axis Station 7+60 to Station 8+00± and was approximately 8 feet wide with its long axis parallel to the centerline 200 feet downstream. It was within this area that numerous blast holes occurred.

The major problem during foundation treatment was the presence of water which entered along the eastern wall and ponded in areas such as the one noted above. In the northern part of the monolith, water was entering from a crack in the floor which ran along dam axis Station 7+96. In the southern portion, water was entering from the keyway in Monoliths TA-4 and TA-6, and to a much lesser extent, from a crack at the base of the eastern wall at Station 7+35.5. Pipes were installed over the fractures to prevent water from accumulating under the placement, but control of the water during foundation preparation was inadequate. Fortunately, there were no freezing temperatures so a minimal amount of work was required on 5 November 1974 and a good foundation was obtained.

(3) Monoliths TA-4 and TA-5

Monoliths TA-4 and TA-5 are founded on the Hannibal Shale. Work progressed very rapidly on these monolithic foundations. The Contractor excavated, treated and placed the protective concrete in approximately one month (from 26 June 1975 to 29 July 1975). No problems were encountered during these operations. It should be noted that the north wall of Monolith TA-5 gave a slight indication of a small shear zone in the shale just downstream of Monolith D-5/D-6.

(4) Monoliths TA-6 through TA-9

These monoliths are also shale founded. The Contractor began excavation of all four foundations on 6 June 1974. Protective concrete for Monoliths TA-7, -8 and -9 was placed on 10 July 1974 and 11 July 1974. Monolith TA-6 received protective concrete on 27 July 1974 and 29 July 1974. No problems were encountered during the work performed. Treatment was minimal with little rock having to be removed and few joints treated.

6. Stilling Basin and Tailrace

(a) Stilling Basin

Foundation excavation for the stilling basin began on 27 April 1976 and was completed by 1 June 1976. Excavation was conducted to within 6 inches of the final foundation surface by Luhr Bros., Inc., while Massman Construction Co. removed the remaining shale during foundation treatment operations. Massman Construction Co. commenced foundation treatment on 19 May 1976 with the first placement of 6-inch protective concrete being placed on 1 June 1976. Each cycle of foundation treatment was followed by protective concrete placement due to the 1-hour exposure limitation for shale. The final placement of protective concrete occurred on 9 July 1976. Very little treatment was required on the shale foundation since no major problems were encountered.

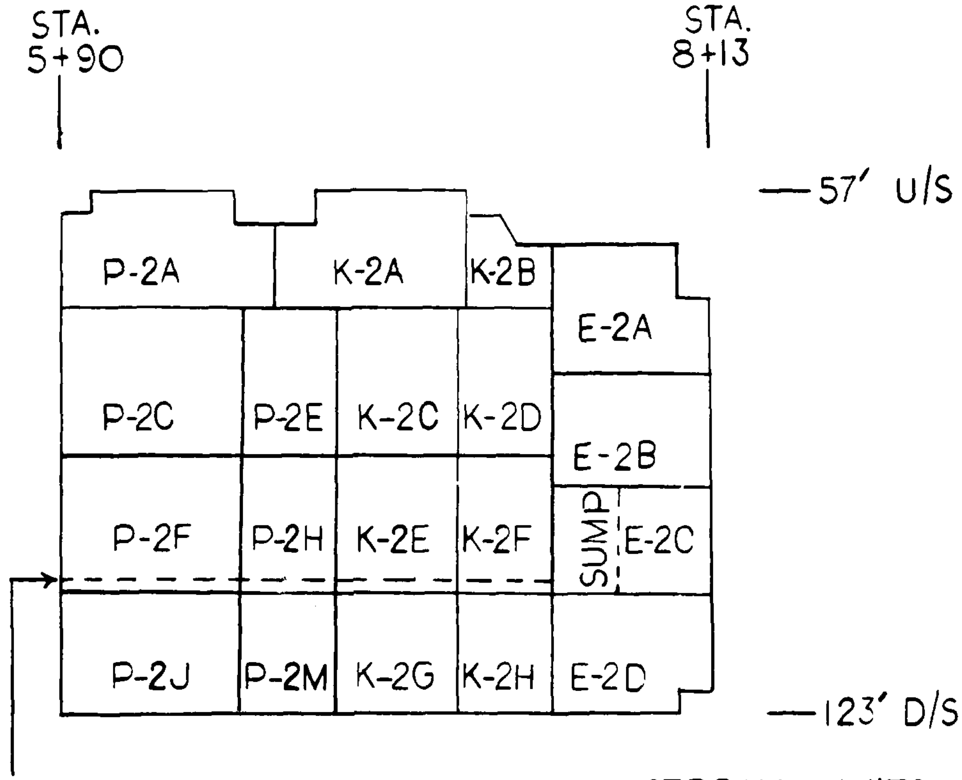
(b) Tailrace

Foundation excavation for the placement of the tailrace pavement commenced in July 1977 with the Contractor (Luhr Bros., Inc.) excavating to within 2 feet of final grade. The shale was left above grade until 19 September 1977 at which time the Contractor resumed excavation operations, and by the following day, an additional 18 inches of shale had been removed. The final 6 inches of shale were removed by Massman Construction Co. using a Gradall.

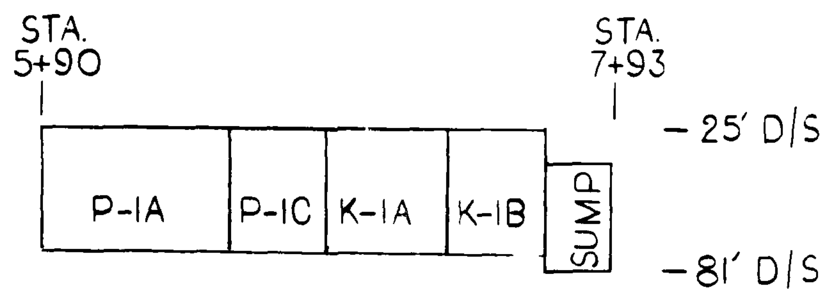
Foundation treatment began on 22 September 1977 and was completed by 17 November 1977. The amount of foundation preparation was greatly reduced when the contract was modified to cover the foundation shale with 6 inches of sand within one hour prior to concrete placement. The Contractor generally prepared a relatively small (20-foot by 20-foot) portion of the foundation at one time.

The shale was sound except around the perimenter. For approximately 3 feet out from the shotcrete-covered excavations for the splitter wall, tailrace wall and powerhouse, approximately 2 feet of shale were removed. The concrete placements began on 21 September 1977 with the final concrete being placed by 22 November 1977.

POWERHOUSE SECTIONS



DASHED LINE REPRESENTS DOWNSTREAM LIMITS OF KEYWAY AND SUMP



DRAFT TUBE KEYWAY AND SUMP

PLATE NO 1

B. Embankment

1. Diversion Channel

An extensive foundation shale preparation for the earthen embankment within the limits shown on Drawing No. 120/2 (pseudo-core) was conducted from 11 September 1979 to 6 November 1979. The foundation work started after the closure of the third-stage cofferdam and after the completion of all mucking operations within the diversion channel upstream of the pseudo-core. The foundation work involved the removal of all highly weathered shale, surface preparation, treatment of all open iron stained joints and the placement of dental concrete or slush grout. The foundation treatment was performed by Luhr Bros., Inc. (subcontractor) utilizing the following equipment:

One (1) CAT 245 Backhoe
One (1) CAT 14G Motorgrader
One (1) Gardner-Denver 900 Air Compressor
One (1) John Deere Model 410 Rubber-tired Backhoe (end loader)
One (1) Allman Portable Light Plant
One (1) Electric Sump Pump
One (1) Gasoline-powered Hand Compactor
Assorted Small Tools (including jack hammers, sledge hammers, shovels, brooms, buckets, picks, rock hammers, wheelbarrows, bullhoses and small air hoses)

The sequence of operations for all foundation work within the pseudo-core can be chronologically divided into three sections: the diversion channel, the left bank of the diversion channel and, lastly, the shale bench adjacent to the limestone embankment 1V:1H contact.

The first priority of work was the treatment of the Hannibal Shale within the limits of the diversion channel from dam axis Station 15+00± to Station 17+00±. Work commenced in the extreme northwest corner of this section on 11 September 1979 with Luhr Bros., Inc., preparing an area approximately 30-foot wide by 65-foot long. Upon completion of all foundation treatment and the covering with an initial loose lift of 18 inches of impervious material, a second area to the east was excavated and prepared. This sequence continued until the downstream limit of the pseudo-core was reached. The Contractor could then move to the south of the last prepared area and work in a westerly direction. In this manner, after several passes back and forth, the shale was exposed, prepared and covered with impervious fill until the tie in with the Phase I fill. The Contractor finished the last area in the extreme southeast corner on 28 September 1979. During this period, Luhr Bros., Inc., labor personnel worked two 10-hour shifts, six days per week. The two shifts allowed the Contractor considerable flexibility as the night shift could initiate foundation treatment in a new area or could complete all foundation treatment so that the day shift could cover the final surface within the 12-hour specification requirement. Occasionally, the Contractor would excavate an area larger than the labor crew could prepare. In this case, the shale was covered with impervious fill or weathered shale to prevent or reduce the depth or dessication. The final shale surfaces were generally covered within at least eight hours of first exposure and those areas left open for more than eight hours required additional dental treatment.

The Contractor utilized a CAT 245 backhoe to strip the highly weathered shale down to the top of firm shale. The backhoe generally worked in a north-south direction so that all bucket grooves were parallel to the dam axis centerline. The laborers would then use blow pipes to remove the remaining rock fragments or the thin scale of shale due to dessication.

The exposed shale surface was then sounded by a Government geologist who painted the drummy areas to be removed, joints to be veed and overhangs to be eliminated. Laborers using jack hammers, rock hammers, sledges, picks and shovels then removed the appropriate rock and blew down the surface again. Occasionally, the CAT 245 backhoe was used to remove an overhang or a large block of shale which had been separated from the rest of the shale mass by open joints and/or bedding plane partings. The geologists then checked the surface again for final approval. If the shale surface was not ready for approval, the process of sounding, painting, excavating and blowing was repeated until a satisfactory final surface was reached.

The surface was then mapped uisng either plane table and telescopic alidade or transit and tape (refer Drawings Nos. 121/2 through 125/2). Photographs were taken of the final surface and any unusual feature (refer Volume 3, Section 3).

Shortly after the initiation of foundation preparation, the use of the "bull" air hose was generally discontinued and small diameter blow pipes and brooms were used to remove debris from the foundation in order to reduce the degree of dessication.

Open or weathered joints were veed to a depth of four to five times their width and backfilled with dental concrete or mortar grout (piable mixture of concrete, sand, water and 1%± expansive agent). Overhangs that developed during dental excavation were eliminated either by further excavation or tuckpointing with mortar grout. Depressions at joint intersections with seeps were filled with concrete, whereas the depressions generated from excavation of drummy rock were filled with hand compacted impervious fill. Bedding plane seeps at the base of ledges (joint faces) were tuckpointed. The Hannibal Shale in the diversion channel is a compaction shale normally bluish grey to greyish green when slightly to moderately weathered. The formation is thin bedded with no appreciable dip and highly jointed. Occasional fossil molds of brachiopods, horn corals, pyritized cephalopods and nodules of pyrite and calcite varying in diameter from 1/4 inch to 1 inch were found. The major (high angle, 85° to 90°) and minor joint systems were well developed as evidenced by their concentration and latitude as shown on Drawings Nos. 121/2 through 123/2. The mineral joint filling (1/20 inch to 1/2 inch in width) ranged from calcite, pyrite to silica. In a number of cases, the joints were found to be open or partially filled with either rust colored, yellow or grey clay, oxidized pyrite or weathered calcite. Typically deep weathering occurred at the intersection of major and minor joint systems which allowed small seeps or springs to develop during dental excavation.

On 1 October 1979, after completion of the tie in to the Phase I fill, Luhr Bros., Inc., began work on the left bank of the diversion channel. The Contractor worked one 10-hour shift, six days per week during this period. The area worked extended approximately from dam axis Station 17+00 to Station 18+00 (refer Drawings Nos. 124/2 and 125/2). Generally, a crew of approximately six laborers performed the foundation preparation.

The Contractor utilized a CAT 245 backhoe to excavate the bulk of the weathered incompetent shale. In several cases, a motorgrader was used to strip the remainder of the weathered shale along the near vertical faces down to firm shale. This practice eliminated unnecessary excavation. The preparation and mapping procedure outlined earlier was followed. Due to concentration of open (1/10 inch) closely spaced relief joints, slush grout was applied in order to seal these joints. A grout mix of 3:1 water/cement ratio was applied and then the excess grout was broomed off.

This section was previously the north bank of the diversion channel excavated by the Phase I Contractor. Due to the steep slope and formation jointing characteristics, the Contractor was forced to excavate the bank in a series of steps which generated a series of benches and high angle slope faces (generally 5 feet high). This practice allowed for the development of an overall 1V:1H slope. The degree of weathering and jointing necessitated considerable shale removal which generated concern that the left abutment concrete fillet would be undercut. Consequently, some variance from the 5-foot face limitation for embankment compaction was waived by SLD Foundation personnel.

Embankment compaction against the high angle slope was performed principally with a motorgrader, whereas in the diversion channel, the Caron wheel roller was used. This change of procedure for compaction drastically reduced the amount of hand compaction.

The lithology of the Hannibal Shale in this section was basically the same as the diversion channel with a few exceptions. Fossil molds of crinoids were found and the concentration of brachiopods was more abundant. An essentially continuous 1 1/2-inch thick bed consisting of pyrite, calcite and silica was found at approximate El. 512 feet NGVD. The predominate jointing in this area was stress-relief which created a cleavage-like appearance in the shale. These joints were founded in essentially parallel sets with a spacing of as little as 1 inch between joints. Common spacing was 4 inches to 12 inches. These stress-relief joints varied from tight to open and occasionally were filled with clay. Jointing in this area was also more often associated with yellow to brown staining as the depth of primary weathering varied from 4 feet to 6 feet deep (refer Drawings Nos. 121/2 through 123/2).

The last section from approximate dam axis Station 18+00 to the Hannibal-Chouteau contact was worked on a 10-hour per day shift, six days per week. The preparation of this area was completed by 6 November 1979 when the last of the shale was covered with impervious fill. The only appreciable difference in preparation of this shale was the use of the motorgrader exclusively to blade off the incompetent shale. Stress-relief jointing was also very prominent in this area and, as such, was treated accordingly. As a last order of foundation treatment, any drummy protective fillet slab concrete or concrete at the shale/limestone contact was removed and replaced with mortar grout.

Due to the depth of primary weathering and the high concentration of joint systems in each section, an approximate 300% overrun in shale excavation occurred.

2. Left Abutment Basal Concrete Fillets

Foundation preparation for the left abutment basal concrete fillets began on 27 July 1978 and was completed on 7 September 1978. The Contractor generally utilized a single 10-hour shift for foundation preparation and protective concrete placements. The principal equipment utilized by Massman Construction Co. during foundation preparation consisted of the following:

One (1) Gardner-Denver 600 Air Compressor with Air Tools
One (1) CAT 14 Motorgrader
One (1) CAT 977 End Loader
One (1) CAT 980 End Loader
One (1) CAT 955 End Loader
One (1) Case 680 End Loader
Jack Hammers, Blow Pipes and Other Assorted Small Tools

The procedure implemented by Massman Construction Co. concerning foundation preparation for the basal fillets consisted of the removal of the highly weathered Hannibal Shale in a downstream-upstream direction by the use of a CAT 14 motorgrader so that foundation preparation could precede the fillet forming operation. A foundation strip approximately 12-foot to 25-foot long and a width of one-half the vertical joint face was excavated under the direction of a Government geologist using a Case 680 end loader and jack hammers to remove the remaining incompetent foundation shale. The final surface would then be blown down, geologically mapped and photographed (refer Volume 3 Photographs, Section 4 and Volume 4 Drawings Nos. 126/2 thru 129/2). Just prior to the placement of the protective concrete, the surface would be blown down again. Generally, the final surface was covered with protective concrete within the one-hour specification limitation (refer Drawings Nos. 13/2 thru 133/2 for the location of the basal fillets and comments on the orientation and spacing of the relief joints within the fillets' foundation).

3. <u>Left Abutment Cutoff Wall Foundation</u>

Foundation preparation for the cutoff wall was done by Massman Construction Co. from 11 August 1978 through 22 August 1978. The Contractor used the same labor force and equipment for this operation that was used in foundation preparation for the left abutment basal fillets.

The Contractor began foundation preparation at the rear of the cutoff wall by excavating all drummy or fractured foundation shale as outlined by a Government geologist from Station 20+00 to Station 19+87. The unsuitable foundation shale was excavated by a Case 680 end loader and jack hammers. When the Government geologist was satisfied with the foundation, the final surface would then be blown down, geologically mapped and photographed, and covered with protective concrete (1-inch top size aggregate) within one hour. The Contractor followed this same procedure as outlined above for the remainder of the shale preparation within the cutoff wall. Due to the bedding characteristic of the Hannibal Shale, most final surfaces were uniform in elevation and free of joints (refer Drawing No. 136/2 for the final shale elevations and the location/orientation of the various joints).

4. Left Abutment Contact Area

Foundation treatment of the left abutment embankment contact area (Burlington and Chouteau Limestone Formations) began on 11 September 1980 and was completed during the 1983 construction season. The limits of the foundation treatment are shown on Drawings Nos. 120/2 and 126/2 thru 130/2. Foundation treatment was performed by Luhr Bros., Inc., by utilizing two 3-man crews with the appropriate equipment and, depending upon the time of year, an 8-hour to 10-hour shift. The Contractor utilized the following equipment for abutment treatment and rock removal:

One (1) CAT 245 Backhoe
One (1) CAT D9 Dozer
One (1) CAT D8 Dozer
One (1) CAT D6 Dozer
One (1) Euclid End Dump
One (1) CAT 988 End Loader
One (1) CAT 977 End Loader
One (1) CAT 951 End Loader
One (1) Terex End Dump 33-05
Three (3) Portable Light Plants
One (1) Grove Hydrocrane R/T
One (1) Gardner-Denver 900 cfm Air Compressor
One (1) John Deere Rubber-tired Backhoe
Assorted Small Hand Tools (including several air hammers, air chisels, rock hammers, blow pipes and spades)

Foundation treatment within the limits of the left abutment embankment contact area was divided into two major areas. The first area was all foundation rock within the confines of the core trench (pseudo-core) and the second area was all foundation rock outside the limits of the pseudo-core. The division of the embankment contact was based upon the type and degree of foundation treatment required by the specifications. The area within the pseudo-core received the most elaborate treatment (foundation preparation, removal of all drummy rock, dental treatment and joint treatment), whereas the area outside received principally dental treatment. Due to the long exposure of the abutment face (since 1977), considerable rock removal was required to provide a competent surface for embankment placement.

Generally, the Contractor's labor force performed foundation treatment by working from upstream to downstream for a height of 5 feet. The foundation treatment was supervised by a Government geologist. The final surface was mapped and photographed, and all approved surfaces were covered with embankment prior to the 7-day limitation in the specifications (refer Photograph Volume No. 3, Section 4).

Foundation preparation of the abutment surface of the foundation rock within the psuedo-core consisted of removal of all weathered, fractured, loose or drummy-sounding rock, and the veeing of all joints, blast fractures and weathered bedding planes to a depth four to five times their width. All veed areas were subsequently backfilled by using a dry pack mortar grout mixture or concrete. In addition, battered (1/2V:1H to 1V:1H) concrete fillets were placed to eliminate overhangs or to maintain the 1V:1H abutment slope where large amounts of rock had to be removed (refer Drawings Nos. 131/2 thru 133/2 and Photograph Volume No. 3, Section 4 for location and details).

The treatment of the upstream foundation section consisted of the removal of highly weathered, fractured, loose or drummy-sounding rock. When the removal of rock was so extensive an overhang or vertical face greater than 7 feet in height was created, battered (1/2V:1H to 1V:1H) concrete fillets were placed to maintain a 1V:1H slope (refer Drawings Nos. 131/2 thru 133/2 for locations). The veeing of open joints, cracks and bedding planes to a depth of four to five times their width was also performed with these areas being subsequently backfilled with a mortar grout mixture or concrete. Foundation treatment in the far upstream reaches of the left abutment and areas near (with 2± feet) the embankment template did not receive as high a degree of treatment as the pseudo-core, but received, in general, a greater degree of treatment than the downstream section of the left abutment outside the pseudo-core.

Treatment of the Burlington and Chouteau Limestone Formations downstream of the pseudo-core received the lowest degree of treatment because the rock surface was covered with a 10-foot wide pervious blanket (refer Drawings Nos. 107/2 and 108/2). Treatment consisted of the removal of detached rock slabs, removal of extremely drummy-sounding rock, treatment of the overhangs and the sealing of clay-filled joints and bedding planes by dental concrete. The same method of foundation treatment for drummy rock, joints or fractures and overhangs was followed as outlined in previous paragraphs. The criteria for fillet placement was the presence of a 10-foot vertical rock face.

Foundation treatment performed on the Chouteau Limestone during the 1980 construction season was minimal. Work started on 11 September 1980 and completed on 31 October 1980. Two problem areas were encountered which required extension foundation excavation and dental treatment. The large amount of excavation resulted in the placement of two battered concrete fillets (390 feet upstream, El. 567± feet NGVD and 20 feet upstream, El. 575± feet NGVD) (refer Drawing No. 131/2). The final approved limits for the 1980 construction season are shown on Plate No. 2.

Foundation treatment for the 1981 construction season began on 7 April 1981 and continued intermittently until 20 October 1981. Luhr Bros., Inc., began by removing up to 5 feet of impervious fill against the left abutment embankment contact area due to frost damage and to tie into the upper limits of the previously prepared surface. The principal area of foundation treatment occurred between dam axis Station 18+35 and dam axis Station 18+40 at approximately 165 feet upstream due to the presence of extremely drummy rock and the intersection of several major joints. Foundation treatment resulted in the removal of approximately 5 feet of rock and the establishment of several benches. The quality of rock improved only slightly with depth and concrete wedges were placed around the perimeter of the benches to stop any potential seepage through the bedding planes (refer Drawings Nos. 126/2 and 127/2 for locations). Between 23 July 1981 and 17 August 1981, all foundation work was suspended due to the July flood.

The second period of foundation treatment for the 1981 construction season began with the re-examination and treatment of the foundation rock within the Embankment Protection Zone (EP) template (refer Section 10, Part D). Since the EP was designed to control a flood of the July 1981 magnitude, foundation treatment within the EP Zone paralleled that within the pseudo-core. No major rock removal was required due to flood damage although much of the dental tuckpointing and dry pack required removal with subsequent backfill. By 15 September 1981, the Contractor had prepared the rock surface to El. 590 feet NGVD (refer Photograph Volume No. 3, Section D for examples of the final foundation surface within the EP). Luhr Bros., Inc., finished the 1981 construction season by treating the foundation surface upstream of the EP (from 450 feet upstream to 270 feet upstream) (refer Plate No. 2 for the limits of final approved limits on the left abutment).

Foundation treatment for the 1982 construction season began in June 1982 and continued intermittently until 11 November 1982. The Contractor worked the entire length of the abutment and raised the extreme upstream and downstream portions of the embankment up to finished main dam template. There were several areas which required extensive foundation excavation within psuedo-core and downstream of the pseudo-core (centerline to 225 feet downstream). The extensive excavation required the removal of large detached limestone blocks by a CAT 245 backhoe and CAT D8 dozer in order to expose and treat several clay-filled to open major joints. In addition, a continuous battered concrete fillet was placed at Station 18+40 from 90 feet to 110 feet downstream at El. 585 feet NGVD in order to seal off several major/minor joints and to eliminate an overhang.

A 4-inch diameter drill hole was encountered within the pseudo-core at Station 18+52, Offset 65 feet downstream, El. 590 feet NGVD, which extended to El. 555.5 feet NGVD. The drill hole was No. 638C drilled by Government forces. It was backfilled using non-shrink grout poured down a 2-inch diameter PVC pipe.

The final approved limits of the left abutment embankment contact area for the 1982 construction season are shown on Plate No. 2.

During the 1983 construction season, foundation treatment on the left abutment commenced in late April 1983 and was completed by 20 August 1983. The Contractor's labor crews (under Government direction) prepared the remaining portion of the left abutment/embankment contact zone, the cutoff trench and the right abutment water temperature control weir embankment/rock contact. The Contractor shifted his labor crews between the abutment face and the cutoff trench in order for foundation treatment to be performed on a continual basis and to maximize fill placement operations.

As described in earlier portions of this narrative, the same treatment procedures for drummy rock, joints and overhangs were followed and the degree was dictated by the location of rock defect. The area within the limits of the pseudo-core received the maximum amount of treatment (refer Drawing No. 120/2).

On the abutment face, there were several battered concrete fillets (22.5° to 45°) placed for the purpose of sealing the solutioned feature, eliminating overhangs or vertical faces by extensive rock removal (refer Drawings Nos. 131/2 thru 133/2). A typical example would be exposure and treatment of the solution feature at Station 19+05, 50 feet upstream, El. 628 feet NGVD. This joint was an extension of Cavity No. 5 which had been backfilled and sealed off with concrete. The joint/cavities were cleaned out to El. 623 feet NGVD and backfilled with dental concrete.

Foundation preparation and dental treatment within the limits of the cutoff trench were performed up to El. 638 feet NGVD on the upstream and downstream faces, and El. 640 feet NGVD on the rear face. Generally, the downstream face was blown down using a blow pipe with very little, if any, jack hammering being performed due to its contact with the sand drainage system. Treatment on the upstream face consisted principally of the removal of the highly fractured chert with consideration given to the elevation and the fact the chert seams did not improve significantly with depth.

The floor received full dental treatment. Joints and drill holes, regardless of their location, were cleaned out and backfilled with dental concrete and grout, respectively. The exposure of a solutioned major joint at Station 19+55 on the upstream and downstream faces required the placement of battered concrete fillets (refer Drawing No. 132/2 for fillet locations and Drawing No. 129/2 for strike and dip of the major/minor joint systems within the limits of the cutoff trench).

In order to prepare for closure in August 1983, the Contractor's labor force prepared the right abutment limestone face within the limits of the water temperature control weir contact. The limits of treatment ranged from El. 552± feet NGVD to El. 565± feet NGVD for an approximate width of 30 feet upstream and downstream of the water temperature control weir centerline (refer Drawing No. 57/2 for general contact limits and photographs for the general condition of the limestone face prior to and after foundation treatment). Foundation treatment consisted of the removal of some very loose limestone block and the cleaning of the rock surface with shovels and brooms. Only a minimal amount of rock was excavated due to the interconnecting of the rock block and the fact that the joints were clay filled. In addition, hand compaction was performed to ensure a good bond between the rock and earthen fill.

APPENDIX

MEMO TO FILE

1. The purpose of this memo is to consider the factors which contributed to the difficulty in obtaining a sound foundation in the Louisiana Limestone. The prime considerations are as follows: (1) the various lithologies within the Louisiana Limestone; (2) the sedimentary and structural features which had a bearing on the excavation procedures; (3) the activities of the contractors and (4) the natural forces which were acting on the foundations.

2. The Louisiana Limestone, as exposed in the excavation, may best be divided into the following three distinct units: a thin upper lithographic limestone unit and a lower unit comprised of a sequence of calcareous siltstones, limestones, dolomites and a sandy dolomitic limestone. The division between these units was marked by a thin (3-inch to 8-inch) horizontal and continuous layer of shale similar to the Hannibal Shale Formation. The upper unit and shale were of limited importance since they formed part of the foundation walls. However, this shale seam did affect the charge distribution within the presplit and production shot for the first lift.

3. The upper unit is 2-foot to 3-foot thick and consists of a tan, homogenous, aphanitic limestone (lithographic) which includes scattered pyrite cubes and occasional inclusions of secondary crystaline calcite and dolomite. This unit may be considered as one continuous horizontal bed of rock having the appearance and brittleness of chert. Underlying the lithographic limestone is a thin layer of shale which is dark grey in color, weathers easily, contains rounded, black, cherty pebbles, pyrite and a few fossils. Occasional stringers of this shale run up into the first unit for a short distance.

4. Beneath this shale seam lies a series of beds varying in thickness from 0.25-inch to a few feet thick. Generally, these beds range from 0.75 inch to 6 inches if fine grained, and 8 inches to 2 feet thick if they are medium grained, e.g., limestones. Most of these beds are argillaceous limestones and fine grained limestones, but other coarser grained and/or less calcareous layers are fairly common, especially south of the powerhouse sump. In the bottom half of the sump excavation, El. 455.0± NGVD, a nearly white, porous calcareous sandstone was encountered, but it is thought to be of limited extent since it was not shown in the exploratory logs and was not encountered elsewhere in the excavation. It may be said that the foundation forming rocks varied in color from very light grey to dark grey with the majority being medium to dark grey. The sandy limestones are invariably light grey, while the finer grained rocks owe their variation in color to variable amounts of darker grains which usually occur in bands. Bedding planes were generally marked by thin shale laminae between beds. This feature is well shown in the cores of this limestone which also show the uneven nature of the bedding; however, these are not unusually weak zones as was shown by the integrity of the cores after considerable handling, storage and direct shear tests performed. The interface between the shale layer and the lower unit shows pitted and mineralogically darkened surface indicating a hiatus which is consistent with the change in lithologies above and below the shale seam.

5. The physical properties of the rock had a limited affect upon the excavation procedure; however, they did have an affect upon blast design and certain methods of hand excavation. Although the upper unit exhibits a different response to blasting (a shattering due to its homogenous and

brittle nature), it has little significance for excavation considerations. The important feature is the bedding of the lower unit in which the foundations are based and their response to shock waves, vibration and wedging action. Its presence is an obvious advantage especially since its near horizontal state provides for clean and even breakage with a minimum of effort and explosive. The incorporation of the presence of these bedding planes into the blasting design could have effectively eliminated the need for subdrilling. The presence of these bedding planes facilities excavation with jack hammers since the rock will break at the bedding planes. The only detrimental aspect of this bedding is its susceptibility to water absorption and the wedging action of ice if continually flooded and allowed to freeze.

6. Jointing within the limestone was conspicuously rare. Those joints which were present were marked by coatings of calcite and pyrite and were of minor structural importance. When the limestone was first exposed, two preferred joint directions were noted with the most persistent joints oriented N25°-30°W with the second set striking perpendicular to these with lengths on the order of 50 feet. Both sets were tight and can best be described as hair line. These sets were highly localized with most of them occurring between dam axis Station 6+00 and Station 8+00; furthermore, they were restricted to the cap rock and did not continue into the underlying units.

7. When first exposed, the top of the Louisiana Limestone showed a very slight anticlinal nature which had an axis trending N15°W, and a very gentle plunge northward. Dips on the flanks and nose never exceeded one degree and the structure was limited to Monoliths D-8, D-7 and draft tube keyway. The remainder of the cap rock was essentially horizontal.

8. The lower limestone unit (all rock below the cap rock and separated by a thin shale bed) had its own structure which was unrelated to that of the cap rock. The bedding in this unit dipped 1° to 7° toward the north in the area of the upstream powerhouse. The only other dipping strata (northwesterly 1° to 5°) was located in the northwestern corner of Monolith D-1. The reason the local features were considered sedimentary rather than structural was shown by their flattening as they approached the upper lift.

9. Thus, the overall condition of the lower portion of the Louisiana Limestone was conducive to obtaining sound foundations with a minimal amount of effort. Structural problems inherent to the rock, such as open joints or soft beds, were essentially non-existent as exhibited by the integrity of the foundation rock cores after extended storage and visual examination of the foundation areas themselves. The only physical properties of the rock which were significant were their layered nature and absorption of water which could not be considered unusual or different from core log descriptions.

10. The Contractor was responsible for the shot design and the sequence of blasting. His standard blasting plan for the 2-foot and 3-foot production lifts called for one cartridge of 1 1/8-inch by 8-inch (40% Special Gelatin) or 1 1/4-inch by 8-inch (70% Red Arrow) placed at the bottom of the hole with the remainder of the hole being backfilled with stemming. The Contractor elected to use 3-inch diameter holes and to space the production holes between 2 feet and 3 feet. The main problem with the selection of the 3-inch diameter holes and spacing was the continued rifling of the shots with resultant poor breakage. Toward the end of their blasting operations, smaller drill bits were used with slightly inclined holes which resulted in the elimination of the rifled shots and better rock breakage.

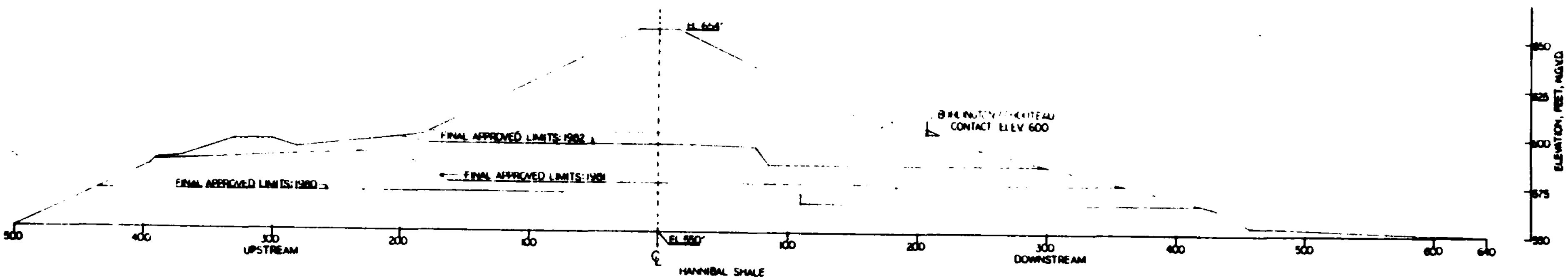
11. Basically, the Contractor used two methods of controlling hole depth for blasting operations. The most precise method was the installation of a string line which had been surveyed into the work area. From this, the drillers would drill a prescribed distance stopping when a mark on the drill steel matched the string. An alternate method was to cease drilling when a mark on the steel reached the top of the rock. Problems occasionally arose when it was intended that the drillers stop when their marked steel reached a string line, but due to misunderstanding, lack of supervision, etc., the drillers drilled until the mark reached the top of rock.

12. Final foundation treatment was divided into three phases. During the initial phase of work when relatively large quantities of rock were to be removed, a hydraulic ram mounted on the back of a tractor was used in conjunction with large (50-pound) jack hammers. The rock was then removed with front end loaders. The foundation was cleaned by washing with high-pressure water hoses.

13. The second phase involved the most time and required the largest crew. Equipment included hand tools, jack hammers (30-pound to 50-pound), high-pressure air and/or water hoses and 5-foot pry bars. The pry bars were used for sounding out "drummy" areas and removing layers under 2 inches thick while jack hammers were used for removing layers in excess of 2 inches thick. Throughout this phase, the working areas were washed whenever debris interferred with the workers. Loose rock was loaded into skips by hand and removed by the "whirley" or other equipment. Once the foundation had most of the "drummy" areas removed, then final cleanup was performed.

14. The last phase involved the cleaning of any walls, trimming of loose parts of ledges of rock and removal of any loose or unsound rock found during the final inspection. Any loose or unsound rock was generally removed in pails. The foundation was washed, followed by air cleaning, to remove puddles of water.

15. This sequence of operations proved to be quite effective when followed; however, it was often interrupted between the second and final phase. Areas which were allowed to freeze became loose with the amount of additional work being generally proportional to the number of cycles of freezing and thawing.



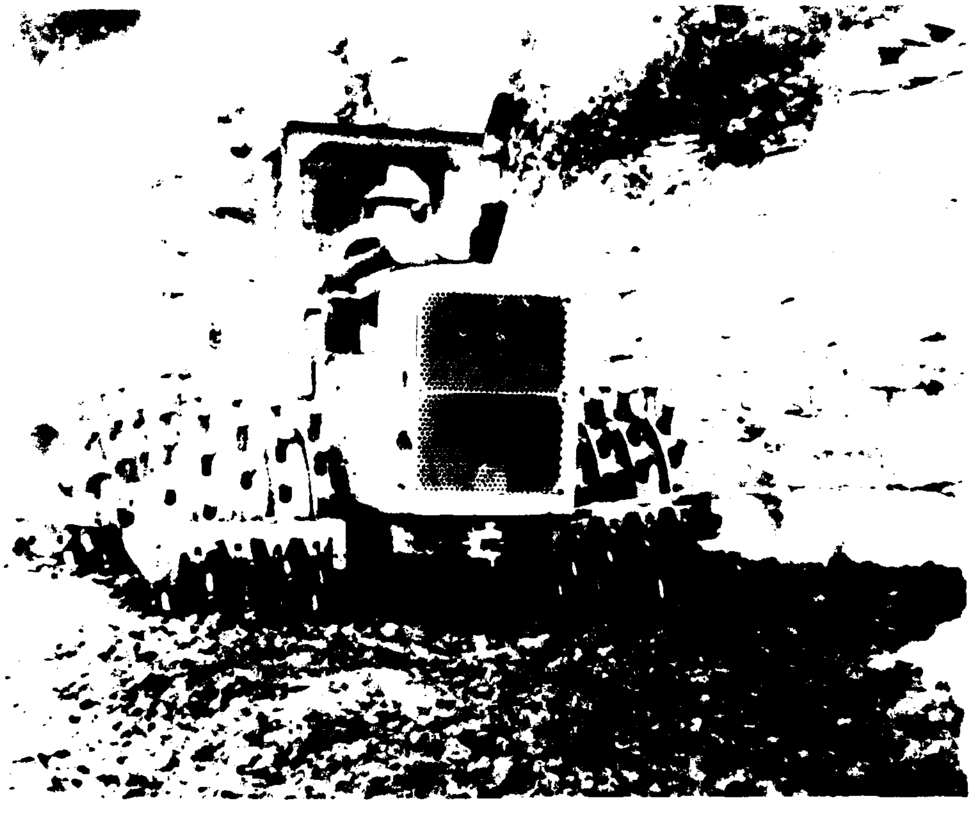
FINAL APPROVED LIMITS: 1982
FINAL APPROVED LIMITS: 1981
FINAL APPROVED LIMITS: 1980
CONTACT ELEV 600
EL 550'
UPSTREAM
DOWNSTREAM
HANNIBAL SHALE
ELEVATION, FEET, NGVD
500
400
300
200
100
100
200
300
400
500
600
640

PLATE NO. 2

Photograph A depicting condition of rock before treatment is performed on right abutment water temperature control weir embankment/rock contact.

Photograph B depicting condition of rock before treatment is performed on right abutment water temperature control weir embankment/rock contact.

Photograph C showing wheel rolling for compaction on rock/embankment contact of water temperature control weir.



Photograph D showing rolling of fill using sheepsfoot roller near rock/empankment contact of water temperature control weir.

Photograph E showing compaction of fill using mechanical tamper on rock/embankment contact of water temperature control weir on right abutment.

SECTION 7

ROCK BOLTS AND STILLING BASIN ANCHORS

A. Rock Bolts

1. Introduction

The contract specifications for the rock bolt and rock anchor program required the Contractor to perform a rock bolt and rock anchor test program, to install and tension production bolts of varying lengths, and to install rock anchors. The specified locations for the pattern bolting and rock anchors are designated on Drawings Nos. 65/2 and 66/2 and Table No. 1 shows a comparison between estimated quantities and as installed quantities. The specifications required a rock bolt and anchor test to assure that the Contractor's systems and installation techniques satisfied the specified criteria. The entire program was conducted by Luhr Bros., Inc. from October 1973 to June 1975. A complete installation sequence for the production rock bolt/anchor required the drilling of a boring to the specified depth, gaging of the hole diameter, insertion of the resin cartridges, installation of the 1-inch diameter bolt/anchor, installation of the bearing plate, bevel, flat washer, hexagonal nut, and lastly the stressing of the bolt. Rock anchors were not stressed. The purpose of the rock bolts was to compensate for unloading and to prevent or minimize the development of relief jointing in the limestone and shale. The rock anchors were used primarily to hold wire mesh in place for shotcreting.

2. Rock Bolt and Rock Anchor Test Program

See Appendix A.

3. Production Bolting

The Contractor began production bolting in October 1973 and installed the last bolt by June 1975. The Contractor used a single shift operation until April 1974, and a double shift operation for the remaining period of time. Principal equipment used was as follows:

ATD 3100 A Gardner Denver Drill
AT 3700 Gardner Denver Drill
750 Gardner Denver Compressor
900 Gardner Denver Compressor
977 CAT Front End Loader
Inland-Ryerson Center Hole 50 Ton Hydraulic Jack
Simplex Forceback Centerhole Jack Model RC3025B
Maxi Light Plants

All rock bolts and rock anchors used were of hot rolled and proof stretched alloy steel, with a rolled-in pattern of deformations for the entire length, and referred to as a "Dywidag Bar". They were manufactured by Inland-Ryerson Construction Products Company of Chicago, Illinois. See Appendix A, Page 4, for characteristics of the "Dywidag Bar". The bars were delivered in stock lengths and cut to the appropriate length in the field.

The installation procedure for the rock bolts as determined by the test program required the Contractor to drill a 1 and 7/8-inch diameter hole to within the final 10 feet of the hole. Rock anchor holes were 1 and 3/8-inch in diameter for the entire length. The last 10 feet of the hole (anchorage zone) were drilled by using a 1 and 3/8-inch drill bit to help ensure a greater bond between the rock bolt and rock. The hole was blown clean using a minimum of 50 psi compressed air.

Generally, only those holes in the foundation walls or floors were gaged for proper diameter. See Appendix A, Figures 5 and 6. The Contractor next installed a sausage shaped fast set polyester resin (set time 0-1 minute) manufactured by Celtite Inc. of Cleveland, Ohio, into the anchorage zone of the hole. A slower setting polyester resin (set time 2-30 minutes) was installed in the remainder of the hole. It was found that wet drill holes did not affect the bond of the celtite to rock, however, the celtite was very temperature sensitive, i.e., the greater the temperature the faster the set of the celtite. Celtite was generally used in place of grout, however, the only exception to this was on the upstream and downstream walls in the right non-overflow section (Monoliths D-13 thru D-16), where the anchorage zone was celtited with the rest of the hole backfilled with non-shrink grout, and on the upstream 1V:1H slope where a sinkhole was encountered. In these areas the bolts were grouted. The resin cartridges consisted of a reinforced polyester resin component together with its catalyst in a single sausage shaped package isolated from each other by a reacted interface (Appendix A, Page 5). The 5 types of Celtite Resin used were:

1. 3212 HV 0001
2. 3212 MV 0510
3. 3512 HV 0001
4. 3512 MV 0204
5. 4012 HV 1530

The bolt or anchor was then placed into the drill and spun into the hole at a minimum of 40 rpm as recommended by the Celtitie representative.

The bolt was spun in order to mix the catalyst agent and polyester resin in the celtite package. The bearing plate, bevel, flat washers and hexagonal nut were then placed on the bolt or anchor. Due to the low compression strength of the shale, a 14-inch by 14-inch by 1/2-inch bearing plate was used, whereas a 5-inch by 7-inch by 1/2-inch plate was used in the limestone. The bolts were then stressed with a 50-ton center hole jack (refer Photographs I and J). In stressing, the hydraulic ram and gages were checked regularly with a Terrametric Load Cell which was calibrated on a regular basis in the concrete lab on the compression testing machine. Bolts were actually overstressed to approximately 35,000 pounds to ensure at least a 25,000-pound load transfer onto the bolt. The procedure for stressing was as follows:

1. Apply a load of 35,000 pounds on gage
2. Use a wrench to tighten hexagonal nut until 5,000 pounds to 10,000 pounds were removed from the gage reading
3. Release load

It should be noted that very few of the bolts failed the load transfer.

Rock bolts and anchors were installed in the pattern bolting design area in the proper lengths and sequence as the rock excavation progressed on the right abutment 1V:1H slope and within the perimeter of the concrete structure with two exceptions (refer Drawings Nos. 67/2 thru 86/2 for as-installed locations of bolts and anchors). The length of the bolts in Monolith D-15 was increased from 15 feet - 20 feet to 20 feet - 25 feet so as to have a 10-foot anchorage zone in the shale. The rock bolts installed in the upstream and downstream faces of the right abutment monoliths were horizontal instead of at a 10° angle below horizontal to avoid intersecting rock bolts installed on the 1V:1H slope 40 feet upstream and downstream from the outside corners of the monoliths, and avert those rock bolts installed on the upper back face of the next lower monolith. Additional rock bolts were required to be installed in Monolith D-14 due to the presence of parallel joints which highly fractured the foundation of the monolith. The rock bolts were installed in an attempt to close the joints prior to the protective slab placement.

Additional rock bolts and anchors were installed outside the limits of the pattern design area due to the presence of solution features and extreme fracturing/jointing. The bolts installed on the upstream portion of the right abutment (directly upstream of Monolith D-17, El. 640 feet NGVD to El. 625 feet NGVD) were to stabilize the rock adjacent to the sinkhole, whereas bolts installed on the downstream abutment 1V:1H face pinned large loose limestone blocks.

On the upstream right abutment slope, open joints trending N70°E were encountered in front of the above-mentioned sink. Both rock bolting and grouting of the voids were necessary in this area for slope stability. The objective of grouting rock bolts instead of using celtite was to grout the bolt instead of the foundation. After grouting several of the bolts, the Contractor was allowed to use the celtite resin in the holes. The bolts were generally not stressed. It was felt that the stressing of the bolts could initiate movement and precipitate a slope failure. The bolts installed on the downstream portion of the right abutment 1V:1H slope primarily acted as dowels to hold the limestone blocks in place (refer Drawing No. 67/2 for exact locations and details).

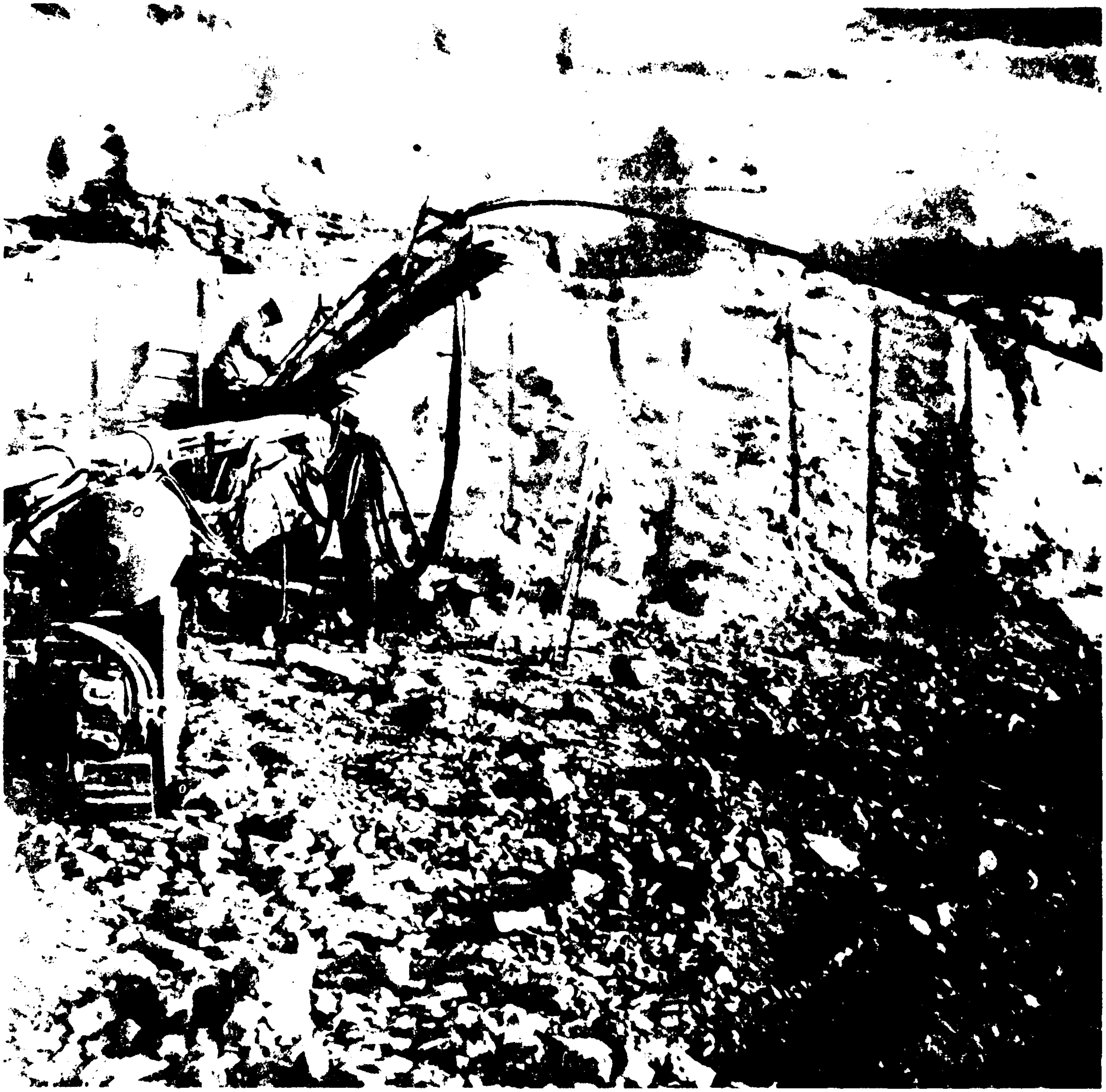
ROCK BOLTS AND ROCK ANCHORS

COMPARISON OF BID QUANTITIES TO FINAL QUANTITIES

	Bid Items	Bid Quantities	Final Quantities
1.	15 Foot Rock Bolts		
	A. First 25 Bolts	25	25
	B. All Over 25 Bolts	15	17
2.	20 Foot Rock Bolts		
	A. First 70 Bolts	70	70
	B. All Over 70 Bolts	15	360
3.	25 Foot Rock Bolts		
	A. First 140 Bolts	140	140
	B. All Over 140 Bolts	45	224
4.	30 Foot Rock Bolts		
	A. First 195 Bolts	195	195
	B. All Over 195 Bolts	60	185
5.	35 Foot Rock Bolts		
	A. First 200 Bolts	200	200
	B. All Over 200 Bolts	60	62
6.	40 Foot Rock Bolts		
	A. First 100 Bolts	100	100
	B. All Over 100 Bolts	30	24
7.	4 Foot Rock Anchor		
	A. First 1,800 Anchors	1,800	1,068
	B. All Over 1,800 Anchors	0	0
8.	8 Foot Anchors		
	A. First 600 Anchors	600	600
	B. All Over 600 Anchors	200	1,008
9.	12 Foot Anchor		
	A. First 300 Anchors	300	300
	B. All Over 300 Anchors	300	970
10.	Testing Rock Anchors		
	A. First 25 Tests	25	25
	B. All Over 25 Tests	25	1

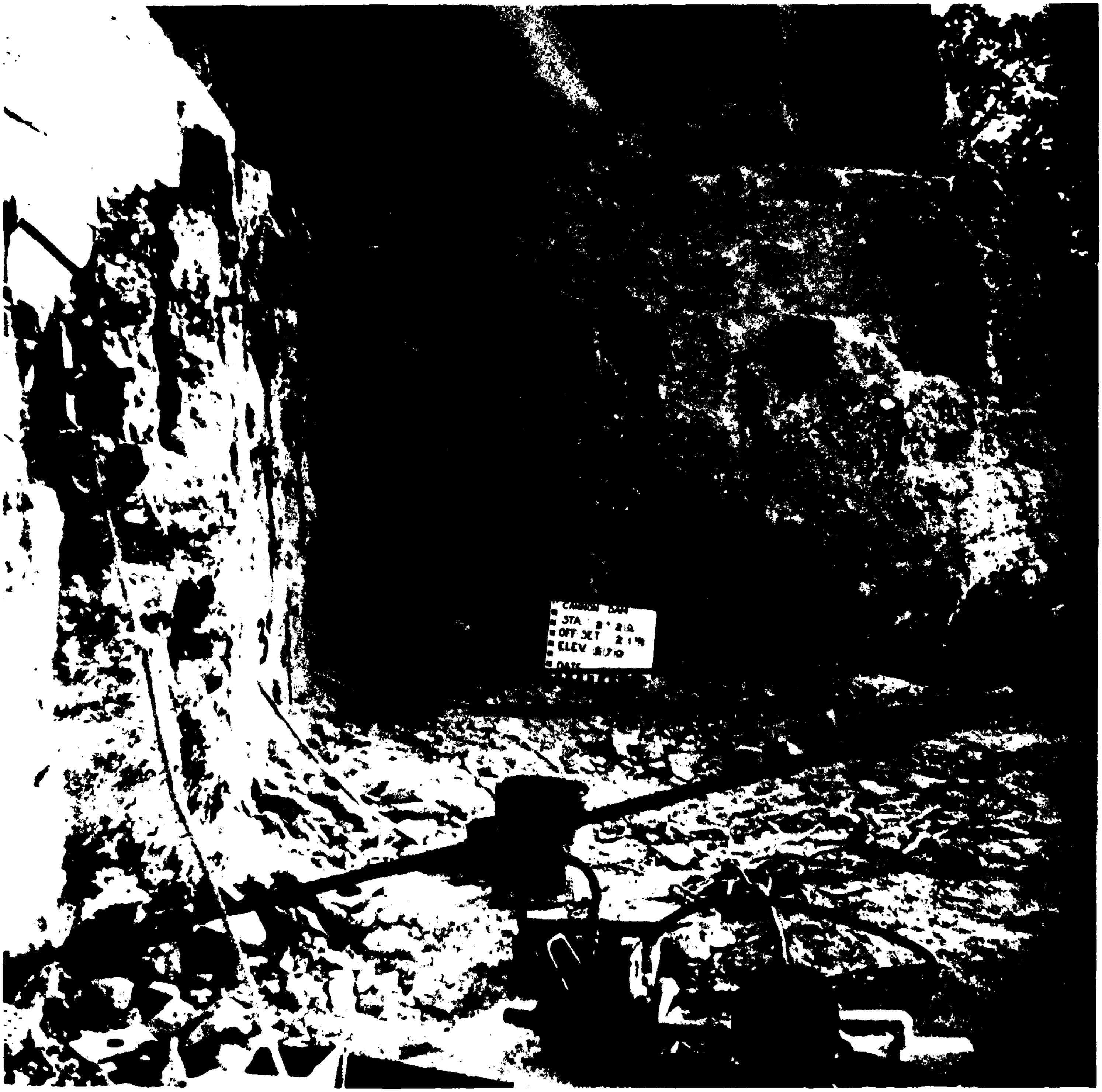
TABLE NO. 1

Typical rock bolt drilling operation on downstream foundation wall for Monolith D-15 with a 3700 Gardner Denver Air Track.

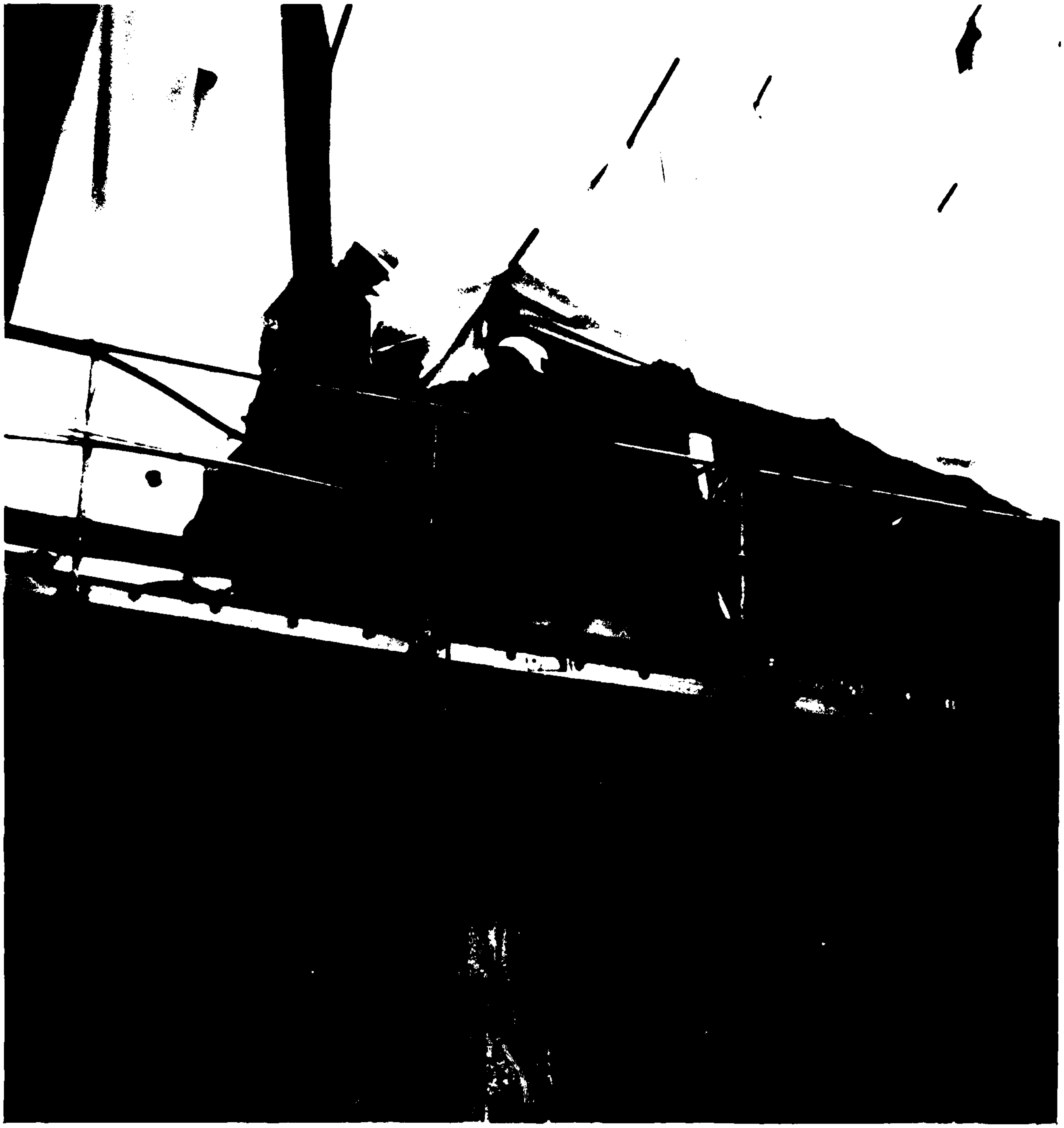


Laborer using a PVC pipe to push the Celtite resin to the back of the hole.

Rock bolt installation with a 3700 Gardner Denver Air Track.



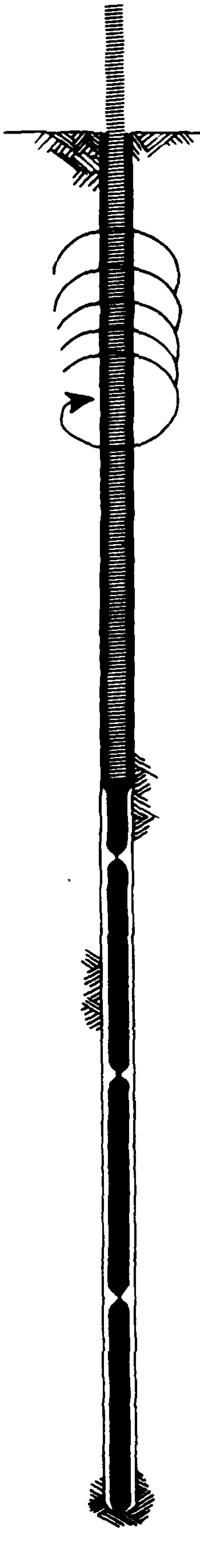
Typical Production Bolting Pattern in Monolith D-15 Foundation Walls

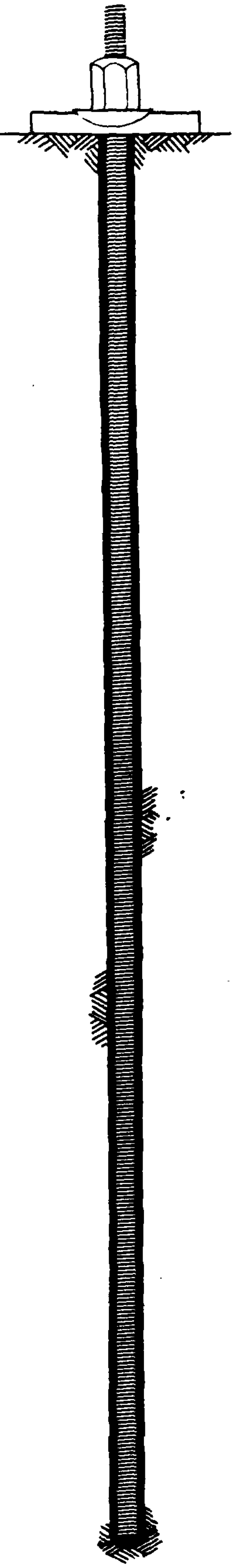


Stressing rock bolt with a 50-ton stressing jack thru back wall of Monolith D-13.

Installing rock anchors for safety curtain on right abutment upstream 1V:1H slope. Note laborer with tool in hand to tighten hexagonal nut.

Appendix A





REPORT OF ROCKBOLT FIELD TESTS CLARENCE CANNON PROJECT MISSOURI

by

John A. Albritton

TESTS OF "CELTITE/DYWIDAG" SYSTEM IN LIMESTONE AND SHALE

U.S. ARMY ENGINEER DISTRICT, ST. LOUIS
CORPS OF ENGINEERS
ST. LOUIS, MISSOURI

ROCKBOLT FIELD TESTS
CLARENCE CANNON PROJECT
MISSOURI

by

JOHN A. ALBRITTON

SEPTEMBER 1974

U. S. ARMY ENGINEER DISTRICT, ST. LOUIS
CORPS OF ENGINEERS
ST. LOUIS, MISSOURI

PREFACE

This report describes Contractor field tests of a rockbolting system utilizing continuously threaded bars anchored and grouted with polyester resins. As demonstrated at the Clarence Cannon project in both the tests and subsequent production work, the system is effective, economic, and simple to install. It was successfully tested and used in both limestone and shale formations.

The purpose of this report is to give dissemination to information concerning thread bar/resin rockbolting applications. The trade names used in the report for the resin and bars (i.e., "Celtite" and "Dywidag") refer to products actually used in the work, and does not imply endorsement of these over any competitive resins or thread bars.

The successful rockbolt and rock anchor tests and the accurate data records of the tests are the result of cooperative efforts of Contractor personnel, manufacturer's representatives, and Corps of Engineers personnel from St. Louis District, Lower Mississippi Valley, and Missouri River Divisions, and the Office of the Chief of Engineers.

Massman Construction Company is the prime contractor on the job, and Luhr Brothers, Inc., are subcontractors for the rockbolting work. Mr. Gary N. Greenfield of Celtite, Inc., and Mr. Eugene A. Lamberson of Inland-Ryerson Construction Products Company furnished useful technical advice and assistance concerning their products.

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ROCKBOLT FIELD TESTS
CLARENCE CANNON PROJECT
MISSOURI

by

JOHN A. ALBRITTON

SEPTEMBER 1974

U. S. ARMY ENGINEER DISTRICT, ST. LOUIS
CORPS OF ENGINEERS
ST. LOUIS, MISSOURI

I. SUMMARY.

Extensive use is being made of rockbolts and anchors at the Clarence Cannon Project in both limestone and weak shale formations. Difficulty was experienced in obtaining sufficient anchorage for required prestressing during preconstruction rockbolt testing in the Hannibal shale formation. The problems in that test program were evaluated as resulting from installation and drilling techniques in the case of grouted anchors and from insufficient strength of the shale in the case of mechanical anchors. A few 4-foot anchors were later installed encapsulated in polyester resin. This type of anchorage seemed to be promising; however, due to the insufficient amount of testing, rockbolt performance was specified rather than an end product. The Contractor elected to use "Dywidag" thread bars anchored and grouted with polyester resins. The system formed by the combined use of these two products exceeded design requirements by a wide margin. The test program provided a large amount of useful data which revealed the system to be simple, economical, versatile, and effective.

II. INTRODUCTION.

Limited rockbolt testing at the Cannon damsite prior to preparing plans and specifications for the main dam indicated that as much as 10 feet of bonded anchorage might be necessary in the Hannibal shale to assure sufficient anchorage for stressing the bolts. Prestressed pattern bolts are designed to reinforce this weak formation in the right abutment where vertical shale cuts and oversteepening of the abutment are necessary to accommodate the structures. Exploration and foundation investigations revealed the shale to be an air-sensitive, compaction shale, essentially unjointed except in the valley and near the abutment slopes, where extensive valley relief jointing is apparent. Horizontal borings into the abutment encountered numerous high angle joints parallel to and dipping toward the valley for about 25 feet. The shale was practically unjointed after the first 25 feet.

The prestressed pattern bolts are anchored, stressed, and fully grouted. Bolting is required from the top of the cut after each lift of excavation to compensate for the excavated material and to minimize the development of relief jointing. Unstressed anchors, or dowels, are used on high angle shale slopes in the valley, which exhibit well developed relief jointing. The shale is temporarily protected from drying out prior to installation of rock-bolts by spray applied bituminous material, and after the bolt or anchor installations by a 6-inch layer of shotcrete.

Rockbolts are also utilized in the right abutment for slope stability in the jointed Chouteau and Burlington limestone formations. Both prestressed bolts and unstressed dowels or anchors are used in the limestone, depending upon geologic conditions.

Since insufficient testing was completed prior to the preparation of plans and specifications for the main dam, performance was specified rather than particular rockbolting systems. A test program for rockbolts and anchors was specified in the contract to assure the Contractor's systems and installation techniques produced the required end results. The rockbolt and anchor systems which were tested and approved for production work consist of continuous thread bars (trade name "Dywidag"), anchored and grouted with polyester resin (trade name "Celtite"). Deformations of the Dywidag bar form threads over the entire length of bar. By installing fast-set resin in the bottom of the hole and slow-set resin in the remaining portion of the hole, a rockbolt can be installed, stressed, and fully grouted (with resin) in one operation. Stressing is accomplished after the fast resin sets but before the slow resin sets. The tests were conducted in both limestone and in some of the weakest shale at the damsite, i.e., a weathered, jointed valley section (FIG. 1 & 2). Results were very good, and a stronger than designed system was demonstrated. A total of 22 tests was conducted: four in limestone and 18 in shale.

Provisions were made to vary the system in badly jointed or cavernous limestone and in installations which are to be stressed and grouted at a later date. In these installations, anchorage is achieved in sound rock using the resin, but the remainder of the hole is grouted with cement grout after stressing. Cement grouting of the pattern bolts which are inclined downward 10° is accomplished through a ½" tube extended to the anchorage zone. The hole is drilled or reamed to a 3-inch diameter in the portion which is grouted with cement grout.

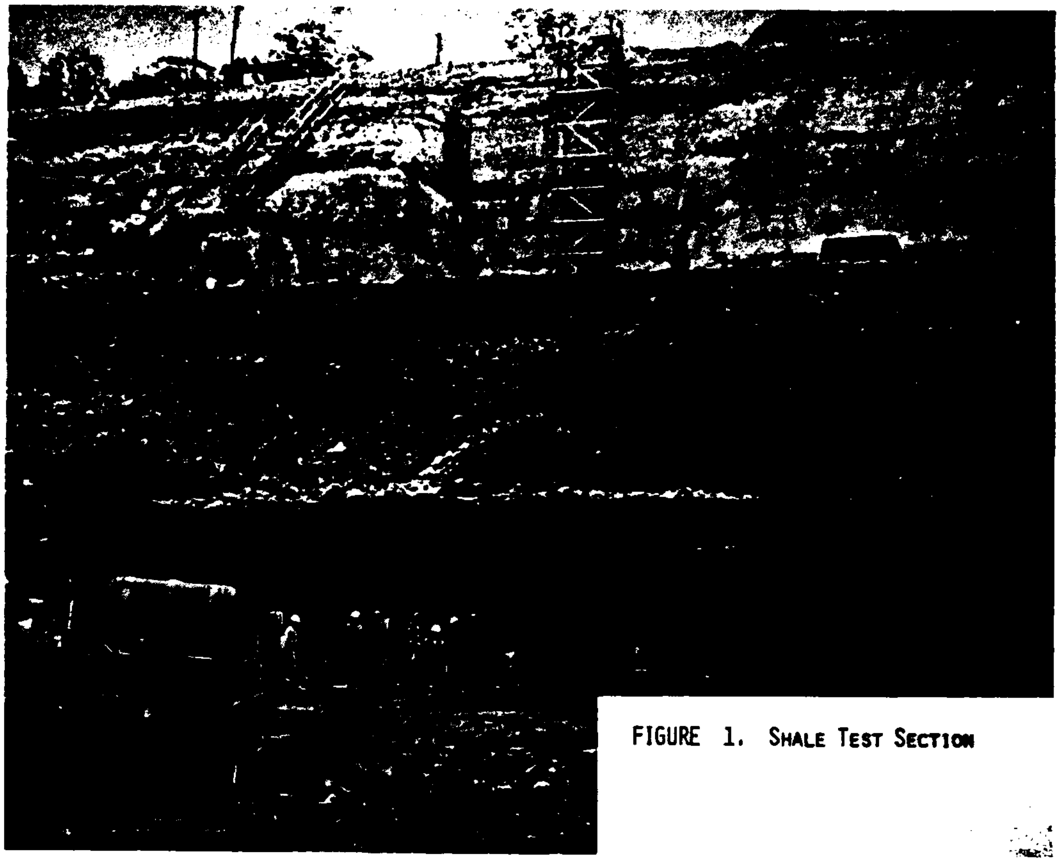

FIGURE 1. SHALE TEST SECTION

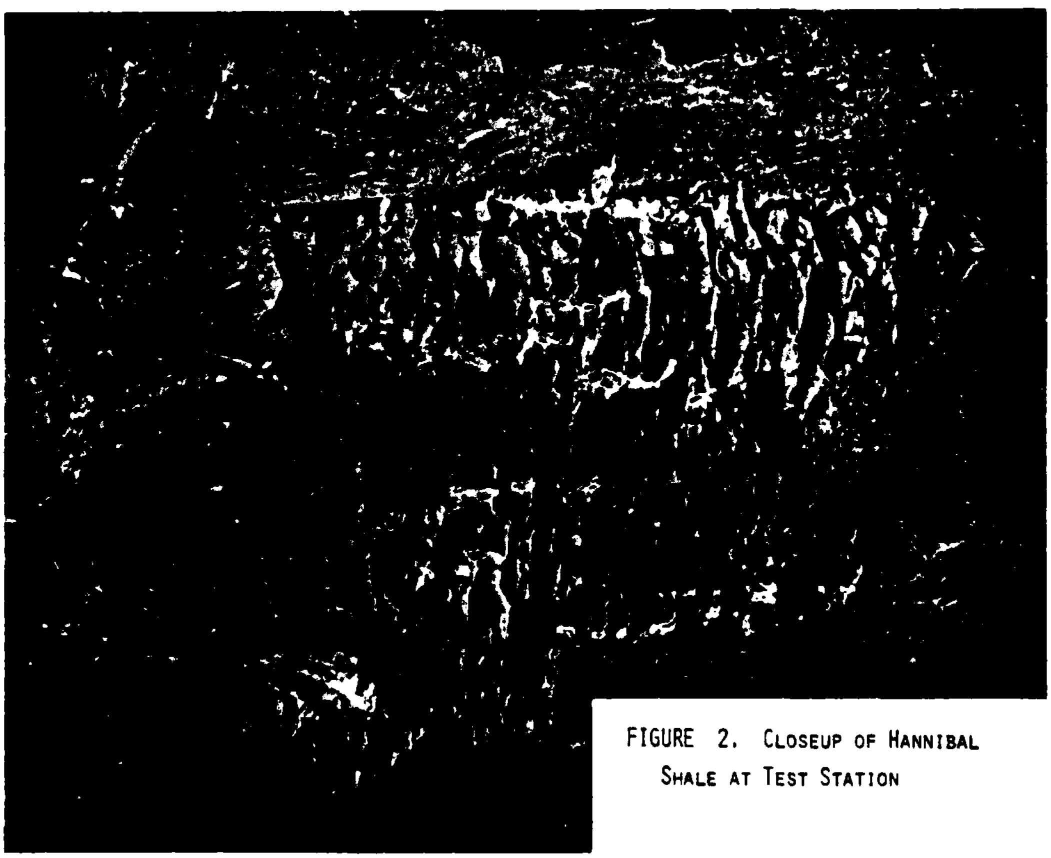

FIGURE 2. CLOSEUP OF HANNIBAL SHALE AT TEST STATION

III. PURPOSE.

The purpose of this report is to disseminate information to other Corps of Engineers offices on the rockbolting system used at Clarence Cannon Dam. The system holds considerable promise for wide application in the rockbolting field. It is hoped that the information included in this report will be useful as a guide for designing rockbolt and anchor systems for other projects. The "Dywidag"/"Celtite" system has proved to be economic, simple, and effective. It may be adapted to many rockbolting situations with possible cost savings.

IV. DESCRIPTION OF MATERIALS - ROCKBOLT AND ANCHOR SYSTEMS.

A. "Dywidag" Bar. The Dywidag thread bars are specially produced tendons developed for post tensioning applications. They are hot-rolled and proof stretched alloy steel, conforming to ASTM A-322 and ASTM A-29, with a rolled-in pattern of deformations along the entire length. These deformations serve as threads, permitting anchorage, coupling, or stressing hardware to be screwed onto the bar at any desired point without end preparation. Bar deformations are designed to give strong mechanical bond with surrounding grout or concrete. The bars are available either cut to length to meet specific job requirements or in stock lengths to be field cut. Stock lengths of No. 8 bars are used for all rockbolt and anchor requirements at the Cannon project. Characteristics of this bar are as follows:

Diameter	1.000	inch
Ultimate strength	150	ksi
Allowable stresses		
Ultimate	127.8	Kips
Temporary	102.2	Kips
Lock off	89.5	Kips
Maximum Effective	76.7	Kips
Cross sectional area	0.852	$in.^2$
Weight	2.960	lbs/ft
Elongation/100' at 102 Kips	4-7/8	inches
Young's modulus	$29.5x10^6$	psi

B. Bearing Plates and Nuts. A special nut designed for use with the thread bar has a conical seat and slotted construction to assure accurate centering and extra clamping action on the bar. Initially, bearing plates used at the Cannon project were 5" x 7" x 1½" for limestone and 8" x 8" x 1" for shale. Because of the low strength of the shale, a 14" x 14" x ½" bearing plate is presently required in the shale.

C. "Celtite" Resin. The resin cartridges consist of a reinforced polyester resin component together with its catalyst in a single sausage shaped package (FIG. 3) isolated from each other by a reacted interface. The cartridges come in a variety of sizes to fit different sized holes. Setting times can also be varied; and by using cartridges with different setting times, an installation can be anchored with resin and fully grouted with resin in one continuous operation. If required, stressing can be accomplished after the fast resin sets up but before the slow resin sets up. Setting times are based on a temperature of about 70°F. They vary with temperature, and are substantially slower at extreme cold temperatures; however, the relative time differences between fast and slow resins remain the same. The resin manufacturer's representative should be consulted concerning setting times for use in extremely cold weather. Care is recommended during hot weather to avoid exposing resin or bars to direct sun, since high temperatures would accelerate setting times. Properties of the cured "Celtite" resin include the following:

Unconfined compressive strength:	16,000 psi
Shear strength:	7,500 psi
Tensile strength:	2,500 psi
Young's modulus	0.3844×10^6 psi

Resin rockbolt and anchor systems are reported to compare favorably with mechanical or cement grout systems in regard to resistance to weathering, aging, vibration, shrinkage, and creep.

V. TEST EQUIPMENT.

Equipment used in the testing included center-hole, hydraulic jacks with hand and electrically operated pumps, jack chairs, wrench with 4-foot cheater bar for load transfer, "Terrametric" load cells and readout box, Ohio Brass Company, 40-foot hole gage (1¼" to 1-5/8"), Ohio Brass Company, 36-foot hole gage (1-5/8" to 2¼"), 3-inch travel extensometer dial gages, air-track drills, and special spin-in equipment mounted on an articulated end loader for installing long bolts. See FIGURES 4-22 for photographs of rockbolt installation and test equipment.

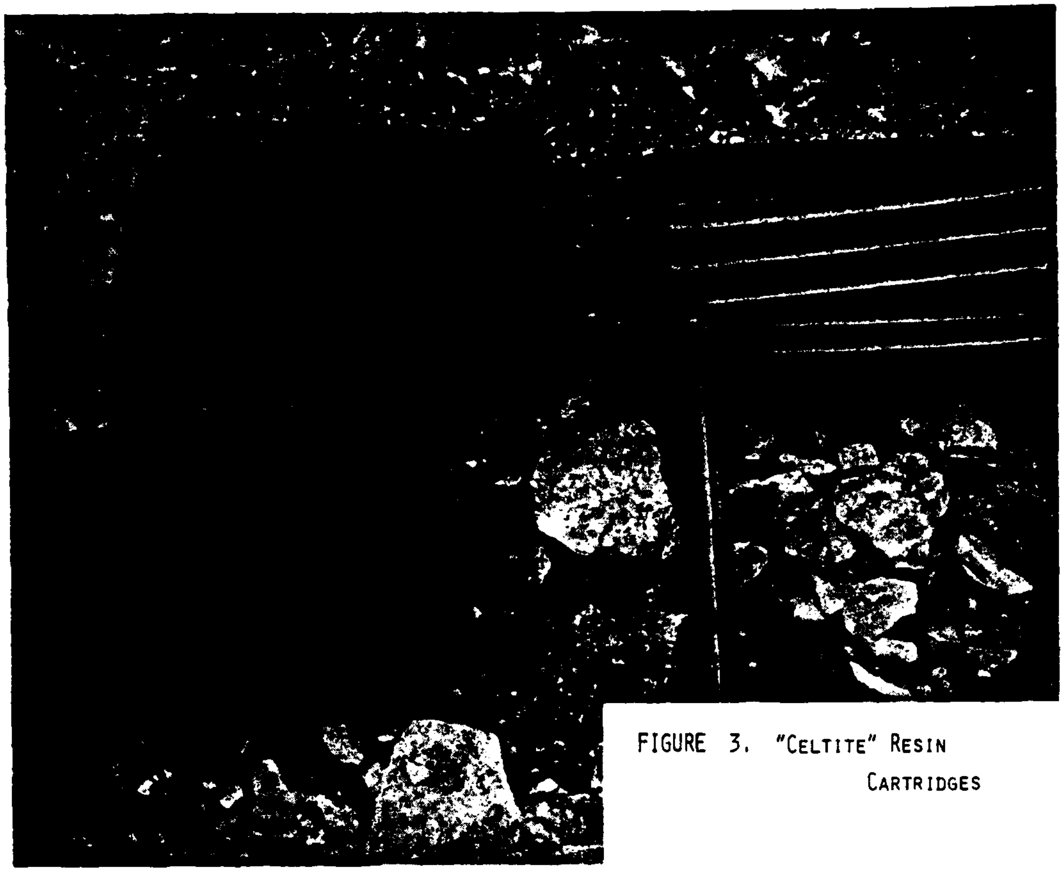

FIGURE 3. "CELTITE" RESIN CARTRIDGES

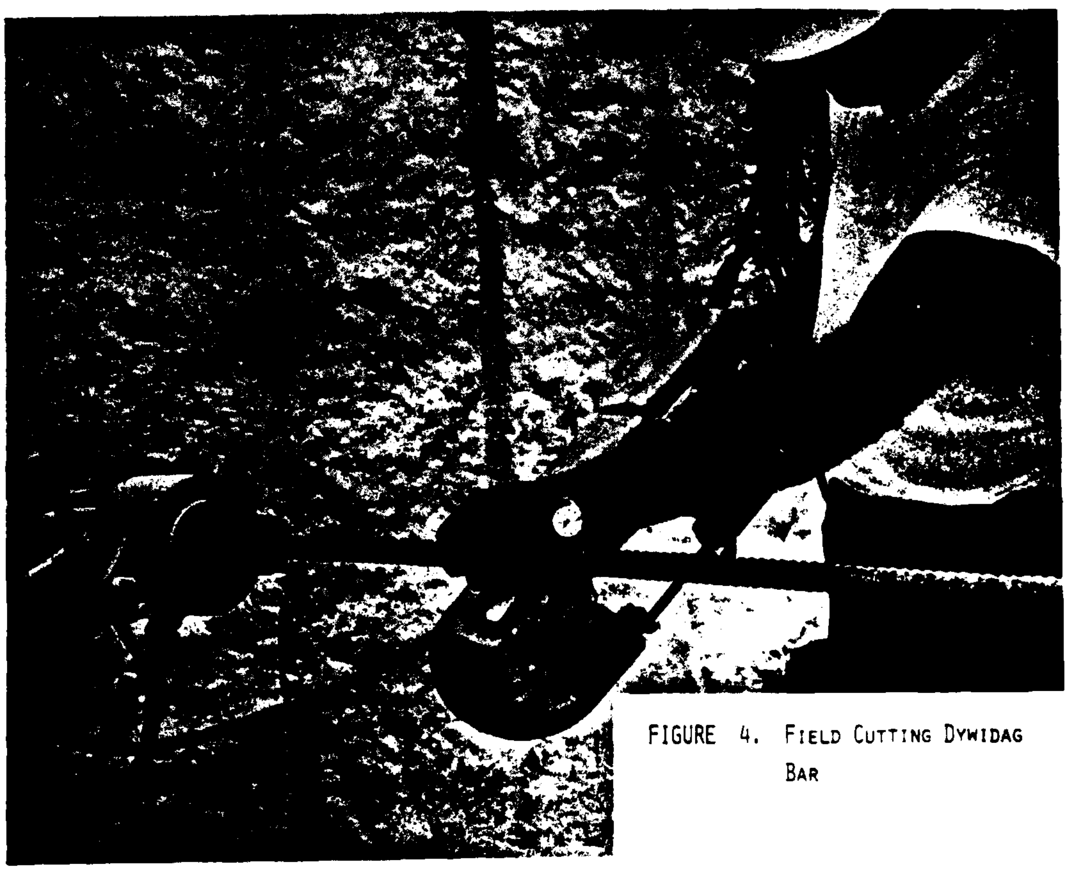

FIGURE 4. FIELD CUTTING DYWIDAG BAR

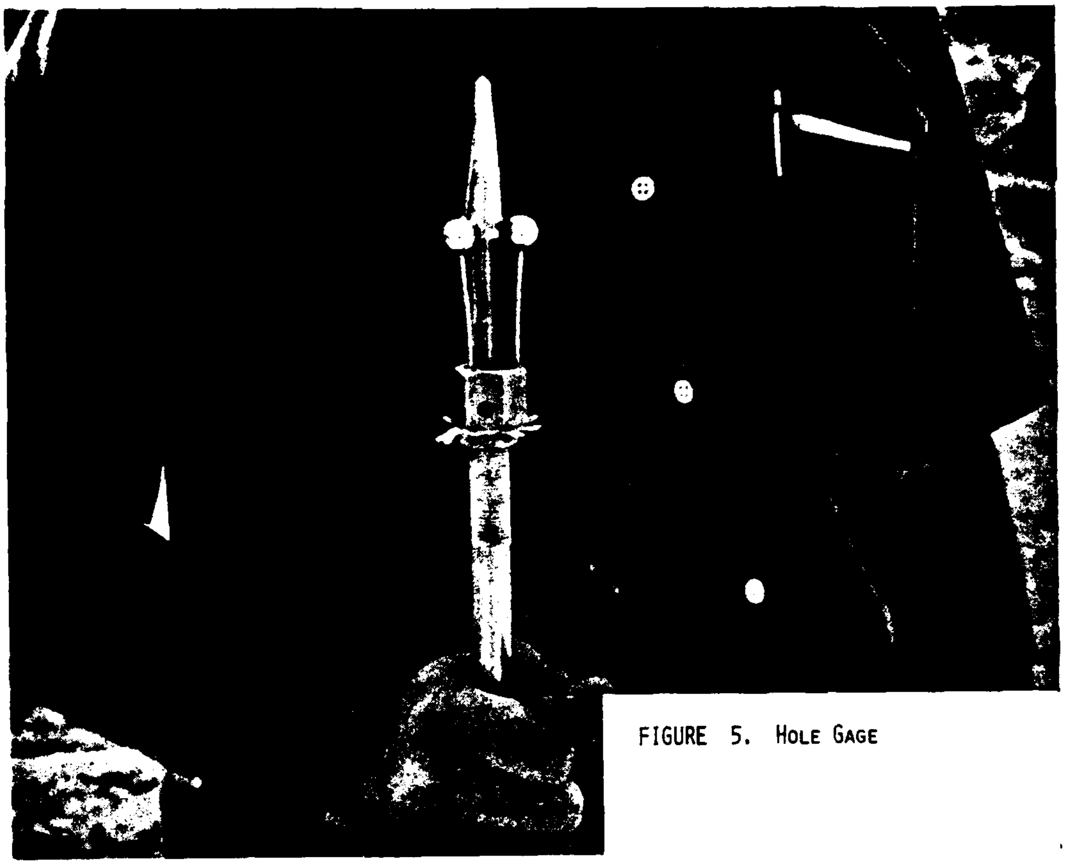

FIGURE 5. HOLE GAGE

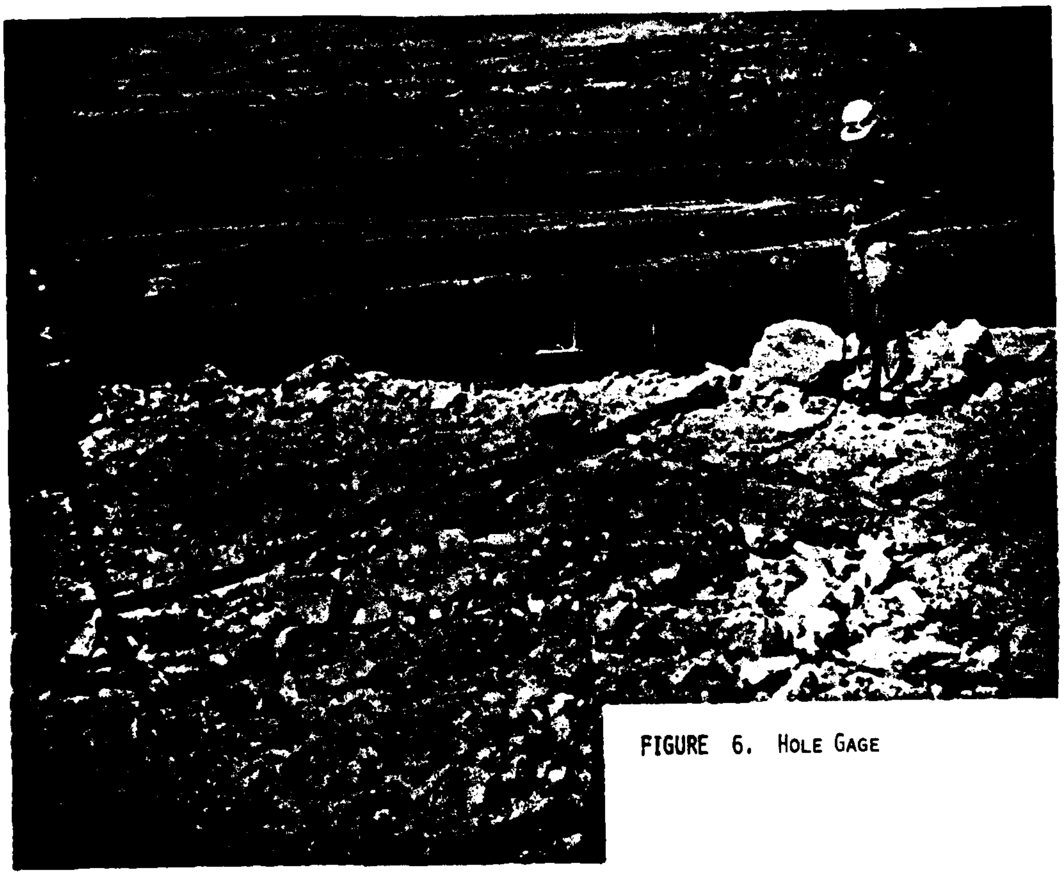

FIGURE 6. HOLE GAGE

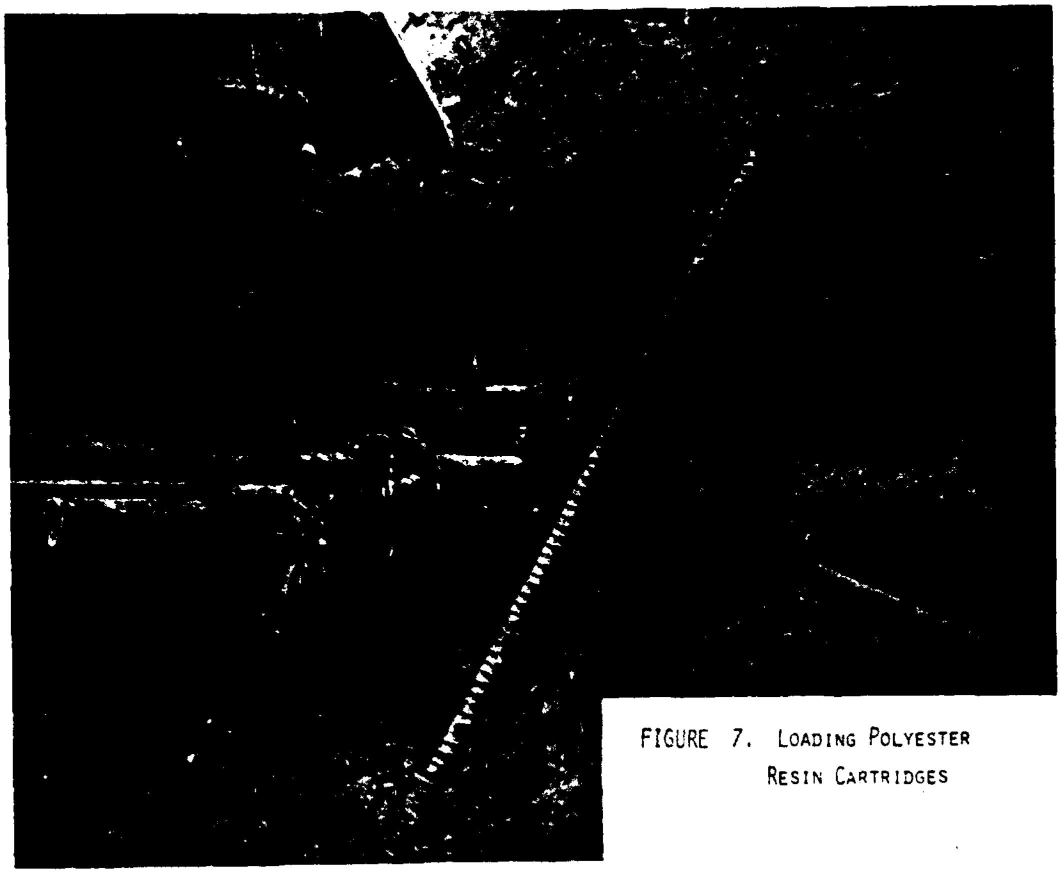

FIGURE 7. LOADING POLYESTER RESIN CARTRIDGES

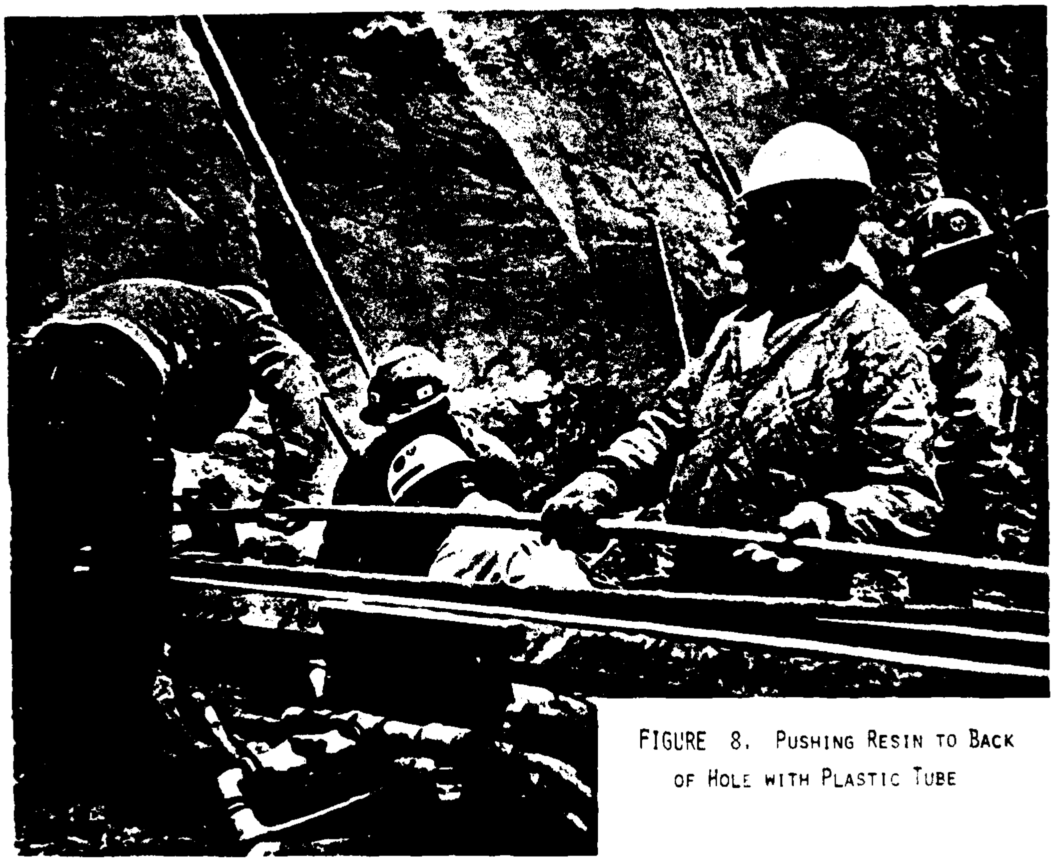

FIGURE 8. PUSHING RESIN TO BACK OF HOLE WITH PLASTIC TUBE

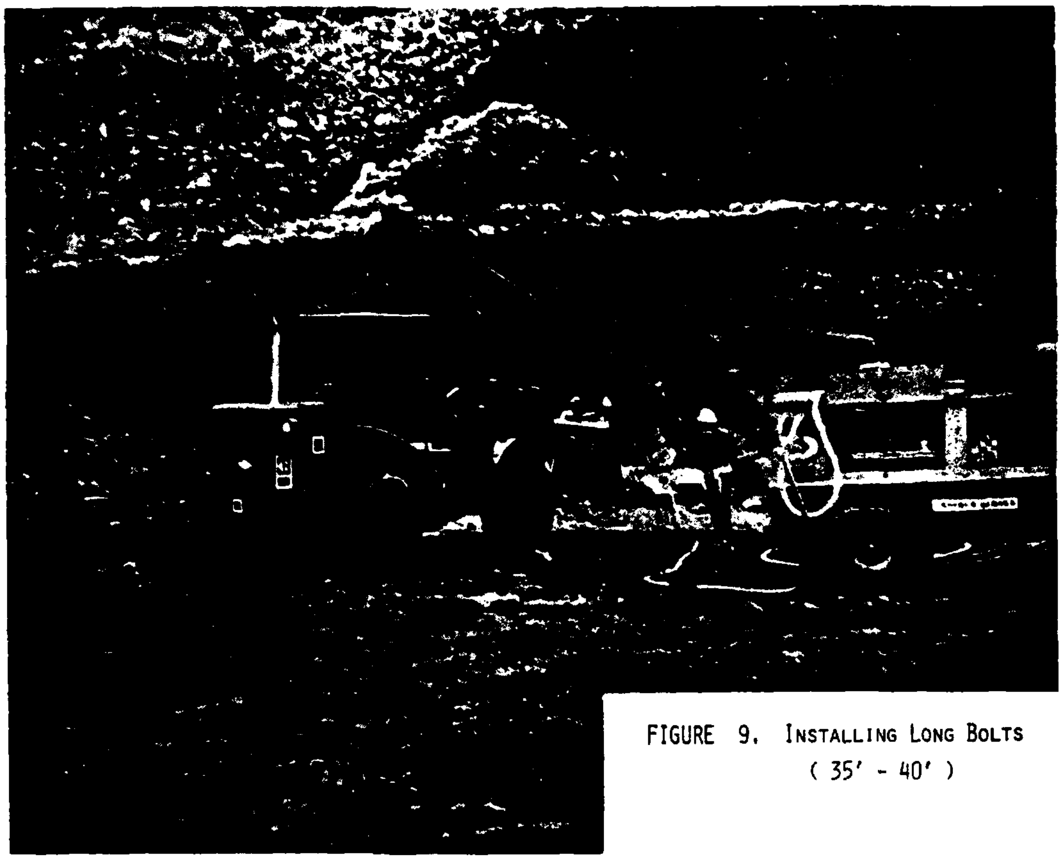

FIGURE 9. INSTALLING LONG BOLTS
(35′ - 40′)

FIGURE 10. INSTALLING LONG BOLTS

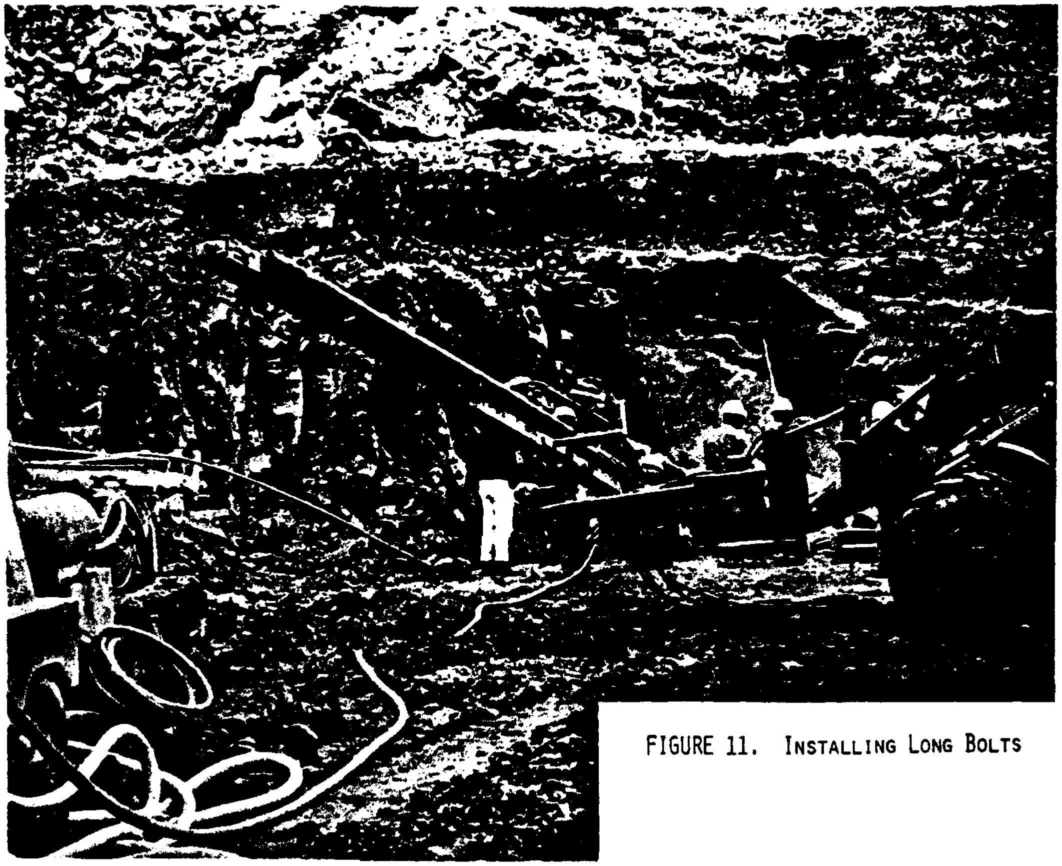

FIGURE 11. Installing Long Bolts

FIGURE 12. Installing Long Bolts

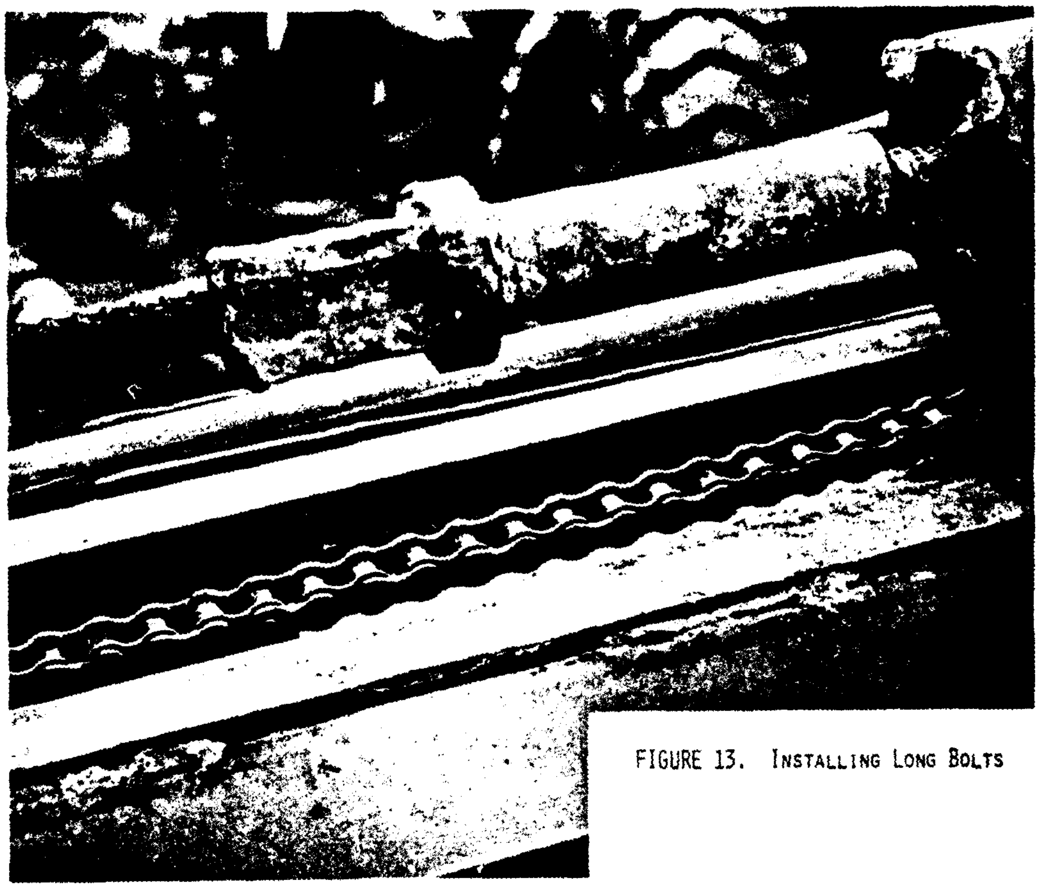

FIGURE 13. INSTALLING LONG BOLTS

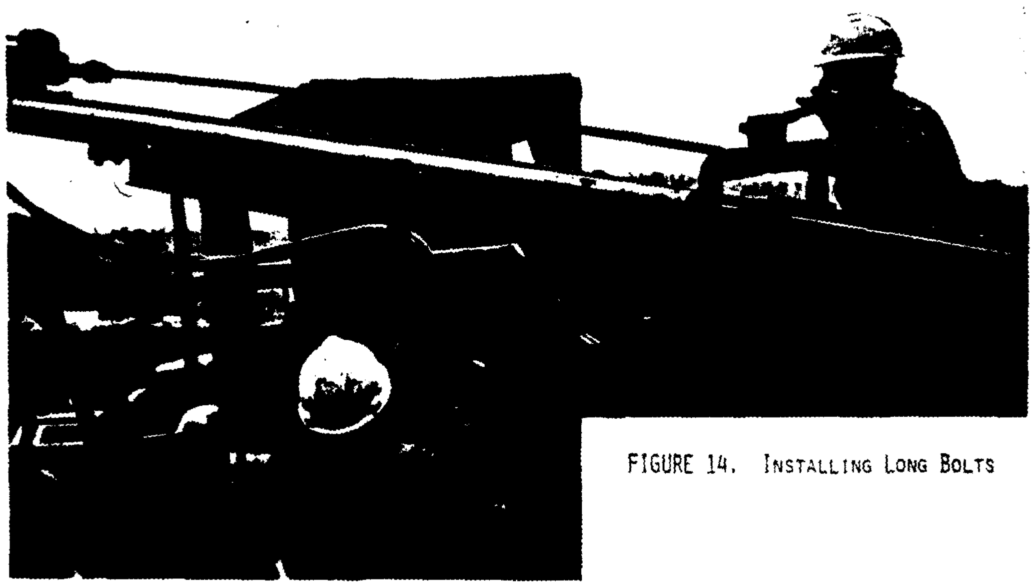

FIGURE 14. INSTALLING LONG BOLTS

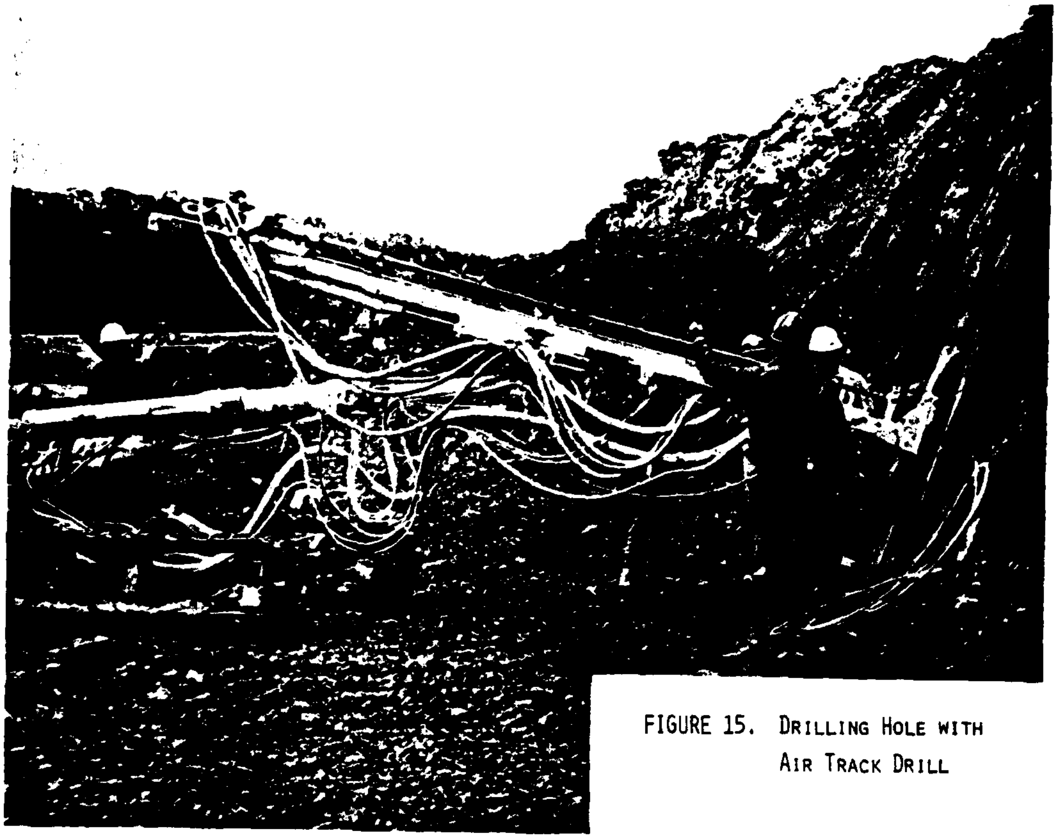

FIGURE 15. Drilling Hole with Air Track Drill

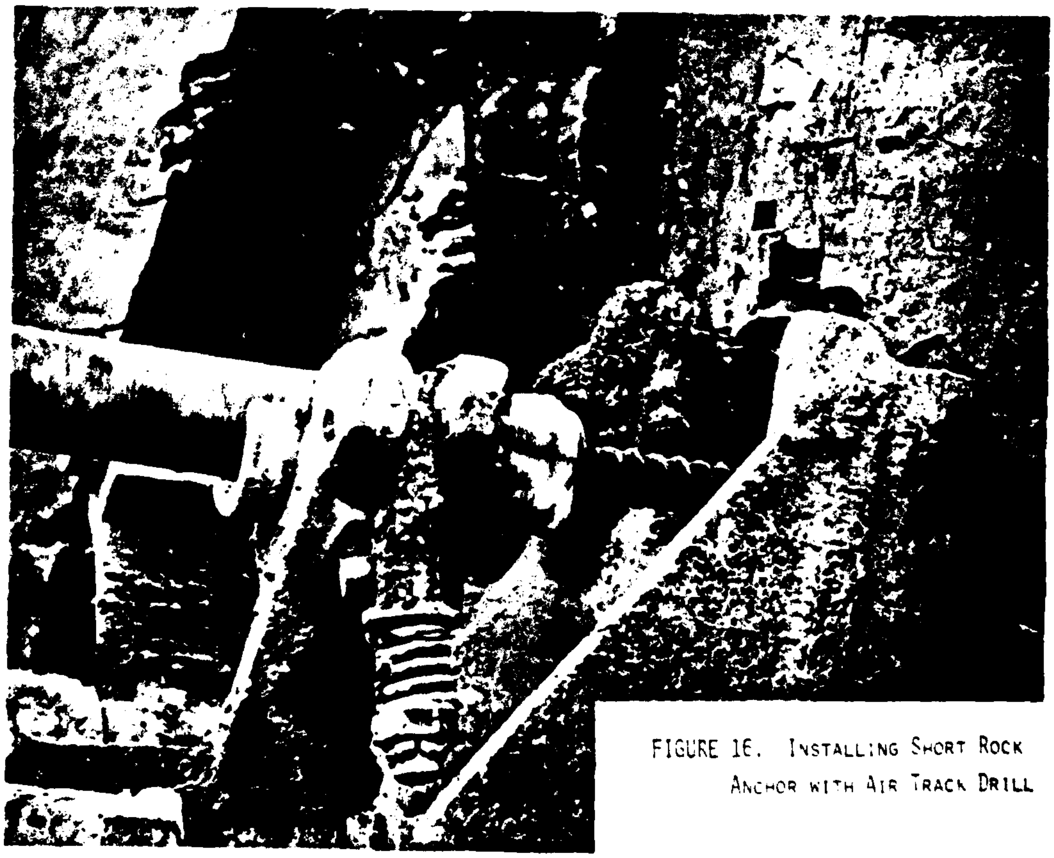

FIGURE 16. Installing Short Rock Anchor with Air Track Drill

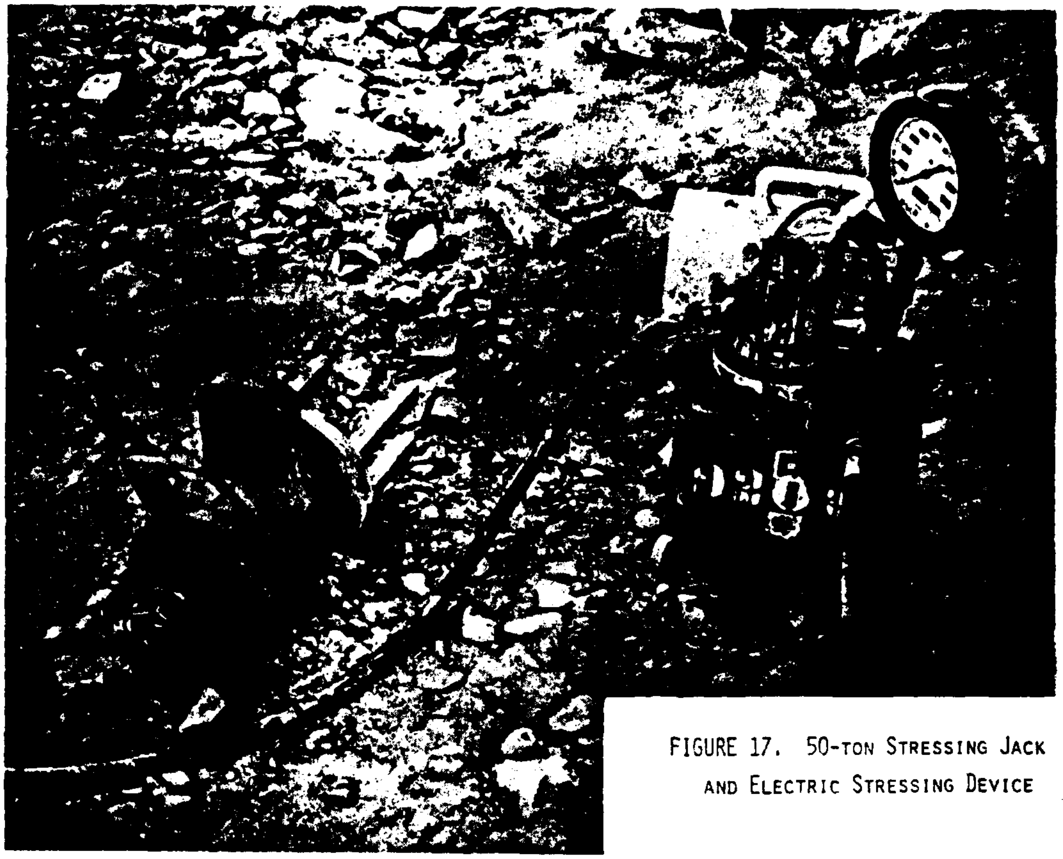

FIGURE 17. 50-TON STRESSING JACK AND ELECTRIC STRESSING DEVICE

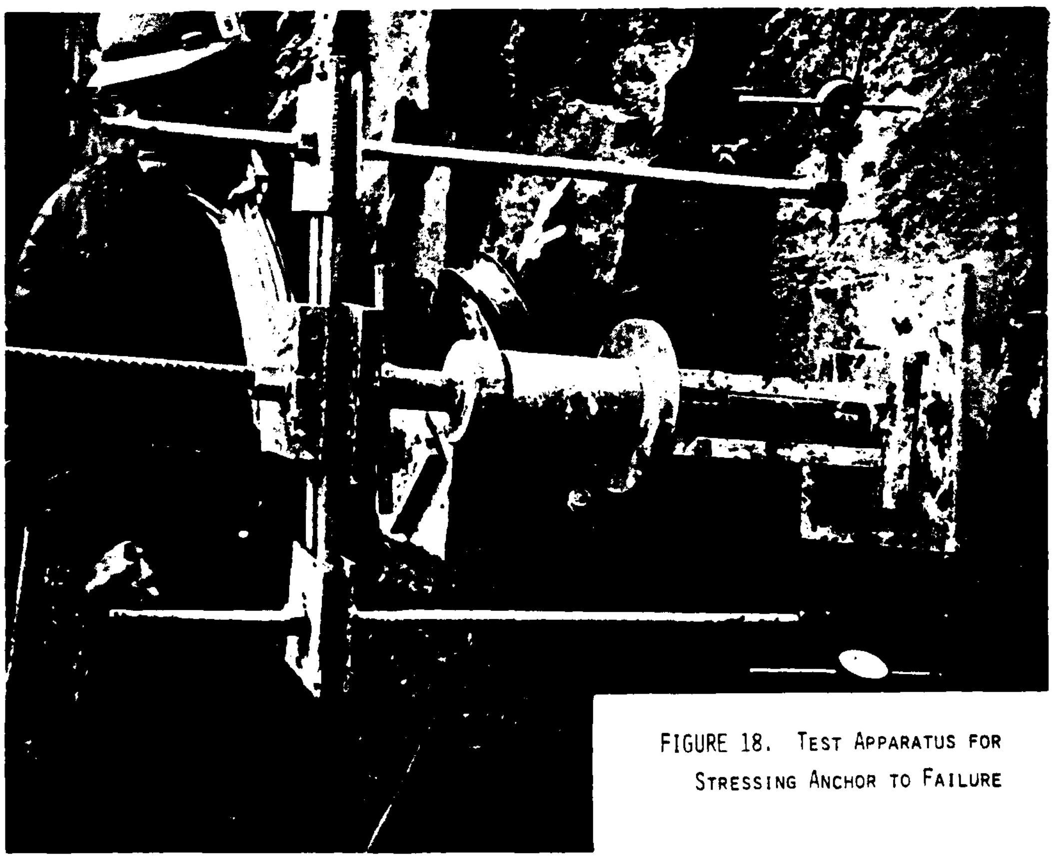

FIGURE 18. TEST APPARATUS FOR STRESSING ANCHOR TO FAILURE

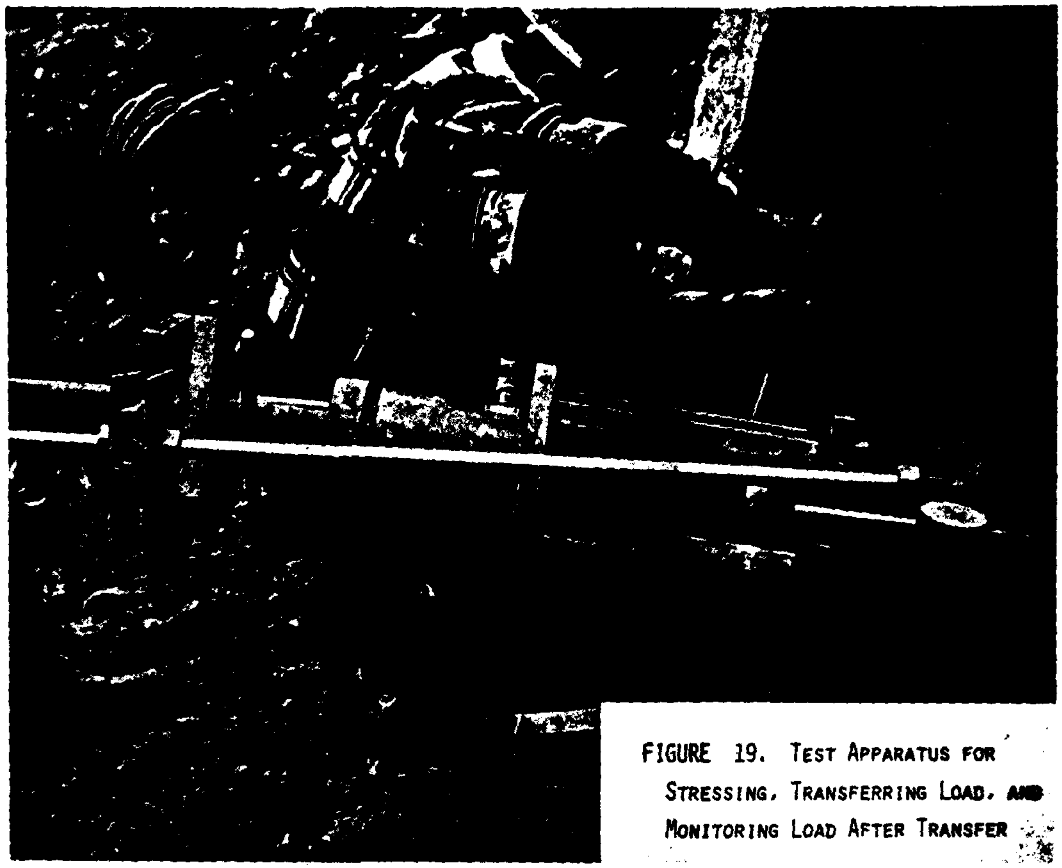

FIGURE 19. TEST APPARATUS FOR STRESSING, TRANSFERRING LOAD, AND MONITORING LOAD AFTER TRANSFER

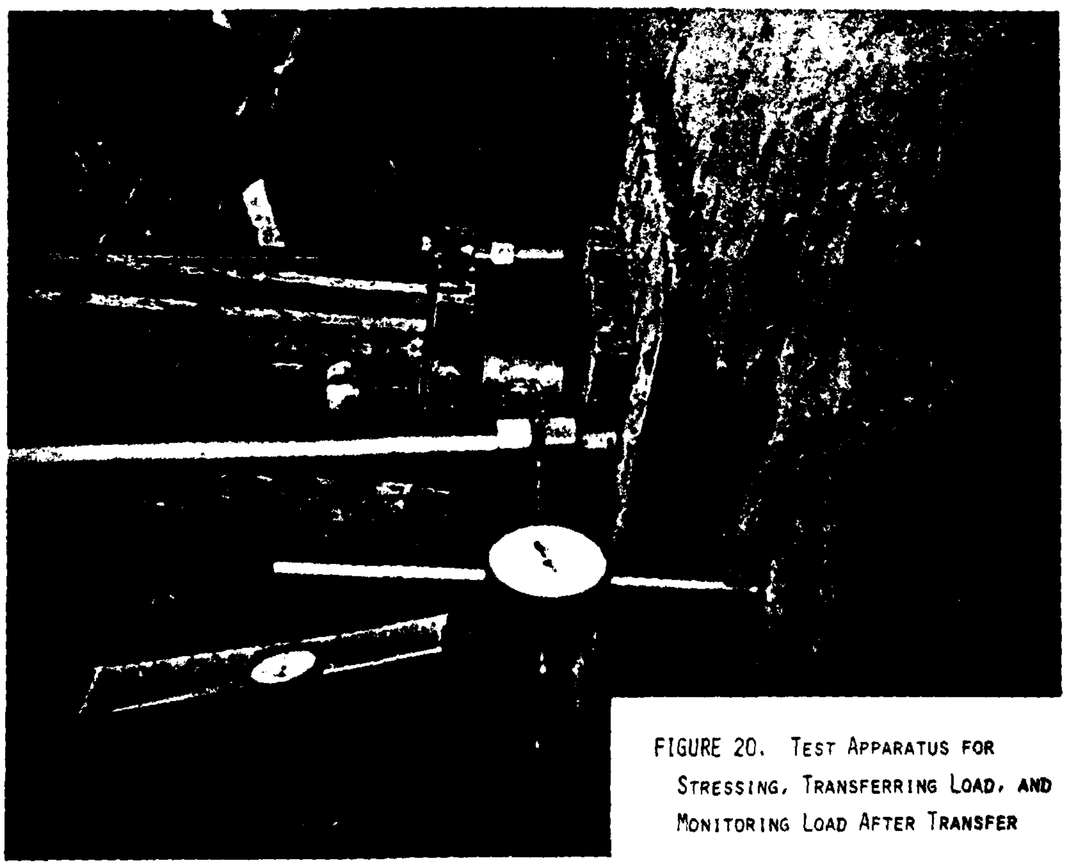

FIGURE 20. TEST APPARATUS FOR STRESSING, TRANSFERRING LOAD, AND MONITORING LOAD AFTER TRANSFER

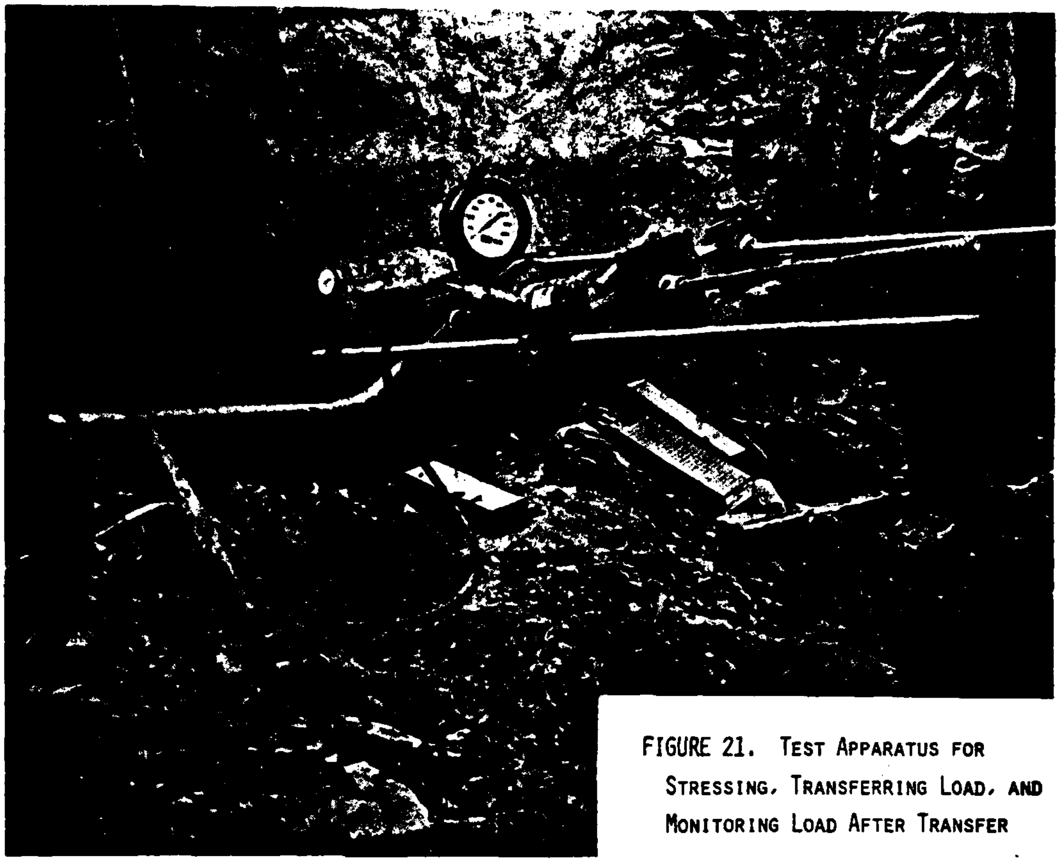

FIGURE 21. Test Apparatus for Stressing, Transferring Load, and Monitoring Load After Transfer

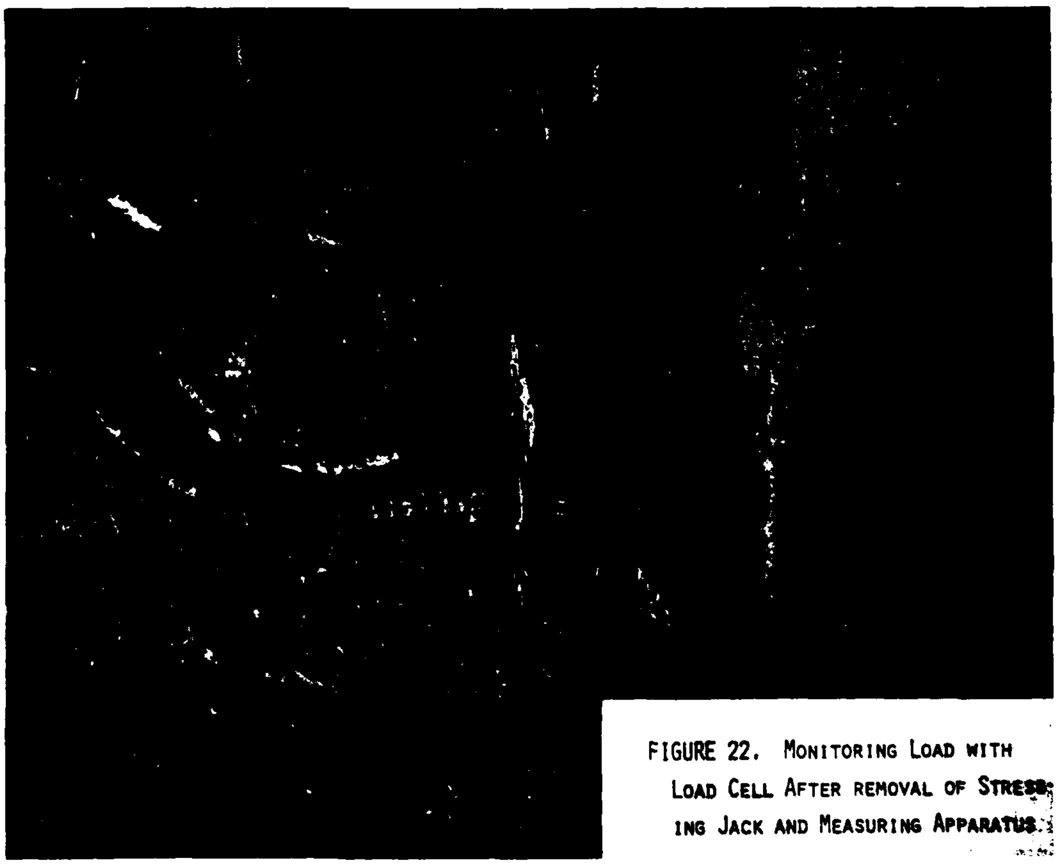

FIGURE 22. Monitoring Load with Load Cell After removal of Stressing Jack and Measuring Apparatus.

VI. GEOLOGY AND ROCK PROPERTIES.

A. General Geology. The site of the Clarence Cannon Dam is at Mile 63.0 on the Salt River in northeastern Missouri, approximately 20 miles southwest of Hannibal, Missouri. The Clarence Cannon Dam and Reservoir Project is located in the Dissected Till Plains Section of the Central Lowland Physiographic Province. The damsite is in a glaciated, maturely eroded plain, preserving only occasional remnants of the original glacial features. Glacial features significant to the project design include a till-filled saddle in the left abutment upland and a buried channel in the valley beneath the embankment section of the main dam. The channel was scoured through some 50 feet of shale to the underlying limestone. Fill in the buried channel consisted of sands and gravels with large boulders at the bottom. Side slopes in the shale were nearly vertical. Paleozoic rocks underlie the project area, varying in age from Ordovician to Pennsylvanian with the Ordovician rocks exposed in the lower reaches of the Salt River. They are overlain by Siluvian or Devonian formations and at the damsite by unassigned Mississippian-Devonian rocks. Mississippian formations crop out extensively in the reservoir area and at the damsite. Sinks filled with Pennsylvanian sediments and Pennsylvanian outliers of clay, shale, sandstone, and coal are found on the uplands. The Lincoln Fold is the nearest major structure to the project and lies a short distance to the southeast. The effects of this northward plunging anticline on rocks at the damsite is slight, and the bedding is practically horizontal. No faults are known in the immediate area and joint patterns are fairly well defined in the formations at the damsite. The pattern most significant to the rockbolt design is the valley relief system which is well developed in the valley shale but rapidly disappears about 25 feet into the right abutment where loading has not been relieved.

B. Stratigraphy and Lithology.

1. General. The stratigraphic succession of bedrock formations at the damsite in descending order are the Mississipian Burlington and Chouteau limestones, Hannibal shale and Louisiana limestone. Pennsylvanian sandstone boulders occur as float on the uplands of the right abutment and Pennsylvanian sediments occur in filled sinks in the Burlington and Chouteau formations. Upland soils are a mixutre of Pennsylvanian or Mississippian residual materials and thin glacial deposits. At the damsite,

a thin topsoil layer of silt and clay overlies a considerable thickness of residual chert which lies upon and grades into the Burlington limestone. Valley soils are comprised of channel fill and sand, talus, colluvial and alluvial flood plain deposits.

2. Burlington-Chouteau Limestone. The youngest and highest bedrock formations exposed at the damsite are the Mississippian Burlington and Chouteau limestones. The upper section of the Burlington is a coarsely crystalline, cherty limestone with the chert occurring in nodular form and in beds ranging in thickness from 0.2 to 0.9 foot. The lower zone consists of some 20 feet of massively-bedded, very coarsely crystalline, chert-free limestone. A maximum of 85 feet of the Burlington is exposed, and the contact with the underlying Chouteau Formation occurs at approximate elevation 600. Thin-bedded (0.1 to 0.5 foot), gray limestones that weather to an earthy texture characterize the approximately 50 feet of Chouteau exposed at the damsite. Considerably less chert occurs in the Chouteau than in the Burlington, but a varied amount of argillaceous material is present in the matrix and on bedding planes. The contact with the underlying Hannibal occurs at approximate elevation 550.

3. Hannibal Shale. At the damsite, the general thickness of the Hannibal shale is 80-85 feet. It is a dark, moderately hard, sublaminated shale, containing some siltstone lenses and numerous pyrite concretions. The siltstone layers are slightly calcareous and are less air-sensitive than the shale mass. Zones of the shale are extremely sensitive, and will begin to crack within 30 minutes of air exposure and completely crumble when desicated, or will disassociate upon immersion in water. The contact with the underlying Louisiana occurs at approximate elevation 470. Underlying the alluvial deposits of the flood plain, the Hannibal shale has been weathered to a depth of from 3 to 8 feet, and has physical properties resembling those of a medium clay (CL). There is a gradual transition into the firm shale, and the weathered section frequently contains fragments of unweathered shale.

4. Louisiana Limestone. Approximately 70 feet of Louisiana limestone unconformably underlies the Hannibal shale at the damsite. Here it may be divided into three localized zones: the upper limestone, the dolomitic zone, and the lower limestone. The upper limestone is a light gray, moderately hard rock, about 3 feet in thickness containing a rather persistent shale layer at its base. The dolomite, of varying magnesium carbonate content, ranges from undulating thin beds to thick beds

of hard dolomite. Much of the dolomite is sandy textured and contains up to 25 percent quartz sand. The lower limestone, which constitutes the majority of the formation, is a very light gray, fine-grained limestone, medium to massively bedded.

C. Engineering Properties of Rock Formations - Design Assumptions.

1. Burlington-Chouteau Formations.

a. Engineering Properties. Although the Burlington and Chouteau limestones differ considerably in many of their geologic properties, their engineering behavior is sufficiently strong as to present no problems, in bearing capacity or sliding stability, to monoliths founded upon them. During early design studies, some concern was expressed over the possibility of sliding failure occurring along the argillaceous or shaly zones within the Chouteau Formation. A sufficient number of tests were performed to conclude that these shaly zones do not, in fact, constitute planes of significant weakness within the formation. Joints and fractures constitute the principal zones of weakness in these formations.

b. Design Assumptions.

Property	Burlington Fm.	Chouteau Fm.
Unit weight	155 pcf	155 pcf
Modulus of elasticity	6.6×10^6 psi	3.6×10^6 psi
Unconfined compressive strength	9,500 psi	9,500 psi

Shear Strength:

Peak	c = 16.2 TSF $\phi = 45^\circ$	c = 16.2 TSF $\phi = 45^\circ$
Residual	c = 3.0 TSF $\phi = 45^\circ$	c = 3.0 TSF $\phi = 45^\circ$
Concrete-Rock	c = 16.2 TSF $\phi = 45^\circ$	c = 16.2 TSF $\phi = 45^\circ$

2. Hannibal Shale.

a. Engineering Properties. The single, most critical factor in the design of Clarence Cannon Dam has been the Hannibal shale. In addition to its low strength, the air-sensitivity of the shale and the existence of soft zones within it have exercised profound influence on the design of the dam. The air-sensitivity of this shale varies widely within the limits of the project area. Several studies have been made in an attempt to recognize some system in the occurrence of this phenomenon, but no clear pattern has emerged. Isolated samples of shale have disintegrated completely after one cycle of wetting and drying, while others are relatively inert to moisture fluctuations. Fractures occur in the valley shale at almost all angles with the majority of fractures occurring at angles of more than 45 degrees from the horizontal. The fractures are frequently filled with calcite. In the abutments, fractures are most common only in the upper 5 to 10 feet of the shale, and are scarce in the remainder of the formation. Several studies have been made in attempts to correlate soft zones within the Hannibal shale. Three distinct zones of soft shale can be detected. The first occurs at the top of rock, and is synonymous with the weathered shale zone in the valley. In the abutments, a second zone of softening occurs near the Chouteau-Hannibal contact, and is apparently caused by ground water migration occurring at the contact; this zone is limited to the area near the abutment face. The third zone occurs in the lowest 10 feet of the shale, and is apparently a reflection of both original lithology and subsequent weathering. In the valley, the shales have been softened by water migrating within the upper 5 feet of the Louisiana Formation. In the valley, shales in the lower 10 feet have compressive strengths as low as 140 psi. The shales in the abutments have been more protected. The upper shales in the abutment have compressive strengths around 1,000 psi, while the lower shales fracture at approximately 600 psi. No other persistent zones of soft shale have been detected.

b. Design Assumptions.

Unit weight: 150 pcf

Modulus of elasticity:

Abutment: 200,000 psi
Valley: 27,000 psi

Unconfined compressive strength: 140 psi

Shear strength:

Peak Undrained:	c = 4.5 TSF, $\phi = 26^\circ$
Residual Undrained:	c = 1.6 TSF, $\phi = 20^\circ$
Peak Drained:	c = 1.4 TSF, $\phi = 19^\circ$
Residual Drained:	c = 0.0, $\phi = 19^\circ$
Concrete-Shale:	c = 0.0, $\phi = 19^\circ$
Anchor Grout-Shale:	c = 2.25 TSF

3. Louisiana Limestone.

a. Engineering Properties. Laboratory testing has indicated that the Louisiana Formation is a competent foundation member and that the numerous shale partings, being extremely irregular, do not constitute any real plane of weakness within the formation. However, near the top of the formation a persistent zone of shale or shaly limestone occurs which varies sufficiently in its properties as to cause some concern. Limited testing indicated this shale zone is relatively sound. Although the Louisiana Formation is highly solutioned in other areas, there is little evidence of solution, even along fractures, at the damsite.

b. Design Assumptions.

Unit weight: 155 pcf

Unconfined compressive strength: 9,500 psi

Modulus of elasticity: 3.0×10^6 psi

Shear strength:

Peak:	c = 16.2 TSF, $\phi = 45^\circ$
Concrete to rock:	c = 16.2 TSF, $\phi = 45^\circ$
Residual:	c = 3.0 TSF, $\phi = 45^\circ$

VII. DESCRIPTION OF ROCK REINFORCEMENT AND PROTECTIVE MEASURES.

The main structures of the Clarence Cannon Dam will consist of an embankment section tying the left abutment to the concrete dam, and the concrete dam consisting of a left non-overflow section, powerhouse section, spillway section, and right non-overflow section. Concrete retaining/training walls will be provided on the right side of the spillway and left side of the powerhouse tailrace with a splitter wall between the two. Except for the right non-overflow, the main dam will be founded on the Louisiana limestone. Most of the retaining wall and splitter wall monoliths will be founded on valley shales. The right non-overflow section will have a large monolith founded on the Louisiana limestone and the remaining monoliths founded successively higher in the right abutment with three on the Hannibal shale, one on the Chouteau limestone and one on the Burlington limestone. Slopes contiguous to the abutment monoliths will be essentially vertical. The vertical steps between monoliths will be as much as 35 feet in the shale and 60 feet in the limestone. Other excavation slopes include 1 on 1 slopes for most of the right abutment downstream of the main dam where oversteepening of the abutment is necessary for the stilling basin retaining wall foundation and 4V on 1H slopes for valley shales contiguous to the concrete structures. Some of the upper valley slopes will be excavated to 1 on 1 above a 10-foot berm. These slopes will be left unprotected. Most of the shale founded monoliths will require passive wedges to meet sliding stability requirements. Excavation of the limestone is primarily by presplitting and primary blasting; however, shale excavation is by ripping, sawing, and line drilling.

Because of its air sensitivity, the Hannibal shale is normally protected from the air immediately upon exposure (one-hour time limit). In addition, the critical right abutment shale foundations and slopes are protected from rebound, differential stresses, freezing, and damaging excavation techniques such as blasting. Both limestone and shale, high-angle slopes are reinforced with prestressed rockbolts and unstressed rock anchors depending on geologic conditions. Approximately 50,000 linear feet of pattern rockbolts and anchors are required for the project in addition to a less specific but significant amount of safety bolts and anchors installed as needed.

Initial shale protection applied within one hour after exposure consists of spray applied bituminous material ("sika seal") on high-angle sawed or line drilled slopes or a 6-inch layer of protective concrete on horizontal foundations. Rockbolts or anchors are then installed on high-angle slopes, followed by 6 inches of shotcrete with drains installed on a maximum spacing of 10-foot centers. Approximately 100,000 square feet of shotcrete is estimated for the project. In addition to shotcreting all high-angle shale slopes, the Chouteau limestone is also shotcreted for slope stability and safety as well as protection of the underlying shale.

Generally speaking, prestressed rockbolts are required in the abutment shale after each step of excavation to compensate for unloading and prevent or minimize the development of relief jointing. In the valley where valley relief jointing is already well developed, unstressed anchors are used in the shale for slope stability.

In addition to the protection described above, the three right abutment main dam monolith shale foundations and vertical slopes will receive the following protection and reinforcement. Initial 2-1/2-foot reinforced concrete lifts for the monoliths will be placed over the 6-inch layer of protective concrete and stressed to the foundation with rockbolts. Following this, a 2-foot reinforced concrete wall will be constructed next to the shotcreted vertical slopes and stressed to the shale with rockbolts. Winter protection of spray-applied urethane foam insulation will then be applied to the wall and initial lift of the monolith surfaces. The specified thermal conductance value of no greater than 0.10 BTU per hour square foot degrees Fahrenheit is designed to prevent the shale from freezing. Thermometers will be installed in the shale to monitor wintertime temperatures.

The details of rock excavation requirements, rock reinforcement, rock protective measures, and instrumentation requirements are purposely avoided in this report. The above description is intended to furnish only enough detail to better understand the rockbolt and anchor test program.

AD-A160 524 CLARENCE CANNON DAM AND MARK TWAIN LAKE FOUNDATION AND 4/5
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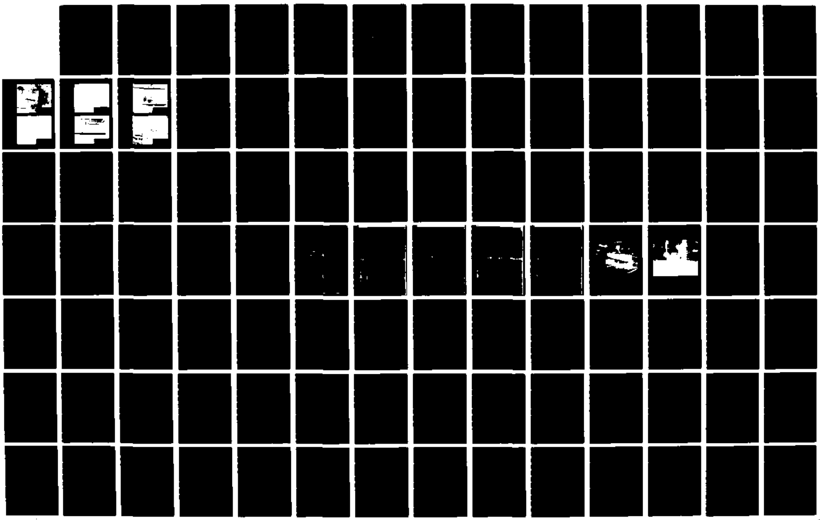

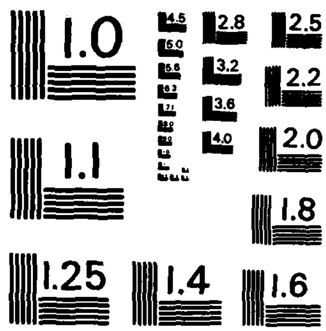
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6.3

VIII. DESCRIPTION OF ROCKBOLT AND ANCHOR TEST PROGRAM.

The total test program consisted of two series of tests - one in limestone and one in shale. The tests were designed to verify anchorage and stress/strain relationships; and to evaluate materials and installation techniques. In addition, if the "Celtite-Dywidag" system proved effective in the weak shale, test results would provide in-situ measurements of the shale compressive strength, shear strength and Young's modulus of the shale. The tests in limestone were conducted in November 1973 in a vertical, production limestone cut. The shale tests were conducted in May 1974 prior to any production excavation in the shale.

A. <u>Limestone Tests</u>. The limestone tests consisted of the installation and testing of two 30-foot and two 35-foot bolts.

1. <u>Test L-1</u> was a 30-foot installation to determine anchorage, load transfer, stress/strain relationship, and for obtaining long term periodic load cell measurements of the anchorage. It was anchored with 5 feet of fast-set resin and stressed to 75 kips in maximum increments of 2,000 lbs. Bolt elongation measurements were made for each increment. The load was then transferred and the load cell was left intact for long term periodic readings.

2. <u>Tests L-2, L-3, and L-4</u>. These tests were to evaluate anchorage, materials and installation techniques. The hole was drilled and gaged for each test and approximately the back 5 feet charged with fast-set resin and the remainder with slow-set resin. The bolts were spun in, in one operation and stressed in increments after the fast resin set up but before the slow resin gelled. Measurements of the bolt elongation were made for each stressing increment. After load transfer and removal of the jack, the load cells were read periodically for 24 hours and the stress level recorded. Tests L-2 and L-4 were in 35-foot holes and L-3 a 30-foot hole.

B. <u>Shale Tests</u>. The tests in shale were performed on the installation of four 35-foot bolts; four 40-foot bolts; two 4-foot anchors; six 8-foot anchors; and two 12-foot anchors. The anchors were all stressed incrementally to failure with a record made of the bar movement for each stressing increment. Four of the long bolts were fully encapsulated with both fast and slow resin and the other four were anchored with fast-set resin only. The holes were all drilled with an air-track drill and gaged prior to the tests. A spinning rate of 120 rpm was initially required during bar installation. A slower spinning (40 rpm) rate was later tested and found to be satisfactory. Details of the individual tests are given in Table 1.

TABLE 1
DESCRIPTION OF ROCKBOLT AND ANCHOR TESTS IN SHALE

Test No.	Bar Length	Hole Depth	Hole Diameter	Resin Used*	Remarks
S-1	4'	3.5'	1-3/8"	3-3212 HV0001	Stressed to failure; pulled from hole for evaluation
S-2	8'	7.5'	1-3/8"	4-3212 HV0001	Stressed to failure; pulled from hole for evaluation
S-3	8'	7.5'	1-3/8"	4-3212 HV0001 1-3212 MV0510	Stressed to failure; pulled from hole for evaluation
S-4	12'	11.5'	1-3/8"	4-3212 HV0001 4-3212 MV0510	Stressed to failure; pulled from hole for evaluation
S-5	30'	34.5'	0 - 24.5', 3" 24.5' - 34.5', 1½"	4-3512 HV0001 4-3512 MV1530	Due to error bar was cut 5' short. Stressed to failure; pulled from hole for evaluation. Resin yield: 3'-6"
S-6	40'	39.5'	0 - 27.5', 1-7/8" 27.5' - 39.5', 1½"	4-3512 MV0001 5-3512 MV0510 4-4012 MV1530	Stressed to 85 Kips Could not fail Resin yield: 15'-4"
S-7	35'	34.5'	0 - 24.5', 1-7/8" 24.5' - 34.5', 1½"	4-3512 HV0001 4-3512 MV1530	Stressed to 29 Kips; load transferred and load cell read for two days. Resin yield: 10'-8"
S-8	40'	39.5'	0 - 27.5', 1-7/8" 27.5' - 39.5', 1½"	4-3512 HV0001 5-3512 MV1530	Stressed to 33 Kips; load transferred and load cell read for two days. Resin yield: 12'
S-9	4'	3.5'	1-3/8"	3-3212 HV0001	Stressed to failure
S-10	8'	7.5'	1-3/8"	4-3212 HV0001 1-3212 MV0510	Stressed to failure

TABLE 1 - Con't

set in minute Time

Test No.	Bar Length	Hole Depth	Hole Diameter	Resin Used*	Remarks
S-11	8'	7.5'	1-3/8"	4-3212 HV0001 1-3212 MV0510 5 to 10 min set time	Stressed to failure.
S-12	12'	11.5'	1-3/8"	4-3212 HV0001 4-3212 HV0510	Stressed to failure.
S-13	35'	34.5'	0 - 24.5', 1-7/8" 24.5' - 34.5', 1½"	4-3512 HV0001 5-3512 MV0204 27-4012 HV1530	12' anchor zone; bolt fully encapsulated. Stressed to 77 Kips. Load not transferred.
S-14	35'	34.5'	0 - 22.5', 1-7/8" 22.5' - 34.5', 1½"	3-3512 HV0001 6-3512 MV0204 24-4012 MV1530	Stressed to 29 Kips; Load transferred; 12' anchor zone.
S-15	40'	39.5'	0 - 29.5', 1-7/8" 29.5' - 39.5', 1½"	4-3512 HV0001 4-3512 MV0204 31-4021 MV1530	Stressed to 29 Kips; Load transferred; 10' anchor zone.
S-16	40'	39.5'	0 - 27.5', 1-7/8" 27.5' - 39.5', 1½"	3-3512 HV0001 6-3512 MV0204 28-4012 MV1530	Stressed to 29 Kips; Load transferred; 12' anchor zone.
S-17	8'	7.5'	1-3/8"	4-3212 HV0001 1-3212 MV0510	Hole filled with water before test. Stressed to failure; pulled from hole for evaluation.
S-18	8'	7.5'	1-3/8"	2-3212 HV0001 3-3212 MV0510	40 rpm spinning rate during installation. Stressed to failure; pulled from hole for evaluation.

*Resin Used code: 4-3212 MV 0510

- 0510 — Setting time range. 5 to 10 minutes.
- MV — Medium viscosity.
- 3212 — Cartridge size: 32mm diameter by 12-inches long.
- 4 — Number of cartridges.

IX. TEST RESULTS.

Summaries of the rockbolt and anchor tests are shown on Tables 2 and 3. Figures 34-55 show for each test individual plots of bar displacement and stressing increments. Generally speaking, the test results verified the manufacturers' claims for the components of the "Celtite/Dywidag" System; and provided some very useful in-situ testing of the Hannibal shale, which tended to confirm the design assumptions.

A. Limestone Test Results. The limestone tests verified the stress/strain curve of the "Dywidag" bar, proved the adequacy of the system for limestone applications, and demonstrated satisfactory installation techniques. In addition, test No. L-1 is still being monitored by periodic readings of the stress level as indicated by the load cell. This bolt is still (after 10 months) maintaining its load even after near-by blasting operations and excavation. Figure 23 is a plot of the load cell readings vs. time since its installation. Most of the variation is considered to be the result of blasting effects, temperature variations, and read-out sensitivity. Sufficient load has been maintained since installation to conclude the anchorage may be permanently bonded in the limestone and no creep of the resin, nor anchorage failure, is apparent after almost a year.

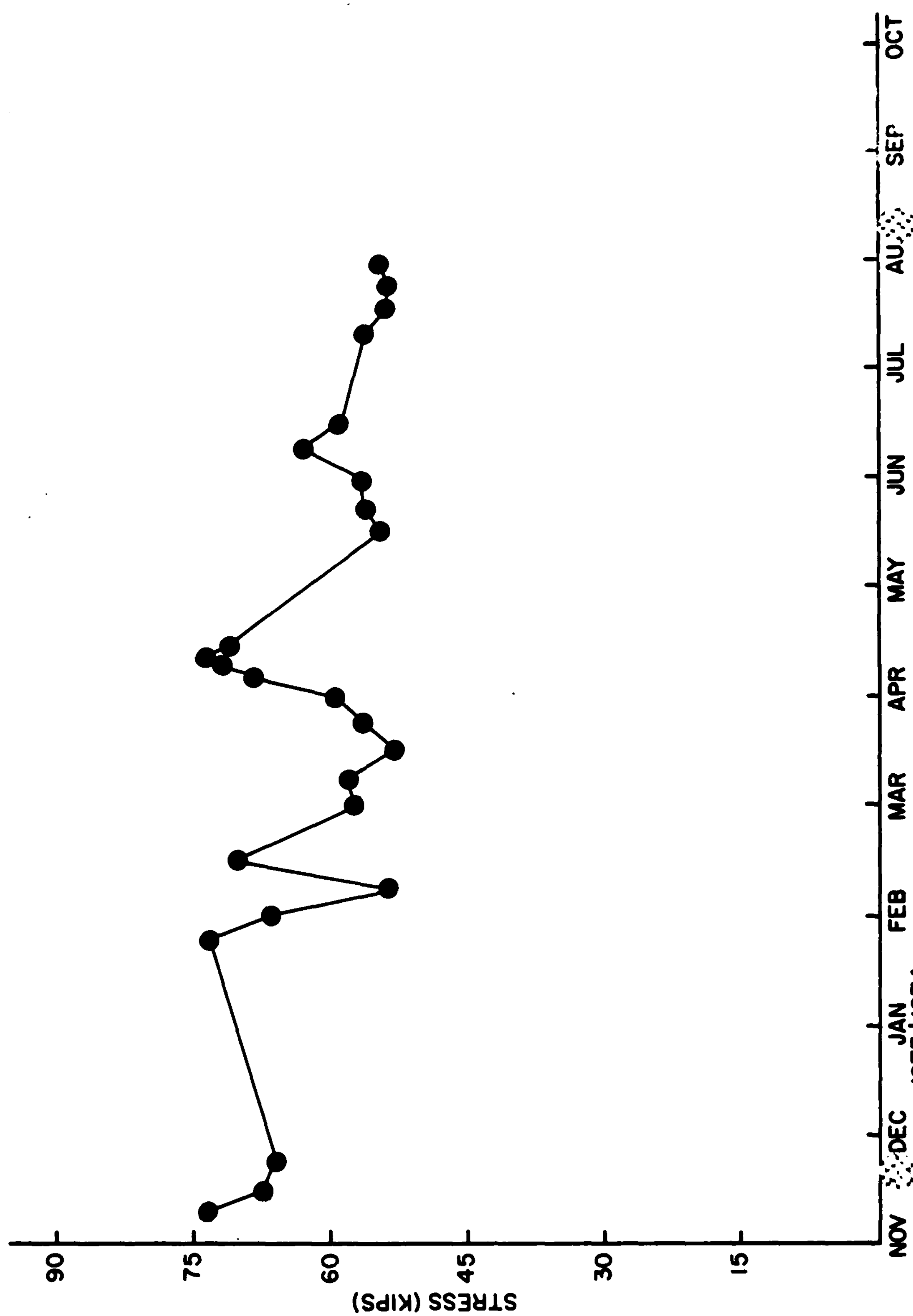
No relationships between the length of encapsulation and the amount of anchorage could be established since the 5-feet of encapsulation used in the limestone tests probably provided anchorage stronger than the system. None of the limestone test bolts were pulled to failure. Based on information furnished by Mr. Gary Greenfield, the "Celtite" representative, and preliminary testing at this project, one ton per inch of encapsulation is a reasonable estimate of anchorage capacity in the lower Burlington limestone formation.

A preliminary test at this project was anchored by 60-inches of resin and stressed to 105 kips, without producing failure.

Figures 24 and 25 may be used for preliminary guides in estimating the anchorage factor or length of anchorage needed for various rocks based on unconfined compressive strengths.

A comparison of the design stress/strain of the "Dywidag" bar and the field determination of stress/strain is shown in Figure 26. The field value determined from Test L-1 falls on the design stress/strain curve.

Installation techniques of the "Dywidag/Celtite" system proved to be quite simple in the competent Burlington limestone. In accepting the system, however, reservations were made to require grouting of bolts with cement grout when cavernous conditions or large open joints were encountered. Such conditions were encountered during production work. The procedure adopted for these conditions



TEST L-1 PLOT OF STRESS VS TIME

FIGURE 23

consisted of obtaining anchorage in 5-feet of good rock even if the hole depth had to be adjusted. The remainder of the hole was reamed to a larger diameter (3-inches) to assure the practicality of installing a 1/2-inch grout tube to the anchor zone at the back of the hole. Normal grouting procedures were then used to grout the hole with nonshrinking cement grout. One area had to be consolidated with grouting prior to bolting. It had open joints and was in a critical location by virtue of the fact that behind the limestone to be bolted was a large filled-sink structure almost 100-feet in depth.

B. Shale Test Results. Test results of rockbolts and anchors installed in the Hannibal shale proved the system to be effective in this formation and provided sufficient data to allow extending anchorage factor estimates to include weaker materials. (See FIG. 24 and 25). FIG. 33 shows a plot of Resin Encapsulation vs. Failure Loads for anchors pulled to failure. Failure in all these tests was shear failure in the shale rather than failure of components of the system. This allowed calculation of in-situ values of direct shear strength, stress/strain, and Young's modulus for the shale. An estimate of the unconfined compressive strength of the shale was based on observations of the first sign of failure of an 8-inch-square bearing plate at 13,000 pounds stress. The average in-situ values of the shale strength compared to design assumptions are as follows:

	In-Situ Value	Design Assumption
Modulus of elasticity:	$.036 \times 10^6$ psi	$.027 \times 10^6$ psi
Unconfined compressive strength:	200 psi	140 psi
Average direct shear strength:	101 psi	--
Average strain at 101 psi stress:	.0030 in/in	

The test values compare reasonably well with the design assumptions, which were based on tests of valley shale.

Rockbolting in the Hannibal shale was considered a very critical item in the Main Dam contract prior to the tests. The ability of the Contractor to obtain more than adequate anchorage in the weak valley shales and the demonstrations of a simple, efficient installation procedure for rockbolts and anchors virtually assured the success of the rock reinforcing program at the Clarence Cannon project. The "Dywidag/Celtite" system is con-

36000
27000 psi

TABLE 2

SUMMARY OF ROCKBOLT TESTS.

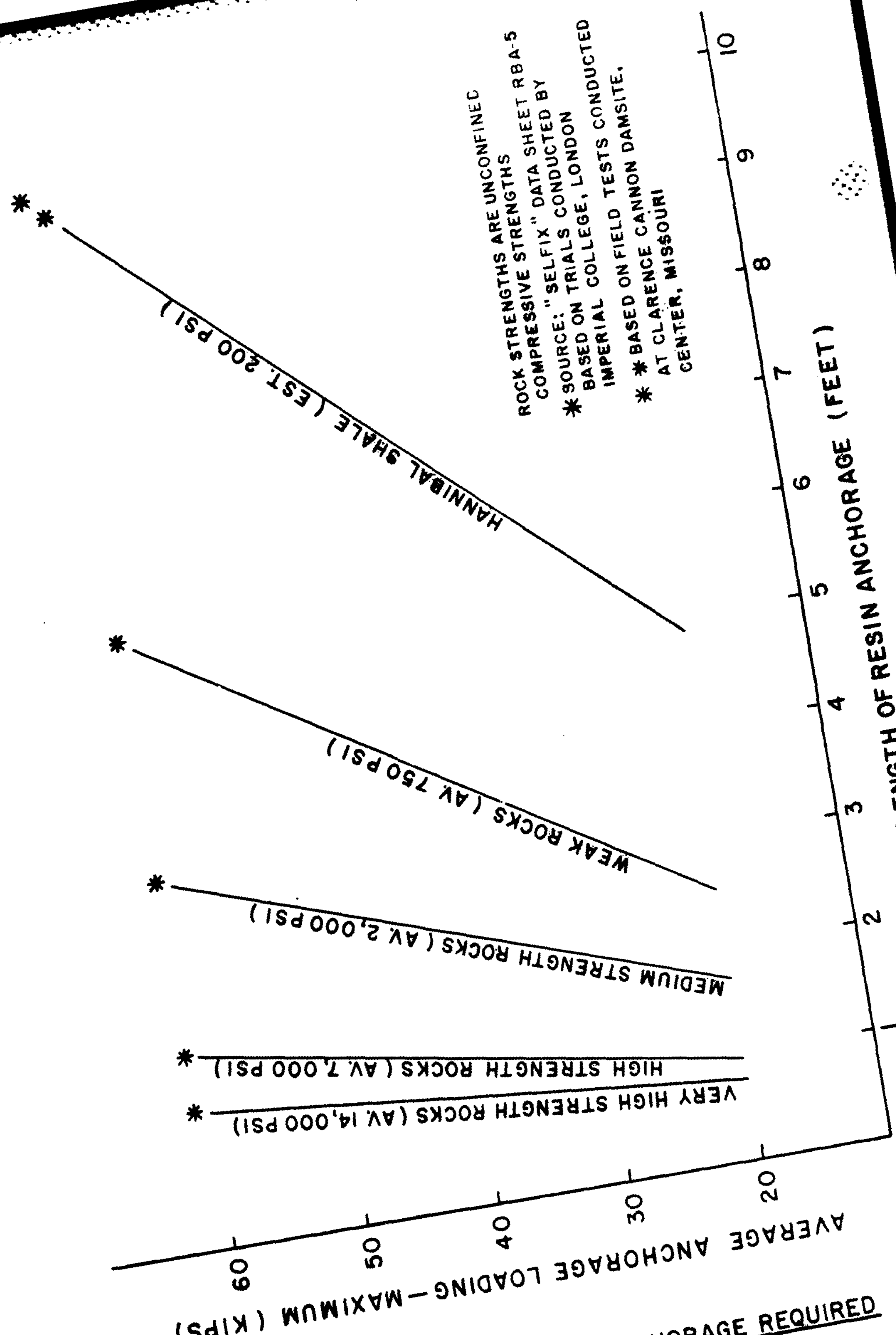
Test No.	L-1	L-2	L-3	L-4	S-7	S-8	S-13	S-14	S-15	S-16
Location Station No. Offset (Feet	1+86 17.5'DS	1+86 12'DS	1+86 CL	1+86 CL	3+40 116'US	3+41 110'US	3+47 56'US	3+40 125'US	3+42 98'US	3+47 65'US
Elevation (Ft-MSL)	603.3	604.2	604.4	604.0	520.9	520.8	520.8	520.2	521.5	520.8
Resin Encapsulation (Feet)	5.3	35	30	35	10.7	12.0	34.5	34.5	39.5	39.5
Anchor Zone Length (Feet)	5.3	5.3	5.3	5.3	10.7	12.0	12.0	12.0	10.0	12.0
Hole Size-Anchor Zone (Inches)	1½	1½	1½	1½	1½	1½	1½	1½	1½	1½
Hole Size-Grout Zone (Inches)	-	1 7/8	1 7/8	1 7/8	-	-	1 7/8	1 7/8	1 7/8	1 7/8
Maximum Stress (Lbs x 1000)	76	34	49	40	29	33	29	29	29	29
Load Transferred (Lbs x 1000)	74	28	49	39	28	30	--	25	26	26
Displacement at Maximum Stress (Inches)	1.13	0.64	0.71	0.66	0.32	0.40	0.22	0.23	0.12	0.34

TABLE 3

SUMMARY OF ROCK ANCHOR TESTS

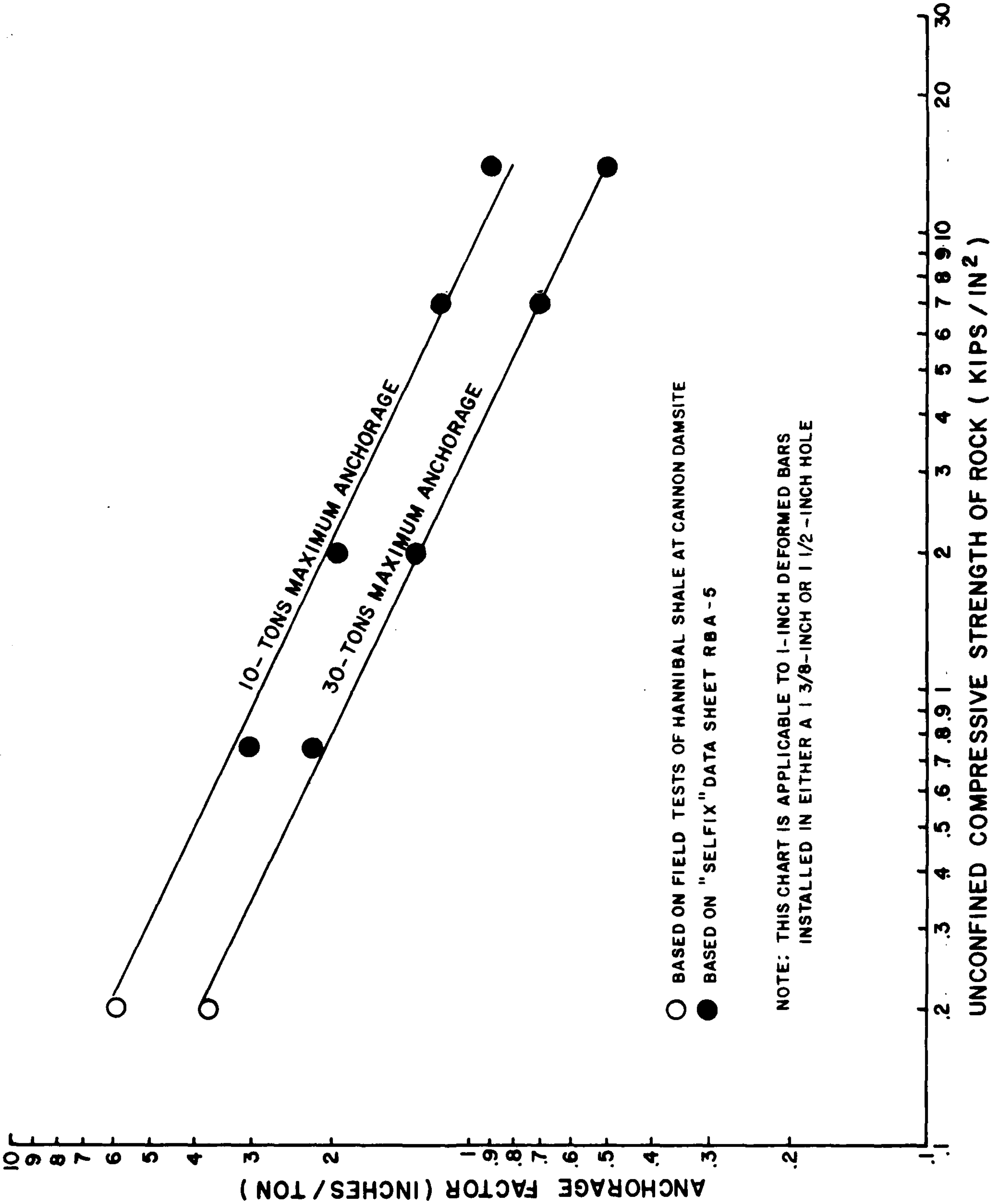
Test No.	S-1	S-2	S-3	S-4	S-5	S-6	S-9	S-10	S-11	S-12	S-17	S-18	AVG
Location Station No. Offset (US)	3+47 43'	3+47 48'	3+47 56'	3+47 62'	3+47 66'	3+44 81'	3+39 135'	3+39 131'	3+40 138'	3+39 124'	3+39 119'	3+38 131'	
Elevation (Ft-MSL)	522.5	522.3	523.1	522.7	522.7	520.9	523.2	520.9	521.0	520.7	520.4	520.6	
Resin Encapsulation (Feet)	3.5	7.5	7.5	11.5	3.5	15.3	3.5	7.5	7.5	11.5	7.5	7.5	
Hole Size (Inches)	1 3/8	1 3/8	1 3/8	1 3/8	1½	1½	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	1 3/8	
Failure Load (lbs x 1000)	12.7	32.7	27.5	48.2	17.1	84.0*	11.0	48.5	43.7	78.0	51.0	62.0	
Displacement at Failure (Inches)	.12	.26	.28	.53	.26	1.1*	.07	.33	.26	.36	.34	.44	
Shear Strength of Rock (psi)	70	84	71	81	86	97+*	61	125	112	131	131	159	101
Anchorage (lbs/ft)	3629	4360	3667	4191	4886	5490+*	3143	6467	5827	6782	6800	8267	5274
Total Strain at Failure Load (inches/inch)	.0028	.0029	.0031	.0038	.0062	.0060+*	.0017	.0037	.0029	.0026	.0038	.0049	.0035
Shale Strain at Failure Load (inches/inch)	.0024	.0025	.0027	.0033	.0053	.0051+*	.0015	.0032	.0025	.0022	.0033	.0042	.0030
Young's Modulus of Shale (PSIx10^6)	.029	.034	.026	.025	.016	.019+*	.041	.039	.045	.060	.040	.038	.036

*Maximum Load Did Not Fail Anchorage
These values not included in averages



GUIDE FOR ESTIMATING LENGTH OF ANCHORAGE REQUIRED FOR STRESS LEVEL

FIGURE 24



GUIDE FOR ESTIMATING ANCHORAGE FACTOR BASED ON UNCONFINED COMPRESSIVE STRENGTH OF ROCKS

FIGURE 25

DESIGN AND TEST VALUES OF STRESS-STRAIN FOR DYWIDAG BAR

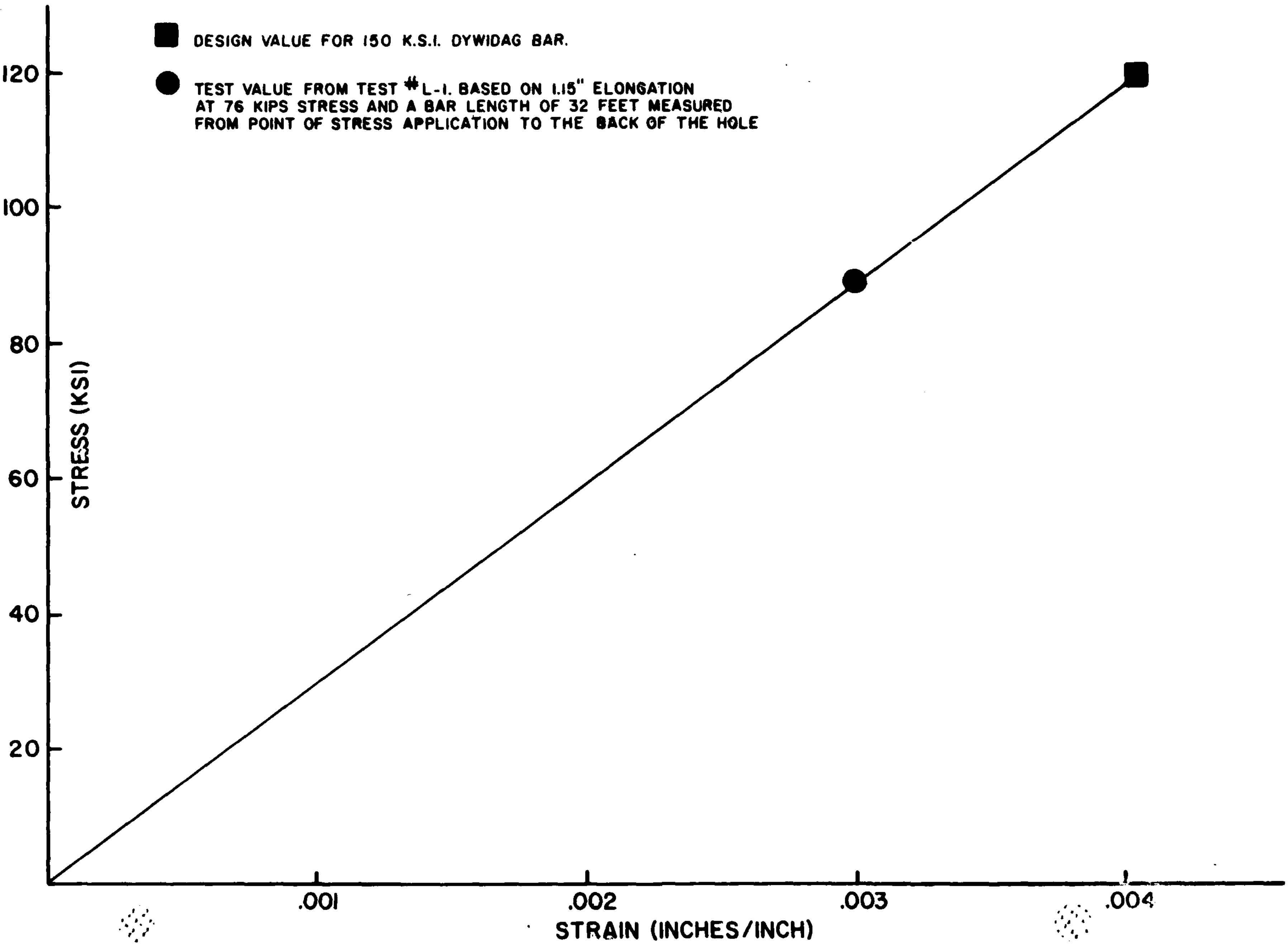


FIGURE 26

siderably stronger than the design requirements; thus providing an increased safety factor in a critical phase of the project. Based on the test results in the shale, a 12-foot anchor zone is required for long bolts, and the bearing plate size was increased to 14" x 14". Overstressing from the required 25 Kips to 35 Kips is a general procedure presently used to allow for any loss in stress due to load transfer.

FIGURES 27, 28, and 29 show bearing surface failure which was due both to conducting the tests in the weak valley shales and to stressing the test installations well above design requirements. Since the system is stronger than required in the design, a positive advantage is achieved with the larger size bearing plates now being used.

FIGURES 30, 31, and 32 show extracted anchors. These anchors all exhibited an intimate mixing of resin with the catalyst. They also served to verify that failure was due to shear failure in the rock itself. An excellent mechanical bond was apparent between the bar and resin and a combination of chemical and mechanical bond seemed to exist between the resin and shale. Apparently, the spinning during installation and mixing in the weak shale served to cause slight deformation of the hole walls, which served to increase the mechanical bond between the resin and the hole wall. Test No. S-17 was filled with water 2 hours before installation, and test No. S-18 was spun-in at a reduced rate (40 rpm) from the 120 rpm required by the specifications. These tests were both effective and neither the wet hole nor the reduced spinning rate seemed to have any adverse effect in the one test.

Based on the design modulus of elasticity, E, for valley shale and a 1-inch bar in a 1-3/8-inch hole, the strain value for shale was computed using the relationship Stress = Strain x E. The Young's moduli as previously given in paragraph IV were used for the resin and bar. Of the total strain, 85.7% could be attributed to the shale in the fully encapsulated anchor installation. TABLE 3 shows both total strain and the adjusted value for the shale.

Plots of the stress-displacement curves are shown on FIGURES 34-55. These curves reflect movement at both the bearing surface and anchorage zone. In weak materials, such as the Hannibal shale, a reference independent of the bearing surface should be considered as a basis for test measurements of displacement. This would enable more accurate evaluation of test results.

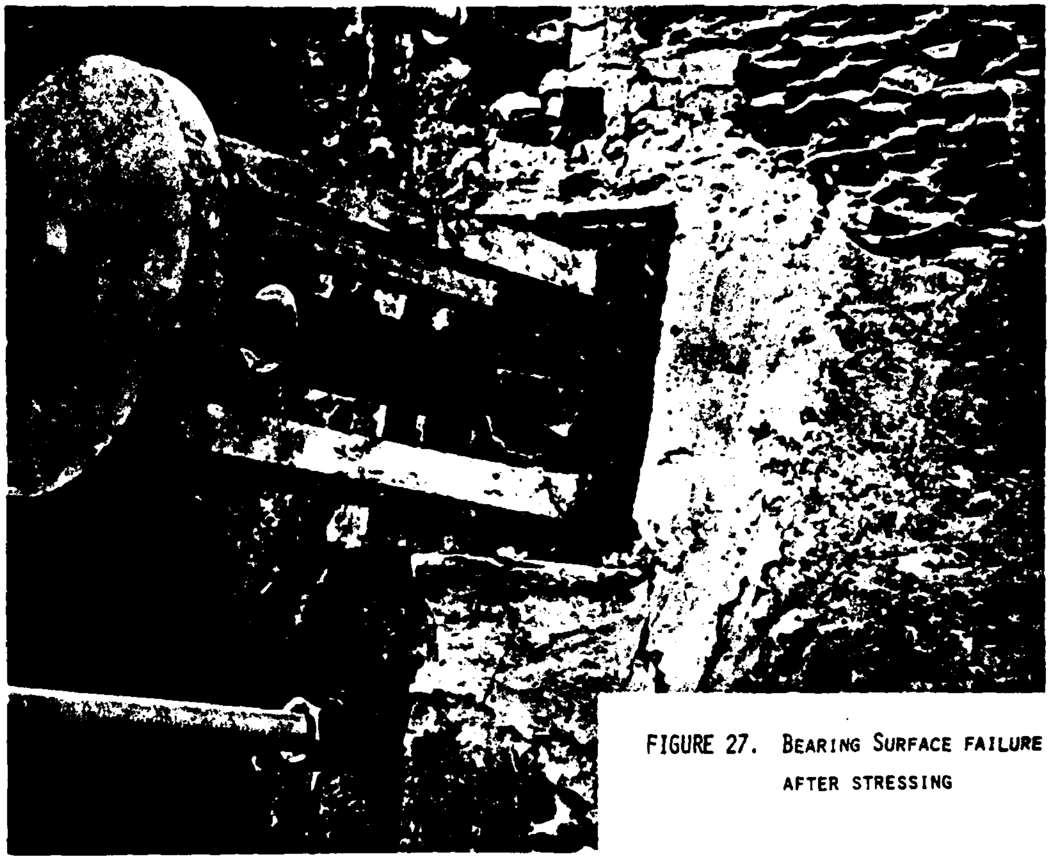


FIGURE 27. BEARING SURFACE FAILURE AFTER STRESSING

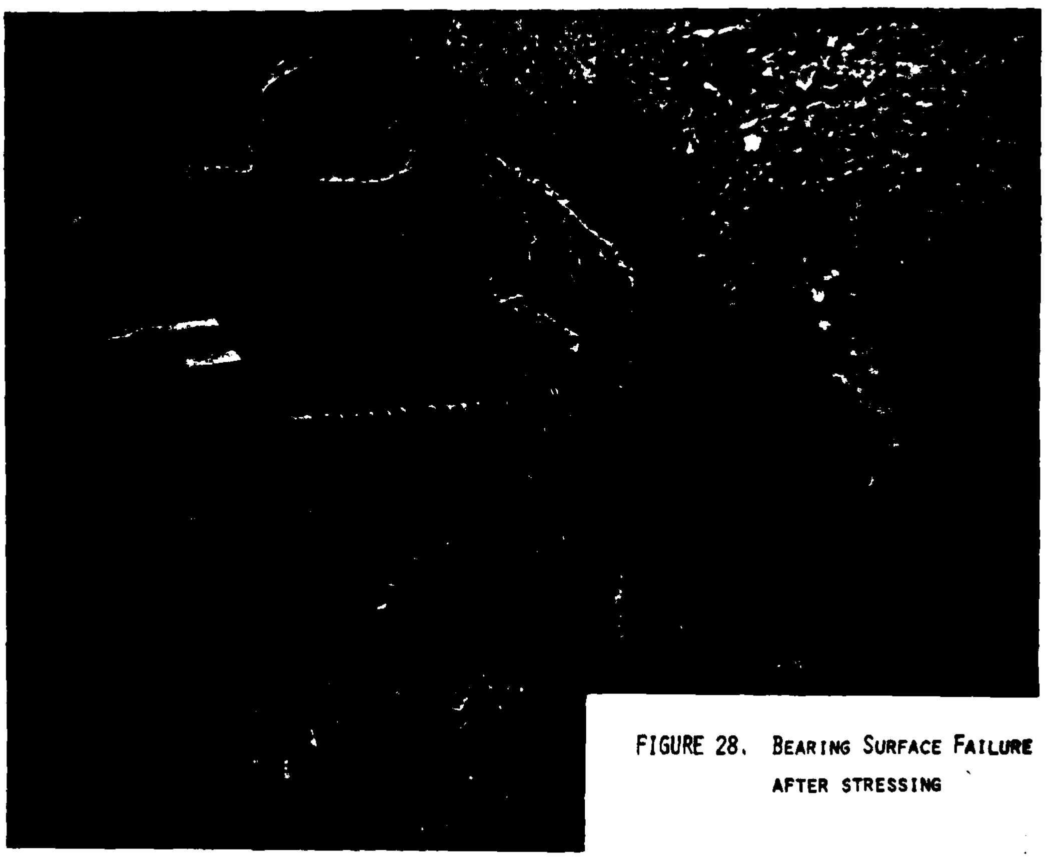


FIGURE 28. BEARING SURFACE FAILURE AFTER STRESSING

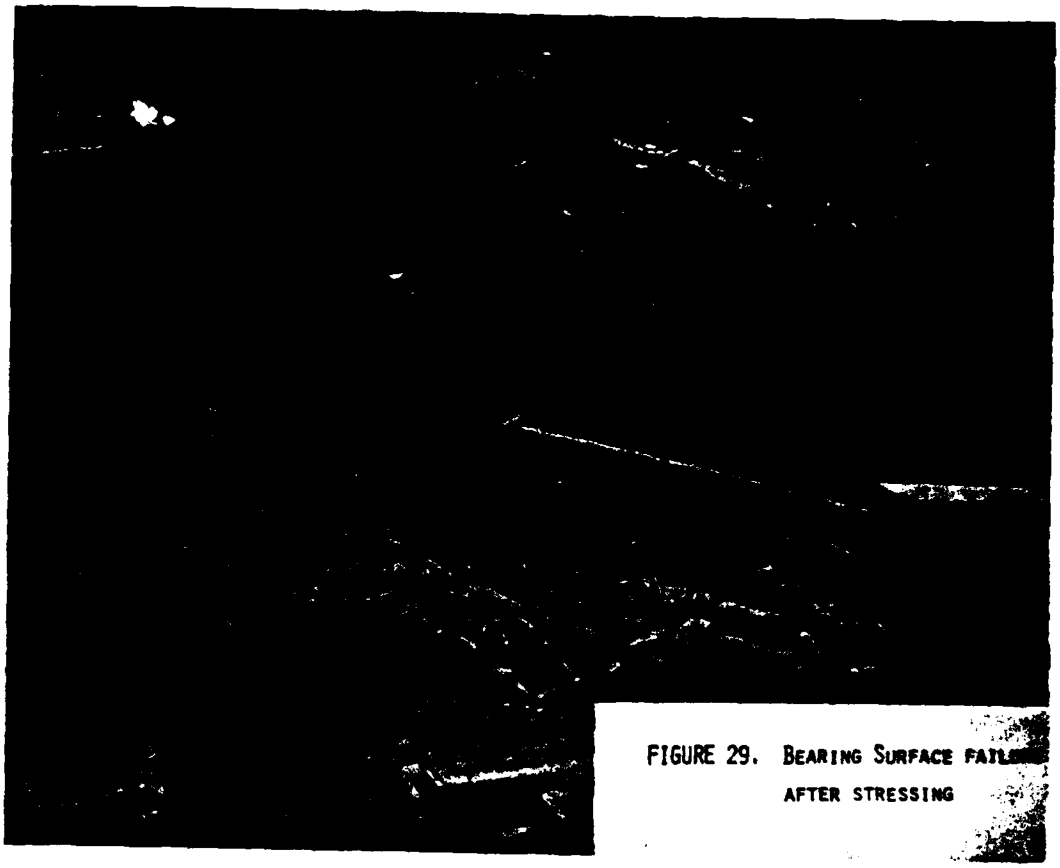


FIGURE 29. Bearing Surface Failure after Stressing

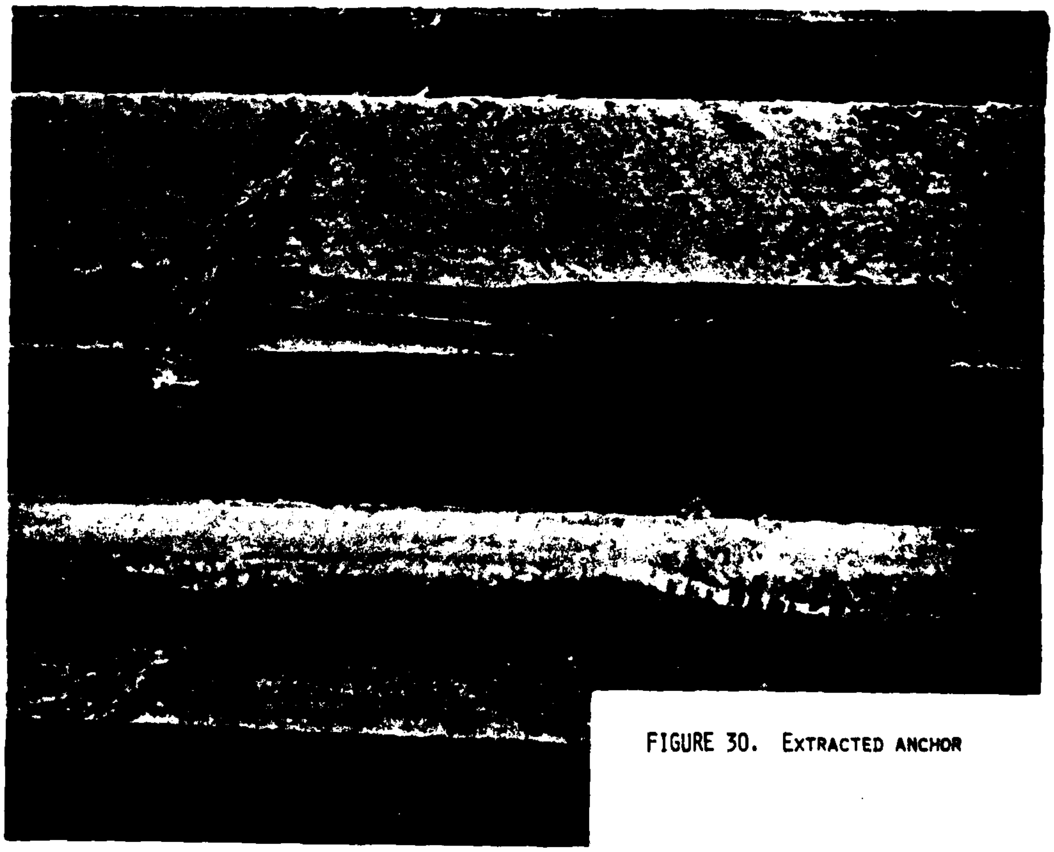


FIGURE 30. Extracted Anchor

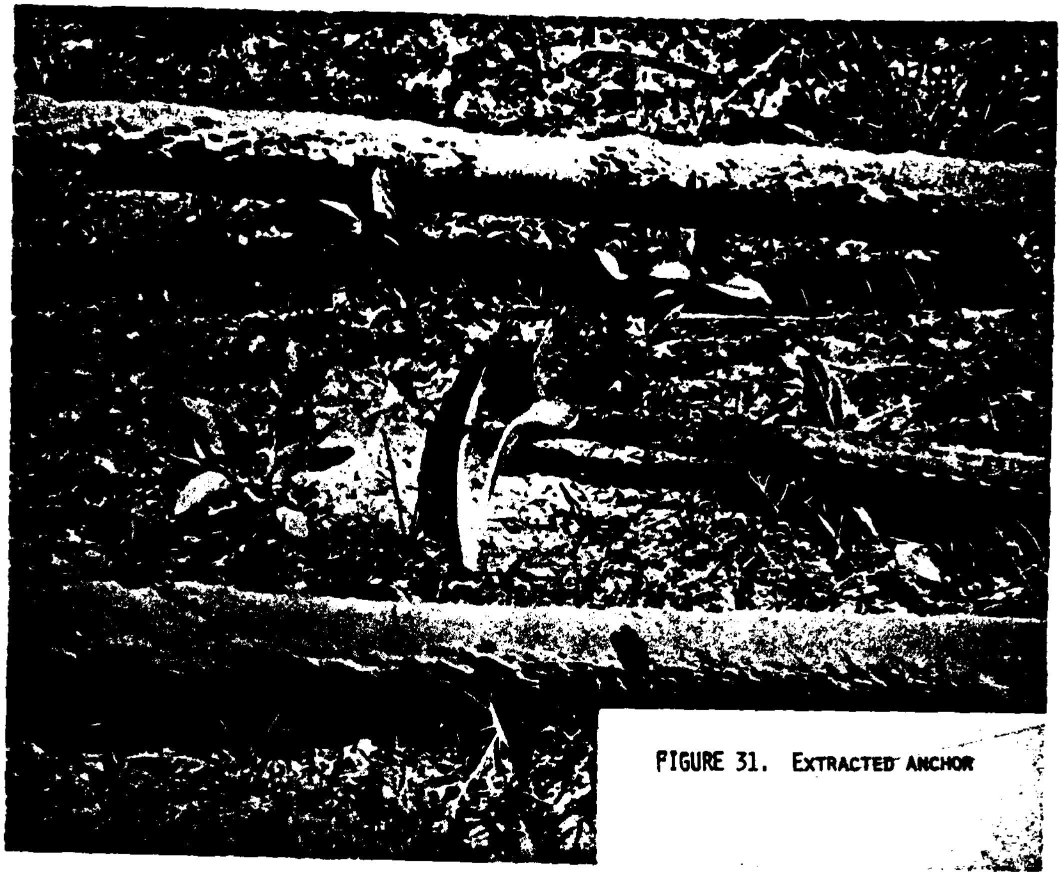


FIGURE 31. EXTRACTED ANCHOR

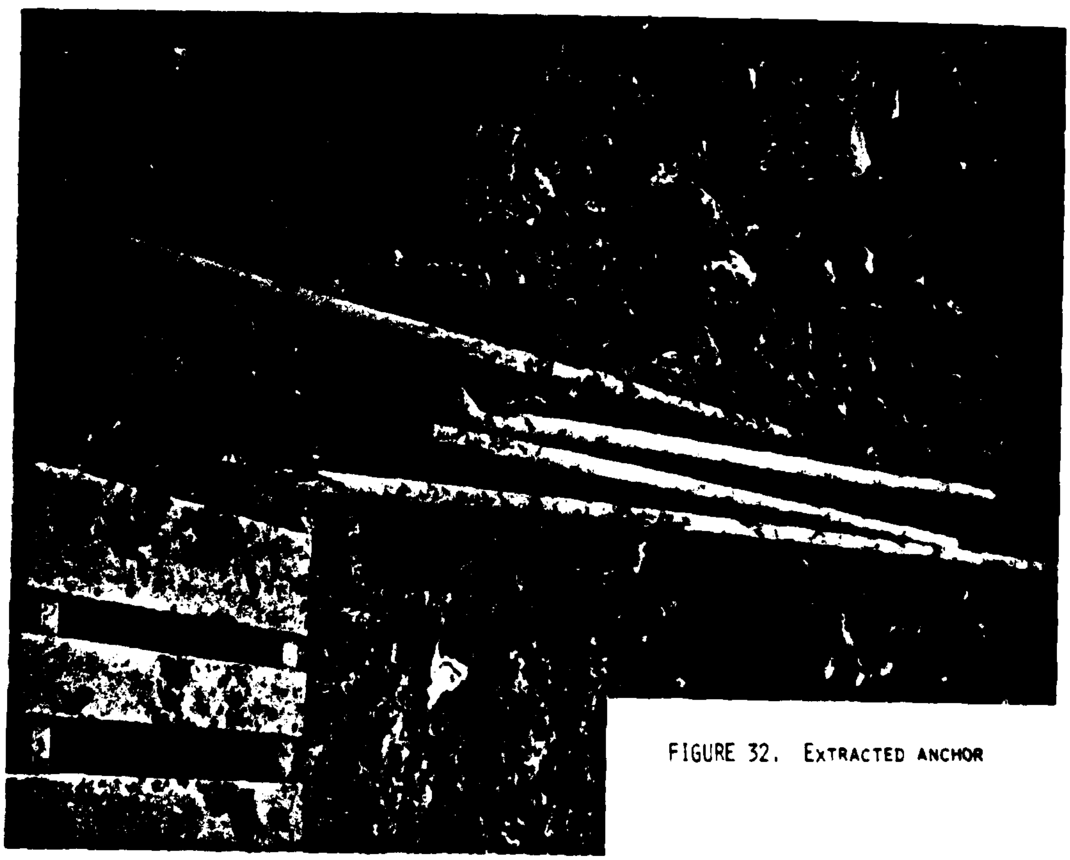
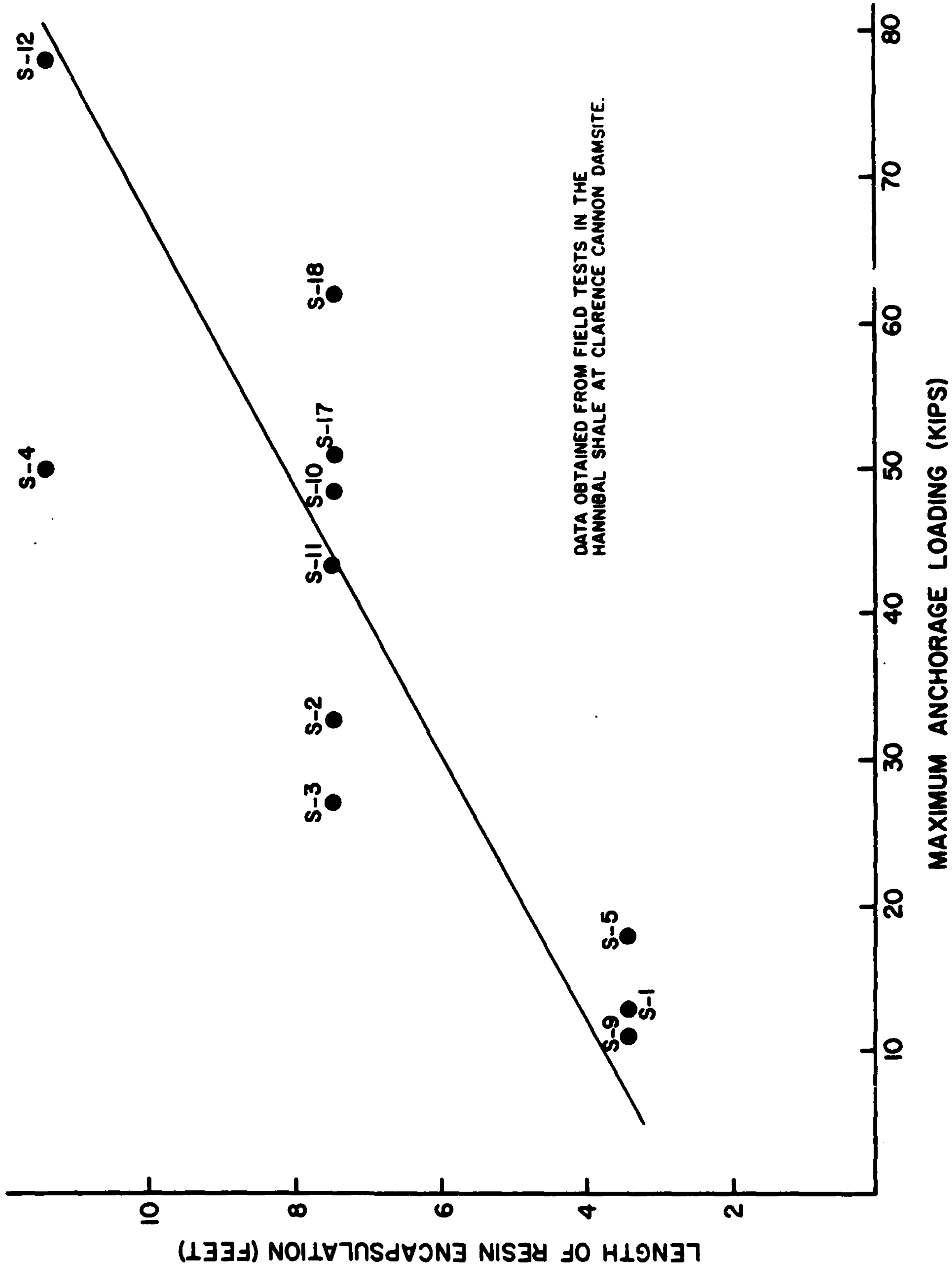
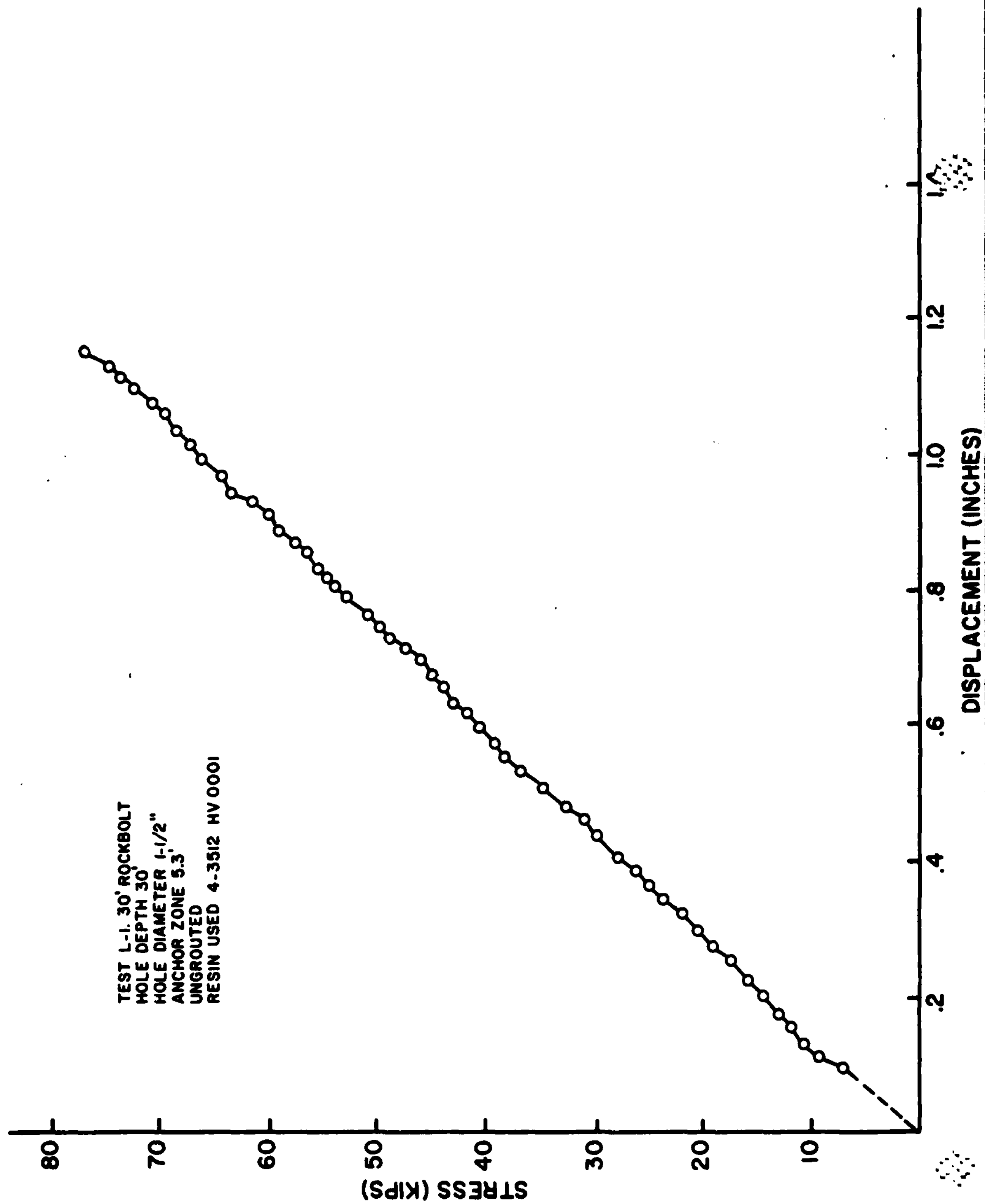


FIGURE 32. EXTRACTED ANCHOR



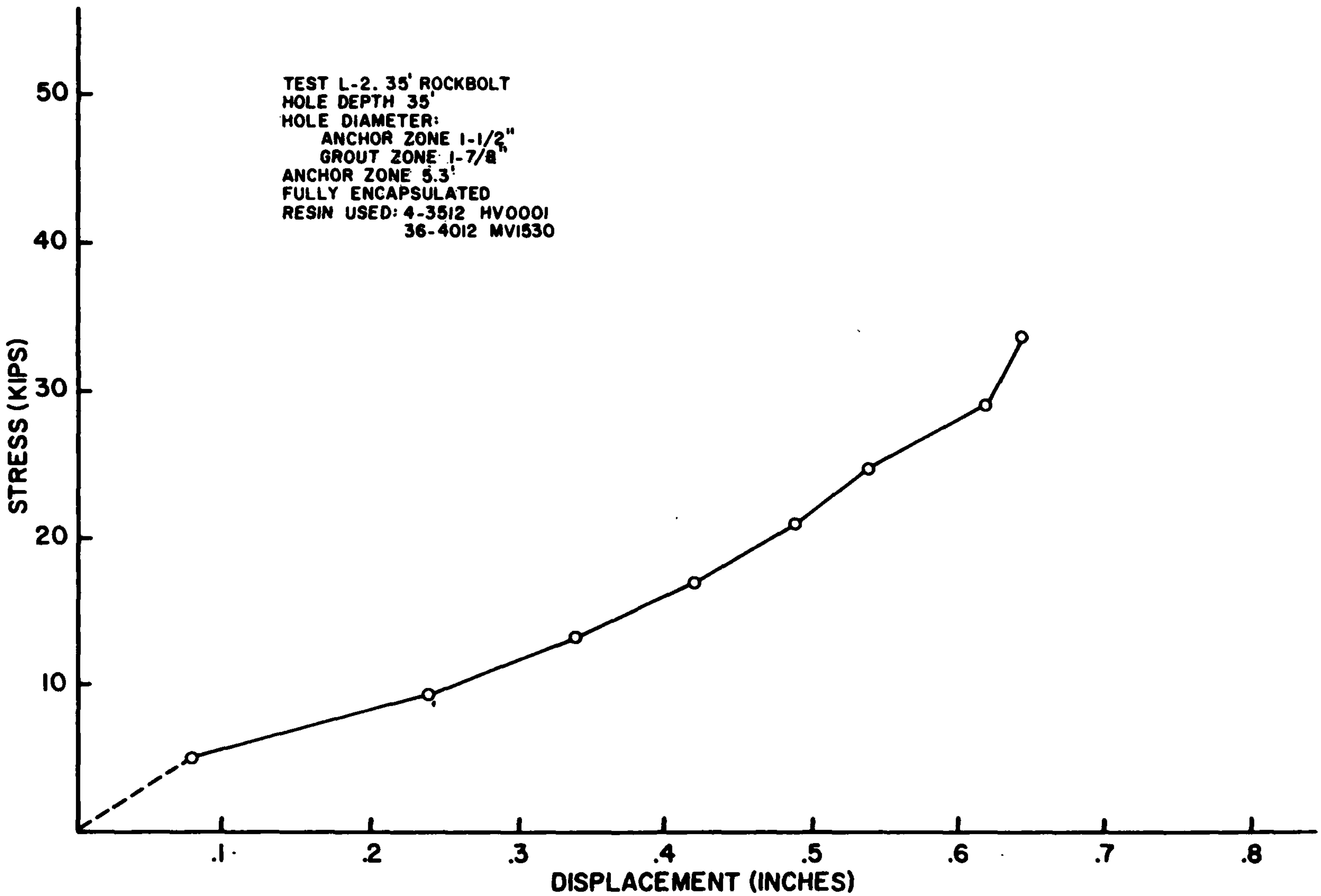
RELATIONSHIP OF MAXIMUM ANCHORAGE TO LENGTH OF ENCAPSULATION IN THE HANNIBAL SHALE.

FIGURE



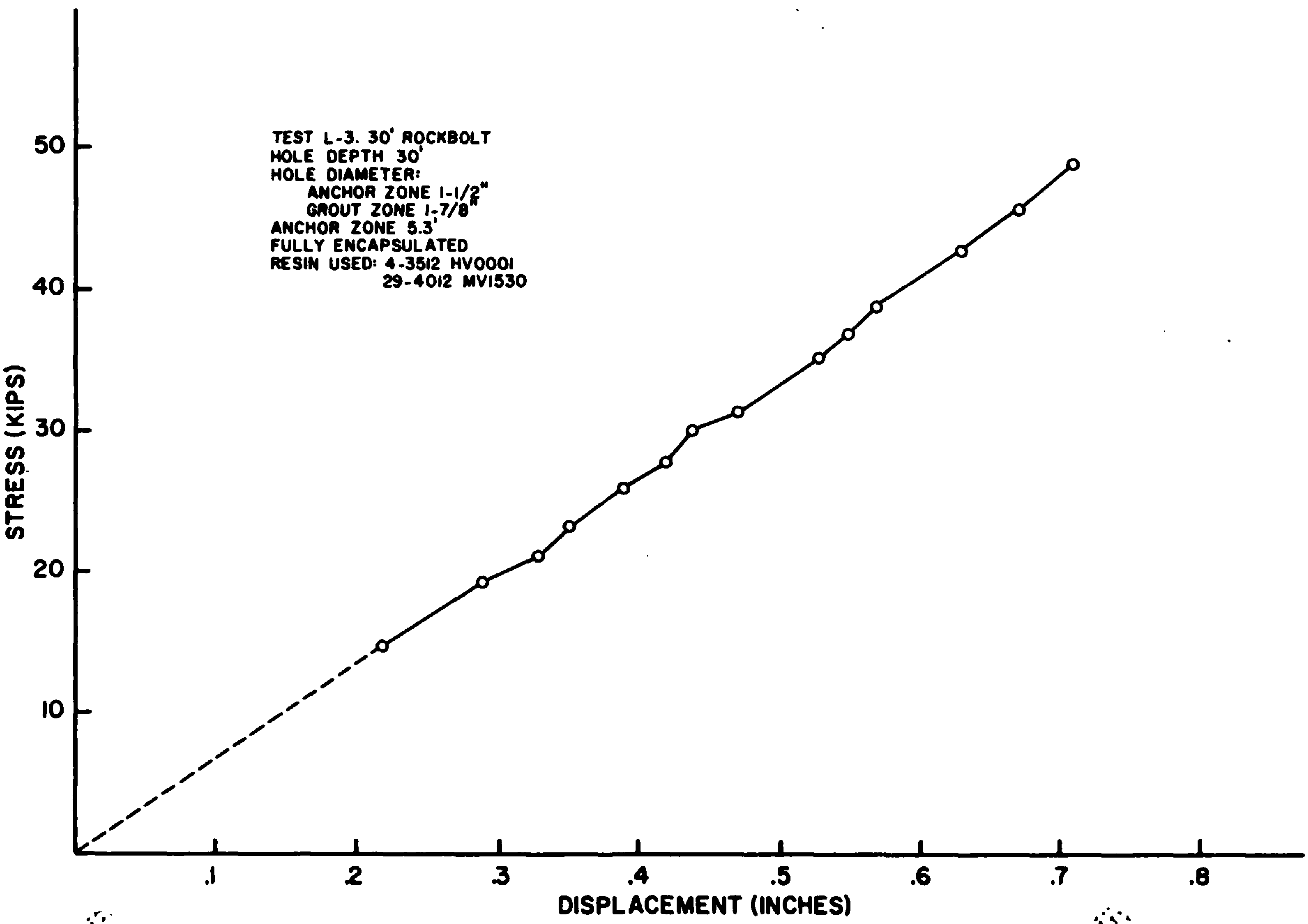
TEST L-I. STRESS DISPLACEMENT CURVE

FIGURE 34



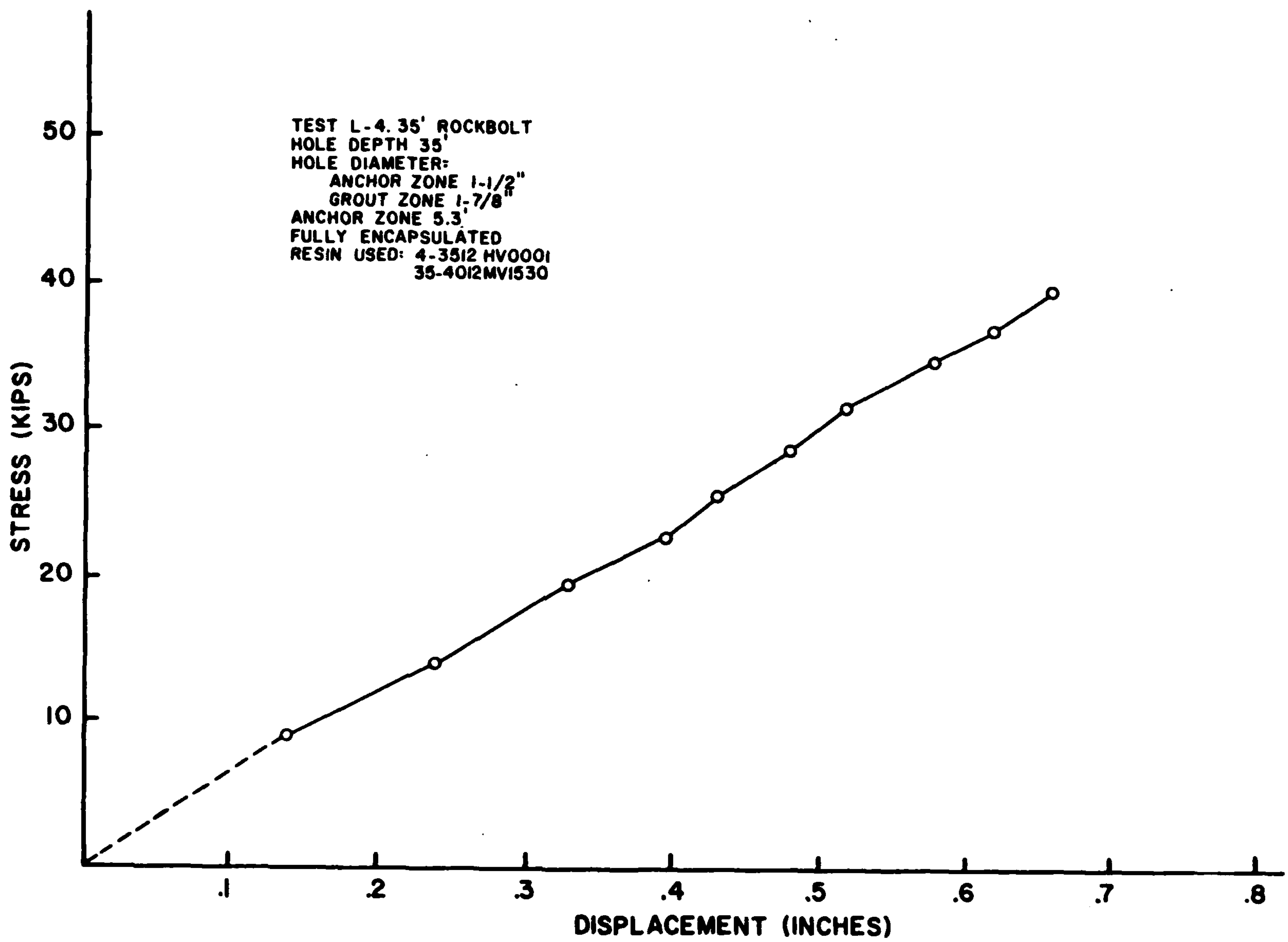
TEST L-2 STRESS DISPLACEMENT CURVE

FIGURE 35



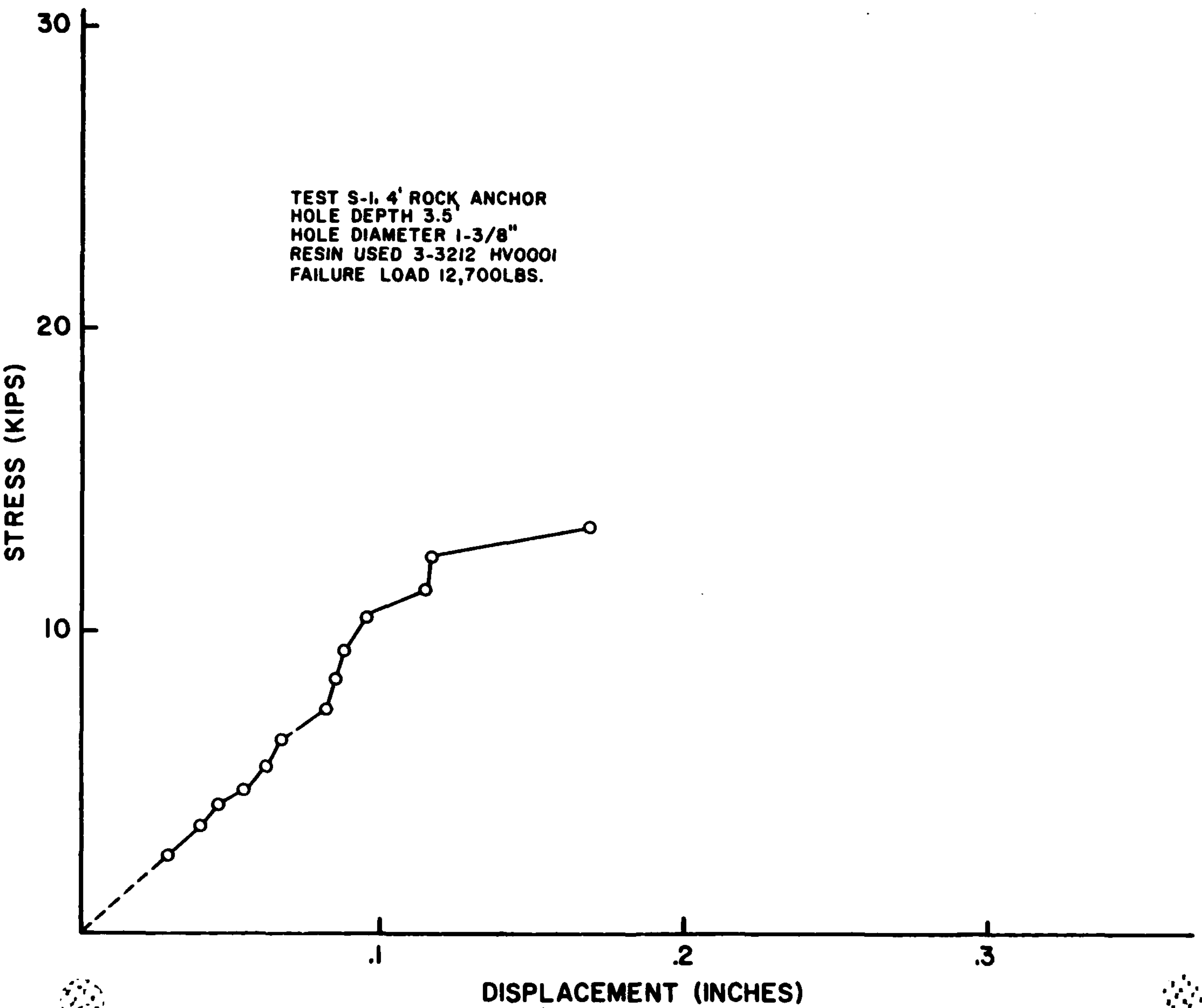
TEST L-3 STRESS DISPLACEMENT CURVE

FIGURE 36



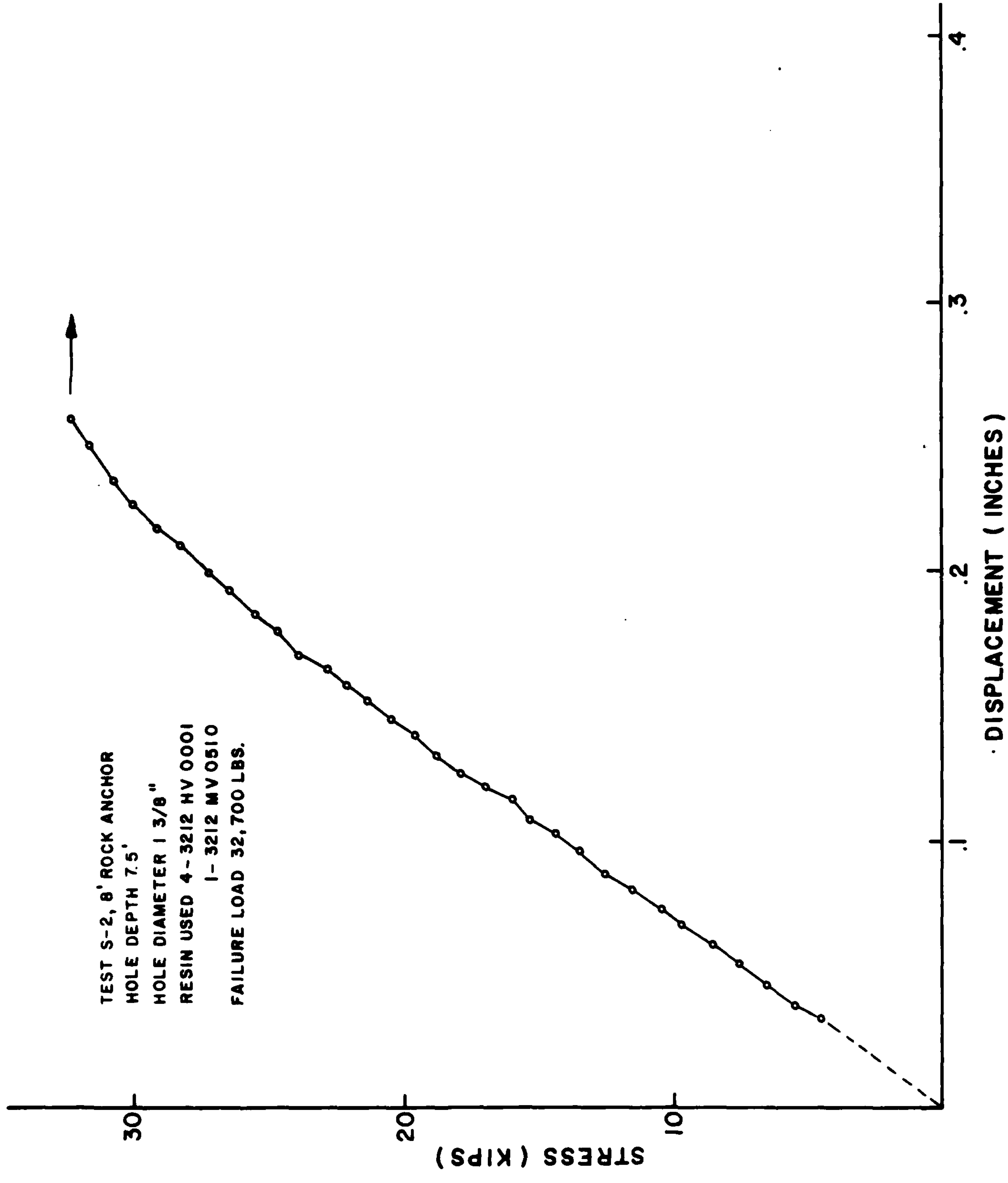
TEST L-4 STRESS DISPLACEMENT CURVE

FIGURE 37



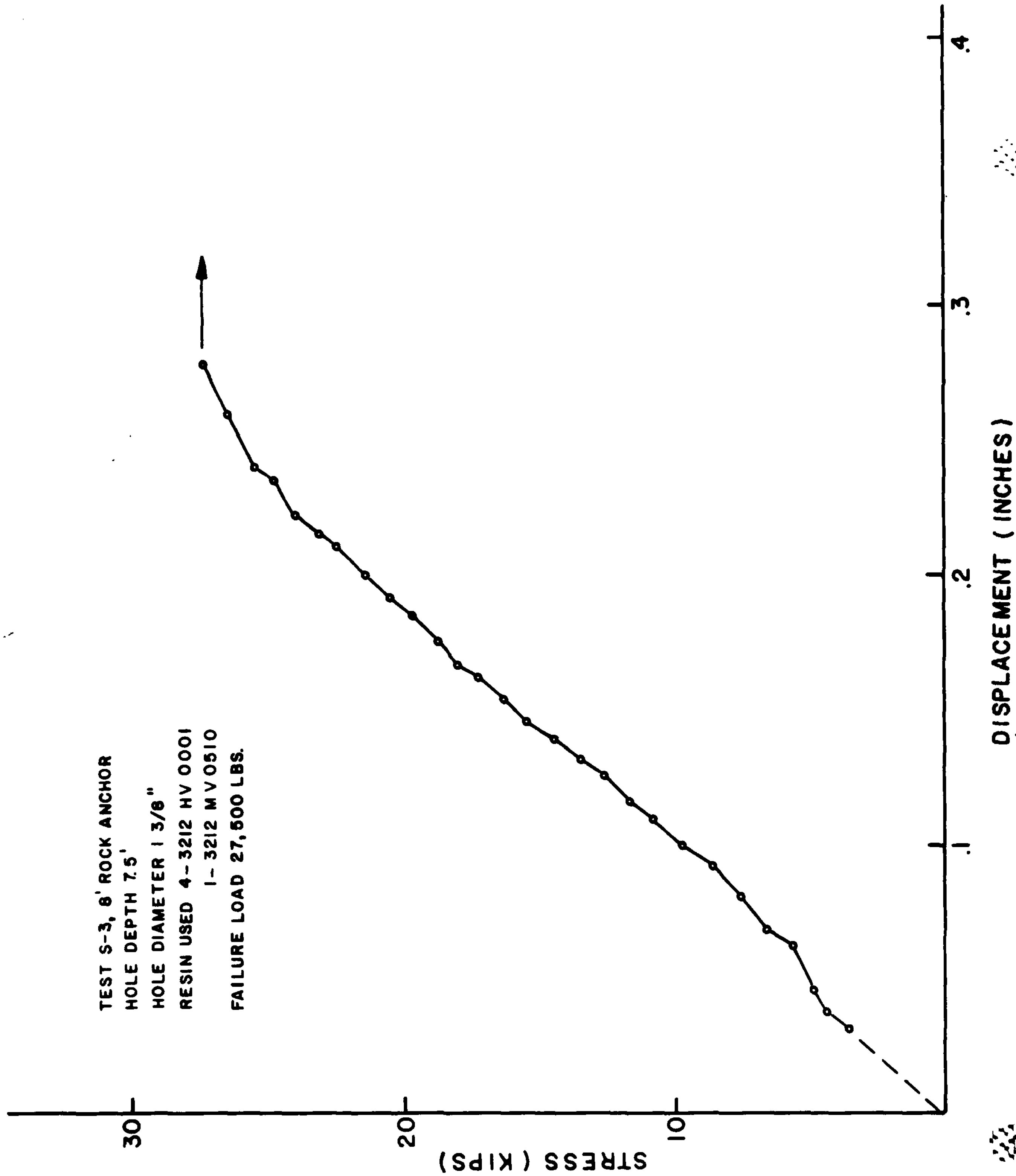
TEST S-1 STRESS DISPLACEMENT CURVE

FIGURE 38



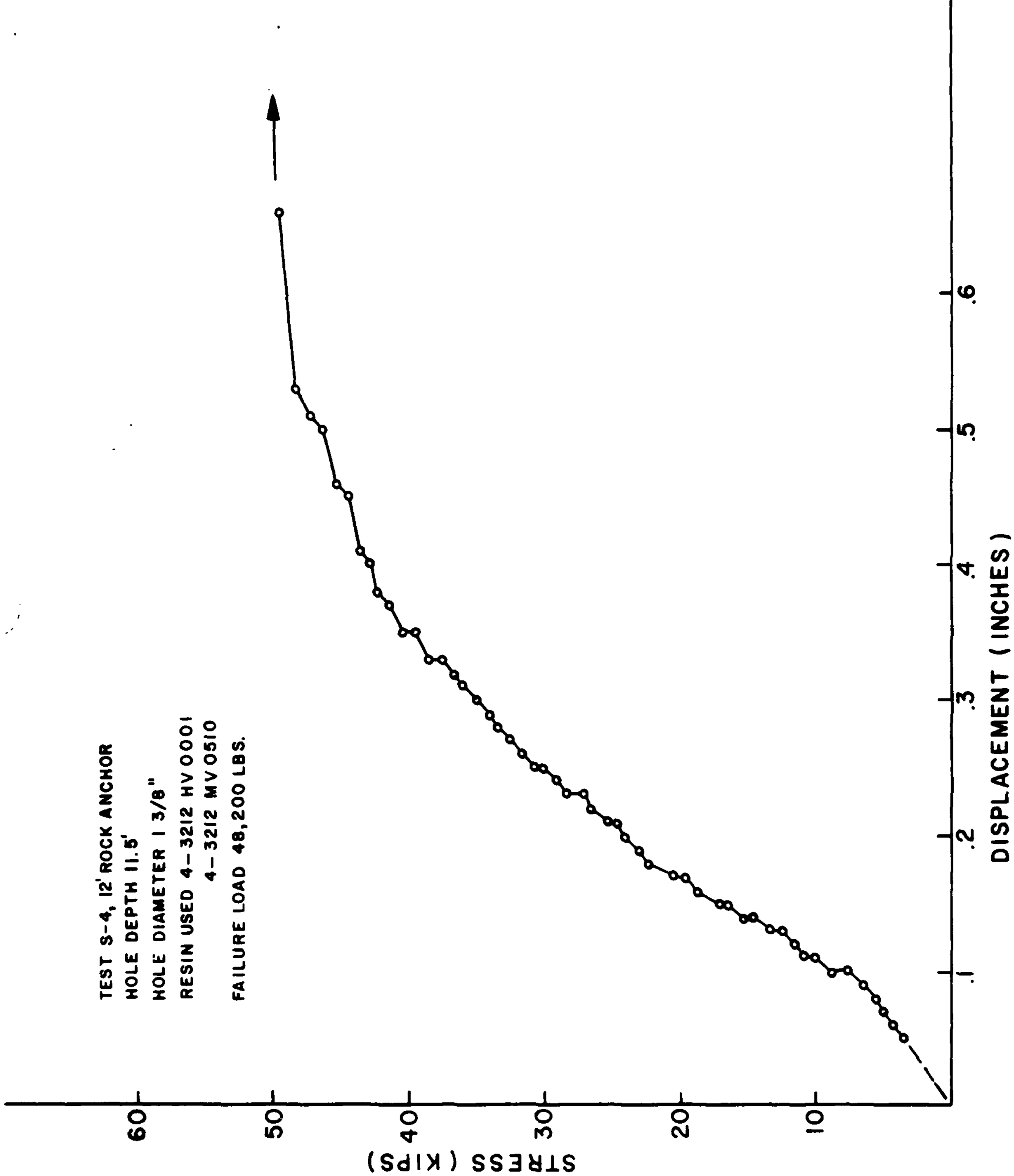
TEST S-2
STRESS DISPLACEMENT CURVE

FIGURE 39



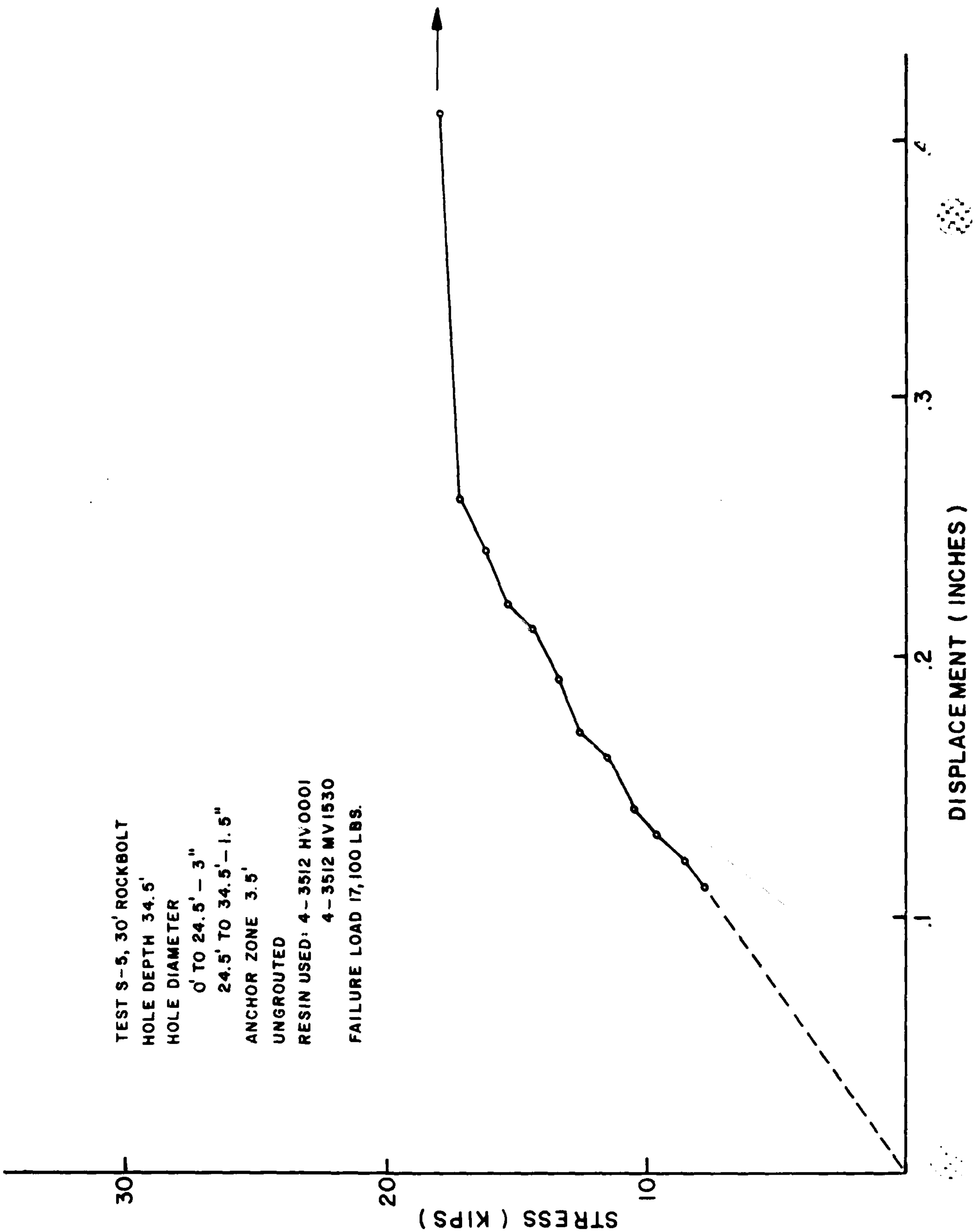
TEST S-3
STRESS DISPLACEMENT CURVE

FIGURE 40



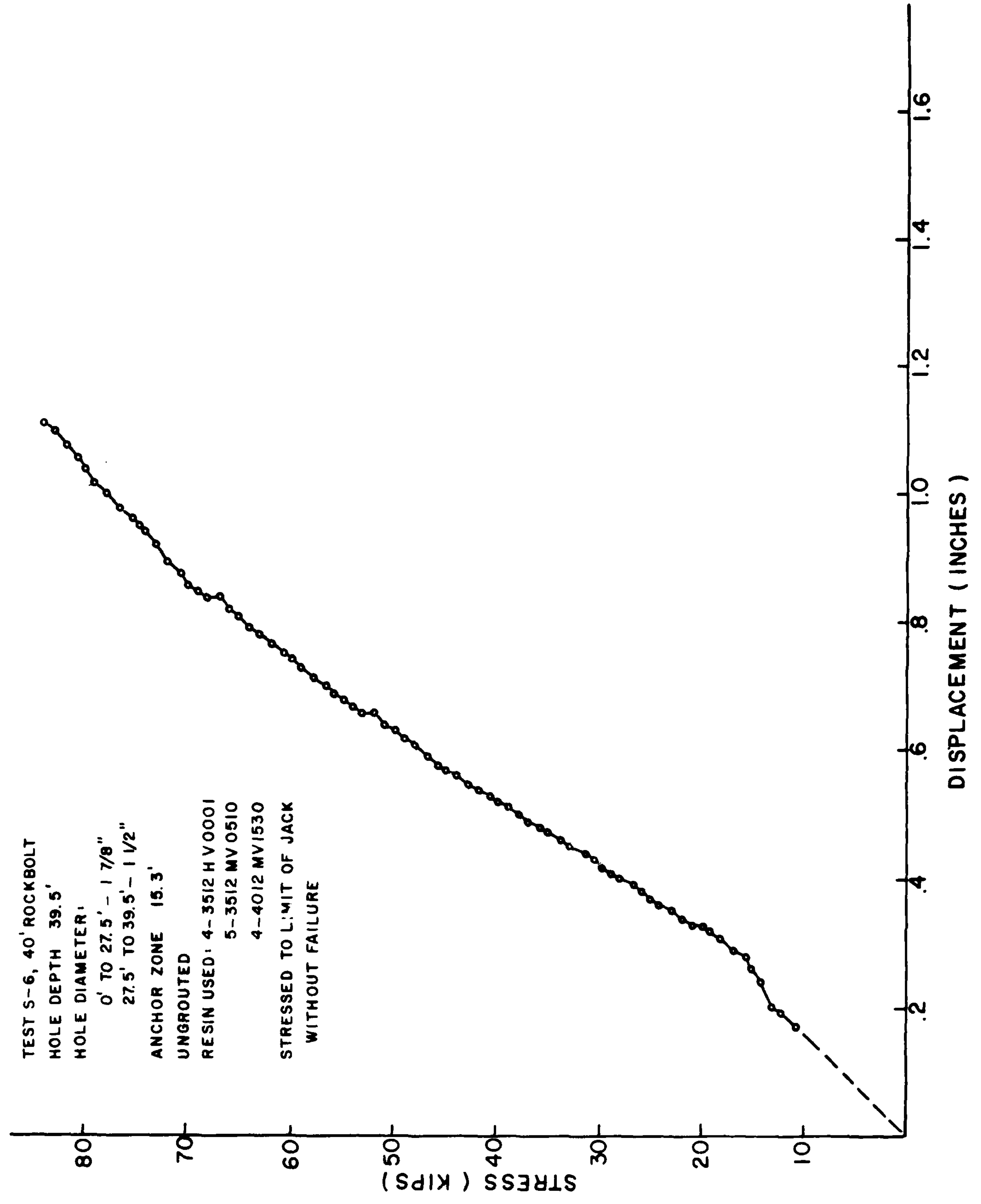
TEST S-4
STRESS DISPLACEMENT CURVE

FIGURE 41



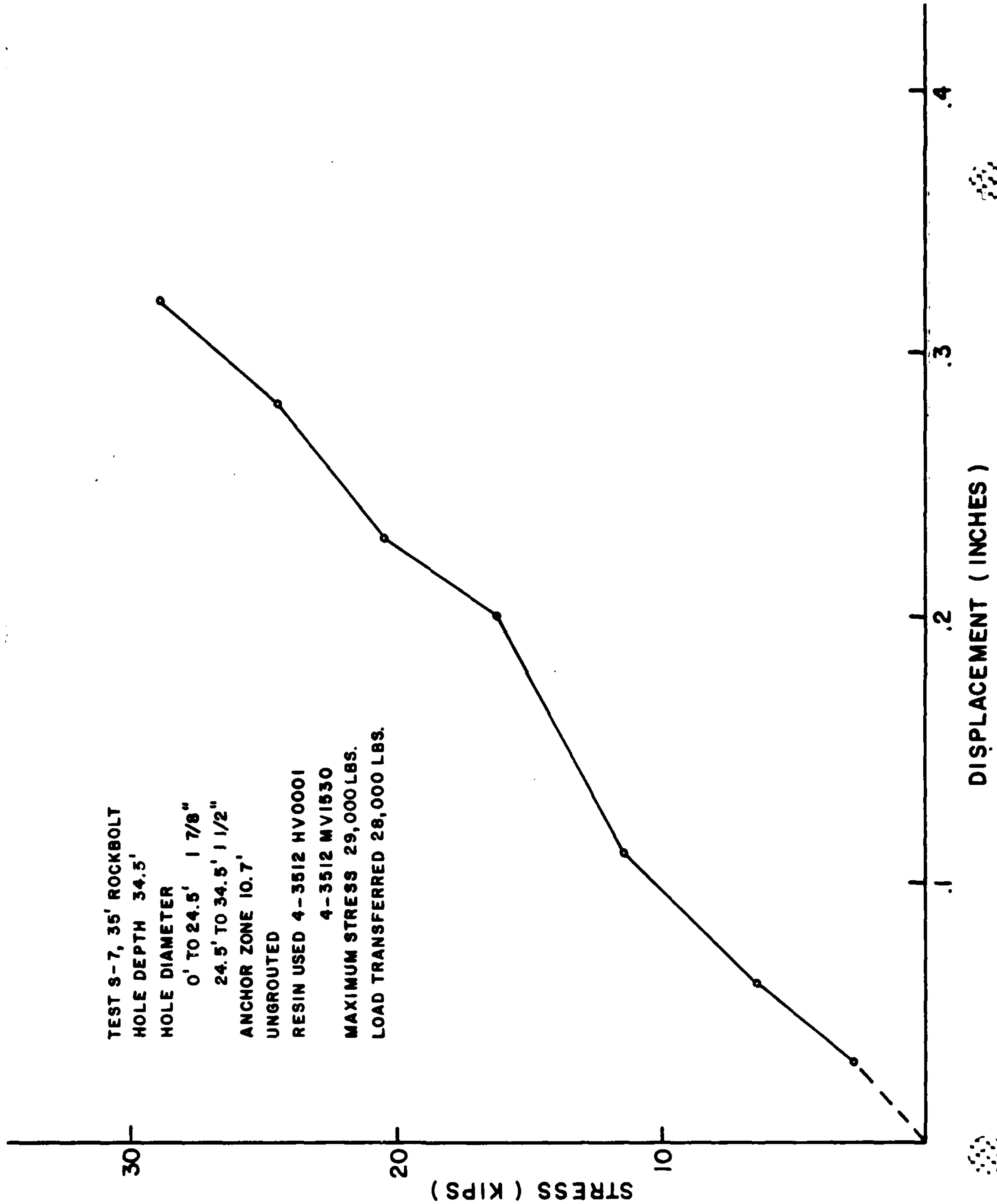
TEST S-5
STRESS DISPLACEMENT CURVE

FIGURE 42



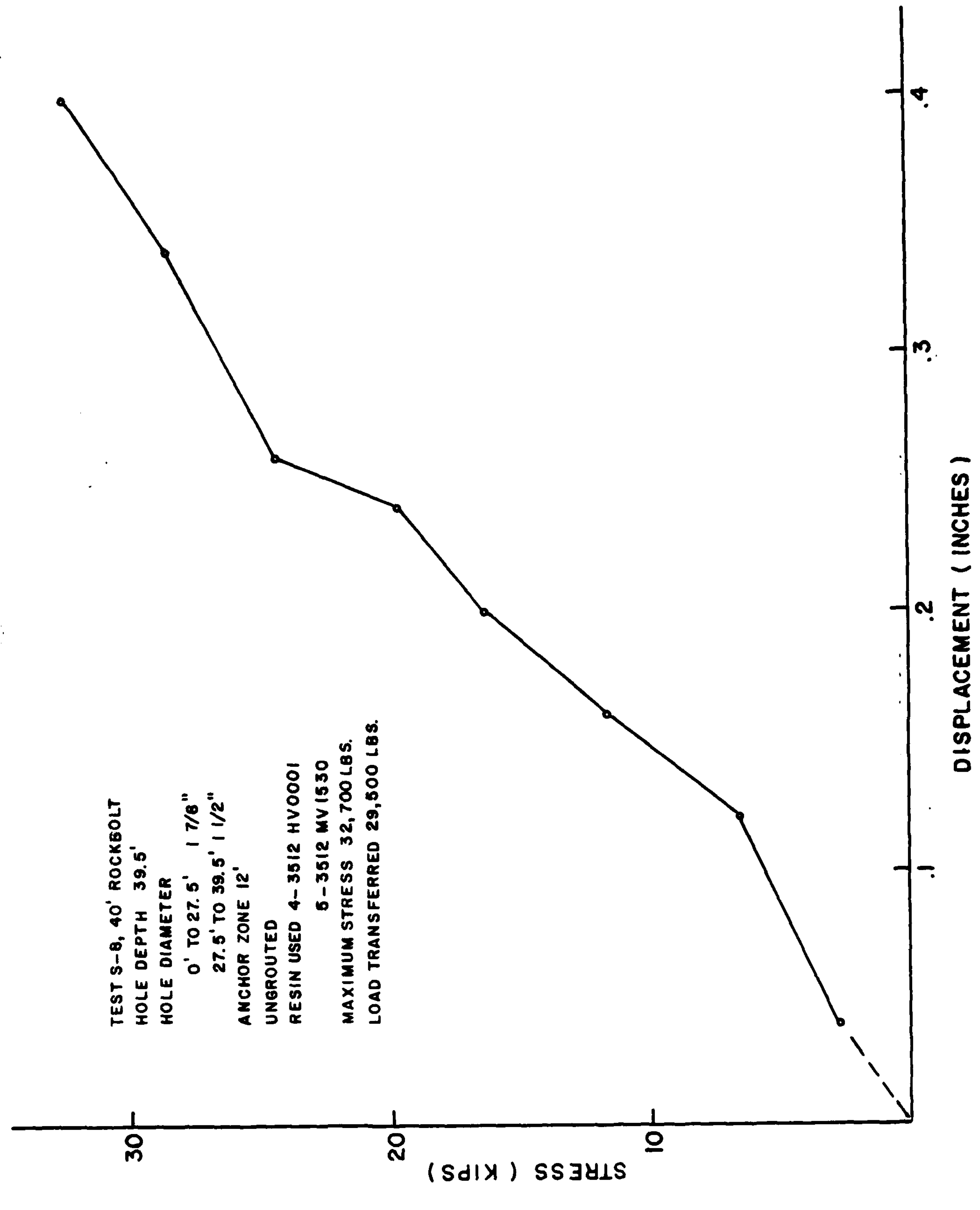
TEST S-6
STRESS DISPLACEMENT CURVE

FIGURE 43



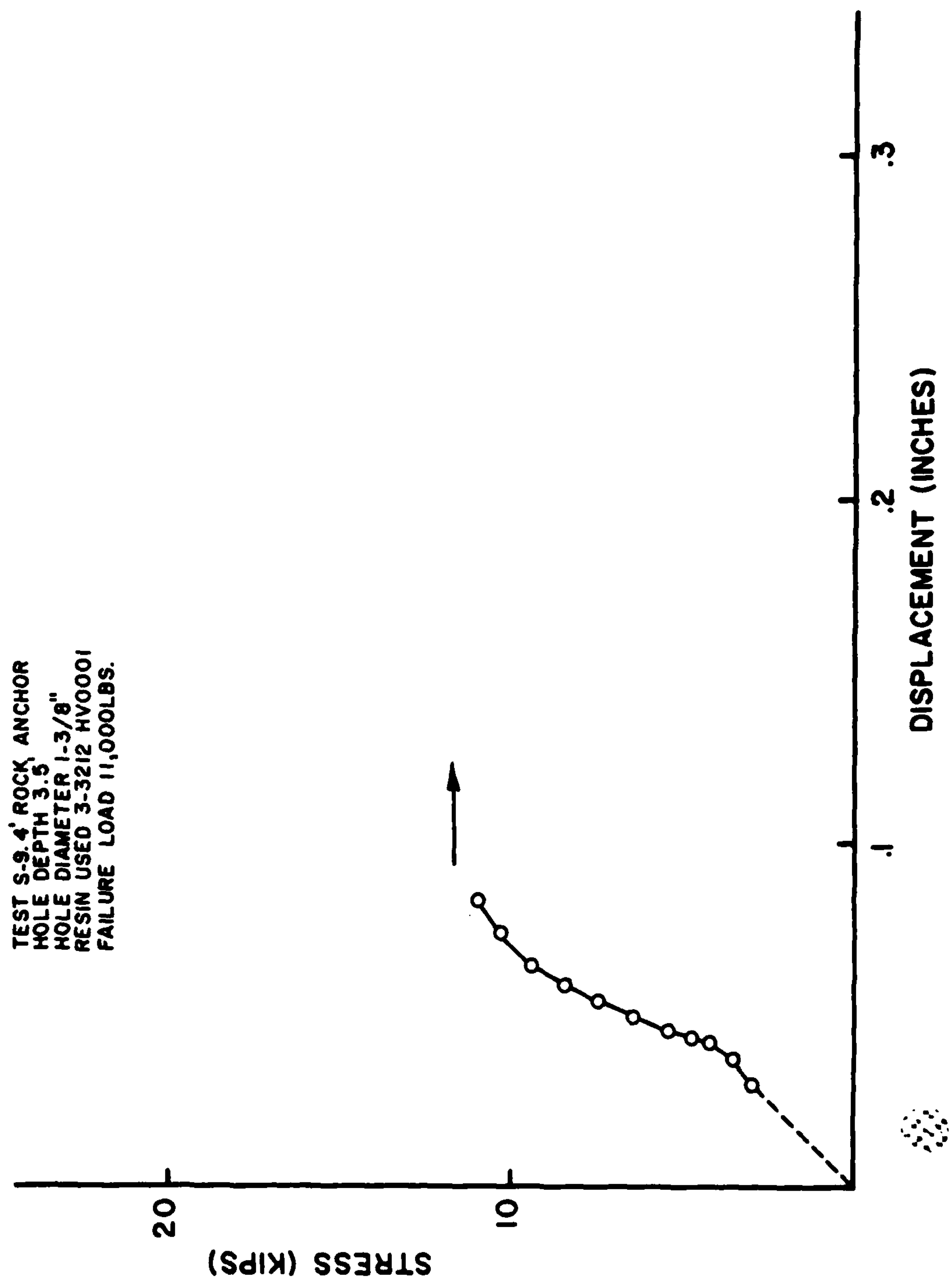
TEST S-7
STRESS DISPLACEMENT CURVE

FIGURE 44



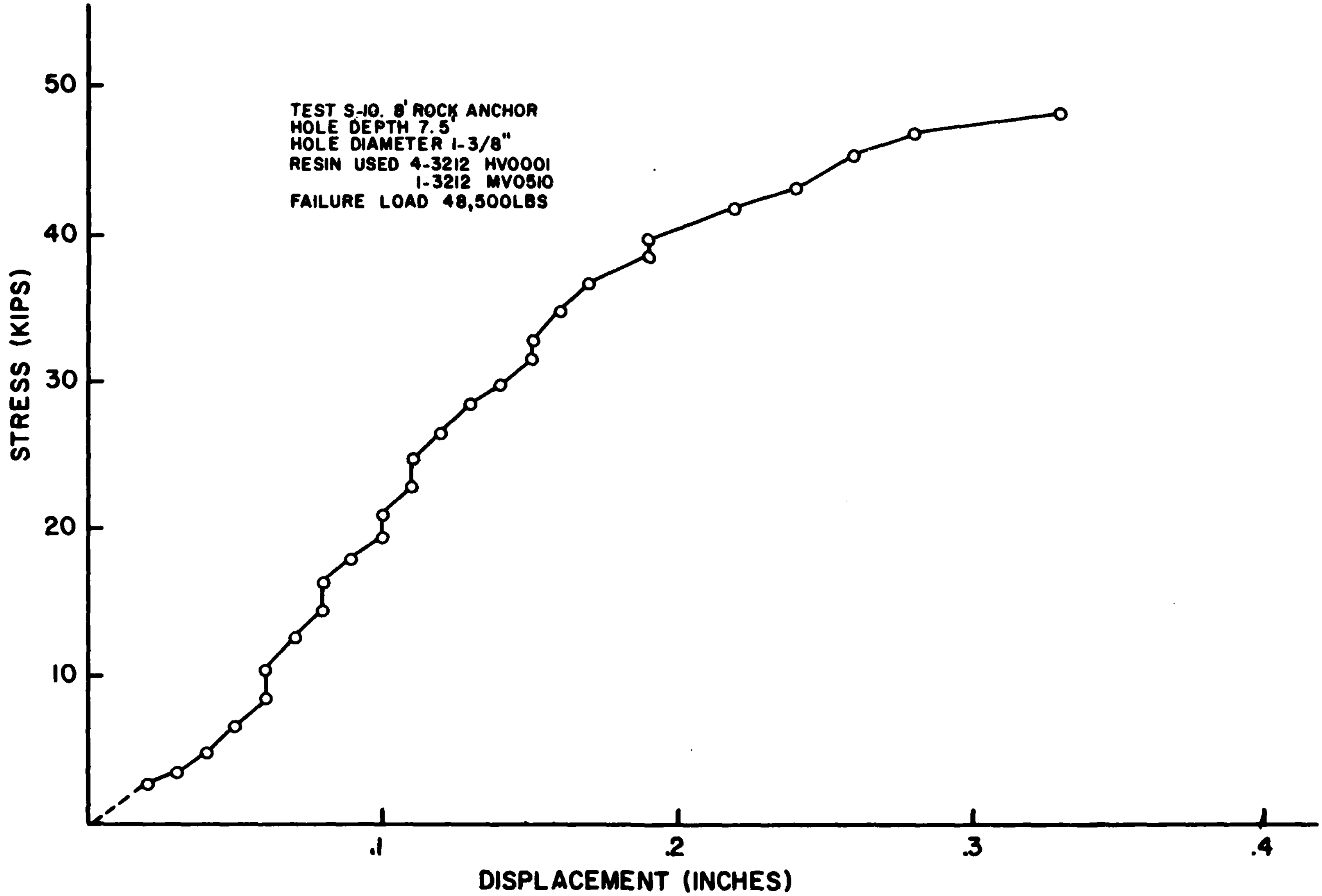
TEST S-8
STRESS DISPLACEMENT CURVE

FIGURE 45



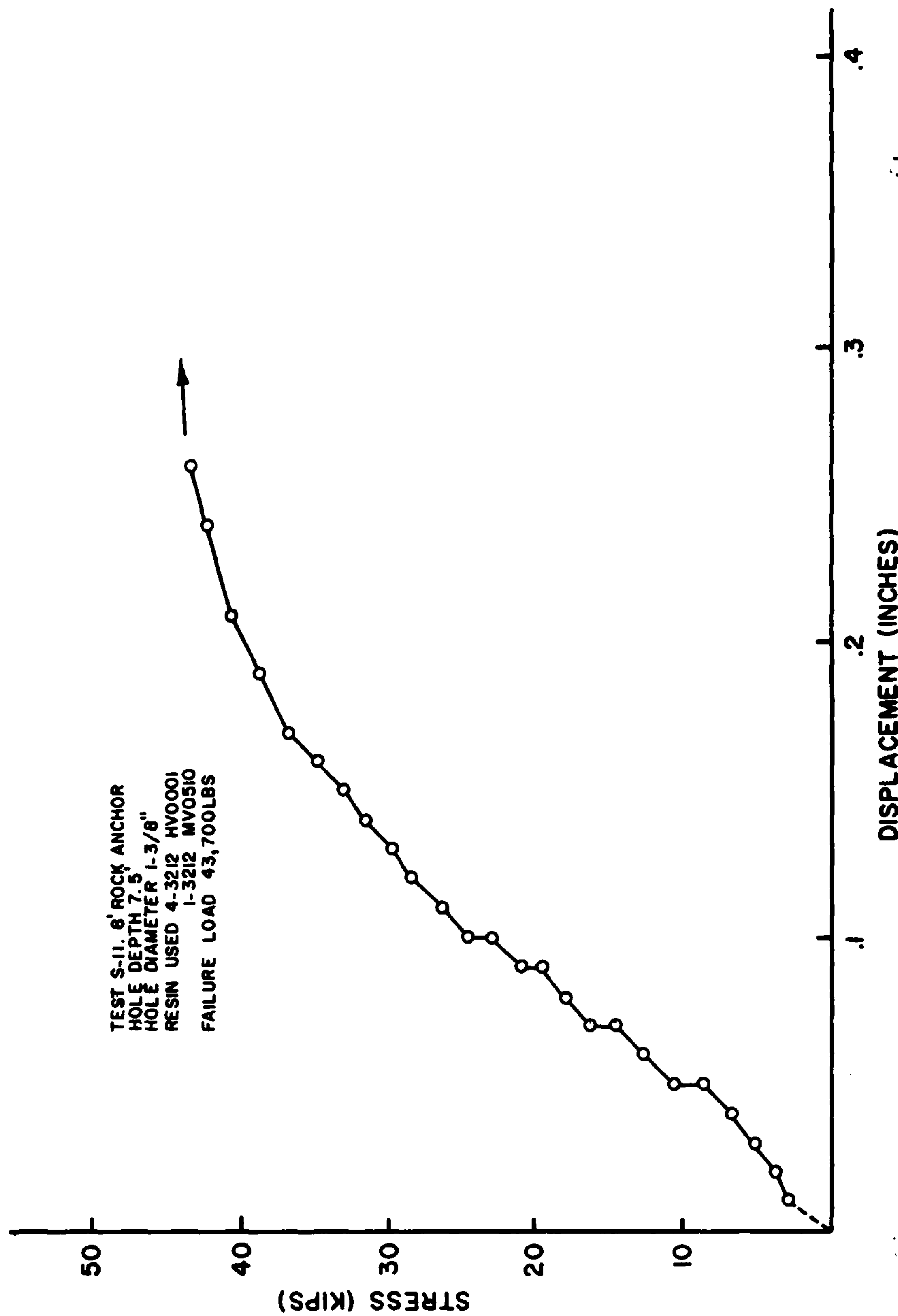
TEST S-9 STRESS DISPLACEMENT CURVE

FIGURE 4



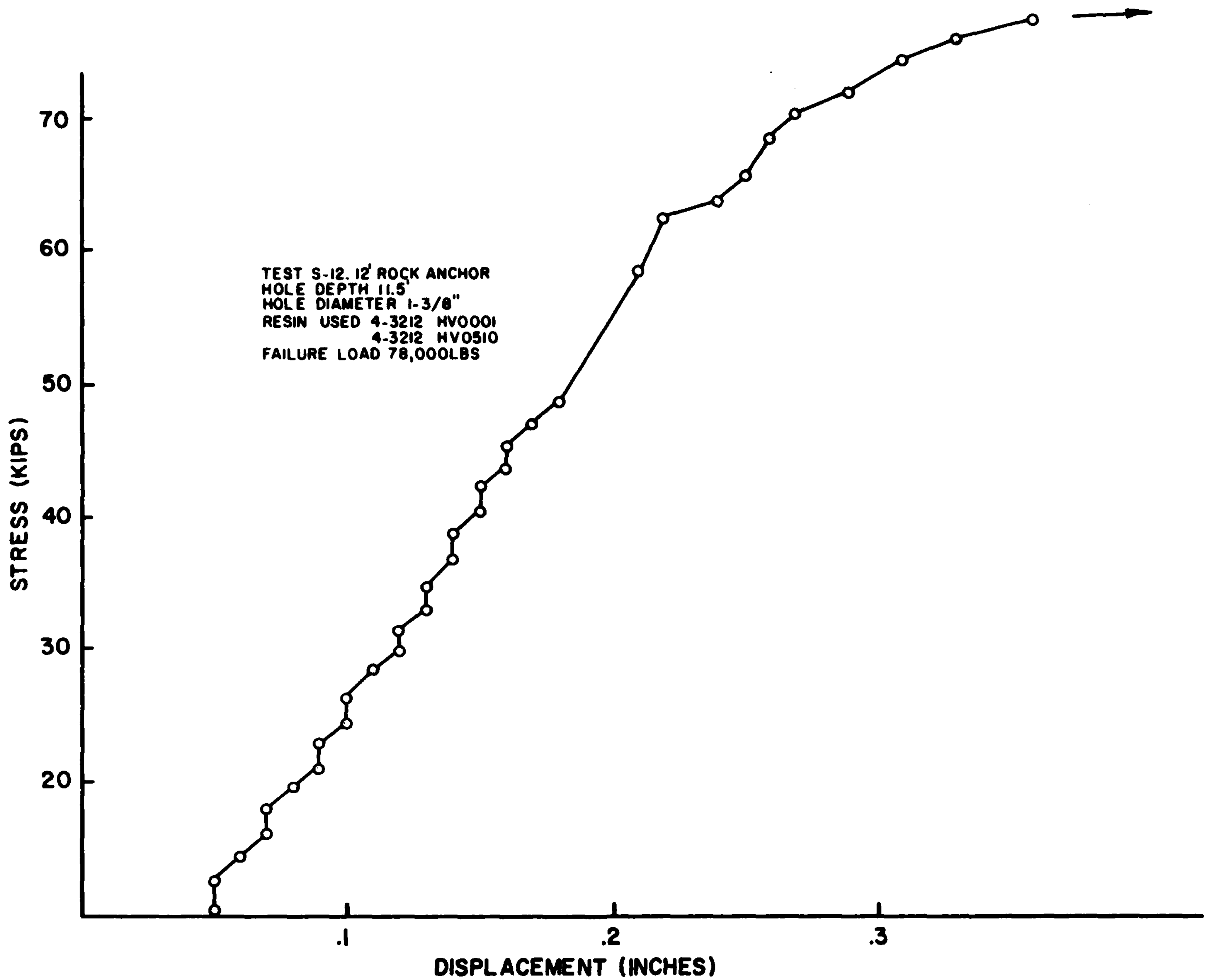
TEST S-10 STRESS DISPLACEMENT CURVE

FIGURE 47



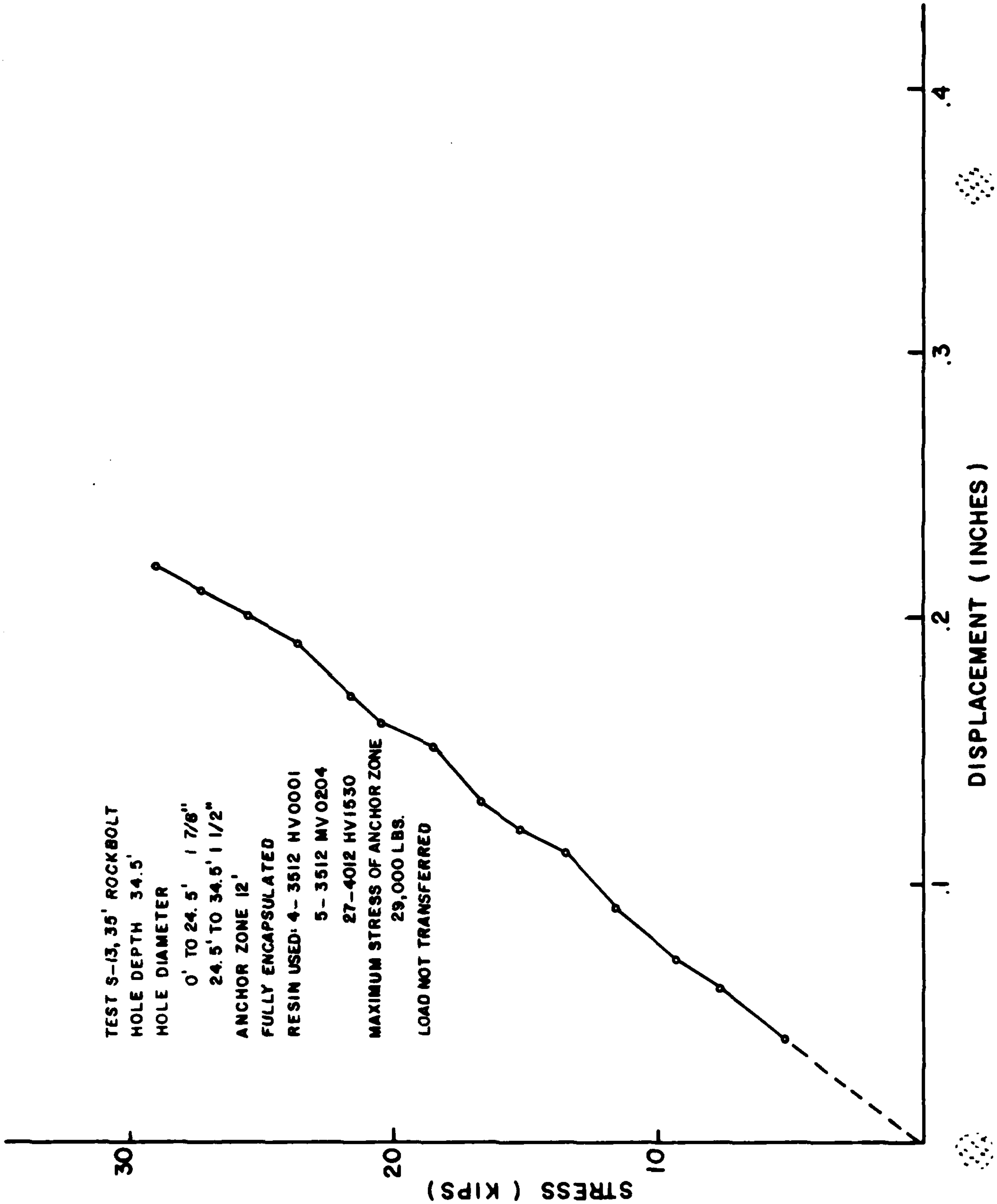
TEST S-11 STRESS DISPLACEMENT CURVE

FIGURE 48



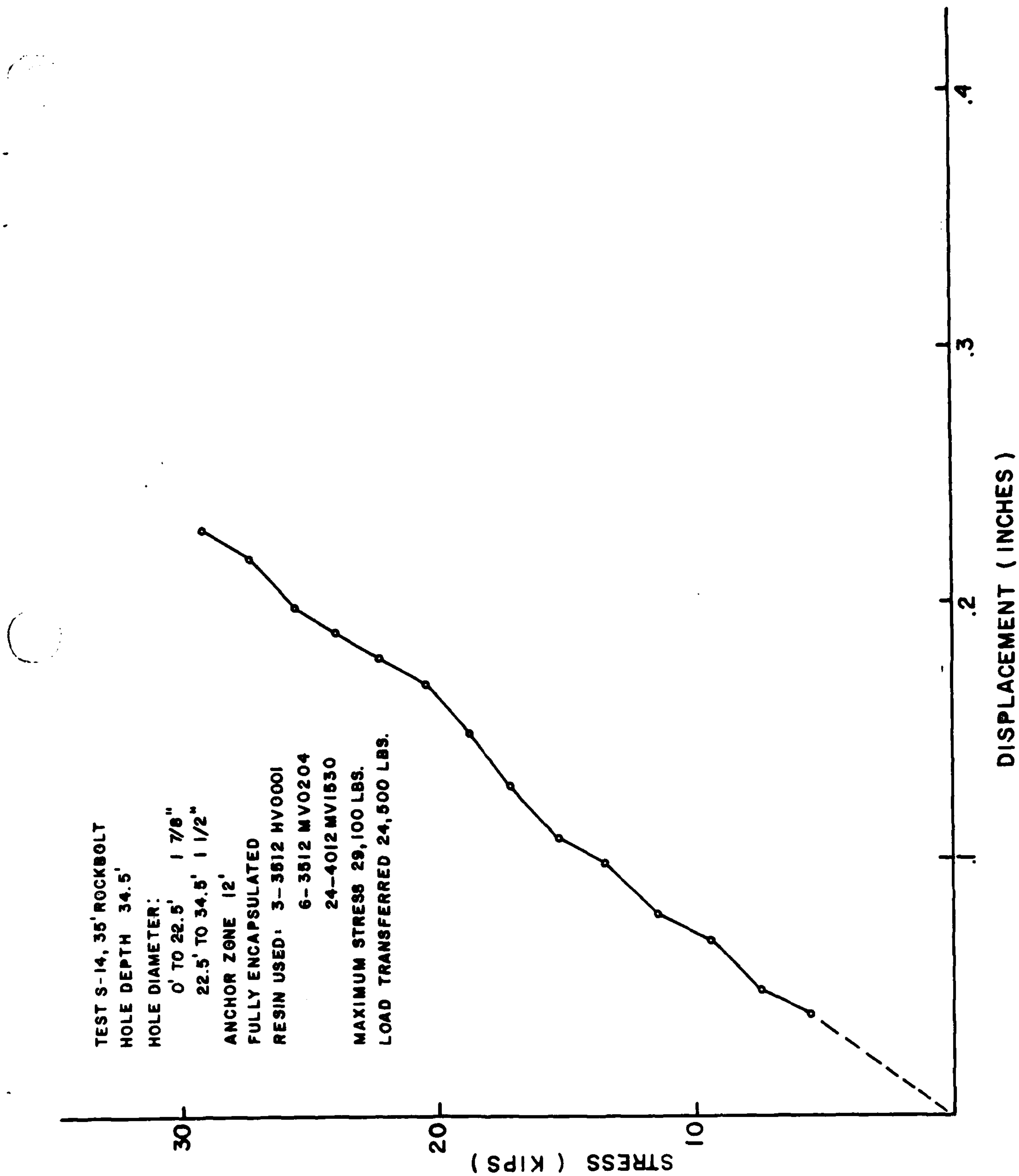
TEST S-12 STRESS DISPLACEMENT CURVE

FIGURE 49



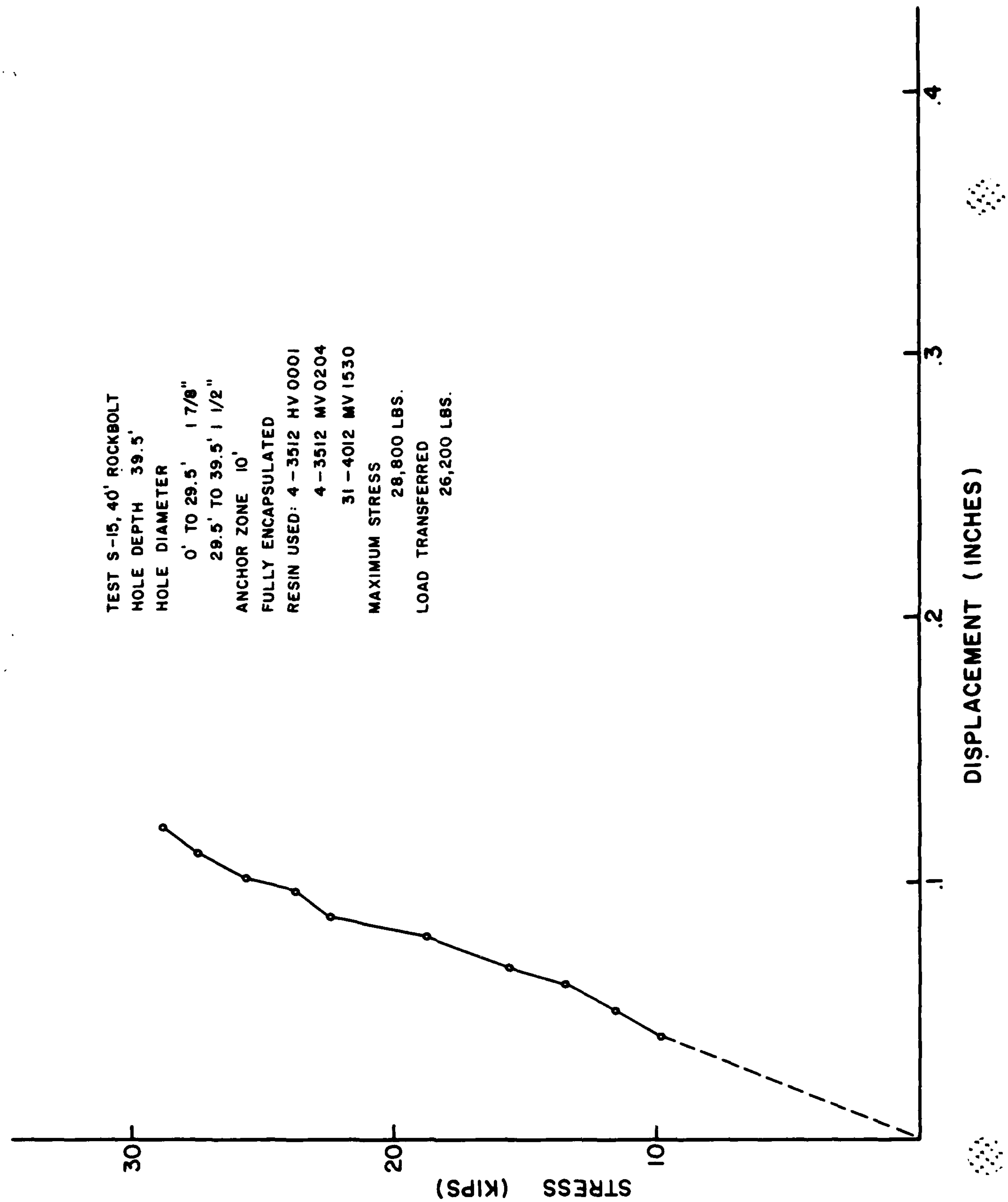
TEST S-13
STRESS DISPLACEMENT CURVE

FIGURE 50



TEST S-14
STRESS DISPLACEMENT CURVE

FIGURE 51



TEST S-15
STRESS DISPLACEMENT CURVE

FIGURE 52

TEST S-16 STRESS DISPLACEMENT CURVE

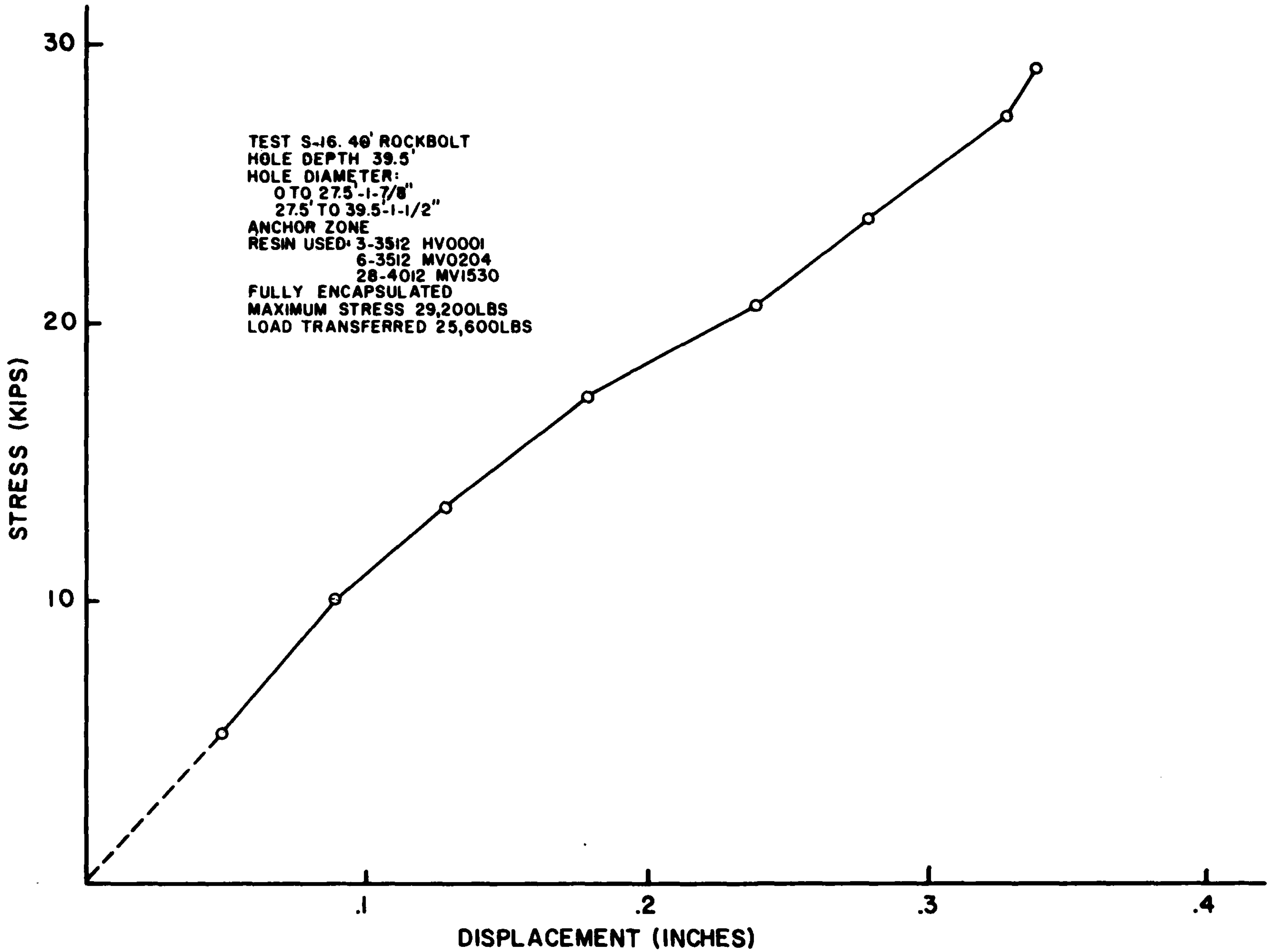
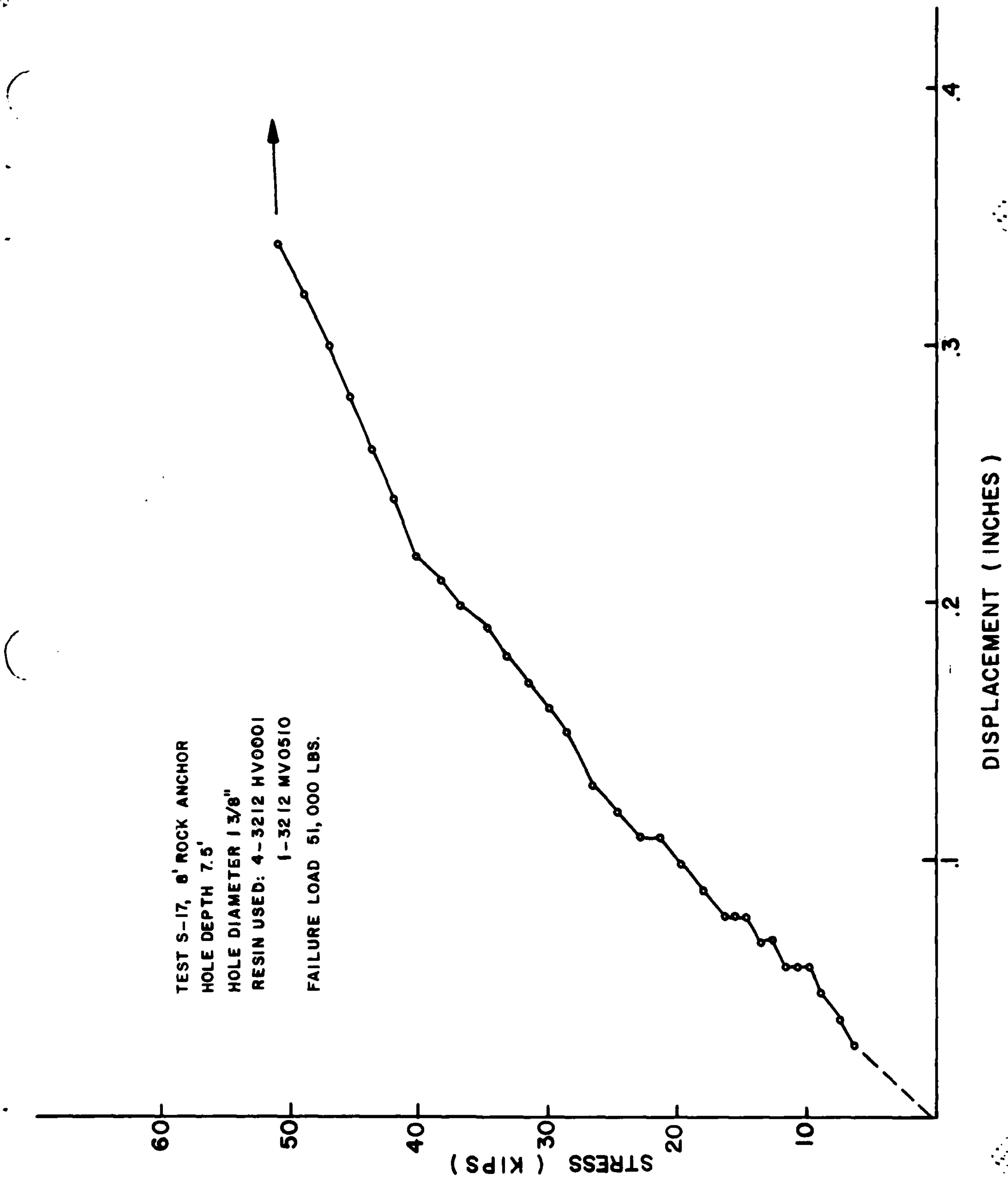
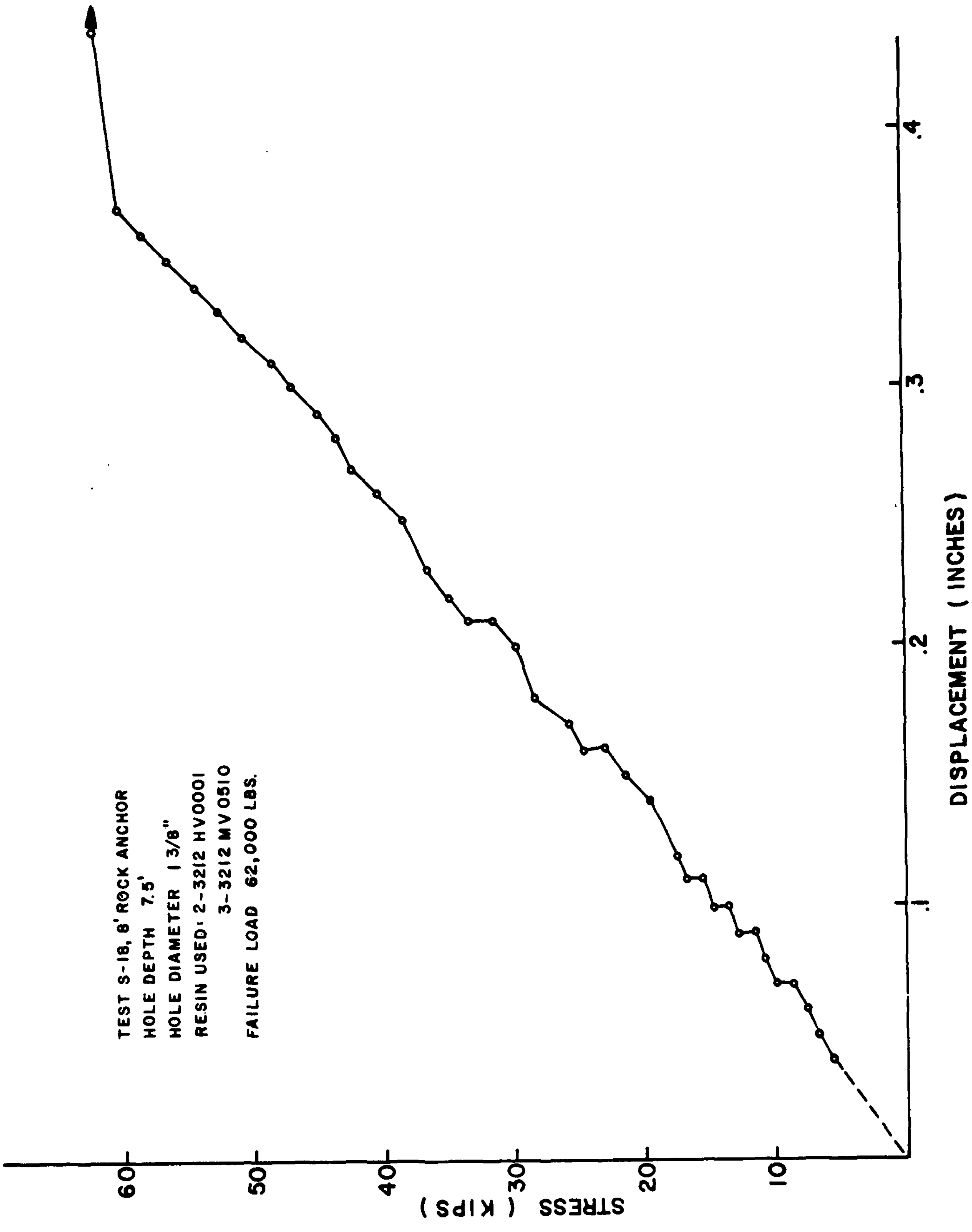


FIGURE 53



TEST S-17
STRESS DISPLACEMENT CURVE

FIGURE 54



TEST S-18
STRESS DISPLACEMENT CURVE

FIGURE 55

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8. Inland-Ryerson Construction Products Company, Catalog 40-30, "Resin Anchored Prestressed Rockbolts," April 1974.

9. Pearson, L. L., "Selfix/Celtite System," Technical Notes, 21.2, 1972.

10. Reed, John J., "Grouted Bolts for Faster Rock Stabilization." Reprinted from The Mines Magazine, March 1970.

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B. Stilling Basin Anchors

1. Introduction

The contract specifications for the stilling basin anchor program required the Contractor to perform an anchor test program and to install 287 permanent anchors as shown on Drawing No. 89A/2. The specifications required a test program to ensure that the design requirements for the permanent anchors were satisfied. The purpose of the stilling basin anchors was to anchor the stilling basin concrete slab securely to the shale foundation. Massman Construction Co. performed the entire operation except for the drilling of the permanent anchor borings which was done by Test Drilling Service Company. Work began in May 1976 and was completed on 14 September 1976.

2. Anchor Test Program

The Contractor began work in May 1976 and completed the program on 6 July 1976. The anchor test program consisted of drilling eight 6-inch diameter borings 21 feet in depth from El. 503± feet NGVD to El. 482± feet NGVD with an Ingersol Rand CM 250 air drill (refer Table No. 2 for exact locations). The boring was cleaned with compressed air and partially backfilled with non-shrink grout (mix design: 94 lb. cement, 124 lb. sand, 39 lb. water and 0.75 lb. Interplast N). The Contractor installed the anchor (No. 18 rebar) and completed backfilling the anchor boring. It was determined after tension testing that Test Anchors Nos. 1, 2 and 3 were improperly installed. The anchor borings were not partially backfilled with grout prior to installation of the anchor as required by the contract specifications. As a result of the improper installations, the Contractor was directed to install two additional anchors (Test Anchors Nos. 7 and 8) for testing.

Prior to the tension tests, calibration of pressure gages and dial indicators was performed by Industrial Testing Laboratories, Inc., and Pittsburgh Testing Laboratory, both of St. Louis, Missouri. The next step of the test program was to tension the anchors with a 50-ton hydraulic jack as shown on Plate No. 1. Stressing of the anchor did not commence until at least seven days after grouting. The testing sequence of the anchor was as follows:

a. Add one dial micrometer to measure deformation between reference angle and bottom of jack support.

b. Increase tension on test anchor in 1,000-pound increments and record deformation of all dial micrometers (per the contract specifications).

c. Tensioning shall not proceed above the following levels until pressure indicators show no loss of jack force for a period of two minutes 2,000 psig (40,000 pounds), 2,500 psig (~50,000 pounds), 3,000 psig (~60,000 pounds), 3,500 psig (~70,000 pounds), 4,000 psig (~80,000 pounds), 4,500 psig (~90,000 pounds). Deformation and pressure (force) shall be recorded each time jacking is required immediately prior to and after jacking to obtain a no loss condition.

d. If pressure indicator shows a loss of jack force at 5,000 psig (~100,000 pounds), the test anchor shall be jacked back to the 5,000 psig level a minimum of three times with a 5-minute waiting period between each reapplication of pressure (force). Deformation and pressure readings shall be taken prior to and after jacking.

e. After completion of the above phase, the jacking force shall be reduced to nearly zero, wait five minutes then increase the load to 5,000 psig and wait five minutes. This procedure shall be repeated a minimum of five cycles. Deformation and pressure readings shall be required before and after loading or unloading.

Upon completion of tension testing, it was found that the test anchors did not fulfill the necessary design requirements (refer Plates Nos. 2, 3 and 4 for test results). Consequently the Contracting Officer directed the Contractor to pull Test Anchors Nos. 1, 3, 6 and 8 in order to determine the cause of failure. Test Anchors Nos. 2, 4, 5 and 7 were cutoff and left in place. The Contractor began extraction operations with Test Anchor No. 6 pulling to the capacity of two 50-ton hydraulic jacks with no success. Removal of Test Anchor No. 6 required the drilling of five 3-inch diameter borings around the anchor and the use of the jacks to pull the anchor. This was found to be a slow process; thus, when the load dropped to approximately 30 tons, the Contractor utilized a motor crane to remove the anchor and grout column. The resultant boring was backfilled with non-shrink grout. Because of the difficulties encountered in removing Test Anchor No. 6, it was decided to forego extracting Test Anchors Nos. 1, 3 and 8. The anchors were cut off and left in place.

Due to the failure of the test anchors, on 21 June 1976, the Contractor was directed to suspend further anchor testing until the anchor system was redesigned. On 24 June 1976, the Contractor was directed to install, grout and tension test three additional test anchors with a tip elevation of 463± feet NGVD. This increased the anchorage length from 21 feet to 40 feet (refer Table No. 2 for location and detail).

As a result of the three additional anchor tests, it was determined that the anchorage length (40 feet) was adequate (refer Plate No. 5 for test results). Accordingly, the tip elevation of all permanent stilling basin anchors was lowered to El. 463 feet NGVD. This change in the contract necessitated the issuance of Modification No. P00067, which in addition to the increased anchorage length, required the Contractor to immediately plug the anchor boring upon completion of drilling in order to exclude surface water and air circulation and to install the anchor within 60 hours of drilling the boring. The design elements felt this would give greater grout-shale shear strength.

3. Permanent Anchors

Drilling began on 4 August 1976 and was completed on 2 September 1976. Grouting of the anchors commenced on 6 August 1976 with the final anchor being grouted on 3 September 1976. The anchors were grouted at least six days prior to the placement of concrete as required by the contract specifications.

The installation procedures were the same as those used during the anchor test program with two exceptions. The first was the absence of anchor tension testing upon completion of grouting and the second was the drilling of the 6-inch diameter anchor borings by Test Drilling Service Company utilizing an Ingersol Rand I-4 Truck-Mounted Drill Rig. This was necessary due to the increased anchorage length of the borings (from 21 feet to 40 feet).

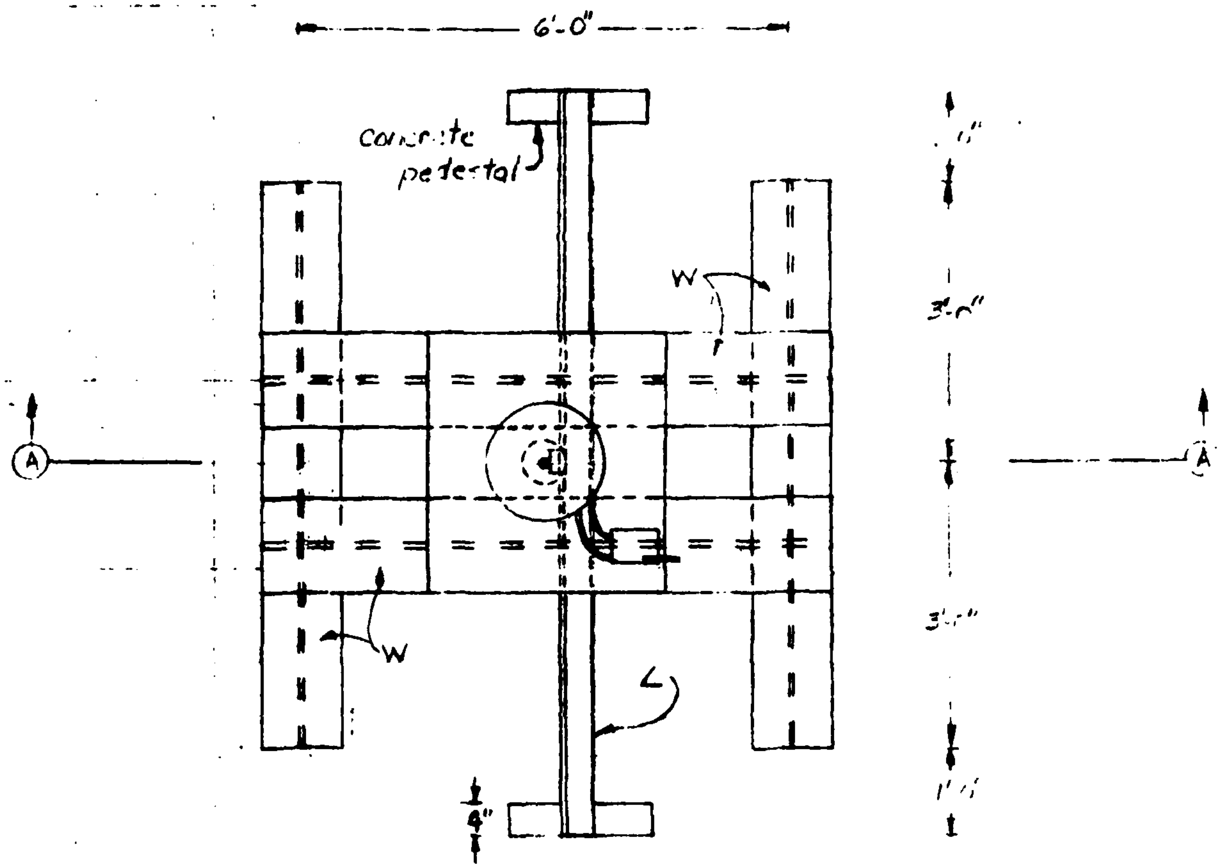
LOCATION OF STILLING BASIN TEST ANCHORS

Test No.	Block	Station	Offset	Tip Elevation Feet NGVD	Date Grouted	Date Tested	Remarks
** 1	B3	4+45	125' D/S	482±	5/26/76	6/09/76	Failed Tension Test
** 2	B2	4+28	145' D/S	482±	5/26/76	6/09/76	Failed Tension Test
** 3	B11	4+78	176' D/S	482±	5/26/76	6/09/76	Failed Tension Test
4	B18	5+09	211' D/S	482±	6/08/76	6/17/76	Failed Tension Test
5	B15	4+28	205' D/S	482±	6/08/76	6/17/76	Failed Tension Test
* 6	B12A	5+07	184' D/S	482±	6/10/76	6/17/76	Failed Tension Test
7	B12B	5+00	166' D/S	482±	6/10/76	6/18/76	Failed Tension Test
8	B24	4+71	267' D/S	482±	6/10/76	6/18/76	Failed Tension Test
9	B17	4+75	200' D/S	463±	6/24/76	7/02/76	Passed Tension Test
10	B21	3+92	258' D/S	463±	6/28/76	7/06/76	Passed Tension Test
11	B6	5+31	132' D/S	463±	6/28/76	7/06/76	Passed Tension Test

*Pulled to determine cause of failure

**Improperly installed

TABLE NO. 2

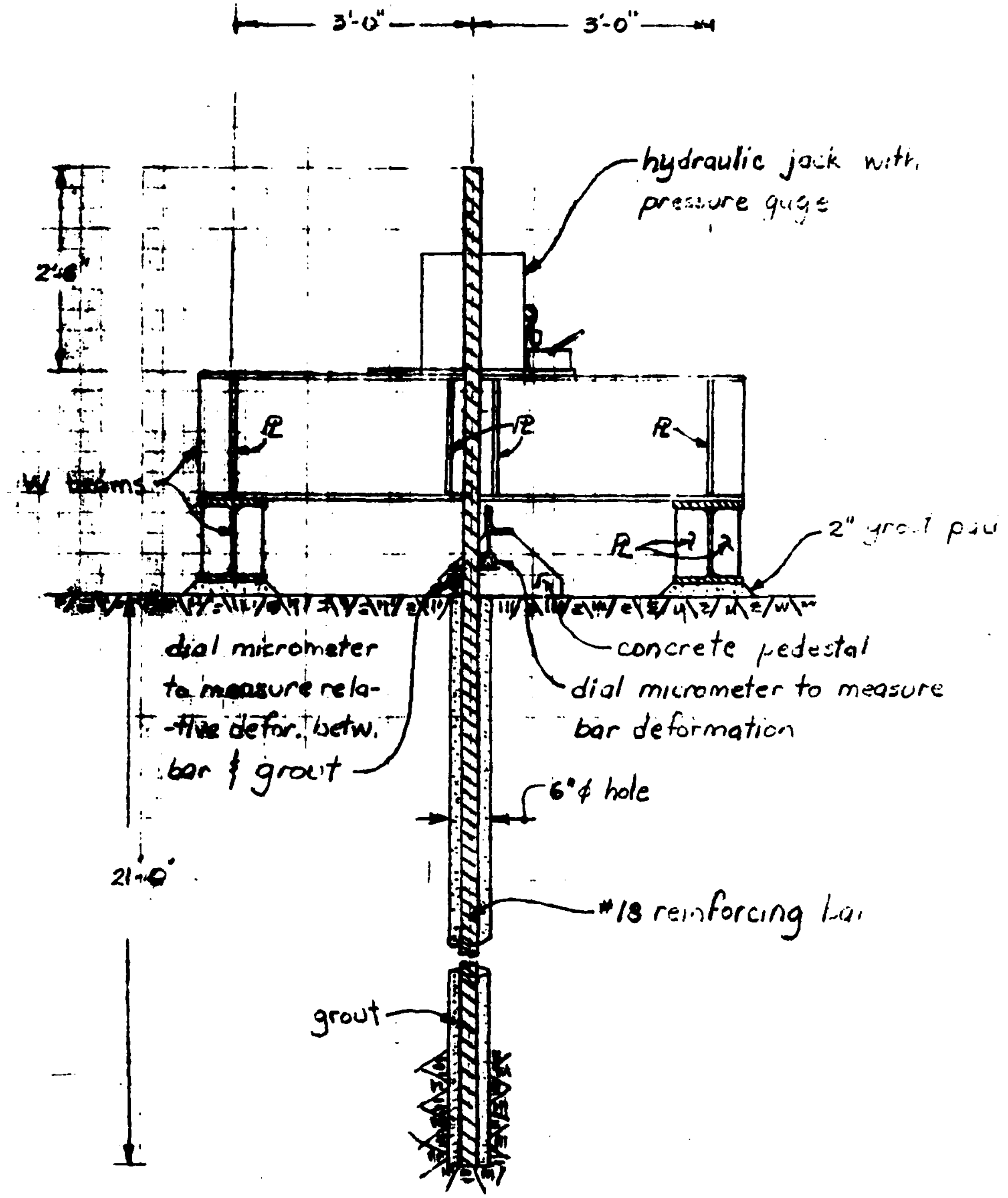


Plan View

(not to scale)

PLATE NO. 1
SHEET NO. 1

TEST ANCHOR BARS, TEST APPARATUS



Section A-A
(not to scale)

PLATE NO. 1
SHEET NO. 2

TEST ANCHOR BARS, TEST APPARATUS

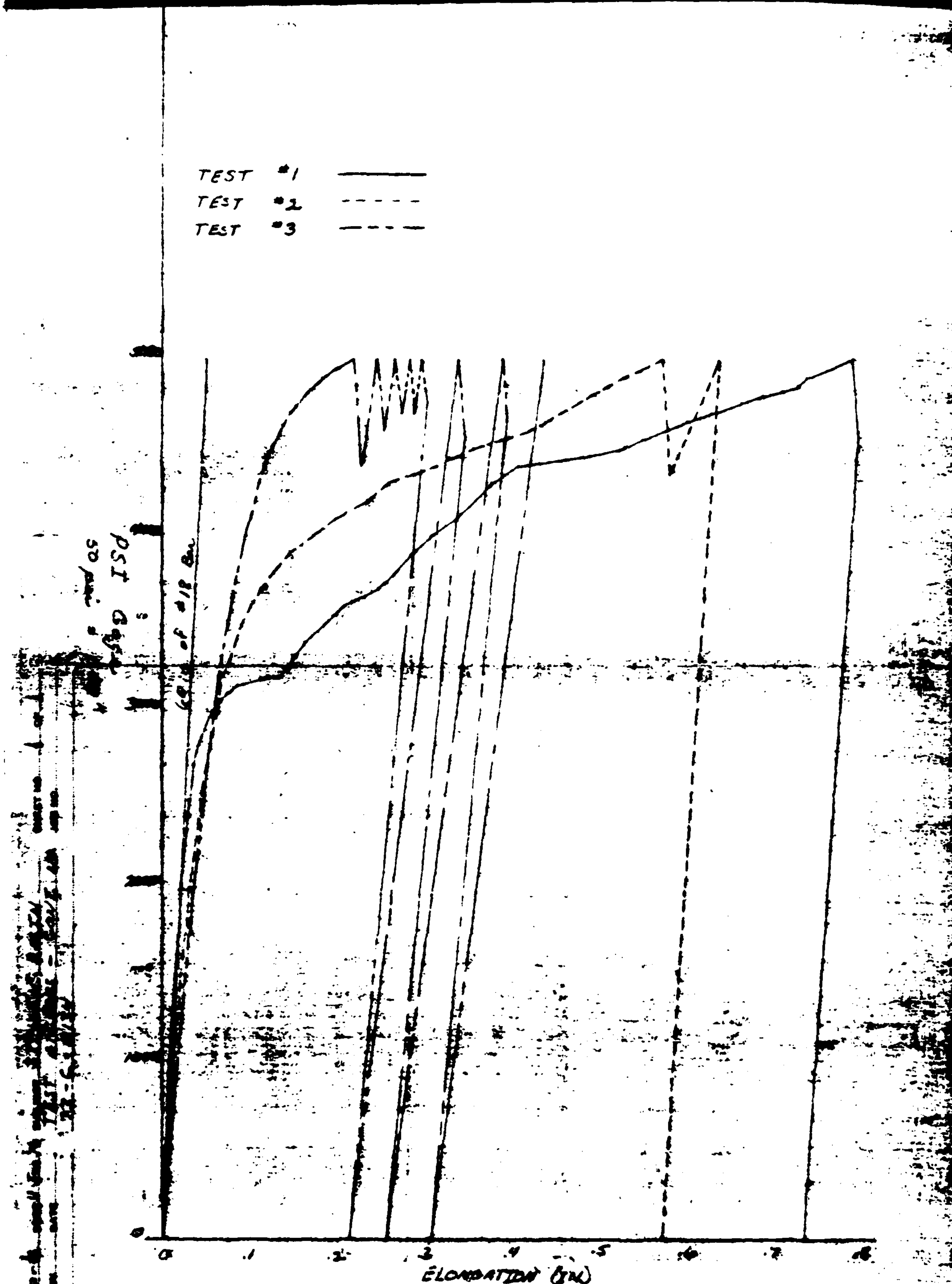
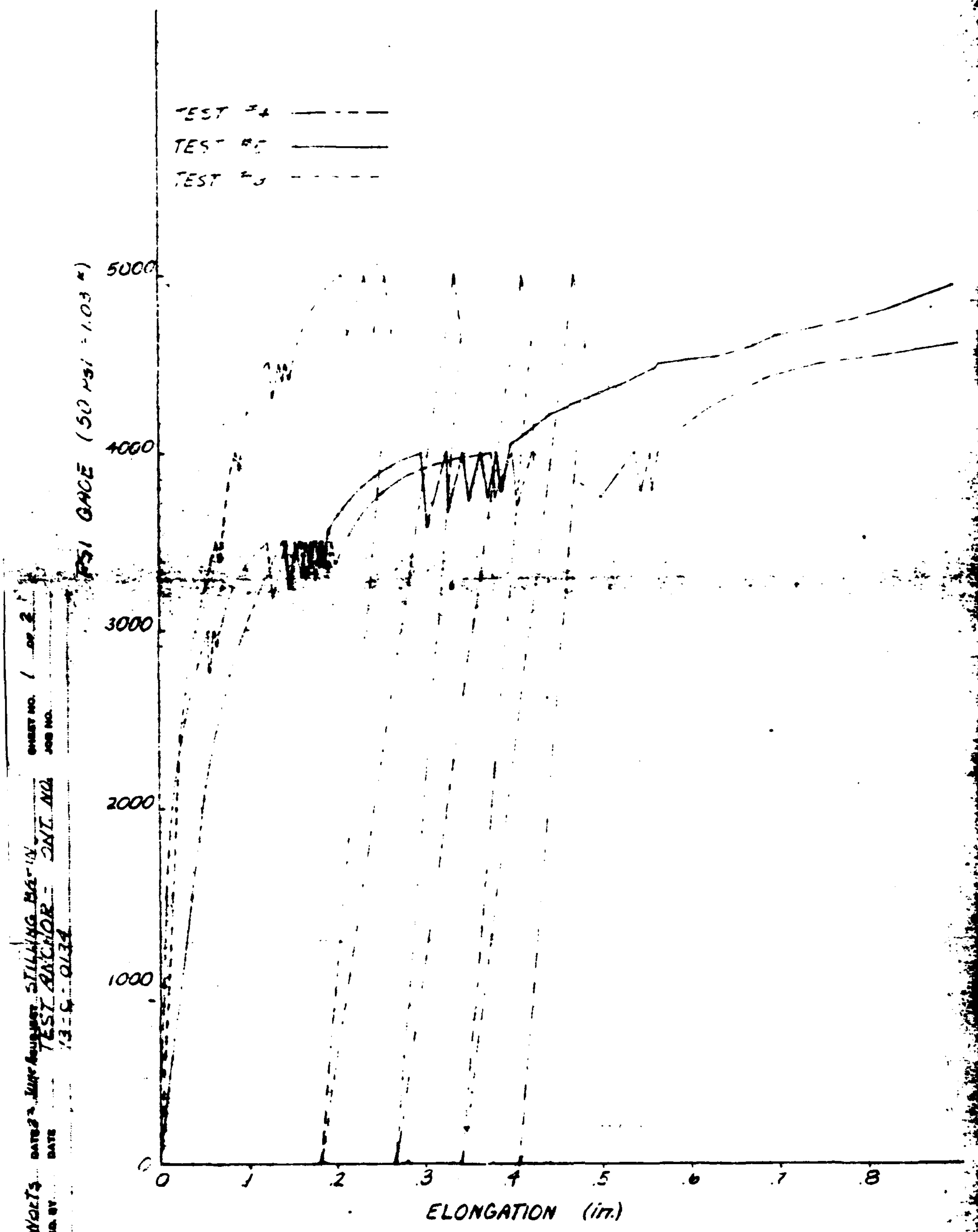


PLATE NO. 2



TEST #4
TEST #5
TEST #3
PSI GAGE (50 KSI = 1.03 K)
5000
4000
3000
2000
1000
0
0
.1
.2
.3
.4
.5
.6
.7
.8
ELONGATION (in.)
BY WOLTS
SUBJECT STILLING BASIN TEST ANCHOR - 2nd T NO
SHEET NO. 1 OF 2
JOB NO. 13-C-0134

PLATE NO. 3

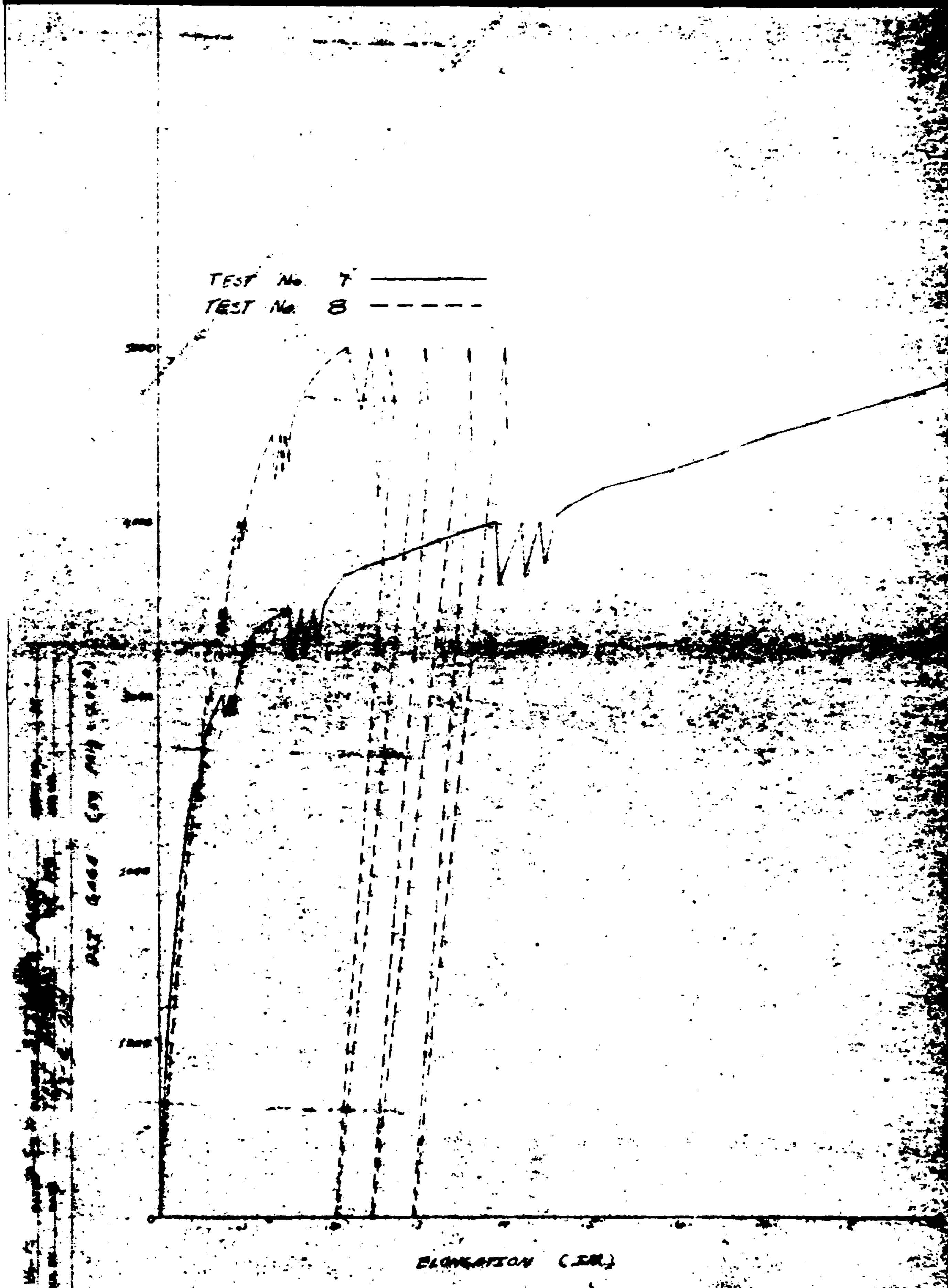


PLATE NO. 4

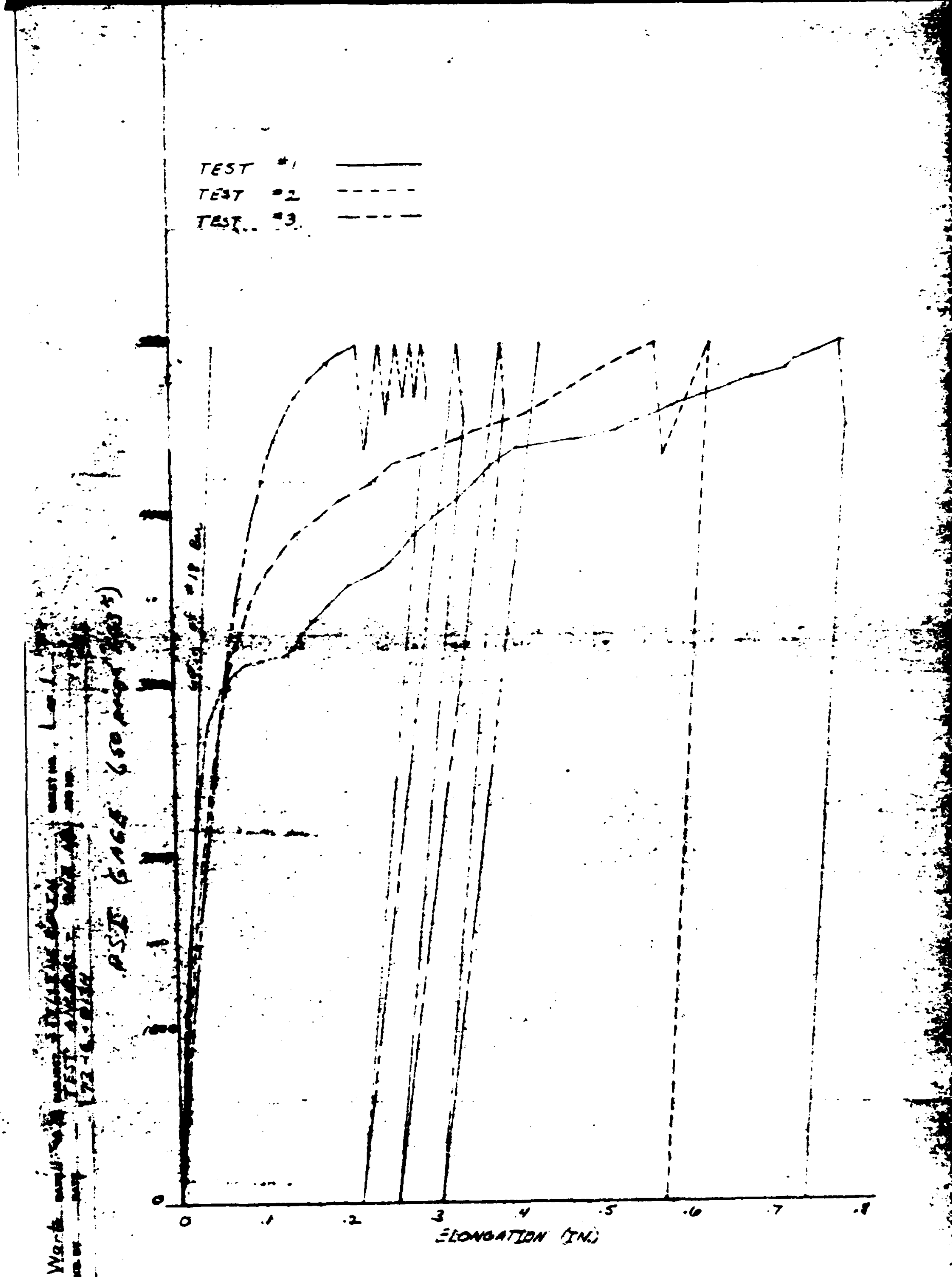
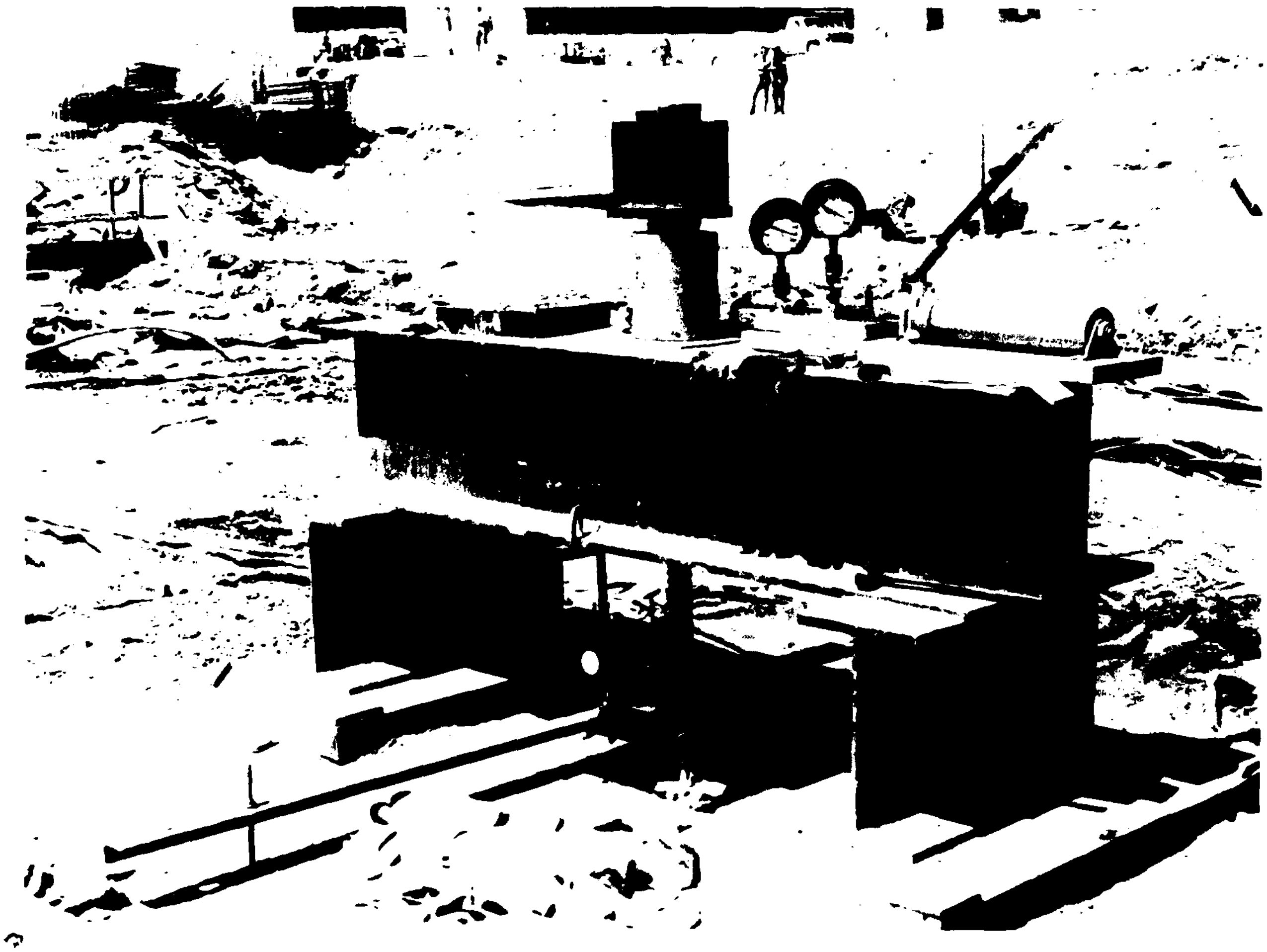
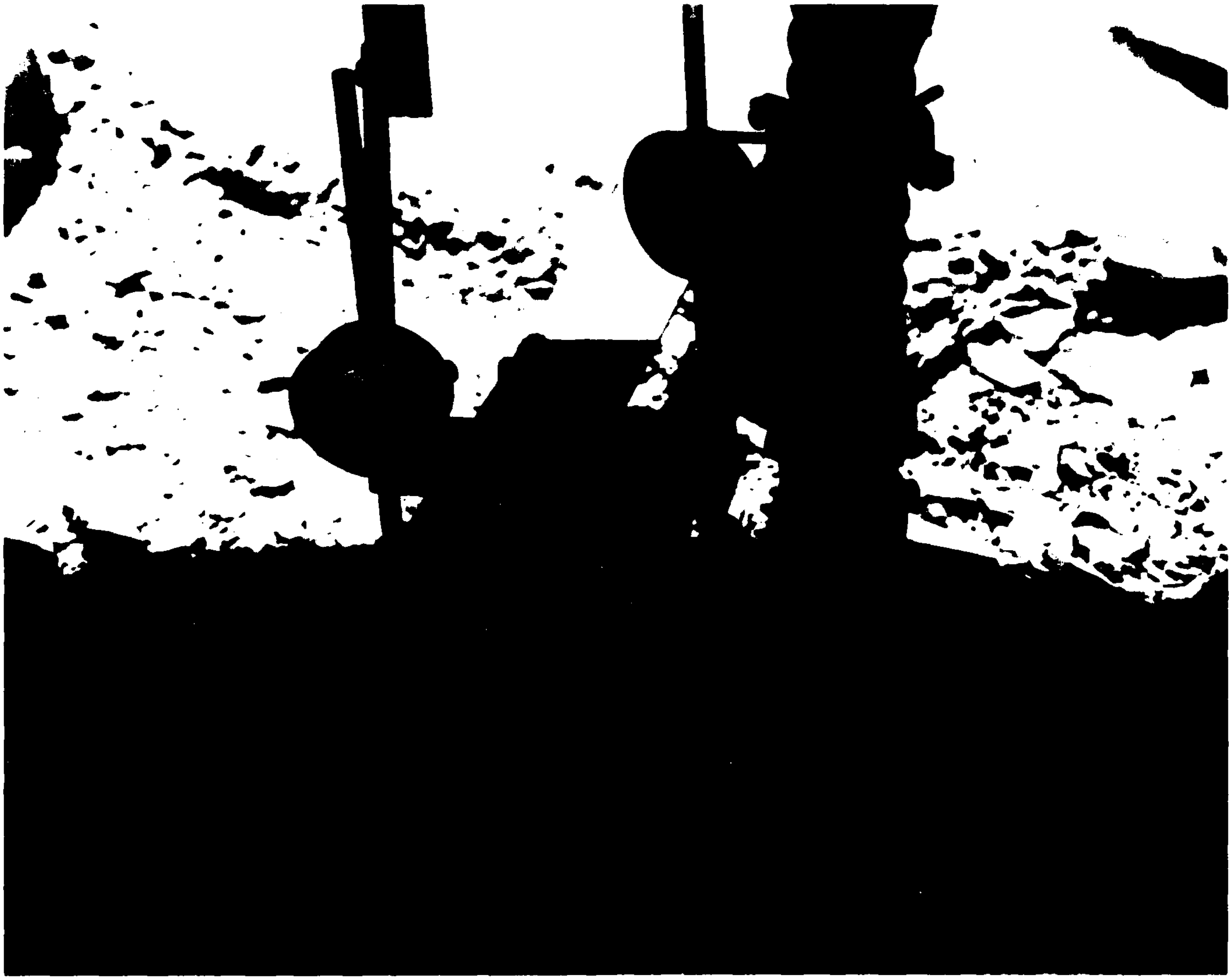


PLATE NO. 5



Typical arrangement of 50-ton hydraulic jack with dial indicators and pressure gages used for anchor test program in June 1976.



Arrangement of dial indicators for anchor test program. Left to right: (1) Beam, (2) Bar-(anchor), and (3) Grout.

SECTION 8

POST TENSIONING

A. General

The post tensioning program required the drilling of 32 anchorage holes into the monolithic limestone foundation, installation of pipe forms during powerhouse concrete placements, installation of post tensioning units, grouting of the rock anchors and the tensioning of units. The purpose of such tendons was to induce a compressive stress into the upstream face of the powerhouse and intakes, thereby eliminating the need for an excessive amount of tension reinforcement in the upstream concrete face.

B. Drilling

The first post tensioning boring was drilled during the latter part of August 1974, with the remainder of the borings being drilled during the months of November and December 1974, by Continental Drilling Company. The 8-inch diameter borings were collared in the Louisiana Limestone at El. 465 feet NGVD and the inclination of the borings was either 5.95° upstream, normal to axis, or 90°. Upon completion, the borings were then washed (30 minutes to 1 hour), pressure tested (15 minutes) and, if necessary, grouted. The specified depth of the post tensioning boring was increased by Modification No. P00042 due to inconclusive data from the specified anchor test which indicated possible anchor slippage (refer Drawing No. 92/2 for the general location and original or modified depth of the A and B post tensioning borings).

Grouting of the post tensioning borings was performed by utilizing a combination of gravity and pressure grouting techniques. The borings were initially filled with neat grout to the surface and then injected under pressure by utilizing a Chemgrout Grout Plant (Model No. CG-500, 7 cubic foot mixes). The grout mixes varied from a water-cement ratio of 2:1 to a water cement ratio of 0.6:1 (typically 2:1 to 1:1) with a maximum gage pressure of 8 psi. Each post tensioning boring was grouted in a single stage with the packer being placed at the collar of the hole. Grouting refusal criteria was the placement of 0.5 cubic foot or less for a 5-minute period. The maximum grout take (31 bags) occurred in borings at Station 7+38.5, Offset 45.58 feet upstream. Upon completion of grouting, the borings were redrilled, washed and plugged until concrete placement was started. A list of grouted/redrilled post tensioning borings is shown on Table No. 1 at the end of this Section.

The initial segment of each post tensioning casing (20-gage galvanized) was seated by reaming the top 8 inches of each foundation boring and then sealed by the placement of grout. The remainder of the casing for each monolithic concrete placement was not allowed to vary more than 0.5 inch in a 10-foot section or 2 inches over the total height of the structure.

C. Installation

Installation of the post tensioning units was performed by Inland Ryerson Company of Melrose Park, Illinois. The post tensioning units were designated as either "A" Unit (128 strands) or "B" Unit (109 strands) depending upon the number of strands. Each strand of the "A" or "B"

Units consisted of a prestressed steel which was an uncoated seven wire stress relieved strand for prestressed concrete. By Modification No. P00089, the top anchorage of the post tensioning units was raised from El. 645 feet NGVD to a blockout 2 feet 5 inches below the top of the concrete except for the vertical tendons located in the spillway pier portion of the pump turbine unit which remained as previously indicated (refer Plate No. 1 for typical installation details and Table No. 2 for the as-built data on tendon installation).

The tendons were coiled in such a manner that the top of the tendon came off the drum first. This was to enable the tendon to be uncoiled from the drum by lifting up and uncoiling the tendon by its top and then lowering it into its hole. As a result of there being nothing available with the necessary reach required to raise the top of the tendon high enough to then lower it into the hole, the Contractor attempted to lower the tendons in the hole by uncoiling them directly from the drum into the hole. A great deal of difficulty was encountered on 10 and 11 October 1978 while attempting to install Tendon Sequence Nos. 32, 29 and 31 in this manner. On 12 October 1978 when attempting to install Tendon Sequence No. 30, the tendon slipped from the drum and free fell into the tendon hole breaking several wires. Following this, a second drum was brought to the job site to allow a tendon to be uncoiled from the first drum onto the second drum then uncoiled from the second drum directly into the tendon hole. Tendon Sequence No. 30 was removed from the site on 23 October 1978 and replaced with a new one.

The dates of tendon installation are listed below.

Date	Tendon Sequence No.
10 October 1978	32 (started)
11 October 1978	32 (completed)
11 October 1978	29 and 31
12 October 1978	30
24 October 1978	27
26 October 1978	30, 23 and 28
27 October 1978	19 and 21
28 October 1978	15 and 17
30 October 1978	9 and 13
31 October 1978	11
1 November 1978	5, 2 and 4
2 November 1978	1, 6 and 3
3 November 1978	8 and 10
4 November 1978	12 and 20
7 November 1978	14 and 22
8 November 1978	16 and 24
10 November 1978	18, 26 and 25

Tendon Sequence No. 25 was placed in Hole No. 30 on 25 October 1978. When the new tendon replacing the broken No. 30 arrived on site, it was placed in Hole No. 25.

In placing Tendon Sequence No. 8 on 3 November 1978, it was found that the grout pipe could not be lowered to the proper depth. Immediately following this, when Tendon Sequence No. 10 was placed, it was found to protrude from the top of the hole. Tendon Sequence Nos. 8 and 10 were measured and it was concluded that the factory numbering had been reversed on them. Tendon Sequence No. 10 was partially pulled and 8 feet of tendon was cut from the top in accordance with the manufacturer's recommendation; the tendon was then field button headed and then replaced in the hole. An attempt was made to pull Tendon Sequence No. 8. While pulling it, the tendon slipped from the drum and free fell into the hole damaging several strands. On 10 November 1978, Tendon Sequence No. 8 was pulled and replaced on 16 November 1978 with a new one.

D. Primary Grouting

Upon completing the placement of the post tensioning tendons, primary grouting (first-stage grouting) of the anchorages of the post tensioning tendons commenced. As per specifications, the post tensioning unit was securely fastened in place to prevent any movement during the grouting. This was accomplished by centering devices placed every 20 feet on the tendon along the weight of the suspended cable itself.

The first-stage grouting used one sack of cement, five gallons of water and one-half pound BBRV expansion agent per batch, and was placed as indicated on Table No. 3.

The grout was placed using a 250 psi Chemgrout Model No. CG-500 Grout Pump and two 7 cubic foot mixers. In checking the depth to top of grout, it was found that additional grout had to be placed in several of the holes to bring it to the required elevation.

E. Stressing

The Contractor started the prestressing operations on 24 November 1978. Each post tensioning tendon was given a sequence number to indicate the order in which it was to be stressed. The sequential number order was changed by Modification No. P00089 as follows: "Tendon stressing will begin with Mark A2 in the middle pier of the Kaplan Unit, then right and left Mark A2 of the Kaplan Unit, Mark A1 in the middle pier of the Kaplan Unit, thereafter alternate right and left tendons except stress Mark A2 in the Pump Turbine Unit prior to Mark A1. Thus, stressing will begin with the tendon nearest the centroid of the total post tension force continuing outward approximately in order of distance from the centroid. A minimum of seven days after completion of the tensioning of each tendon, the prestress force shall be subjected to verification testing. A satisfactory test will be one in which a force equal to the long term working force is sustained without unseating the tendon."

The normal sequence for stressing a tendon was as follows:

1. The jack is lowered over the tendon and the head is screwed onto the top of the tendon.

2. The tendon was jacked to about 1,000 psig to seat the jack.

3. Temporary 1-inch plates were installed under the tendon head.

4. The jack pressure was reduced to zero.

5. The tendon was then stressed to first-stage transfer taking gage and elongation readings at predetermined points.

6. The temporary plates were removed and two 4-inch thick plates were inserted under the head of the tendon.

7. The jack pressure was released and the ram lowered and the lifting nut lowered to contact the ram.

8. Jacking was then continued with measurements taken at the appropriate points until the initial prestress force was reached.

9. Steel plates were added to a height sufficient to maintain elongation at the transfer stress.

10. The head of the tendon was lowered onto the steel plates and the pressure on the jack released.

11. A check was then made of the transfer force by rejacking until liftoff occurred.

12. The jack was then removed and moved to the next tendon where the process was repeated.

The Contractor used a 750-ton Pine Ram with a ram area of 149.5 square inches. The gage used was calibrated in increments of 100 psi from 0 to 10,000 psi.

Readings for the tendon stressing taken and recorded by Inland Ryerson Company are as follows:

Date	Tendon	Gage psi	Elongation Inches	Transfer psi
24 November 1978	A2-1	8,050	13.05	7,200
25 November 1978	A2-2	8,050	13.60	7,200
	A2-3	8,050	13.15	7,150
	A1-4	8,050	13.10	7,125
27 November 1978	A1-5	8,050	13.15	7,150
	A1-6	8,050	13.45	7,125
	B1-7	6,850	12.70	6,150
	B9-8	6,850	13.20	6,150
	B3-9	6,850	13.20	6,050
28 November 1978	B10-10	6,850	12.80	6,125
	B2-11	6,850	12.60	6,050
	B9-12	6,850	12.85	6,100
	B4-13	6,850	13.15	6,000
	A2-14	8,050	13.00	7,125
29 November 1978	B5-15	6,850	13.20	6,100
	A2-16	8,050	13.45	7,125
	B6-17	6,850	12.80	6,025
	A2-18	8,000	13.10	7,200
	B5-19	6,850	12.70	6,100
	A1-20	8,050	12.90	7,150
30 November 1978	B6-21	6,850	12.75	6,125
	A1-22	8,050	13.25	7,100
	B5-23	6,850	12.85	6,100
	A1-24	8,050	12.80	7,125
	B6-25	6,850	13.15	6,100

Date	Tendon	Gage psi	Elongation Inches	Transfer psi
1 December 1978	B11-26	6,850	12.20	6,150
	B5-27	6,850	12.60	6,150
	B6-28	6,850	12.60	6,150
	B5-29	6,850	12.85	6,125
	B6-30	6,850	12.95	6,100
	B7-31	6,850	12.75	6,150
	B8-32	6,800	12.50	6,150

During stressing, load and elongation readings were taken at the initial 1,000 psi gage reading and then at .2, .3, .4, .5, .6 and .7 of ultimate maximum load. All elongation readings were noted to fall on the straight line portion of the stress-strain curve. During stressing, it was necessary to reset the ram at approximately the midpoint of each stressing operation due to the limit of the ram travel. Most transfer loads were set on the high side of .7 of ultimate maximum load.

After the required seven day waiting period, the tendons were check-stressed by loading to liftoff as indicated below.

Tendon Sequence No.	December Liftoff Date	Original Transfer Force psi	Final Liftoff psi
1	2	7,200	7,200
2	2	7,200	7,200
3	2	7,150	7,150
4	2	7,125	7,125
5	4	7,150	7,150
6	4	7,125	7,050
7	4	6,150	6,050
8	4	6,150	6,050
9	4	6,050	6,050
10	5	6,125	6,100
11	5	6,050	6,025
12	5	6,100	6,050
13	5	6,000	6,025
14	5	7,125	7,100
15	6	6,100	6,025
16	6	7,125	7,050

Tendon Sequence No.	December Liftoff Date	Original Transfer Force psi	Final Liftoff psi
17	6	6,025	6,025
18	6	7,200	7,075
19	6	6,100	6,050
20	6	7,150	7,100
21	11	6,125	6,050
22	11	7,100	7,050
23	11	6,100	6,050
24	11	7,125	7,075
25	11	6,100	5,950
26	11	6,150	6,100
27	11	6,150	6,050
28	11	6,150	5,975
29	11	6,125	6,050
30	11	6,100	6,050
31	11	6,150	6,000
32	11	6,150	5,950

F. Secondary Grouting

Following check-stressing, the tendons were grouted with the same grout mix and equipment used for the first-stage grouting. In some cases, it was necessary to return to the tendon hole to complete the fill up of the hole. On 4 and 5 December 1978, a great deal of difficulty was experienced due to inability of the grout pump to overcome the grout head and fill the hole. The apparent problem was that the grout pump, which was in a temporary enclosure, was freezing up and losing the necessary revolutions per minute required to overcome the grout head. On 6 December 1978, a heater was installed in the temporary grouting enclosure and thereafter the grouting operation proceeded smoothly. The grouting dates were as follows:

Tendon Sequence No.	December Date Grouted	December Date Additional Grout Was Added
1	4 and 5	15 and 16
2	5	15 and 16
3	6	15 and 16
4	4	15 and 16
5	5	15 and 16
6	4	15 and 16
7	6	15 and 16
8	7	None
9	12	15
10	7	15 and 16
11	6	None
12	7	None
13	12	15
14	6	None
15	12	15
16	11	15 and 16
17	13	15
18	12	None
19	13	None
20	6	15 and 16
21	13	None
22	11	15
23	13	15
24	11	15 and 16
25	13	15
26	11	None
27	14	None
28	14	15
29	14	15 and 16
30	14	15
31	14	None
32	15	16

Upon completion of secondary grouting, the top of the anchorage blockouts were cemented.

GROUTED/REDRILLED POST TENSIONING
ANCHORAGE BORINGS

Station	Offset (U/S)	Duration of Washing (Min.)	Pressure Test (Cu.Ft./Min.)	Grout Take (Bags/Mix)
6+81.00	46.83	30	0.2	12/2:1(8) - 1:1(4)
6+81.00	50.80	50	0.4	16 Bags
7+04.00	46.83	75	0.16	12/2:1(8) - 1:1(4)
7+04.00	50.83	20	0.4	8 Bags
7+27.00	46.83	35	0.5	1:19 (Injected)/1:1
7+27.00	50.83	105	0.4	0 (Injected)/1:1
7+32.00	35.74	30	0.8	18/1:1
7+32.50	45.58	30	0.3	12 Bags
7+38.00	34.75	35	1.5	8.5/1:1
7+38.50	45.58	30	18.8	31/2:1 - 0:6
7+44.00	34.75	55	1.0	5/1:1(4) - 2:1(1)
7+50.00	34.75	45	2.1	29/2:1 - 1:1
7+56.00	34.75	40	1.6	5.5/2:1
7+62.00	34.75	35	2.1	2/5/2:1
7+74.00	34.75	35	0.12	6/1:1
6+68.00	34.75	60	0.2	5/1:1(1) - 2:1(4)
7+80.00	34.75	40	0.12-.02	-
7+86.00	34.75	30	0.3	9.5/1:1
7+92.00	34.75	30	1.4	10/2:1(8) - 1:1(2)
8+03.25	15.75	55	0	8/2:1
8+08.25	15.75	60	0	8 Bags

Notes: 1. The majority of the pressure tests were conducted for 15 minutes.

2. Grout take generally includes the volume of cement required to fill the 8-inch diameter boring.

TABLE NO. 1

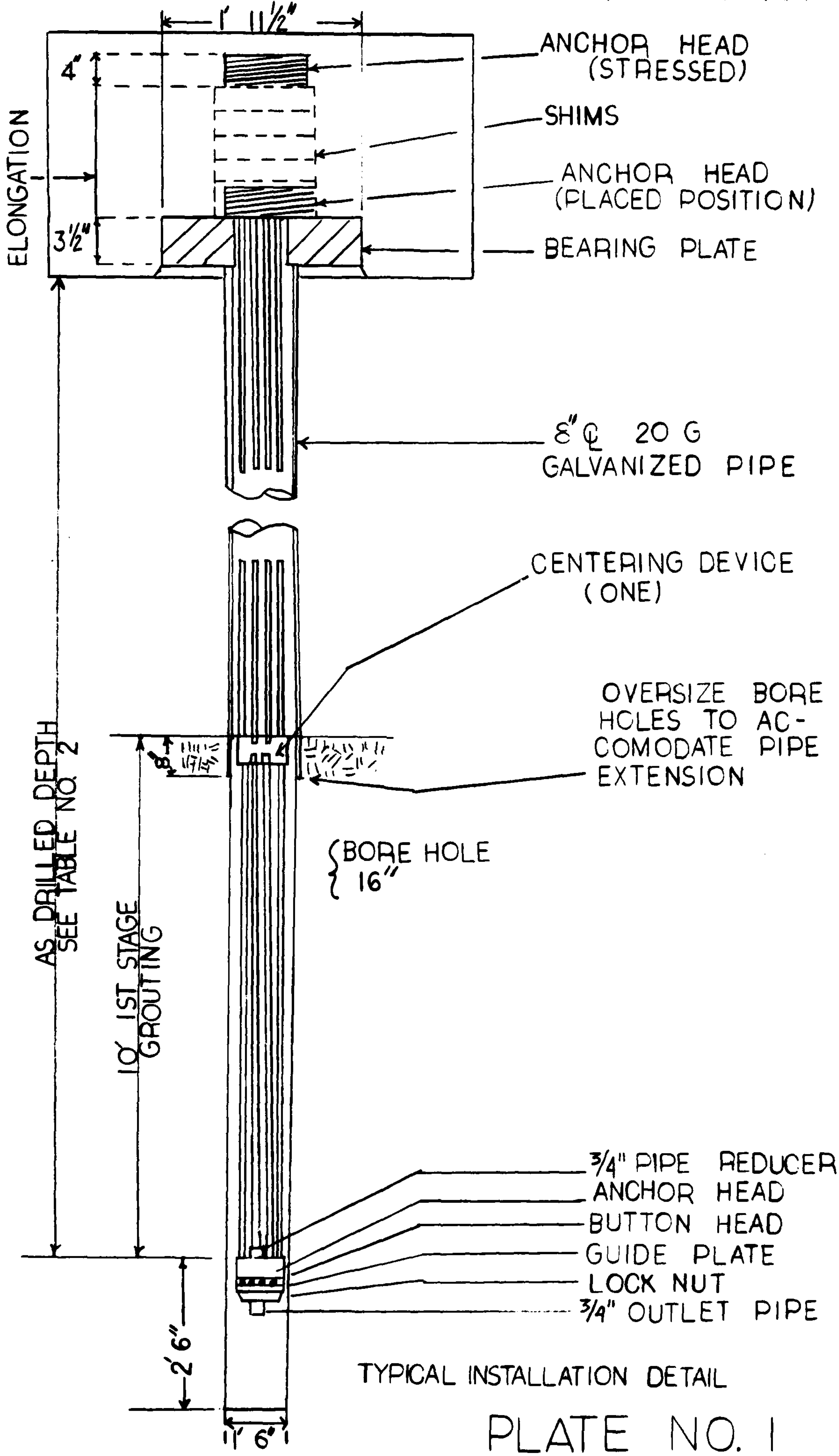
Tendon Mark	Original Tendon No.	As-Built Tendon No.	Station	Offset (U/S)	Depth In Rock In Ft.	Elevation BOH NGVD	Total Depth Of Hole In Ft.	Angle
B-1	18	7	7+32.50	45.58	33.00	432.00	220.00	Vertical
B-2	20	11	7+38.50	45.48	43.00	422.00	226.00	Vertical
B-3	17	9	7+32.00	34.75	45.00	420.00	231.00	5.95° U/S
B-4	19	13	7+38.00	34.75	35.00	430.00	221.00	5.95° U/S
B-5	21	15	7+44.00	34.75	45.00	420.00	231.00	5.95° U/S
B-6	22	17	7+50.00	34.75	35.00	430.00	226.00	5.95° U/S
B-5	23	19	7+56.00	34.75	45.00	420.00	231.00	5.95° U/S
B-6	24	21	7+62.00	34.75	35.00	430.00	226.00	5.95° U/S
B-5	25	23	7+68.00	34.75	45.50	419.50	231.00	5.95° U/S
B-6	26	25	7+74.00	34.75	35.00	430.00	221.00	5.95° U/S
B-5	27	27	7+80.00	34.75	45.00	420.00	231.00	5.95° U/S
B-6	28	28	7+86.00	34.75	35.00	430.00	221.00	5.95° U/S
B-5	29	29	7+92.00	34.75	45.00	420.00	231.00	5.95° U/S
B-6	30	30	7+96.42	34.75	35.50	429.50	221.00	5.95° U/S
B-7	31	31	8+03.25	15.75	45.00	420.00	230.00	5.95° U/S
B-8	32	32	8+08.25	15.75	34.00	431.00	220.00	5.95° U/S
B-9	8	8	6+74.75	33.75	44.00	421.00	231.00	Vertical
B-10	9	10	6+65.75	33.75	35.00	430.00	221.00	Vertical
B-9	10	12	6+54.75	33.75	44.00	421.00	231.00	Vertical
B-11	1	26	5+96.00	22.75	45.00	420.00	231.00	Vertical
A-1	2	24	6+02.50	50.83	40.00	425.00	226.00	Vertical
A-2	3	18	6+02.50	46.83	51.50	413.50	236.00	Vertical
A-1	4	22	6+25.50	50.83	40.00	425.00	226.00	Vertical
A-2	5	16	6+25.50	46.83	51.50	413.50	236.00	Vertical
A-1	6	20	6+48.50	50.83	40.00	425.00	226.00	Vertical
A-2	7	14	6+48.50	46.83	50.00	415.00	236.00	Vertical
A-1	12	6	6+81.00	50.83	38.83	426.17	226.00	Vertical
A-2	11	3	6+81.00	46.83	48.00	417.00	236.00	Vertical
A-1	13	4	7+04.00	50.83	40.25	424.75	226.00	Vertical
A-2	14	1	7+04.00	46.83	48.58	415.42	236.00	Vertical
A-1	16	5	7+27.00	50.83	38.83	426.17	226.00	Vertical
A-2	15	2	7+27.00	46.83	48.00	417.00	236.00	Vertical

TABLE NO. 2

Tendon Sequence No.	Theoretical Cement Bags	Nov. Placement Date	Cement Used Bags	Measured Depth Ft.	Second Nov. Placement Date	Cement Used Bags	Measured Depth Ft.
1	12.2	10	13	181.00	17	1	179.00
2	13.5	15	15	190.00	17	2	178.00
3	10.7	10	11	-	-	-	183.00
4	10.7	10	11	186.00	17	0.5	181.50
5	13.4	10	14	188.00	17	1	179.00
6	10.3	10	11	-	-	-	185.00
7	13.1	10	14	188.00	17	1	179.00
8	12.5	15	13	190.00	17	2	182.00
9	9.6	15	10	187.00	17	1	182.00
10	12.2	17	14	190.00	20	3	180.00
11	12.8	9	14	-	-	-	185.00
12	10.1	9	11	-	-	-	180.00
13	10.1	9	11	-	-	-	184.00
14	12.8	9	13	-	-	-	183.00
15	12.9	10	13	-	-	-	185.00
16	10.3	9	11	-	-	-	184.00
17	11.9	9	12	-	15	0.5	180.00
18	9.0	10	10	-	-	-	182.00
19	9.3	10	10	-	-	-	182.00
20	12.0	9	13	-	-	-	184.00
21	11.7	9	12	190.00	15	2	180.00
22	9.6	9	10	190.00	15	2	177.00
23	11.8	15	12	186.00	17	1	184.00
24	9.3	10	10	188.00	15	1	181.00
25	11.8	10	12	187.00	15	1	182.00
26	9.1	15	10	-	-	-	180.00
27	12.0	15	14	-	-	-	172.00
28	9.4	10	10	-	-	-	184.00
29	11.9	10	12	185.00	17	0.5	180.00
30	9.7	10	10	-	-	-	184.00
31	12.4	15	13	-	-	-	184.00
32	9.5	15	10	-	-	-	183.00

NOTE: Tendon Sequence No. 7 had a third placement of three bags of cement on 20 November 1978, to bring the final measured depth to top of grout to 179.00'.

TABLE NO. 3



TYPICAL INSTALLATION DETAIL

PLATE NO. 1

SECTION 9

STRUCTURAL CONCRETE

A. General

On 17 March 1973, the contract for the Construction of the Main Dam, Spillway, Power Plant Substructure and Intake was awarded to Massman Construction Co. of Kansas City, Missouri.

The contract called for the placement of approximately 422,000 cubic yards of concrete. The mass concrete was to contain aggregates up to 6 inches in diameter and the major portion of the structure to be placed using concrete not exceeding 50°F temperature when measured 20 minutes after placement. These requirements were major factors in the Contractor's plant type, size, location, aggregate sources, cement type, pozzolan, ice making facilities, etc. In addition, the Contractor was responsible for the establishment of a Quality Control Program.

Concrete placements for the structure began on 9 December 1974 and continued through August 1979. The batch plant fire, strikes, extreme cold weather and other delays occurred which caused the extensive period of time for completion of concrete under the Phase II contract.

B. Materials

1. Aggregate Sources

(a) Coarse Aggregates

The coarse aggregates were required to be produced in four sizes: 6-inch to 3-inch, 3-inch to 1 and 1/2-inch, 1 and 1/2-inch to 3/4-inch and 3/4-inch to No. 4. The coarse aggregates were produced by Central Stone Company at their Huntington, Missouri plant (Central Stone No. 1). The aggregates were of crushed limestone obtained from

Upper Kimmswick Formation which was an approximate 40-foot to 42-foot face. The aggregates from this source were evaluated during 1967 and again in 1974 when a portion of the Lower Kimmswick Formation was approved to be included in the production of concrete aggregates. The upper 14 feet of the Upper Kimmswick Formation were very vuggy and contained clay filled vugs. In 1974, the quarry was asked not to include the upper 10 feet in their shots for quarrying. This allowed an approximate 40-foot face to be shot for coarse aggregate production and consisted of approximately 30 feet of the Upper Kimmswick and approximately 10 feet of the Lower Kimmswick Formation to be used. The inclusion of the Lower Kimmswick aggregate caused work difficulties with absorption during mixing and degradation of the coarse aggregate sizes during handling and stockpiling.

(b) Fine Aggregates

The Contractor elected to use manufactured sand as the fine aggregate. Central Stone Company's Huntington Quarry utilized an existing stockpile of screenings to produce a fine aggregate through a washing and screening process. The fine aggregate produced met the contract requirements without the use of an admix sand as generally required of river sands in the project area. The control of gradation and fineness modulus was somewhat difficult due to the variation of the material used in manufacturing the fine aggregate.

(c) Samples for Mixture Designs

Samples of aggregates were obtained during June 1974 and were sent to Waterways Experiment Station at Vicksburg, Mississippi, for use in the design of the various concrete mixtures.

2. Cement and Flyash

The contract bidding schedule allowed the Contractor to bid one of three options for cementing materials: Option 1, for concrete made with Portland cement; Option 2, for concrete made with a blend of Portland cement and flyash; and Option 3, for concrete made with Type IP Portland cement. The Contractor bid the contract exercising Option 3. Later investigation by the Contractor revealed problems were likely to occur with availability and transportation of Type IP Portland cement to the project site. In July 1973, the Contractor proposed the substitution of Option 2, using Type II Portland cement and flyash. The change was considered to be acceptable and the contract was modified (giving the Government a credit for the substitution) by Modification No. P00027.

The Portland cement was to conform to Federal Specification SS-C-192g, Type II, with requirements for Low Alkali, Heat of Hydration and False Set. The flyash was to conform to CRD-C-262, Class F. The Contractor's sources for Portland cement and flyash were Universal Atlas Portland Cement Co., Independence, Kansas Plant and the flyash was to be furnished by the Walter N. Handy Co. of Springfield, Missouri, with the flyash coming from the Montrose Plant at LaDue, Missouri.

The Portland cement was shipped from Independence, Kansas by rail to the Universal Atlas Plant at Hannibal, Missouri, where it was unloaded directly into trucks for shipment to the project. The trucks were furnished by Schwerman Trucking Co. and were of the type normally used in handling cement in bulk form. The flyash was shipped by truck directly from the Montrose Plant to the project site. Walter N. Handy Co. and Tiona Truck Lines were charged with the hauling operation.

All cement and flyash shipments were made from previously tested and approved silos located at the point of manufacture. All testing was accomplished by the Waterways Experiment Station.

3. Air Entraining Admixtures

The Contractor proposed the use of a powdered material to be mixed with water at the site for air entraining admixture. The material was pulverized Vinsol NVS as manufactured by Hercules, Inc., and as described in their technical data bulletin PC-178. The Foundations and Materials Branch, St. Louis District, felt that field mixing of air entraining admixtures was not acceptable; that the intent of the contract specifications was the admixture was to be supplied in a solution ready to batch. They felt that field mixing would allow too much variation in the admixture strengths and would thus affect the uniformity of the concrete mixtures. The use of powdered admixture was disapproved in June 1974. However, in September 1974, the Contractor voiced objection to this decision and again set forth procedures, etc., to be followed by his Quality Control Organization, and asked that the Government reconsider its position. In October 1974, approval was given for the use of the powdered admixture provided that the procedures would be followed and other conditions were met. Also, the job-mixed solution was to be sampled by project personnel and shipped to Waterways Experiment Station for testing.

Job site mixing of the air entraining admixture was accomplished and samples obtained and sent to Waterways Experiment Station in late October 1974. Initially, it was felt that testing of each batch would be required; however, the quantity of material being used would require an unrealistic amount of sampling. It was then determined that periodic samples would be obtained and sent to Waterways Experiment Station and that each batch would be tested by project personnel using a hydrometer to assure uniformity in the solution's specific gravity. The checking of specific gravity of the job-mixed admixture by project personnel was performed and results indicated the Contractor was able to control the uniformity within acceptable limits.

In August 1975, due to a strike at the Hercules Plant, the supply of Vinsol NVX became short and the Contractor requested the approval of "Air-In", a liquid admixture manufactured by Hunt Process Corporation of Ridgeland, Mississippi, for use through the completion of the project. Approval was given and the use of Hunt Process "Air-In" began in September 1975. The admixture was used in the double-strength solution.

4. Curing Compound and Curing Water

Due to limitations on the use of water curing in areas of the shale foundation exposure, the Contractor used a white pigmented curing compound as manufactured by Hunt Process Corporation of Ridgeland, Mississippi.

Water for curing in other locations was obtained from an on-site well located upstream of the structure in the buried channel of the Salt River. The water was tested and approved for use by Waterways Experiment Station.

5. Mixing Water

Mixing water was obtained from the same well as mentioned in Paragraph 4 above.

C. Concrete Mixture Designs

1. General

The design mixtures were proportioned by the Waterways Experiment Station for the St. Louis District. Preliminary concrete mixtures were furnished in September 1974. Final mixture proportions were furnished when the 90-day strength tests were completed.

Eight concrete mixtures were proportioned and were designated as follows:

Type A — Exterior Concrete, 1 and 1/2-inch maximum size aggregate, maximum water content of 5.0 gal./bag of cement, slump 1 1/2 ±1/2-inch, air content 5.0 ±.05 percent, 25 percent of volume of cement replaced with flyash and have a compressive strength f'c 5,000 psi using coefficient of variation of 10 percent. 1 1/2" T.S.A.

Type B — Exterior Concrete, 3/4-inch maximum size aggregate, maximum water content of 5.5 gal./bag of cement, slump 3 ±1/2-inch, air content 6.0 ±0.5 percent, 25 percent of volume replacement of cement with flyash and have a compressive strength f'c 4,000 psi using a coefficient of variation of 10 percent. 3/4" T.S.A.

Type C — Concrete for Walls, 3-inch maximum size aggregate, maximum water content of 6.0 gal./bag of cement, slump 1 1/2 ±1/2-inch, air content 5.0 ±0.5 percent on minus 1 1/2-inch fraction, 25 percent of volume of cement replaced with flyash and have a compressive strength f'c 3,000 psi using a coefficient of variation of 15 percent. 3" T.S.A.

Type D — Exterior Concrete, 6-inch maximum size aggregate, maximum water content of 6.0 gal./bag of cement, slump 1 1/2 ±1/2-inch portion, air content 5.0 ±0.5 percent on minus 1 1/2-inch portion, 25 percent volume of cement replaced with flyash and have a compressive strength f'c 3,000 psi using a coefficient of variation of 15 percent. 6" T.S.A.

Type E-1 Interior Concrete, 6-inch maximum size aggregate, maximum water content of 9.0 gal./bag of cement, slump 1 1/2 ±1/2-inch on minus 1 1/2-inch portion, air content 5.0 ±0.5 percent on minus 1 1/2-inch portion, 35 percent volume of cement replaced with flyash and have a compressive strength f'c 2,000 psi using a coefficient of variation of 15 percent. 6" T.S.A.

Type E-2 Exterior Concrete, 6-inch maximum size aggregate, maximum water content of 8.0 gal./bag of cement, slump 1 1/2 ±1/2-inch on minus 1 1/2-inch portion, air content 5.0 ±0.5 percent on minus 1 1/2-inch portion, 30 percent volume of cement replaced with flyash and have a compressive strength f'c 2,500 psi using a coefficient of variation of 15 percent. 6" T.S.A.

Type G "Ogee Cover" High Velocity Concrete, 3-inch maximum size aggregate, maximum water content 5.0 gal./bag of cement, slump 1 1/2 ±1/2-inch on minus 1 1/2-inch portion, air content 5.0 ±0.5 percent on minus 1 1/2-inch portion, 25 percent volume of cement replaced with flyash and have a compressive strength f'c 3,000 psi using a coefficient of variation of 15 percent. 3" T.S.A.

Type H Channel Paving Concrete, 1 1/2-inch maximum size aggregate, maximum water content of 5.0 gal./bag of cement, slump 1 1/2 ±1/2-inch, air content 5.0 ±0.5 percent, 25 percent volume of cement replaced with flyash and have a compressive strength f'c 3,000 psi using a coefficient of variation of 15 percent. 1 1/2" T.S.A.

Note T.S.A. is abbreviation for Top Size Aggregate.

Batch weights for one cubic yard in pounds, S.S.D. basis were as follows:

Materials	A	B	C	D	E-1	E-2	G	H
Cement	458	423	289	213	129	161	352	405
Flyash	126	116	80	59	58	57	97	112
Sand	1121	1204	1095	793	865	840	1039	1172
Stone	2004	1812	2345	2916	2916	2914	2326	2005
Water	227	265	198	147	147	147	200	230
A.E.A.	Field Determination							

Characteristics	A	B	C	D	E-1	E-2	G	H
CM/F (bags/cu.yd.)	6.5	6.0	4.1	3.0	2.1	2.45	5.0	5.75
W/CM (gal./bag)	4.2	5.3	5.8	5.8	8.3	7.2	4.8	4.8
W/CM (weight)	0.39	0.49	0.54	0.54	0.78	0.67	0.445	0.44
S/A (%)	36	40	32	22	23	23	31	37
Air % (-1 1/2" portion)	5.0	6.0	5.0	5.0	5.0	5.0	5.0	5.0
Slump (inches) (-1 1/2" portion)	1½	3	1½	1½	1½	1½	1½	1½
Unit Weight (lb./ft.^{3})	145.8	141.5	148.4	152.9	152.4	152.6	148.7	145.3

2. Additional Interior Concrete Mixtures

After the placement of concrete mixtures had been initiated in the structure, the Contractor expressed concern regarding the strength of interior mixtures. The anchorages for forming required a strength of 1,000 psi and the time required to accomplish such strength was in excess of what the Contractor had anticipated. Several factors were apparent--one being the cold weather placement, and secondly, the total cementing material content of the Interior Mix E-1 was below the specified minimum in the contract specifications. In order to correct this problem, it was decided to eliminate the E-1 and E-2 concrete mixtures and have only one mixture instead. The new mixture design was designated as Type E.

Mix Type E		Batch Wt. Lb. S.S.D. Basis
Cement	165)	
Flyash	59)	
Sand	819)	
Stone	2,922)	
Water	150)	
CM/F	2.50)	18 January 1975
W/CM	7.2)	
W/CM	0.670)	
S/A	22)	
Air	5.0)	
Slump	1 1/2)	
Unit Weight	152.4)	

3. Changes to Concrete Mixtures

To assure design strengths during the cold weather and initial shakedown of concreting operations, the concrete mixtures in use (C, D, E and H) during February 1975, an increase was made in the cementing materials content. This was a temporary change and the cementing materials content was reduced in all except the Type E mixture at a later date.

4. Concrete Mixtures "CH" and "J"

(a) It became apparent that a mix similar to C and D but with smaller aggregate was needed for various locations in the structure where placement became difficult to accomplish due to congestion and/or poor access. A mixture designated as type CH was designed for this purpose and used 1 1/2 inch maximum size aggregate.

This mixture could be tremied or placed by buckets.

One Cubic Yard S.S.D. Batch Weights and Characteristics of Mix

Cement	330
Flyash	89
Water	222
Sand	1,047
Stone	2,236
A.E.A.	Field Determination
W/C (by weight)	0.530
S/A (percent)	32
Air (percent)	5.0
Slump (inches)	2
Strength f'c	3,000 psi

Concrete mixture Type "CH" was used throughout the various portions of the structure and performed as expected.

(b) The need for a concrete mixture requiring a compressive strength f'c of 5,000 psi and 3/4-inch maximum size aggregate became apparent when concrete repairs were needed in the highly congested areas of the trunnion girders and trunnion anchorages. The mixture would be available as a substitute for the 1 1/2-inch maximum size aggregate mixture Type "A". The new mixture was designated as Type "J".

One Cubic Yard S.S.D. Batch Weights and Characteristics of Mix

Cement	525
Flyash	136
Water	280
Sand	1,142
Stone	1,720
A.E.A.	Field Determination
W/C (by weight)	0.400
S/A (percent)	40
Air (percent)	6.0
Slump	2
Strength f'c	5,000 psi

NOTE: Concrete mixtures "CH" and "J" were proportioned by the Resident Materials Section, with approval from the Engineer Division, Material and Exploration Section.

5. Ready-Mix Concrete

Concrete mixtures from local concrete plants were approved for use several times throughout the construction of the structure. Protective concrete for the foundation shale, the El. 485-foot NGVD berm concrete, backfill concrete, foundation wall and concrete placed in the structure during 1979 (following the dismantling of the batch plant) all consisted of concrete manufactured by L.W. Riney Co., Hannibal, Missouri; Paris Concrete, Paris, Missouri; and Bleigh Concrete, Monroe City, Missouri.

The mix designs, in general, were the same as previously used with the exceptions that Type I cements and natural river sands were used.

D. Concrete Batching Facilities

1. Batch Plant Type and Location

The Contractor elected to erect the concrete manufacturing facilities in the river valley rather than either abutment. The plant was located approximately at Station 12+00 and 140 feet downstream of the centerline of the dam. The plant was erected on earthfill placed on a portion of the Phase I embankment. H-piling was driven to refusal to support the batch plant proper, cement storage silos and flyash storage silo.

The concrete plant was a C.S. Johnson 1,100 cubic yard Octobin Batch Plant with Serial No. 60739. The plant had been used on several mass concrete projects over the period of 15 to 20 years. Due to the age of the plant, it was necessary to do considerable maintenance and rehabilitation during erection.

The storage bins for aggregates were located on top of the plant and had capacities of 160 cubic yards of 3/4-inch - No. 4 stone, 112 cubic yards of 1 1/2-inch - 3/4-inch stone, 224 cubic yards of 3-inch - 1 1/2-inch stone, 224 cubic yards of 6-inch - 3-inch stone and 168 cubic yards of sand. The storage capacity of cement and flyash were 685 barrels each. The storage of cement and flyash was accomplished by using one circular storage bin divided into two parts and was located in the center of the aggregate storage bins.

Located above the aggregate storage bins was the coarse aggregate rescreening plant which is discussed in Paragraph 2 below.

Material batching and weighing facilities were provided for each stone size, sand, cement, flyash, water, ice and air entraining agent. The batching was controlled by a mix control panel capable of 12 separate mixes by selection of the plant operator. The materials after weighing were discharged to one of four Koehring Tilt Mixtures (four cubic yard capacity each). After mixing, the concrete was discharged into a holding hopper with a capacity of 16 cubic yards. The holding hopper was divided with two mixers discharging into each one half. This allowed several mixtures to be batched and loaded out for various placements simultaneously. Loading of buckets was accomplished directly under the holding hopper. The mixing time for four cubic yards of concrete was 90 seconds based on results of Mixer Performance Testing performed by Waterways Experiment Station.

2. Aggregate Stockpiling, Washing, Rescreening and Testing

Aggregate sources, their locations and routes for transportation of aggregates to the project site led the Contractor to select the left or north abutment as the location of the aggregate stockpiles. Excavation of a portion of the abutment where State Route J was to be relocated was accomplished to facilitate the construction of the conveyor "tunnel".

The stockpiling of aggregate was over this tunnel through the use of an overhead belt conveyor with a tripper to discharge at the various stone size compartments. A belt conveyor located in the C.M.P. "tunnel" was loaded by gravity feed and/or Syntron vibratory feeders. The loading of the aggregates on the conveyor from the stockpiles was controlled by the operator of the rescreening plant. All sizes of coarse aggregates were conveyed at the same time to the washing facilities and then to the rescreening area where they were once again separated into the various size ranges. In order that sand would not interfere with the conveying of stone, the Contractor constructed a smaller sand storage facility near the wash plant. This allowed sand to be conveyed from the stockpile area during slack periods of production. All aggregates were transported from the stockpile area across the diversion channel via a conveyor belt suspended from cables anchored on the left abutment and near the wash plant.

The coarse aggregates were washed prior to rescreening. The washing plant was located downstream of the batch plant and consisted of two double-deck 5-foot by 14-foot screens with spray bars. The washing process removed a large percentage of the -3/16-inch materials present in the aggregate.

From the washing plant, the coarse aggregates were conveyed to the rescreening plant located at the top of the concrete batch plant.

The rescreening facilities consisted of a dual screening plant made up of two 6-foot by 16-foot double-decked units for each one half of the plant. The upper units produced the 6-inch - 3-inch and 3-inch - 1 1/2-inch stone sizes while the lower units produced the 1 1/2-inch - 3/4-inch and 3/4-inch - No. 4 sizes. Screen sizes were adjusted periodically to accomplish the required gradations of the stone as delivered to the mixes. The -No. 4 size screenings were wasted into a sump at the base of the plant for ease in removal. From the rescreening plant, the coarse aggregates were deposited into the aggregate storage bins located directly below.

The contract specifications called for the aggregates to be within prescribed grading limits as delivered to the mixers. Also, the specifications required the Contractor to furnish a sampling and test screening device capable of screening large samples of all of the corase aggregate sizes. The unit was to be furnished for acceptance testing by Government Quality Assurance personnel. Engineering Technical Letter No. 1110-2-46 dated 12 July 1968 sets forth requirements and describes a unit capable of performing the required testing. The concrete batch plant, as provided for use on this project, had as part of its auxiliary equipment a sampling device as described by the ETL. The device was manufactured by the Curtis Manufacturing Co. of Spokane, Washington. The device had been used previously at the Lower Granite Dam Project with less than satisfactory performance. After considerable problems, meetings and correlation, operational procedures were developed which led to satisfactory performance of the device for this project. The Curtis Sampler will be discussed in this Report, Paragraph E.5.

3. Storage at Site for Cement and Flyash

Storage of cement was in two each 5,846 barrel capacity silos located at the base of the concrete plant. Flyash was stored in a 4,753 barrel capacity silo also located at the base of the plant. The transport trucks pneumatically discharged the cement and flyash into the respective silos. From the silos the cement and flyash were delivered to the storage silo at the top of the plant by a Fueller-Kenyon cement-flyash system with attendant dust collector systems.

4. Shaved Ice - Production, Storage, Conveying and Batching

The specifications required that all massive concrete maintain a temperature between 40°F and 50°F within 20 minutes after placing. Consequently, the use of shaved ice in the mixing process was required. Depending on the season of the year, relative humidity, ambient temperatures, etc., the use of the shaved ice varied from approximately 100 pounds per cubic yard to none.

The Contractor, in anticipation of the use of ice in large quantity, provided nine North Star Model 60 Ice Makers. The ice makers shaved ice from freezing coils and had a production capacity of approximately 20 tons each per day. The Contractor constructed an ice storage area directly below the ice makers with a storage capacity sufficient to maintain a stockpile of shaved ice at all times. The stockpiled shaved ice was removed by an ice rake to a twin screw conveyor where the ice was conveyed and discharged into a pneumatic conveying system which transported the shaved ice to an insulated surge bin at the weighing level of the concrete plant. The ice in the surge bin was conveyed by a screw conveyor to the ice scale where it was weighed and then discharged with the other ingredients into the mixer. The batching of ice presented a few problems due to freeze-ups and accuracy of the batching equipment.

5. Refrigeration Plant, Water Chiller and Air Conditioning of Aggregate Bins

The use of ice makers, the need for chilled (40°F) water and the need to cool the coarse aggregates as much as possible, required a refrigeration plant capable of providing cooling during the production of concrete up to 300 cubic yards per hour. Each of the four coarse aggregate storage bins was capable of being cooled by cold air forced through the aggregates from near the bottom of the bin. This allowed the aggregates to be exposed to cold air for the longest amount of time. Chilled water and shaved ice were used as the mixing water in order to attain further cooling of the concrete mixture. The refrigeration plant was of the liquid ammonia type.

6. Batch Plant Fire

On 22 February 1975, the concrete batching plant and other related facilities were destroyed by fire. Concrete production did not resume until 1 August 1975. During the time of rebuilding, some ready-mix concrete was placed as protective concrete. A report of the investigation of the fire is available at the District Office.

7. Operational Problems of the Concrete Batching Plant and Related Facilities

(a) General

Much time could be spent on discussion of the problems that arose during the production of concrete; however, many were of the type that normally occur when equipment is being used for long periods of time with minimal maintenance being performed. A large percentage of the delays, cold joints, etc., can be attributed to equipment failure.

A few of the operational problems were unique to this project and are discussed in this Report. Some problems such as those involving shortages of materials will not be discussed.

(b) Shaved Ice Batching and Weighing Accuracy

The batching of shaved ice is somewhat difficult due to the simple fact that ice will freeze together if it remains exposed to warmer temperatures for a short length of time. Freeze-ups occurred in the surge or temporary storage bin and in the screw conveyor which fed the ice into the scale hopper. The weighing accuracy of ice was less than desirable. The contract specifications allow an accuracy of ±1% for mixing water. The Contractor experienced many difficulties in attempting to attain this degree of accuracy. Changes were made in the rate of feed by the conveyor as well as a more rapid cutoff gate. Ice, due to its own qualities, may or may not fall onto the scale in the same amount each time as is experienced by other ingredients for concrete. Approximate variations in batch quantities were from 10 pounds to 20 pounds over or under on each batch. This amounts to a variation of 2.5% to 5.0% in accuracy. However, a variation of one gallon to two gallons of mixing water per 4-cubic yard batch did not appear to be detrimental to the quality of concrete produced for the project. Future plant design perhaps will see improvements in the batching facilities for ice.

(c) Graphical Record of Batching

Problems were experienced throughout the production of concrete with the graphical record of batching. The printing of graphs was somewhat troublesome due to lines not being straight, too little or too much ink, tearing, poor quality paper, etc. Large rolls of the graphical records also presented storage and handling problems.

(d) Gradations of Coarse Aggregates

A great many of the problems associated with the grading of aggregates can be attributed to maintaining stockpiles, rescreening plant operation and maintaining adequate levels of aggregate in the plant bins. The overhead feed belt for the stockpiles resulted in "cone shaped" piles, while the gates for the tunnel recovery belt under the stockpiles resulted in inverted cone feed to that gate. Both contributed greatly to segregation of the aggregate. Concrete placement usually was at night, and by morning the stockpiles were normally very low. The stockpiles were then built up to a high cone during the day by trucks hauling from the quarry. This rapid depletion and filling also contributed to segregation.

E. Sampling and Testing of Materials

1. Quality Assurance by Government

Quality Assurance Inspection and Testing requirements for this project were contained in a report entitled "Engineering Considerations and Construction Control Report", published by the Engineering Division, St. Louis District. The report pointed out areas of construction requiring special attention and also gave an indication of the amount of testing and inspection required for the proper control of concrete production and placement in the structure.

The Quality Assurance inspections and testing of all materials, fresh concrete, hardened concrete, mixing of concrete mixtures, proper placement, curing and protection were performed by personnel of the Main Dam Project Office and the Materials Section of the Resident Office. To assure that all personnel were aware of the role of the Government, a Quality Assurance Plan for Concrete Batching and Mixing was prepared. A flow chart of all operations for concrete production was prepared showing testing location, as well as points of inspection. A copy of the Quality Assurance Plan and a copy of the flow chart are attached at the end of this Section.

Quality Assurance inspections of placement of concrete and all related items were performed and records kept.

2. Quality Control by Contractor

The contract called for Quality Control inspections and testing to be performed by the Contractor as outlined in the specifications. The specifications required a minimum number of personnel to be furnished and that certain personnel have the necessary experience, education and registration to assure adequate control of the construction. The Contractor was generally able to furnish the qualified personnel as part of the program. The portion of quality control that was responsible for the proper production of concrete had more consideration by the Contractor than other features of work. However, the Contractor did have a quality control program that worked to accomplish the proper construction of the structure. Attached at the end of this Section is an outline of the Contractor's Quality Control Plan for tests and inspection of concrete production.

3. Concrete Strengths

(a) Concrete Mixture Design

The concrete mixture designs called for the following minimum compressive strengths to be attained:

Mix Type	Minimum Compressive Strength, f'c	Maximum Compressive Strength, f'c
A	5,000 psi	5,800 psi
B	4,000 psi	4,600 psi
C	3,000 psi	3,500 psi
D	3,000 psi	3,800 psi
E	2,000 psi	2,550 psi
E-1	2,000 psi	Not Used
E-2	2,500 psi	Not Used
G	3,000 psi	3,600 psi
H	3,000 psi	3,750 psi
CH	3,000 psi	3,600 psi
J	5,000 psi	5,800 psi

In order to obtain a more true reflection of concrete strengths, a random method of sampling fresh concrete mixtures was adhered to by Quality Assurance personnel. Test specimens were cast and cured within the applicable provisions of the CRD Test Procedures. The coefficient of variation was higher than hoped for, but is believed to present a true view.

The use of computer printouts was made available by a computer program set up by the Foundations and Materials Section, St. Louis District. Test results from specimens cast for each concrete mixture were forwarded to the F&M Branch on a biweekly to monthly basis. Plots at the project were maintained on a daily to weekly basis. The computer plots indicated test number, date, location in the structure, slump, air content, 7-, 28- and 90-day compressive strengths.

The final printouts for this project indicate the following data:

Mix Type	Average Slump	Average Air Content	Average 90-Day Strength
A	2.29 inches	5.4%	6,828 psi
B	2.66 inches	6.0%	6,325 psi
C	2.36 inches	5.2%	5,071 psi
D	2.29 inches	5.2%	4,441 psi
E	2.50 inches	5.3%	3,542 psi
G	2.22 inches	5.1%	5,907 psi
H	2.23 inches	5.3%	5,977 psi
CH	2.33 inches	5.2%	4,571 psi
J	2.64 inches	5.8%	7,837 psi

Mix Type	90-Day Tests Coefficient of Variation
A	7.39%
B	13.62%
C	12.54%
D	13.87%
E	15.88%
G	12.21%
H	12.03%
CH	12.86%
J	13.36%

Refer ACI 214-65 for the rating given for the above.

(b) Record Samples

LMVD-G Regluation No. 1110-1-300 dated 2 June 1966 required that 5% of the concrete control samples be tested at the Waterways Experiment Station Concrete Laboratory, Vicksburg, Mississippi. To comply with this regulation, two additional specimens were cast on every 20th set of specimens cast for a particular concrete mixture. For example, Set No. 20 for Mix Type H required the casting of six rather than the usual four test specimens. The two additional specimens were cured at the project for approximately 60 days prior to shipment. Shipment of the specimens was accomplished through the use of metal curing cans sealed to prevent moisture loss and crated in wooden crates for shipment by commercial truck line to Vicksburg, Mississippi. At the concrete laboratory in Vicksburg, the specimens were cured until the specified testing date. Results of the tests performed at Vicksburg vs. results of tests performed at the project indicated little or no difference in compressive strength determinations.

(c) Accelerated Curing of Concrete Test Specimens

At the request of OCE through LMVD, this project performed accelerated curing in accordance with Method CRD-C-97, Procedure A (using warm water). Three concrete mixtures were selected for the accelerated curing study. Mix Types C, E and H were selected and 20 sets of each mixture were tested. The results and findings of this program were reported to LMVD-G and a copy of the report is made a part of this Report.

4. Mixer Performance Tests

(a) As Performed by Contractor Quality Control

The contract called for complete mixer performance testing to be performed by the Contractor as part of his Quality Control Testing. The tests were to be in accordance with Method CRD-C-55. The complete tests were to be performed at the start of concrete production and once per year thereafter. An abbreviated mixer performance test was to be performed approximately six months following the complete test. The Contractor provided the necessary equipment and facilities for the mixer performance testing and was very competent in accomplishing the tests.

(b) As Performed by Government (W.E.S. Personnel)

Following the rebuilding of the batching plant due to the fire in February 1975, the Government, in August 1975, performed complete mixer performance testing to determine the minimum mixing times needed to produce concrete of the required uniformity. The mixing time was reduced as a result of this mixer performance testing. A copy of the testing performed by W.E.S. is attached as a part of this Report.

5. Curtis Sampler for Coarse Aggregates

As stated previously in Paragraph D.2. of this Report, the contract specifications required the Contractor to furnish a sampling device capable of performing grading analysis on large samples of coarse aggregates. The unit furnished was manufactured by the Curtis Manufacturing Co. of Spokane, Washington.

Engineering Technical Letter No. 1110-2-46 dated 12 July 1968 described the Curtis Aggregate Sampler; however, the requirements for screen sizes, operational procedures, correlation of results with existing standard sieving methods, etc., were not covered in sufficient detail. The Contractor purchased the Sampler as a part of the concrete batching plant and did not obtain any guidance pertaining to the use of the Sampler. Also, some features for operation and control were absent. The Contractor erected the Sampler and related equipment, along with the batch plant, during 1974.

The Sampler was initially used by the Contractor to determine the necessary screening requirements of the rescreening plant. The Sampler was used to perform acceptance testing of coarse aggregates during the production of concrete. Failures of aggregates to meet the specifications for grading led to investigation of the results obtained from the Curtis Sampler vs. those obtained from a Gilson Laboratory Sieve Shaker. It was found that failing tests on the Sampler were actually passing tests on the Gilson. Screen changes were thought to be necessary and were made with no apparent success. After many problems, delays, etc., a meeting was held with representatives from the manufacturer, the Government and the Contractor.

As a result, proper screen sizes were installed, proper operational procedures developed and correlation testing performed. The correlation testing provided the necessary information and data needed. The Curtis Sampler usage for testing throughout the remainder of concrete production was satisfactory for the control of gradations. The contract was modified to include the periodic correlation testing. The use of such a device on contracts using aggregates in the 6-inch - 3-inch and 3-inch - 1 1/2-inch sizes is recommended. The ability to perform sieve analysis on large samples in a short period of time is considered very beneficial to the control of concrete manufacture.

Results of correlation testing are maintained in the project files for future reference and the results of one correlation test are made a part of this Report.

F. Placement, Curing and Protection

1. Placement

(a) Transportation and Placement of Freshly Mixed Concrete

Two methods of transporting the concrete mixtures in 2-cubic yard and 4-cubic yard bottom dump hydraulic or air-operated buckets were used. Trucks capable of transporting three 4-cubic yard buckets (one empty, two loaded) were used where the placements were inaccessible by the other method and to supplement the other method. Self-propelled rail cars with a capacity of five 4-cubic yard buckets (one empty, four loaded) were used to transport fresh concrete to a point of pickup by one of two Washington gantry cranes. The gantry cranes operated from a construction trestle erected downstream of the stilling basin wall near the right abutment. The trestle also contained a dual-track system for the rail cars. Concrete was routed to the proper pickup point by switches. Each car load of concrete was identified as to mixture type and destination through the use of tags and lights, respectively. A dispatcher at the plant loaded the proper mixtures from the plant holding hopper into the buckets. He was responsible for the proper identification and destination.

Upon arrival at the proper pickup point, the concrete was transported to the placement site by the crane.

Placement of concrete, for the most part, was accomplished through the use of cranes and buckets. Some areas of placement required the use of tremies. Placement of concrete in 7 and 1/2-foot lifts in the massive sections required the Contractor to carry five 18-inch benches of fresh concrete.

(b) Temperature Restrictions of Freshly Placed Concrete

Ultimate strain capacity tests and computer thermal studies were conducted by the Waterways Experiment Station during the design stage. Results of these studies are reported in Supplement No. 1 to Design Memorandum No. 5, Availability of Construction Materials.

These studies developed optimum lift thickness and placing temperature of the mass concrete to control cracking associated with the heat of hydration. Based on these studies the mass concrete placements were required to be placed at a maximum concrete temperature of 50°F when measured twenty minutes after mixing. Thin slabs, generally less than 3 feet thick, had a maximum allowable placing temperature of 85°F.

(c) "Ogee" Section

The placement of concrete in the overflow monoliths called for the use of the 3-inch MSA concrete mixture, Type G. This mixture was to be placed as the surface concrete. Initially, this mixture was used; however, the development of "honeycombed" areas and a large amount of surface voids led to the use of the 1 1/2-inch MSA mixture, Type H, in the outer shell of the Ogee Section.

2. Curing and Protection

Curing of concrete was accomplished by water, water and burlap, and white pigmented curing compound. Curing during cold weather required the use of insulated blankets, as well as heat, to protect the concrete during the curing period.

The placement of concrete during cold weather also required the use of insulated forms.

G. Concrete Cracking

A concrete condition survey was completed in September 1983. This survey included inspection of the stilling basin which had been dewatered, at the time of closure, after several large flows through the diversion sluices. A report has been prepared documenting the extent of cracking in the completed structure. The cracking observed is very minimal for such a large and complicated structure. The few cracks are in the fine category, less than 1mm in width.

H. Sluices

As the result of numerous periods of high water after diversion of the river through the three sluices, the protective coverings for the waterstops were torn away, the PVC waterstops were destroyed, the concrete in the area of waterstop blockouts was moderately eroded and the dowels embedded in the surrounding concrete at the downstream end of the sluices were damaged. Repair work performed by the Contractor, as directed, included patching of eroded concrete, replacement of damaged dowels, and relocation and replacement of the PVC waterstop with stainless steel.

Installation of the steel blukheads and repair described above were performed in August and September 1983.

The sluices were filled by two methods as shown on the sketch attached at the end of this Section; the upstream portion by preplaced aggregate concrete and the downstream segment by pumped concrete. The upstream 34 feet 2 inches of the sluices were filled with preplaced aggregate concrete in two placements each; the upstream placement being 14 feet 2 inches and the second placement 20 feet. Coarse aggregate was supplied by Central Stone Co. from their quarry at Huntington, Missouri, and was crushed limestone graded from 3-inch to 3/8-inch. Intrusion grouting was performed under the supervision of the Prepakt Concrete Co., Cleveland, Ohio. Two simplex (2.5-inch single piston) grout pumps were used with normal working pressure of 10± psi. Grout design used was as follows:

1,303 lb. Type IP cement per cubic yard

1,194 lb. masonry sand per cubic yard

796 lb. blow sand per cubic yard

69.3 gal. water per cubic yard

13 lb. fluidifier per cubic yard

Suppliers of grout materials were as follows:

Cement - Dundee Cement Co., Clarksville, Missouri

Masonry Sand - Missouri Sand and Gravel Co., LaGrange, Missouri

Blow Sand - Missouri Sand and Gravel Co., Marcelline, Illinois

Fluidifier (Intrusion Aid Type R) - Prepakt Concrete Co., Cleveland, Ohio

Bleigh Ready-Mix Co., Monroe City, Missouri, delivered the grout to the job site in ready-mix trucks.

Preplaced aggregate concrete was started in September 1983 and was completed in November 1983.

In areas other than those described above, the sluices were filled with pumped concrete. A Schwing Model BPI 801 pump with 5-inch lines was used. Concrete design used was as follows:

564 lb. Type IP cement per cubic yard

1,103 lb. #4 - #100 sand per cubic yard

123 lb. blow sand per cubic yard

1,838 lb. 1-inch crushed limestone per cubic yard

32.5 gal. water per cubic yard

*64 oz. superplasticizer per cubic yard

6% entrained air

*Superplasticizer was used only in the top 2 feet to 3 feet.

Sources of concrete materials were as follows:

Cement - Dundee Cement Co., Clarksville, Missouri

Sand (#4 - #100) - Missouri Sand and Gravel Co., LaGrange, Missouri

Blow Sand - Missouri Sand and Gravel Co., Marcelline, Illinois

Coarse Aggregate - Central Stone Co., Huntington, Missouri

Superplasticizer (Melment Type A) - American Admixtures & Chemical Corporation, Chicago, Illinois

Air Entrainment (MB-AEA-10) - Master Builders Co., Cleveland, Ohio

Bleigh Ready-Mix Co., Monroe City, Missouri, delivered the concrete to the job site in ready-mix trucks.

Concrete was pumped in November and December 1983.

Following mass filling of the sluices, the Ogee section forms were covered with insulated blankets and a period of curing and cold weather protection was provided in accordance with contract requirements.

On 5 January 1984, the insulated blankets were removed and the Contractor commenced shrinkage grouting operations along the sluice ceilings as required by the contract. Grouting was performed through a system of 3/4-inch galvanized pipes installed in each sluice prior to mass filling. The piping arrangement consisted of eight groups of pipes for each sluice as shown in the sketch attached at the end of this Section. Each group was composed of two grout pipes placed near the outside corners of the sluice ceilings and one vent pipe located near the center. Four groups were placed in the four mass placement zones and the remaining four groups were located upstream and downstream of the two stainless steel waterstops. The system resulted in 16 grout and 8 vent pipes available for each sluice.

Grout and vent pipes were installed with 1/8-inch holes drilled on approximately 6-inch centers along each particular placement zone. Grout pipe holes were covered by expandable rubber sleeves to preclude intrusion of mortar during mass filling but to allow extrusion of grout during that operation. Vent pipes were similarly perforated and protected by masking tape; tape seals were broken with air pressure prior to grouting.

Shrinkage grouting was performed using a ChemGrout CG-500 grout plant with two mixing tanks (eight cubic feet each) and a Moyno pump. Grouting was controlled at the header of each grout pipe by use of ITT Grinnell ball valves.

Initial grout mixes of 3:1 and 2:1 (water to cement by volume) were examined and determined too thin for job conditions. Actual grouting was performed using a 1:1 mix proportioned by volume as follows:

94 lb. cement (1 cubic foot bulk)

7.5 gal. water (1 cubic foot)

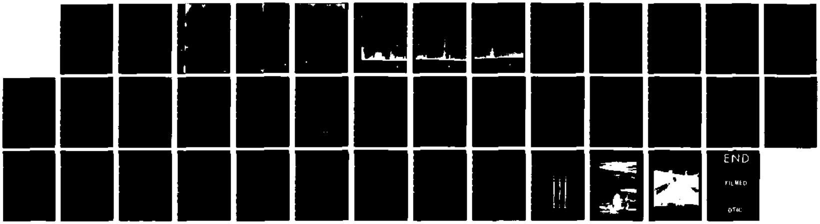
.75 lb. intrusion aid fluidifier

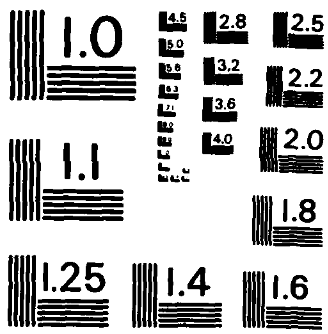
Batching and grout takes were monitored using a calibrated dipstick.

Grouting was initiated at the upstream end of each sluice and was advanced progressively downstream. No specific contract requirements were established to regulate grout takes; however, the shrinkage grouting was continued until return flow appeared at vent pipes or until grout takes dropped below 0.1 cubic foot per minute and, in most cases, below 0.05 cubic foot per minute under header pressures ranging from 35 psi to 50 psi. Total grout take was 128 cubic feet in Sluice D8 where a previously delayed pumpcrete placement had resulted in a large void. Total takes in Sluices D9 and D10 were 15 cubic feet and 9 cubic feet, respectively, indicating little shrinkage.

AD-A160 524 CLARENCE CANNON DAM AND MARK TWAIN LAKE FOUNDATION AND 5/5
EMBANKMENT COMPLET..(U) ARMY ENGINEER DISTRICT ST LOUIS
MO DEC 84

UNCLASSIFIED F/G 13/13 NL





MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

Following completion of shrinkage grouting and form removal, the Ogee section surfaces were inspected and unsatisfactory areas were chiseled, ground and smoothed or veed out and filled with SET 45 as manufactured by Set Products, Inc. Form bolt holes were similarly repaired.

I. Attachments

1. Concrete Materials Flow Diagram

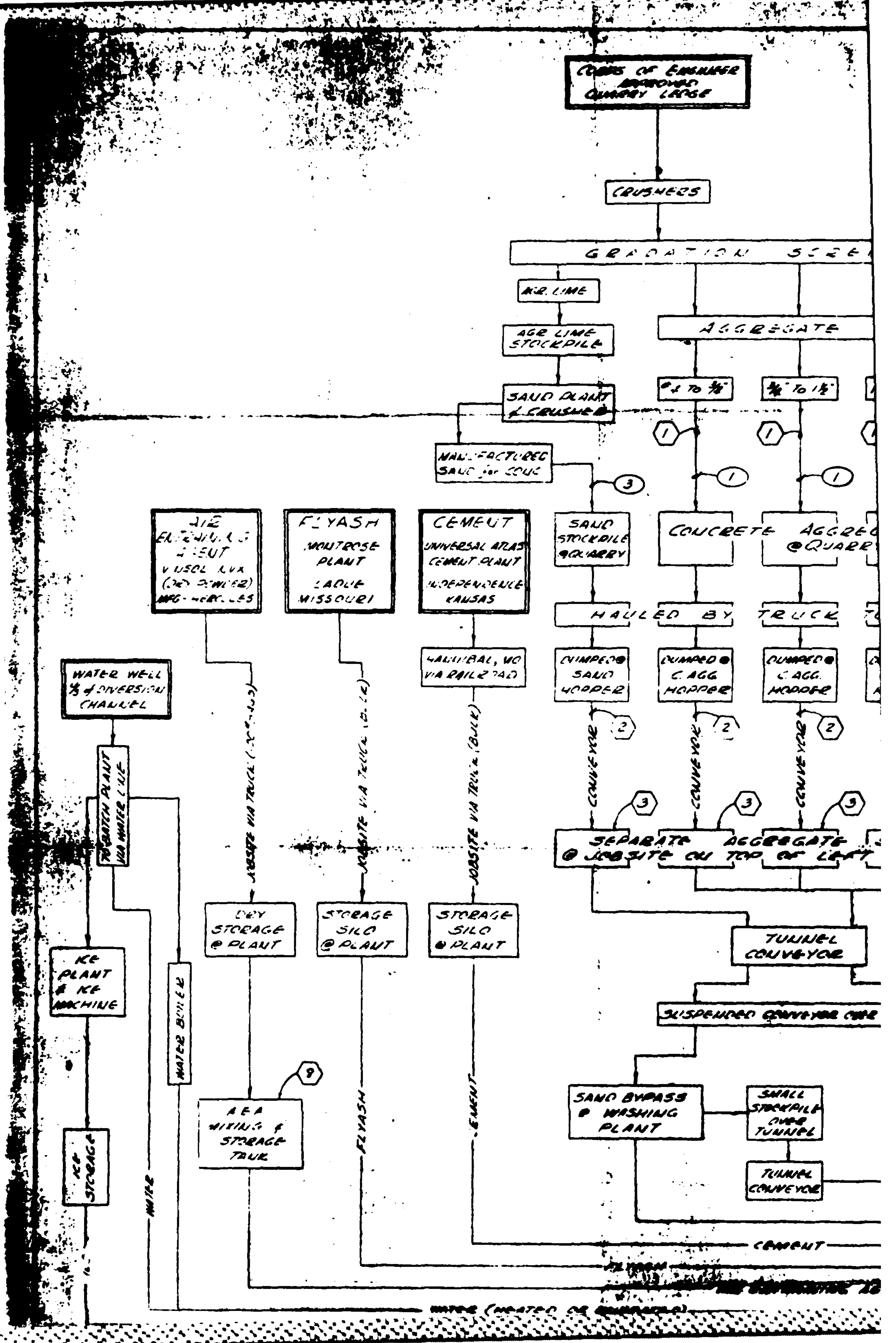
2. Quality Assurance Plan for Concrete Batching and Mixing

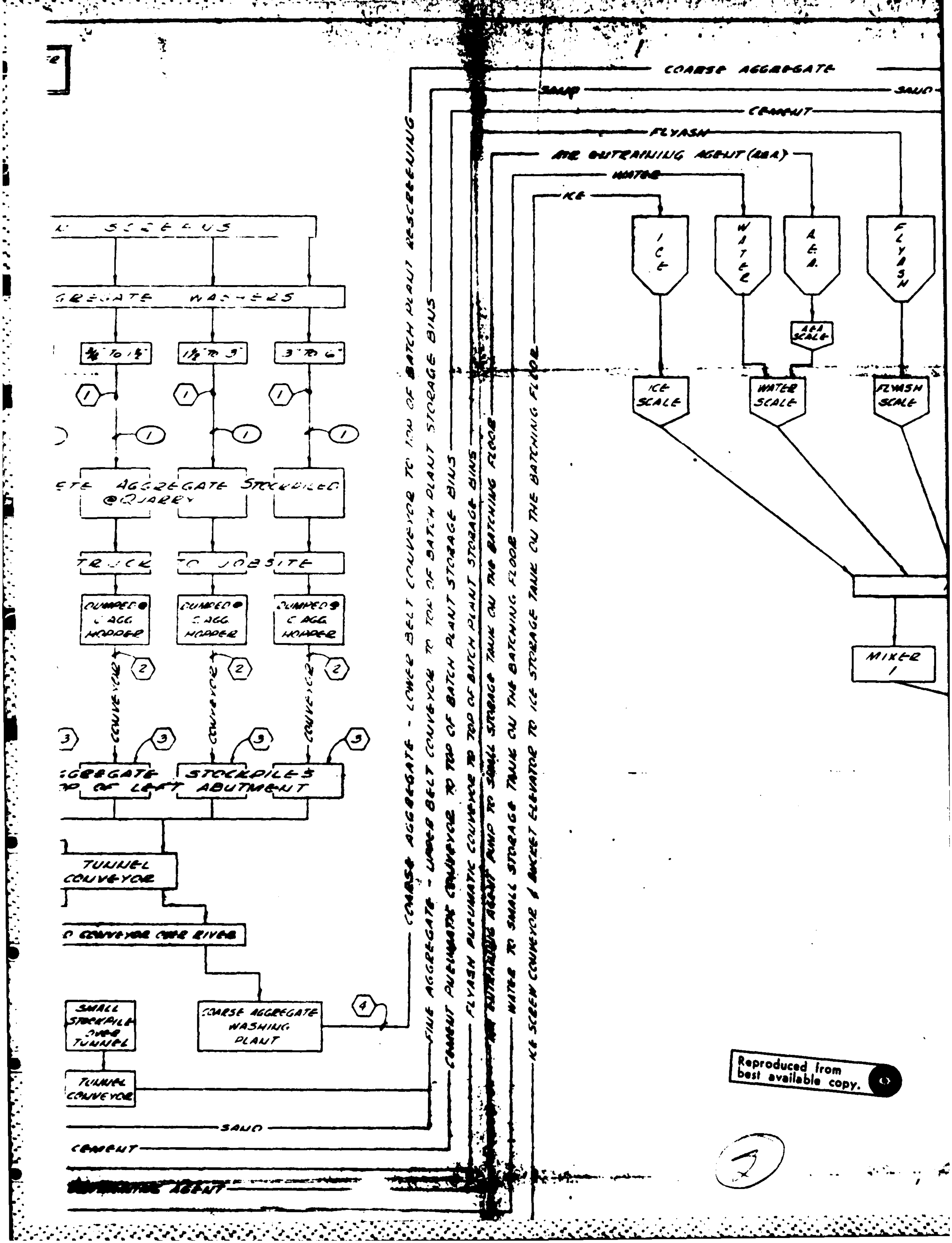
3. Contractor's Quality Control Plan

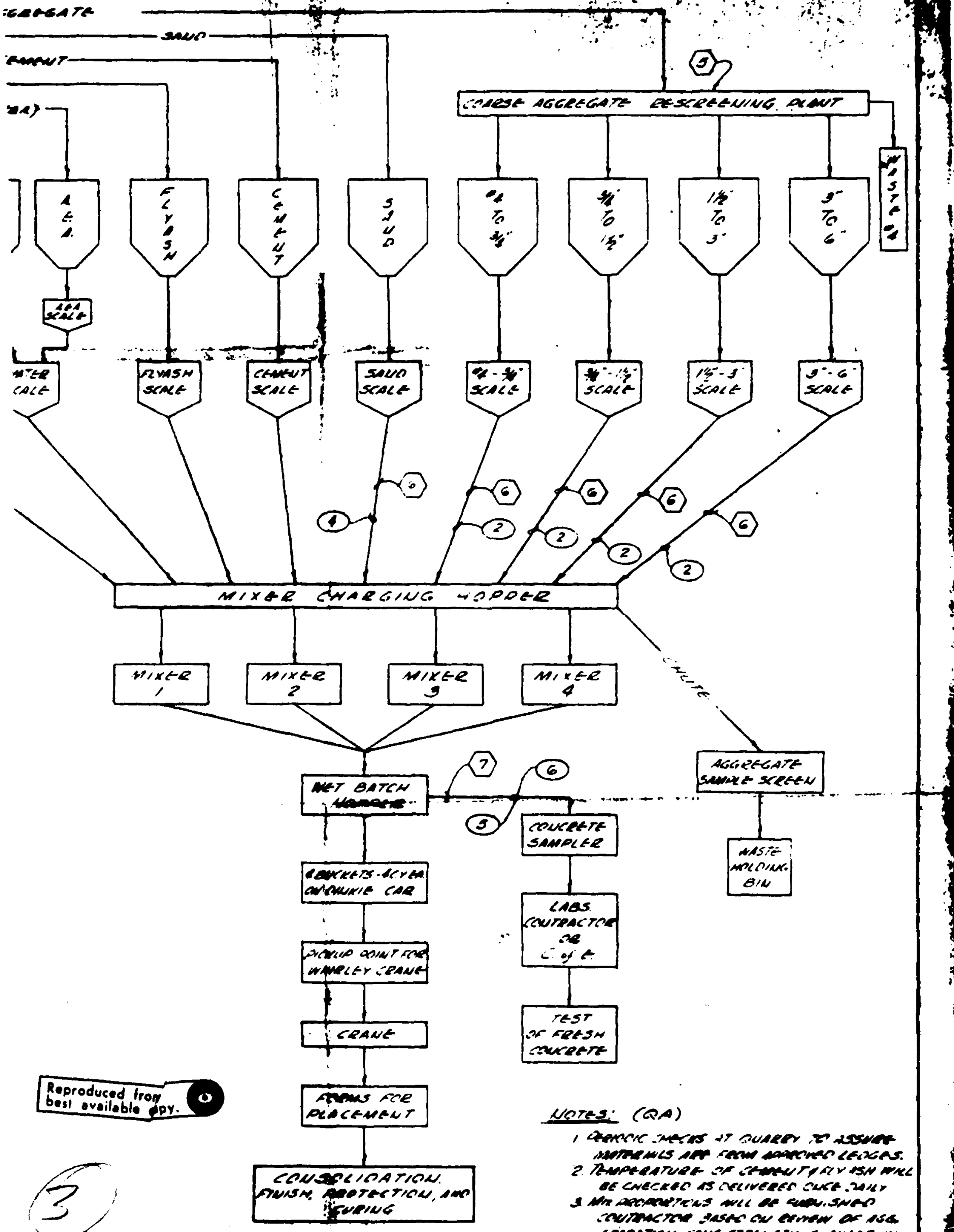
4. Mixer Performance Test by W.E.S

5. Correlation of Curtis Sampler

6. Piping Arrangement for Shrinkage Grouting and Concrete Placement Sequence for Sluices


CORPS OF ENGINEER APPROVED QUARRY LEDGE
CRUSHERS
GRADATION SCREEN
AGR. LIME
AGR LIME STOCKPILE
SAND PLANT & CRUSHER
MANUFACTURED SAND for CONC
AGGREGATE
3/4" TO 1 1/2"
CONCRETE AGGREG @QUARRY
SAND STOCKPILE @QUARRY
HAULED BY TRUCK
DUMPED @ SAND HOPPER
DUMPED @ C. AGG. HOPPER
CONVEYOR
SEPARATE AGGREGATE @ JOBSITE ON TOP OF LEFT
TUNNEL CONVEYOR
SUSPENDED CONVEYOR
SAND BYPASS & WASHING PLANT
SMALL STOCKPILE OVER TUNNEL
TUNNEL CONVEYOR
FLYASH
MONTROSE PLANT
LADUE MISSOURI
CEMENT
UNIVERSAL ATLAS CEMENT PLANT
INDEPENDENCE KANSAS
HANNIBAL, MO VIA RAILROAD
JOBSITE VIA TRUCK (BULK)
STORAGE SILO @ PLANT
DRY STORAGE @ PLANT
AEA MIXING & STORAGE TANK
WATER WELL
TO BATCH PLANT VIA WATER LINE
ICE PLANT & ICE MACHINE
WATER BOILER
ICE STORAGE
WATER
FLYASH
CEMENT
WATER (HEATED OR CHILLED)

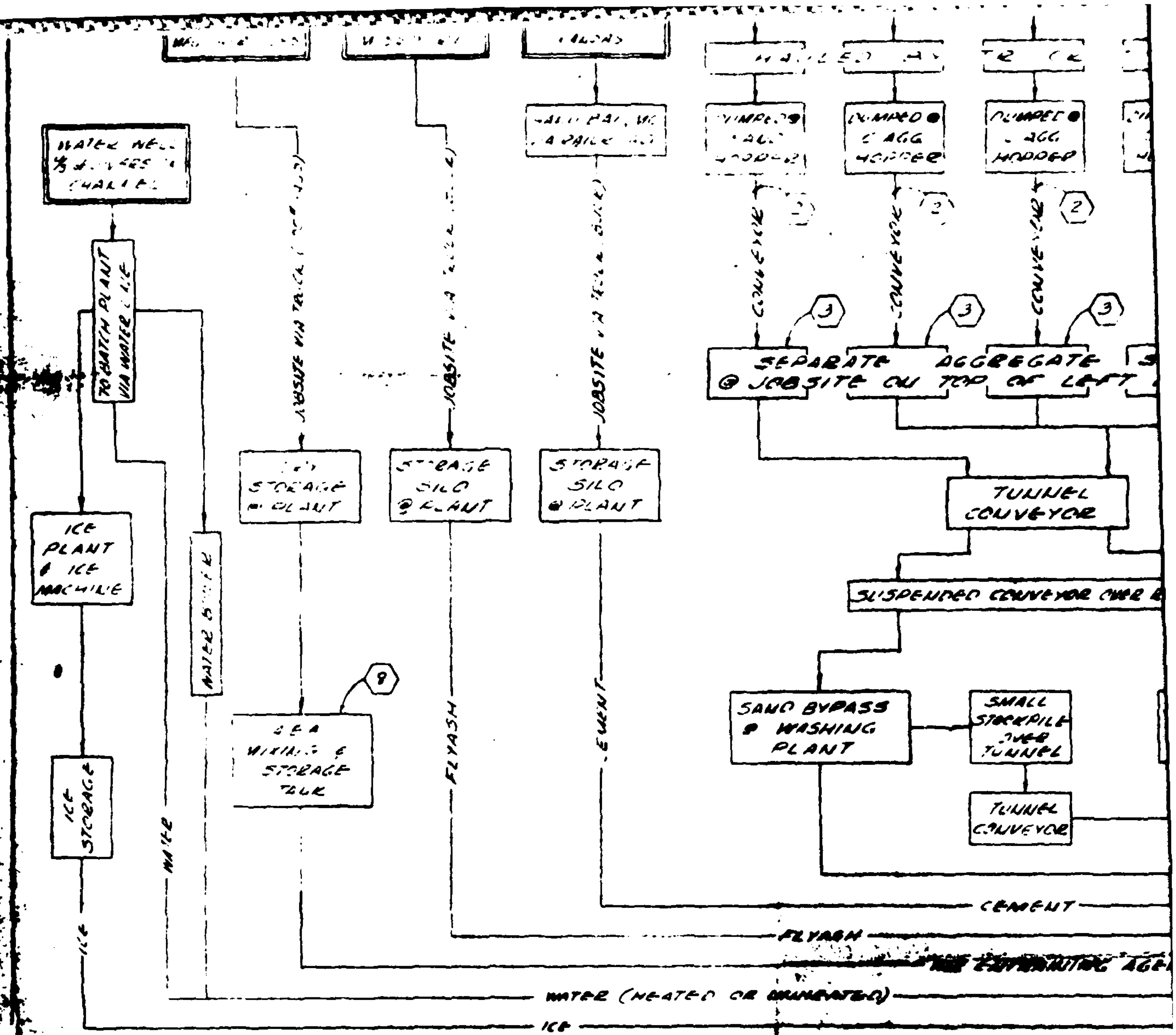

COARSE AGGREGATE
SAND
CEMENT
FLYASH
AIR ENTRAINING AGENT (AEA)
WATER
ICE
ICE
WATER
AEA
FLYASH
AEA SCALE
ICE SCALE
WATER SCALE
FLYASH SCALE
MIXER 1
SCREENS
AGGREGATE WASHERS
3/4" TO 1 1/2"
1 1/2" TO 3"
3" TO 6"
AGGREGATE STOCKPILED @QUARRY
TRUCK TO JOBSITE
DUMPED @ C AGG HOPPER
CONVEYOR
AGGREGATE STOCKPILES
OF LEFT ABUTMENT
TUNNEL CONVEYOR
CONVEYOR OVER RIVER
COARSE AGGREGATE WASHING PLANT
SMALL STOCKPILE OVER TUNNEL
TUNNEL CONVEYOR
SAND
CEMENT
AGENT
COARSE AGGREGATE - LOWER BELT CONVEYOR TO TOP OF BATCH PLANT RESCREENING
FINE AGGREGATE - UPPER BELT CONVEYOR TO TOP OF BATCH PLANT STORAGE BINS
CEMENT PNEUMATIC CONVEYOR TO TOP OF BATCH PLANT STORAGE BINS
FLYASH PNEUMATIC CONVEYOR TO TOP OF BATCH PLANT STORAGE BINS
AIR ENTRAINING AGENT PUMP TO SMALL STORAGE TANK ON THE BATCHING FLOOR
WATER TO SMALL STORAGE TANK ON THE BATCHING FLOOR
ICE SCREW CONVEYOR & BUCKET ELEVATOR TO ICE STORAGE TANK ON THE BATCHING FLOOR



3

NOTES: (QA)

1 PERIODIC CHECKS AT QUARRY TO ASSURE MATERIALS ARE FROM APPROVED LEDGES.
2 TEMPERATURE OF CEMENT & FLY ASH WILL BE CHECKED AS DELIVERED ONCE DAILY
3 MIX PROPORTIONS WILL BE FURNISHED CONTRACTOR BASED ON REVIEW OF AGG. GRADATION, CONC STRENGTH, CHANGES IN CEMENT, FLYASH AND WORKABILITY.
4 ABBREVIATED MIXER PERFORMANCE TEST MAY BE PERFORMED IN ADDITION TO CONTRACTORS
5 [illegible]



LOCATION	CONTRACTOR QUALITY CONTROL	FREQUENCY
1	SIEVE ANALYSIS FOR EACH SIZE	1 EA. PRODUCTION DAY
2	SIEVE ANALYSIS FOR EACH SIZE	1 EA. SHIFT
	MOISTURE CONTENT	1 EA. SHIFT
	PARTICLE SHAPE	WEEKLY IF NEEDED
3	SIEVE ANALYSIS	1 EA. PROD DAY
4	SIEVE ANALYSIS	1 EA. SHIFT
	MOISTURE CONTENT	
	PARTICLE SHAPE	WEEKLY, IF NEEDED
5	AIR CONTENT	
	SLUMP	
6	MIXER	

(QC)

1. ACCURACY OF SCALES
2. ACCURACY OF BATCHING
3. CONTROL CHARTS TO
4.
5.

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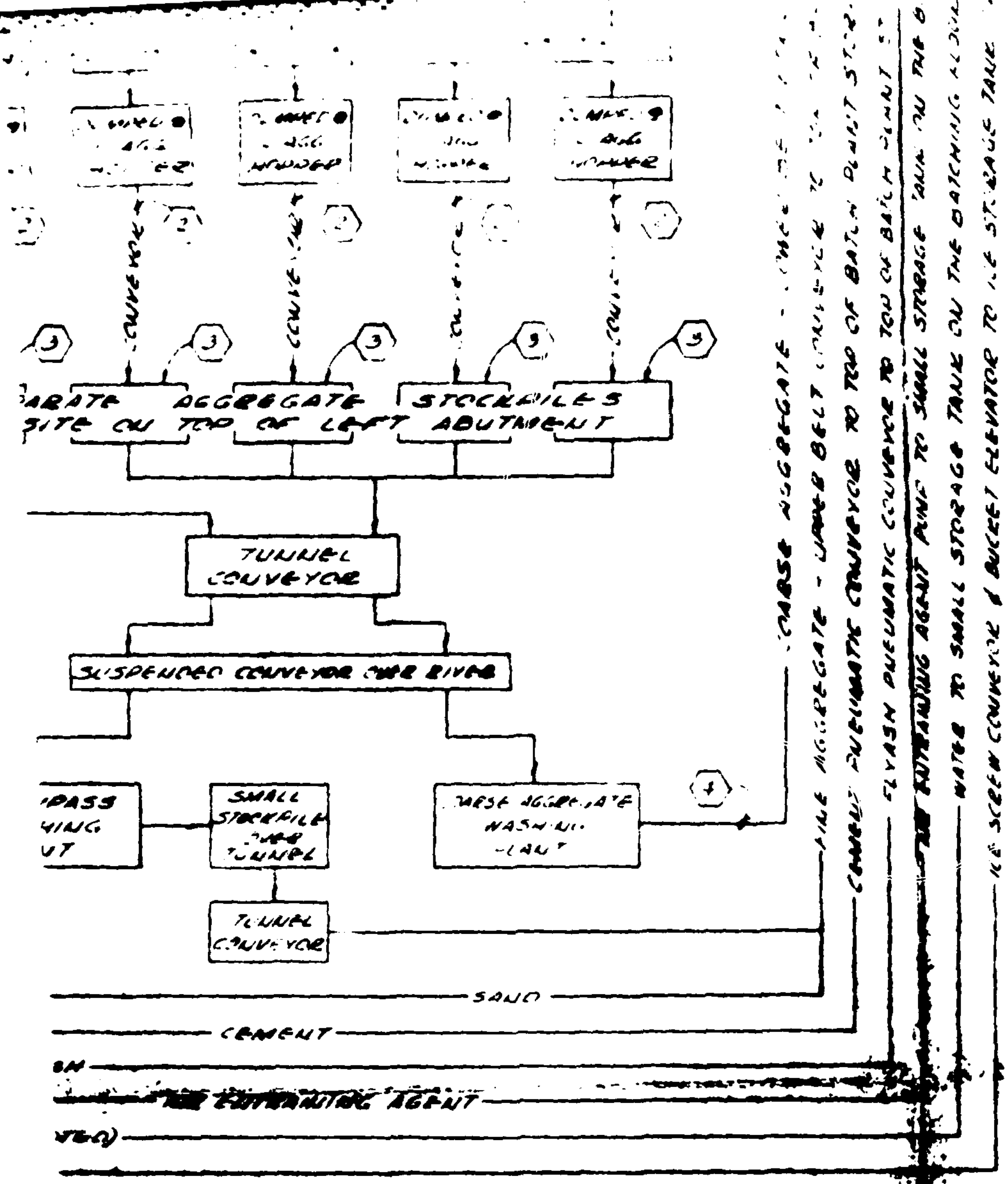
(4)



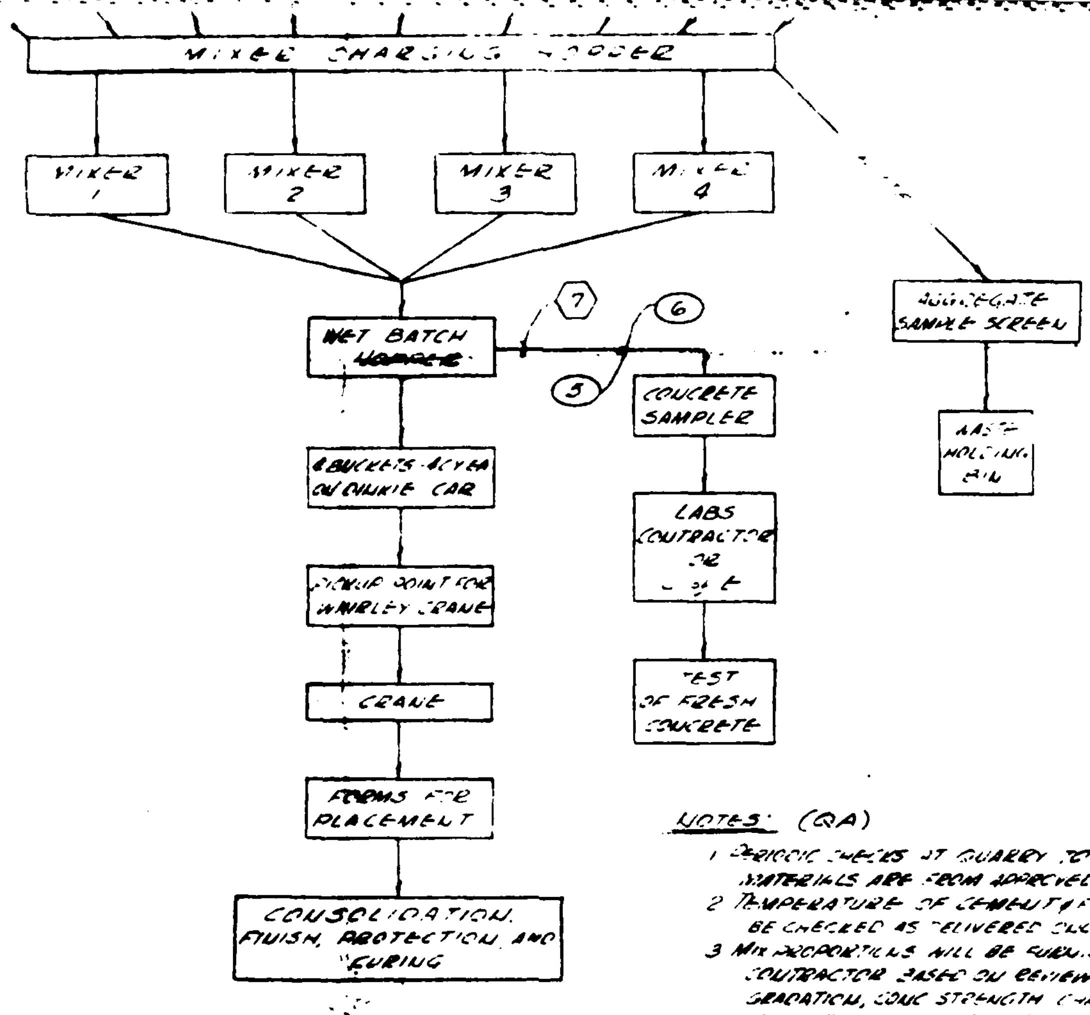
NOTES: (CONT)

1. ACCURACY OF SCALES COMPLETELY CHECKED EVERY 30 DAYS.
2. ACCURACY OF BATCHING & RECORDING CHECKED ONCE EVERY WEEK.
3. CONTROL CHARTS TO BE CONTINUOUSLY PLOTTED INDICATING SLUMP, AIR CONTENT, & FINENESS MODULUS OF SAND & REPORTED TO CORPS OF ENGINEERS WEEKLY.
4. [illegible] WEIGHT DETERMINATION TO BE TAKEN ONCE PER SHIFT PER MIX USED.
5. [illegible] CONTROL TO BE CONTINUOUS.

[illegible]

LOCATION	QUALITY ASSURANCE
1	SIEVE ANALYSIS, SPECIFIC GRAVITY, AND ABSORPTION.
2	STOCKPILING MONITORED, SIEVE ANALYSIS, SPECIFIC GRAVITY & ABSORPTION.
3	VISUAL INSPECTION.
4	CHECK WASHING OPERATIONS.
5	[illegible] COARSE AGG., [illegible] AND SCREEN MAINTENANCE.
	[illegible] AGGREGATE [illegible]

15



NOTES: (QA)

1 PERIODIC CHECKS AT QUARRY TO ASSURE MATERIALS ARE FROM APPROVED LEDGES.
2 TEMPERATURE OF CEMENT & FLYASH WILL BE CHECKED AS DELIVERED ONCE DAILY
3 MIX PROPORTIONS WILL BE FURNISHED CONTRACTOR BASED ON REVIEW OF AGG. GRADATION, CONC STRENGTH CHANGES IN CEMENT, FLYASH AND WORKABILITY.
4 ABBREVIATED MIXER PERFORMANCE TEST MAY BE PERFORMED IN ADDITION TO CONTRACTORS.
5. CORPS OF ENGR. INSPECTION FORCE WILL MONITOR CONTRACTOR QUALITY CONTROL TESTING, INSPECTION, EQUIPMENT USED IN TESTING AND RECORDS DAILY.
6 MIXING WATER - CHECK TEST AT SLD. ONCE EVERY 6. MONTHS
7. 5% OF THE TEST SPECIMENS WILL BE SHIPPED TO WES.

RANGE	FREQUENCY	LOCATION	QUALITY ASSURANCE	FREQUENCY
... GRAVITY,	MONTHLY EITHER AT	6	MOISTURE CONTENTS.	1 EA. SIZE EA. DAY
...VE ANALYSIS,	QUARRY OR	7	AIR CONTENT.	2/MIN/8 HR. SHIFT
...RATION.	STOCKPILE		SLUMP.	2/MIN/8 HR SHIFT
			UNIT WEIGHT.	1 PER WEEK
	EA. PROD. DAY		TEMPERATURE. 5	CONTINUOUSLY
			TEST SPECIMENS ([illegible]/SET/MIN).	1 SET/SHIFT [illegible]
...TIONS.	DAILY	8	[illegible] OBSERVED.	EA. MIXING
., CLEANLINESS ...ON	DAILY		[illegible]	EA. MIXTURE
...RATION.	1 PER 8 HR. SHIFT		[illegible]LING	SAMPLE TO WES/[illegible]
...TURES	1 PER 8 HR. SHIFT			
[illegible]	1 PER [illegible]			

CLARENCE CANNON DAM

U.S. ARMY CORPS OF ENGINEERS
CANNON RESIDENT OFFICE
CONTRACT NO. [illegible]

[illegible]

6

CANNON DAM
QUALITY ASSURANCE PLAN
for
CONCRETE BATCHING AND MIXING

I. AGGREGATES

II. FRESH CONCRETE

III. CEMENT AND FLYASH

IV. AIR ENTRAINING AGENT

V. BATCH PLANT INSPECTION

VI. DOCUMENTATION

I. AGGREGATES

A. PRODUCTION - Periodic checks of the producers' facilities will be made to assure that materials come from approved ledges or pits. Samples may be obtained for Sieve Analysis, Specific Gravity, and Absorptions.

B. STOCKPILING AT JOBSITE - The contractor's operations involving stockpiling of all aggregates will be monitored (for such items as segregation of individual sizes, maintenance of sufficient quantities for making placements, degradation of sizes, etc. Samples again may be taken to check the change in gradation due to handling.

C. WASHING AND RESCREENING FACILITIES - A daily inspection will be made to check the adequacy of the washing and rescreening processes. Coarse Aggregates will be checked for cleanliness immediately after going thru the washing process. Rescreening facilities will be checked for maintenance of screens and removal of fines.

D. ACCEPTANCE TESTING OF AGGREGATES - Tests for acceptance will be made on all aggregates as they are delivered to the mixers. This will be accomplished by obtaining samples from the scales at the Batching Plant. The schedule of tests are as follows:

1. Coarse Aggregate - Each size of coarse aggregate will be tested for gradation once per 8-hour shift.

2. Fine Aggregate - A sieve analysis and fineness modules determination will be made once per 8-hour shift. A FM Control Chart is maintained.

E. MISCELLANEOUS TESTS PERFORMED ON AGGREGATES - The following tests will be performed and at the frequency stated below:

1. Particle Shape - Once per week.

2. Bulk Specific Gravity - Once per month.

3. Absorption - Once per month.

4. Material finer than #200 sieve - Once per month.

5. Moisture Contents - One per size each day.

II. FRESH CONCRETE

A. AIR CONTENT - Tests for air content will be performed at the rate of two tests per mix used per 8-hour shift. A control chart will also be maintained.

B. SLUMP - Tests for slump will be performed at the rate of two tests per mix used per 8-hour shift. A control chart will also be maintained.

C. UNIT WEIGHT - A test to determine unit weights will be performed on each mix used at the rate of one test per week.

D. TEMPERATURE - Constant checks of temperature of the fresh concrete will be made to assure compliance with the specification requirements.

E. TEST SPECIMENS FOR STRENGTH - A set of four 6"x12" specimens will be cast for each mix used at the rate of one set per shift.

III. CEMENT AND FLYASH

A. All shipments of these materials will be checked to assure they are from tested and approved sources.

B. TEMPERATURE - Cement will be checked once daily for temperature.

C. All test reports of these materials will be reviewed so that changes in mix strength, workability, air content, etc., may be anticipated and proper action taken.

IV. AIR ENTRAINING AGENT

A. Air Entraining Agent on this project is job mixed.

B. MIXING - The mixing of air entraining agent will be inspected to assure proper procedures are used.

C. TESTING AND SAMPLING - The testing and sampling procedures as set forth by the Resident Office will be followed.

V. BATCH PLANT INSPECTION

A. MIX PROPORTIONS - These will be furnished to the contractor based upon determinations made by the review of all data regarding gradations of aggregates, strength of concrete, changes in cement, and flyash and workability.

B. WEIGHING, BATCHING, AND MIXING - As follows:

1. Scales - They will be checked for accuracy every 20 batching days by the contractor under the supervision of the Corps Inspector. An additional scale check will be performed anytime repairs are made to the scales or it is apparent that a problem exists.

2. Batching and Recording - A check of batching accuracy and recording accuracy will be performed a minimum of four times per 8-hour shift by the Plant Inspector. The record of batching (graph) will be removed and checked in detail at the end of each placing day.

3. Mixing - An abbreviated Mixer Performance Test may be performed in addition to the contractor's tests to assure proper mixing action and time so that uniformity is maintained.

C. MONITORING OF CONTRACTOR QUALITY CONTROL - As follows:

1. The Corps Inspection force will constantly monitor the contractor in his performance of the specified Quality Control Testing and Inspection.

2. All records maintained by the contractor and equipment used in testing will be subjected to a daily review and inspection for completeness and operation.

VI. DOCUMENTATION

A. RECORDS - All testing performed for Quality Assurance will be recorded on appropriate standard forms and local forms.

B. OTHER DATA - Miscellaneous data regarding problems with specifications, materials, etc., will be compile..

CONCRETE PRODUCTION - CANNON DAM
CONTRACTOR QUALITY CONTROL
TESTS AND INSPECTION

I. COARSE AGGREGATES

A. Quarry Production - one sieve analysis per size each production day.

B. Batch Plant - one sieve analysis per size per shift, one moisture content per size per shift, and one particle shape per week if needed.

II. FINE AGGREGATES (SAND)

A. Sand Plant Production - one sieve analysis with FM per production day.

B. Batch Plant - one sieve analysis with FM per shift, two pairs moisture contents per 8-hours, and one particle shape per week if needed.

III. SCALES -

A. Scale Accuracy - one complete scale check every 20 days.

B. Accuracy of Batching and Recording - one per week.

IV. FRESH CONCRETE

A. Air Content - two per mix per 8-hour shift.

B. Slump - two per mix per 8-hour shift.

V. MIXER PERFORMANCE TEST

A. Complete - Each mixer once per year

B. Abbreviated - Each mixer once every six months.

VI. CONTROL CHARTS

A. Slump - continuous plots reported to Corps every week.

B. Air Content - continuous plots reported to Corps every week.

C. Fineness Modulus of Sand - continuous plots reported to Corps every week.

VII. BATCH WEIGHT DETERMINATION - Once per shift for each mix used.

VIII. BATCH PLANT CONTROL - Continuous

DEPARTMENT OF THE ARMY
WATERWAYS EXPERIMENT STATION, CORPS OF ENGINEERS
P. O. BOX 631
VICKSBURG, MISSISSIPPI 39180

REPLY REFER TO: WESCC 24 September 1975

SUBJECT: Mixer Performance Test for Clarence Cannon Dam

District Engineer
U. S. Army Engineer District, St. Louis
210 N. 12th St.
St. Louis, MO 63101

1. Reference is made to your DA Form 2544, No. ED 57-75, dated 9 October 1974.

2. Concrete mixer performance tests as outlined in CRD-C 55 were conducted at the site on an automatic mixing plant with four 4-cu-yd-capacity tilting mixers manufactured by the Koehring Company. The tests were conducted by personnel of the Waterways Experiment Station (WES) during the period 19-27 August 1975.

3. The mixing plant was operating on a limited schedule when WES personnel arrived at the jobsite on 19 August 1975. Sampling and testing were conducted so as to result in a minimal amount of interference with the contractor concrete mixing operations. The District personnel at the site were very helpful in coordinating work with the contractor.

4. Twelve 4-cu-yd batches of concrete representing three mixtures were sampled and tested for variations in water content of mortar, coarse aggregate content, unit weight of air-free mortar, and cement content of dried mortar. Mixture proportions are given in Incl 1.

5. Samples were taken from the wet-batch hopper representing the first, middle, and last portions of the mixer discharge and tested as outlined in CRD-C 55. The Standard Guide Specifications for Concrete, CE 1401.01, July 1973, gives the maximum allowable variations for the test results in paragraph 10.10 for automatic batching and mixing plants. The test results of the 12 batches sampled and tested are given in Incl 2. The cycle time includes 12-15 sec charging time in addition to the mixing time indicated.

6. The concrete from all batches appeared well mixed. Some segregation was observed during sampling of mixture E, due primarily to the high slump in combination with the 6-in. maximum size aggregate material. The test

results indicate that a 90 sec mixing time is satisfactory for all mixtures and all mixers. A 75 sec mixing time appears to be satisfactory only for concrete having slumps greater than 2 in., therefore 90 sec is the recommended minimum mixing time. The water content variation exceeded the allowable when a 75 sec mixing time was used to mix lower slump concrete (2 in. or less).

FOR THE DIRECTOR:

2 Incl
as

BRYANT MATHER
Engineer
Chief, Concrete Laboratory

CF w incl:
LMVD, ATTN: LMVED-G

Mixtures

Mix Designation	Maximum Size Aggregate, in.	Water Content	Cement,* lb/cu yd	f'c, psi
C	3	0.54	368	3000
E	6	0.67	218	2500
H	1-1/2	0.45	517	3000

* Cement plus fly ash.

All mixtures proportioned to have 1-1/2 ± 1/2 in. slump and 5.0 ± 0.5 percent air.

Test Results

Mix No.	Mixer No.	Mixing Time, sec	Slump, in.	Variability Index: Unit Weight of Air-Free Mortar	Variability Index: Coarse Aggregate Content	Variability Index: Water Content of Mortar	Variability Index: Cement Content of Dried Mortar
Specification Limit (min)				98.5	90.5	91.5	82.5
C	1	90	1	98.3*	98.6	87.0*	92.9
C	1	90	1-1/4	99.3	93.0	97.0	98.6
C	1	75	1-3/4	99.2	95.0	90.6*	97.0
C	2	90	2-3/4	98.6	90.5	92.1	92.4
C	3	90	2	99.1	92.9	96.0	93.8
C	3	75	2-1/4	99.6	95.2	94.3	99.2
C	4	90	2-3/4	99.5	96.3	95.7	96.1
E	1	90	3-1/2	99.7	98.7	97.2	85.1
E	2	90	3-1/2	98.6	94.0	94.4	95.6
E	3	90	3	99.8	95.6	94.1	86.5
E	4	90	4	99.6	93.3	95.1	95.8
H	1	90	1	99.6	97.5	99.3	98.4

* Exceeds specification limit.

MASSMAN CONSTRUCTION CO. — CANNON DAM PROJECT
KANSAS CITY, MISSOURI

Webb

COPY

R. R. # 2
Center, Missouri 63436

October 22, 1976

Mr. Otto K. Steffens
U. S. Army Corps of Engineers
Clarence Cannon Resident Office
R. R. # 2
Center, Missouri 63436

Re: Clarence Cannon Dam
Contract No. DACW 43-73-C-0134

Dear Sir:

Enclosed are results of correlation tests on Curtis Sampler.

Tests were performed from September 22 to October 15, 1976 by Massman Construction Co. Quality Control Technicians.

The following methods will be used to operate Curtis Sampler for gradation testing.

#4 - 3/4" Aggregate
11° Slope, 6 Min @ 1400 R.P.M., 1 Min @ 2000 R.P.M. Fast Feed, Gate down

3/4" - 1 1/2" Aggregate
10° Slope, 4 @ 1800 R.P.M.,2 Min @ 2200 R.P.M., Fast Feed, Gate down

1 1/2 - 3" Aggregate
8° Slope, 5 Min @ 2300 R.P.M., Fast Feed, Gate 1/2 open

3" - 6" Aggregate
8° Slope, 5 Min @ 2300 R.P.M., Fast feed, Gate Open.

These methods are same as derived from correlation tests performed February,1976.

Very truly yours,

MASSMAN CONSTRUCTION CO.
CANNON DAM PROJECT

Paul L. Baker
Quality Control

PLB:pjp
CC: KC Office
Encl.

SUMMARY

#4 - 3/4" AGGREGATE

3/4"	Curtis	Gilson	Diff.
1	96.4	94.5	1.9
2	96.5	94.5	2.0
3	95.0	92.7	2.3
3/8"			
1	37.5	36.5	1.0
2	36.4	35.9	0.5
3	28.9	27.6	1.3

NOTE: # 4 Screen out of Curtis

Total Material Run '

Curtis	1167
Gilson	1155.5

METHODS OF OPERATION

TEST NUMBER

1 — 11^{o} Slope, 6 Min @ 1400 R.P.M., 1 Min @ 2000 R.P.M.
Fast Feed, Gate Down

2 — 11^{o} Slope, 6 Min @ 1400 R.P.M., 1 Min @ 2000 R.P.M.
Fast Feed, Gate Down

3 — 11^{o} Slope, 6 Min @ 1400 R.P.M., 1 Min @ 2000 R.P.M.
Fast Feed, Gate Down

SUMMARY

3/4" - 1 1/2" AGGREGATE

1 1/2"	CURTIS	GILSON	DIFF.
1	97.6	96.8	0.8
2	97.9	98.1	0.2
3	94.0	92.9	1.1
4	97.8	96.8	1.0
1"			
1	19.9	24.3	4.4
2	39.2	40.4	1.2
3	24.0	25.7	1.7
4	39.2	39.3	0.1
3/4"			
1	3.3	6.6	3.3
2	8.6	5.3	3.3
3	3.8	4.8	1.0
4	6.0	6.3	0.3
3/8"			
1	1.8	5.1	3.3
2	1.4	1.4	0.0
3	1.5	2.6	1.1
4	2.0	3.0	1.0

Total Material Run

Curtis	1703
Gilson	1682.5

METHODS OF OPERATION

TEST NUMBER	
1	10^{o} Slope, 4 Min @ 1800 R.P.M., 2 Min @ 2200 R.P.M., Fast Feed, Gate Down
2	10^{o} Slope, 4 Min @ 1800 R.P.M., 2 Min @ 2200 R.P.M., Fast Feed, Gate Down
3	10^{o} Slope, 4 Min @ 1800 R P.M., 2 Min @ 2200 R.P.M., Fast Feed, Gate Down
4	10^{o} Slope, 4 Min @ 1800 R.P.M., 2 Min @ 2200 R.p.m., Fast Feed, Gate Down

SUMMARY

1 1/2" - 3" AGGREGATE

	CURTIS	GILSON	DIFF.
3"			
1	90.1	90.0	0.1
2	92.7	91.3	1.4
3	96.2	95.8	0.4
2"			
1	21.4	18.7	2.7
2	26.1	23.7	2.4
3	39.9	37.3	2.6
1 1/2"			
1	4.4	4.5	0.1
2	6.0	6.0	0.0
3	6.7	6.4	0.3
1"			
1	2.7	3.0	0.3
2	2.7	3.2	0.5
3	2.9	3.5	0.6

Total Material Run

Curtis	2507
Gilson	2483

METHODS OF OPERATION

TEST NUMBER	
1	8^{o} Slope, 5 Min @ 1800 R.P.M., Fast Feed, Gate 1/2 Open
2	8^{o} Slope, 5 Min @ 1800 R.P.M., Fast Feed, Gate 1/2 Open
3	8^{o} Slope, 5 Min @ 1800 R.P.M., Fast Feed, Gate 1/2 Open

SUMMARY

3" - 6" AGGREGATE

	CURTIS	GILSON	DIFF.
6"			
1	95.4	95.3	0.1
2	98.0	98.1	0.1
3	97.2	97.1	0.1
4"			
1	53.0	51.6	1.4
2	58.2	58.9	0.7
3	47.2	47.5	0.3
3"			
1	24.9	25.5	0.6
2	14.0	15.8	1.8
3	10.3	11.7	1.4
2"			
1	15.5	15.3	0.2
2	3.5	4.0	0.5
3	3.2	3.6	0.4

Total Material Run

Curtis	3703
Gilson	3679

METHODS OF OPERATION

TEST NUMBER	
1	8^{o} Slope, 5 Min @ 2300 R.P.M., Fast Feed, Gate open
2	8^{o} Slope, 5 Min @ 2300 R.P.M., Fast Feed, Gate Open
3	8^{o} Slope, 5 Min @ 2300 R.P.M., Fast Feed, Gate Open

CONCRETE AGGREGATE TESTS — MASSMAN CONSTRUCTION CO.

CANNON DAM PROJECT — DACW-43-73-C-0134

SAND ANALYSIS

DATE 10-15-76

	1			2			3			
e Size	Wt.Ret.	% Ret.	F.M.	Wt.Ret.	% Ret.	F.M.	Wt.Ret.	% Ret.	F.M.	% Spec.
#4										0-5
#8										5-20
#16										10-22
#30										18-32
#50										18-32
#100										10-25
Pan										5-10
Total										

AGGREGATE ANALYSIS #4 - 3/4"

	1 #1 Curtis			2 #1 Gilson			3 Diff.			
Sieve Size	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
1"	—	—	100	—	—	100		—		100
3/4"	7	3.6	96.4	10.5	5.5	94.5		1.9		90-100
3/8"	120	62.5	37.5	121.0	63.5	36.5		1.0		20-45
#4	192	100	0	190.5	100	0		—		0-5
Total	192			190.5						

AGGREGATE ANALYSIS 3/4" - 1 1/2"

	1			2			3			
Sieve Size	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
										100
1/2"										90-100
1"										20-45
3/4"										0-10
3/8"										0-5
Total										

AGGREGATE ANALYSIS 1 1/2" - 3"

	1			2			3			
Sieve Size	Wt.Ret.	% Ret	% Pass	St.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
4"										100
3"										90-100
2"										20-55
1 1/2"										0-10
1"										0-5
Total										

AGGREGATE ANALYSIS 3" - 6"

	1			2			3			
Si Size	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
										100
6"										90-100
4"										20-55
										0-15
										0-5
Total										

REMARKS: 6 MIN @ 1400 R.P.M., 1 MIN @ 2000 R.P.M., FAST FEED, GATE DOWN 11° SLOPE.

Paul L. Baker

LAB TECH

CONCRETE AGGREGATE TESTS — CANNON DAM PROJECT

MASSMAN CONSTRUCTION CO. — DACW-43-73-C-0134

SAND ANALYSIS

DATE 10-18-76

Sieve Size	1 Wt.Ret.	1 % Ret.	1 F.M.	2 Wt.Ret.	2 % Ret.	2 F.M.	3 Wt.Ret.	3 % Ret.	3 F.M.	% Spec.
#4			~			~			~	0-5
#8										5-20
#16										10-22
[illegible]										18-32
#50										18-32
#100										10-25
Pan			~			~			~	5-10
Total										~

AGGREGATE ANALYSIS #4 - 3/4"

1 #2 CURTIS — 2 #2 GILSON — 3 DIFF.

Sieve Size	1 Wt.Ret.	1 % Ret.	1 % Pass	2 Wt.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
1"	—	—	100	—	—	100		—		100
3/4"	18	3.5	96.5	28.0	5.5	94.5		2.0		90-100
3/8"	327	63.6	36.4	326.0	64.1	35.9		0.5		20-45
#4	514	100	0	508.5	100	0		0		0-5
Total	514			508.5						~

AGGREGATE ANALYSIS 3/4" - 1 1/2"

Size	1 Wt.Ret.	1 % Ret.	1 % Pass	2 Wt.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
										100
1 1/2"										90-100
1"										20-45
3/4"										0-10
3/8"										0-5
Total										~

AGGREGATE ANALYSIS 1 1/2" - 3"

Sieve Size	1 Wt.Ret.	1 % Ret	1 % Pass	2 St.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
4"										100
3"										90-100
2"										20-55
1 1/2"										0-10
1"										0-5
Total										~

AGGREGATE ANALYSIS 3" - 6"

Sieve Size	1 Wt.Ret.	1 % Ret.	1 % Pass	2 Wt.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
7"										100
6"										90-100
[illegible]										20-55
[illegible]										0-15
2"										0-5
Total										~

REMARKS: 6 MIN @ 1400 R.P.M, 1 MIN @ 2000 R.P.M, FAST FEED, GATE DOWN 11° SLOPE.

Paul L. B[illegible]

CONCRETE AGGREGATE TESTS — CANNON DAM PROJECT

MASSMAN CONSTRUCTION CO. — DACW-43-73-C-0134

SAND ANALYSIS

DATE 10-19-76

Sieve Size	1 Wt.Ret.	1 % Ret.	1 F.M.	2 Wt.Ret.	2 % Ret.	2 F.M.	3 Wt.Ret.	3 % Ret.	3 F.M.	% Spec.
#4			~			~			~	0-5
#8										5-20
#16										10-22
[illegible]										16-32
#50										18-32
#100										10-25
Pan			~			~			~	5-10
Total										~

AGGREGATE ANALYSIS #4 - 3/4"

1 #3 Curtis — 2 #3 Gilson — 3 Diff.

Sieve Size	1 Wt.Ret.	1 % Ret.	1 % Pass	2 Wt.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
1"	—	—	100	—	—	100		—		100
3/4"	23	5.0	95.0	33.5	7.3	92.7		2.3		90-100
3/8"	328	71.1	28.9	330.5	72.4	27.6		1.3		20-45
#4	461	100	0	456.5	100	0		—		0-5
Total	461			456.5						~

AGGREGATE ANALYSIS 3/4" - 1 1/2"

Sieve Size	1 Wt.Ret.	1 % Ret.	1 % Pass	2 Wt.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
[illegible]										100
1 1/2"										90-100
1"										20-45
3/4"										0-10
3/8"										0-5
Total										~

AGGREGATE ANALYSIS 1 1/2" - 3"

Sieve Size	1 Wt.Ret.	1 % Ret	1 % Pass	2 St.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
4"										100
3"										90-100
2"										20-55
1 1/2"										0-10
1"										0-5
Total										~

AGGREGATE ANALYSIS 3" - 6"

Sieve Size	1 Wt.Ret.	1 % Ret.	1 % Pass	2 Wt.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
[illegible]										100
6"										90-100
4"										20-55
[illegible]										0-15
[illegible]										0-5
Total										~

REMARKS: 6 MIN @ 1400 R.P.M, 1 MIN @ 2000 R.P.M, FAST FEED, GATE DOWN 11° SLOPE

Paul L. Baker

CONCRETE AGGREGATE TESTS — CANNON DAM PROJECT

HASSMAN CONSTRUCTION CO. — DACW-43-73-C-0134

SAND ANALYSIS

DATE 9-22-76

Sieve Size	1 Wt.Ret.	1 % Ret.	1 F.M.	2 Wt.Ret.	2 % Ret.	2 F.M.	3 Wt.Ret.	3 % Ret.	3 F.M.	% Spec.
#4			~			~			~	0-5
#8										5-20
#16										10-22
[illegible]										18-32
#50										18-32
#100										10-25
Pan			~			~			~	5-10
Total										~

AGGREGATE ANALYSIS #4 - 3/4"

Sieve Size	1 Wt.Ret.	1 % Ret.	1 % Pass	2 Wt.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
1"										100
3/4"										90-100
3/8"										20-45
#4										0-5
Total										~

AGGREGATE ANALYSIS 3/4" - 1 1/2"

1 #1 CURTIS — 2 #1 GILSON — 3 DIFF

Sieve Size	1 Wt.Ret.	1 % Ret.	1 % Pass	2 Wt.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
	—	—	100	—	—	100		—		100
1 1/2"	8	2.4	97.6	11	3.2	96.8		0.8		90-100
1"	270	80.1	19.9	258	75.7	24.3		4.4		20-45
3/4"	326	96.7	3.3	318.5	93.4	6.6		3.3		0-10
3/8"	331	98.2	1.8	323.5	94.9	5.1		[illegible]		0-5
Total	337			341.0						~

AGGREGATE ANALYSIS 1 1/2" - 3"

Sieve Size	1 Wt.Ret.	1 % Ret	1 % Pass	2 St.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
4"										100
3"										90-100
2"										20-55
1 1/2"										0-10
1"										0-5
Total										~

AGGREGATE ANALYSIS 3" - 6"

Sieve Size	1 Wt.Ret.	1 % Ret.	1 % Pass	2 Wt.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
7"										100
6"										90-100
[illegible]										20-55
[illegible]										0-15
[illegible]										0-5
Total										~

REMARKS: DIRTY SAMPLE, EXCESSIVE PAN
1800 R.P.M, 4 MIN - 2200 R.P.M, 2 MIN 10° SLOPE F.E.

Paul L. Baker

CONCRETE AGGREGATE TESTS — MASSMAN CONSTRUCTION CO.

CANNON DAM PROJECT — DACW-43-73-C-0134

SAND ANALYSIS

DATE 9-23-76

Sieve Size	1 Wt.Ret.	1 % Ret.	1 F.M.	2 Wt.Ret.	2 % Ret.	2 F.M.	3 Wt.Ret.	3 % Ret.	3 F.M.	% Spec.
#4			~			~			~	0-5
#8										5-20
#16										10-22
[illegible]0										18-32
#50										18-32
#100										10-25
Pan			~			~			~	5-10
Total										~

AGGREGATE ANALYSIS #4 - 3/4"

Sieve Size	1 Wt.Ret.	1 % Ret.	1 % Pass	2 Wt.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
1"										100
3/4"										90-100
3/8"										20-45
#4										0-5
Total										~

AGGREGATE ANALYSIS 3/4" - 1 1/2"

1 #2 CURTIS — 2 #2 GILSON — 3 DIFF

Sieve Size	1 Wt.Ret.	1 % Ret.	1 % Pass	2 Wt.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
	—	—	100	—	—	100		—		100
1 1/2"	6	2.1	97.9	5.5	1.9	98.1		0.2		90-100
1"	177	60.8	39.2	170.0	59.6	40.4		1.2		20-45
3/4"	266	91.4	8.6	270.0	94.7	5.3		3.3		0-10
3/8"	287	98.6	1.4	281.0	98.6	1.4		0.0		0-5
Total	291			285.0						~

AGGREGATE ANALYSIS 1 1/2" - 3"

Sieve Size	1 Wt.Ret.	1 % Ret	1 % Pass	2 St.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
4"										100
3"										90-100
2"										20-55
1 1/2"										0-10
1"										0-5
Total										~

AGGREGATE ANALYSIS 3" - 6"

Sieve Size	1 Wt.Ret.	1 % Ret.	1 % Pass	2 Wt.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
7"										100
6"										90-100
[illegible]										20-55
										0-15
2"										0-5
Total										~

REMARKS: 1800 R.P.M 4 MIN ; 2 MIN @ 2200 R.P.M

10° SLOPE FAST FEED.

Paul L. [illegible]

CONCRETE AGGREGATE TESTS
CANNON DAM PROJECT

MASSMAN CONSTRUCTION CO.
DACW-43-73-C-0134

SAND ANALYSIS

DATE 9-27-76

	1			2			3			
Sieve Size	Wt.Ret.	% Ret.	F.M.	Wt.Ret.	% Ret.	F.M.	Wt.Ret.	% Ret.	F.M.	% Spec.
#4										0-5
#8										5-20
#16										10-22
#30										18-32
#50										18-32
#100										10-25
Pan										5-10
Total										

AGGREGATE ANALYSIS #4 - 3/4"

	1			2			3			
Sieve Size	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
1"										100
3/4"										90-100
3/8"										20-45
#4										0-5
Total										

AGGREGATE ANALYSIS 3/4" - 1 1/2"

	1 #3 CURTIS			2 #3 GILSON			3 DIFF.			
Sieve Size	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
	—	—	100	—	—	100		—		100
1 1/2"	31	6.0	94.0	36.5	7.1	92.9		1.1		90-100
1"	396	76.0	24.0	379.5	74.3	25.7		1.7		20-45
3/4"	501	96.2	3.8	486.0	95.2	4.8		1.0		0-10
3/8"	513	98.5	1.5	497.0	97.4	2.6		1.1		0-5
Total	521			510.5						

AGGREGATE ANALYSIS 1 1/2" - 3"

	1			2			3			
Sieve Size	Wt.Ret.	% Ret	% Pass	St.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
4"										100
3"										90-100
2"										20-55
1 1/2"										0-10
1"										0-5
Total										

AGGREGATE ANALYSIS 3" - 6"

	1			2			3			
Sieve Size	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
7										100
6"										90-100
4"										20-55
										0-15
										0-5
Total										

REMARKS: 4 MIN @ 1800 R.P.M — 2 MIN @ 2200 R.P.M
10° SLOPE, F.F. GATE DOWN.

Paul L. Baker

CONCRETE AGGREGATE TESTS — MASSMAN CONSTRUCTION CO.

CANNON DAM PROJECT — DACW-43-73-C-0134

SAND ANALYSIS

DATE 9-29-76

Sieve Size	1 Wt.Ret.	1 % Ret.	1 F.M.	2 Wt.Ret.	2 % Ret.	2 F.M.	3 Wt.Ret.	3 % Ret.	3 F.M.	% Spec.
#4			~			~			~	0-5
#8										5-20
#16										10-22
[illegible]										18-32
#50										18-32
#100										10-25
Pan			~			~			~	5-10
Total										~

AGGREGATE ANALYSIS #4 - 3/4"

Sieve Size	1 Wt.Ret.	1 % Ret.	1 % Pass	2 Wt.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
1"										100
3/4"										90-100
3/8"										20-45
#4										0-5
Total										~

AGGREGATE ANALYSIS 3/4" - 1 1/2"

1 #4 CURTIS — 2 #4 GILSON — 3 DIFF.

Size	1 Wt.Ret.	1 % Ret.	1 % Pass	2 Wt.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
	—	—	100	—	—	100		—		100
1 1/2"	12	2.2	97.8	17.5	3.2	96.8		1.0		90-100
1"	337	60.8	39.2	331.5	60.7	39.3		0.1		20-45
3/4"	521	94.0	6.0	511.5	93.7	6.3		0.3		0-10
3/8"	543	98.0	2.0	529.5	97.0	3.0		1.0		0-5
Total	554			546.0						~

AGGREGATE ANALYSIS 1 1/2" - 3"

Sieve Size	1 Wt.Ret.	1 % Ret	1 % Pass	2 St.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
4"										100
3"										90-100
2"										20-55
1 1/2"										0-10
1"										0-5
Total										~

AGGREGATE ANALYSIS 3" - 6"

Sieve Size	1 Wt.Ret.	1 % Ret.	1 % Pass	2 Wt.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
7"										100
6"										90-100
[illegible]										20-55
[illegible]										0-15
[illegible]										0-5
Total										~

REMARKS: 4 MIN @ 1800 R.P.M - 2 MIN @ 2200 R.P.M

10° SLOPE, FAST FEED, GATE DOWN

Paul L. Baker

CONCRETE AGGREGATE TESTS
CANNON DAM PROJECT

MASSMAN CONSTRUCTION CO.
DACW-43-73-C-0134

SAND ANALYSIS

DATE 9-29-76

Sieve Size	1 Wt.Ret.	1 % Ret.	1 F.M.	2 Wt.Ret.	2 % Ret.	2 F.M.	3 Wt.Ret.	3 % Ret.	3 F.M.	% Spec.
#4			~			~			~	0-5
#8										5-20
#16										10-22
#30										18-32
#50										18-32
#100										10-25
Pan			~			~			~	5-10
Total										~

AGGREGATE ANALYSIS #4 - 3/4"

Sieve Size	1 Wt.Ret.	1 % Ret.	1 % Pass	2 Wt.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
1"										100
3/4"										90-100
3/8"										20-45
#4										0-5
Total										~

AGGREGATE ANALYSIS 3/4" - 1 1/2"

Sieve Size	1 Wt.Ret.	1 % Ret.	1 % Pass	2 Wt.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
										100
1 1/2"										90-100
1"										20-45
3/4"										0-10
3/8"										0-5
Total										~

AGGREGATE ANALYSIS 1 1/2" - 3"

1 #1 CURTIS — 2 #1 GIBSON — 3 DIFF.

Sieve Size	1 Wt.Ret.	1 % Ret	1 % Pass	2 St.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
4"	—	—	100	—	—	100		—		100
3"	65	9.9	90.1	65	10.0	90.0		0.1		90-100
2"	515	78.6	21.4	529.0	81.3	18.7		2.7		20-55
1 1/2"	626	95.6	4.4	621.5	95.5	4.5		0.1		0-10
1"	637	97.3	2.7	631.0	97.0	3.0		0.3		0-5
Total	655			650.5						~

AGGREGATE ANALYSIS 3" - 6"

Sieve Size	1 Wt.Ret.	1 % Ret.	1 % Pass	2 Wt.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
7"										100
6"										90-100
										20-55
										0-15
2"										0-5
Total										~

REMARKS: 5 MIN @ 2300 R.P.M, FAST FEED, 1/2 GATE
8° SLOPE

[signature]

CONCRETE AGGREGATE TESTS CANNON DAM PROJECT

MASSMAN CONSTRUCTION CO. DACW-43-73-C-0134

SAND ANALYSIS

DATE 9-30-76

	1			2			3			
Sieve Size	Wt.Ret.	% Ret.	F.M.	Wt.Ret.	% Ret.	F.M.	Wt.Ret.	% Ret.	F.M.	% Spec.
#4			~			~			~	0-5
#8										5-20
[illegible]										10-22
[illegible]										18-32
#50										18-32
#100										10-25
Pan			~			~			~	5-10
Total										~

AGGREGATE ANALYSIS #4 - 3/4"

	1			2			3			
Sieve Size	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
1"										100
3/4"										90-100
3/8"										20-45
#4										0-5
Total										~

AGGREGATE ANALYSIS 3/4" - 1 1/2"

	1			2			3			
Sieve Size	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
										100
1 1/2"										90-100
1"										20-45
3/4"										0-10
3/8"										0-5
Total										~

AGGREGATE ANALYSIS 1 1/2" - 3"

	1 #2 CURTIS			2 #2 GILSON			3 DIFF.			
Sieve Size	Wt.Ret.	% Ret	% Pass	St.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
4"	—	—	100	—	—	100		—		100
3"	67	7.3	92.7	79.0	8.7	91.3		1.4		90-100
2"	681	73.9	26.1	696.5	76.3	23.7		2.4		20-55
1 1/2"	867	94.0	6.0	858.0	94.0	6.0		0.0		0-10
1"	897	97.3	2.7	883.5	96.8	3.2		0.5		0-5
Total	922			912.5						~

AGGREGATE ANALYSIS 3" - 6"

	1			2			3			
Sieve Size	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
7"										100
6"										90-100
										20-55
										0-15
2"										0-5
Total										~

REMARKS: 5 MIN @ 2300 R.P.M, FAST FEED 1/2 GATE 8° SLOPE

Paul L. Parker

CONCRETE AGGREGATE TESTS
CANNON DAM PROJECT

MASSMAN CONSTRUCTION CO.
DACW-43-73-C-0134

SAND ANALYSIS

DATE 10-1-76

	1			2			3			
Sieve Size	Wt.Ret.	% Ret.	F.M.	Wt.Ret.	% Ret.	F.M.	Wt.Ret.	% Ret.	F.M.	% Spec.
#4										0-5
#8										5-20
[illegible]										10-22
#30										18-32
#50										18-32
#100										10-25
Pan										5-10
Total										

AGGREGATE ANALYSIS #4 - 3/4"

	1			2			3			
Sieve Size	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
1"										100
3/4"										90-100
3/8"										20-45
#4										0-5
Total										

AGGREGATE ANALYSIS 3/4" - 1 1/2"

	1			2			3			
Size	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
										100
1 1/2"										90-100
1"										20-45
3/4"										0-10
3/8"										0-5
Total										

AGGREGATE ANALYSIS 1 1/2" - 3"

	1 #3 CURTIS			2 #3 GIBSON			3 DIFF.			
Sieve Size	Wt.Ret.	% Ret	% Pass	St.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
4"	—	—	100	—	—	100		—		100
3"	35	3.8	96.2	39	4.2	95.8		0.4		90-100
2"	559	60.1	39.9	577.0	62.7	37.3		2.6		20-55
1 1/2"	868	93.3	6.7	861.0	93.6	6.4		0.3		0-10
1"	903	97.1	2.9	887.5	96.5	3.5		0.6		0-5
Total	930			920.0						

AGGREGATE ANALYSIS 3" - 6"

	1			2			3			
Sieve Size	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
7"										100
6"										90-100
										20-55
										0-15
2"										0-5
Total										

REMARKS: 5 MIN @ 2300 R.P.M., FAST FEED 1/2 GATE
8° SLOPE.

[illegible]

CONCRETE AGGREGATE TESTS — CANNON DAM PROJECT

MASSMAN CONSTRUCTION CO. — DACW-43-73-C-0134

SAND ANALYSIS

DATE 10-4-76

Sieve Size	1 Wt.Ret.	1 % Ret.	1 F.M.	2 Wt.Ret.	2 % Ret.	2 F.M.	3 Wt.Ret.	3 % Ret.	3 F.M.	% Spec.
#4			~			~			~	0-5
#8										5-20
[illegible]										10-22
[illegible]30										18-32
#50										18-32
#100										10-25
Pan			~			~			~	5-10
Total										~

AGGREGATE ANALYSIS #4 - 3/4"

Sieve Size	1 Wt.Ret.	1 % Ret.	1 % Pass	2 Wt.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
1"										100
3/4"										90-100
3/8"										20-45
#4										0-5
Total										~

AGGREGATE ANALYSIS 3/4" - 1 1/2"

S[illegible] Size	1 Wt.Ret.	1 % Ret.	1 % Pass	2 Wt.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
										100
1 1/2"										90-100
1"										20-45
3/4"										0-10
3/8"										0-5
Total										~

AGGREGATE ANALYSIS 1 1/2" - 3"

Sieve Size	1 Wt.Ret.	1 % Ret	1 % Pass	2 St.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
4"										100
3"										90-100
2"										20-55
1 1/2"										0-10
1"										0-5
Total										~

AGGREGATE ANALYSIS 3" - 6"

1 #1 Curtis — 2 #1 Gilson — 3 Diff.

Sie[illegible] Size	1 Wt.Ret.	1 % Ret.	1 % Pass	2 Wt.Ret.	2 % Ret.	2 % Pass	3 Wt.Ret.	3 % Ret.	3 % Pass	% Spec.
7"	—	—	100	—	—	100		—		100
6"	31	4.6	95.4	31	4.7	95.3		0.1		90-100
	315	47.0	53.0	320.5	48.4	51.6		1.4		20-55
	503	75.1	24.9	493.5	74.5	25.5		0.6		0-15
2"	566	84.5	15.5	561.0	84.7	15.3		0.2		0-5
Total	670			662.5						~

REMARKS: 5 MIN @ 2300 R.P.M, FAST FEED, GATE OPEN
8° SLOPE

Paul L. [illegible]

CONCRETE AGGREGATE TESTS — CANNON DAM PROJECT

MASSMAN CONSTRUCTION CO. — DACW-43-73-C-0134

SAND ANALYSIS

DATE 10-6-76

	1			2			3			
Sieve Size	Wt.Ret.	% Ret.	F.M.	Wt.Ret.	% Ret.	F.M.	Wt.Ret.	% Ret.	F.M.	% Spec.
#4			~			~			~	0-5
#8										5-20
[illegible]										10-22
[illegible]										18-32
#50										18-32
#100										10-25
Pan			~			~			~	5-10
Total										~

AGGREGATE ANALYSIS #4 - 3/4"

	1			2			3			
Sieve Size	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
1"										100
3/4"										90-100
3/8"										20-45
#4										0-5
Total										~

AGGREGATE ANALYSIS 3/4" - 1 1/2"

	1			2			3			
S. Size	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
										100
1 1/2"										90-100
1"										20-45
3/4"										0-10
3/8"										0-5
Total										~

AGGREGATE ANALYSIS 1 1/2" - 3"

	1			2			3			
Sieve Size	Wt.Ret.	% Ret	% Pass	St.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
4"										100
3"										90-100
2"										20-55
1 1/2"										0-10
1"										0-5
Total										~

AGGREGATE ANALYSIS 3" - 6"

	1 #2 CURTIS			2 #2 GILSON			3 DIFF.			
Sieve Size	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
7"	—	—	100	—	—	100		—		100
6"	29	2.0	98.0	28.5	1.9	98.1		0.1		90-100
	621	41.8	58.2	606.5	41.1	58.9		0.7		20-55
	1277	86.0	14.0	1243.5	84.2	15.8		1.8		0-15
2"	1433	96.5	3.5	1417.0	96.0	4.0		0.5		0-5
Total	1485			1476.5						~

REMARKS: 5 MIN @ 2300 R.P.M, EAST FEED, GATE OPEN
8° SLOPE

D... [illegible]

CONCRETE AGGREGATE TESTS
CANNON DAM PROJECT

MASSMAN CONSTRUCTION CO.
DACW-43-73-C-0134

SAND ANALYSIS

DATE 10-7-76

	1			2			3			
Sieve Size	Wt.Ret.	% Ret.	F.M.	Wt.Ret.	% Ret.	F.M.	Wt.Ret.	% Ret.	F.M.	% Spec.
#4										0-5
#8										5-20
#16										10-22
										16-32
#50										18-32
#100										10-25
Pan										5-10
Total										

AGGREGATE ANALYSIS #4 - 3/4"

	1			2			3			
Sieve Size	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
1"										100
3/4"										90-100
3/8"										20-45
#4										0-5
Total										

AGGREGATE ANALYSIS 3/4" - 1 1/2"

	1			2			3			
Size	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
										100
1 1/2"										90-100
1"										20-45
3/4"										0-10
3/8"										0-5
Total										

AGGREGATE ANALYSIS 1 1/2" - 3"

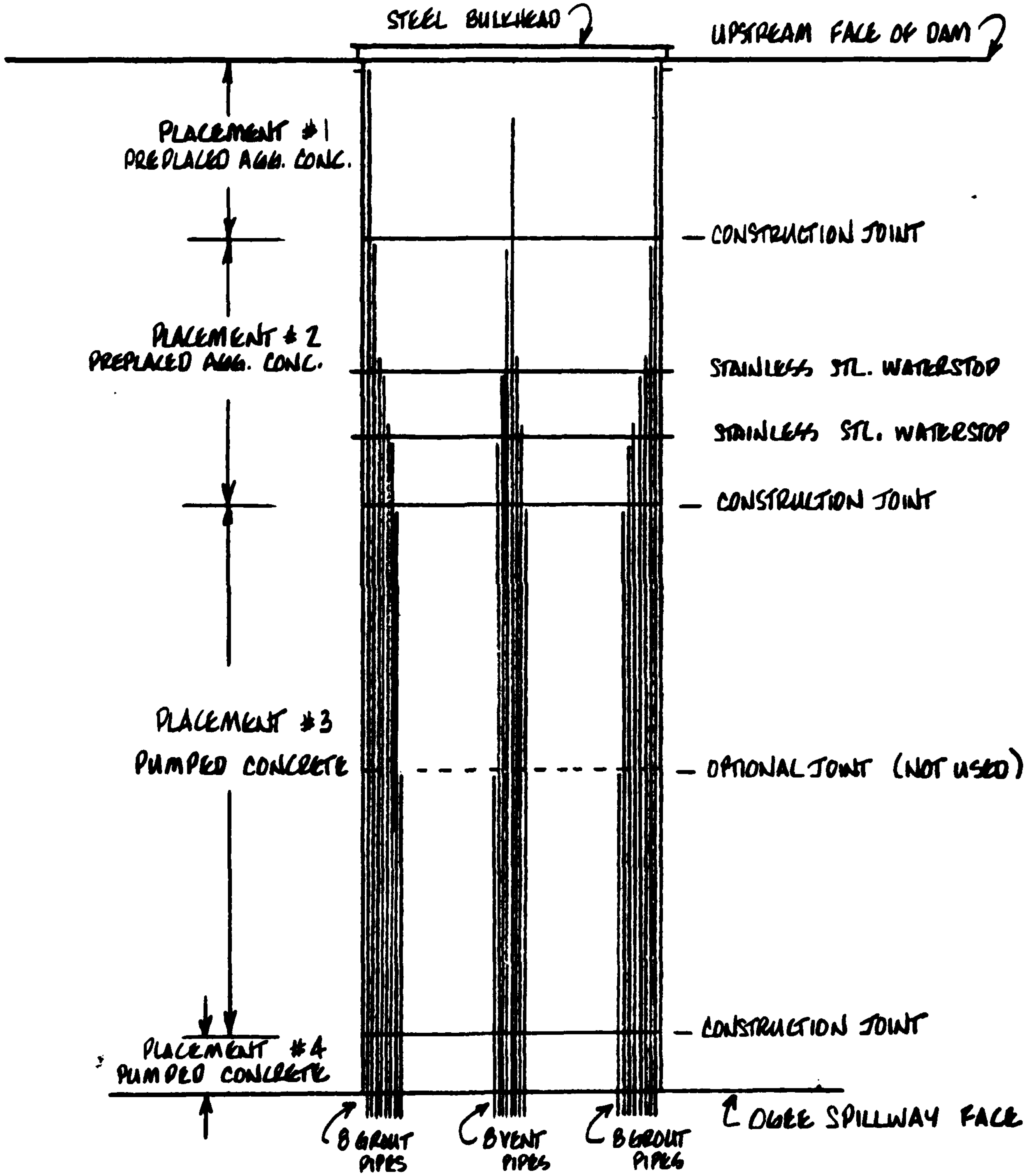
	1			2			3			
Sieve Size	Wt.Ret.	% Ret	% Pass	St.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
4"										100
3"										90-100
2"										20-55
1 1/2"										0-10
1"										0-5
Total										

AGGREGATE ANALYSIS 3" - 6"

	1 #3 CURTIS			2 #3 GILSON			3 DIFF.			
Sieve Size	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	Wt.Ret.	% Ret.	% Pass	% Spec.
7"	—	—	100	—	—	100		—		100
6"	44	2.8	97.2	44.5	2.9	97.1		0.1		90-100
	818	52.8	47.2	808.5	52.5	47.5		0.3		20-55
	1388	89.7	10.3	1360.0	88.3	11.7		1.4		0-15
2"	1499	96.8	3.2	1484.0	96.4	3.6		0.4		0-5
Total	1548			1540.0						

REMARKS: 5 MIN @ 2300 R.P.M , FAST FEED , GATE OPEN
8° SLOPE

Paul L. Baker



PIPING ARRANGEMENT FOR SLUICE SHRINKAGE GROUTING

TYPICAL PLAN VIEW

Concrete Plant before February 1975 fire.

Concrete Plant after fire.

Concrete placement in Monolith D-8.

END

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