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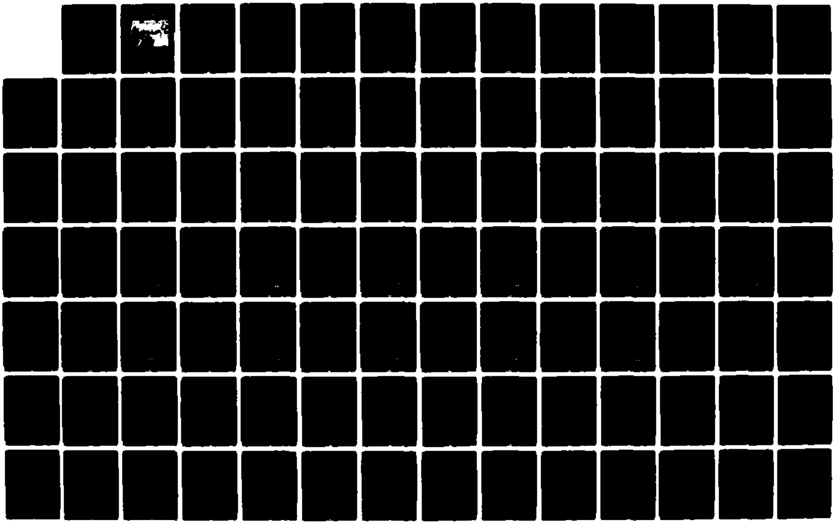

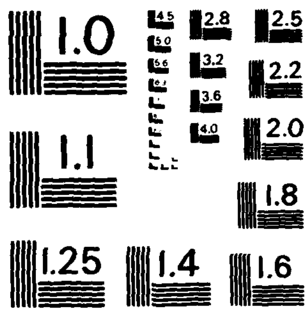

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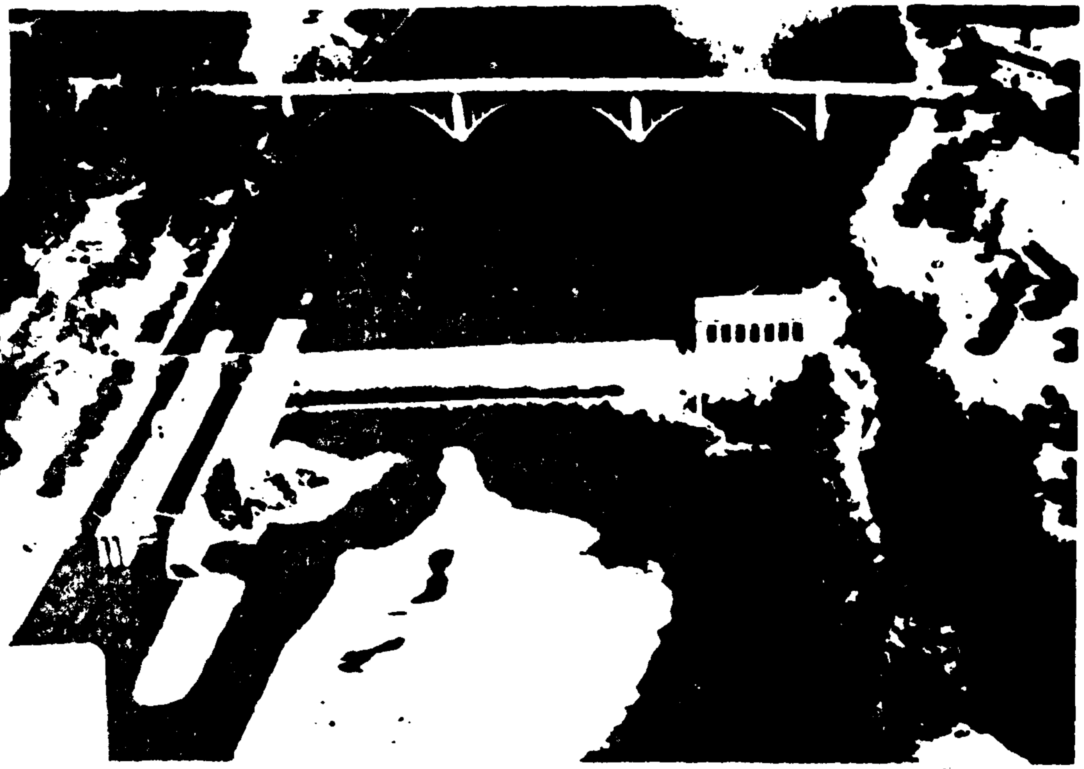
MISSISSIPPI RIVER

STUDY OF ALTERNATIVES FOR REHABILITATION OF LOCK AND DAM NO. 1

MINNEAPOLIS, MINNESOTA



VOLUME III

APPENDICES B-F

DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT, CORPS OF ENGINEERS
ST. PAUL, MINNESOTA

APRIL 1976

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>It is recommended that both the landward lock, the riverward lock and the dam at Lock & Dam no. 1, Minneapolis, Minnesota be completely rehabilitated. Based on studies completed to the date of this report, more detailed studies are required to firmly establish cost estimates, environmental effects, and the construction scheduling necessary to insure the work can be completed in the proposed two year construction period without delaying navigation.
This appendix covers the geologic data relevent to foundation condions at Lock &

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Dam #1, ~~Mississippi River~~; soil and rock parameters used in the structural analysis ; problems associated with the hydraulic filling, emptying, and venting system in the lock; a concrete evaluation study; and a study of the cofferdams.

DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT, CORPS OF ENGINEERS
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MISSISSIPPI RIVER

STUDY OF ALTERNATIVES FOR REHABILITATION OF LOCK AND DAM NO. 1

MINNEAPOLIS, MINNESOTA

APPENDIX B-1

GEOLOGY, FOUNDATIONS AND SOILS INVESTIGATIONS

GEOLOGY AND FOUNDATION EXPLORATION

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2. General Design Memorandum No. 1, Lock and Dam No. 1 Replacement.

3. Schwartz, 1936, Geology of the Minneapolis-St. Paul Area, Minnesota Geological Survey Bulletin - No. 27, pp. 21-127.

APPENDIX B-1

GEOLOGY AND FOUNDATION EXPLORATION

INTRODUCTION

Purpose and Scope

The purpose of this appendix is to present and evaluate all available geologic data relevant to the foundation conditions at Lock and Dam No. 1. These data result from an extensive drilling and testing program conducted during the summer of 1974, and include selected data from previous studies conducted by the Corps of Engineers.

Previous Investigations

The Corps of Engineers completed several exploratory drilling programs at Lock and Dam No. 1, prior to the current study. A total of 40 wash borings were completed from 1915 through 1950. In addition, a total of 13 AX and NX core holes were drilled in 1959 and 1964. The location of USCE boreholes is shown in plate B-1-1. The Corps of Engineers correlated the stratigraphy derived from these borings, where possible, to a core hole (test hole F) drilled at lower St. Anthony's Falls approximately five miles upstream. Data from borings drilled before 1959 could not be correlated since they were essentially overburden borings and no core was recovered.

Present Investigation

Forty four boreholes were completed at Lock and Dam No. 1 during the summer of 1974. The holes were located to establish the detailed geologic framework of the project site. Representative sampling of all materials at the site, including soil, backfill, concrete and rock, was also an integral part of the program. Particular emphasis was given to the concrete of the lock structures and the foundation materials beneath each structural element. Borehole locations are shown in Plate B-1-1.

Compilation of data from the drilling program provides comprehensive stratigraphic coverage of the site. Data from these borings have also been correlated with geologic data from earlier USCE studies.

All materials were logged in the field as drilling progressed and selected samples were sent to the laboratory for testing. Water levels were recorded during drilling and piezometers were installed in 15 borings (see plate B-1-15). Upon completion of the program, the piezometers were used for a study of seepage beneath the lock structures

GEOLOGIC SETTING

The Minneapolis-St. Paul metropolitan area, shown in Figure 1, is in the Central Lowland Physiographic Province of North America. Regional topography is characterized by a flat to gently rolling ground surface,

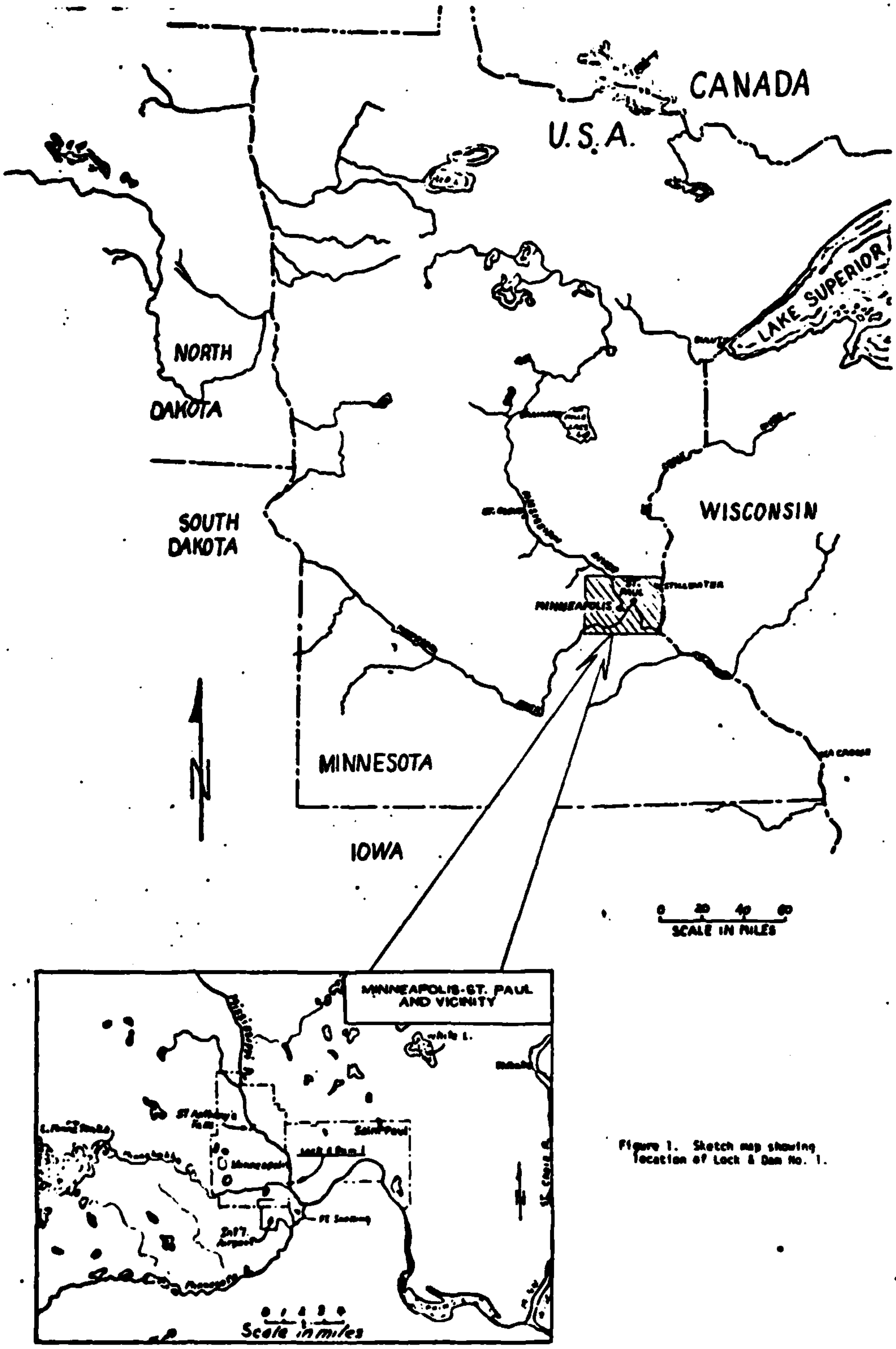


Figure 1. Sketch map showing location of Lock & Dam No. 1.

frequently dissected by entrenched, post-glacial, river valleys. The valleys of the Mississippi, Minnesota and St. Croix Rivers are particularly prominent. Considerable shifting of these river channels in the past is evidenced by numerous abandoned valleys.

The regional structure is dominated by a broad, shallow and almost circular basin. The basin is slightly elongated in the north east, south west direction. The lowest point of the basin, where bedding is essentially horizontal, is located along the Mississippi River, several miles upstream of Lock and Dam No. 1. Artesian water from five geologic formations in this basin is currently being utilized as a major water resource.

The Twin Cities area has been subjected to most of the major glacial advances recognized in North America, and is mantled with varying thicknesses of glacial drift. Outcrops of bedrock are generally restricted to road cuts and deeper river valleys. The formations frequently exposed are the Ordovician age Galena Shale (Decorah member), Platteville Limestone, St. Peter Sandstone and Shakopee Dolomite. Deep wells reveal sequences of dolomite and sandstone underlying these units. Precambrian granite was penetrated at a depth of 2,135 feet in a well in Minneapolis. The stratigraphy of the Twin Cities Metropolitan area is summarized below.

Table 1

Regional Stratigraphy (after Schwartz, 1936)

Period	Formation	Apparent range in thickness	Average thickness
Recent	River alluvium	0-150	-
Pleistocene	Glacial drift, etc.	0-400	100
Ordovician	Galena Shale	0-20	Variable thickness-top eroded
	Decorah Shale	0-75	75
	Platteville Limestone	25-35	30
	Glenwood Shale	2-7	5
	St. Peter Sandstone	145-165	158
	Shakopee Dolomite	35-60	45
	New Richmond Sandstone	0-15	11
	Oneota Dolomite	70-90	80
Cambrian	Jordan Sandstone	80-105	90
	St. Lawrence Formation	160-200	180
	Franconia Sandstone	45-80	65
	Dresbach Formation	125-200	155
Cambrian or	Hinckley Sandstone	limited data-one well	220
Keweenawan	Red Clastic Series	limited data-one well	1,012
Precambrian	Basalt flows; granites		Unknown

No faulting has been described in the immediate vicinity of Lock and Dam No. 1. Two faults have been located in Hastings, Minnesota, 20 miles southeast of the site. Identification of fault zones and similar structures in this area is difficult due to the lack of bedrock outcrops.

SITE GEOLOGY

Topography

Topography at Lock and Dam No. 1 is dominated by a post-glacial valley of the Mississippi River about 1000 feet wide and bounded by steep bluffs. The maximum relief of the valley is approximately 140 feet, south of the dam, and about 100 feet, north of the dam. The bluffs rise about 80 feet above the top of the lock structures. The Mississippi River channel occupies most of the valley floor.

The valley is subject to seasonal flooding. An island about 850 feet long and 400 feet wide immediately southeast of the locks, and small land areas adjacent to the southeastern side of the locks normally do not become inundated. The remainder of the valley floor is relatively flat with scattered hummocky topography. The river bottom immediately upstream of the dam is quite irregular and hummocky, possibly a result of excavation during construction.

Topography above the bluffs is generally flat. Immediately west of the locks, an abandoned channel of the Mississippi River, shown in Figure 2, is represented by a dry waterfall and a shallow, crescent shaped trough. Minnehaha Creek, which enters the Mississippi River about one-half mile downstream, flows southeast through a deep entrenched valley.

Stratigraphy

Rock Units: Plate B-1-2 presents the detailed stratigraphy of the site. The stratigraphic column is primarily a compilation of data from all borings; therefore, specific features may not occur in any individual borehole.

This stratigraphic column includes the entire Platteville Formation, the entire St. Peter Formation and a small portion of the Shakopee Formation. Lithologic and physical characteristics of each formation are described in descending order below:

The Platteville Formation is the well-exposed cap rock for the east and west valley walls. It is a hard, bluish grey, fine to medium grained, fossiliferous limestone locally containing clay seams and interbeds of

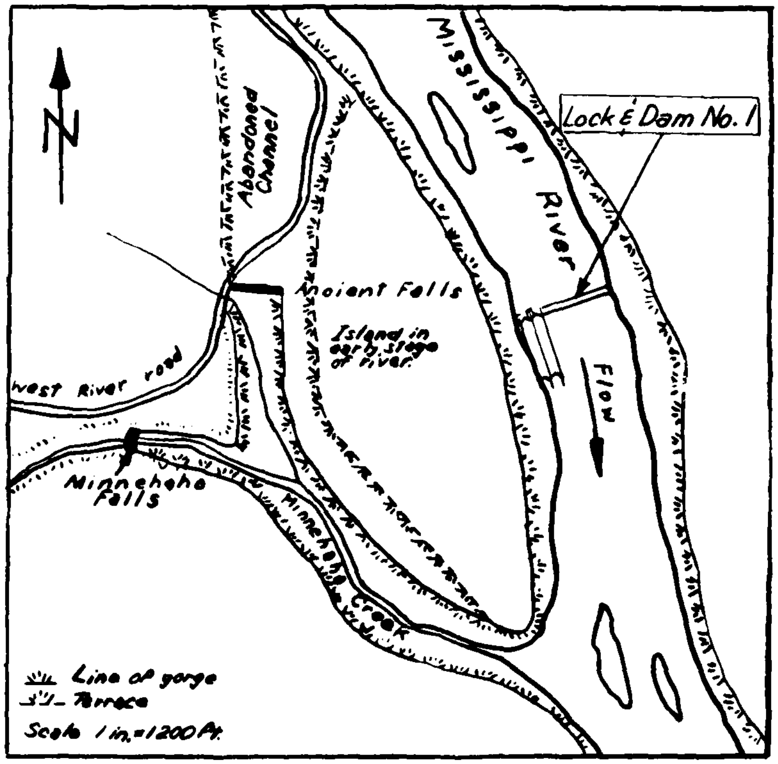


Figure 2. Sketch map of abandoned falls near Lock & Dam No. 1 (from Schwartz, 1936).

limey siltstone. The limestone locally contains small, calcite coated solution cavities. The Glenwood Member at the base of the Platteville is a five foot section of moderately fissile, silty shale, which grades into the underlying St. Peter Formation.

Individual lithologic units within the St. Peter Formation have been assigned letter designations to correspond to previous studies (see plate B-1-2). Units A, B, C and D were established by the Corps of Engineers as a result of the 1964 investigation. Units E, F, G and H were arbitrarily assigned to deeper units as a result of the present study. USCE Unit A has been further subdivided into A and A^1 in this report.

The upper, approximately 105 feet of the St. Peter Formation forms two distinct lithologic zones. This interval, designated as Units A and A^1, is exposed in both valley walls and is the foundation rock for many of the lock structures. The rock is a light grey, poorly cemented, very friable, very fine to fine grained, quartzitic sandstone. Core of Unit A is generally recovered as loose sand, while Unit A^1 is recovered as friable sandstone. Sieve analyses of Unit A indicate the sand is bimodal, containing fractions of both very fine grained and fine grained sand. The very fine grained sand is generally angular to subangular, while the fine grained sand is rounded and frosted. Microscopic and chemical analyses of this part of the St. Peter formation indicate that Unit A and A' contains 97 to 98 per cent quartz.

The lower part of the St. Peter formation, Units B through H, consists of alternating beds of very fine grained, olive gray, moderately cemented silty sandstone, and bluish gray, medium grained, moderately cemented sandstone. The medium grained sands are rounded and frosted. There are two units in the lower St. Peter which do not conform to this sequence. Unit C is a five to six foot thick interbed of dark gray siltstone, containing several thin seams of soft material, ranging in composition from silty clay to clayey silt. The most prominent of these seams is found in the upper foot of Unit C and ranges in thickness from 0.15 to 0.3 feet. One seam 0.5 feet thick was found in hole 75-6. Unit D is a 12 foot thick interval of fine grained, friable sandstone more similar to Unit A^1 in the upper St. Peter Formation than to the generally siltier lower St. Peter.

Two feet of the Oneata Member of the Shakopee Formation were cored in borehole 74-32. This rock is a hard, crystalline, medium grained dolomitic limestone, which is sandy at the contact with the St. Peter. Fractures and solution cavities in the rock are coated with soft, bluish material. Regional stratigraphy indicates that the Shakopee Formation extends to an elevation of approximately 556 feet.

River Alluvium: Significant areas of the project facilities are underlain by river alluvium consisting of poorly graded sands and gravels with very small fractions of non-plastic fines. Most of the coarser sand, and all of the gravel is hard, and well rounded, and consists primarily of quartz, granite and basalt. Finer sands tend to be mostly subangular quartz. Limestone debris, ranging in size from coarse gravel to boulders and slabs several feet in size occur frequently in the alluvium. Well graded quartzitic sand and varying amounts of slightly plastic fines also occur locally.

Structure

Bedding: Primary bedding at the site is essentially horizontal, as seen in bedrock exposures and in the cores. Units A and A^1 exhibit local crossbedding that ranges in dip from 15 to 25 degrees. Local deviations in dip, as indicated in some geologic cross sections, apparently are the result of drill-stem measurement errors.

Jointing: Three well-developed joint sets are exposed in the Platteville Formation at the site.

Two sets of vertical joints, striking approximately parallel and perpendicular to the face of the bluff, occur at spacings of from one to three feet. These joints are continuous through the vertical exposure of the limestone but do not appear to extend through the Glenwood member of the formation. The joints are generally planar, although minor irregularities are found locally, and most joint surfaces are clean. Minor manganese staining was found on a vertical joint surface encountered in Borehole 74-2. The third joint system in the Platteville occurs as essentially horizontal bedding-plane joints spaced from several inches to about nine feet apart. Cores from borehole 74-2 suggest that the spacing of bedding joints is much closer between elevations 803.5 to 787.0 than between 787.0 to 768.0. Bedding joint surfaces from elevation 803.5 to 796.0, are generally stained with manganese and are locally coated with calcite. Joint surfaces are clean below elevation 796.0.

Four manganese-stained joints, dipping from five to fifteen degrees, were observed in the cored interval 17 through 27 feet, in boring 74-2. This joint set was not identified elsewhere.

Two joint sets, exposed in the St. Peter Formation, were also encountered during drilling. A prominent set of horizontal bedding-plane joints, spaced from several inches to as much as ten feet, occurs in Units A and A^1 and, to a far lesser extent, in the lower part of the formation. These joints appear to be iron stained only in the lower part of Unit A and approximately the upper half of Unit A^1. The friable nature of much of Units A and A^1 made positive identification of jointing and/or staining difficult. A vertical to near vertical joint set is exposed at the site and was locally encountered in the cores. These joints strike from N70° to N80°E (USCE design memorandum, 1964), and are spaced from three to ten feet apart. The joints tend to be irregular, and occur sporadically in cores over considerable vertical intervals. An iron stained vertical joint was found in Unit A^1 in borehole 74-16 at a depth of 45 to 50 feet. Joint surfaces occurring below this were clean.

A poorly developed joint set, paralleling the face of the bluff may represent a surficial stress relief feature. These joints are a primary cause of scaling which occurs from the exposed sandstone in the bluff.

Faulting: No faults or shear zones were encountered during the investigation.

Ground Water Conditions

Analysis of available data indicates that the Mississippi River Valley acts as a discharge area for the St. Peter Sandstone. The water level in Borehole 74-2, located on top of the bluff south of the dam centerline, is higher than the lower pool elevation, suggesting movement toward the river.

The St. Peter Sandstone, in the vicinity of Lock and Dam No. 1, can be divided into three arbitrary hydrologic units. The Upper Sandstone (Units A A^1 and B), and Lower Sandstone (Units D through H) are considered to be aquifers, separated by an aquitard (Unit C). The bedrock surface slopes to the east beneath the lock structures so that unconsolidated alluvium is in contact with the Upper Sandstone and the aquitard. In the western portion of the site and beneath the bluff, flow in the Lower Sandstone, primarily Unit D, is confined by the aquitard, producing an artesian effect. Water in the Upper Sandstone is naturally unconfined. When beneath the concrete slabs and walls of the project structures uplift pressures are created.

BORING AND SAMPLING PROGRAM

Various drilling techniques were utilized during this investigation to optimize the acquisition of data. These techniques are described below.

Drilling Equipment

Three drill rigs were used during the program. A truck mounted Mobile Drill Model 50B drilled nine of the 14 borings through the lock walls and ten of the eleven borings in the lock chambers. An Acker "hillbilly" skid rig, initially utilized to drill inside the lock chambers, was replaced by the Mobile Drill when the Acker was found too light to complete the borings efficiently. HQ, four inch and six inch borings, as well as several NX borings were drilled with a truck mounted Joy Model 12B skid rig.

Five different core sizes were recovered during the field program. These included:

1) Conventional NX using a standard five foot longyear double tube barrel and internal discharge diamond bits.
2) HQ wireline using the longyear "Q-Series" wireline system and either bottom-discharge "step" or internal discharge. The "step" bits were used in the friable portions of the St. Peter sandstone while the internal discharge bits were used in the relatively soft, sticky siltstone interbed (Unit C) to reduce blocking.
3) Four inch using a five foot longyear double tube core barrel and bottom or internal discharge bits.
4) Six inch using a five foot longyear double tube core barrel and bottom discharge bits.
5) HQ triple tube wireline using the longyear "HQ-3 Series" triple tube barrel. The triple tube is a modification of the standard HQ wireline system designed to improve core recovery and quality in friable or broken rock. The modification involves the addition of a thin wall third tube, chrome-plated and split lengthwise, to the standard wireline barrel. Minor changes are also made in the wireline recovery mechanism and in the core catcher assembly. A bottom discharge step bit, cutting a slightly smaller core than standard HQ wireline, was used. This system is designed to reduce the friction between the core and the core barrel, thereby reducing mechanical damage, and to preserve the in-situ condition of the rock during drilling, and when the core is removed from the barrel. The split tube is pumped out of the standard barrel using couplings provided with the conversion kit. The core can be logged before being placed in the core box, by separating the two halves of the tube.

Table 2 summarizes the dimensions of each core size and the approximate footages drilled during the field investigation.

Table 2. Dimensions and approximate total footages* of five core systems utilized.

Equipment designation	Core diameter (inches)	Hole diameter (inches)	Total linear feet*
NX Conventional	2.125	2.98	928
HQ Wireline	2.50	3.782	623
4 inch	3.97	5.495	72
6 inch	5.97	7.75	122
HQ-3 triple tube wireline	?	2.98	333

*Footages are approximate and reflect drilling in rock or concrete only.

Drilling Mud

It was necessary to use bentonite drilling mud in coring the St. Peter Formation. This tended to improve core recovery, helped keep the boreholes clean, and reduced fluid losses. Significant core losses, especially in Units A, A^I and D could be traced to insufficiently thick mud or to the use of clear water. Mud was unnecessary when coring limestone or less friable units of the St. Peter Formation.

Equipment Performance

The performance of the various core systems used during the field program varied widely and is important with regards to possible further investigations. Table 3 summarizes equipment performance in each of six materials with regards to core recovery and overall core quality. The six materials include concrete and Units A and A^I, B, C, D, and E through G at the St. Peter Formation (See Plate B-1-2 for unit descriptions). The Platteville limestone is not included as only HQ wireline core was used and both core recovery and quality were excellent. In the St. Peter Formation, Units A and A^I, and Units E through G, have been considered together because of generally similar equipment performance. Too little of the Shakopee Formation was cored to provide meaningful data.

Table 3. Performance of core drilling systems with regards to core recovery and overall core quality.

MATERIAL DRILLED

Core Size	Concrete	St. Peter Formation Units A, A[1]	B	C	D	E - G	Core Quality
NX	99%	24%	84%	90%	53%	80%	1
HQ	99%	48%	97.5%	100%	70%	100%	2
4 inch	100%	91%	100%	100%	100%	71.4%	3
6 inch	100%	90%	14%	100%	14.5%	98.5%	4
HQ-3	Not used	94%	98.5%	97.5%	84.5%	100%	5

Core Quality*

1. Good to excellent in concrete with locally considerable mechanical breakage. Very poor in friable sandstone with severe core erosion, and loss of soft seams. Poor in siltstone with mechanical breakage and loss of soft seams. Poor to good in harder rock units. Mechanical breakage locally high, some core erosion.

2. Excellent in concrete and harder rock units. Poor in friable sandstone. Soft sand seams usually lost and core eroded. Soft seams in siltstone generally disturbed while removing from barrel.

3. Good to excellent in concrete and hard rock. Poor to fair in friable sandstone and siltstone, mostly due to difficulty in removing core from barrel. Soft sand seams generally lost.

4. Good to excellent in concrete and hard rock. Poor to good in friable sandstone and siltstone. Soft sand seams often lost, core frequently severely disturbed during removal from barrel, soft silt seams usually lost by spinning.

5. Good to excellent in all materials. Soft sand and silt seams usually recovered intact. Core removed from barrel undisturbed. Not used in concrete.

<u>Soil Sampling</u>: Standard penetration tests in unconsolidated material were made every five feet with a two-inch diameter split spoon. Bentonite mud and casing were used to keep the holes clean where required.

Sample quality and recovery with the split spoon varied considerably with the type of material being tested. Recovery of fine-to-medium grained backfill and alluvial sand generally ranged from nine inches to one foot per test interval. Sample quality in these areas was excellent. Recovery in coarser sand and gravel ranged from zero to one foot.

Continuous Shelby Tube samples were taken in boring 74-9 behind the land wall. The tubes were driven with the 140 pound hammer after attempts to advance the hole hydraulically proved unsuccessful. An average recovery of 1.3 feet for each 2.5 foot sample interval was achieved. Several of the samples were disturbed, and a number of tubes severely damaged by gravel and cobbles in the backfill.

Piezometer Installations

Piezometer tubes were installed in fifteen boreholes in the lock walls, behind the land wall and behind the upper and lower guidewalls. Periodic water level measurements were taken in the piezometers throughout the program. The piezometers were used for an extensive study of seepage and permeability beneath the locks after drilling was complete.

Construction of the piezometers was designed to measure water levels from a) the upper sandstone, b) the lower sandstone (Unit C being considered an aquiclude), or c) a composite of the two. A double piezometer was installed in borehole 74-8, to measure upper and lower zones simultaneously. Subsequent readings indicate that the cement plug isolating the two tubes may be ineffective. Details of piezometer construction are shown in plate B-1-15.

Core Photography

Two sets of 8 x 10 color prints of the core recovered during the study were produced using special procedures requested by the Corps of Engineers. A tripod mounted 35 mm Canon twin lense reflex camera with a cable shutter release, and a F 1.1, 50 mm lense was used for core photography. Light intensities were measured from incident light with a Luna pro light meter. Kodachrome II color negative film (ASA 85, DIN 25) was used: all film was from the same emulsion group. The film was kept refrigerated before use.

Each core box was placed on an inclined frame, positioned so that the direct sunlight fell a number of degrees off of normal, thereby reducing the glare in the subject area. All cores except those of concrete, were photographed dry, also to reduce glare. Photos were taken between the hours of 9:00 AM to 11:15 AM and 1:30 PM to 4:30 PM on cloudless days to avoid direct noonday sun. A white background was used, where possible, to provide greater contrast.

A Kodak Neutral Test Card (grey card at 18% reflectance), a Kodak color control scale and a linear scale were included in each picture. Most photographs were taken at F-11 with a shutter speed of 1/250 of a second. The higher F-stop settings help to increase the depth of focus. Care was taken to protect unexposed and exposed film from extreme heat and from x-rays (at airport security posts) since both tend to distort colors.

Custom procedures were used in developing and printing each final picture, thus allowing them to be corrected for color to the actual conditions under which they were taken.

If custom developing is not desired, corrective stops, which can be taken during the photography itself, are inclusion of test cards in the picture and the use of incident, rather than reflected, light measurements. The test cards provide a means of estimating color distortions in the resultant print, and incident light automatically provides a great deal of standardization, especially if there is a large degree of light contrast in the subject. These procedural changes can be achieved with very little additional cost or time.

The most important advantage of the custom developing and printing portion of the program is the use of an accurate densiometer to match and correct for exposure and color. The densionometer provides the means by which corrections can be made in the final product. Other advantages of custom printing result simply from the greatly increased care taken in all stages of the program.

LABORATORY TEST PROGRAM

Concrete

Unconfined compressive strengths were determined on NX, HQ, and six inch concrete cores from each of the lock walls. The tests indicate concrete strengths ranging from 2595 psi to 9420 psi and an average strength of 5251 psi. The lowest strengths occurred in the river wall constructed in 1906. The wide range in values is largely indicative of the diameter of the various samples. Relatively large aggregate (up to three inches in diameter), made selection of representative NX and HQ cores difficult. Consistent results were recorded for six inch cores from boring 74-34 (6065 psi to 6535 psi for three samples) in which the effect of large aggregate was minimized. Microscopic examinations of concrete thin sections were also made. Reference is made to Appendix F for further test results on concrete.

Rock

Triaxial shear tests were performed on cores from the St. Peter sandstone Units A, A', and B. Consolidated-drained tests were made on Unit A and A' cores, and unconsolidated-undrained tests on cores of Unit B. HQ, four inch, and six inch cores were used for these tests.

Constant head permeability tests were performed on two, 4 inch cores of Unit A' from borehole 74-36A. Gradation tests were made on five samples from borehole 74-36A.

Reference is made to Appendix B-2 for further test results and analyses of the laboratory testing program.

Soil

Consolidated-drained triaxial shear, gradation and density tests were made on composite Shelby Tube samples of backfill from boring 74-9. Moisture content, wet density, and dry density were measured on individual samples from this boring, and specific gravity on four selected samples. Testing of river alluvium is largely restricted to standard penetration tests in the field as mentioned previously. Reference is made to Appendix B-2 for results and analyses of laboratory testing of soil materials.

ENGINEERING GEOLOGY

Physical Properties

The physical properties of all lithologic units at the site are described below. Reference should be made to plate B-1-2 for more detailed lithologic descriptions.

Rock: The limestone section of the Platteville Formation, as drilled in borehole 74-2, is hard, strong, and unweathered below an elevation of 803.5 feet. Three soft, clay seams in the upper part of the formation and sporadic siltstone interbeds are potential planes of horizontal weakness. Most of the rock contains very small, circular cavities caused by the exsolution of fossil debris. These are usually partially filled with calcite. No significantly large voids were encountered.

The <u>Glenwood Member</u> of the Platteville is a moderately soft silty shale which showed only moderate fissility in borehole 74-2. Locally severe horizontal and vertical slaking occurs in this shale. The slaking property and weakness of the shale might also effect the stability of the bluff at the site.

The physical properties of each unit of the <u>St. Peter Formation</u> are described below.

<u>Unit A</u>, partially exposed at the site, is a fine grained, very friable sandstone generally recovered as loose sand. Core recovery in this unit was very poor in all boreholes except those drilled with the HQ triple tube. Thin zones were locally recovered as poorly cemented sandstone, easily broken in the fingers, rather than loose sand. The lower portions of Unit A locally contain diffused iron staining and iron stained-bedding and vertical joints. The sandstone contains scattered, fresh pyrite crystals.

Unit A sandstone generally disintegrated to loose sand when immersed in water. Precipitation causes quantities of the exposed sandstone to lose cohesion and be washed down as loose sand. The effect of this property on the stability of the bluff is discussed on page 25.

Utility tunnels have been excavated in the metropolitan area, in this portion of the St. Peter Formation. Routes for these tunnels are usually restricted to areas in which a significant section of the Platteville limestone remains to protect the sandstone from water. When protected in this way the rock requires no special support treatment. The sandstone disintegrates rapidly, if left unprotected.

The upper portion of the St. Peter Formation has been found to be an adequate foundation material in the Twin Cities area, although several excavations have revealed natural sandstone caves requiring special treatment.

<u>Unit A^1</u> is similar, lithologically, to Unit A, however cores were generally recovered as poorly cemented sandstone instead of loose sand. The contact between Units A and A^1 is irregular and not easily defined.

Unit A^1 sandstone is extremely friable and can generally be broken manually with light pressure, or in the fingers. Unlike Unit A, it generally does not disintegrate when immersed in water. Minor amounts of water are retained by the rock after immersion. The unit locally contains thin zones of loose sand which can be easily cut with a knife. The majority of the sandstone can be scratched with the fingernail.

Results of consolidated drained triaxial shear tests of Unit A[1] and of Gradation tests are provided in Appendix B-2.

Unit B is a very fine grained, moderately well cemented silty sandstone containing scattered friable zones and very thin zones of loose sand. The core can be scratched by the fingernail with difficulty, and is scratched easily with a knife. NX and HQ core of Unit B can generally be broken with strong manual pressure while larger size core usually required moderate hammer blows. The core breaks along thin, irregular siltstone partings.

During drilling operations, frequent loss of drilling fluid was experienced in Unit B. Cores of those intervals having loss of drilling fluid generally were recovered as broken discs. A one-inch cavity was encountered in Unit B by borehole 74-34 on the downstream end of the center wall.

Unit C is dark gray siltstone, locally containing thin, irregular lenses of fine grained, white sandstone. NX cores of Unit C can be broken manually with light to moderate pressure. Breaks invariably occur along irregular bedding planes, although the rock is only moderately fissile. The core can be scratched by the fingernail with moderate ease.

Unit C exhibits a minor slaking characteristic when immersed in water. This is seen as slaking of silt size material from the core surface. During drying, the core slakes severely, breaking into very closely spaced discs and wedges along the bedding.

Thin, variably plastic silt-clay seams occur in Unit C. The most prominent of these are found in the upper foot of the unit. This silt-clay seam ranges from 0.15 to 0.3 feet thick in most borings. A thinner seam, between 0.05 and 0.15 feet thick, occurs in several boreholes near the contact with the underlying Unit D. A number of very thin seams, generally less than 0.02 feet thick were found locally in the middle of the unit. The seams contain varying quantities of silt which govern their toughness. Seams containing relatively significant amounts of silt can be cut with some difficulty with a knife, while more clayey zones can be gouged with the finger. These silt-clay seams constitute potential planes of horizontal weakness at the site.

Vertical stress release fractures were located by several borings in Unit C. Thin seams of uncemented sand or, possibly, voids were found in the transition zone to the urderlying Unit D in boreholes 74-82 and 74-86A upstream of the dam. The voids were located by a sharp drop in drill stem of about 0.1 feet and the immediate loss of all drilling fluids. No other cavities were encountered in the unit during the drilling program.

Unit D is a friable, poorly to moderately cemented sandstone locally containing thin zones recovered as loose sand. Traces of unweathered pyrite occur in some cores. Cores of Unit D can be scratched with the fingernail and broken with moderate to strong manual pressure. There are no apparent planes of weakness in the rock. Unit D can be reduced to loose sand with light hammer blows. Immersion in water has no deleterious effect on the rock.

Unit E is a very fine grained, moderately well cemented silty sandstone, similar to Unit B. It locally contains zones of moderately friable, coarse grained sandstone. Siltstone partings are generally more continuous across the core than in Unit B. Unit E can be scratched with difficulty by a knife and NX core can be broken with light to moderate manual pressure. Larger size cores can be broken with very strong manual pressure or light hammer blows. The core generally breaks along siltstone partings. Immersion in water has no deleterious effect.

Unit F is a moderately cemented sandstone locally containing slightly friable zones. It is easily scratched with a knife and NX core can be broken with moderate to strong manual pressure. The unit has no apparent planes of weakness. Traces of unweathered pyrite occur in some cores. Immersion in water has no deleterious effect. Minor amounts of water are retained by the rock after immersion.

Unit G is a very fine grain sandstone similar in most respects to Units B and E. Continuous siltstone partings generally about 1/8 to 1/4 inch thick were recovered in several cores. Siltstone partings in Units B and E were usually much thinner and less continuous. Local concentrations of coarse grained, slightly friable sandstone were found in several cores. Unit G can be scratched with difficulty by a knife and six inch core can be broken along the siltstone partings with strong hammer blows. Immersion in water has no deleterious effect.

Unit H, the basal unit of the St. Peter Formation, is a medium grained, moderately cemented quartitic sandstone, generally similar to Unit F. The presence of interbeds and irregular concentrations of silty sandstone is common.

Unit H can be easily scratched with a knife and broken with strong manual pressure. Finer grained zones, particularly near the lower contact, are friable and locally can be broken between the fingers. Immersion in water has no deleterous effects on the rock and minor amounts of water are retained.

The Shakopee Formation was encountered in only one borehole. Prominent features of the Shakopee as seen in boring 74-32 include the presence of solution cavities, and the soft, bluish silt, or possibly clay that coated core breaks and cavities. The core recovered in 74-32 breaks with moderate to strong hammer blows and showed no preferred planes along which breaks occurred. The rock can be scratched with a knife. Immersion in water had no deleterious effect on the limestone.

Alluvium: The contact between alluvium and bedrock beneath the lock structures is an irregular line along the eastern edge of the center wall as shown in plate B-1-1. The thickness of alluvium beneath the river wall ranges from 20 to 26 feet as shown on plate B-1-7. The alluvium lies on Unit A beneath the western side of the river lock and on Unit B beneath the eastern side of the lock and the river wall. Drilling performed in 1929 by the Corps of Engineers suggests the possibility of a number of shallow scour channels in the bedrock surface beneath the river wall. Insufficient borings were drilled to confirm this in the present study. Beneath the dam, Unit B has been almost completely eroded and the top of rock is located between one and three feet above the contact with Unit C. The thickness of alluvium beneath the dam ranges from 32 to 38 feet. Moderately well graded sands, (SP), and sandy gravels (GP), comprised the bulk of the alluvium. Standard penetration blow counts of 25 to 30 were most common for these materials. Blow counts less than 25 were fairly common, while counts between 30 and 50 were less common. Refusal (100 blows per six inch interval) was infrequent, and was probably due to the presence of limestone cobbles.

Within the alluvium concentrations of limestone cobbles, boulders and slabs are frequently found. Slabs up to 5 feet long and 2 feet thick are found at the surface near boring 74-32 and two to three foot boulders were commonly encountered at depth. Cobbles and smaller boulders were generally weathered to the point where they could be broken manually. It was difficult to advance casing through larger slabs. These materials were identified largely from drilling characteristics and wash samples.

In addition to these materials, silty sand (SM) and clayey sand (SC) were locally encountered. Two logs were identified by chips in the return mud and by drilling characteristics in borehole 74-86A upstream of the dam.

Backfill: Almost all backfill material sampled at the site consisted of fine grained, well graded, subrounded sand, closely resembling unconsolidated Unit A of the St. Peter Formation. Varying amounts of rounded gravel, concrete blocks, wood and miscellaneous steel, were also recovered. Borings 74-24 and 74-4 behind the upper and lower guidewalls respectively, drilled through concrete rip rap of the wall structures.

Standard penetration blow counts in the backfill material generally ranged from 20 to 25. Blow counts from 1 to 10 were common in intervals of borehole 74-8 behind the land wall. More than 50 blows were recorded occasionally, due to the presence of gravel or cobbles.

Sieve analyses of Shelby Tube samples from boring 74-9 behind the land wall show that 66 to 89 percent passed the No. 40 sieve (U.S. Standard Sieve numbers, approximately 0.42 mm mesh) with a trace of non-plastic fines. Nine percent of the sample, as sandstone fragments and gravel, was retained on the No. 40 sieve. 35 to 40 percent fell between 0.15 mm and 0.25 mm diameter. The specific gravity of the backfill ranged from 2.62 to 2.66.

Consolidated triaxial shear tests of combined Shelby Tube samples from 74-9 resulted in the following:

Table 4. Triaxial shear tests of backfill: borehole 74-9

Sample Depth (feet)	Dry Density (pcf)	Angle of Internal Friction Ø	Maximum Dry Density (pcf)	Minimum Dry Density (pcf)	% Relative Density
5 27.5	109.0	43.0	118.3	93.8	70.0
29 - 49	108.5	38.3	119.2	96.5	58.0
5 - 27.5	116.2	47.5	118.3	93.8	92.0
29 - 49	117.0	39.9	119.2	96.5	92.0

Reference is made to Appendix B-2 for details of these test results.

Fluid Losses: The return of drilling fluids, either bentonite mud or clear water, were at least partially lost in a majority of holes during drilling. The depth at which circulation was lost, and the circumstances surrounding loss and recovery of drilling fluids provides important secondary information about possible seepage paths in the foundation rock of the lock and dam structure. Table 5 is a summary of the circulation history of holes in which at least partial loss occurred. The percentage of fluid loss when less than total is an estimate made in the field during drilling.

Some of the circulation losses recorded during drilling are a result of insufficiently thick drilling fluid. In several cases circulation was regained by switching from clear water to mud, or by thickening the mud mixture after a loss. In other holes circulation may have been regained if this had been done. In these instances no significant problem is suggested.

Table 5

SUMMARY OF CIRCULATION HISTORY
OF BOREHOLES IN WHICH PARTIAL OR TOTAL
LOSS OF DRILLING FLUID WAS RECORDED

Hole No.	Fluid loss (%)	Depth of fluid loss(ft)	Depth where fluid return was regained (ft.)	Fluid used	Material in which loss occurred	Remarks
74-2	100	158.0	-	mud/ water	Unit B	Barrel clogging with sand suspended in mud. Switched to clear water at 158'.
74-6	100	3.4	-	mud	Below concrete	Poor casing seal in concrete apron.
74-8	100	22.6	22.6	water/ mud	Unit A'	Switched to mud and regained return.
	100	55.0	79.4	mud	Unit B	Reseated casing and regained return.
	100	84.3	-	mud	Unit C	Could not regain. Hole stopped by caving.
74-14	100	72.5	-	mud	Unit B	Could not regain.
74-16	100	74.0	76.5	mud	Unit B	Regained partially at 76.5.
	70	76.5	-	mud		
74-24	100	78.0	-	mud	Unit B	
74-32	20	54.4	-	mud	Unit D	Poor casing seal possible. Artesian flow in hole.
74-34	100	75.0	-	water	Unit B	Void at 75.0' Did not use mud.
74-35	100	76.0	77.0	mud	Unit B	Regained at 77.0'.
74-36	100	77.0	99.0	mud	Unit B	Return was regained by pumping very thick mud after hole was completed.
74-36A	10	79.5	-	mud	Unit B	Loss not significant.
74-38	100	28.0	31.0	water	Concrete	Regained at 31.0.
	100	38.0	-	water/ mud	Concrete	Could not regain. Some of loss is through construction joint in concrete.
74-39	100	65.0	-	water	Unit A'	Did not use mud.
74-41	100	50.5	51.5	mud	Unit B	Regained partially at 51.5'.
	45	51.5	-	mud	Unit B	Could not regain.
74-42	100	82.0	-	water	Unit C	Poor casing seal possible.
74-44	100	72.5	-	water/ mud	Alluvium	Could not regain when drove casing. Poor casing seal.
74-46A	40	63.0	96.0	water	Alluvium	Poor casing seal possible.
	100	96.0	-	water	Unit D	Did not use mud.
74-48	100	56.0	58.0	water	Alluvium	Regained partially at 58'.
	20	58.0	76.6	water	Alluvium	
	100	76.6	-	water	Alluvium	Alluvium /Unit B contact. Did not use mud.

Table 5 (Continued)

SUMMARY OF CIRCULATION HISTORY
OF BOREHOLES IN WHICH PARTIAL OR TOTAL
LOSS OF DRILLING FLUID WAS RECORDED

Hole No.	Fluid loss (%)	Depth of fluid loss (ft)	Depth where fluid return was regained (ft.)	Fluid used	Material in which loss occurred	Remarks
74-50	100	75.0	-	water	Unit B	Did not use mud.
74-52	100	61.2	-	water/ mud	Unit A'	Could not regain with mud.
74-60 to 74-80						Holes experienced intermittent fluid return throughout due to poor casing seals in the concrete beneath the water.
74-82	100	63.5	-	water/ mud	Unit D	Void at 63.5. Could not regain with mud.
74-84	30	40.5	-	water/ mud	Unit B	Poor casing seal probable.
74-86A	100	69.3	-	mud	Unit D	Void at 69.3' Could not regain.
74-88	40	36.3	-	mud/ water	Alluvium	Poor casing seal probable. Could not regain.

The difficulty of attaining positive casing seals in rock account for a significant number of fluid losses, especially in those holes drilled into concrete through water. Positive seals in these holes would require cementing the casing into the concrete, a process made impractical by constantly changing river conditions and by the limited time allowed for drilling inside of and adjacent to the locks. Fluid losses attributable to these causes also do not suggest any significant seepage problem.

Most of the fluid losses which cannot be explained by either of the conditions, occur in the lower portion of Unit B of the St. Peter Formation. In several cases (i.e. 74-24) fluid loss was accompanied by severe breakage at the core along bedding planes, suggesting at least some degree of seepage in parts of this unit. The inconsistency of fluid losses in this unit from hole to hole is an indication that seepage does not have significant horizontal continuity, and, therefore is not generally considered a major problem.

Losses in other rock units are inconsistent and appear to represent only localized seepage conditions. Fluid losses in the highly permeable alluvium occurred as expected and were generally recovered when casing was driven. Those cases in which circulation was not regained are probably a result of poor casing seals due to the very coarse, blocky nature at the alluvium.

The instantaneous losses which occurred at the same depths as thin voids and/or seams of loose sand in holes 74-34, 82, and 86A, might suggest more significant seepage paths and warrant further treatment.

Foundation Evaluation

The foundation for structural elements of Lock and Dam No. 1 varies with the elevation of the foundation and with the position of each structure in the river valley. The foundation conditions for each structure, as determined by this study, are described below. Reference is made to plates B-1-3 through B-1-8 for the lock structures, B-1-9 and B-1-10 for the upper lower guidewalls, respectively, and B-1-11 through B-1-13 for the dam structure. Summary logs of all boreholes are provided on plates B-1-16 through B-1-19.

Several factors effect the foundation of the entire lock and dam, or of large portions of it. First, the soft clayey seams, located in Unit C throughout the area, may represent horizontal planes of weakness along which sliding could occur. Second, the friable Unit A^1 sandstone, found under the west portions of the locks, is easily subject to erosion by water, creating a potential for channeling and scour. Third, the alluvium beneath the east portion of the locks and beneath the dam might present difficulties due to its relatively high permeability, and to the difficulty with which steel sheet and wood friction piles are driven through it. Seepage in these areas may be relatively high and difficult to control.

Land Wall: The land wall is founded on Unit A^1 of the St. Peter Formation. Boreholes 74-50, -52 and -58 were drilled through the wall and, although core recovery was very poor immediately below the concrete, no deterioration of the sandstone was indicated by wash samples or drilling characteristics. The contact between concrete and sandstone was recovered in 74-50 and -52. Two thin seams recovered as loose sand were identified in Unit A^1 in boring 74-50, and correlation with boreholes behind the land wall suggests that similar features were washed away by the drilling. The top of Unit B is approximately 13 feet below the concrete. The siltstone interbed (Unit C) is inferred to be approximately 22 feet below the concrete although not penetrated by the holes drilled in the wall. Projection of borings 74-8 and 74-16 behind the land wall suggests the occurrence of a silt-clay seam as much as 0.3 feet thick in the upper foot of Unit C.

Movement of the land wall has been of negligible proportions and the foundation of the wall appears satisfactory in all respects. Seepage tests indicate hydraulic communication between the locks and the area behind the land wall.

Land Lock: Boreholes 74-60 through 74-68 and USCE borings 59-1M through 59-4M show the land lock to be founded on Unit A^1. While core recovery in these borings was very poor, no indication of deterioration of the sandstone was given by either recovered samples or drilling characteristics.

Seepage tests indicate hydraulic communication between the foundation of the land lock and that of the land wall. Dye injection studies indicate some degree of piping between borehole 74-70 in the river lock, at the downstream end of the center wall, and the east edge of the land lock. Communication under the land lock could occur in open channels beneath the concrete floor of the lock or, more likely, through drainage pipes installed beneath the lock. This communication appears to have more significance to the integrity of the center wall and river lock than to the land lock. The foundation of the land lock is considered satisfactory.

Center Wall: The center wall is founded primarily on Unit A^1, although two to three feet of alluvial sand (SP) is found beneath the upstream and, possibly, the downstream ends. In addition, borehole 74-36A encountered 1.7 feet of fine grained loose sand with rounded gravel beneath the concrete. This is inferred to be backfill material rather than alluvium.

Boreholes 74-34 and 74-41 recovered numerous thin zones of loose sand in Unit A^1, which is inferred to continue beneath the entire structure. A 0.1 foot open cavity was found in boring 74-34 in the center of Unit B approximately 17 feet below the concrete. A soft silt-clay seam, up to 0.2' thick, located in the upper foot of Unit C was recovered in boreholes 74-34 and 74-41. A 0.2 foot thick silt-clay seam was found at the bottom of Unit C in borehole 74-41, and several thinner seams were located throughout the Unit in boreholes 74-34, -36, and -41.

Only minor movement has been recorded in monoliths of the center wall, however, stability calculations performed by the USCE (1966) on Monolith No. 18 (the lower miter gate monolith) indicated a low factor of safety against movement. Seepage and dye tests performed in this area suggest communication beneath Monolith No. 18, through either the void located in borehole 74-34, or open channels beneath the concrete, (which were not indicated by the drilling). The foundation of the remainder of the wall appears satisfactory, although seepage tests indicate some hydraulic communication between the land and river locks and the center wall.

River Lock: The river lock rests primarily on unconsolidated river alluvium. Unit A^1 underlies the concrete in several areas. The contact is somewhat irregular along the western side of the lock. The top of rock drops off quickly to the east and as much as 16.5 feet of alluvium were found beneath the eastern side of the lock. The alluvium beneath the western three-quarters of the lock lies on Unit A[1]. The alluvium lies on Unit B beneath its eastern quarter.

Most of the alluvial material in this area grades from sandy gravel (GP) to gravelly sand (SP), although 5.5 feet of clayey sand (SC) and seven feet of silty sand (SM) were found in boreholes 74-70 and 74-76, respectively. The alluvium on the west side of the lock tends to be finer grained and better grained than that on the east side.

A small, highly irregular cavity was penetrated immediately below the top of rock in borehole 74-70, approximately seven feet below the concrete. The top six inches of rock in this hole were slightly harder than normally encountered and core breaks were coated with a soft, light brown material. This may be evidence of foundation grouting, last performed by the USCE in 1959.

Tests indicate at least some hydraulic communication between the foundations of the river lock and those of the center and river walls. The majority of the communication probably reflects the inherently high permeability of the alluvium. Injection of dye in borehole 74-70, inside the lock, suggests communication from the southwest corner of the lock to areas beneath the center wall and along the east side of the land lock, as well as below the miter gates of the land lock. Communication could occur through open channels beneath the concrete, or through voids as penetrated in 74-34. In this case, vertical communication must also be assumed in order to explain dye "shows" below the miter gate. This communication could be provided by joints in the St. Peter Formation, although no suggestion of water movement in these joints was revealed during drilling.

River Wall: The river wall is founded on friction piles driven into unconsolidated river alluvium. This material consists primarily of sand and gravel (SP to GP) with varying concentrations of limestone slabs and boulders. A thin interval of silty sand (SM) was encountered in borehole 74-46A. The top of rock beneath the river wall is at an approximate elevation of 655.

USCE Design Memo No. 1 (1966) states that Monolith R-16 of the river wall moved landward approximately 1-1/2 inches and settled approximately 1/2 inch between the years 1938 and 1949. At that time the Corps of Engineers attributed the movement to deterioration of the old concrete miter gate sill, upon which the monolith was built. This condition was stabilized by injection grouting and no major wall movement has been recorded since. Minor movement in other portions of the wall has occurred, however.

The current stability of the wall suggests that the alluvial material provided an adequate foundation, however, the underlying friction piles could not be driven uniformly to bedrock due to the presence of limestone slabs. Settling of these piles is a possibility. Difficulty in driving sheet steel piling through material containing large boulders indicates a possibility of water movement to the east into the river channel. Seepage tests performed during this study suggest at least some communication between the river lock and piezometers set in the wall.

Upper Guidewall: Boreholes 74-22, -24, and -28 show that the upper guidewall is founded on Unit A of the St. Peter Formation. The upper ten feet of sandstone in borehole 74-28 is soft and was recovered as sand. Iron stained bedding joints and horizontal zones of diffuse iron staining were observed in both Units A and A^1. There is a 0.2 foot soft clay seam at the top of Unit C, 56 feet below the concrete of the guidewall.

Minor movement of the upper guidewall was recorded between 1934 and 1963, particularly of Monolith No. 3 which tilted landward to a maximum vertical deviation of 0.45 feet. Minor movement might result from deterioration of the sandstone beneath the structures or, possibly, to the effects of repeated impacts from barge traffic. Iron staining along bedding planes suggests water movement beneath the guidewall, although it is not known whether such seepage is occurring at the present time.

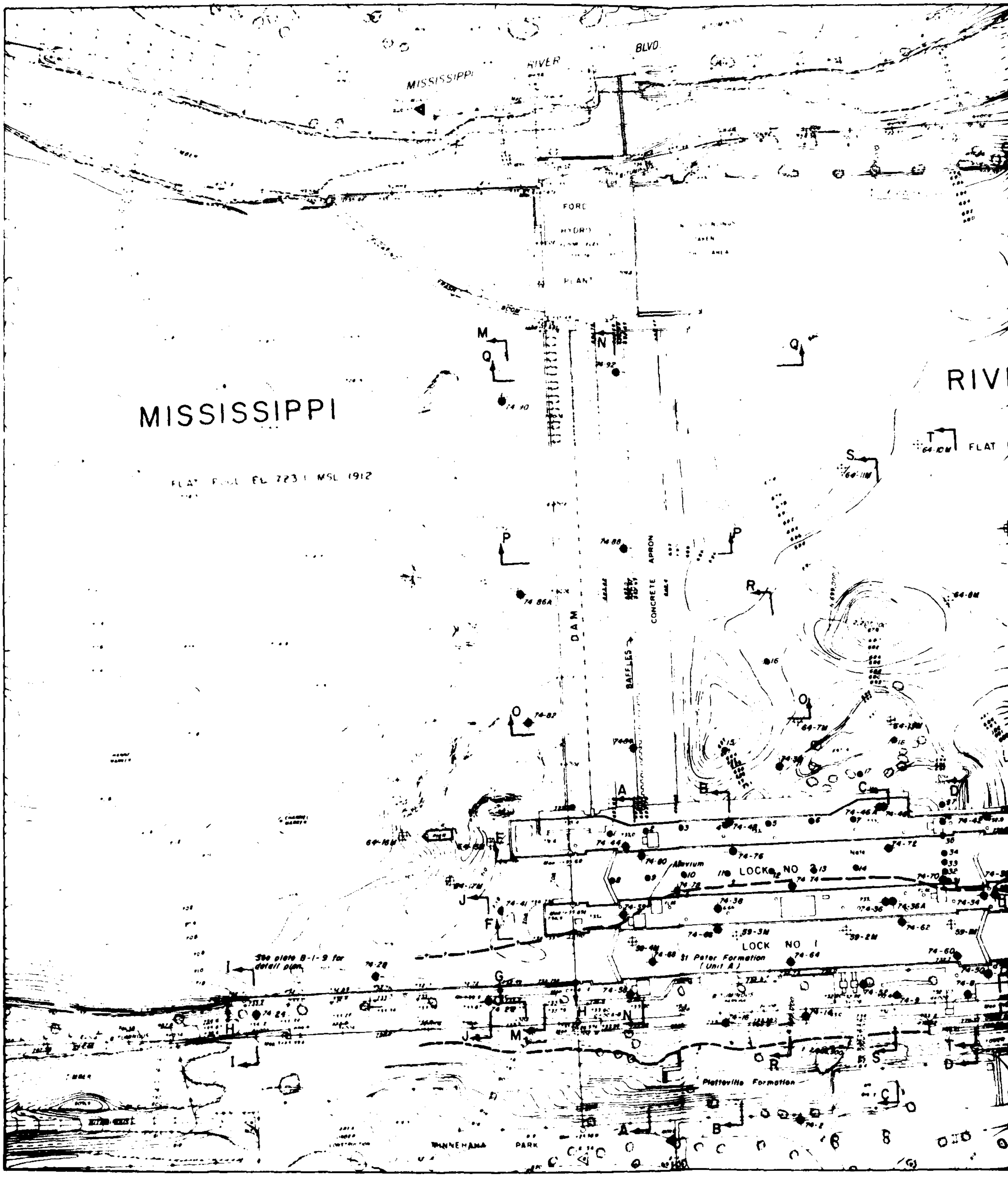
Lower Guidewall: The lower guidewall rests on Unit A^1 Sandstone as determined from borings 74-4 and -6. Numerous thin zones of uncemented, or poorly cemented sand were located in this material. Soft silt-clay seams, 0.3 and 0.5 feet thick, were found in the upper foot of Unit C in borehole 74-4 and 74-6 respectively. The top of Unit C is approximately 25 feet below the structure.

Major riverward movement of the lower guidewall monoliths occurred in the period from 1931 to 1935. Stability was attained by removal of backfill from behind the wall. Periodic removal of backfill since then has maintained stability to date. The USCE determined that the factor contributing most to this movement was differential pressure acting on the wall from behind, rather than failure of the foundation itself. The foundation is considered adequate for support of the wall. Dye "shows" in boring 74-66, however, indicate hydraulic seepage between the foundation area of the guidewall and those of the river lock and center wall. The path of communication may be through scour channels beneath the concrete apron or, possibly, through a continuation of the open cavity penetrated in borehole 74-34 in Unit B. Piping in these areas should be considered as a possibility, since the grout pumped into 74-6 obviously was not effective. This problem is discussed further in Appendix B-3 Sections 3, 9, and 13.

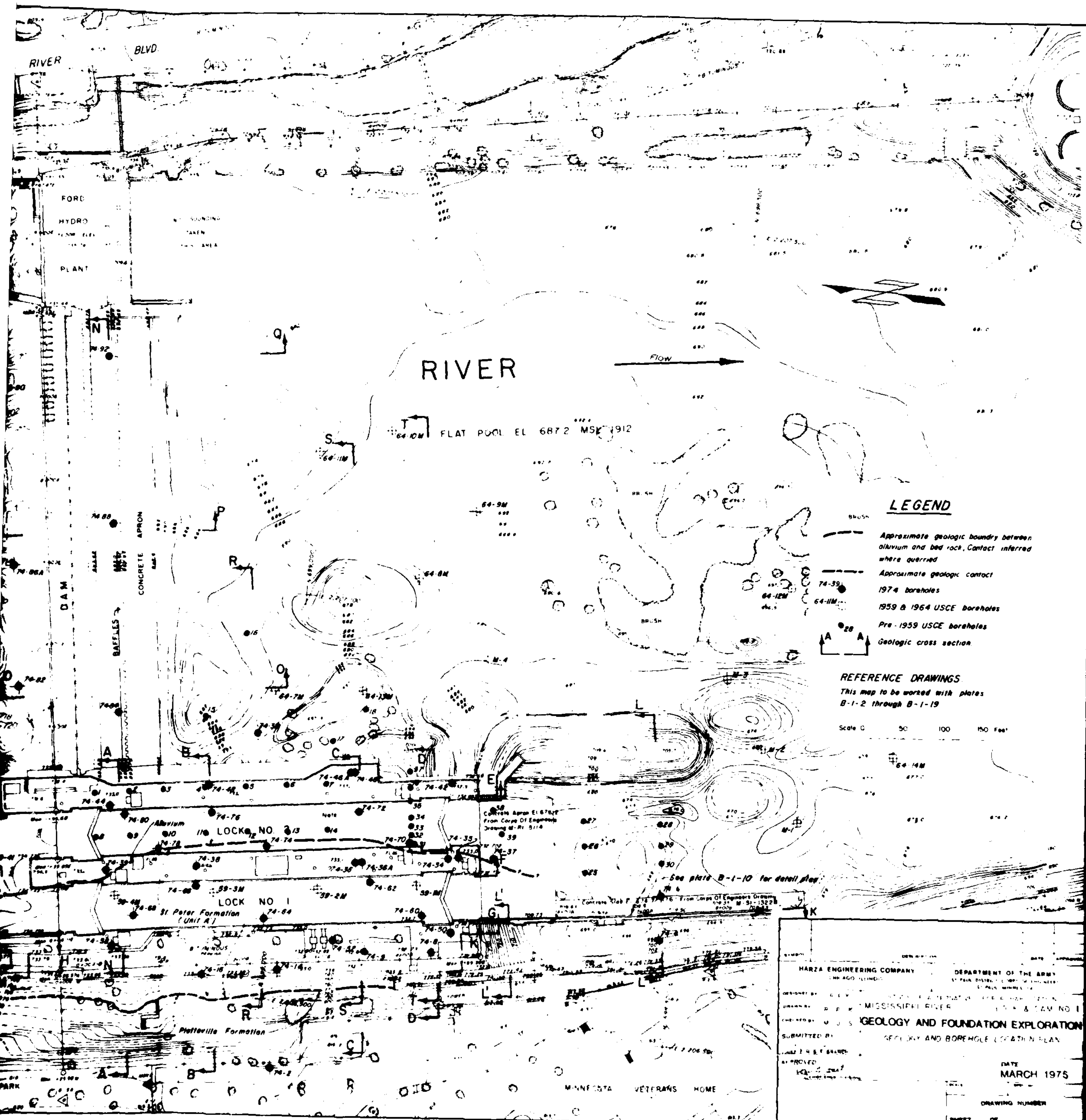
Dam: The dam structure is founded on river alluvium, consisting of sand, gravel and limestone slabs. The top of rock is in Unit B of the St. Peter Formation approximately 36 feet below the dam. The contact with Unit C is one to three feet below the top of rock. Soft silt-clay seams up to 0.3 feet thick were commonly encountered in the upper foot of Unit C. One inch open cavities were penetrated immediately below Unit C in boreholes 74-82 and -86A, upstream of the dam. Voids were not encountered in other holes drilled at the dam.

The lack of any appreciable settlement along the dam since it was built in 1917, indicates that the foundation of the structure is satisfactory. Seepage beneath the dam is occurring, however, it does not seem to be a significant problem and appears to be adequately controlled.

Shape of Buried Cut-slope Surfaces: Corps of Engineers structural drawings of existing conditions indicate a 3:1 slope cut into rock behind the land wall and both guidewalls. The cut-slope surface is buried by backfill material up to an elevation of 732.7 behind the land wall and the upper guidewall, and to approximately 695 behind the lower guidewall. Boreholes 74-4, -8, -14, -16, -22 and -24 suggest a buried 3:1 slope with minor variations. These variations are easily explained by irregularities in excavation of the rock.



MISSISSIPPI RIVER BLVD.
FORD HYDRO PLANT
MISSISSIPPI
RIVE
FLAT POOL EL 723.1 MSL 1912
DAM
CONCRETE APRON
BAFFLES
Alluvium
LOCK NO 2
LOCK NO 1
St Peter Formation (Unit A)
Platteville Formation
See plate B-1-9 for detail plan.
MINNEHAHA PARK

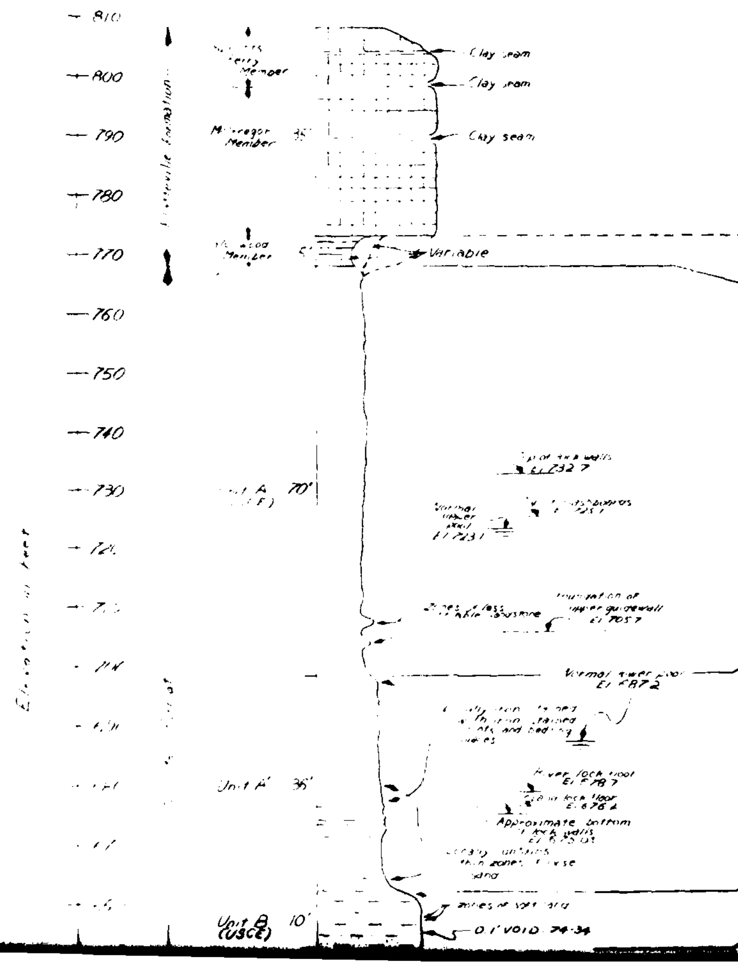


RIVER
BLVD.
FORD
HYDRO
PLANT
RIVER
Flow
FLAT POOL EL 687.2 MSL 1912
DAM
CONCRETE APRON
BAFFLES
LOCK NO. 2
LOCK NO. 1
St Peter Formation (Unit A)
Platteville Formation
Alluvium
See plate B-1-10 for detail plan
MINNESOTA VETERANS HOME
PARK
LEGEND
Approximate geologic boundry between alluvium and bed rock, Contact inferred where querried
Approximate geologic contact
74-39 1974 boreholes
64-11M 1959 & 1964 USCE boreholes
28 Pre-1959 USCE boreholes
A A Geologic cross section
REFERENCE DRAWINGS
This map to be worked with plates B-1-2 through B-1-19
Scale 0 50 100 150 Feet
HARZA ENGINEERING COMPANY
DEPARTMENT OF THE ARMY
MISSISSIPPI RIVER
GEOLOGY AND FOUNDATION EXPLORATION
GEOLOGY AND BOREHOLE LOCATION PLAN
SUBMITTED BY
APPROVED
DATE
MARCH 1975
DRAWING NUMBER
SHEET OF

PLATE B-1-1

Lock & Dam No. 1

Stratigraphic Column at Site



Platteville Formation, LIMESTONE, gray (N3-N5) to bluish gray (5B5/1), fossiliferous, generally fine grained, locally coarse grained, bedding is massive, horizontal, locally contains siltstone partings and interbeds and clay seams to 0.5' thick, locally contains small, calcite coated vugs, moderately to little fractured along bedding, occasional vertical joints, fracture surfaces are locally stained with Manganese and coated with Calcite, can be scratched with knife, breaks along bedding with light to moderate hammer blow, clay seam can be gouged with fingers, unweathered. Sharp contact with Glenwood Member.

Glenwood Member, SILTY SHALE, gray (N3-N5), limy, contains closely-spaced limestone interbeds less than 0.01' thick, thinly bedded, bedding is horizontal, core surface is pitted, locally contains small vugs, locally sandy, moderately fissile, no apparent jointing, can be scratched with fingernail, breaks with light manual pressure, severe horizontal and vertical slaking when dry, unweathered. Grades over 1' to St. Peter Formation.

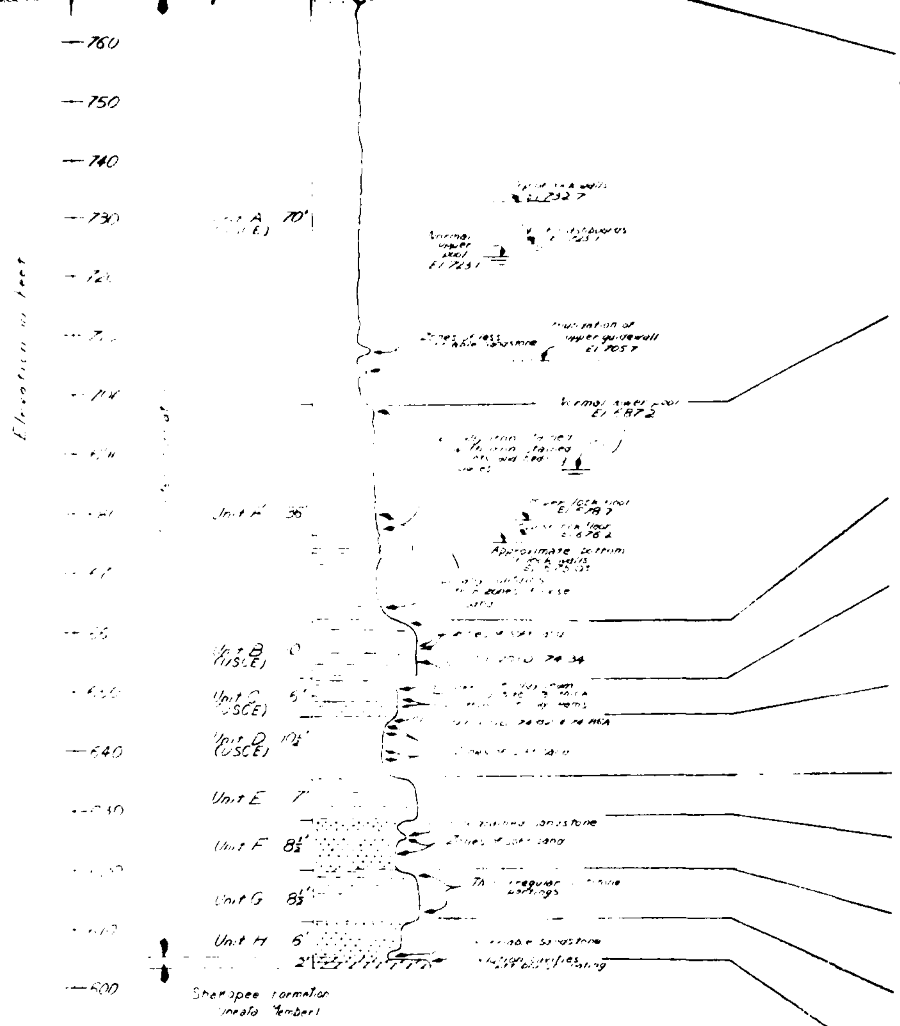
St. Peter Formation,
Unit A, SANDSTONE, light gray (N7) to yellowish gray (5Y8/4), friable, very poorly cemented with silica, generally recovered as loose sand, fine grained, well sorted, subrounded to subangular, frosted, core recovery very poor, locally contains thin zones recovered as friable sandstone rather than loose sand, bedding is massive, horizontal, locally cross-bedded, locally iron-stained, no apparent jointing, can be scratched with fingernail, can be cut with knife, can be crushed in fingers, generally loses integrity when immersed in water, unweathered. Grades to Unit A'.

Unit A', SANDSTONE, light gray (N7), friable, poorly cemented with silica, locally contains thin zones recovered as loose sand, lithologically similar to Unit A, locally contains thin interbeds of silty sandstone and small, silty concentrations, core surface locally pitted, moderately to little fractured along bedding, locally contains vertical joints, joint surfaces locally iron stained, most surfaces clean, can be scratched with fingernail, breaks with light to moderate manual pressure, locally can be crushed in fingers, generally retains integrity when immersed in water, unweathered. Grades over 1' to Unit B.

Unit B, SILTY SANDSTONE, light olive gray (5Y6/1), moderately cemented with silica, slightly friable, contains thin, irregular to wavy, discontinuous siltstone partings, sand is very fine grained, well sorted, thinly bedded, bedding is horizontal, locally contains closely spaced bedding fractures, fracture surfaces clean, can be scratched with knife, breaks along bedding with strong manual pressure or light hammer blow, retains integrity in water, unweathered. Grades over 1' to Unit C.

Unit C, SILTSTONE, dark gray (N3), contains thin, closely spaced, irregular sandstone interbeds, sticky when wet, slight organic odor when wet, locally contains soft, variably plastic silt-clay seams to 0.35' thick, thickly laminated, bedding is horizontal, no apparent jointing, can scratch with fingernail, breaks with light to moderate manual pressure along bedding, silt-clay seams can be gouged with finger, minor surface slaking when immersed in water, severe slaking along bedding when dry, unweathered; grades over 1.5' to Unit D.

silog; can be scratched with fingernail; breaks with light manual pressure; severe horizontal and vertical slaking when dry; unweathered. Grades over 1' to St. Peter Formation



St. Peter Formation.

Unit A, SANDSTONE, light gray (N7) to yellowish gray (5Y8/4), friable, very poorly cemented with silica, generally recovered as loose sand, fine grained, well sorted, subrounded to subangular, frosted, core recovery very poor. Locally contains thin zones recovered as friable sandstone rather than loose sand, bedding is massive, horizontal, locally cross-bedded, locally iron-stained, no apparent jointing, can be scratched with fingernail, can be cut with knife, can be crushed in fingers, generally loses integrity when immersed in water, unweathered. Grades to unit A'

Unit A', SANDSTONE, light gray (N7), friable, poorly cemented with silica, locally contains thin zones recovered as loose sand, [illegible] similar to unit A, locally contains thin interbeds of silty sandstone and small silty concentrations, core surface [illegible] pitted, moderately to little fractured along bedding, [illegible] contains vertical joints, joint surfaces locally iron-stained, most surfaces clean, can be scratched with fingernail, breaks with light to moderate manual pressure, locally can be crushed in fingers, generally retains integrity when immersed in water, unweathered. Grades over 3' to unit B

Unit B, SILTY SANDSTONE, light olive gray (5Y6/1), moderately cemented with silica, slightly friable, contains thin, irregular to wavy, discontinuous siltstone partings, sand is very fine grained, well sorted, thinly bedded, bedding is horizontal, locally contains closely spaced bedding fractures, fracture surfaces clean, can be scratched with knife, breaks along bedding with strong manual pressure or light hammer blow, retains integrity in water, unweathered. Grades over 1' to unit C

Unit C, SILTSTONE, dark gray (N3), contains thin, closely spaced, irregular sandstone interbeds, sticky when wet, slight organic odor when wet, locally contains soft, variably plastic silt clay seams to 0.35' thick, thinly laminated, bedding is horizontal, no apparent jointing, can scratch with fingernail, breaks with light to moderate manual pressure along bedding, silt-clay seams can be gouged with finger, minor surface slaking when immersed in water, severe slaking along bedding when dry, unweathered. Grades over [illegible] to unit D

Unit D, SANDSTONE, light gray (N7), friable, poorly cemented with silica, fine grained, well sorted, subangular to rounded, locally frosted, locally contains thin zones recovered as loose sand, bedding is massive, horizontal, locally contains traces of fresh pyrite, no apparent jointing, can be scratched with fingernail, breaks along bedding with light to moderate manual pressure, locally can be crushed in fingers, retains integrity when immersed in water, unweathered. Grades over [illegible] to unit E

Unit E, SILTY SANDSTONE, light olive gray (5Y6/1), similar to unit B, locally contains siltstone partings to 1/4 inch thick, no apparent jointing, can be scratched with knife, breaks along partings with moderate to strong manual pressure, retains integrity when immersed in water, unweathered. Grades over [illegible] to unit F

Unit F, SANDSTONE, medium bluish gray (5B5/1), slightly friable, moderately cemented with silica, medium grained, well sorted, rounded, frosted, locally contains traces of fresh pyrite, bedding is massive, horizontal, no apparent jointing, can be scratched with fingernail, breaks with light hammer blow or very strong manual pressure, retains integrity when immersed in water, unweathered. Grades over [illegible] to unit G

Unit G, SILTY SANDSTONE, light olive gray (5Y6/1), similar to units B and E, contains closely spaced, irregular siltstone partings to 1/4 inch thick, locally contains concentrations of medium grained, rounded sand, thinly bedded, bedding is horizontal, no apparent jointing, can be scratched with fingernail, breaks along partings with light hammer blow, retains integrity when immersed in water, unweathered. Grades over 1' to unit H

Unit H, SANDSTONE, medium bluish gray (5B5/1), similar to unit F. Locally contains concentrations of silty sandstone similar to units B, E, and G, thinly to thickly bedded, bedding is horizontal, no apparent jointing, can be scratched easily with knife, six inch core breaks with very strong hammer blows, retains integrity when immersed in water, unweathered. Grades over 2' to Shakopee dolomite

Shakopee Formation, DOLOMITIC LIMESTONE, gray (N6), locally sandy, medium grained, locally iron stained, contains numerous, small vugs, vugs partially filled with blue clay. Closely fractured, fractures are irregular, fracture surfaces coated with blue clay, can be scratched with knife, six inch core breaks with very strong hammer blows, unweathered

Notes:

1. Elevations are mean sea level (1912 adj.)
2. Lithologic units A, B, C and D defined by USCE, units A', E, F, G and H are defined in this study
3. Stratigraphic column is compilation from 1974 drilling, refer to borehole logs for individual features
4. Colors are taken from the Rock Color Chart distributed by the Geological Society of America (GSA)

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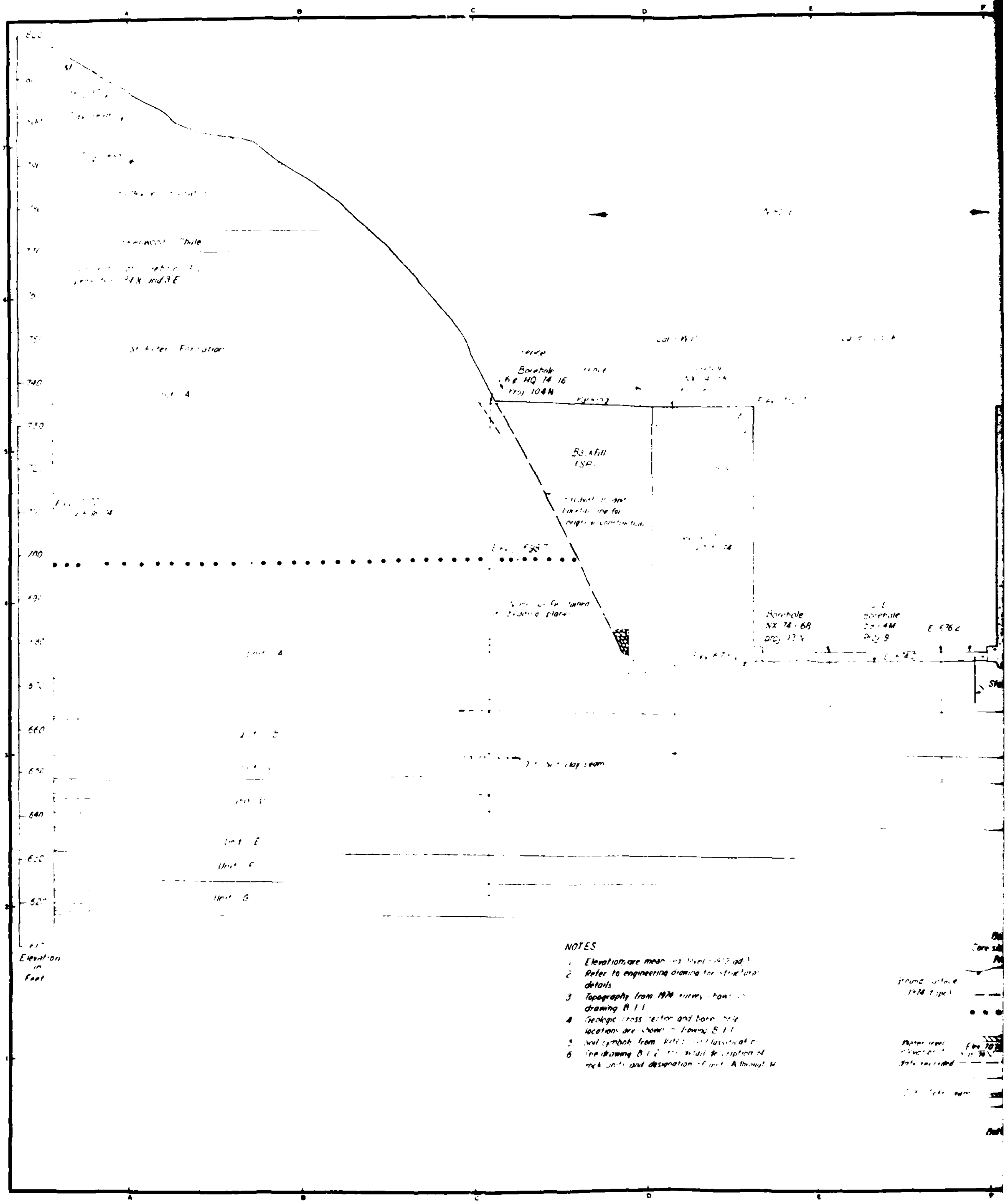
DESIGNED BY: R.E.K.
DRAWN BY: R.P.B.
CHECKED BY: M.[illegible]
SUBMITTED BY:
APPROVED:

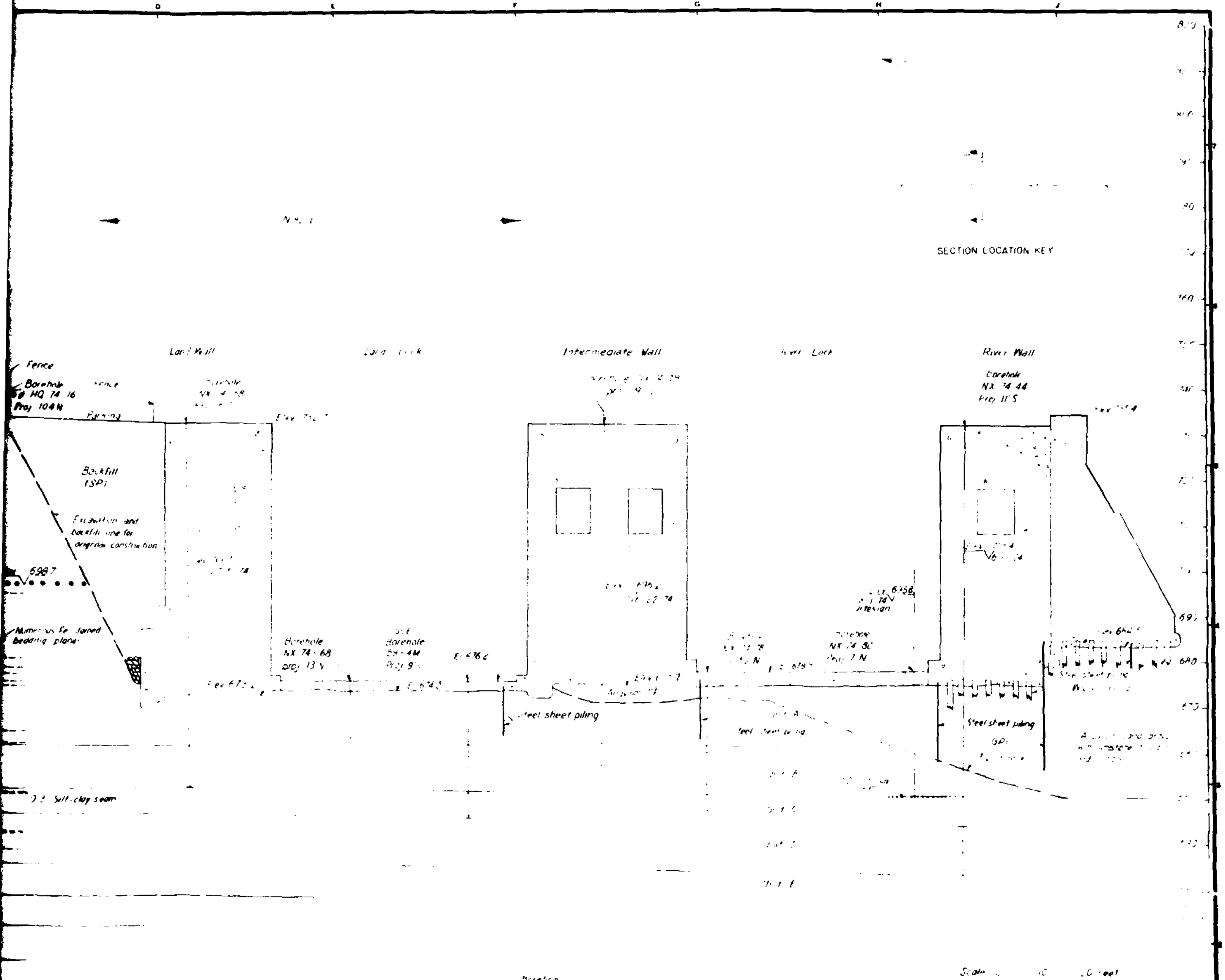
STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO. 1
GEOLOGY AND FOUNDATION EXPLORATION
STRATIGRAPHIC COLUMN

DATE: MARCH 1975
DRAWING NUMBER
SHEET OF

PLATE B-I-2



Shale
St Peter Formation
Borehole
HQ 74 16
104N
Backfill
(SP)
silt clay seam
Borehole
NX 74 - 6B
Borehole
Unit E
Unit F
Unit G
740
700
660
640
Elevation
in
Feet
NOTES
Elevations are mean
Refer to engineering
details
Topography from
drawing B 1 1
locations
See drawing B 1 2
and designation



NOTES

1. Elevations are mean sea level (1912 adj.)
2. Refer to engineering drawing for structural details
3. Topography from 1974 survey, drawing B-1-1
4. Geologic cross section and bore hole locations are shown in drawing B-1-1
5. Soil symbols from Unified soil classification
6. See drawing B-1-2 for detail description of rock units and designation of unit A through K

Borehole
Core size/Hole num.
Projection

Ground surface (1974 Topo)

Water level elevation and date recorded

Bottom of hole

Scale 0 10 20 Feet
Horizontal and vertical

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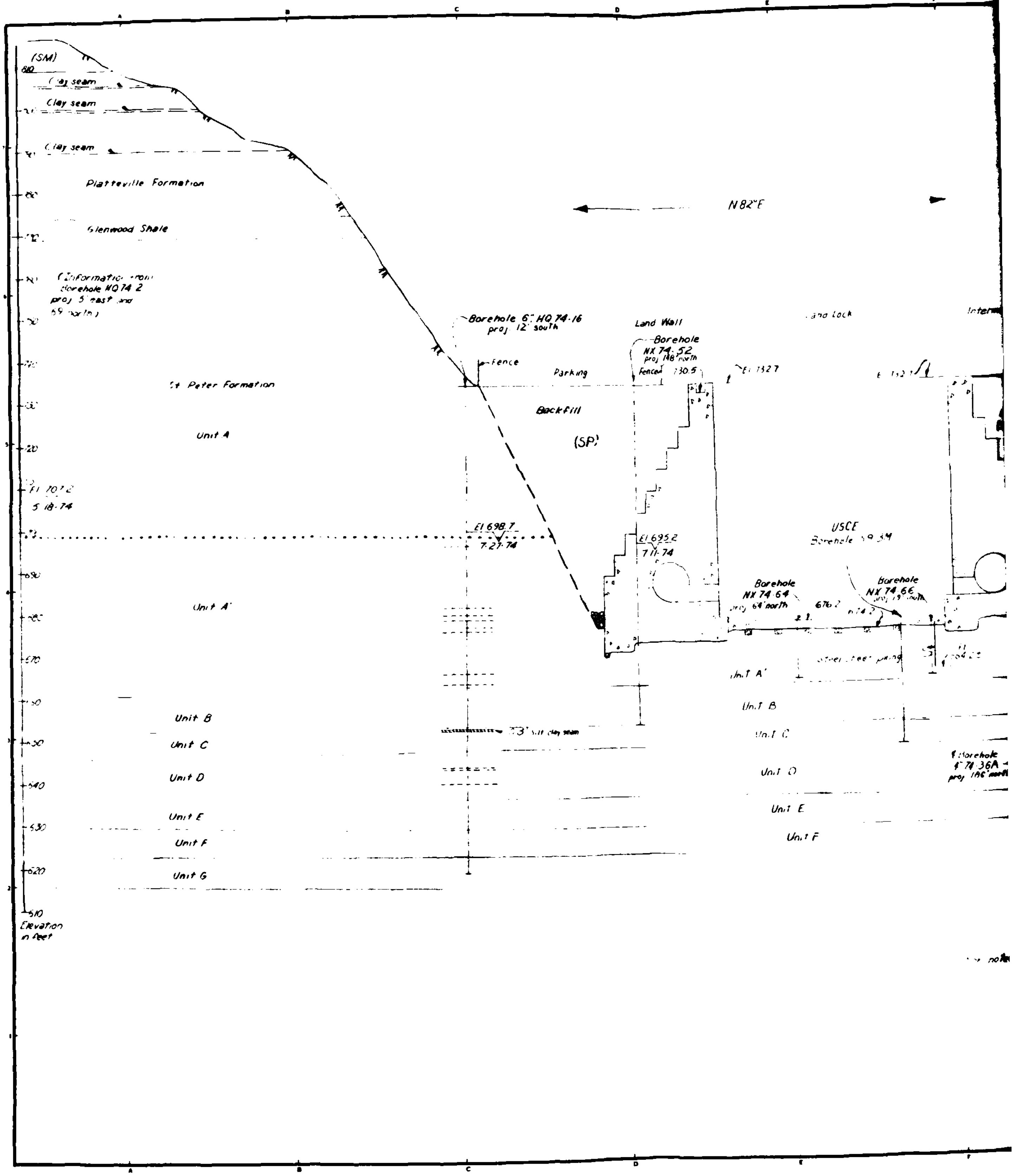
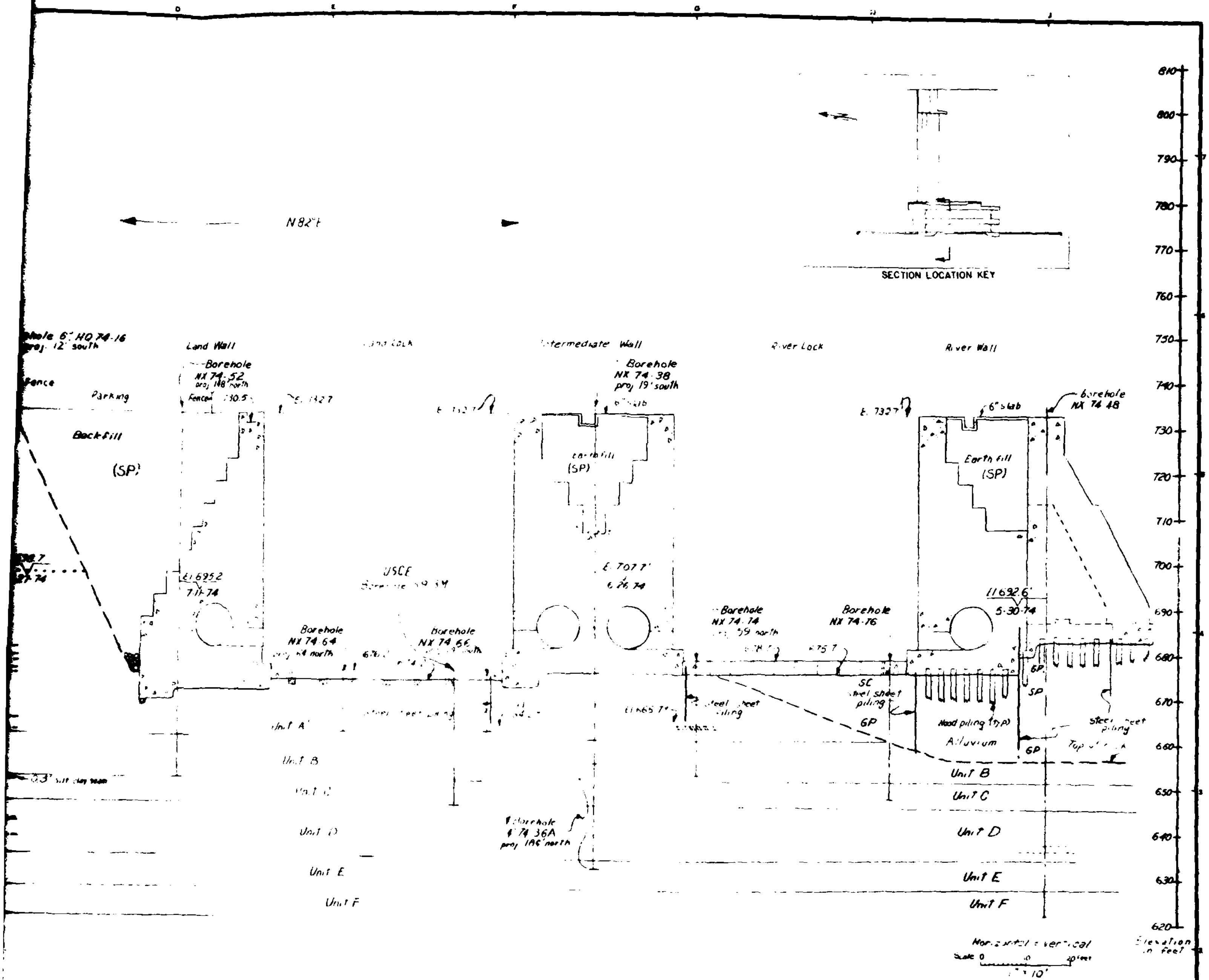
DESIGNED BY R P K	STUDY OF ALTERNATIVES FOR REHABILITATION
DRAWN BY R P K	MISSISSIPPI RIVER LOCK & DAM NO 1
CHECKED BY M J S	GEOLOGY AND FOUNDATION EXPLORATION
SUBMITTED BY	GEOLOGIC SECTION A-A
APPROVED	DATE MARCH 1975
	DRAWING NUMBER
	SHEET OF

PLATE B-I-3



(SM)
Clay seam
Clay seam
Clay seam
Platteville Formation
Glenwood Shale
(Information from borehole HQ 74 2 proj 5' east and 59' north)
St Peter Formation
Unit A
El 707.2
5 18-74
Unit A'
Unit B
Unit C
Unit D
Unit E
Unit F
Unit G
Elevation in Feet
N 82° E
Borehole 6" HQ 74-16 proj 12' south
Fence
Parking
Backfill
(SP)
El 698.7
7-27-74
Land Wall
Borehole NX 74-52 proj 148' north
Fenced
El 695.2
7-11-74
USCE
Borehole
Borehole NX 74-64 proj 64' north
Borehole NX 74-66
Unit A'
Unit B
Unit C
Unit D
Unit E
Unit F
Land Lock



For notes and explanations see Plate B-1-3

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ST. PAUL, MINNESOTA

R.P.K.
R.P.K.
M.J.S.

STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO 1
GEOLOGY AND FOUNDATION EXPLORATION
GEOLOGIC SECTION B-B

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MARCH 1975

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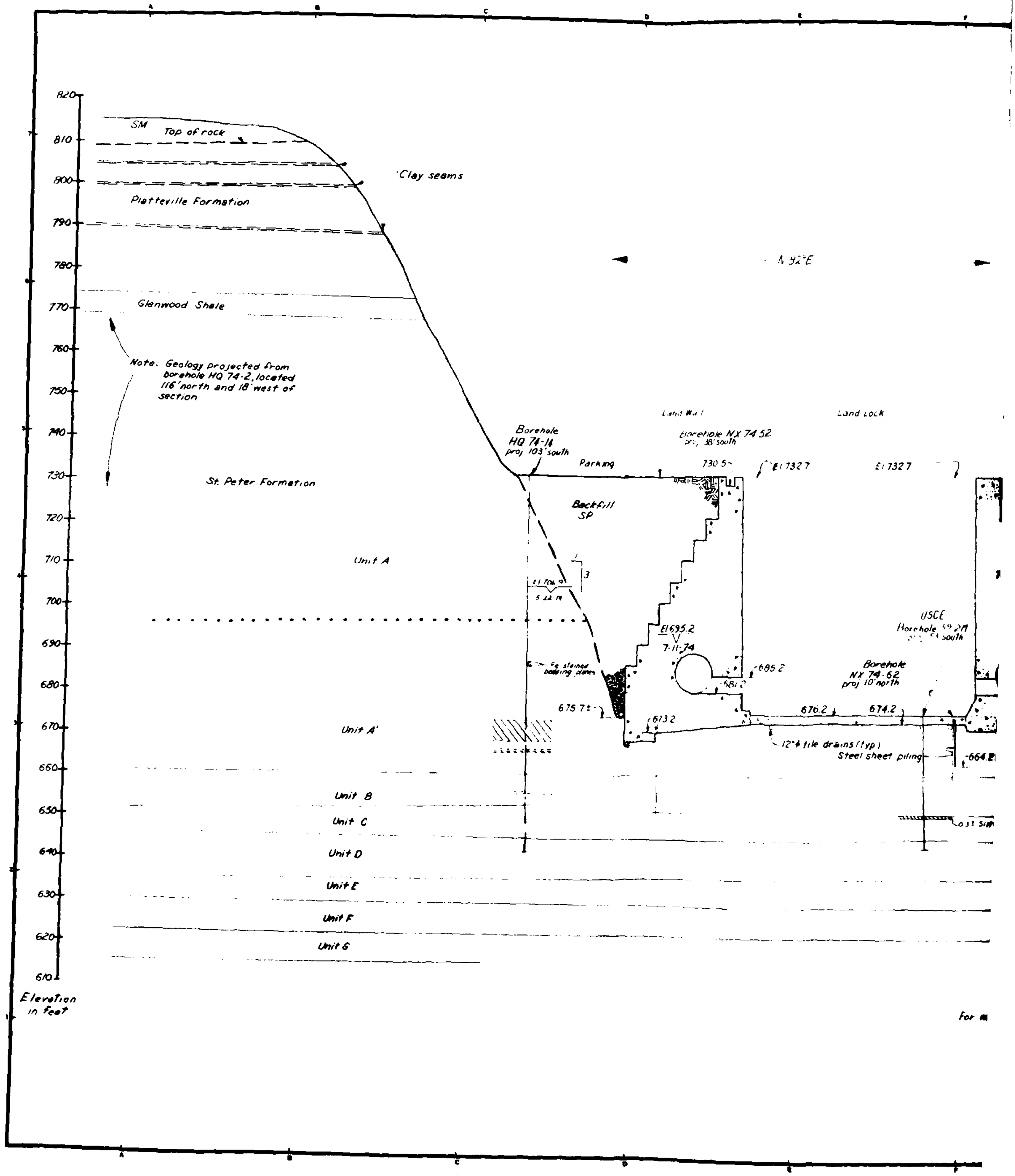
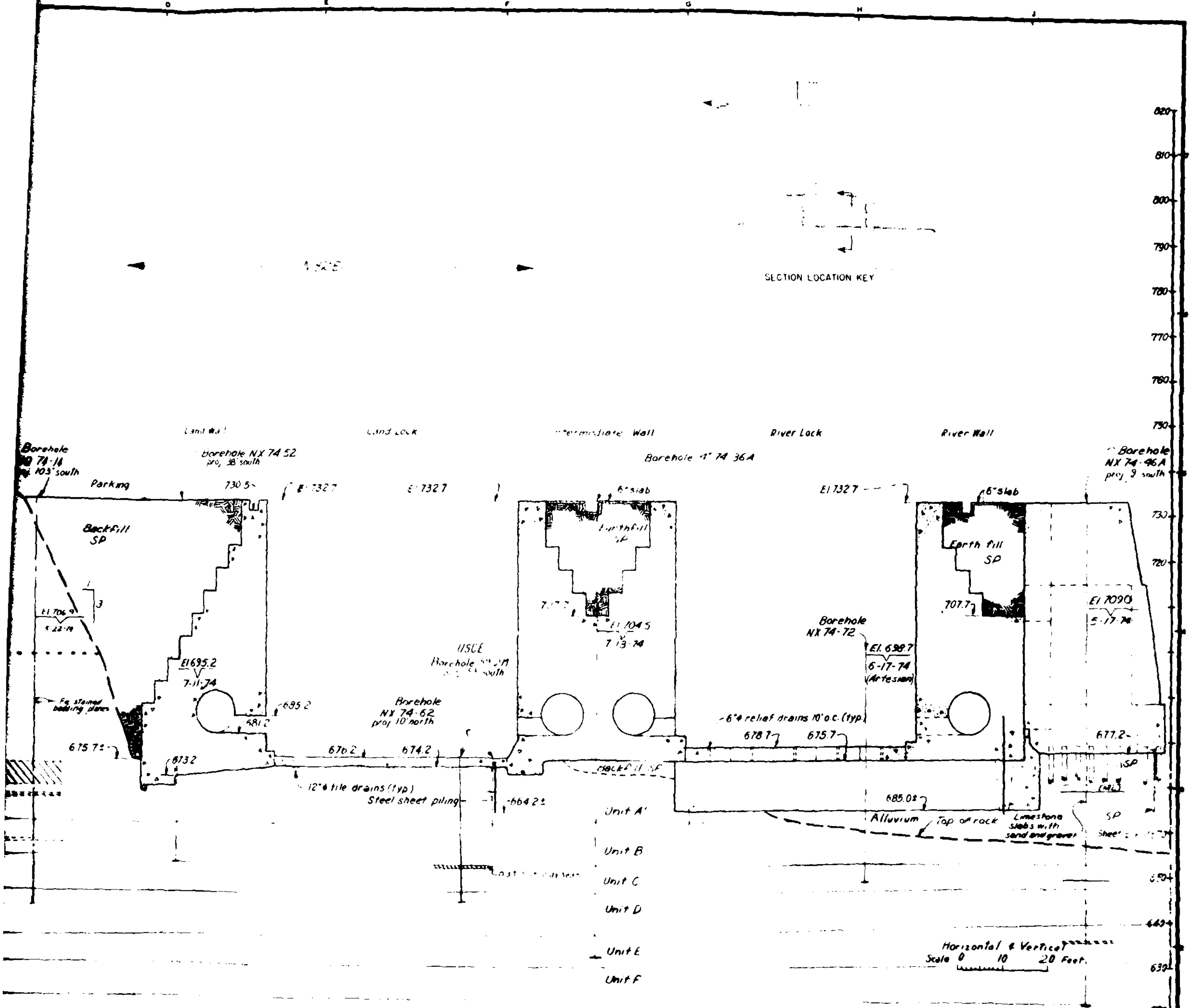
PLATE B-1-4



SM
Top of rock
Clay seams
Platteville Formation
N 92°E
Glenwood Shale
Note: Geology projected from borehole HQ 74-2, located 116' north and 18' west of section
Land Wall
Land Lock
Borehole HQ 74-14 proj 103' south
Borehole NX 74 52
Parking
St. Peter Formation
Backfill SP
Unit A
El 706.9
El 695.2
7-11-74
USCE
Borehole NX 74-62 proj 10' north
Unit A'
12"φ tile drains (typ)
Steel sheet piling
Unit B
Unit C
Unit D
Unit E
Unit F
Unit G
Elevation in feet



For notes & explanations see Plate B-1-3

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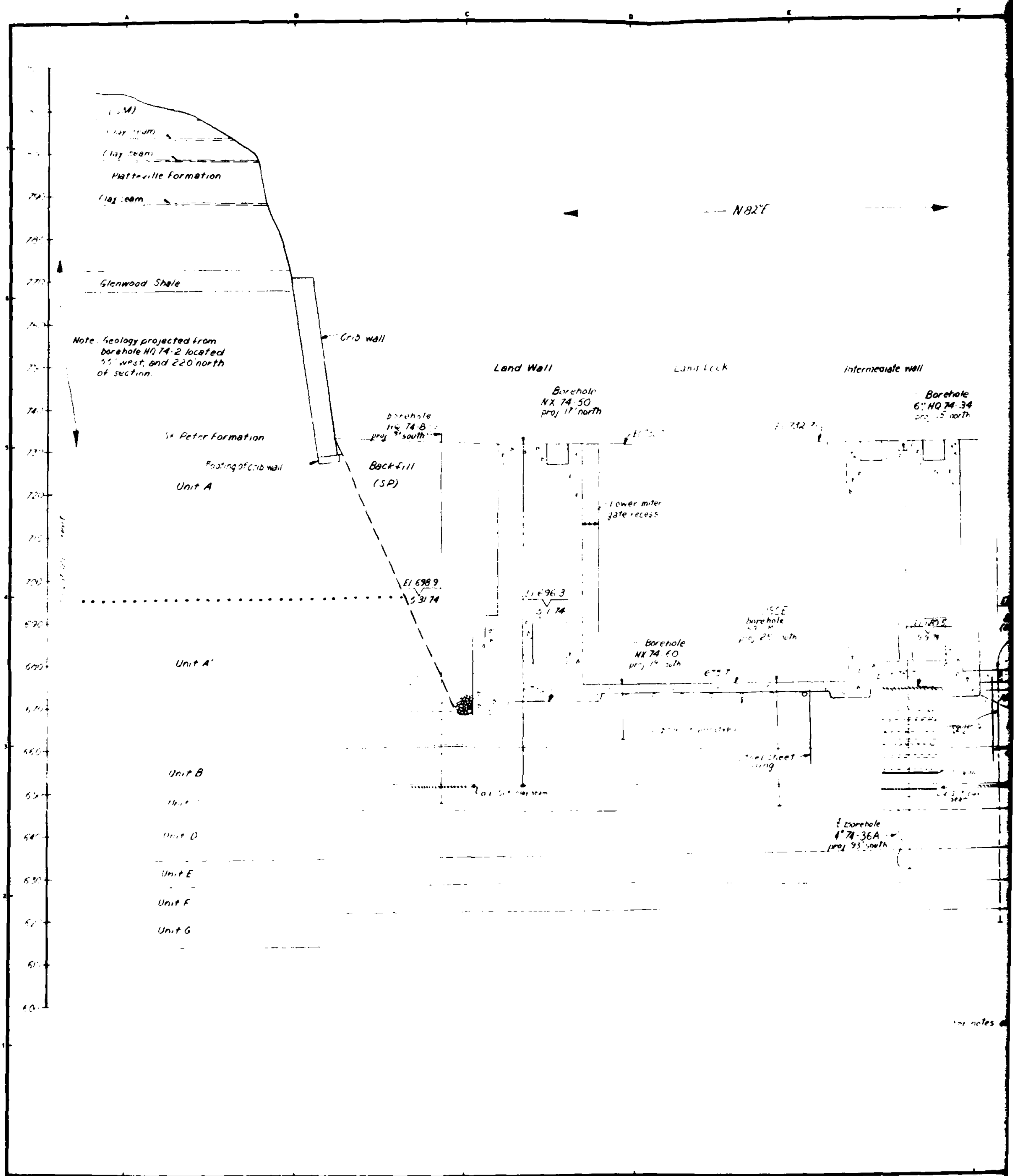
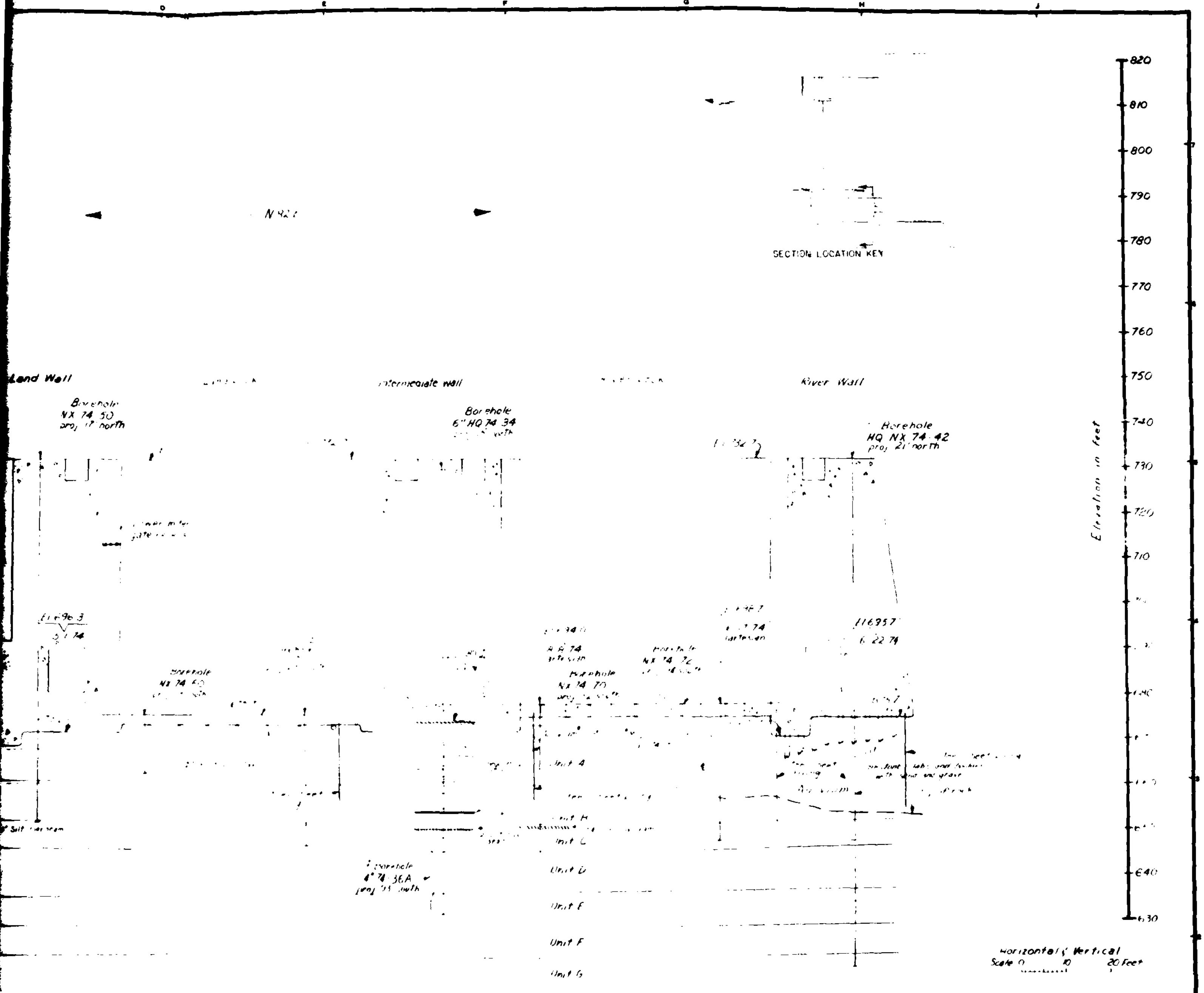
R. P. K. R. P. K. M. J. S.	STUDY OF ALTERNATIVES FOR REHABILITATION MISSISSIPPI RIVER LOCK & DAM NO. 1 GEOLOGY AND FOUNDATION EXPLORATION GEOLOGIC SECTION C-C
	MARCH 1975

SAFETY PAYS



(SM)
Clay seam
Clay seam
Platteville Formation
Clay seam
Glenwood Shale
Note. Geology projected from borehole HQ 74-2 located 55' west, and 220' north of section.
Crib wall
Land Wall
Land Lock
Intermediate wall
N82°E
Borehole NX 74-50 proj 17' north
Borehole 6" HQ 74-34
borehole HQ 74-8 proj 9' south
St Peter Formation
Footing of Crib wall
Backfill (SP)
Unit A
Lower miter gate recess
El 698.9 5-31-74
Unit A'
Borehole NX 74-60
borehole
Steel sheet piling
Unit B
Unit D
Unit E
Unit F
Unit G
Borehole 4" 74-36A proj 93' south



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STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO 1

GEOLOGY AND FOUNDATION EXPLORATION
GEOLOGIC SECTION D-D

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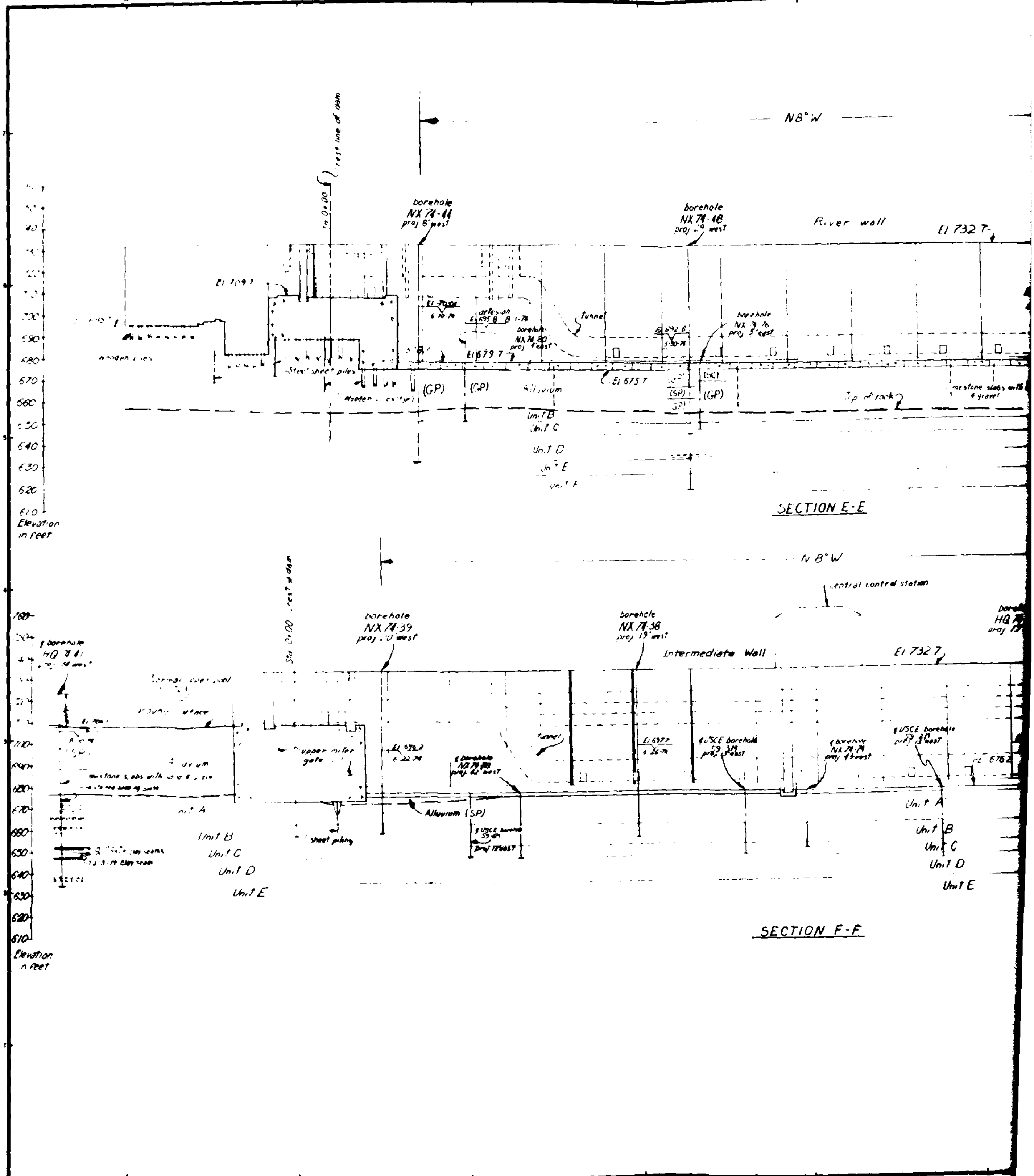
APPROVED

DATE
MARCH 1975

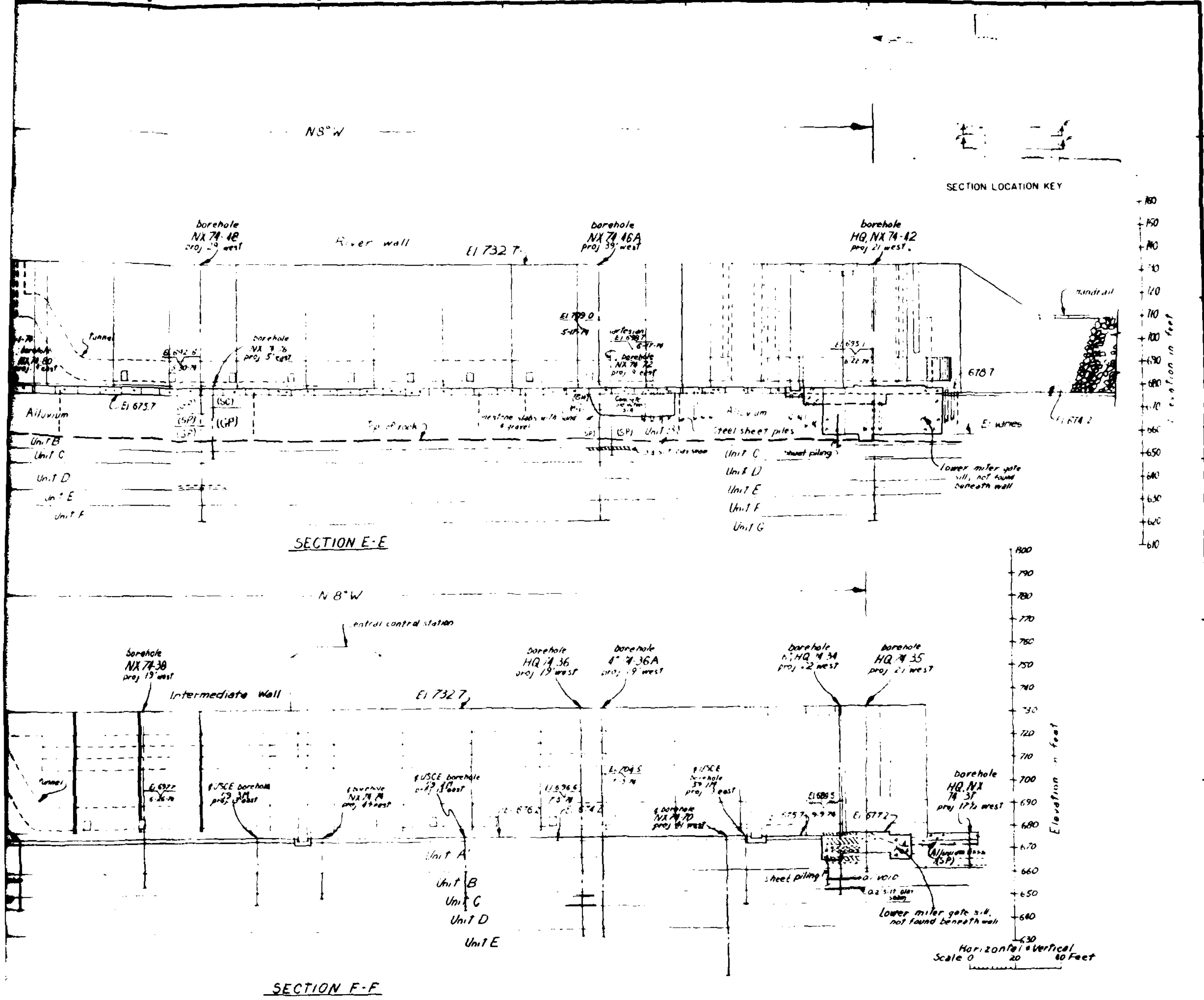
DRAWING NUMBER

SHEET OF

PLATE B-1-6



N8°W
Crest line of dam
Sta 0+00
borehole
NX 74-44
proj 8' west
borehole
NX 74-48
River wall
El 732.7
El 709.7
tunnel
borehole
NX 74-80
El 679.7
Steel sheet piles
wooden piles
(GP)
(GP)
Alluvium
El 675.7
(SC)
(SP)
(GP)
Top of rock
Unit B
Unit C
Unit D
Unit E
Unit F
Elevation in feet
SECTION E-E
N 8° W
Central control station
borehole
NX 74-39
borehole
NX 74-38
proj 19' west
Intermediate Wall
El 732.7
borehole
HQ 74-41
Alluvium
upper miter gate
tunnel
USCE borehole
borehole
Alluvium (SP)
sheet piling
Unit A
Unit B
Unit C
Unit D
Unit E
Elevation in feet
SECTION F-F



For notes and explanations see Plate B-1-3.

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R P K — STUDY OF ALTERNATIVES FOR REHABILITATION

R P K — MISSISSIPPI RIVER — LOCK & DAM NO 1

M J S — GEOLOGY AND FOUNDATION EXPLORATION

GEOLOGIC SECTIONS E-E AND F-F

SUBMITTED BY:

APPROVED:

DATE: MARCH 1975

DRAWING NUMBER

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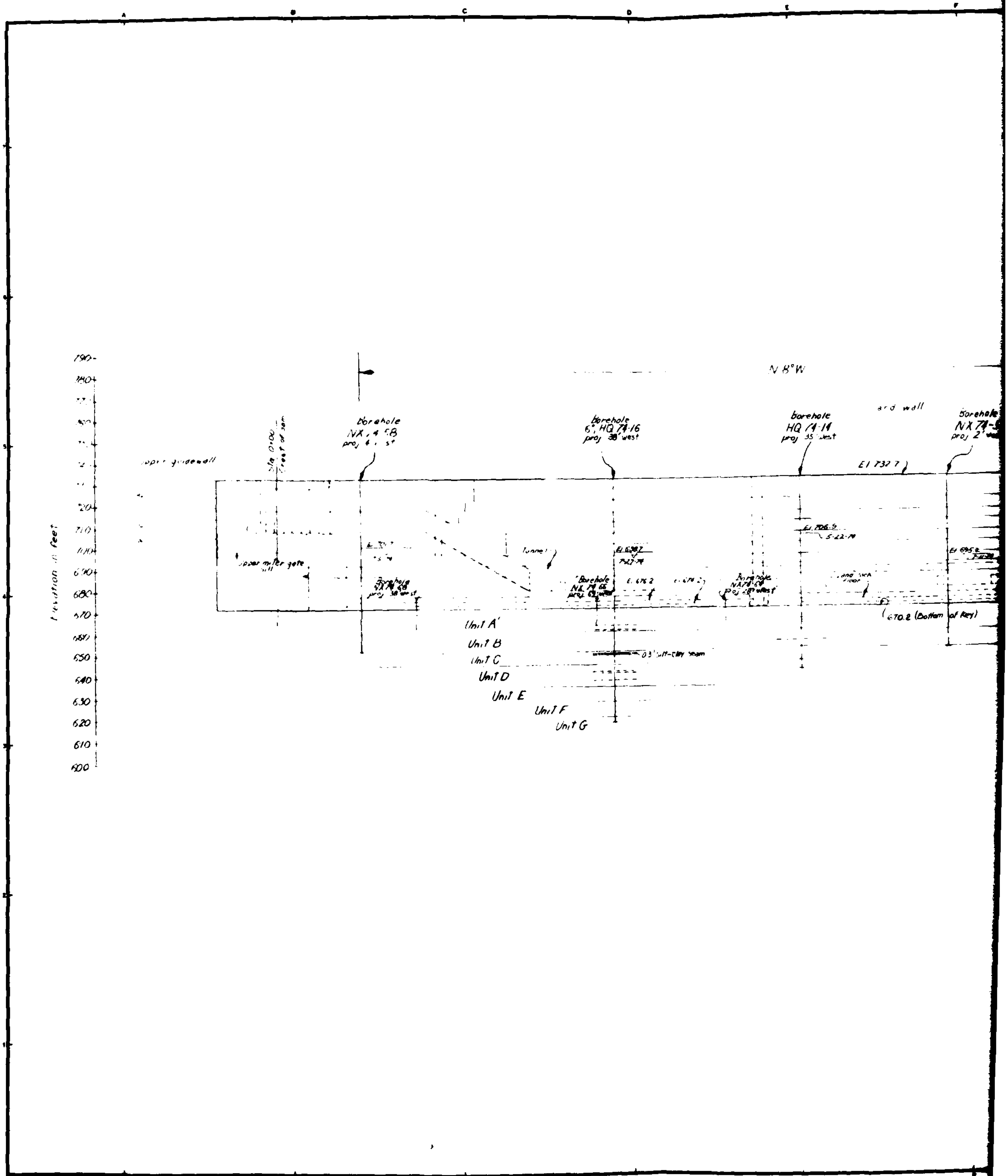
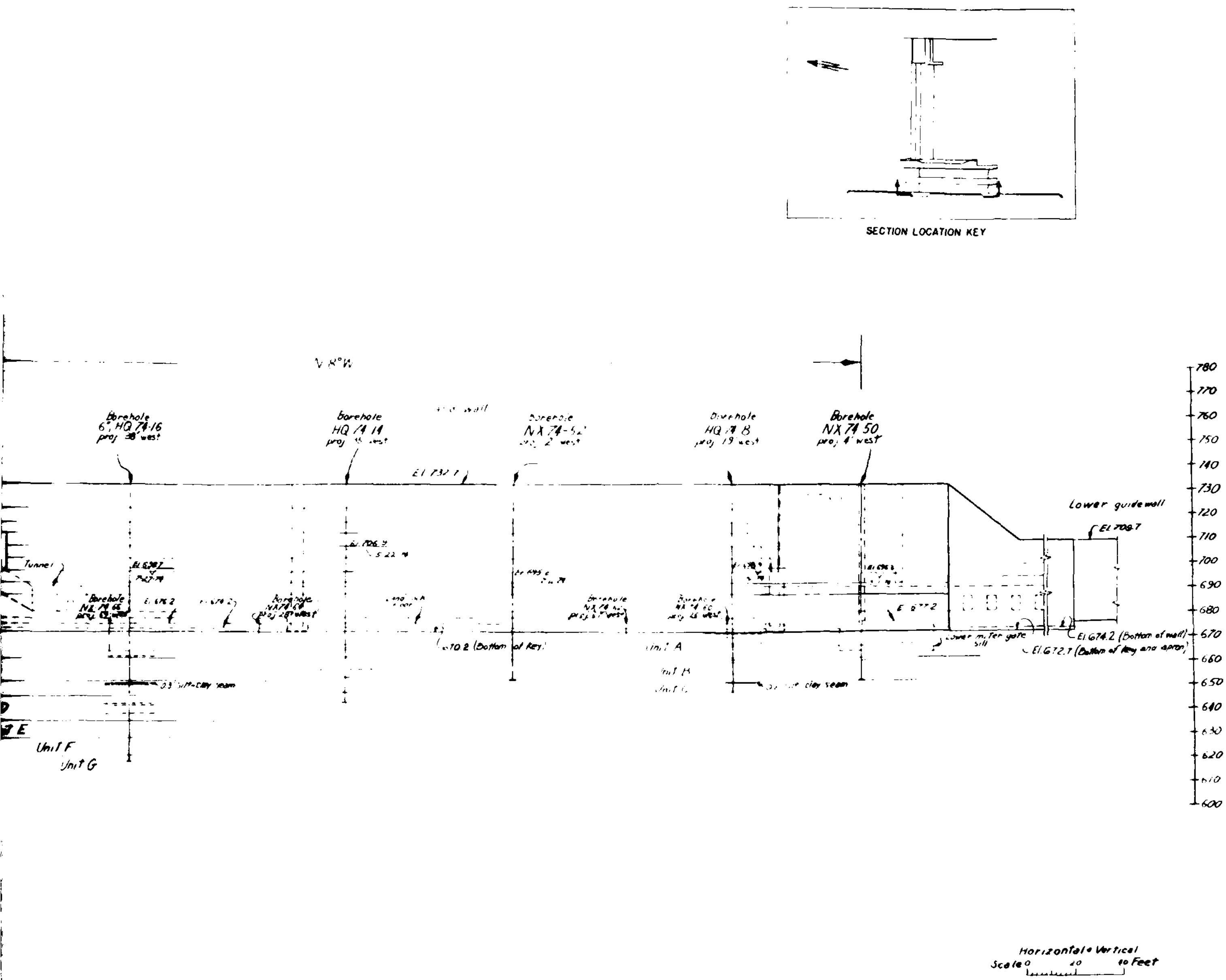
PLATE B-1-7

2



A
B
C
D
E
F
Elevation in feet
790
780
720
710
700
690
680
670
650
640
630
620
610
600
N 8°W
Borehole
6" HQ 74-16
proj 38' west
Borehole
HQ 74-14
proj 35' west
El 732.7
Tunnel
Unit A'
Unit B
Unit C
Unit D
Unit E
Unit F
Unit G
0.3' silt-clay seam
670.2 (Bottom of key)



For notes and explanations see Plate B-1-3.

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ST. PAUL DISTRICT CORPS OF ENGINEERS
ST. PAUL, MINNESOTA

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DRAWN BY R. P. K.
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SUBMITTED BY
APPROVED

STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO 1

GEOLOGY AND FOUNDATION EXPLORATION
GEOLOGIC SECTION G-G

DATE MARCH 1975

DRAWING NUMBER

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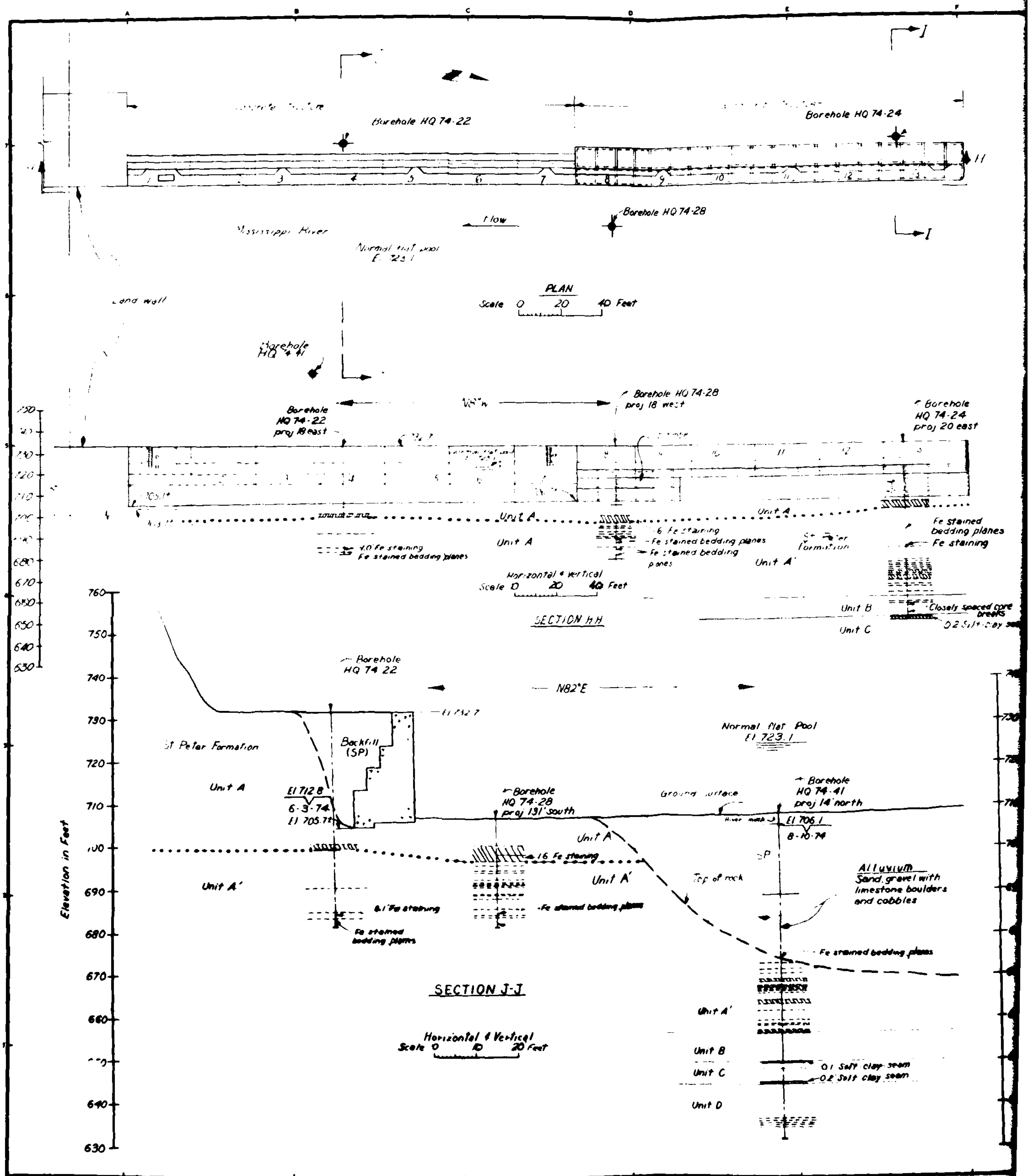
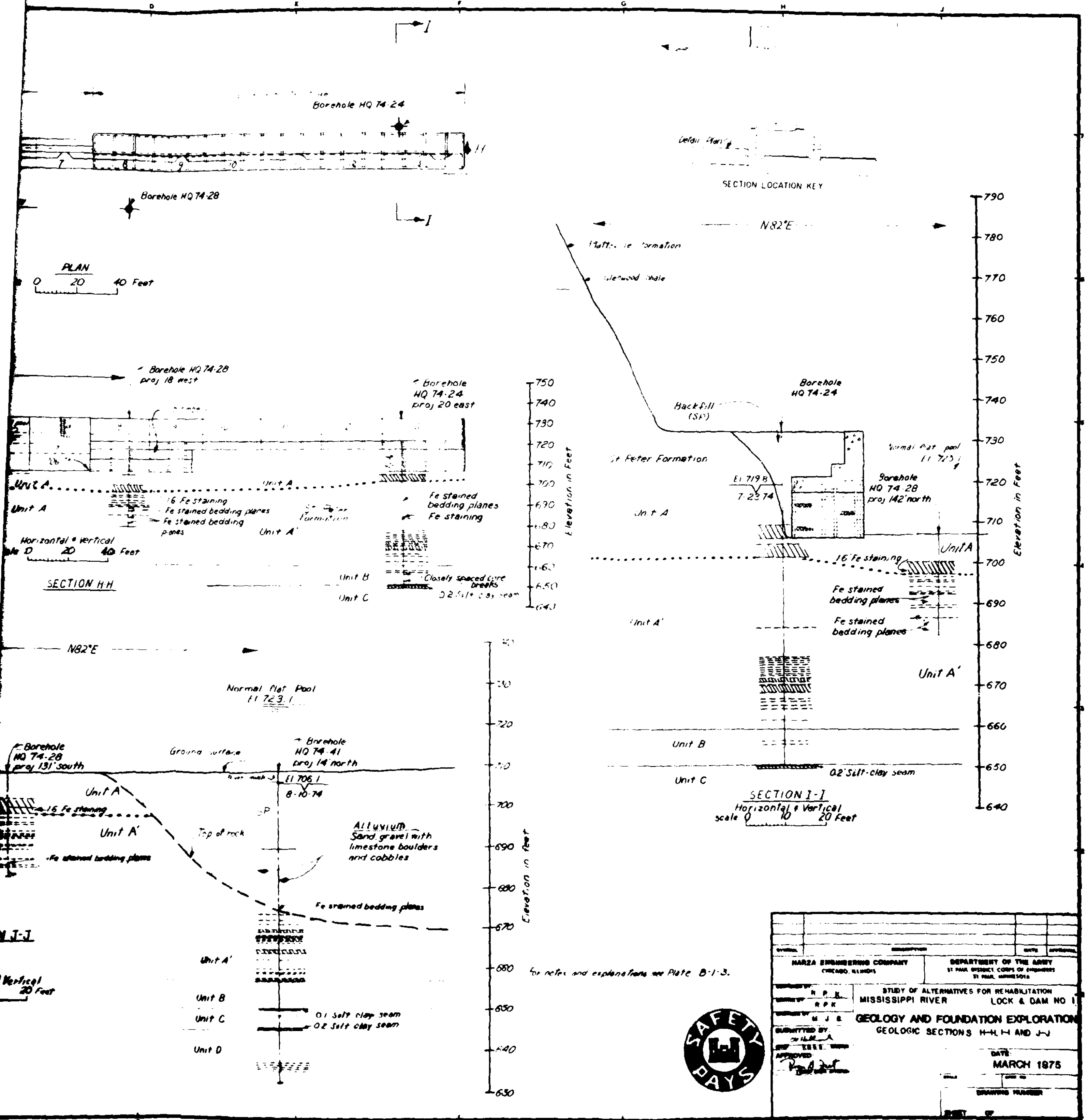
SAFETY PAYS

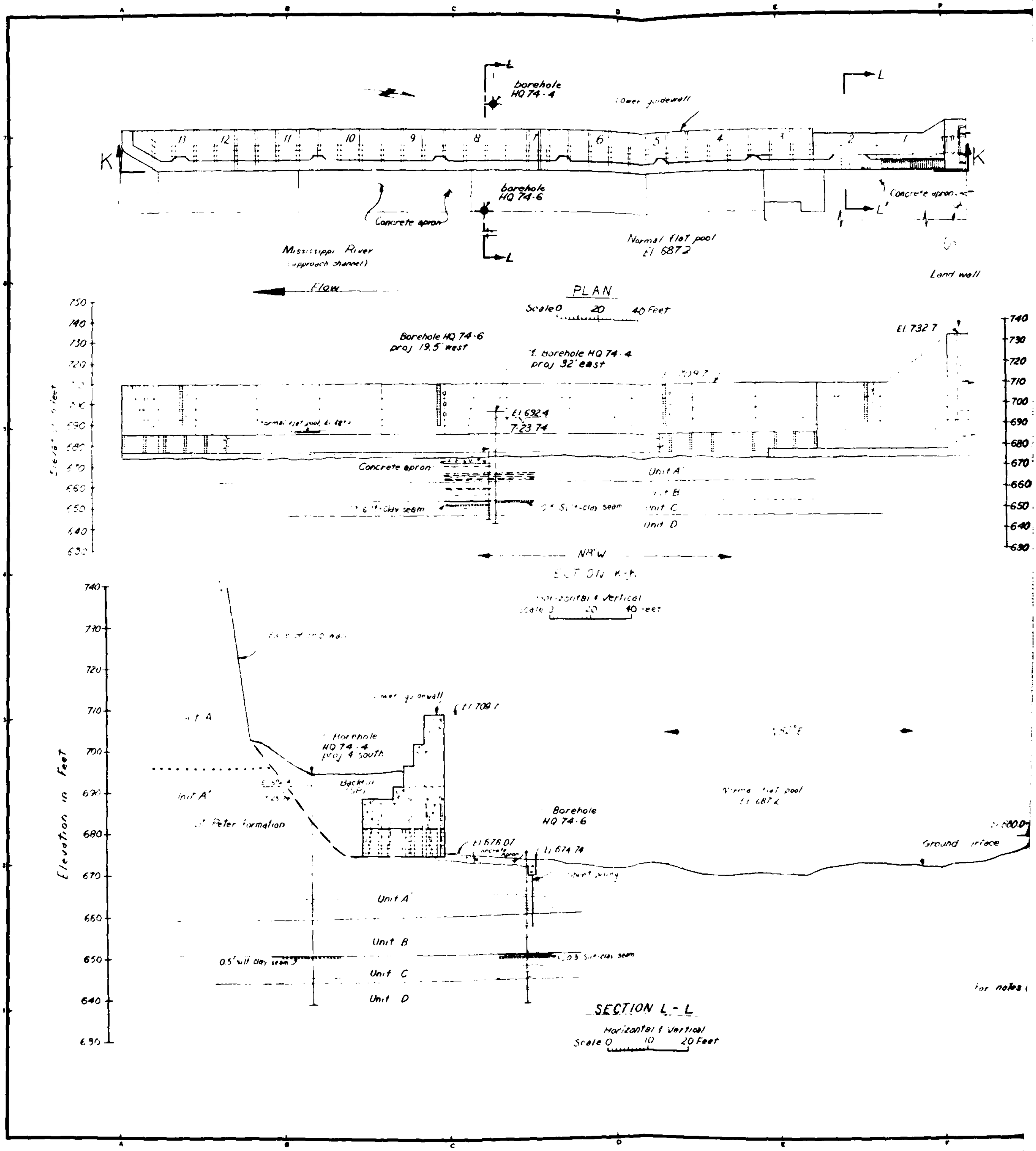
PLATE B-1-8



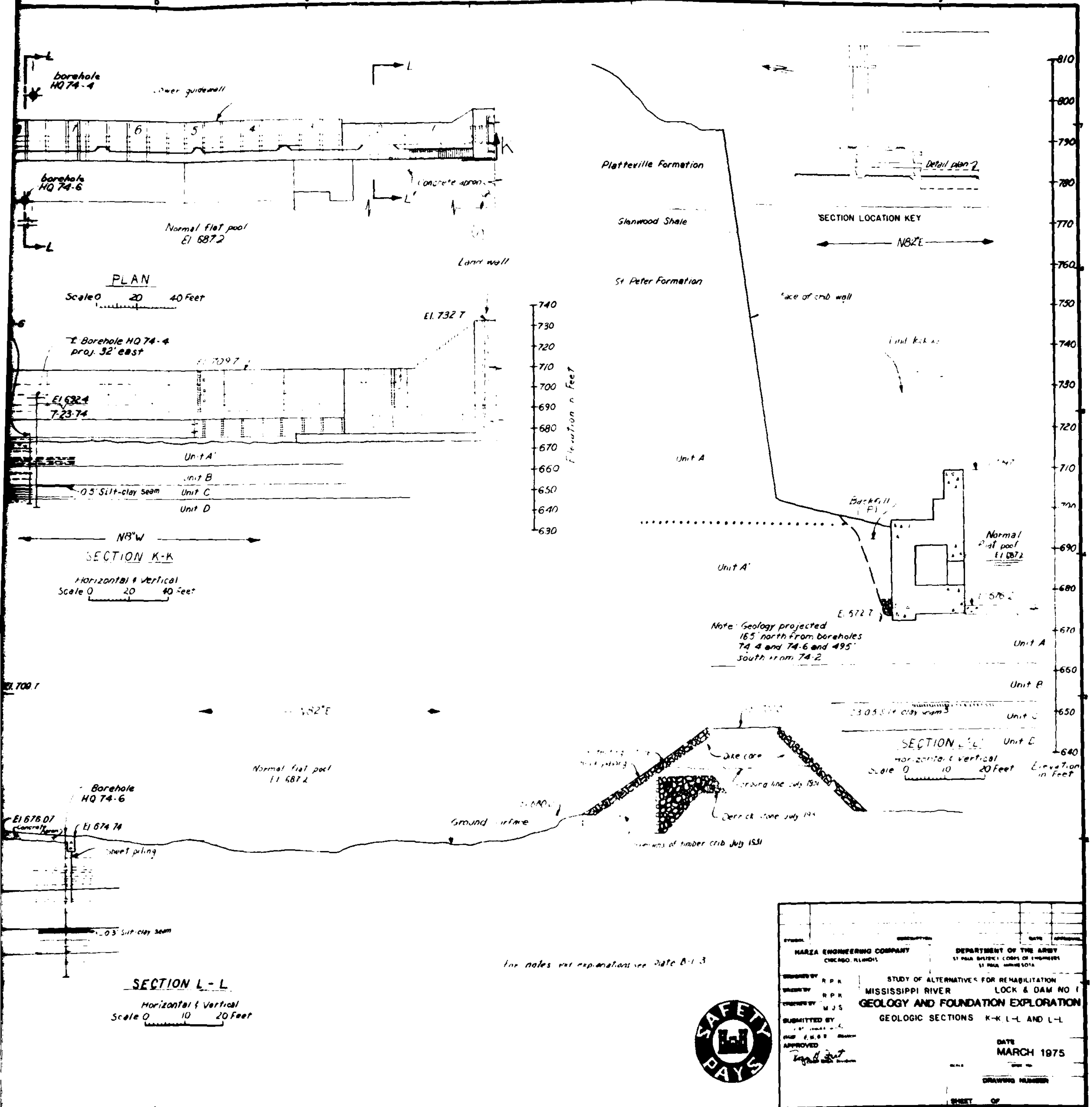
Borehole HQ 74-22
Borehole HQ 74-24
Borehole HQ 74-28
Mississippi River
Flow
Normal flat pool El 723.1
PLAN
Scale 0 20 40 Feet
Land wall
Borehole HQ 74-41
Borehole HQ 74-22 proj 18 east
Borehole HQ 74-28 proj 18 west
Borehole HQ 74-24 proj 20 east
Unit A
Unit A'
4.0' Fe staining
Fe stained bedding planes
.6 Fe staining
Fe stained bedding planes
St Peter Formation
Fe stained bedding planes
Fe staining
Horizontal & vertical Scale 0 20 40 Feet
SECTION H-H
Unit B
Unit C
Closely spaced core breaks
0.2 Silt-clay seam
Elevation in Feet
Borehole HQ 74 22
N82°E
El 732.7
Backfill (SP)
St Peter Formation
Normal flat Pool El 723.1
El 712.8
6-3-74
El 705.7
Borehole HQ 74-28 proj 131' south
Ground surface
Borehole HQ 74-41 proj 14' north
El 706.1
8-10-74
SP
Alluvium
Sand, gravel with limestone boulders and cobbles
Top of rock
1.6 Fe staining
8.1' Fe staining
Fe stained bedding planes
Fe stained bedding planes
Fe stained bedding planes
SECTION J-J
Horizontal & Vertical Scale 0 10 20 Feet
Unit D
0.1 Silt clay seam
0.2 Silt clay seam



Borehole HQ 74-24
Borehole HQ 74-28
SECTION LOCATION KEY
PLAN
0 20 40 Feet
N82°E
Borehole HQ 74-28 proj 18' west
Borehole HQ 74-24 proj 20' east
Unit A
.16 Fe staining
Fe stained bedding planes
Fe staining
Unit A'
Horizontal & Vertical scale 0 20 40 Feet
SECTION H-H
Unit B
Unit C
Closely spaced core breaks
0.2 Silt clay seam
Elevation in Feet
Borehole HQ 74-24
Backfill (SP)
St Peter Formation
El 719.8 7-23-74
Borehole HQ 74-28 proj 142' north
Normal flat pool El 723.1
.16 Fe staining
Unit A'
Unit B
Unit C
0.2' Silt-clay seam
SECTION I-I
Horizontal & Vertical scale 0 10 20 Feet
Normal flat Pool El 723.1
Borehole HQ 74-28 proj 131' south
Ground surface
Borehole HQ 74-41 proj 14' north
El 706.1 8-10-74
Top of rock
Alluvium
Sand, gravel with limestone boulders and cobbles
Fe stained bedding planes
Unit D
0.1 Silt clay seam
0.2 Silt clay seam
For notes and explanations see Plate B-1-3.
HARZA ENGINEERING COMPANY CHICAGO, ILLINOIS
DEPARTMENT OF THE ARMY ST PAUL DISTRICT CORPS OF ENGINEERS ST PAUL, MINNESOTA
STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO 1
GEOLOGY AND FOUNDATION EXPLORATION
GEOLOGIC SECTIONS H-H, I-I AND J-J
MARCH 1975
SAFETY PAYS
PLATE B-1-9



borehole HQ 74-4
Lower guidewall
borehole HQ 74-6
Concrete apron
Concrete apron
Normal flat pool El 687.2
Mississippi River (approach channel)
Land wall
Flow
PLAN
Scale 0 20 40 Feet
Borehole HQ 74-6 proj 19.5' west
Borehole HQ 74-4 proj 32' east
El 732.7
El 692.4 7-23-74
Concrete apron
Unit A'
Unit C
Unit D
NB'W
Horizontal & Vertical
Elevation in Feet
Borehole HQ 74-4 proj 4' south
El 709.7
St. Peter formation
Unit A'
Borehole HQ 74-6
El 676.07
El 674.74
Normal flat pool El 687.2
Ground surface
Unit A'
Unit B
0.5' silt clay seam
Unit C
Unit D
SECTION L - L
Horizontal & Vertical
Scale 0 10 20 Feet



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CHICAGO, ILLINOIS

DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT, CORPS OF ENGINEERS
ST. PAUL, MINNESOTA

DESIGNED BY R.P.K.
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SUBMITTED BY
APPROVED

STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO 1
GEOLOGY AND FOUNDATION EXPLORATION
GEOLOGIC SECTIONS K-K, L-L AND L'-L'

DATE MARCH 1975

DRAWING NUMBER

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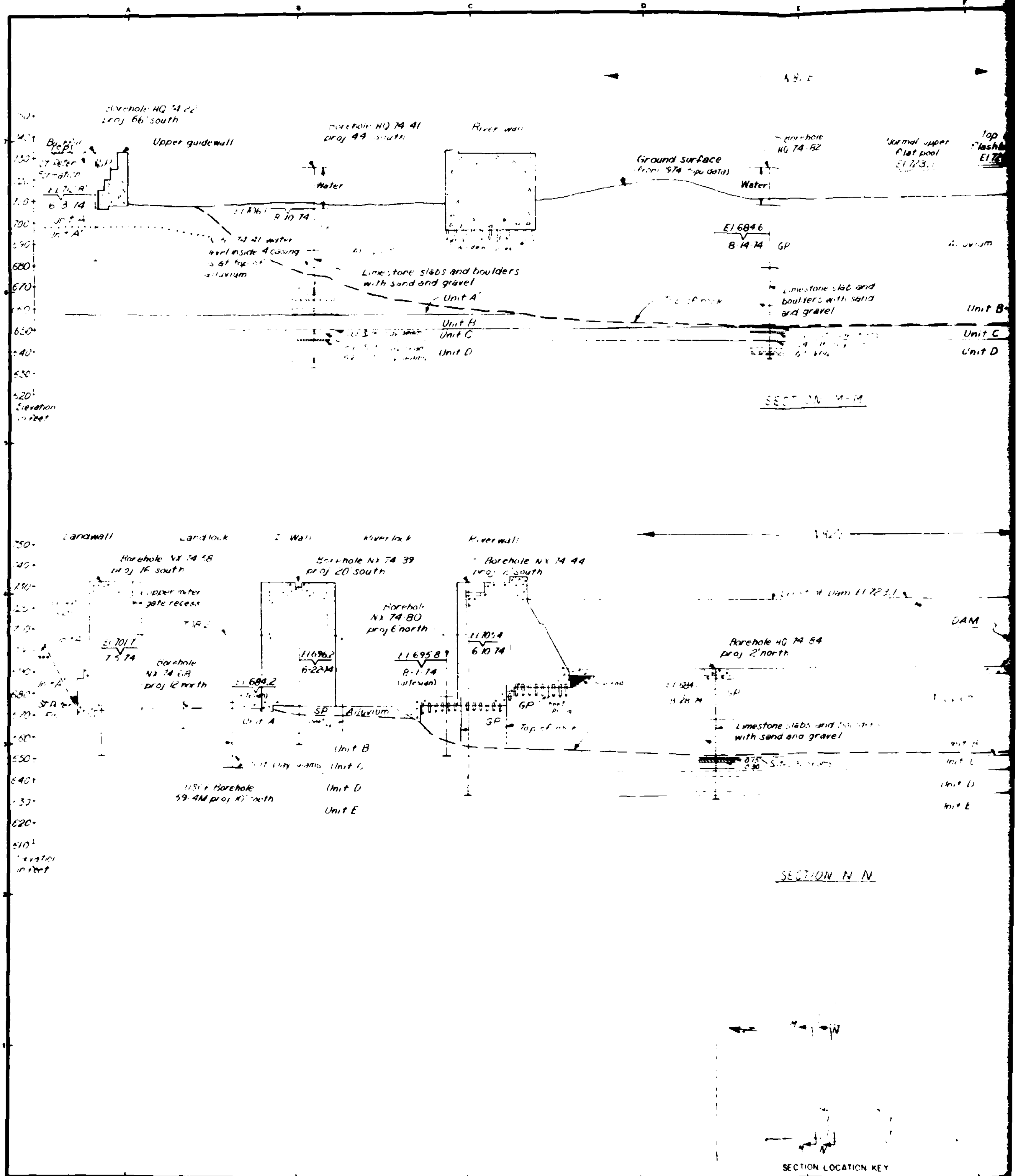
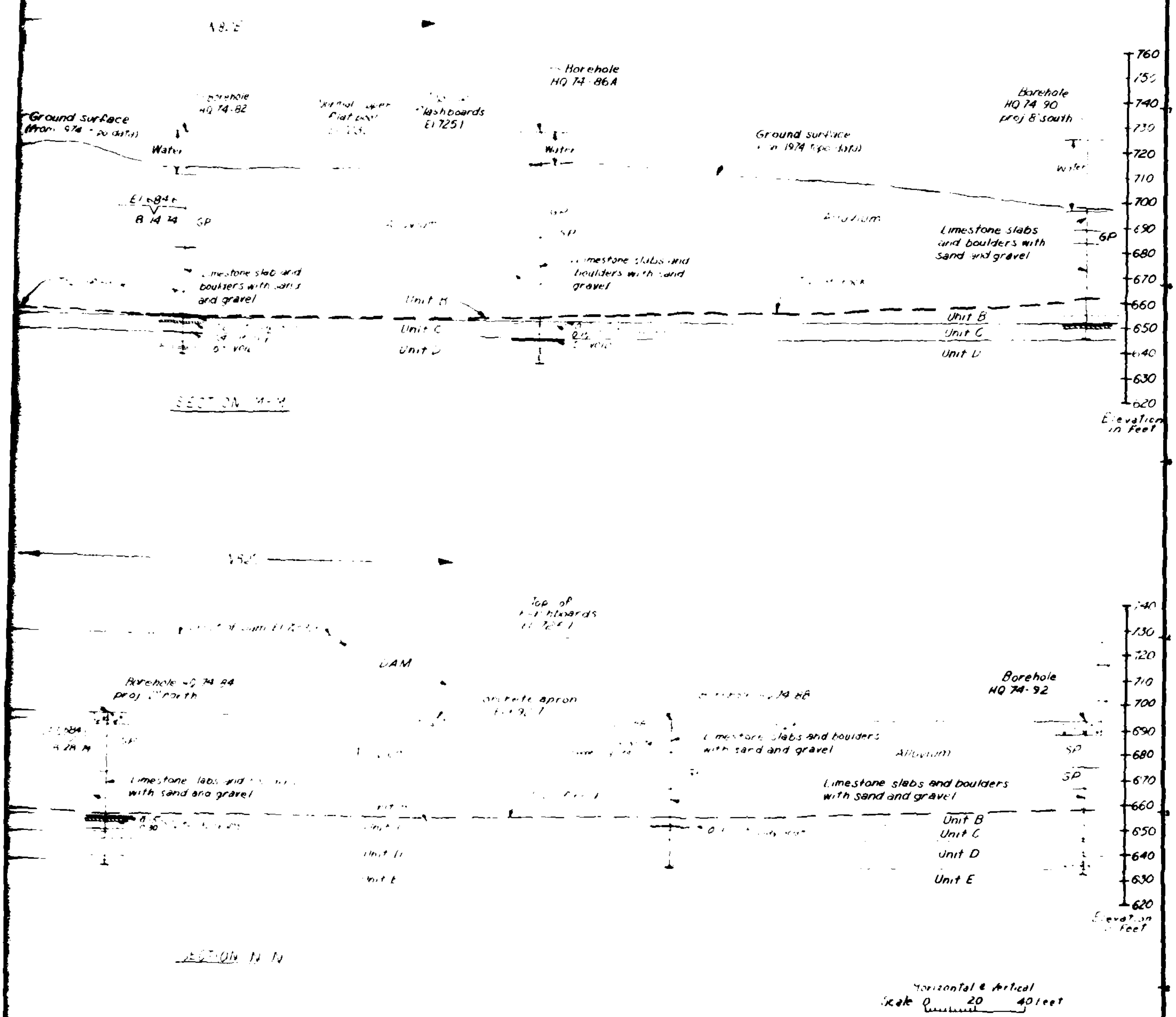
PLATE B-1-10



Borehole HQ 74-22
proj 66' south
Upper guidewall
Borehole HQ 74-41
proj 44' south
River wall
Ground surface
Water
Borehole
HQ 74-82
Normal upper
Flat pool
El 684.6
8-14-74
GP
Limestone slabs and boulders
with sand and gravel
Unit A'
Unit B
Unit C
Unit D
Limestone slab and
boulders with sand
and gravel
Alluvium
Elevation
in feet
SECTION M-M
Landwall
Riverwall
Borehole NX 74-48
proj 16' south
Upper miter
gate recess
Borehole NX 74-39
proj 20' south
Borehole NX 74-44
Borehole
NX 74-80
proj 6' north
El 701.7
7-5-74
El 696.2
6-22-74
El 695.8
8-1-74
El 705.4
6-10-74
Borehole
NX 74-68
proj 12' north
El 684.2
Unit A
SP
Alluvium
GP
Top of rock
Borehole HQ 74-84
proj 2' north
Limestone slabs and boulders
with sand and gravel
DAM
Unit B
Unit C
Unit D
Unit E
USGS Borehole
59-4M proj 10' north
Elevation
in feet
SECTION N-N
SECTION LOCATION KEY



HARZA ENGINEERING COMPANY CHICAGO, ILLINOIS	DEPARTMENT OF THE ARMY ST. PAUL DISTRICT CORPS OF ENGINEERS ST. PAUL, MINNESOTA

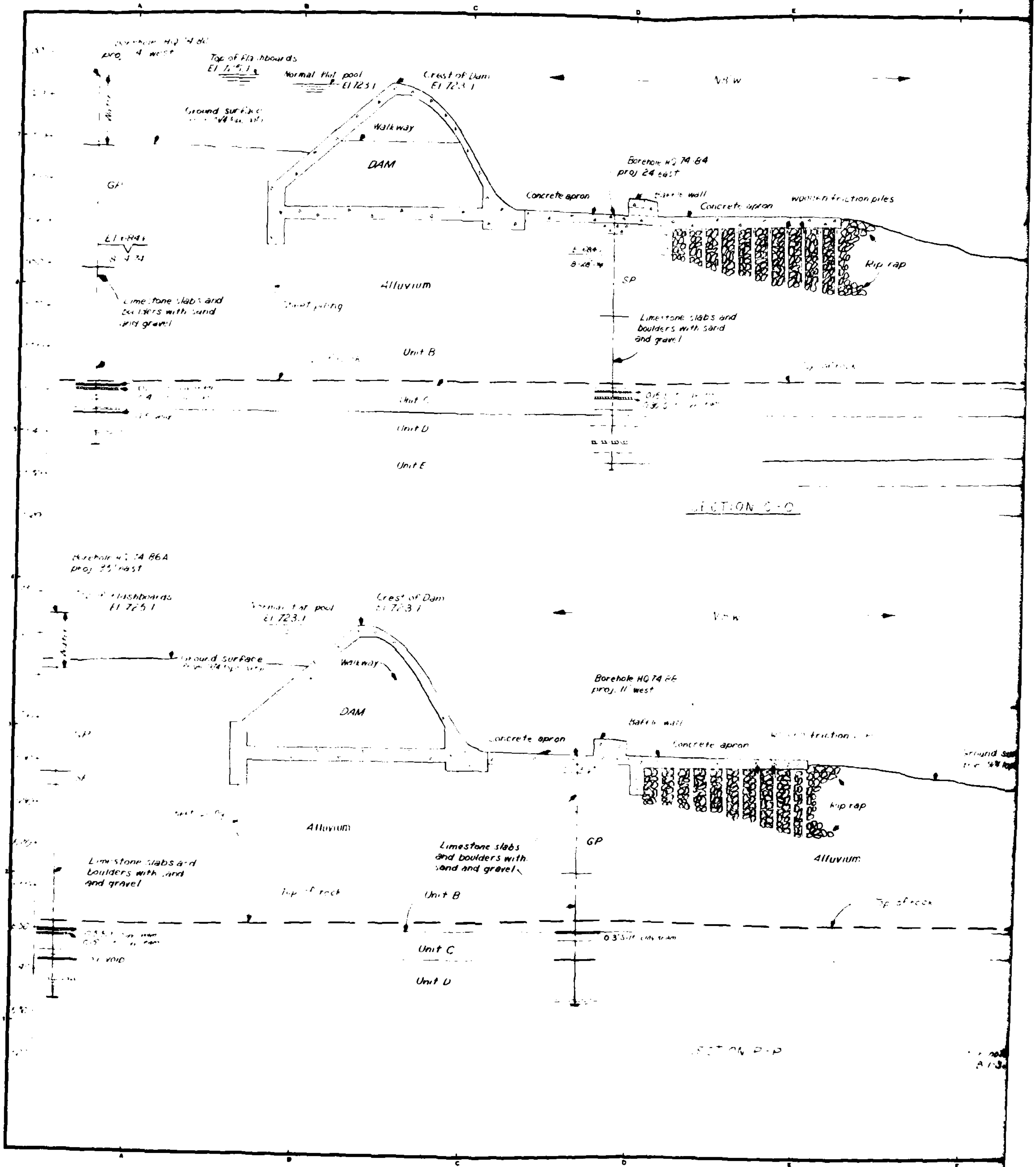
STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO. 1
GEOLOGY AND FOUNDATION EXPLORATION
GEOLOGIC SECTIONS M-M AND N-N

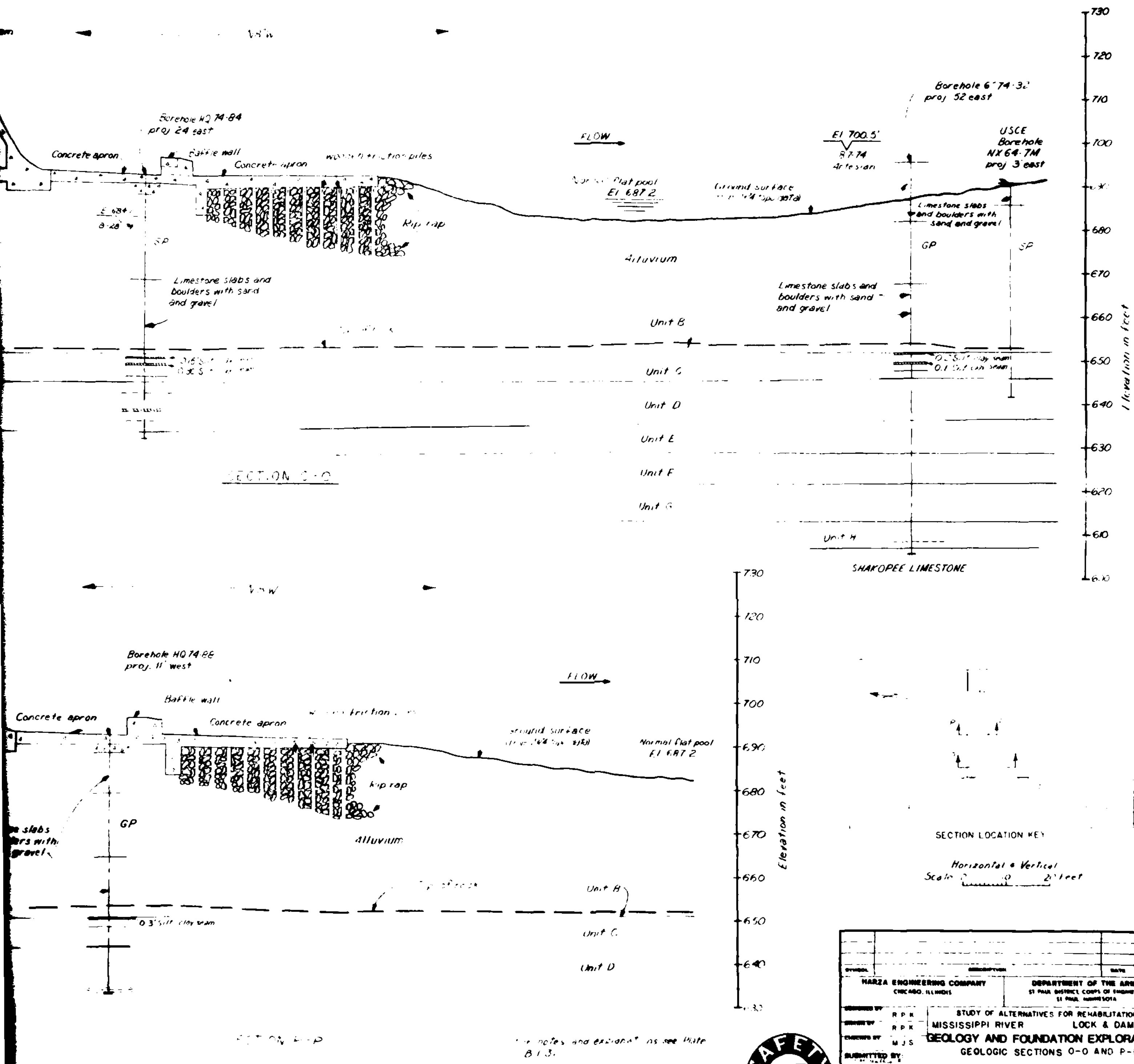
R.P.K.
R.P.K.
M.J.S.
SUBMITTED BY
APPROVED

DATE: MARCH 1975

SAFETY PAYS

PLATE B-I-11


Top of Flashboards
Normal flat pool El 723.1
Crest of Dam El 723.1
Ground surface
Walkway
DAM
Borehole HQ 74-84
proj. 24' east
Concrete apron
Baffle wall
Concrete apron
wooden friction piles
Rip rap
Alluvium
SP
Limestone slabs and boulders with sand and gravel
Limestone slabs and boulders with sand and gravel
Unit B
Unit C
Unit D
Unit E
SECTION C-C
Borehole HQ 74-86A
Flashboards El 725.1
Normal flat pool El 723.1
Crest of Dam El 723.1
Ground surface
Walkway
DAM
Borehole HQ 74-86
proj. 11' west
Baffle wall
Concrete apron
Concrete apron
Rip rap
Alluvium
Alluvium
GP
Limestone slabs and boulders with sand and gravel
Limestone slabs and boulders with sand and gravel
Top of rock
Unit B
Unit C
Unit D
SECTION D-D



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ST. PAUL DISTRICT, CORPS OF ENGINEERS
ST. PAUL, MINNESOTA

DESIGNED BY R P K
DRAWN BY R P K
CHECKED BY M J S
SUBMITTED BY:
APPROVED

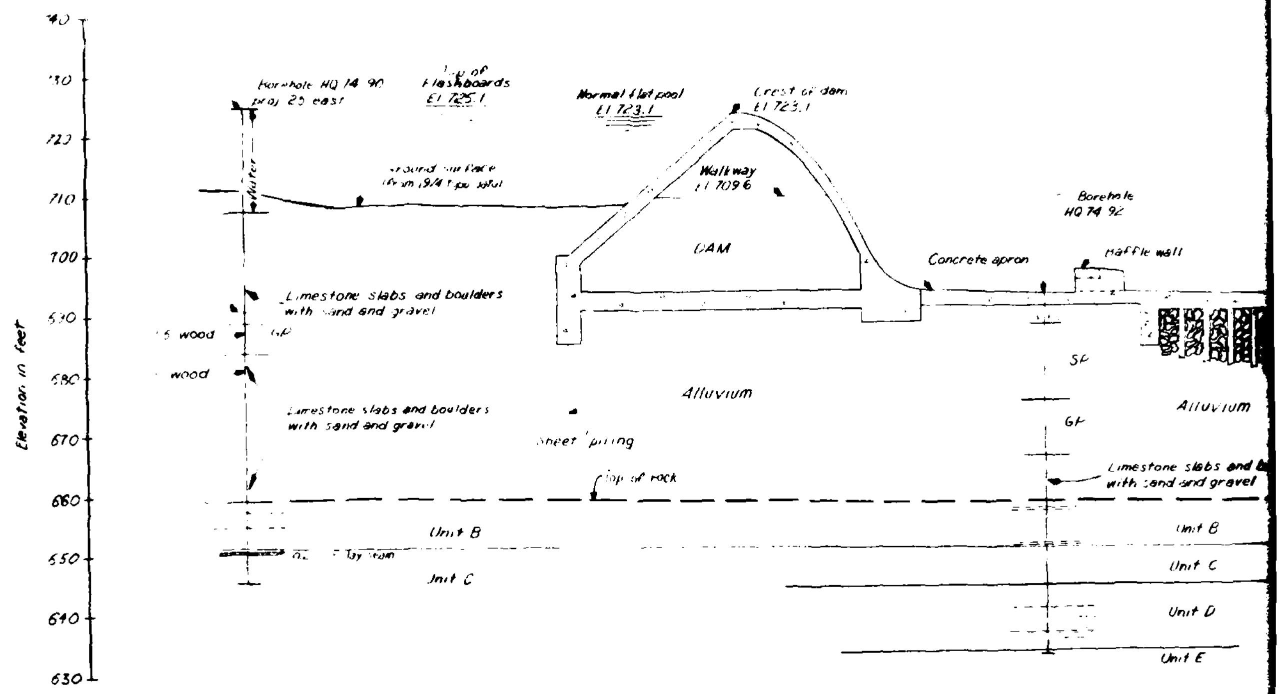
STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO 1
GEOLOGY AND FOUNDATION EXPLORATION
GEOLOGIC SECTIONS O-O AND P-P

DATE: MARCH 1975

DRAWING NUMBER

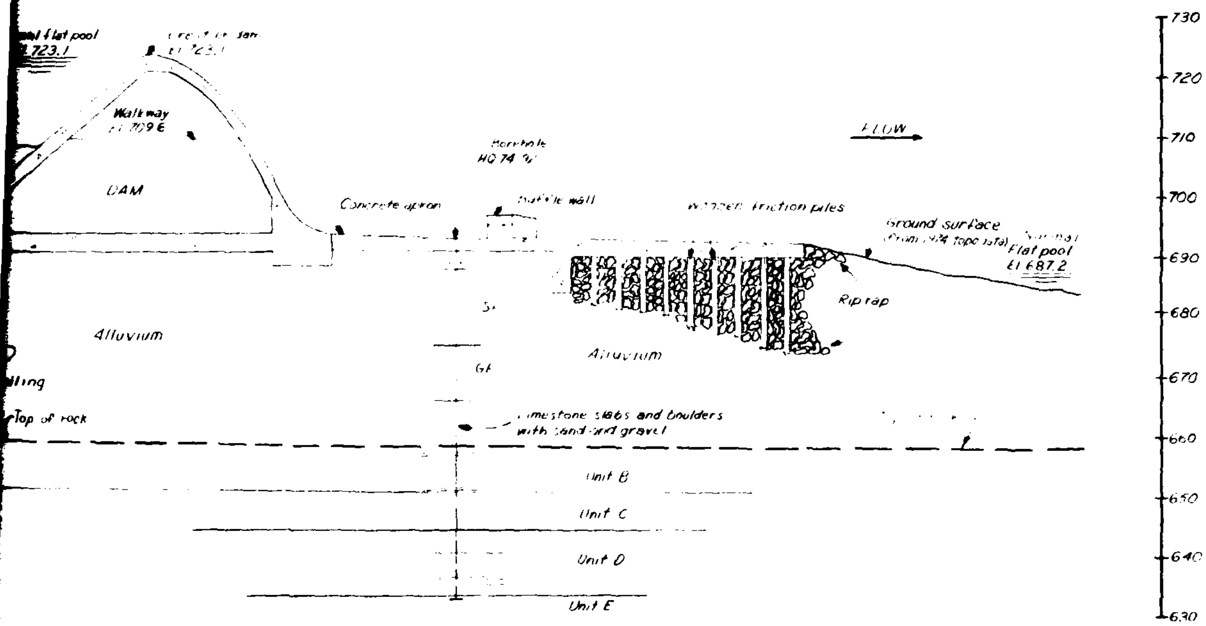
SHEET OF

PLATE B-1-12



Elevation in feet
Borehole HQ 74 90 proj 25 east
Top of Flashboards El 725.1
Normal flat pool El 723.1
Crest of dam El 723.1
Ground surface (from 1974 topo data)
Walkway El 709.6
Water
DAM
Borehole HQ 74 92
Concrete apron
Baffle wall
Limestone slabs and boulders with sand and gravel
wood
GP
SP
Alluvium
Sheet piling
Top of rock
Unit B
Unit C
Unit D
Unit E

SECTION LOCATION KEY



Horizontal & Vertical
Scale 0 10 20 Feet

For notes and explanations see Plate B 1-3.

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ST PAUL, MINNESOTA

R P K
R P K
M J S

STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO 1
GEOLOGY AND FOUNDATION EXPLORATION
GEOLOGIC SECTION Q-Q

SUBMITTED BY:
APPROVED:

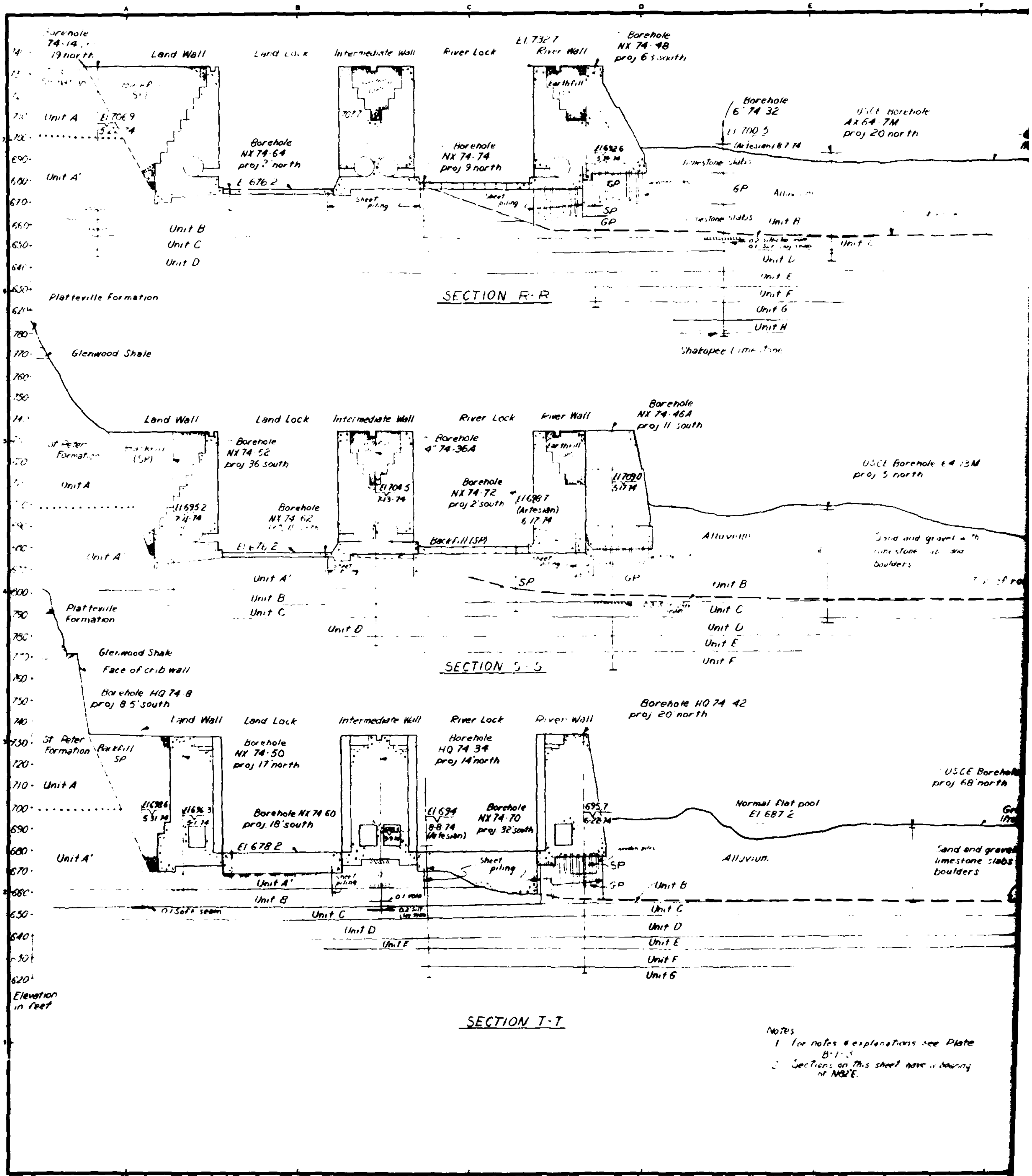
DATE:
MARCH 1975

DRAWING NUMBER

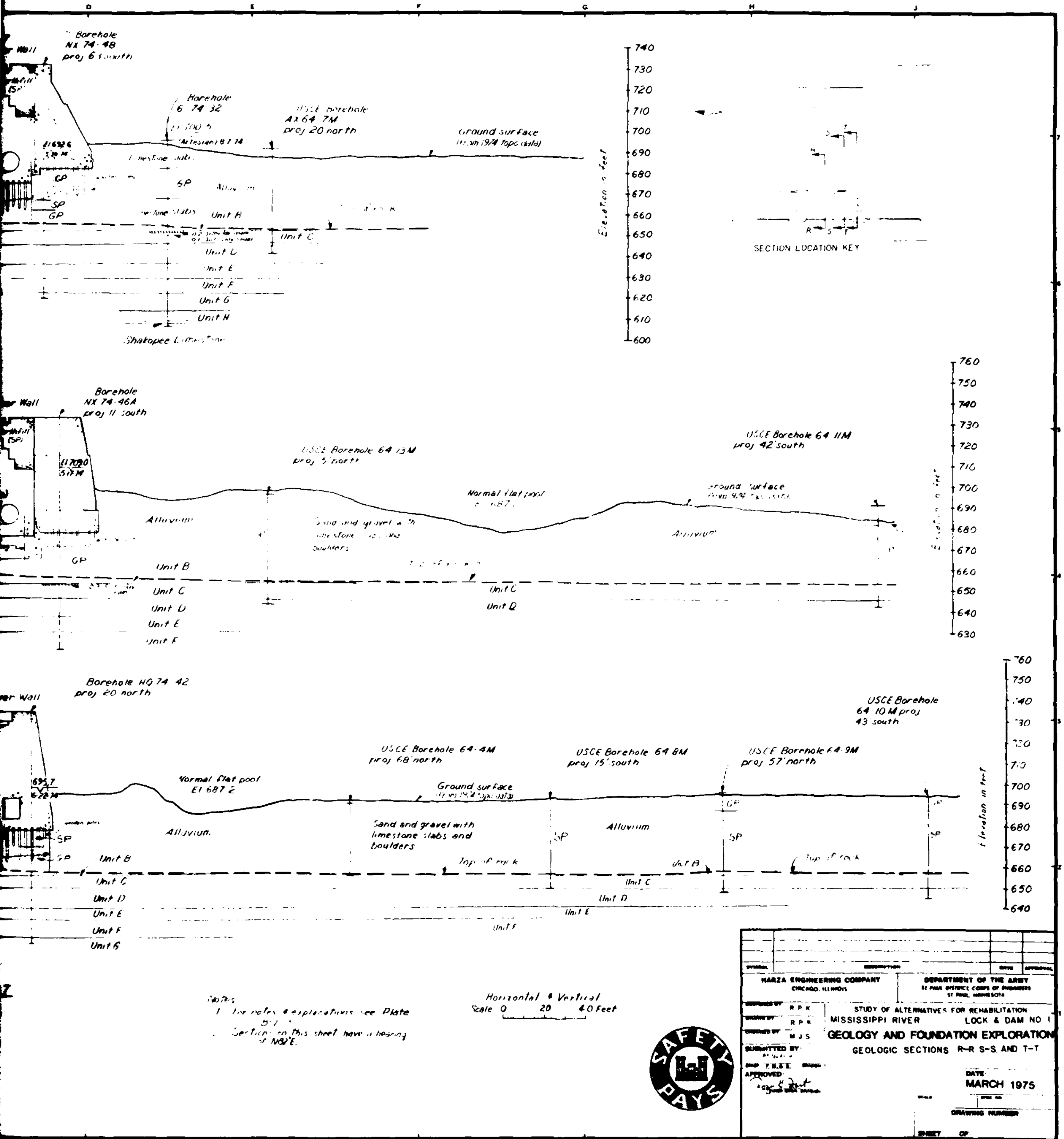
SHEET OF

SAFETY PAYS

PLATE B-1-13



Land Wall
Land Lock
Intermediate Wall
River Lock
River Wall
Borehole NX 74-48 proj 6.5 south
Borehole NX 74-64 proj 2 north
Borehole NX 74-74 proj 9 north
Unit A
Unit A'
Unit B
Unit C
Unit D
Platteville Formation
SECTION R-R
Unit E
Unit F
Unit G
Unit H
Shakopee Limestone
Glenwood Shale
Borehole NX 74-46A proj 11 south
Borehole NX 74-52 proj 36 south
Borehole NX 74-72 proj 2 south
USCE Borehole 64-13M proj 5 north
Alluvium
Backfill (SP)
Platteville Formation
SECTION S-S
Face of crib wall
Borehole HQ 74-8 proj 8.5 south
Borehole HQ 74-42 proj 20 north
Borehole NX 74-50 proj 17 north
Borehole HQ 74-34 proj 14 north
Borehole NX 74-60 proj 18 south
Borehole NX 74-70 proj 32 south
St Peter Formation
Backfill SP
Normal flat pool El 687.2
Sand and gravel limestone slabs boulders
Sheet piling
Elevation in feet
SECTION T-T
Notes
1. For notes & explanations see Plate B-1-3
2. Sections on this sheet have a bearing of N62°E.


Borehole NX 74-48 proj 6' south
Borehole 6 74 32
USCE Borehole AX 64-7M proj 20' north
Ground surface (from 1974 topo data)
Unit B
Unit C
Unit D
Unit E
Unit F
Unit G
Unit H
Shakopee Limestone
Elevation in feet
SECTION LOCATION KEY
Borehole NX 74-46A proj 11' south
USCE Borehole 64-13M proj 5' north
Normal flat pool
USCE Borehole 64-11M proj 42' south
Ground surface
Alluvium
Sand and gravel with limestone slabs and boulders
Borehole HQ 74-42 proj 20' north
USCE Borehole 64-10M proj 43' south
USCE Borehole 64-4M proj 68' north
USCE Borehole 64-8M proj 15' south
USCE Borehole 64-9M proj 57' north
Normal flat pool El 687.2
Ground surface (from 1974 topo data)
Top of rock
GP
SP
Notes
1. For notes & explanations see Plate B-1-1
2. Sections on this sheet have a bearing of N82°E.
Horizontal & Vertical Scale 0 20 40 Feet
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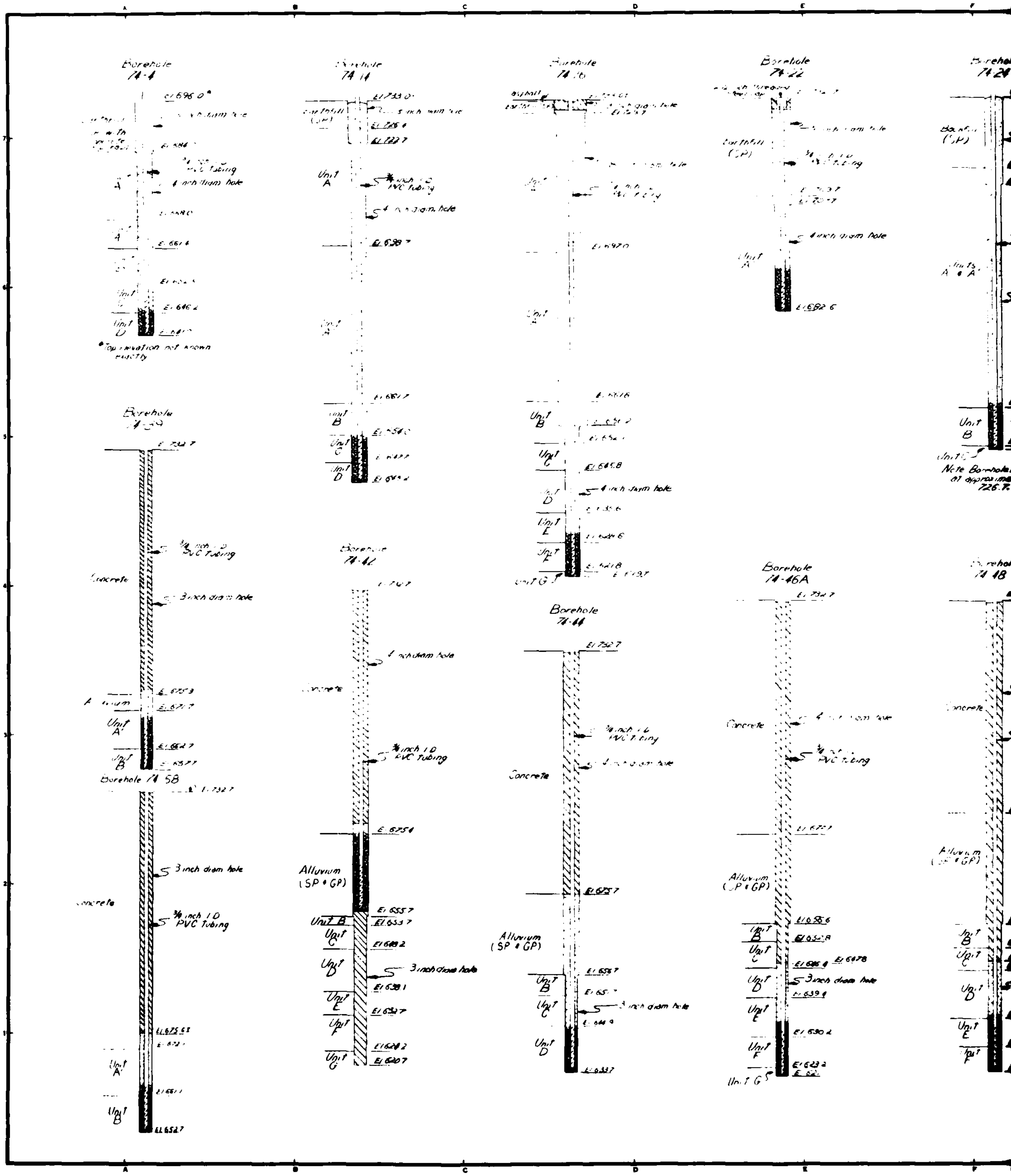
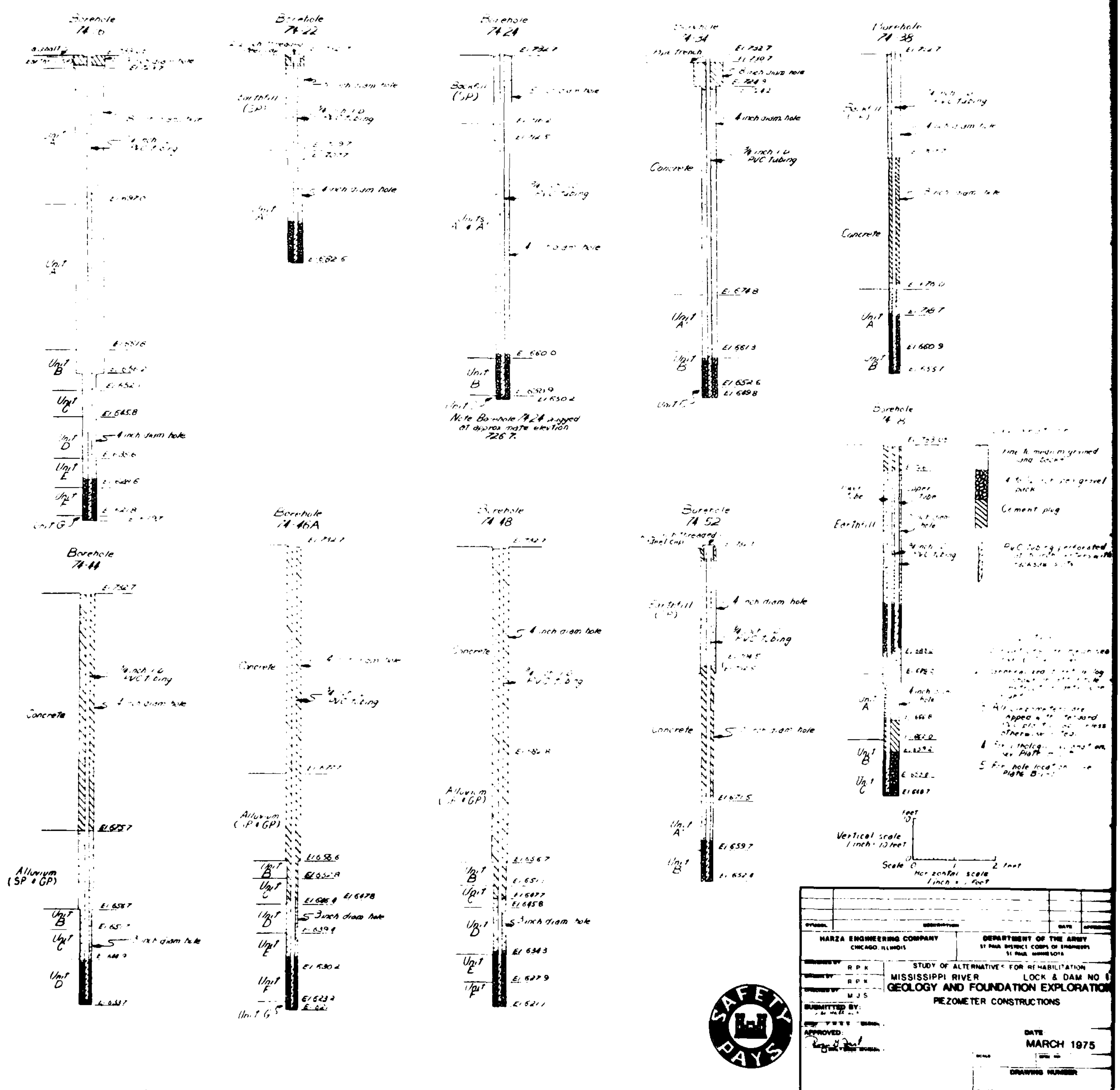
STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO. 1
GEOLOGY AND FOUNDATION EXPLORATION
GEOLOGIC SECTIONS R-R, S-S AND T-T
MARCH 1975
SAFETY PAYS

PLATE B-1-14


Borehole 74-4
*Top elevation not known exactly
Borehole 74-14
Borehole 74-16
Borehole 74-22
Borehole 74-39
Borehole 74-42
Borehole 74-44
Borehole 74-46A
Borehole 74-58
Concrete
Alluvium (SP + GP)
¾ inch I.D. PVC Tubing
3 inch diam hole
4 inch diam hole
Unit A
Unit A'
Unit B
Unit C
Unit D
Unit E
Unit F
Unit G
El. 652.7
El. 655.7
El. 653.7
El. 675.4
El. 692.6



Borehole 74-22
Borehole 74-24
Borehole 74-34
Borehole 74-38
Borehole 74-44
Borehole 74-46A
Borehole 74-48
Borehole 74-52
4 inch diam hole
3/4 inch I.D. PVC Tubing
Concrete
Alluvium (SP + GP)
Earthfill
Backfill (SP)
Unit A
Unit B
Unit C
Unit D
Unit E
Unit F
Unit G
Note: Borehole 74-24 logged at approximate elevation 726.7
Fine to medium grained sand backfill
Cement plug
Vertical scale 1 inch = 10 feet
Horizontal scale
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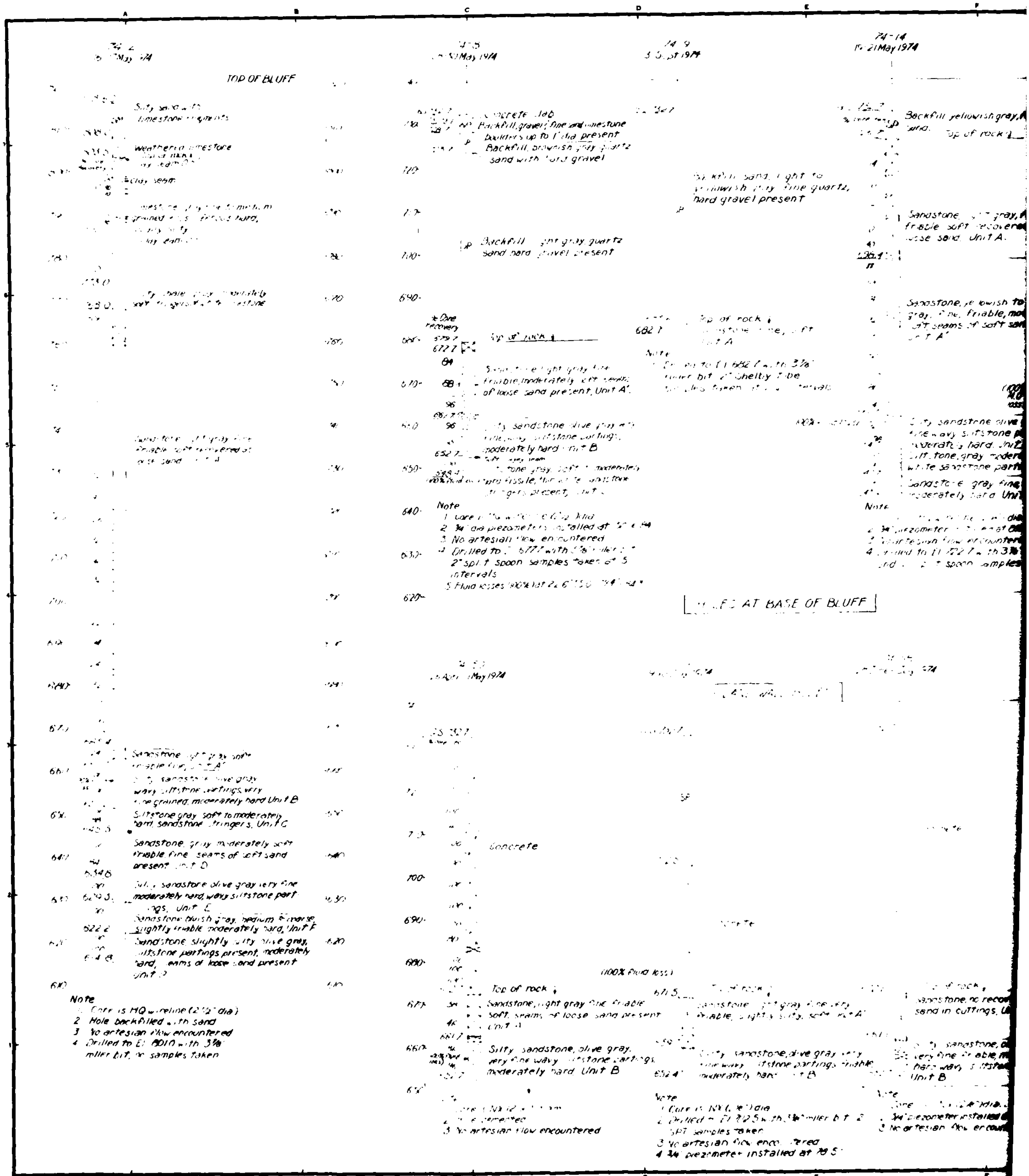
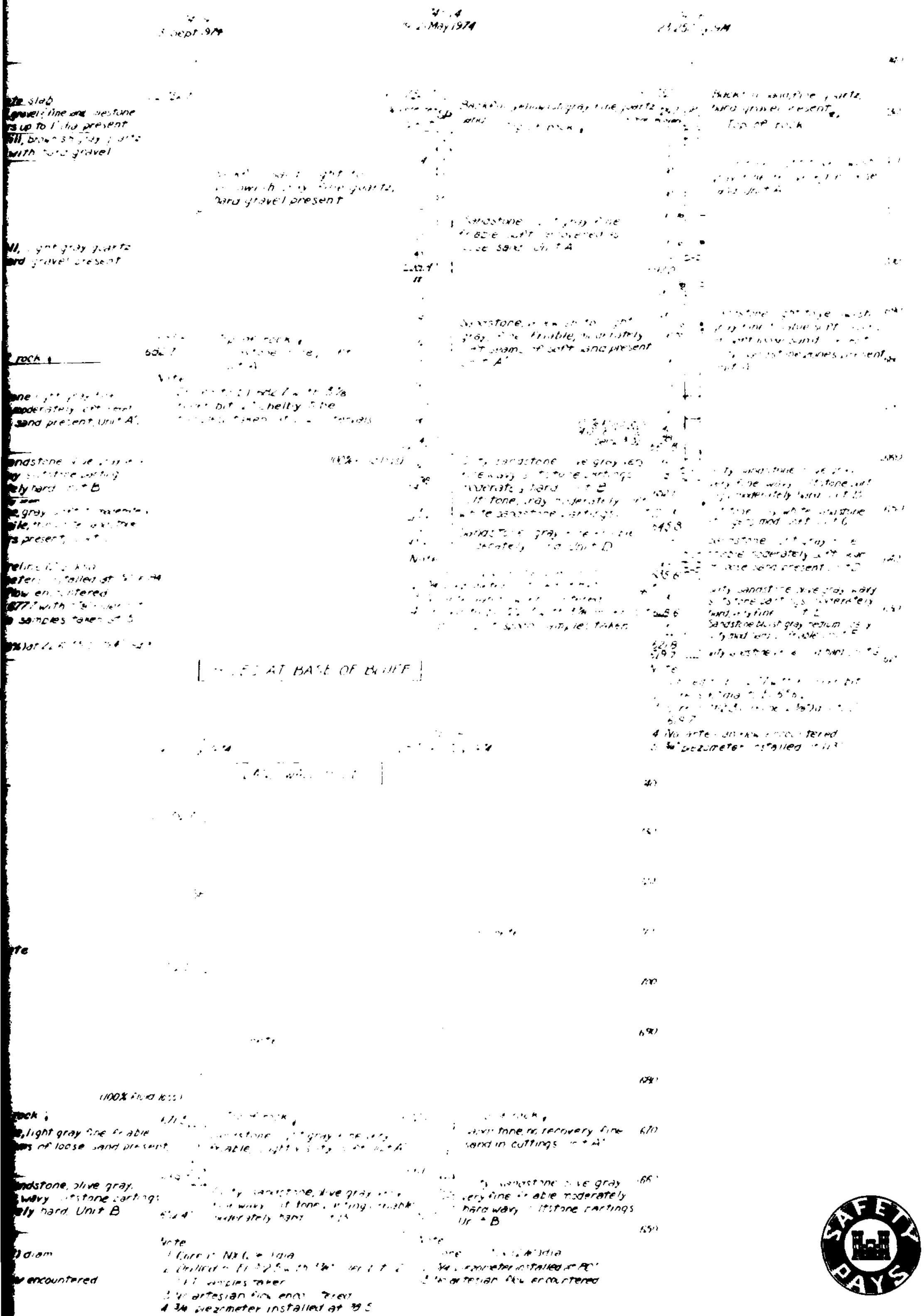
ST PAUL, MINNESOTA
STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO 1
GEOLOGY AND FOUNDATION EXPLORATION
PIEZOMETER CONSTRUCTIONS
SUBMITTED BY:
APPROVED:
DATE
MARCH 1975
DRAWING NUMBER
SAFETY PAYS

PLATE B-1


74-14
21 May 1974
TOP OF BLUFF
Backfill, gravel, fine and limestone boulders up to 1' dia present
sand with hard gravel
Backfill yellowish gray
Top of rock
Weathered limestone
hard gravel present
sand hard gravel present
Top of rock
of loose sand present, Unit A
moderately hard Unit B
Note
No artesian flow encountered
2" split spoon samples taken at 5' intervals
AT BASE OF BLUFF
Sandstone light gray soft friable fine, Unit A
Silty sandstone olive gray wavy siltstone partings very fine grained, moderately hard Unit B
Siltstone gray soft to moderately hard, sandstone stringers, Unit C
Sandstone, gray moderately soft friable fine, seams of soft sand present Unit D
Silty sandstone olive gray very fine moderately hard, wavy siltstone partings, Unit E
Sandstone bluish gray, medium to coarse, slightly friable moderately hard, Unit F
Sandstone slightly silty olive gray, siltstone partings present, moderately hard, seams of loose sand present
Note
1. Core is HQ wireline (2 1/2" dia)
2. Hole backfilled with sand
3. No artesian flow encountered
4. Drilled to El. 601.0 with 3 7/8" miller bit, no samples taken
Concrete
(100% fluid loss)
Top of rock
Sandstone, light gray fine friable soft, seams of loose sand present Unit A
Silty sandstone, olive gray, very fine wavy siltstone partings, moderately hard Unit B
3 No artesian flow encountered
3 No artesian flow encountered
4 3/4 piezometer installed at



SOILS LEGEND

Sand or gravelly sand, poorly graded

Gravel or sandy gravel, poorly graded

Silty sand or silty gravelly sand

Clayey sand or clayey gravelly sand

Silt, sandy or gravelly silt, low to med plasticity

Silty gravels, gravel sand silt mixtures

STRUCTURAL LEGEND

ROCK LEGEND

Elevation in feet	
740	Platteville Limestone (in 74-2 only)
730	Glenwood shale (in 74-2 only)
	St Peter Formation sandstone
720	Silty sandstone
	Siltstone
	Shakopee Dolomite (in 74-32 only)
	Core loss
	...
	Clayey seams in Unit C

Notes
2 Water level ... of drilling
3 Per cent core recovery shown ...
4 See drawing ... for hole locations
5 See drawing ... for detailed description of rock unit, and of unit designations A through ...
Identification of soils based on visual inspection of split spoon samples and by drilling resistance

SYMBOL	DESCRIPTION	DATE / APPROVAL

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

DEPARTMENT OF THE ARMY
ST PAUL DISTRICT CORPS OF ENGINEERS
ST PAUL, MINNESOTA

DESIGNED BY R P K
DRAWN BY R P K
CHECKED BY M J S
SUBMITTED BY
APPROVED

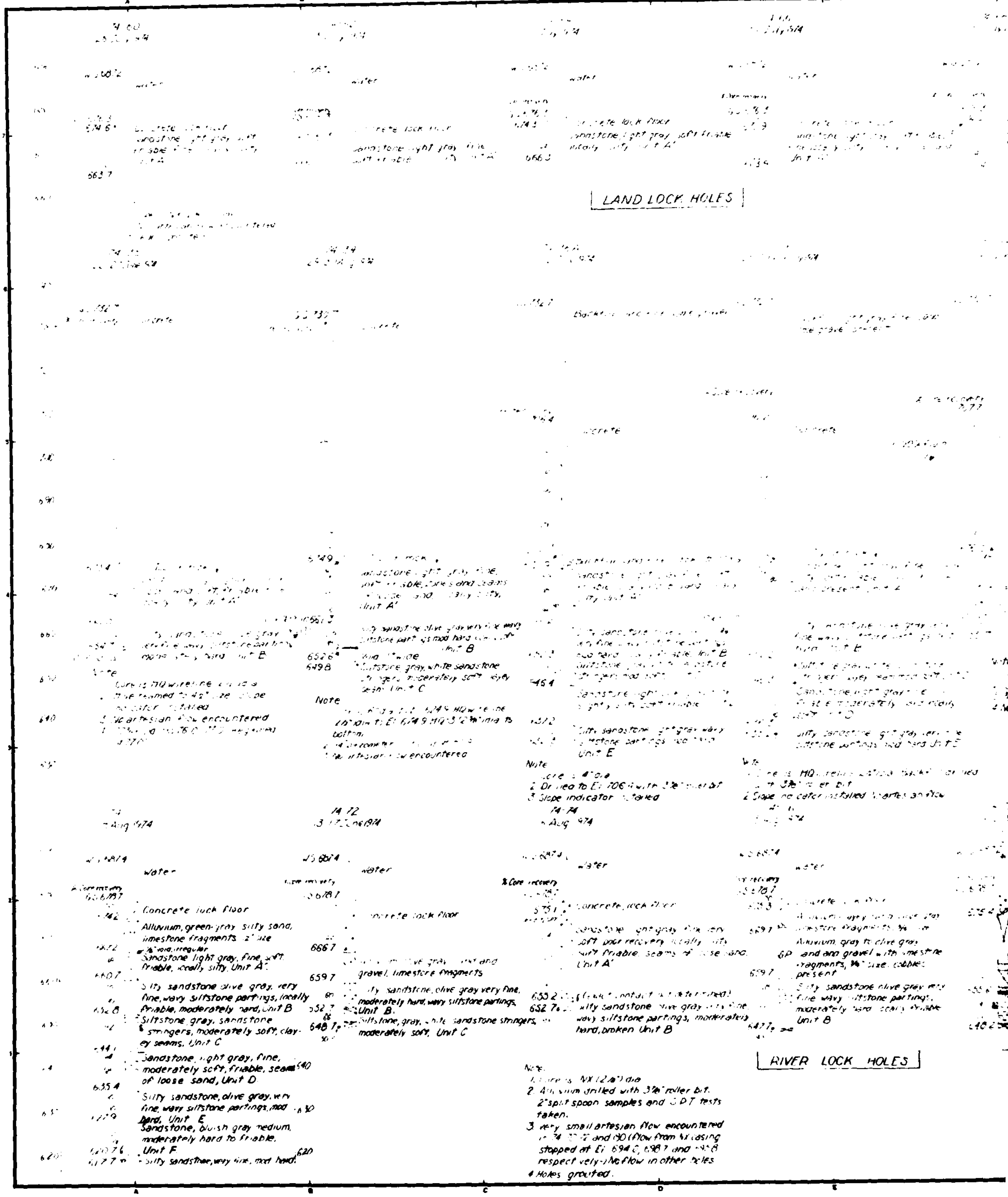
STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO 1
GEOLOGY AND FOUNDATION EXPLORATION
SUMMARY BOREHOLE LOGS
BLUFF BASE OF BLUFF LAND WALL

DATE
MARCH 1975

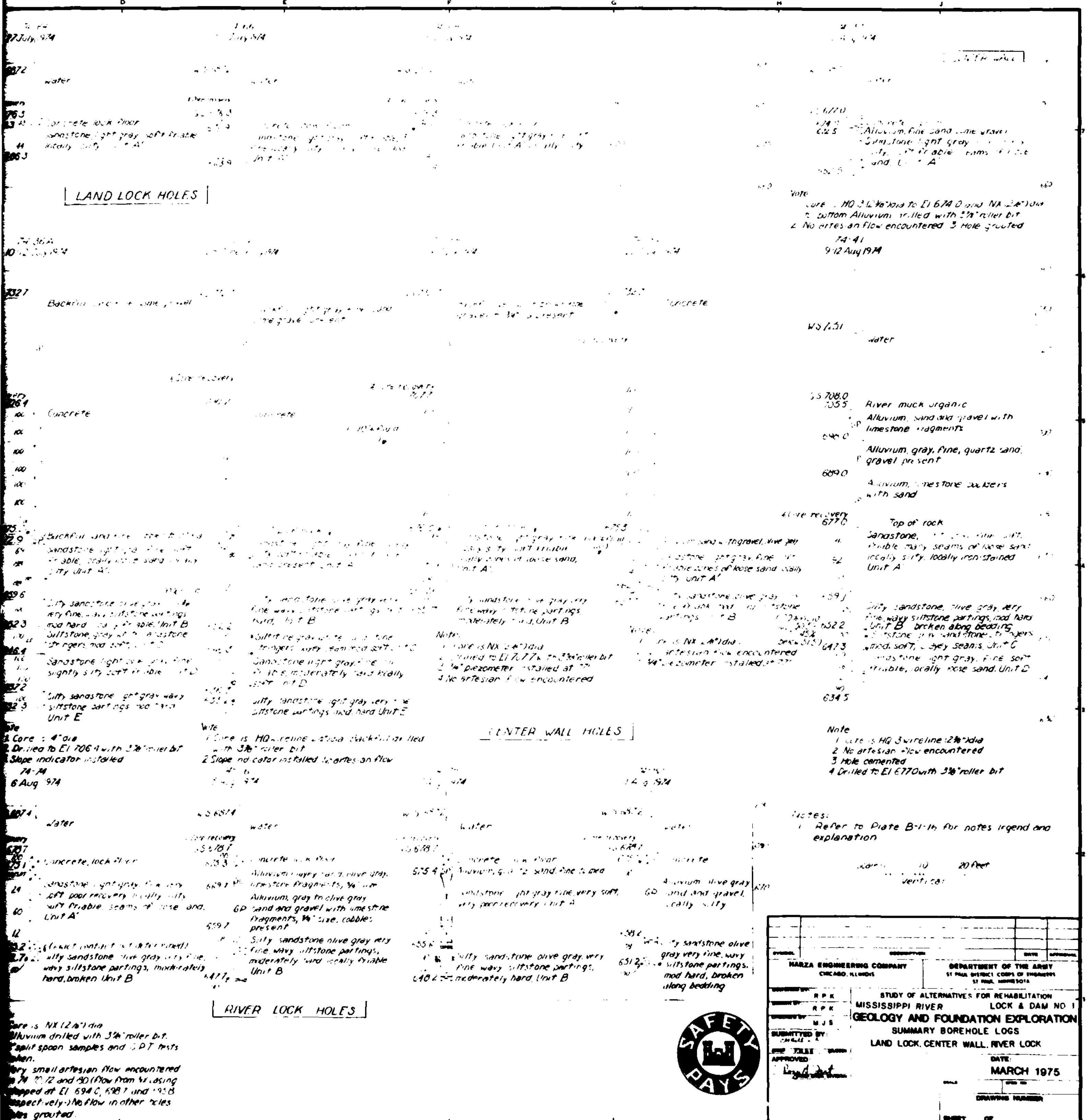
DRAWING NUMBER

SHEET OF

PLATE B-1-16



LAND LOCK HOLES
RIVER LOCK HOLES
Concrete lock floor
Alluvium, green-gray silty sand, limestone fragments
Sandstone light gray, fine, soft, friable, locally silty, Unit A
Silty sandstone olive gray, very fine, wavy siltstone partings, locally friable, moderately hard, Unit B
Siltstone gray, sandstone stringers, moderately soft, clayey seams, Unit C
Sandstone, light gray, fine, moderately soft, friable, seams of loose sand, Unit D
Silty sandstone, olive gray, very fine, wavy siltstone partings, mod hard, Unit E
Sandstone, bluish gray medium, moderately hard to friable, Unit F
Silty sandstone, very fine, mod hard
Silty sandstone, olive gray very fine, moderately hard, wavy siltstone partings, Unit B
Siltstone, gray, white sandstone stringers, moderately soft, Unit C
Silty sandstone olive gray very fine, wavy siltstone partings, moderately hard, broken Unit B
Silty sandstone olive gray very fine wavy siltstone partings, moderately hard, locally friable Unit B
Alluvium, gray to olive gray sand and gravel with limestone fragments, cobbles present
Note
2. Alluvium drilled with 3⅞" roller bit. 2" split spoon samples and SPT tests taken.
3. Very small artesian flow encountered
4. Holes grouted.
Note
3. Slope indicator installed
2. Slope indicator installed



LAND LOCK HOLES
CENTER WALL HOLES
RIVER LOCK HOLES
Alluvium, fine sand, some gravel
74-41
9-12 Aug 1974
River muck organic
Alluvium, sand and gravel with limestone fragments
Alluvium, gray, fine, quartz sand, gravel present
Top of rock
Note
2 No artesian flow encountered
3 Hole cemented
4 Drilled to El 677.0 with 3⅞" roller bit
Notes:
1 Refer to Plate B-1-16 for notes legend and explanation
Very small artesian flow encountered
HARZA ENGINEERING COMPANY
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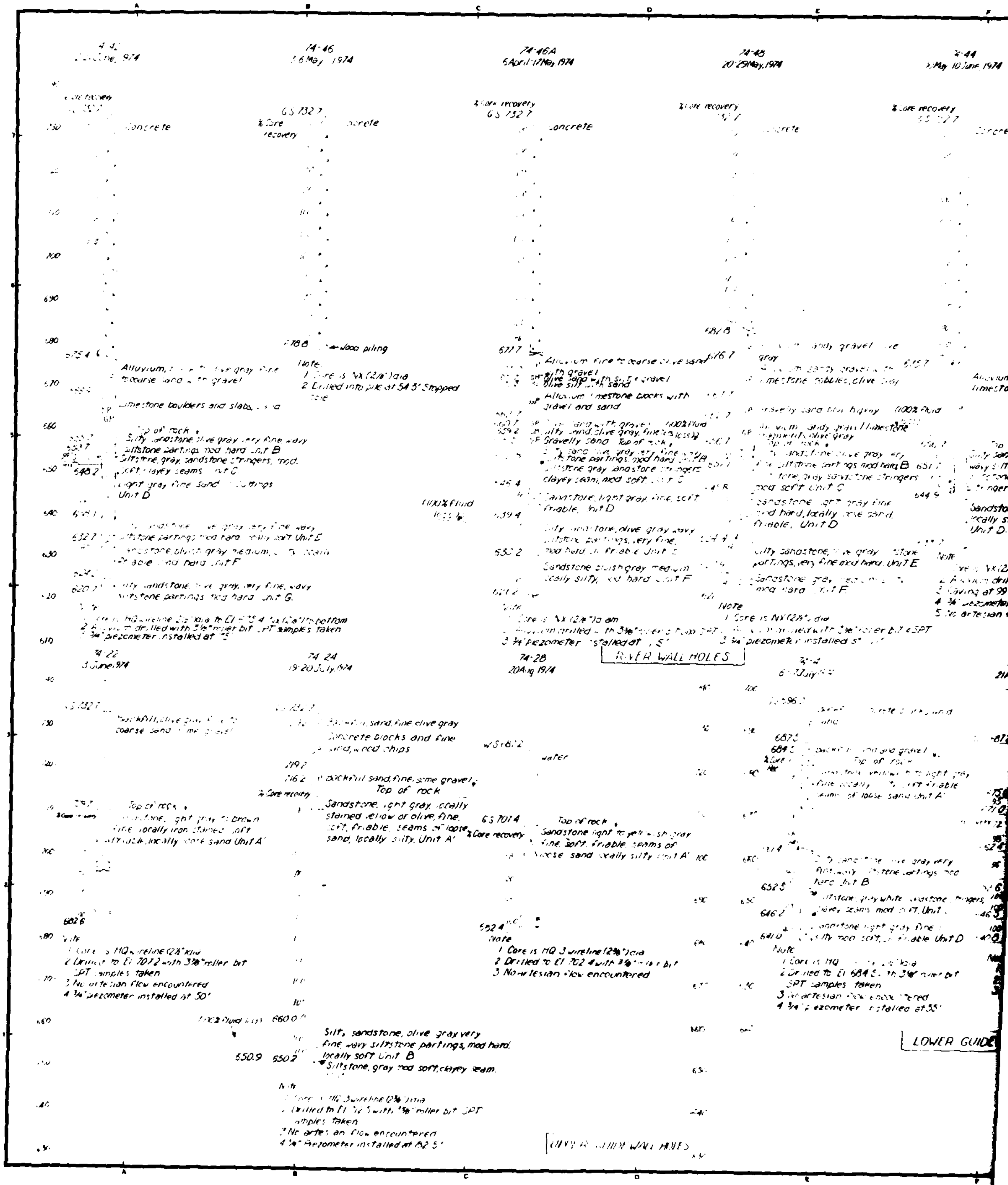
ST. PAUL DISTRICT CORPS OF ENGINEERS
ST. PAUL, MINNESOTA
STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO 1
GEOLOGY AND FOUNDATION EXPLORATION
SUMMARY BOREHOLE LOGS
LAND LOCK, CENTER WALL, RIVER LOCK
MARCH 1975
SAFETY PAYS

PLATE B-1-17



74-46
74-46A
74-45
74-44
Concrete
Wood piling
Note
1 Core is NX (2⅛") dia
2 Drilled into pile at 54.5' Stopped hole
Alluvium, fine to coarse olive sand with gravel
Alluvium, limestone blocks with gravel and sand
Top of rock
Silty sandstone, olive gray, very fine, wavy siltstone partings, mod hard, Unit B
Siltstone, gray, sandstone stringers, mod. soft, clayey seams, Unit C
Light gray fine sand, cuttings, Unit D
Sandstone, light gray, fine, soft, friable, Unit D
Sandstone, bluish gray, medium, locally silty, mod hard, Unit F
Silty sandstone, olive gray, very fine, wavy siltstone partings, mod hard, Unit G
3 ¾" piezometer installed at
RIVER WALL HOLES
74-22
3 June 1974
74-24
19-20 July 1974
74-28
20 Aug 1974
Backfill, olive gray, fine to coarse sand, some gravel
Backfill, sand, fine, olive gray
Concrete blocks and fine sand, wood chips
Backfill sand, fine, some gravel
Top of rock
Sandstone, light gray, locally stained yellow or olive, fine, soft, friable, seams of loose sand, locally silty, Unit A
% Core recovery
Water
Note
1 Core is HQ 3 wireline (2⅜") dia
2 Drilled to El 702.4 with 3⅞" roller bit
3 No artesian flow encountered
4 ¾" piezometer installed at 50'
Silt, sandstone, olive gray, very fine, wavy siltstone partings, mod hard, locally soft, Unit B
Siltstone, gray, mod soft, clayey seam
4 ¾" Piezometer installed at 62.5'
SPT samples taken
LOWER GUIDE
UPPER GUIDE WALL HOLES

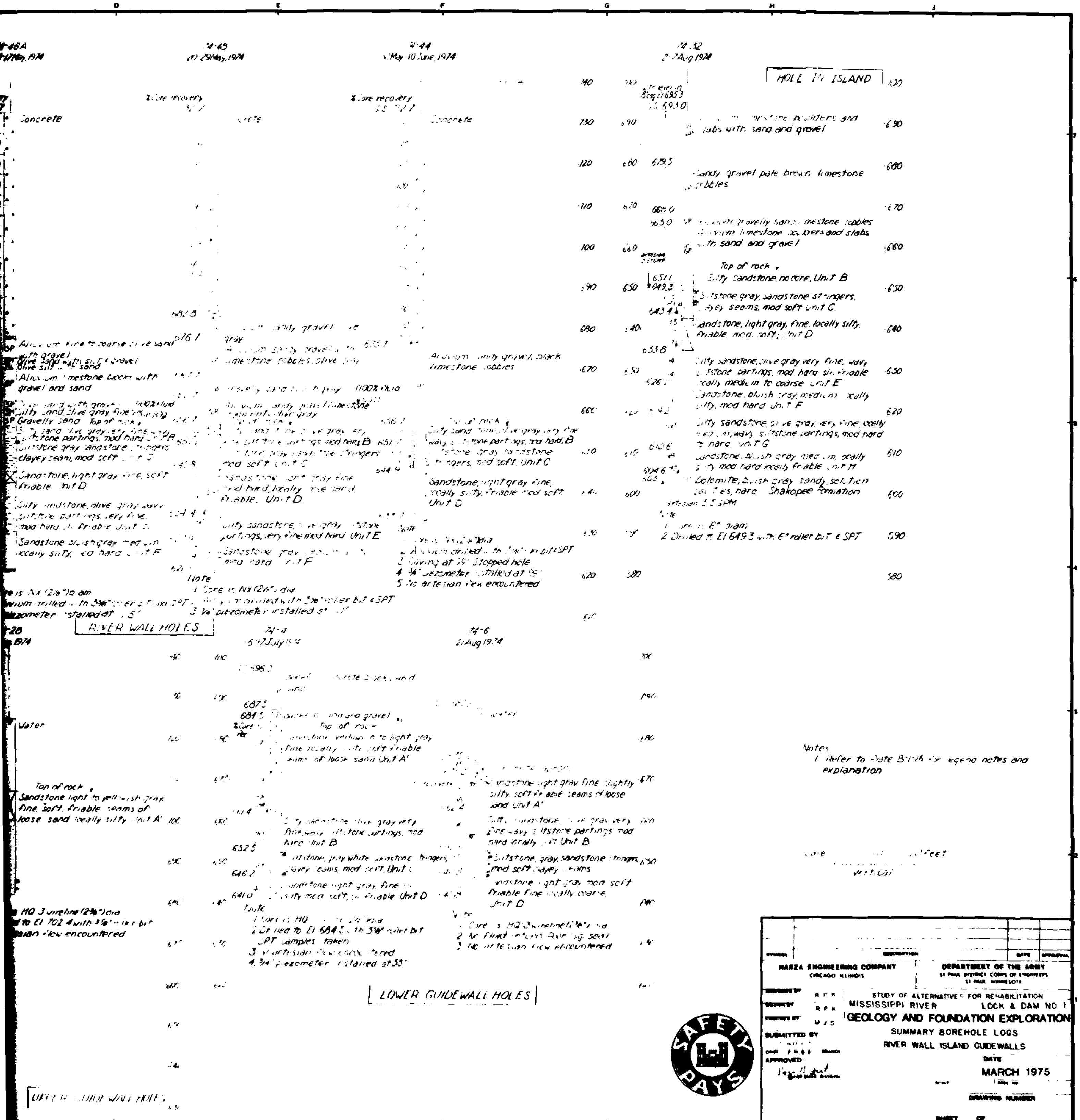
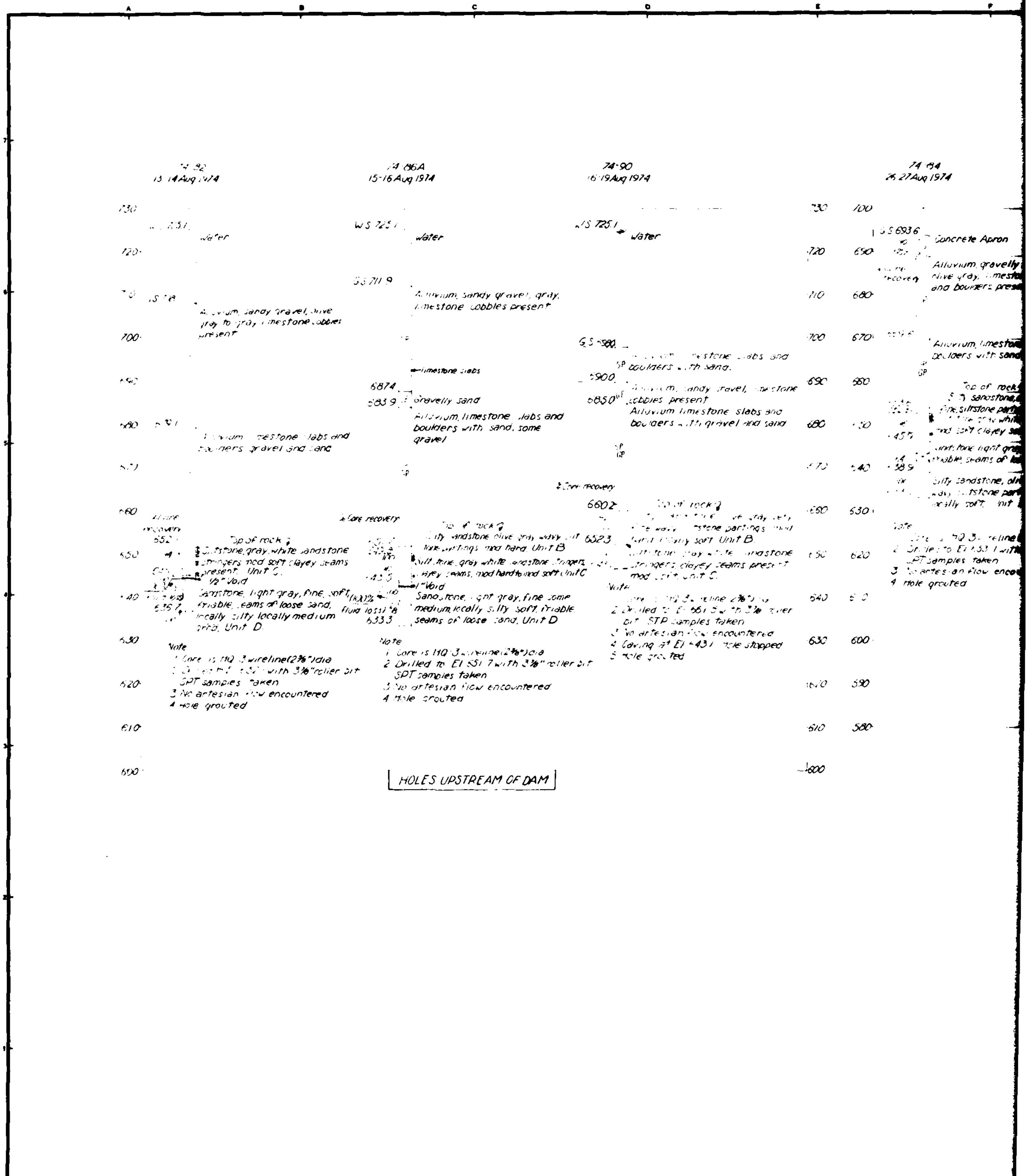
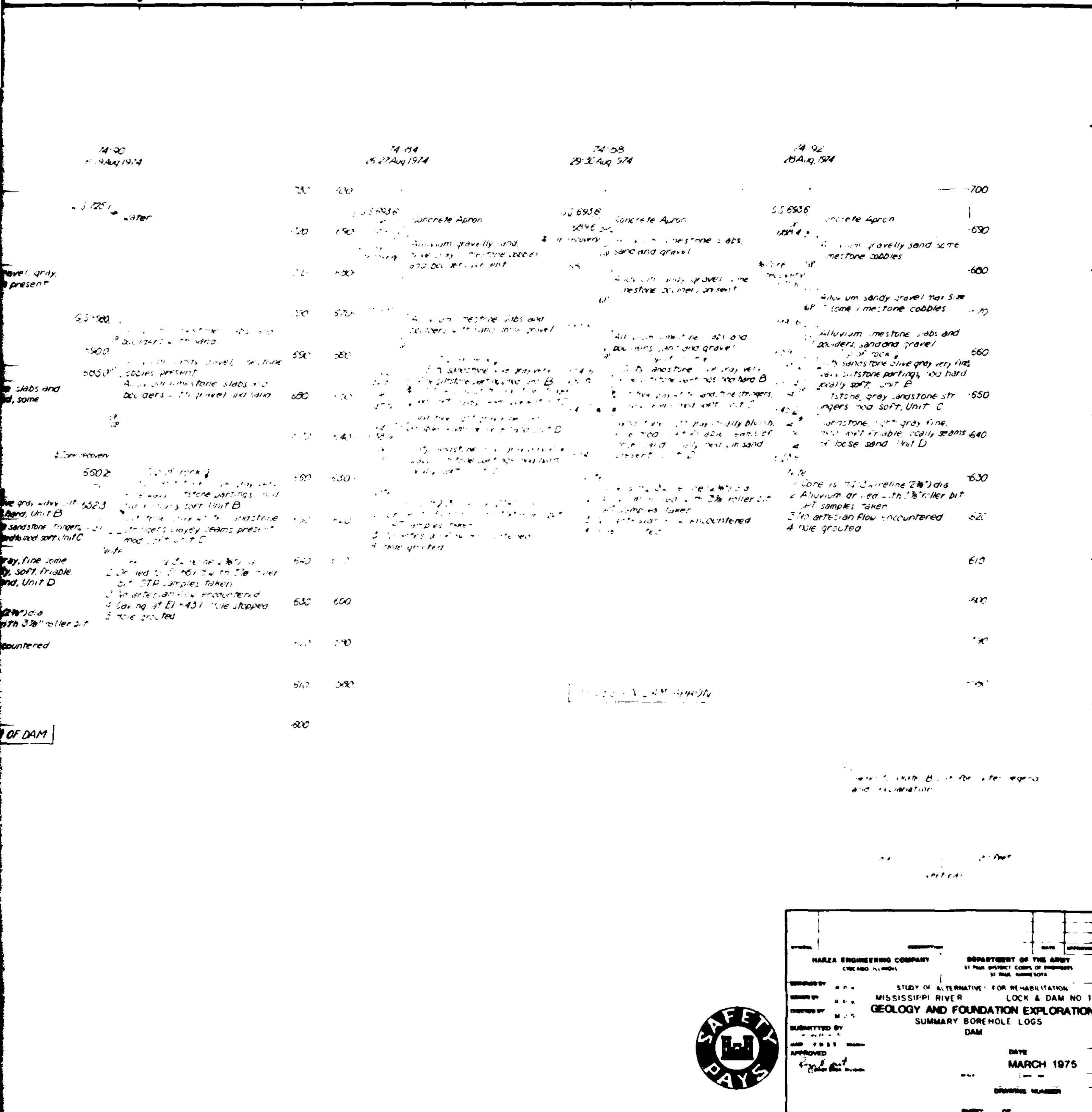

74-45
20-29 May 1974
74-44
May 10 June 1974
74-52
2-7 Aug 1974
HOLE IN ISLAND
Concrete
Sandy gravel pale brown limestone cobbles
Top of rock
Silty sandstone, no core, Unit B
Siltstone gray, sandstone stringers, clayey seams, mod soft Unit C
Sandstone, light gray, fine, locally silty, friable, mod. soft, Unit D
Silty sandstone, olive gray, very fine, wavy siltstone partings, mod hard
Sandstone, bluish gray, medium, locally silty, mod hard Unit F
Dolomite, bluish gray, sandy, solution cavities, hard. Shakopee Formation
2 Drilled to El 649.3 with 6" roller bit & SPT
Alluvium sandy gravel, black limestone cobbles
Sandstone, light gray, fine, locally silty, friable, mod soft, Unit D
Note
1 Core is NX (2⅛") dia
5 No artesian flow encountered
RIVER WALL HOLES
74-4
74-6
21 Aug 1974
Water
Top of rock
Sandstone light to yellowish gray, fine, soft, friable, seams of loose sand, locally silty, Unit A'
Siltstone, gray, sandstone stringers, mod soft, clayey seams
Sandstone light gray, mod soft, friable, fine, locally coarse, Unit D
3 No artesian flow encountered
4 ¾" piezometer installed at 55'
2 No fluid return during test
3 No artesian flow encountered
LOWER GUIDEWALL HOLES
UPPER GUIDEWALL HOLES
Notes
1. Refer to Plate B-1-16 for legend notes and explanation
HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS
DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT CORPS OF ENGINEERS
ST. PAUL, MINNESOTA
STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO 1
GEOLOGY AND FOUNDATION EXPLORATION
SUMMARY BOREHOLE LOGS
RIVER WALL ISLAND GUIDEWALLS
MARCH 1975
SAFETY PAYS

PLATE B-1-18



74-86A
15-16 Aug 1974
74-90
16-19 Aug 1974
74-84
26-27 Aug 1974
Water
Concrete Apron
Alluvium, sandy gravel, gray, limestone cobbles present
Limestone slabs
Gravelly sand
Alluvium, limestone slabs and boulders with sand, some gravel
Alluvium, sandy gravel, limestone cobbles present
Alluvium, limestone slabs and boulders with gravel and sand
Core recovery
Top of rock
Siltstone, gray, white sandstone stringers, mod soft clayey seams present. Unit C
1/2" Void
Sandstone, light gray, fine, soft, friable, seams of loose sand, locally silty, locally medium hard, Unit D
Note
1. Core is HQ 3 wireline (2⅜") dia
SPT samples taken
3. No artesian flow encountered
4. Hole grouted
1" Void
Sandstone, light gray, fine, some medium, locally silty, soft, friable, seams of loose sand, Unit D
2. Drilled to El 551.7 with 3⅞" roller bit
SPT samples taken
3. No artesian flow encountered
4. Hole grouted
4. Caving at El 643.1, hole stopped
5. Hole grouted
HOLES UPSTREAM OF DAM



74-90
9 Aug 1974
74-84
26-27 Aug 1974
74-58
29-30 Aug 1974
74-92
28 Aug 1974
Concrete Apron
Alluvium, gravelly sand
Alluvium, sandy gravel, limestone cobbles present
Alluvium, limestone slabs and boulders, sand and gravel
Sandstone, olive gray, very fine, with siltstone partings, mod hard, locally soft, Unit B
Sandstone, gray, sandstone stringers, mod soft, Unit C
Sandstone, light gray, fine, mod soft, friable, locally seams of loose sand, Unit D
Note
1 Core is NX wireline (2⅛") dia
2 Alluvium drilled with 3⅞" roller bit, SPT samples taken
3 No artesian flow encountered
4 Hole grouted
OF DAM
700
690
680
670
660
650
640
630
620
610
600
590
580
HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS
DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT, CORPS OF ENGINEERS
ST. PAUL, MINNESOTA
STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER
LOCK & DAM NO 1
GEOLOGY AND FOUNDATION EXPLORATION
SUMMARY BOREHOLE LOGS
DAM
SUBMITTED BY
APPROVED
DATE
MARCH 1975
DRAWING NUMBER
SAFETY PAYS

PLATE B-I-19

DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT, CORPS OF ENGINEERS
1210 U.S. Post Office & Custom House
St. Paul, Minnesota 55101

MISSISSIPPI RIVER

STUDY OF ALTERNATIVES FOR REHABILITATION OF LOCK AND DAM NO. 1

MINNEAPOLIS, MINNESOTA

APPENDIX B-2

GEOLOGY, FOUNDATIONS AND SOILS INVESTIGATION

GEOTECHNICAL TEST RESULTS AND PARAMETERS FOR ANALYSES

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

PLATES

TABLE OF CONTENTS (Continued)

PLATES

SUPPLEMENT A

Report Entitled:

"Results of Testing Program on Three HQ Core Samples From Unit C, St. Peter Formation, Minneapolis, Minnesota."

prepared for

Harza Engineering Company
Chicago, Illinois
by
Alberto S. Nieto
January 6, 1975

APPENDIX B-2

GEOTECHNICAL TEST RESULTS AND PARAMETERS FOR ANALYSES

INTRODUCTION

This appendix presents the results of the geotechnical laboratory testing program and recommendations for the geotechnical parameters to be used in the various analyses of the structures for existing conditions and for the construction conditions anticipated during rehabilitation.

The boring locations for the 1974 exploration program; the logs of the 1974 borings and the longitudinal and transverse sections showing the subsurface stratigraphy in relation to the structures are all shown in Appendix B-1, which deals with the Geology of the project site.

The samples available for testing purposes consisted of disturbed jar samples recovered from standard penetration tests; two inch diameter shelby tube samples from boring 74-9; unwaxed core samples from all the borings and selected waxed core samples from various borings and units of the geologic column. The extent of the testing program has, in part, been limited by the type and availability of samples of certain materials. Where undisturbed samples, representative of in-situ conditions, were not available values of index and physical properties have been estimated utilizing the available exploration data. Quantitative values of the index and physical properties were obtained for certain granular backfill materials and for foundation materials underlying the land and intermediate lock walls.

The nature and properties of the various materials are discussed subsequently in the following order. Backfill materials are treated first followed by the foundation materials in order of units as defined in the geologic report. The factual information is followed by recommendations of parameters which may be utilized in the various analyses. Certain geologic details, which are believed to be important with regard to the structural analyses, are noted and commented on for each unit discussed.

The geotechnical laboratory tests reported herein were carried out by Soil Testing Services, Inc., Northbrook, Illinois, 60062, as requested by Harza Engineering Company.

The direct shear tests on the thin silt-clay seams from the Unit C member of the St. Peter formation were carried out by Prof. A. S. Nieto of the Department of Geology, University of Illinois as requested by Harza Engineering Company. Professor Nieto's report on this testing program is included as Supplement A to this appendix.

BACKFILL MATERIALS

The backfill materials existing behind the land and guide-walls and along the top of the intermediate wall were explored by means of borings 74-4, 74-8, 74-9, 74-14, 74-16, 74-22, 74-24, 74-36, 74-36A, 74-38, 74-52.

Land Wall Backfill

Within the longitudinal limits of the land wall and below the pavement and topsoil cover, the backfill was found to consist of a compact to dense fine to coarse quartz sand with inclusions of pieces of sandstone, limestone boulders; pieces of wood; pieces of concrete and the occasional piece of steel. The upper 10 to 15 feet of the backfill appears to contain a greater concentration of limestone and sandstone boulders. The sand is believed to be derived from the St. Peter sandstone formation. A schematic stratigraphic section is shown on plate B-2-1.

Standard penetration test results versus depth for borings behind the land wall are plotted graphically on the above drawing. The comparatively large blow count values in the upper 10 to 15 feet of the backfill reflect the greater presence of coarse material and these values should not be taken as being fully representative of the true relative density of the granular backfill within this depth range.

Continuous shelby tube samples of the granular backfill were recovered from boring 74-9. Natural moisture contents and natural unit weights were determined in the laboratory for all samples recovered. In-situ dry and saturated unit weights were calculated for each sample as was the relative density. The above index properties are plotted graphically with depth on plate B-2-1. The specific gravity of the minus 1/4 inch quartz sand was found to vary between 2.62 and 2.66. A value of 2.65 was used in the calculations of index properties.

The individual sand samples from boring 74-9 were combined to form two larger samples; one representing a depth range of 5 through 27.5 feet; the other representing a depth range of 29 throuth 49 feet. The grain size distribution curves for these two samples are shown on plates B-2-2 and B-2-3 respectively together with the gradation of the minus 1/4 inch fraction of these samples. Photographs of the individual quartz sand grains are shown enlarged on plate B-2-4. Photograph A is representative of the total 1/4 inch fraction and photograph B represents the material passing the no. 100 sieve and that retained on the no. 200 sieve.

The maximum and minimum dry densities for the minus 1/4 inch materials, as determined in the laboratory, were as follows:

Boring	Material	Depth Range-ft.	Max. Dry * Density-pcf.	Min. Dry * Density-pcf.
74-9	Quartz sand	5-27.5	118.3	93.8
74-9	Quartz sand	29-49	119.2	96.5

The two combined minus 1/4 inch sand samples were utilized to carry out a series of consolidated-drained triaxial compression strength tests at different confining pressures and different relative densities. The results of these tests are plotted on plates B-2-5, B-2-6, and B-2-7 which show the relationship between the angle of internal friction and initial void ratio, initial relative density and initial dry unit weight respectively. The sample data and stress-strain curves for these drained tests are shown on plate B-2-8.

The above results were obtained from tests on samples from boring 74-9. A review of the logs and examination of available jar samples from other borings behind the land wall indicate that the backfill material elsewhere is similar for all practical purposes. Therefore, the results presented on plates B-2-5, B-2-6 and B-2-7 may be used to sensibly estimate the angle of internal friction of the backfill material for the determination of lateral earth pressures acting on the land wall. From the above mentioned data the following average parameters are recommended. For backfill above the water table a partially saturated unit weight of 115 pounds per cubic foot. For sands below the water table a saturated unit weight of 130 pounds per cubic foot and a submerged unit weight of 67.5 pounds per cubic foot. For an average relative density of 60 per cent an angle of internal friction of 38 degrees is recommended for the sand backfill.

Upper Guide Wall Backfill

Borings 74-22 and 74-24 revealed the following backfill materials behind the upper land guide wall.

Boring 74-22, behind the downstream half of the upper guide wall, revealed a backfill consisting of brown to grey fine to coarse sand with some gravel sizes and the occasional limestone fragments. A visual estimate indicated the samples recovered consisted of about 90 per cent sand sizes and 10 per cent gravel sizes with the gravel sizes being found to occur primarily in the upper ten feet of the boring. Four standard penetration test results gave values varying between 20 and 60 blows per foot.

Boring 74-24, near the upstream end of the upper guide wall revealed a backfill consisting of 3 feet of loose to compact fine to medium sand; 10 feet of concrete blocks, wood fragments and sand and 3 feet of grey compact to dense fine to medium sand overlying the St. Peter sandstone. One standard penetration test in the lower sand stratum gave a value of 34 blows per foot.

* Values determined in accordance with ASTM Method D2049.

The above two borings indicate that there is a significant variation in the nature of the backfill existing behind the upper land guide wall.

For purposes of analyses it is suggested that the backfill behind monoliths 1 through 7 be considered similar to that revealed in boring 74-22. The angle of internal friction and the unit weights of the granular backfill behind these monoliths may be taken to be equal to that recommended for the land wall backfill.

The backfill behind monoliths 8 through 13 of the upper guide wall should be considered similar to that revealed by boring 74-24. An angle of internal friction of 35 degrees is recommended for determining the magnitude of earth pressure acting on these monoliths. Saturated and partially saturated unit weights of 125 and 115 pounds per cubic foot respectively may be utilized for earth pressure calculations.

Lower Guide Wall Backfill

The backfill behind the lower land guide wall was explored by boring 74-4 which revealed the following conditions.

Concrete blocks and fragments with sand were encountered between a depth of zero and 8.5 feet. Below this depth there is a 3 foot thick stratum of gravelly fine to coarse sand overlying the St. Peter sandstone. No standard penetration tests were possible in this boring.

It is suggested that the backfill behind all the monoliths of the lower land guide wall be considered similar to that revealed in boring 74-4. An angle of internal friction of 35 degrees is recommended for determining earth pressure coefficients. Saturated and partially saturated unit weights of 125 and 115 pounds per cubic foot respectively may be utilized for earth pressure calculations.

Intermediate Wall Backfill

Borings along the intermediate lock wall indicate that the backfill at the top of the wall consists of a grey to brown loose to compact quartz sand with some gravel and the occasional fragment of concrete and steel.

For purposes of analyses a partially saturated unit weight of 115 pounds per cubic foot is recommended for this backfill.

River Wall Backfill

No borings penetrated the backfill at the top of and between the old and new river walls. It is believed that this backfill is a material similar in kind and character to that found along the top of the intermediate wall.

For purposes of analyses a partially saturated unit weight of 115 pounds per cubic foot is suggested together with an angle of internal friction of 32 degrees for the determination of earth pressure due to this backfill.

FOUNDATION MATERIALS

The types of foundation materials upon which the lock structures and the dam are founded are shown on the transverse and longitudinal stratigraphic sections of the geologic report. The land walls, the intermediate wall and the floor of the land lock bear directly upon the Unit A and A' members of the St. Peter sandstone. The river walls and the dam are founded on the Mississippi river alluvium as is the major portion of the floor of the river lock.

The discussion which follows describes the characteristics and properties of the foundation materials as revealed by the available information. Units A', B and C are described in that order followed by the river alluvium.

Units A & A'

The Unit A members of the St. Peter formation are a friable to well cemented quartz grained sandstone and are the strata upon which the land wall, the upper guide wall, the lower guide wall, the floor of the land lock and the intermediate wall are founded. The thickness of the Unit A' sandstone remaining beneath the main structures varies between 10 and 15 feet.

The boring logs, in a qualitative sense, report three distinct degrees of cementation for the Unit A' member. A degree of cementation which was so low that it was not possible to recover any core specimens. A somewhat higher degree of cementation where drilling produced intact cores measured in inches of depth between which there were horizons of uncemented sand also measured in inches of depth. The third degree of cementation is where intact core recovery was measured in feet with no significant horizons of uncemented sand. Of significance to the structural analyses is the fact that a minimum degree of cementation is reported to occur in the Unit A' sandstone to significant depths immediately below the bottoms of the structures founded on this member. This indicates that the land and intermediate walls do not bear directly upon highly cemented sandstone but on sandstone or sand having very little if any cementation. Hence, the behavior of this unit of the St. Peter sandstone, for stability analyses, should be considered as being that of a sand rather than that of a well indurated or cemented sandstone.

It was not possible to determine the in-situ or in-place index and physical properties of the very poorly cemented sandstone because of the lack of any samples from immediately beneath the structures. However, upon completion of triaxial tests on waxed core samples of the Unit A' sandstone, the sand remaining was utilized to carry out a series of strength tests at different relative densities and confining pressures to provide realistic values of parameters for use in the stability analyses of walls founded upon this unit. The following results were obtained. The results of tests on waxed core samples are described subsequently.

Core samples from boring 74-36A, between a depth range of 60.1 and 66.4 feet, were broken down into sand and combined to form a sample having the grain size distribution shown on plate B-2-9. It is noted that some 20 per cent of this sample passes the number 200 sieve. An examination of the material passing the number 200 sieve under a microscope indicates that the fines consist of pure quartz grains. Photographs of magnified individual quartz sand grains are shown on plate B-2-10. The specific gravity of the combined sample of quartz sand was 2.66. The maximum and minimum dry densities for the sand were 114 and 84 pounds per cubic feet respectively.

The results of the consolidated-drained triaxial compression tests on the sand are shown on plates B-2-11, B-2-12 and B-2-13 which give the relationships between the angle of internal friction and initial void ratio, initial relative density and initial dry unit weight respectively. The sample data and stress-strain curves for these tests are shown on plate B-2-14.

A visual examination of available core samples of the Unit A' sandstone from other borings which penetrated beneath the land and intermediate walls indicates that the gradation of the sand or sandstone is, within practical limits, the same. It is therefore, suggested that the results presented on plates B-2-11, B-2-12 and B-2-13 be utilized to analyse the stability of the land and intermediate walls and monoliths 1 through 7 of the upper guide wall.

An angle of internal friction of 37 degrees and a cohesion of zero are recommended for use in determining the ultimate bearing capacity of the sandstone beneath the land and intermediate walls and beneath monoliths 1 through 7 of the upper guide wall. The bearing capacity calculations should utilize relationships which consider the inclination of the resultant load acting at foundation level. A saturated unit weight of 132.5 pounds per cubic foot may be used to determine the submerged unit weight of the sandstone to be used in the bearing capacity relationships. The above values correspond to a relative density of approximately 90 per cent on the argument that though the degree of cementation has been reduced to a negligible amount the in-situ relative density of the foundation materials has decreased only a minor amount.

Monoliths 8 through 13 of the upper guide wall and all the monoliths of the lower guide wall are supported on rock filled timber cribs which bear on the Unit A & A' members respectively. The actual foundation stresses acting along the individual timbers bearing on the sandstone will be greater than those for concrete monoliths. In determining the bearing capacity factors of safety for monoliths supported on timber cribs the magnitude of such stress concentrations should be used with due consideration being given to the minimum depths of cover which exist.

It is difficult to assess the effects which the construction of the cribs may have had upon the character of the sandstone and therefore, difficult to recommend parameters for use in determining ultimate bearing capacity beneath the cribs. It is suggested that two conditions be analysed. An upper boundary condition utilizing the parameters given for the land wall. A lower boundary condition utilizing an angle of internal friction of 32 degrees and a cohesion of zero.

For sliding stability analyses along horizontal planes beneath the land and intermediate walls and monoliths 1 through 7 of the upper guide wall an angle of internal friction of 32 degrees and a cohesion of zero are recommended.

The actual mode of sliding behavior of timber cribs resting on poorly cemented sandstone or sand is a complex one. As a first approximation, it is suggested that sliding stability of monoliths supported on timber cribs be analysed in a manner similar to the land and intermediate walls using the same parameters.

The subsequent test results are those obtained from laboratory tests on samples of the Unit A & A' sandstone having a significant degree of cementation. Waxed core samples of this unit, suitable for triaxial compressive strength tests, were available from borings 74-16 and 74-36A. The samples from the former boring are from above the base of the land wall and behind it. The samples from the latter boring are from beneath the plan area of the downstream half of the intermediate wall. No samples of the Unit A' sandstone were available from beneath the river walls as this member has been eroded away in this area.

Index Properties - Units A & A'

St. Peter Sandstone

Boring or Location	Sample Depth ft.	Sample Elevation	Dry Unit Weight pcf.	Initial Void Ratio	Horizontal Permeability cm/sec.
74-16	13.9-15.1	718.7-717.6	119.6	0.390	
74-16	38.0-39.2	694.7-693.6	118.6	0.405	
74-16	40.7-41.8	692.0-690.9	113.2	0.466	
74-36A	60.1-60.4	672.6-672.3	109.5	0.520	1.3×10^{-4}
74-36A	62.0-62.3	670.7-670.4	110.0	0.505	1.1×10^{-4}
74-36A	64.4-65.1	668.3-667.6	114.0	0.449	
74-36A	66.0-66.4	666.7-666.3	113.9	0.451	
74-36A	70.8-71.4	661.9-661.3	116.9	0.413	
*Ford Mine Block Samples	1001	710±	114.4	0.445	
	1002		116.9	0.413	
	1003		113.2	0.466	
	1004		117.4	0.408	
	1005		114.2	0.447	
	1006		113.8	0.453	
	1007		115.5	0.431	
	1008		114.4	0.445	

*From report entitled "Mississippi River, St. Anthony Falls Project, Minneapolis, Minnesota, Design Memorandum No. 3, Upper Lock, Part UL-3, Foundations and Geology, Appendix A, September, 1958." The test results are derived from work carried out in 1938 at Harvard University.

The grain size distribution curves for samples from boring 74-36A are shown on plates B-2-15, B-2-16, B-2-17, B-2-18 and B-2-19. A typical grain size curve for the sand from the Ford mine is shown on plate B-2-20.

In a review of the above results it is noted that the dry unit weights of the cemented Units A & A' sandstone vary between 109.5 and 119.6 pounds per cubic foot. This variation in dry unit weights suggests that the strength parameters of the sandstone will vary also. To observe the nature of this variation the results of the 1974 consolidated drained compressive strength tests and the 1938 Harvard tests are plotted on plates B-2-21 and B-2-22 respectively, as values of $\frac{\sigma_1 - \sigma_3}{2}$ versus $\frac{\sigma_1 + \sigma_3}{2}$ for approximately equal values of dry unit weight and maximum deviator stress conditions. The resulting K_f-lines are utilized to determine the angle of internal friction and cohesion for the sandstone utilizing the relationships shown on the above exhibits. The sample data and stress-strain curves for the 1974 tests are given on exhibit C-2-23. The following strength parameters were obtained for the Units A & A' sandstone.

Member	Dry Unit Weight pcf.	Strain Max. Deviator Stress - %	c' k_g/cm^2	Ø' degrees	Remarks
Unit A'	113 - 114	1 - 2	12.2	46.2	Borings
Unit A'	118.5-119.5	1 - 2	24.7	47.3	74-16- 74-36A
Unit A	114.5±	0.5 - 1.5	0.48	58.5	Block Samples
Unit A	117±	0.5 - 1.0	2.86	58.5	from Ford mine

The strength parameters for the block samples from the Ford mine are the results of tests where the axial load was cycled one or more times prior to failure. Under cyclic loading conditions a reduction or a breakdown of the cementation and an increase in the relative density would be expected to occur. This method of testing is believed to explain the difference in the strength parameters between the 1938 and the 1974 tests which were non-cyclic.

The above test results show that the greater the dry unit weight the greater is the value of the cohesion with a much lesser variation in the angle of internal friction with dry density.

Unit B

The Unit B member of the St. Peter formation is a cemented silty sandstone to sandy siltstone grading in that order from top to bottom. Locally it is much less cemented and displays an unusual structural feature. The thickness of this stratum varies between 7 and 10 feet except beneath the river walls and the dam where it has been severely eroded away.

The unusual structural feature which the boring logs report is that nearly 65 per cent of all the drilling fluid losses reported occur within this stratum. The median elevation of these fluid losses is about 656.5 which is approximately the mid-height of the stratum. In boring 74-34 a one inch void was reported at approximately this same elevation. This evidence indicates the presence of a more pervious horizon in an otherwise much less pervious stratum. It is suggested that this structural feature of the Unit B member is a zone along which horizontal movements have occured in past geologic history. The processes causing such a movement would be similar to those causing the formation of the soft seams within the Unit C member as discussed by Prof. Nieto in his report.

Triaxial compressive strength tests were carried out on waxed core samples of the Unit B member from borings 74-4 and 74-16. Consolidated drained tests were conducted on samples from boring 74-4 behind the lower land guide wall, whereas unconsolidated-undrained tests were carried out on samples from boring 74-16 behind the land wall. The results of the drained tests are shown on plate B-2-24 in a manner similar to that previously described. The sample data and stress-strain curves for these tests are shown on plate B-2-25. The results of the undrained triaxial tests are shown on plate B-2-26 as confining pressure versus compressive strength. The sample data and stress-strain curves for the undrained tests are shown on plate B-2-27.

The samples utilized for the above tests came from an elevation range of 654.6 and 652.2, or below the structural feature noted above. The dry unit weights of the samples vary between 120.2 and 131.5 pounds per cubic foot and increase with the depth of the sample. The consolidated-drained triaxial tests on samples from boring 74-4 indicate an angle of internal friction of 52 degrees and a cohesion ordinate of 31.6 kg/cm^2 for dry unit weights which are approximately equal. As for the Unit A sandstones there appears to be a tendency for the cohesion of the Unit B member to vary with the dry unit weight of the material. However, the number of test results are rather limited.

Unit C

The Unit C member of the St. Peter formation is a laminated siltstone varying in thickness between 5 and 7 feet. This member contains horizontal seams of soft to firm silt-clay near the top and bottom limits of the stratum. The soft seams were found to vary in thickness between 0.5 inches and 3 inches. The results of the 1974 exploration indicate that the seams are a continuous geologic feature beneath the structures at this site.

The laboratory testing program on the silt-clay and siltstone samples from Unit C was carried out by Prof. A. S. Nieto, University of Illinois, Urbana, Illinois as requested by Harza Engineering Company. The details of the testing procedures and the test results are given in Supplement A to this appendix.

The samples of silt-clay seams available for testing were disturbed samples with the result that it was not possible to obtain data on the in-situ strength of the silt-clay seams. The reason for the lack of undisturbed samples was due to the fact that blocking of the core bit was common while penetrating this member with the result that the core was caused to spin at or within the silt-clay seam. Where intact core was recovered, removal from the core barrel resulted in breaks occuring along the top or bottom of the seam.

In view of the above the testing program was directed toward determining the residual strength characteristics of the silt-clay seams. Samples of the silt-clay and siltstone from borings 74-16, 74-34 and 74-88 were utilized and tested in direct shear under drained conditions. The mineralogical composition of the silt-clay and the siltstone was determined by means of x-ray diffraction and differential thermal analysis methods. The predominant active clay minerals in the silt-clay seams and in the siltstone were found to be kaolinite and illite. The Atterberg limits and the grain size distributions of silt-clay seam samples were also determined. These latter results are given in the summary table below and Appendix B of Supplement A respectively.

The results of the drained direct shear tests for the silt-clay seams are presented below as maximum and minimum values of the angle of internal friction. The maximum value is that equal to the peak shear force on the shear force displacement diagram for the remolded sample. The minimum value is that equal to the residual shear force for the remolded sample at a large horizontal displacement. Shear along surfaces of intact siltstone did not display any maximum or minimum values for the samples tested.

Summary of Direct Shear Tests
Remolded Silt-Clay Seams and Siltstone Samples
Unit C - St. Peter Sandstone

Sample No.	Sample Depth	Sample Elevation	Normal Stress Peak (psi)	Normal Stress Residual (psi)	ϕ_p (1) degrees	ϕ_r (1) degrees	Remarks
SR-1 74-34 CL-1			15.4	18.0	24.1	22.6	"Clayey" portion of seam-
(LL = 46.9, PI = 27.6)	78.8'	653.9	30.1	37.5	22.7	21.8	test run on 1/8 in. layer
			62.1	75.6	20.7	19.8	
SR-1 74-34 CL-2			14.9	16.7	24.5	23.6	"Silty" portion of seam-
(LL = 40.4, PI = 23.1)	to	to	30.2	35.5	23.9	22.9	test run on 1/8 in layer
			62.1	76.9	23.8	22.7	
SR-1 74-34 LR			31.5	31.5	24.1	24.1	Siltstone lapped with
			64.4	(2)	37.6	(2)	#80 grit
SR-1 74-34 SR	79.5'	653.2	31.5	31.5	21.2	21.2	Induced fracture in siltstone
SR-1 74-34 GR			18.9	22.7	28.2	22.3	Ground rock, passing #80
(LL = 39.7, PI = 18.8)			50.1	58.0	25.9	21.4	sieve--1/8 in. layer
S-7 74-16	81.1'-	651.6-	19.6	25.3 (3)	24.2	18.5 (3)	Soft seam--1/8 in. layer
(LL = 39.8, PI = 19.4)	81.3'	651.4	49.7	56.7	23.0	18.3	
SR-1 74-88	42.2'-	651.4-	19.8	22.2	25.6	23.7	Soft seam--1/8 in. layer
(LL = 38.0, PI = 19.7)	42.6'	651.0					

NOTES: (1) ϕ_p = Maximum Value; ϕ_r = Minimum Value. (2) Sample toppled and failed before residual was reached. (3) Residual was not reached; actual value may be slightly (less than 1°) lower.

For sliding stability analyses along horizontal planes passing through the Unit C member of the St. Peter formation it is recommended that an angle of internal friction of 23 degrees and a cohesion of zero be utilized. These recommended values allow for the differences between laboratory and in-situ conditions in the field.

Vertical and near vertical joint planes were reported in a number of the borings. For example in boring 74-16 behind the land wall in Units A and A'; in boring 74-6 on the water side of the lower guide wall in Units A' and B; in boring 74-24 behind the upper guide wall in Unit A'; in borings 74-34, 74-36A, 74-38, 74-39 and 74-41 along the length of the intermediate wall in Units A', B and C. The orientation of these joint planes is not know. However, from earlier reports prepared by the St. Paul District it is known that two sets of vertical and near vertical joints traverse the site perpendicular and parallel, respectively, to the alignment of the river valley. It is suggested that the orientation of the joint planes reported in the 1974 borings be considered oriented in similar directions and that such joint plants may be taken as forming the vertical or near-vertical upstream boundaries of sliding blocks passing through the Unit C member of the foundation sequence.

Mississippi River Alluvium

The floor of the river lock, the river walls and the dam are founded on the Mississippi River alluvium.

The floor of the river lock is founded on clayey, silty and gravelly sands the maximum thickness of which was about 18 feet in boring 74-80. Boring 74-70, near the downstream end of the lock encountered a one-half inch horizon of soft brown material at the base of the alluvium. This material may originate from previous foundation grouting in this area. On the other hand its soft consistency suggests a possible different origin.

Standard penetration tests, in borings penetrating the alluvium beneath the river lock floor, varied between a minimum value of 16 blows per foot and a maximum value of 80 blows per foot; the average being about 55 blows per foot. The results of these tests suggest that the relative density of the alluvium beneath the river lock floor varies, in a qualitative sense, between compact, dense and very dense. All borings within the river lock indicated a greater or lesser amount of limestone fragments present in the samples recovered. Therefore, the standard penetration test results should be interpreted with caution. It is suspected that a good proportion of the alluvium beneath the river lock floor is in fact a backfill mixture originating in part from the river alluvium and in part from the St. Peter sandstone. No jar samples of this alluvium were found to remain for index and physical property tests.

For purposes of analyses an average saturated unit weight of 125 pounds per cubic foot is suggested together with an angle of internal friction of 35 degrees and a cohesion of zero for the alluvium beneath the river lock floor.

The river walls are founded on timber piles which penetrate the river alluvium to variable depths. In general the borings revealed the alluvium beneath the river walls to consist of dense to very dense sandy gravels to gravelly sands with inclusions of limestone slabs and boulders. The thickness of the alluvium beneath the river walls varies between 19 and 26 feet. In borings 74-42 and 74-46A horizons of closely packed limestone slabs and boulders were encountered at depths of 8.5 and 7.5 feet respectively below the base of the wall. These horizons of limestone were reported to be 11 and 8 feet thick respectively. Timber piles in such areas would not be expected to penetrate the limestone horizons. If the piles were overdriven to achieve greater penetration, it is conceivable that they could have been broomed at the ends or sheared along their length. The load carrying capacity of such piles would be less than the design load.

The standard penetration tests results in borings penetrating beneath the river walls should be viewed with caution because of the coarse gradations reported. This fact not withstanding the penetration results do indicate qualitatively, that the relative density of the alluvium beneath the river walls is noticeably greater than that beneath the river lock floor and beneath the dam. A result to be expected because of the pile driving operations during construction.

Jar samples from the standard penetration tests in boring 74-46A were combined to form a sample of sufficient size for testing purposes. The specific gravity, minimum and maximum dry density and grain size distribution tests were carried out. There was not a sufficient volume of material to carry out any physical property tests. The grain size distribution for the minus 1 1/2 inch material is shown on plate B-2-28. The specific gravity of this sample was 2.74 and the coefficient of uniformity was 160. The minimum and maximum dry densities were 110.5 and 146 pounds per cubic foot respectively. There is some question about the value of the maximum dry density yet the rather well graded nature of the minus 1 1/2 inch material and the greater specific gravity may explain this high value.

Assuming that the average in-situ dry density of the river alluvium beneath the river walls is at least equal to 110.5 pounds per cubic foot gives an average saturated unit weight for the alluvium of 133 pounds per cubic foot. For purposes of analyses it is suggested that a saturated unit weight of 135 pounds per cubic foot be used. An angle of internal friction of 40 degrees and a cohesion of zero may be used to evaluate the ultimate pile capacities and the ultimate bearing capacity of the alluvium when assessing the factor of safety of pile groups beneath individual monoliths.

The main structure of the dam bears directly upon a river alluvium consisting of compact to dense sandy gravels and gravelly sands with significant horizons of closely packed limestone slabs and boulders. Upstream of the dam the alluvium varies in thickness between 37 and 60 feet, the minimum thickness occuring in the vicinity of the sluiceway intake area. Downstream the thickness of the alluvium varied between 35 and 42 feet with the maximum thickness occuring immediately east of the downstream half of the old river wall. The borings upstream and downstream of the dam reveal that over 50 per cent of the thickness of the river alluvium consists of horizons of closely packed limestone slabs and boulders.

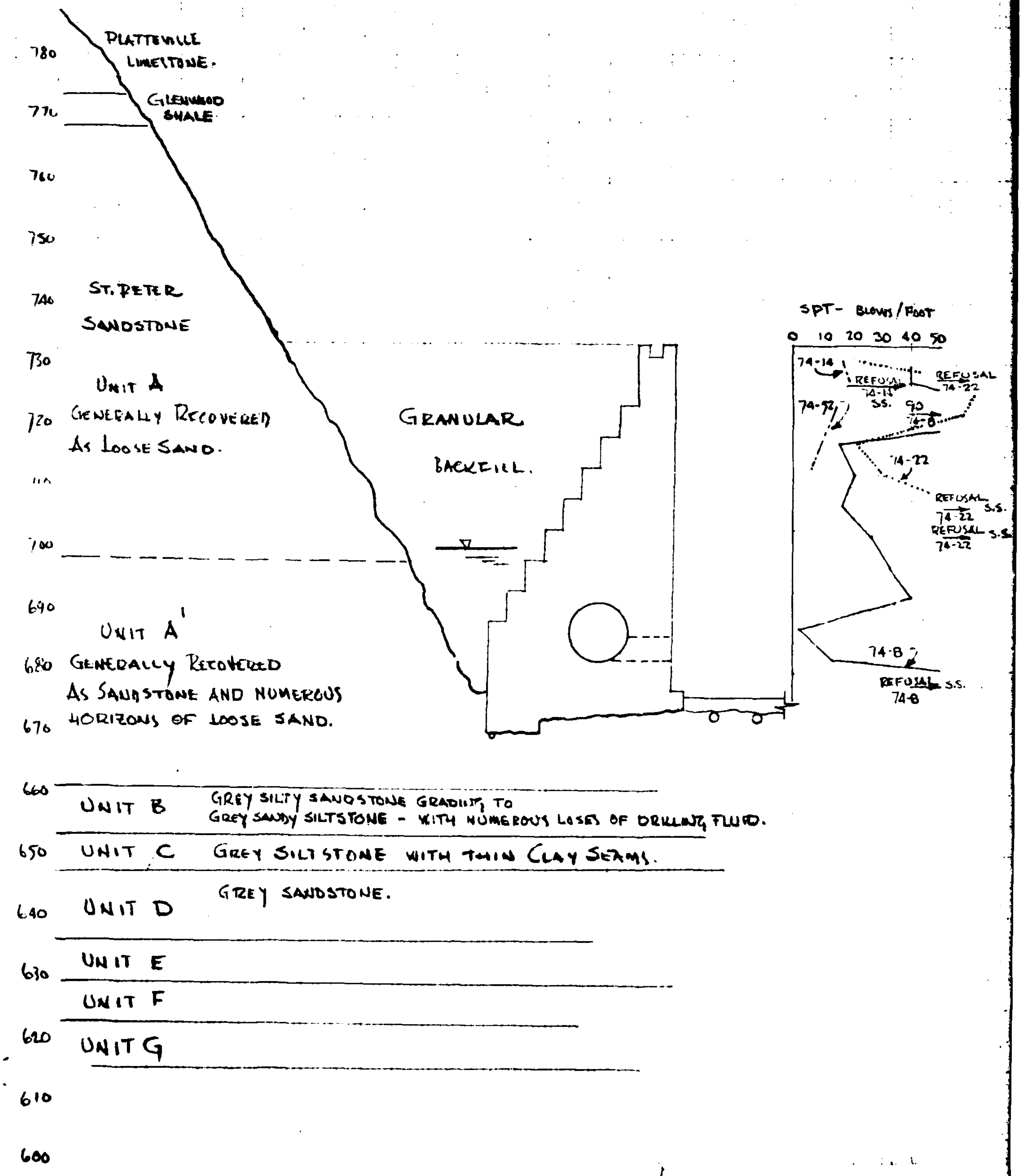
The standard penetration test results in borings upstream and downstream of the dam varied between 12 and 60 blows per foot with over 60 per cent of the tests having values less than 30 blows per foot. The average penetration resistance was 25 blows per foot. These values contrast markedly with those reported from beneath the river walls and indicate that in the area of the dam the qualitative relative density of the river alluvium is significantly less than elsewhere, all other things being equal.

For testing purposes jar samples from boring 74-82 were combined to form a larger sample. Specific gravity, minimum and maximum dry density and grain size distribution tests were carried out. The grain size distribution for the minus 1 1/2 inch material is shown on plate B-2-29. The average specific gravity was 2.76 and the uniformity coefficient was 60. The minimum and maximum dry densities were 110.6 and 146 pounds per cubic foot respectively.

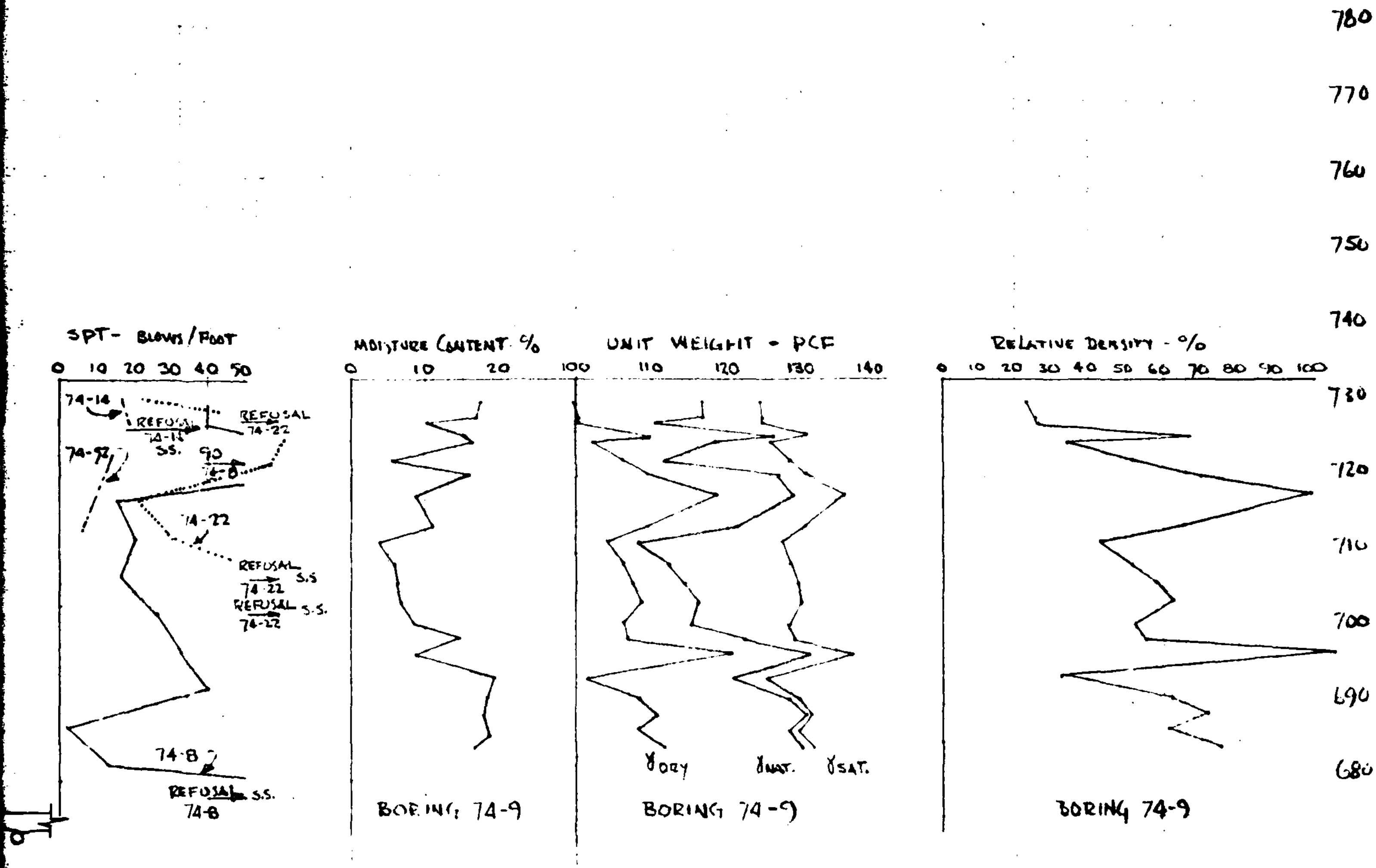
The penetration test results and judgement suggest that the in-situ properties of the river alluvium in the area of the dam should have values less than those estimated elsewhere. A saturated unit weight of 120 pounds per cubic foot and an angle of internal friction of 33 degrees are suggested for design analyses in the area of the dam. A coefficient of sliding resistance equivalent to an angle of 26.5 degrees is suggested for analyses of sliding along the base of the dam. For planes of sliding below the base of the dam a coefficient of sliding equivalent to an angle of 33 degrees is recommended. For assessing the sliding stability of monoliths of the river walls it is suggested that the passive resisting force due to the river alluvium be determined using a passive earth pressure coefficient of 1.5 for the alluvium immediately east of the river monoliths. This same coefficient may be utilized where concrete keys exist beneath the dam.

PLATES

APPENDIX B-2



780
770
760
750
740
730
720
700
690
680
670
660
650
640
630
620
610
600
PLATTEVILLE LIMESTONE.
GLENWOOD SHALE
ST. PETER SANDSTONE
UNIT A
GENERALLY RECOVERED AS LOOSE SAND.
GRANULAR BACKFILL.
UNIT A'
GENERALLY RECOVERED AS SANDSTONE AND NUMEROUS HORIZONS OF LOOSE SAND.
SPT- BLOWS / FOOT
0 10 20 30 40 50
74-14
REFUSAL 74-14 SS.
REFUSAL 74-22
74-52
90 74-8
74-22
REFUSAL S.S. 74-22
REFUSAL S.S. 74-22
74-8
REFUSAL S.S. 74-8
UNIT B
GREY SILTY SANDSTONE GRADING TO GREY SANDY SILTSTONE - WITH NUMEROUS LOSES OF DRILLING FLUID.
UNIT C
GREY SILTSTONE WITH THIN CLAY SEAMS.
UNIT D
GREY SANDSTONE.
UNIT E
UNIT F
UNIT G



STUDY of ALTERNATIVES for REHABILITATION

MISSISSIPPI RIVER LOCK & DAM NO.1

INDEX PROPERTIES

GRANULAR BACKFILL-LAND WALL

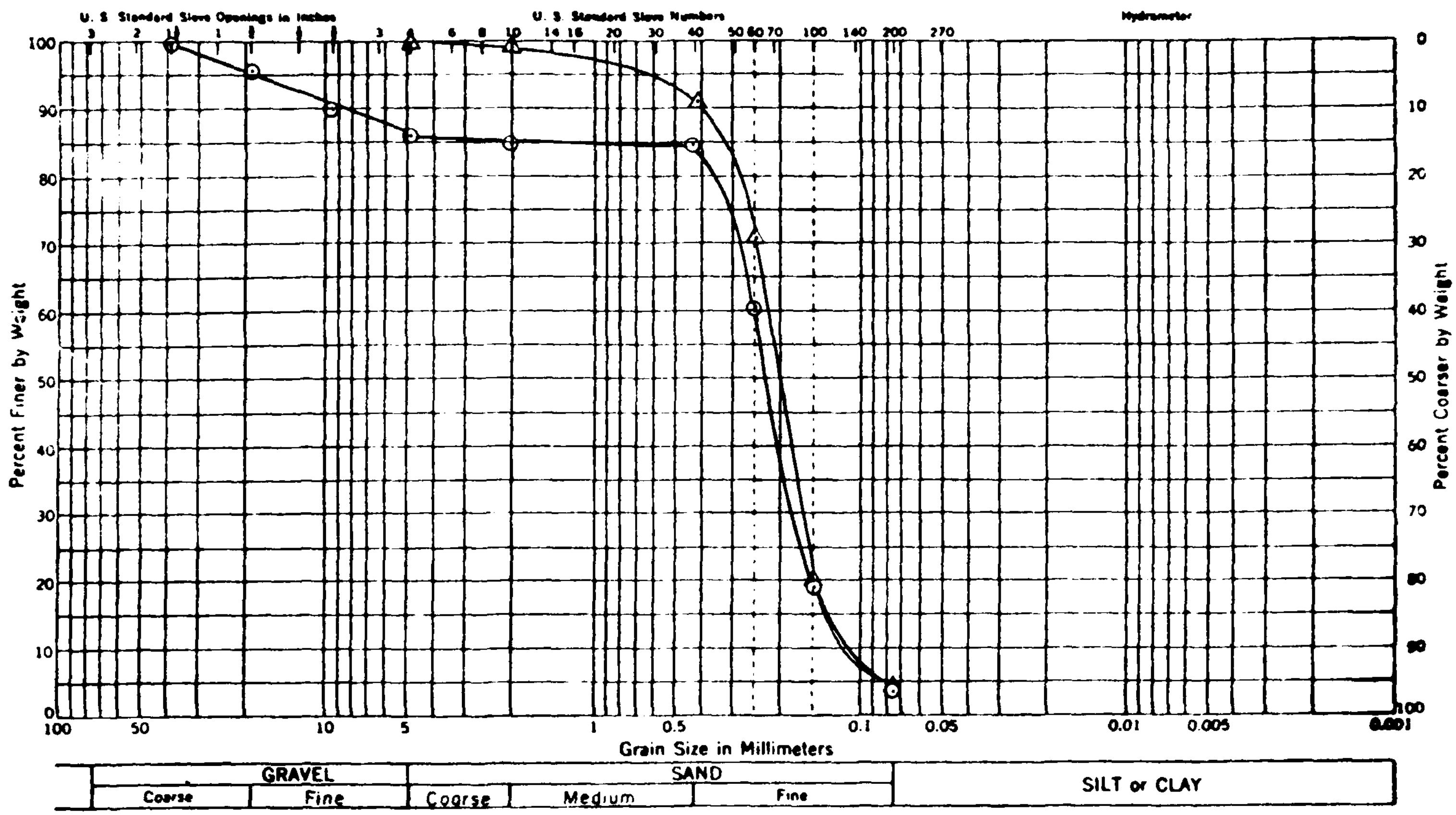
BORING 74-9

ST. PAUL, MINNESOTA DISTRICT

FILE NO. DATED: MARCH 1975

PLATE B-2-1

UNIFIED CLASS CATION SYSTEM



SAMPLE NO.	W.C.	LL	PL	PI	CLASSIFICATION
					⊙—⊙ : Sample as received
					$Cu = D_{60}/D_{10} = 2.2$
					△—△ : Sample used for consolidated
					drained triaxial shear tests
					$Cu = D_{60}/D_{10} = 1.9$

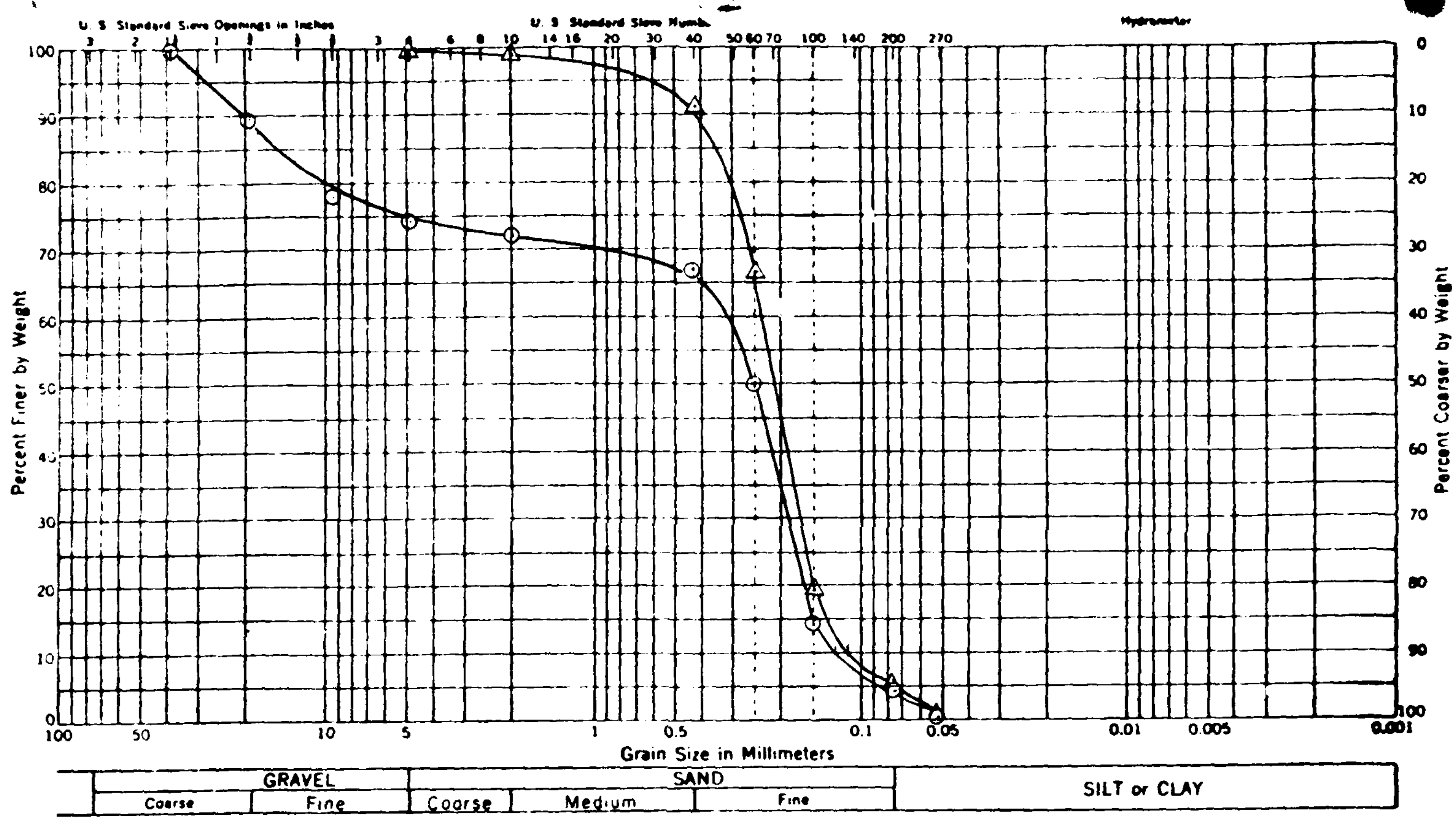
HARZA ENGINEERING COMPANY
LOCK & DAM NO.1
HOLE NO. 74-9
MINNEAPOLIS, MINN.
COMBINED SAMPLE : S-2 to S-10
DEPTH : 5.0' - 27.5'

SOIL TESTING SERVICES, INC.
111 PFINGSTEN ROAD
NORTHBROOK ILLINOIS

DRAWN	APPROVED	DATE	JOB No.

PLATE B-2-2

UNIFIED CLASS CATION SYSTEM



SAMPLE NO.	W.C.	LL	PL	PI	CLASSIFICATION
					⊙—⊙ : Sample as received
					$C_U = D_{60}/D_{10} = 3.4$
					△—△ : Sample used for consolidated
					drained triaxial shear tests
					$C_U = D_{60}/D_{10} = 2.0$

HARZA ENGINEERING COMPANY
LOCK & DAM NO.1
HOLE NO. 74-9
MINNEAPOLIS, MINN.
COMBINED SAMPLE : S-11 to S-19
DEPTH : 29' - 49'

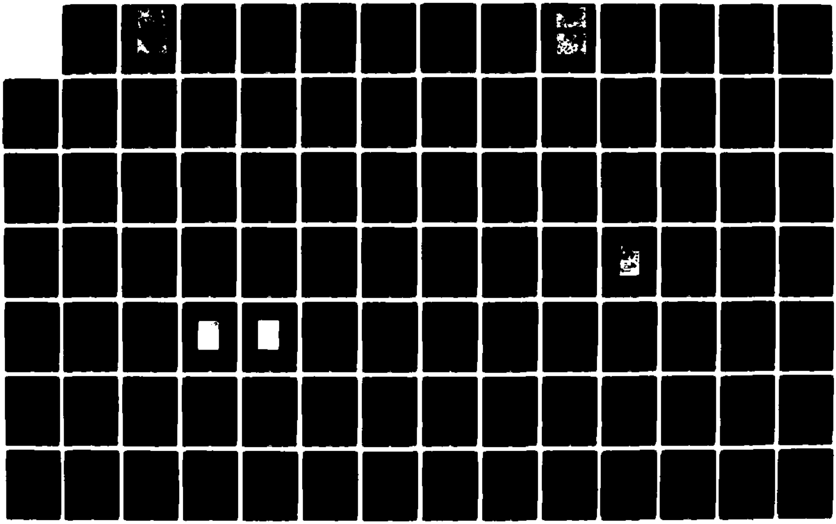
SOIL TESTING SERVICES, INC.
111 PFINGSTEN ROAD
NORTHBROOK ILLINOIS

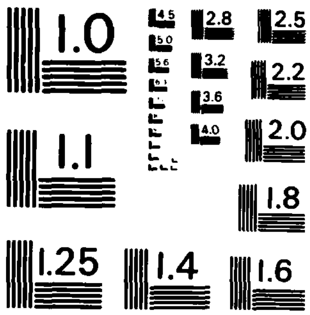
DRAWN	APPROVED	DATE	JOB No.
SZ	CHC	10-22-74	17326-D

PLATE B-2-3

AD-A134 023 MISSISSIPPI RIVER: STUDY OF ALTERNATIVES FOR REHABILITATION OF LOCK AND D..(U) CORPS OF ENGINEERS ST PAUL MN ST PAUL DISTRICT APR 76 2/4

UNCLASSIFIED F/G 13/2 NL





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

PHOTO A

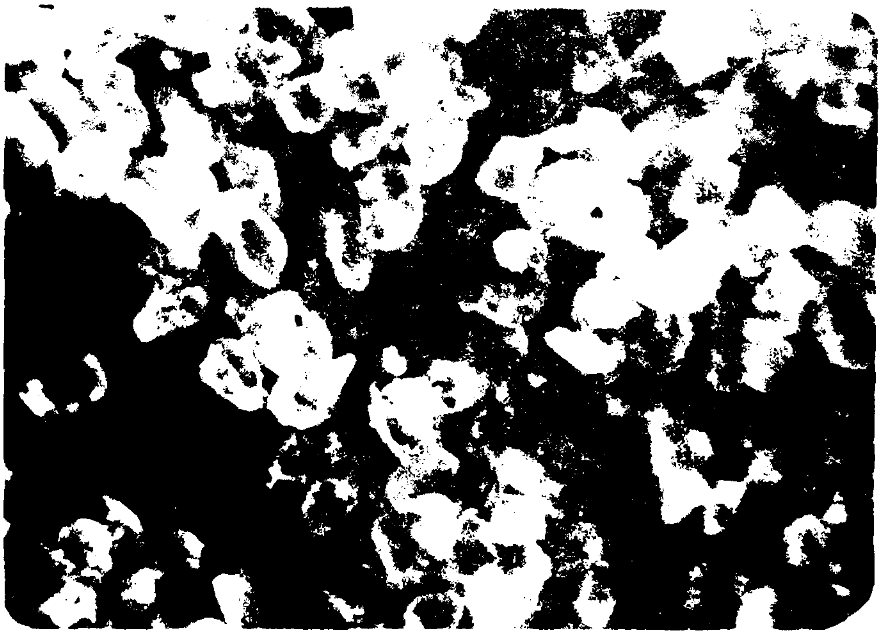


PHOTO B

STUDY of ALTERNATIVES for REHABILITATION

MISSISSIPPI RIVER LOCK & DAM NO.1

SAND PARTICLES

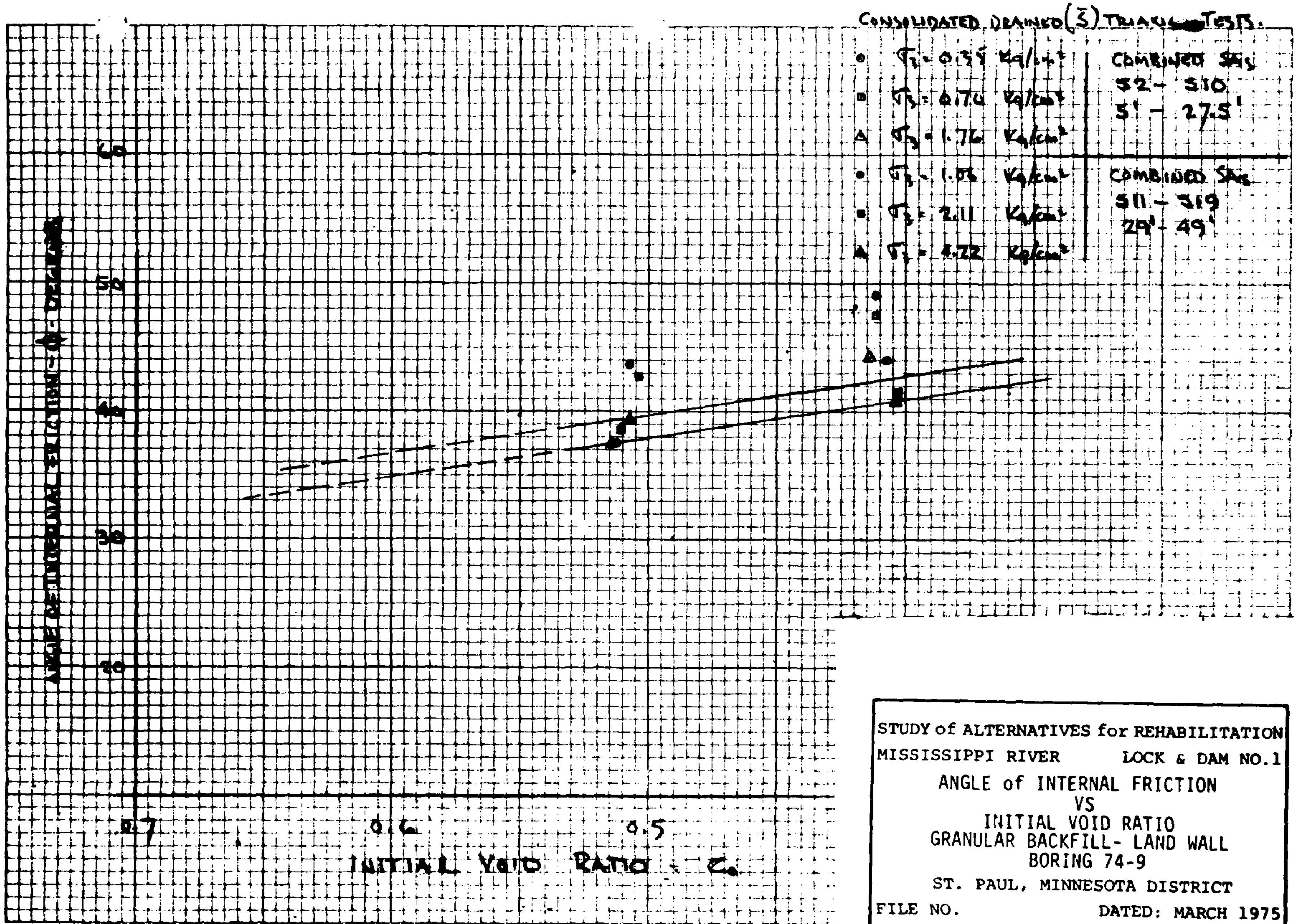
GRANULAR BACKFILL-LAND WALL

BORING 74-9

ST. PAUL, MINNESOTA DISTRICT

FILE NO. DATED: MARCH 1975

PLATE B-2-4



STUDY of ALTERNATIVES for REHABILITATION

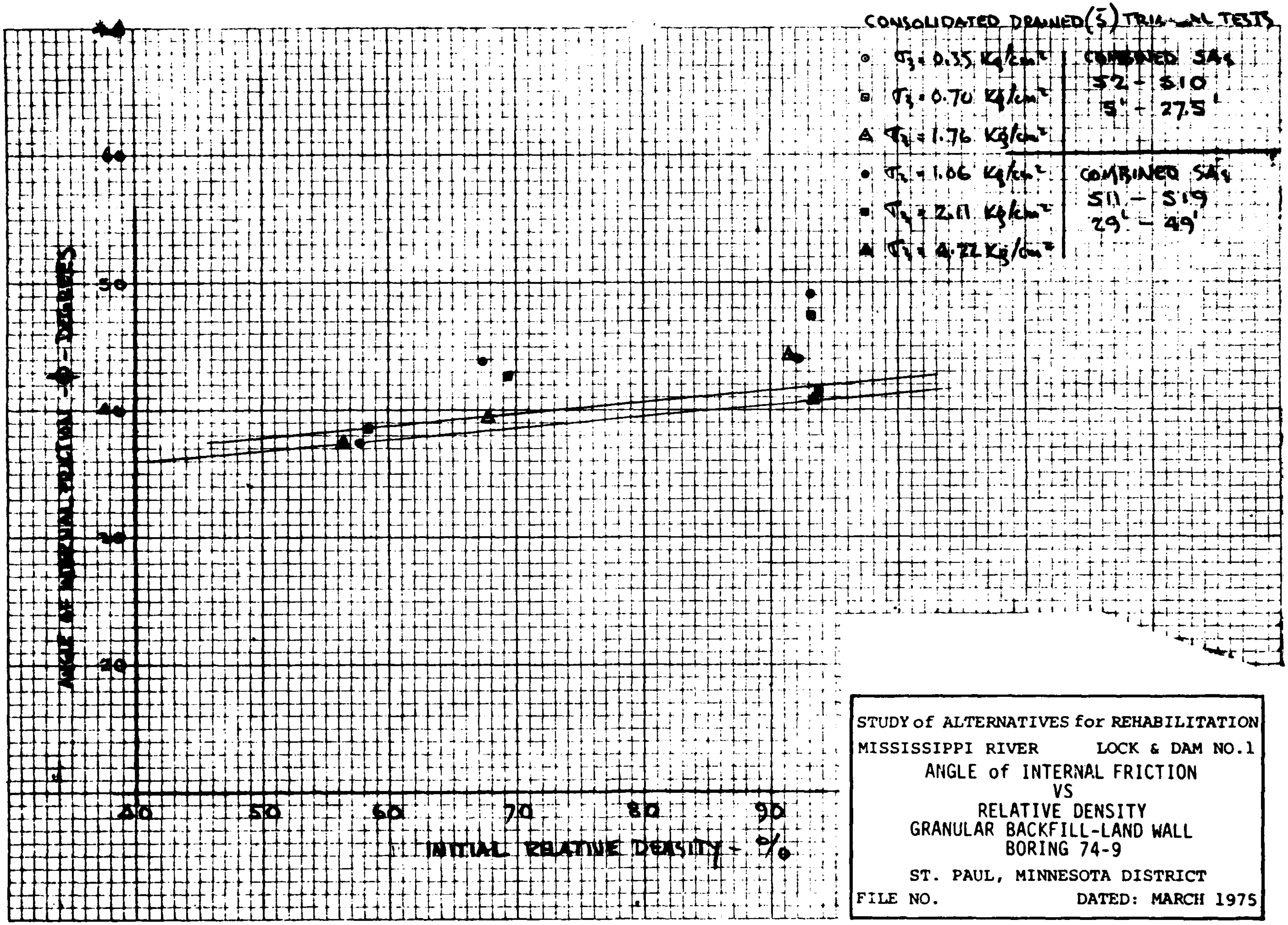
MISSISSIPPI RIVER LOCK & DAM NO.1

ANGLE of INTERNAL FRICTION
VS
INITIAL VOID RATIO
GRANULAR BACKFILL- LAND WALL
BORING 74-9

ST. PAUL, MINNESOTA DISTRICT

FILE NO. DATED: MARCH 1975

PLATE B-2-5



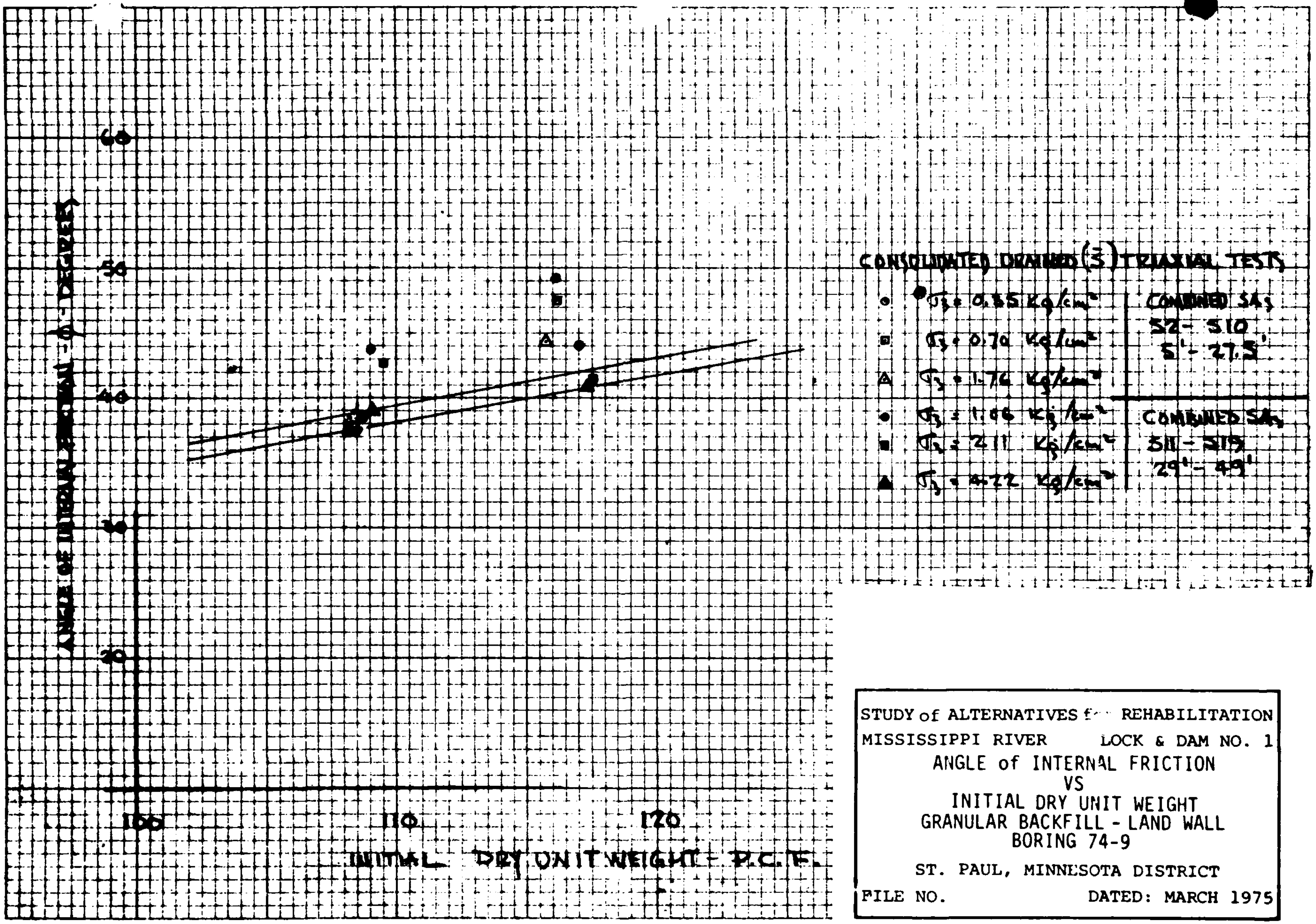
STUDY of ALTERNATIVES for REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO.1

ANGLE of INTERNAL FRICTION
VS
RELATIVE DENSITY
GRANULAR BACKFILL-LAND WALL
BORING 74-9

ST. PAUL, MINNESOTA DISTRICT

FILE NO. DATED: MARCH 1975

PLATE B-2-6



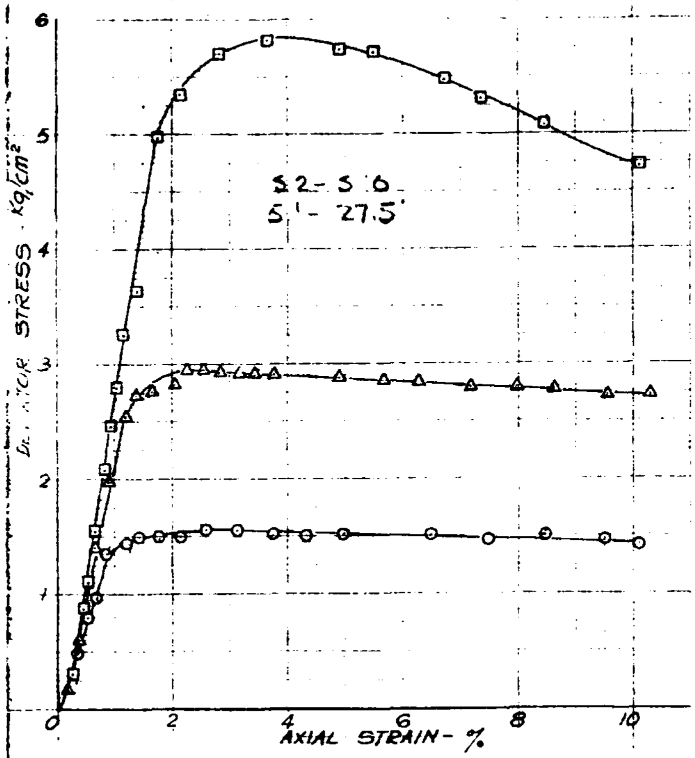
STUDY of ALTERNATIVES f·· REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO. 1

ANGLE of INTERNAL FRICTION
VS
INITIAL DRY UNIT WEIGHT
GRANULAR BACKFILL - LAND WALL
BORING 74-9

ST. PAUL, MINNESOTA DISTRICT

FILE NO. DATED: MARCH 1975

PLATE B-2-7

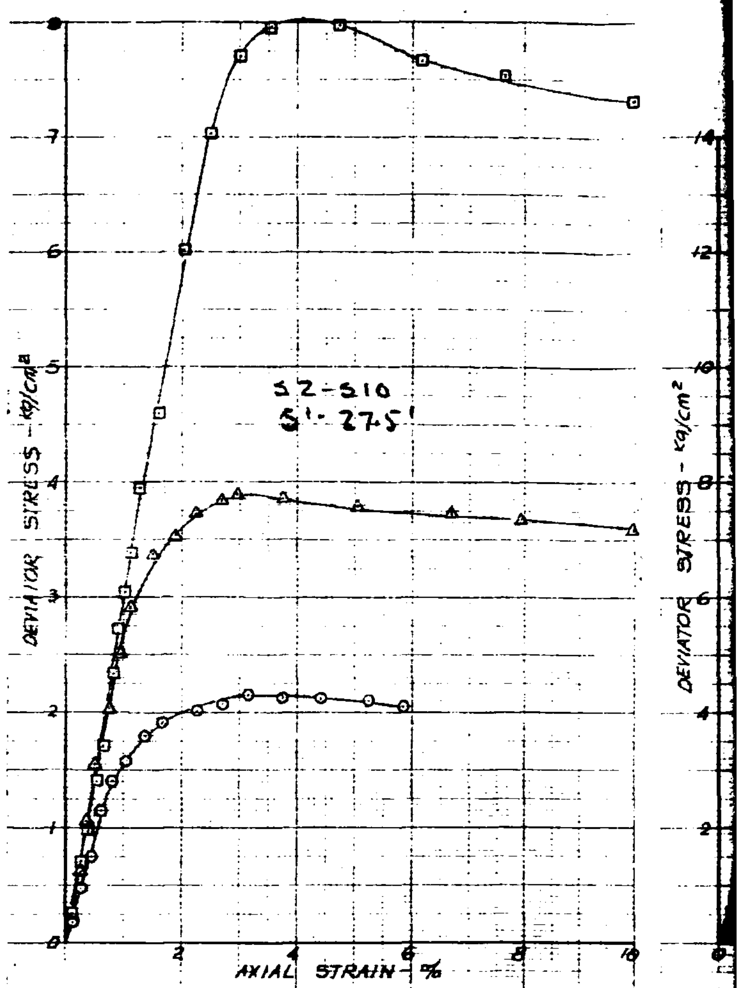


LEGEND

- ○—○ 0.35 kg/sq cm(5 psi) Consolidation Pressure
- △—△ 0.70 kg/sq cm(10 psi) Consolidation Pressure
- □—□ 1.76 kg/sq cm(25 psi) Consolidation Pressure

INITIAL SAMPLE		DRY DENSITY pcf	WATER CONTENT %		CHAMBER PRESSURE kg/cm²	CONSOLIDATION PRESSURE kg/cm²	STRAIN AT MAX. $(\sigma_1 - \sigma_3)$ %	MAX DEV STRESS kg/cm²
DIAMETER in.	HEIGHT in.		INITIAL	FINAL				
2.000	4.075	109.0	9.1	14.2	2.46	0.35	2.59	156
2.000	4.025	109.5	9.1	14.1	2.81	0.70	2.25	2.95
2.000	4.090	109.1	9.0	14.0	3.87	1.76	3.68	5.82

30 psi(2.11 kg/sq cm) of back pressure was applied prior to shear.
Sample : Fine sand,trace silt - Very light brown(SP)
Test samples were molded at approximately 75% of relative density



LEGEND

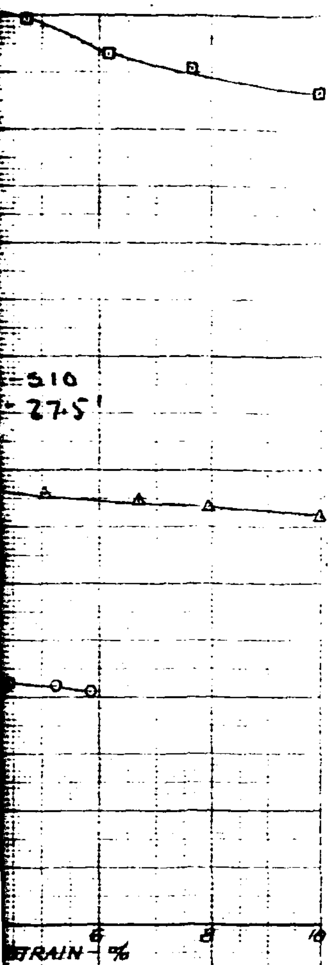
- ○—○ 0.35 kg/sq cm(5 psi) Consolidation Pressure
- △—△ 0.70 kg/sq cm(10 psi) Consolidation Pressure
- □—□ 1.76 kg/sq cm(25 psi) Consolidation Pressure

INITIAL SAMPLE		DRY DENSITY pcf	WATER CONTENT %		CHAMBER PRESSURE kg/cm²	CONSOLIDATION PRESSURE [illegible]	STRAIN AT MAX. [illegible]	MAX DEV [illegible]
DIAMETER in.	HEIGHT in.		INITIAL	FINAL				
2.000	3.950	116.2	9.1	14.7	2.46	0.35	3.13	2.15
2.000	3.975	116.2	9.1	14.9	2.81	0.70	2.95	3.88
2.000	3.950	115.7	9.1	15.0	3.87	1.76	4.15	8.04

30 psi(2.11 kg/sq.cm) of back pressure was applied prior to shear.
Sample : Fine sand, trace silt-very light brown (SP)
Test samples were aimed to compact at 100% of relative density
- Actual relative density=93%

INITIAL SA[illegible]	
DIAMETER in	H[illegible]
2.000	4.[illegible]
2.000	4.[illegible]
2.000	4.[illegible]

30 psi(2.[illegible]
Sample : [illegible]

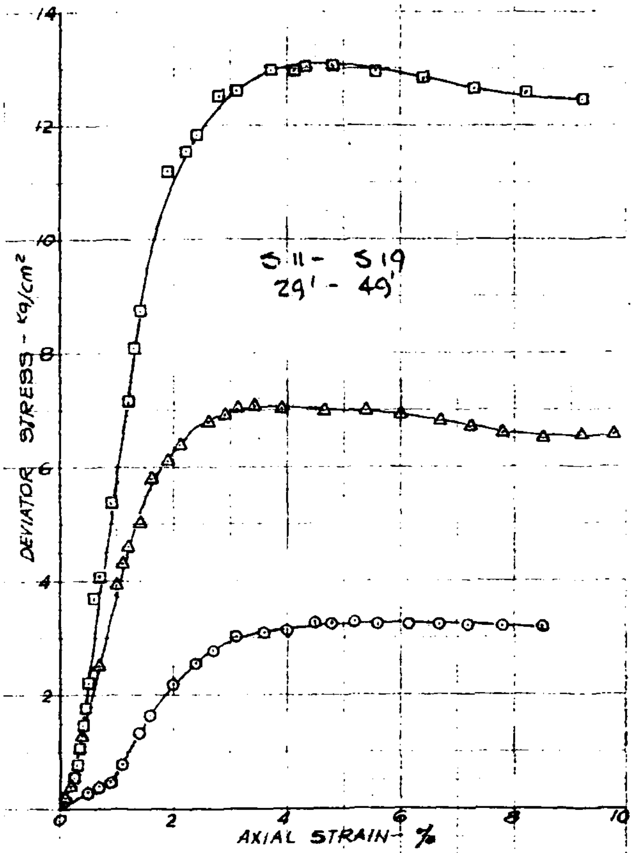


si) Consolidation Pressure
si) Consolidation Pressure
si) Consolidation Pressure

...NTENT FINAL	CHAMBER PRESSURE kg/cm²	CONSOLIDATION PRESSURE kg/cm²	STRAIN AT MAX $(\sigma_1 - \sigma_3)$ %	MAX DEV STRESS kg/cm²
4.7	2.46	0.35	3.13	2.15
4.9	2.81	0.70	2.95	3.88
5.0	3.87	1.76	4.15	8.04

...ure was applied prior to shear.
... light brown (SP)
... compact at 100% of relative density

...ive density=93%

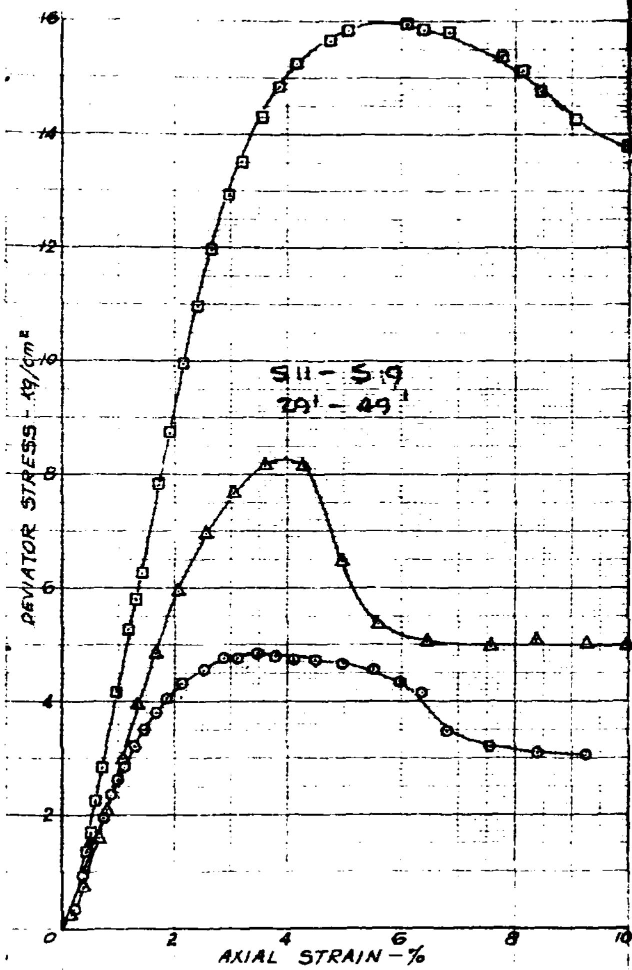


LEGEND

O——O 1.06 kg/sq.cm (15 psi) Consolidation Pressure
Δ——Δ 2.11 kg/sq.cm (30 psi) Consolidation Pressure
□——□ 4.22 kg/sq.cm (60 psi) Consolidation Pressure

INITIAL SAMPLE DIAMETER in	INITIAL SAMPLE HEIGHT in	DRY DENSITY pcf	WATER CONTENT % INITIAL	WATER CONTENT % FINAL	CHAMBER PRESSURE kg/cm²	CONSOLIDATION PRESSURE kg/cm²	STRAIN AT MAX $(\sigma_1 - \sigma_3)$ %	MAX DEV STRESS kg/cm²
2.000	4.075	108.5	9.1	17.9	3.17	1.06	5.20	3.29
2.000	4.075	108.7	8.6	16.5	4.22	2.11	3.90	7.02
2.000	4.075	108.2	9.1	16.2	6.33	4.22	4.83	13.03

30 psi (2.11 kg/sq.cm) of back pressure was applied prior to shear.
Sample : Fine sand, trace silt - Very light brown (SP)
Test samples were molded at approximately 75% of relative density



LEGEND

O——O 1.06 kg/sq.cm (15 psi) Consolidation Pressure
Δ——Δ 2.11 kg/sq.cm (30 psi) Consolidation Pressure
□——□ 4.22 kg/sq.cm (60 psi) Consolidation Pressure

INITIAL SAMPLE DIAMETER in	INITIAL SAMPLE HEIGHT in	DRY DENSITY pcf	WATER CONTENT % INITIAL	WATER CONTENT % FINAL	CHAMBER PRESSURE kg/cm²	CONSOLIDATION PRESSURE kg/cm²	STRAIN AT MAX $(\sigma_1 - \sigma_3)$ %	MAX DEV STRESS kg/cm²
2.000	3.975	117.0	9.0	14.6	2.46	1.06	3.32	4.85
2.000	3.950	117.5	9.1	14.5	2.81	2.11	3.61	8.14
2.000	3.975	117.3	9.0	14.3	3.87	4.22	5.75	16.00

30 psi (2.11 kg/sq.cm) of back pressure was applied prior to shear.
SAMPLE : Fine sand, trace silt - very light brown (SP)
Test samples were aimed to compact to 100% of relative density

Actual relative density=93%,

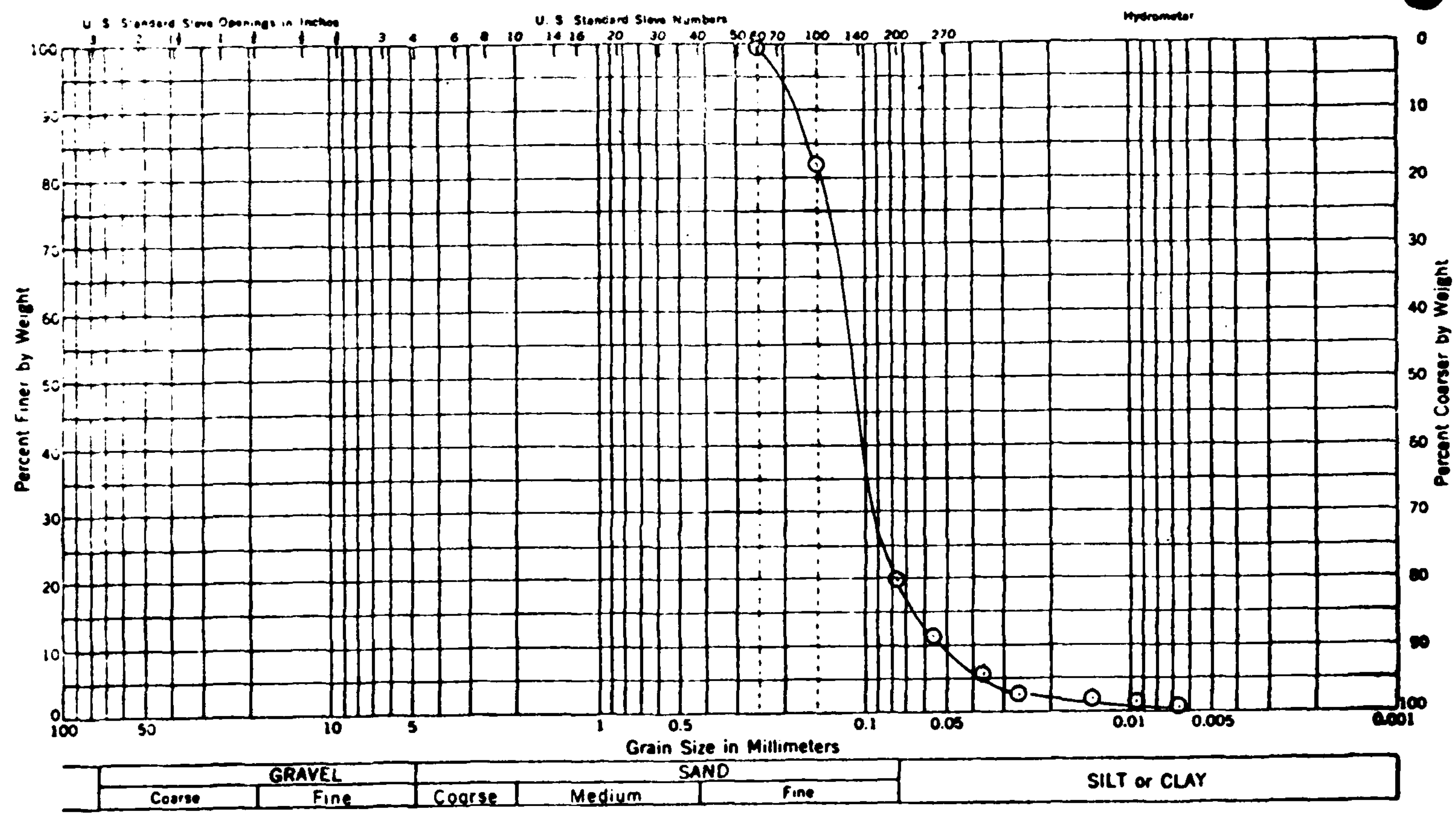
STUDY of ALTERNATIVES for REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO.1

CONSOLIDATED-DRAINED TRIAXIAL TESTS
SAMPLE DATA AND STRESS-STRAIN CURVES
GRANULAR BACKFILL-LAND WALL
BORING 74-9

ST. PAUL, MINNESOTA DISTRICT

FILE NO. DATED: MARCH 1975

UNIF[illegible]D [illegible]ION SYSTEM



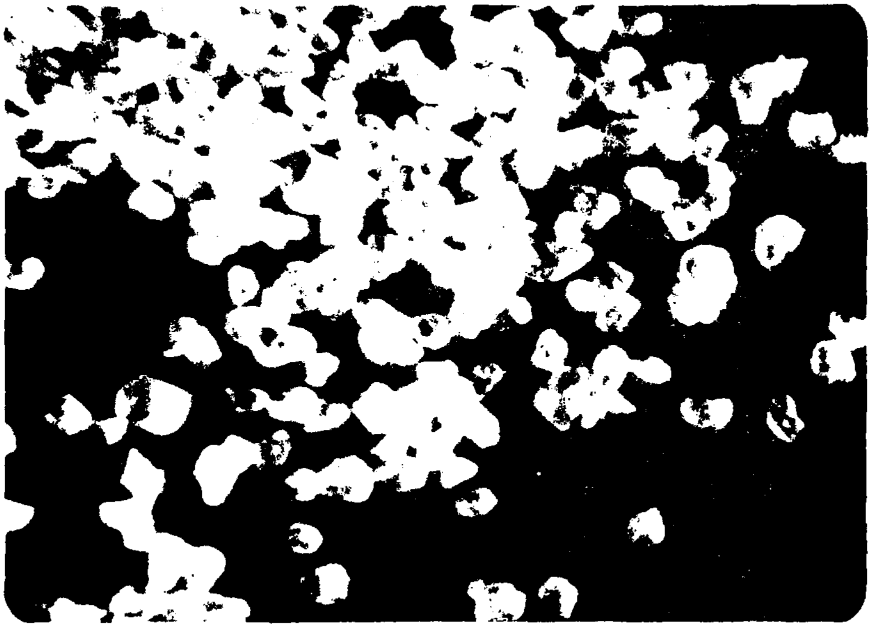
SAMPLE NO.	W.C.	LL	PL	PI	CLASSIFICATION
					Silty fine sand - Very light grey (SM)
					Specific Gravity = 2.66
					$C_u = D_{60}/D_{10} = 2.5$

Harza Engineering Company
Lock & Dam No.L
Minneapolis, Minn.
Boring No. : 74-36 A
Sample No. : SA-1,2,3 & 4
Depth : 60.1'-66.4'

SOIL TESTING SERVICES, INC.
111 PFINGSTEN ROAD
NORTHBROOK ILLINOIS

DRAWN	APPROVED	DATE	JOB No.
SZ	CHC	12-17-74	17326-E

PLATE B-2-9



TOTAL SAMPLE

PASSING No. 200

...NATIVES for REHABILITATION
... LOCK & DAM NO. 1

SAND PARTICLES

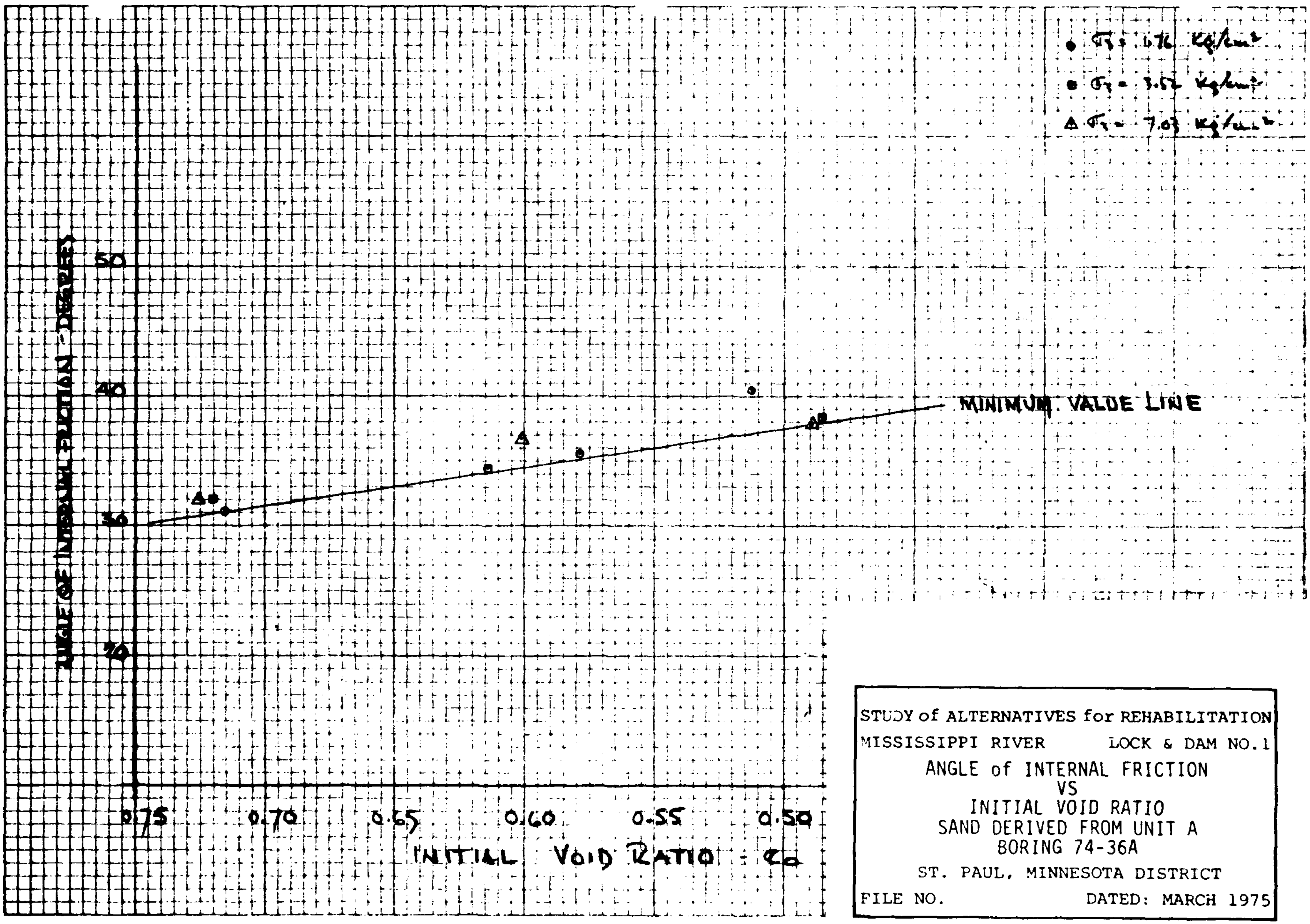
SAND DERIVED FROM UNIT A

BORING 74-36A

... MINNESOTA DISTRICT

DATED: MARCH 1975

PLATE B-2-10



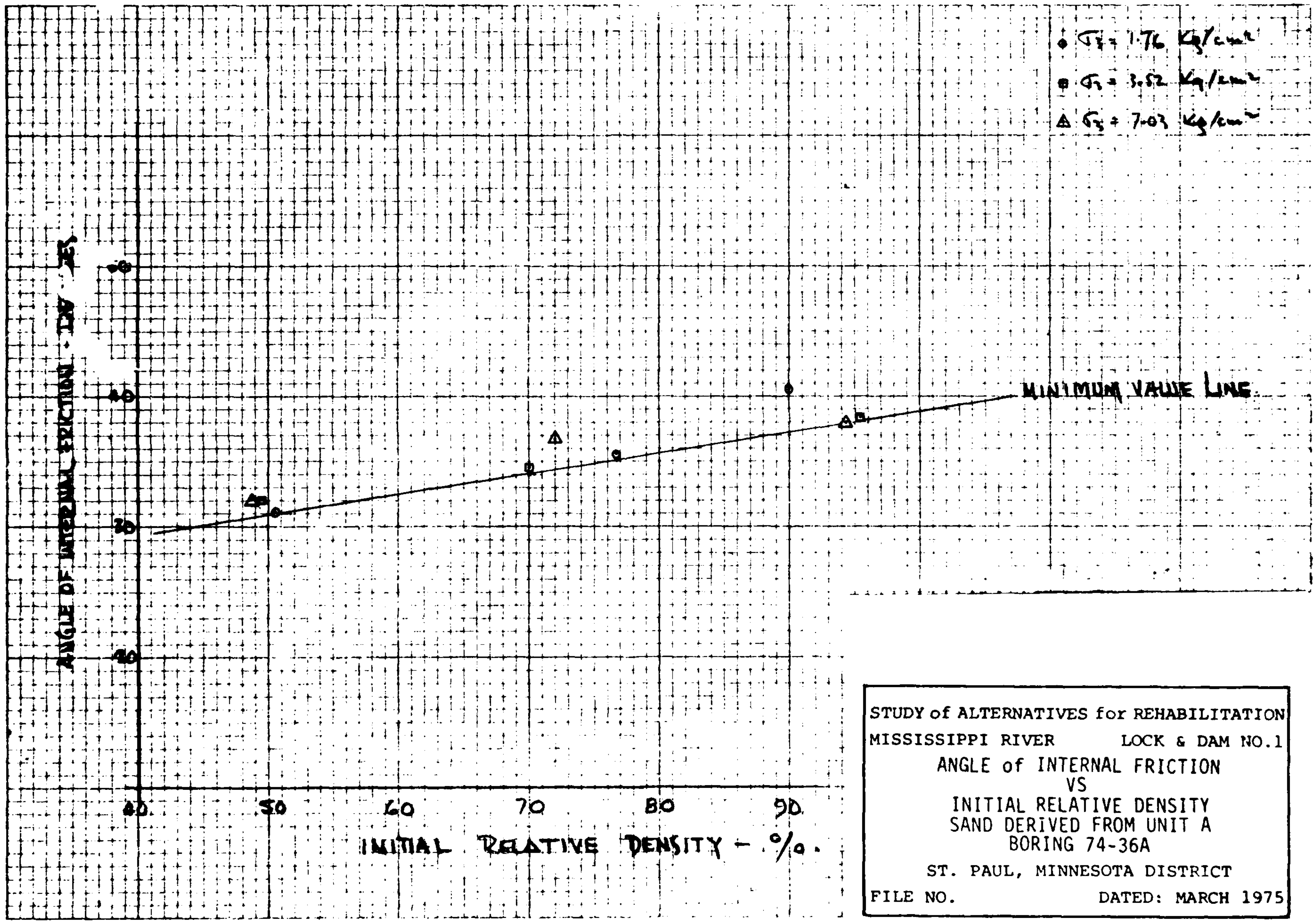
STUDY of ALTERNATIVES for REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO. 1

ANGLE of INTERNAL FRICTION
VS
INITIAL VOID RATIO
SAND DERIVED FROM UNIT A
BORING 74-36A

ST. PAUL, MINNESOTA DISTRICT

FILE NO. DATED: MARCH 1975

PLATE B-2-11



STUDY of ALTERNATIVES for REHABILITATION

MISSISSIPPI RIVER LOCK & DAM NO.1

ANGLE of INTERNAL FRICTION

VS

INITIAL RELATIVE DENSITY

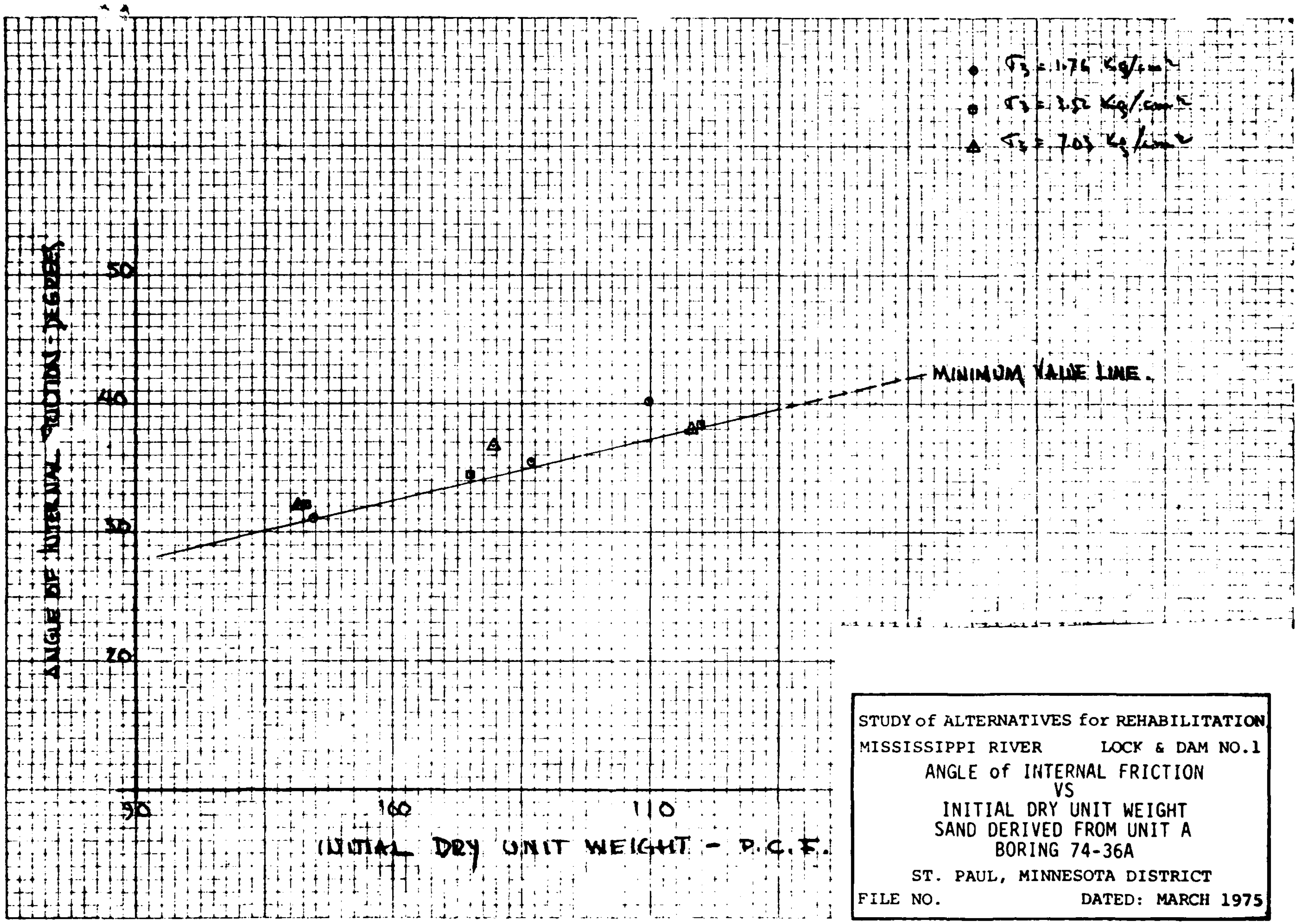
SAND DERIVED FROM UNIT A

BORING 74-36A

ST. PAUL, MINNESOTA DISTRICT

FILE NO. DATED: MARCH 1975

PLATE B-2-12



STUDY of ALTERNATIVES for REHABILITATION

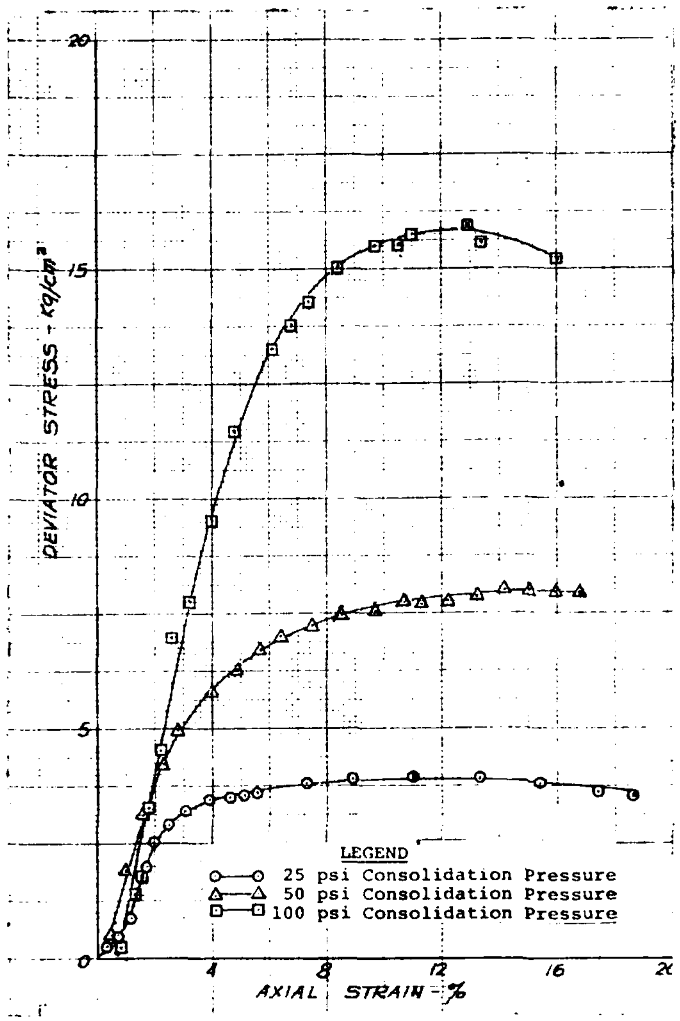
MISSISSIPPI RIVER LOCK & DAM NO.1

ANGLE of INTERNAL FRICTION
VS
INITIAL DRY UNIT WEIGHT
SAND DERIVED FROM UNIT A
BORING 74-36A

ST. PAUL, MINNESOTA DISTRICT

FILE NO. DATED: MARCH 1975

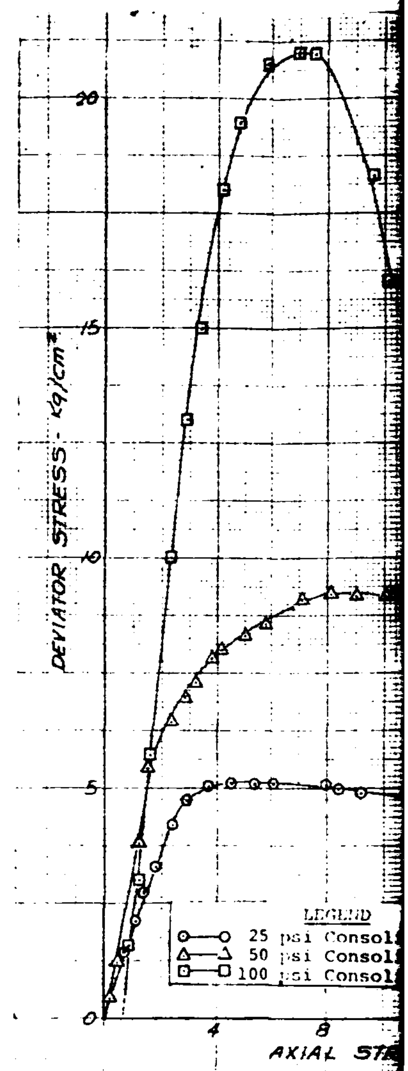
PLATE B-2-13



Diameter cm	Ht. cm	w/c %	γd pcf	AFTER w/c %	CONSOLIDATION kg/cm²	AT MAXIMUM ε %	DEVIATOR STRESS kg/cm²
5.080	10.16	11.0	96.9	19.0	1.76	8.79	3.89
5.080	10.16	11.0	96.6	18.3	3.52	15.13	7.99
5.080	10.03	11.0	96.3	18.0	7.03	12.88	15.87

Samples were saturated by using 30 psi (2.11 kg/sq.cm) of back pressure prior to shear.
Sample Description : Silty fine sand - Very light grey to white (SM)
MAXIMUM DRY DENSITY = 114.0 pcf MINIMUM DRY DENSITY = 84.0 pcf
Test samples were molded at approximately 50 % of relative density.

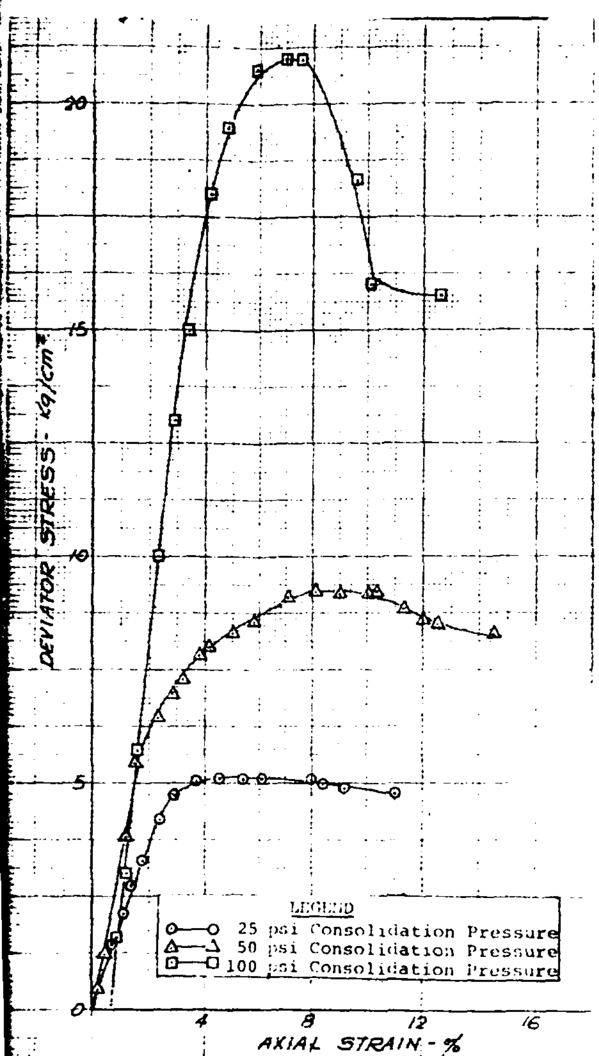
STRAIN RATE : 0.08 %/min.



DIAMETER cm	HEIGHT cm	w/c %	γd pcf	AFTER w/c %
5.080	10.09	10.0	105.3	18.5
5.080	10.16	10.0	103.0	18.4
5.080	10.09	10.0	103.8	17.8

Samples were saturated by using 30 psi (2.11 kg/
Sample Description : Silty fine sand - Very li
MAXIMUM DRY DENSITY = 114.0 pcf MINIMUM
Test samples were molded at approximately 70%

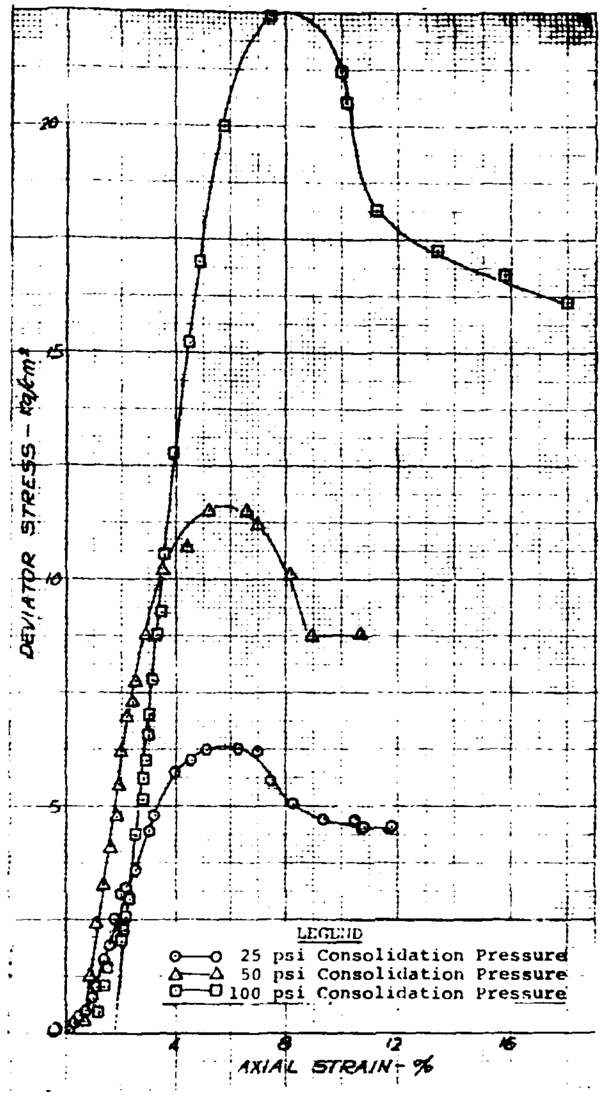
STRAIN RATE : 0.08 %/



TER	HEIGHT cm	W/C %	γ_d pcf	AFTER W/C %	CONSOLIDATION kg/cm²	AT MAXIMUM ϵ %	DEVIATOR STRESS kg/cm²
	10.09	10.0	105.3	18.5	1.76	6.08	5.16
	10.16	10.0	103.0	18.4	3.52	10.30	9.25
	10.09	10.0	103.8	17.8	7.03	6.97	21.04

were saturated by using 30 psi (2.11 kg/sq.cm) of back pressure prior to shear.
scription : Silty fine sand - Very light grey to white (SM)
RY DENSITY = 114.0 pcf MINIMUM DRY DENSITY = 84.0 pcf
les were molded at approximately 70% of relative density.

STRAIN RATE : 0.08 %/min.



DIA. cm	HEIGHT cm	W/C %	γ_d pcf	AFTER W/C %	CONSOLIDATION kg/cm²	AT MAXIMUM ϵ %	DEVIATOR STRESS kg/cm²
5.080	10.160	8.6	109.9	17.5	1.76	6.93	6.40
5.080	10.160	7.9	111.9	13.9	3.52	6.67	11.32
5.080	10.180	9.0	111.6	12.6	7.03	7.50	22.48

Samples were saturated by using 30 psi (2.11 kg/sq.cm) of back pressure prior to shear.
Sample Description : Silty fine sand - Very light grey to white (SM)
MAXIMUM DRY DENSITY = 114.0 pcf MINIMUM DRY DENSITY = 84.0 pcf
Test samples were molded at approximately 90 % of relative density.

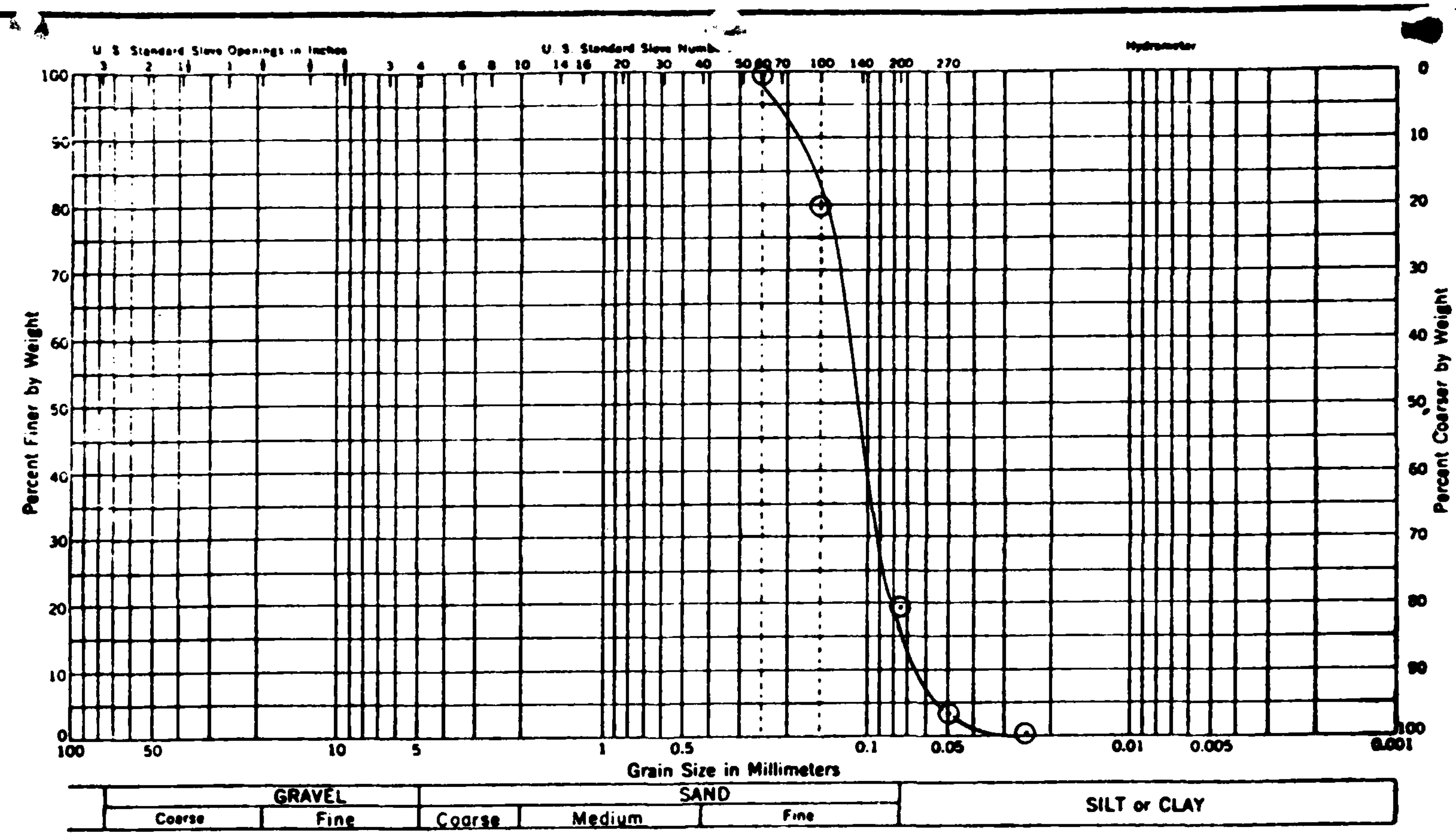
STRAIN RATE : 0.08 %/min.

STUDY of ALTERNATIVES for REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO.1

CONSOLIDATED-DRAINED TRIAXIAL TESTS
SAMPLE DATA AND STRESS-STRAIN CURVES
SAND DERIVED FROM UNIT A
BORING 74-36A

ST. PAUL, MINNESOTA DISTRICT
FILE NO. DATED: MARCH 1975

UNIFIED CLAS' FICATION SYSTEM



SAMPLE NO.	W.C.	LL	PL	PI	CLASSIFICATION
					Silty fine sand -Very Light grey(SM)
					$Cu = D_{60}/D_{10} = 1.78$

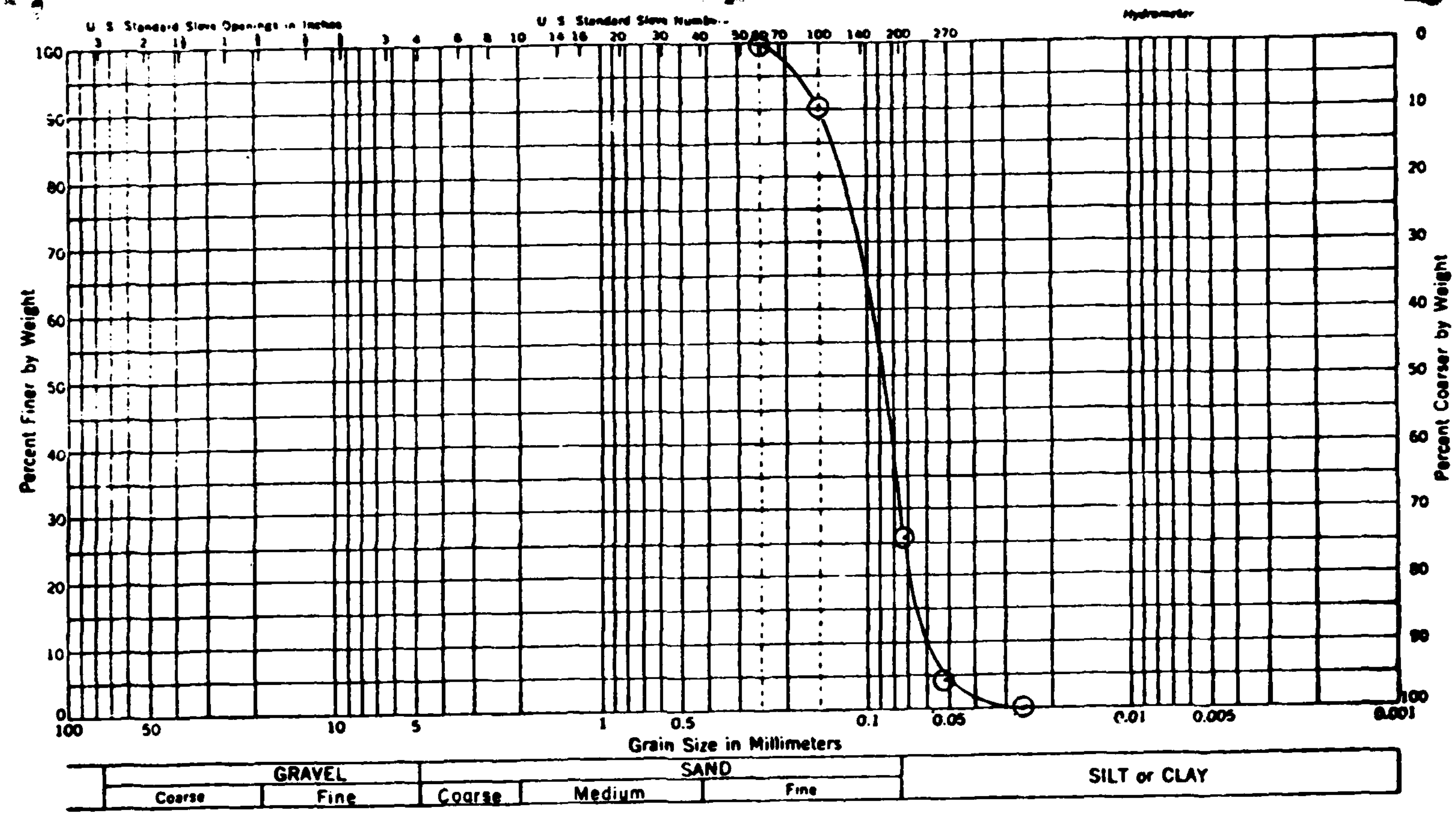
HARZA ENGINEERING COMPANY
LOCK & DAM NO.1
HARZA PROJRCT NO. 800A
HOLE NO. 74-36A
SAMPLE NO. 1
DEPTH : 60.1'- 60.4'

SOIL TESTING SERVICES, INC.
111 PFINGSTEN ROAD
NORTHBROOK ILLINOIS

DRAWN	APPROVED	DATE	JOB No.
SZ	CHC	11-19-74	7326-c

PLATE B-2-15

UNIFIED CLAS FICATION SYSTEM



SAMPLE NO.	W.C.	LL	PL	PI	CLASSIFICATION
					Silty fine sand - Very light grey(SM)
					$C_u = D_{60}/D_{10} = 1.62$

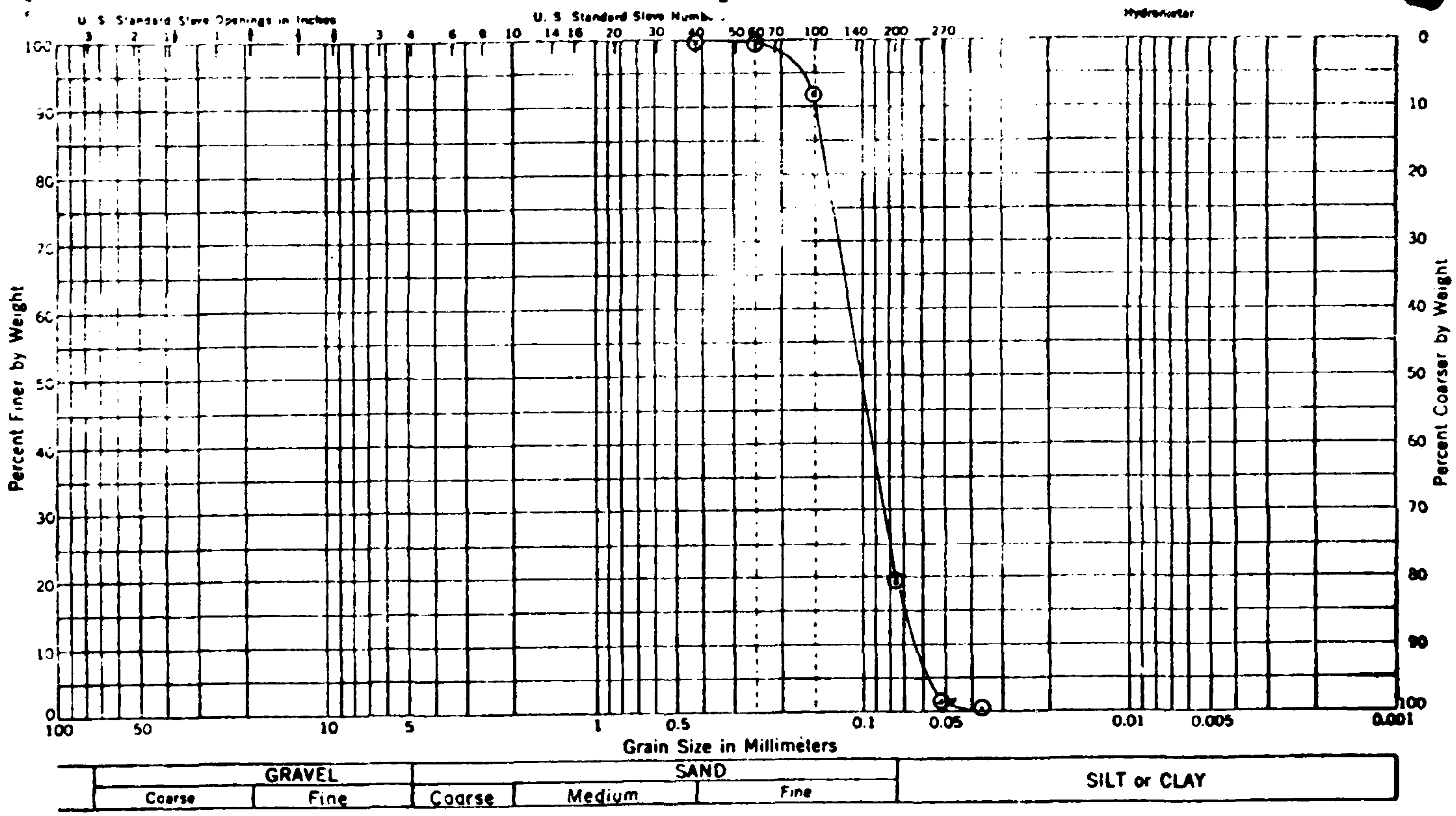
HARZA ENGINEERING COMPANY
LOCK & DAM NO. 1
HARZA PROJECT NO.800A
HOLE NO. 74-36A
SAMPLE NO.2
DEPTH : 62.0'-62.3'

SOIL TESTING SERVICES, INC.
111 PFINGSTEN ROAD
NORTHBROOK ILLINOIS

DRAWN	APPROVED	DATE	JOB No.
SZ	CHC	11-19-74	17326C

PLATE B-2-16

UNIFIED CLA SIFICATION SYSTEM



SAMPLE NO.	W.C.	LL	PL	PI	CLASSIFICATION
					Silty fine sand - Very light grey
					(SM)
					$C_u = D_{60}/D_{10} = 1.78$

HARZA ENGINEERING COMPANY

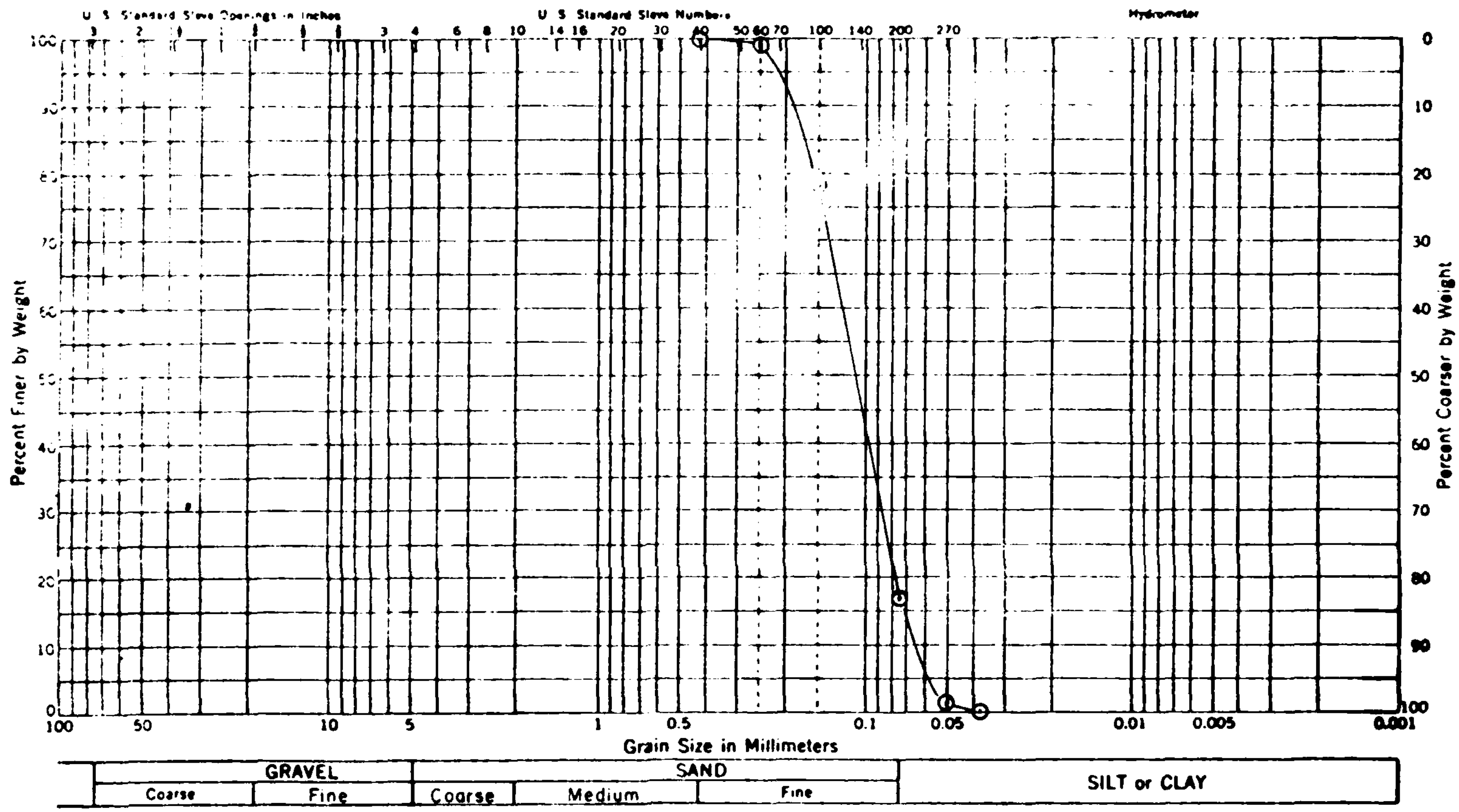
LOCK & DAM NO.I
HARZA PROJECT NO. 800A
HOLE NO. 74-36A
SAMPLE NO. 3
DEPTH : 64.4'-64.7'

SOIL TESTING SERVICES, INC.
111 PFINGSTEN ROAD
NORTHBROOK ILLINOIS

DRAWN	APPROVED	DATE	JOB No.
SZ	CHC	11-19-74	17326-C

PLATE B-2-17

UNIFIED CL/ SIFICATION SYSTEM



SAMPLE NO.	W.C.	LL	PL	PI	CLASSIFICATION
					Silty fine sand-Very light grey
					(SM)
					$C_u = D_{60}/D_{10} = 1.89$

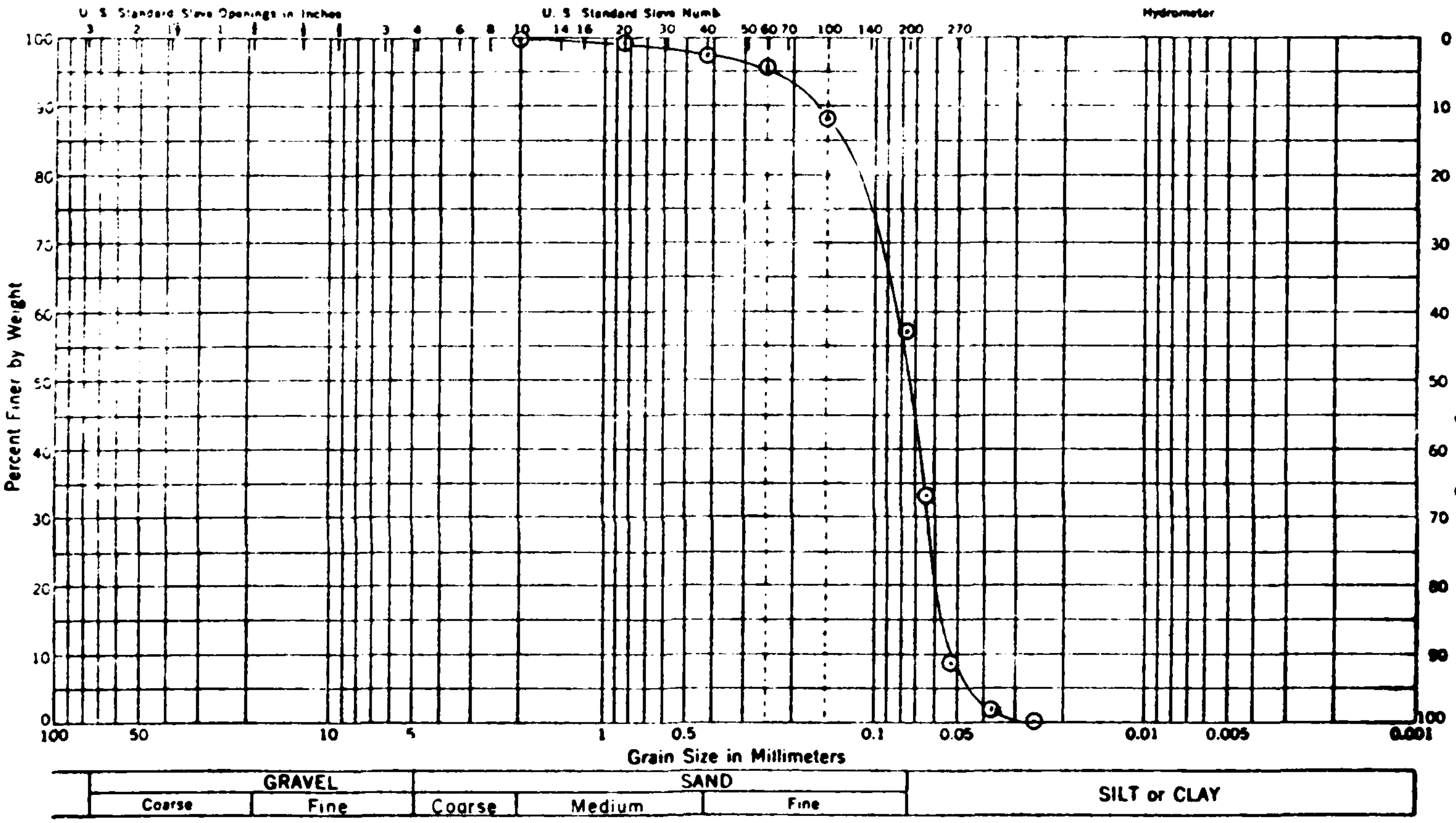
HARZA ENGINEERING COMPANY
LOCK & DAM NO. I
HARZA PROJECT NO.800A
HOLE NO. 74-36A
SAMPLE NO.4
DEPTH : 66.0'-66.4'

SOIL TESTING SERVICES, INC.
111 PFINGSTEN ROAD
NORTHBROOK ILLINOIS

DRAWN	APPROVED	DATE	JOB No.
SZ	CHC	11-19-74	17326-C

PLATE B-2-18

UNIFIED CL/ SIFICATION SYSTEM



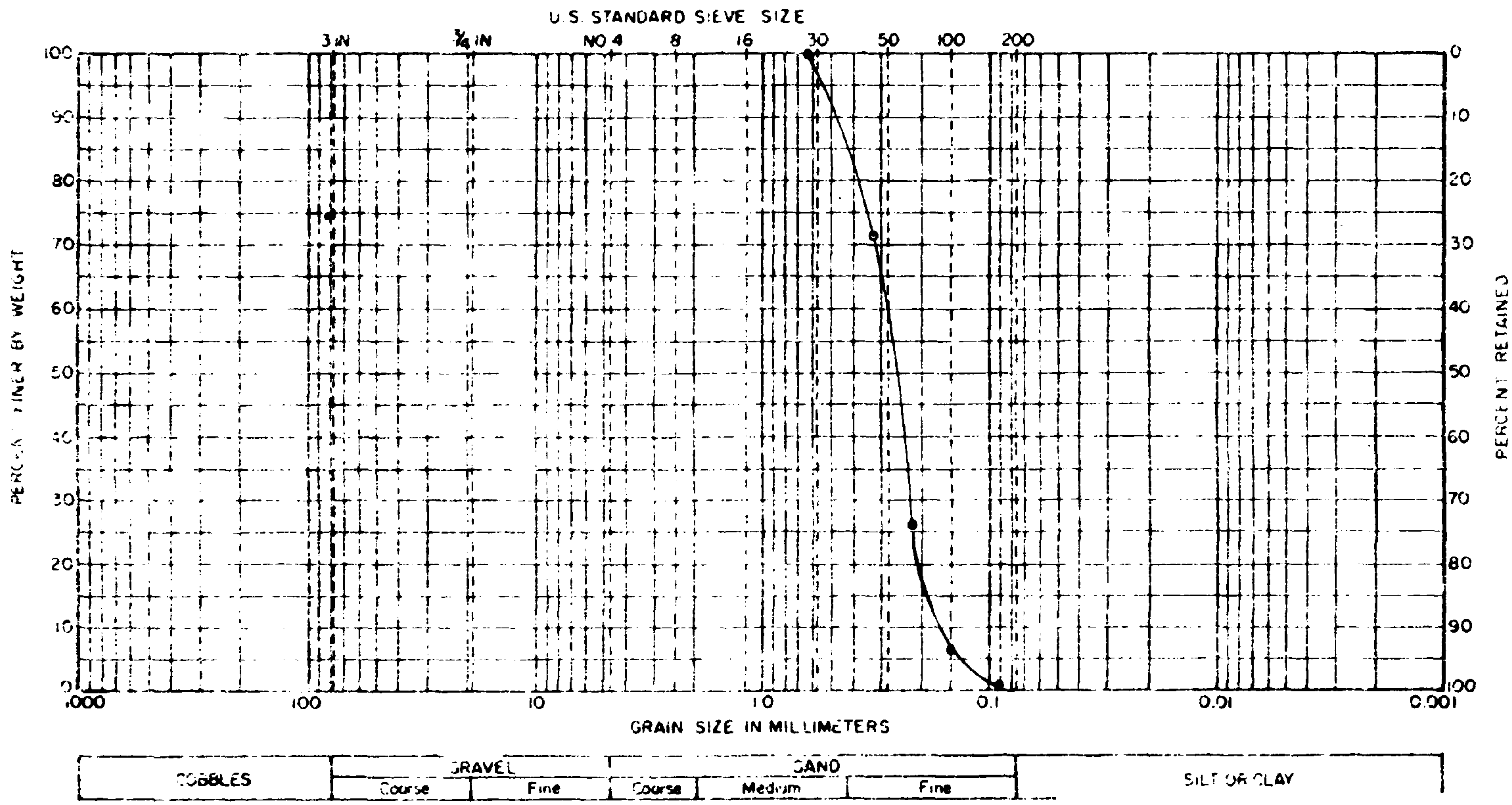
SAMPLE NO.	W.C.	LL	PL	PI	CLASSIFICATION
					Silt and fine sand -White(ML-SM)
					$C_u = D_{60}/D_{10} = 1.60$

HARZA ENGINEERING COMPANY
LOCK & DAM NO.1
HARZA PROJECT NO.800A
HOLE NO.74-36A
SAMPLE NO. 5
DEPTH : 70.8'-71.4'

SOIL TESTING SERVICES, INC.
111 PFINGSTEN ROAD
NORTHBROOK ILLINOIS

DRAWN	APPROVED	DATE	JOB No.
SZ	CHC	11-19-74	17326-C

PLATE B-2-19



Sample No.	Elev. or Depth	Classification	Nat WC	LL	PL	PI	
							$C_u = D_{60}/D_{10} = 1.75$

GRADATION CURVES

STUDY of ALTERNATIVES for REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO.1

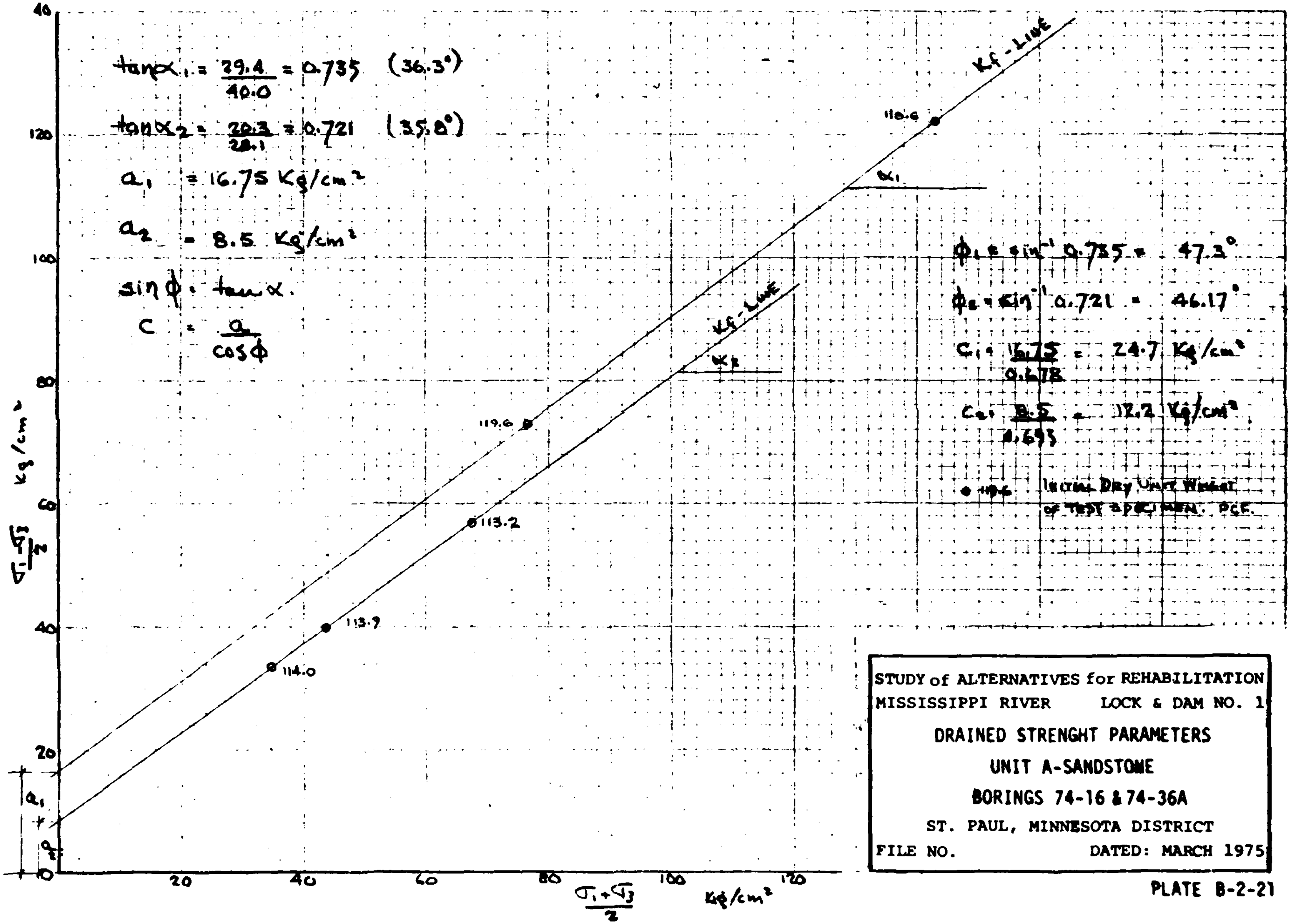
GRAIN SIZE DISTRIBUTION

SAND FROM BLOCK SAMPLE

FROM FORD MINE

ST. PAUL, MINNESOTA DISTRICT
FILE NO. DATED: MARCH 1975

PLATE B-2-20



STUDY of ALTERNATIVES for REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO. 1

DRAINED STRENGHT PARAMETERS

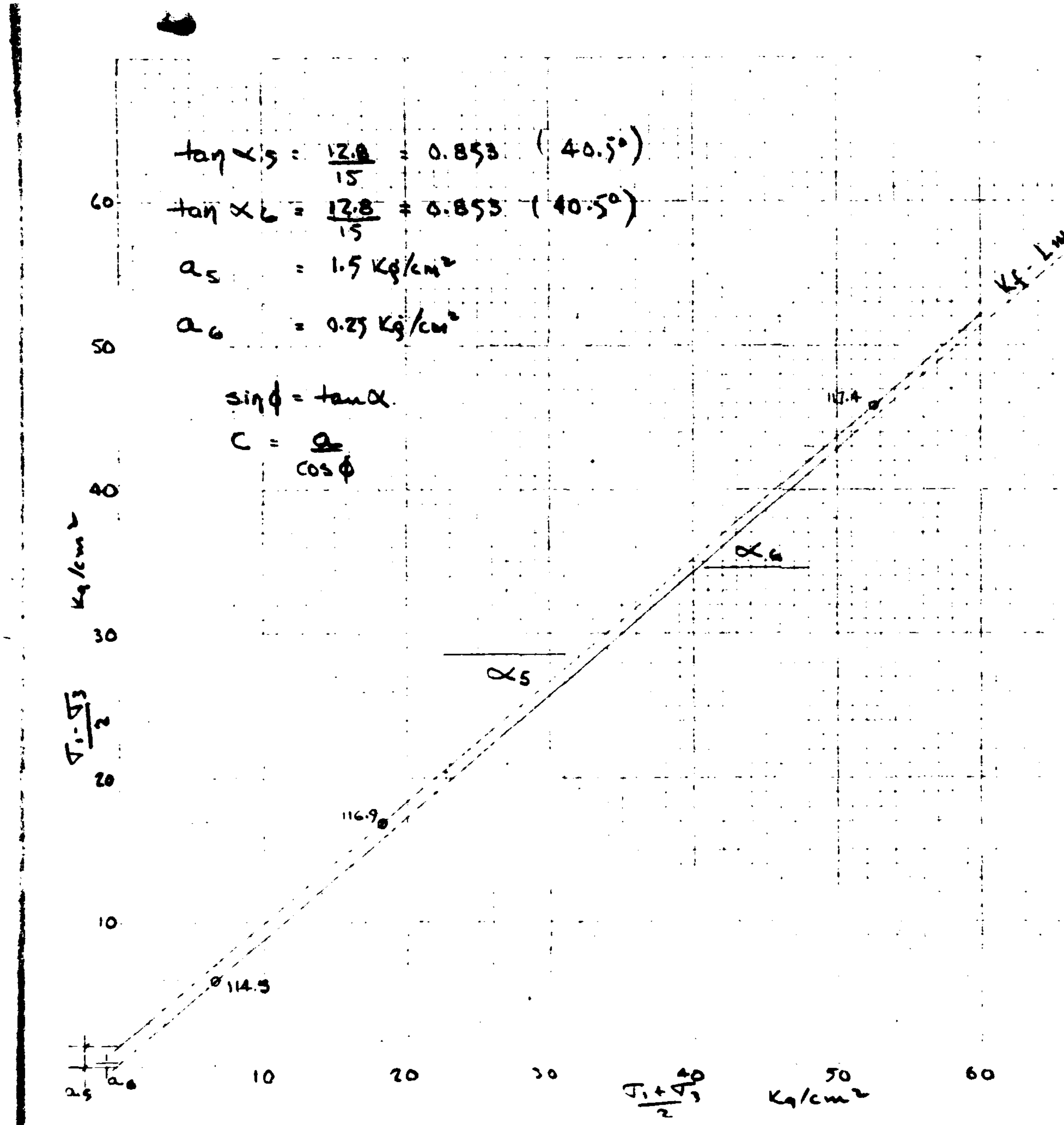
UNIT A-SANDSTONE

BORINGS 74-16 & 74-36A

ST. PAUL, MINNESOTA DISTRICT

FILE NO. DATED: MARCH 1975

PLATE B-2-21



STUDY of ALTERNATIVES for REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO. 1

DRAINED STRENGHT PARAMETERS

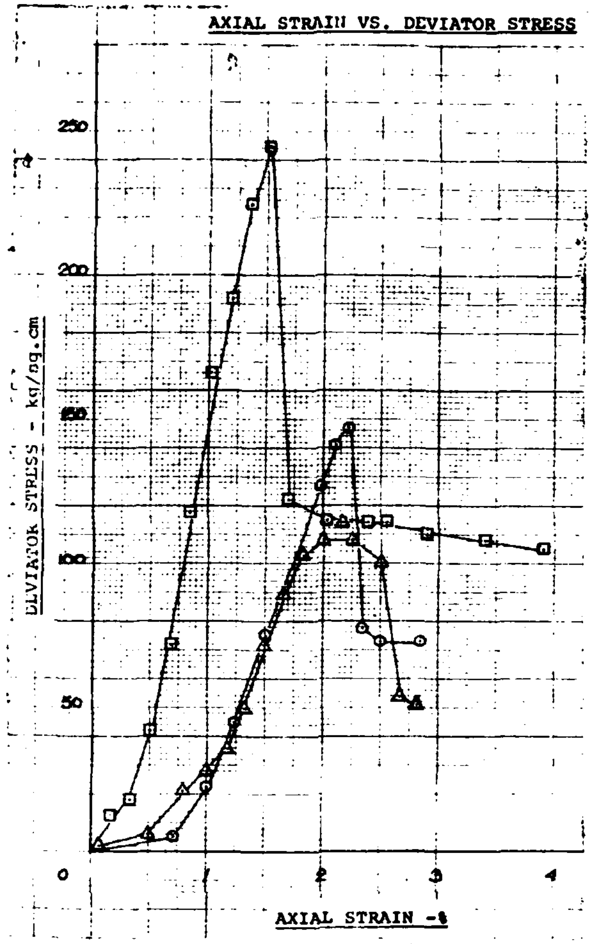
UNIT A-SANDSTONE

BLOCK SAMPLE FROM FORD MINE

ST. PAUL, MINNESOTA DISTRICT

FILE NO. DATED: MARCH 1975

PLATE B-2-22



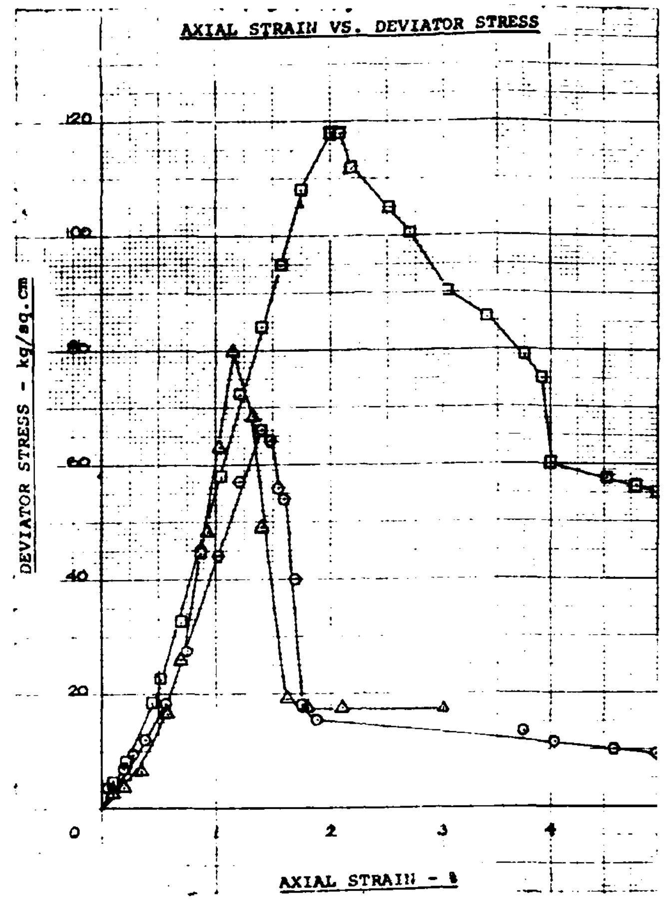
LEGEND

○—○ 3.52 kg/sq.cm Consolidation Pressure
△—△ 10.56 kg/sq.cm Consolidation Pressure
□—□ 21.10 kg/sq.cm Consolidation Pressure

SAMPLE NO.	DEPTH FT.	HEIGHT IN.	DIA. IN.	W/C %	DRY γ_d PCF	σ_3 kg/sq.cm	MAX. ε %	MAX. σ_d kg/sqcm
1	13.9-15.1	5.25	2.45	12.9	119.6	3.52	2.22	147
3	40.7-41.8	12.00	6.00	12.0	113.2	10.56	2.17	114
2	38.0-39.2	11.75	6.00	12.4	118.5	21.10	1.53	245

Strain Rate = 0.0050 in. per min. 30 psi of back pressure was applied during consolidation and shear.
Sample : Friable Sandstone * Box No.7 Sample No.1 (side portion) was disturbed and trimed to a smaller size

BORING 74-16



LEGEND

o—o 1.76 kg/sq.cm Consolidation Pressure
△—△ 3.52 kg/sq.cm Consolidation Pressure
□—□ 7.04 kg/sq.cm Consolidation Pressure

SAMPLE NO.	INITIAL SAMPLE		DRY DENSITY	WATER CONTENT %		CHAMBER PRESSURE	CONSOLIDATION PRESSURE	STRAIN AT MAX	MAX DEV STRESS
	DIAMETER in	HEIGHT in	pcf	INITIAL	FINAL	kg/sqcm	kg/sqcm	$(\sigma_1 - \sigma_3)$%	PSI
3	2.775	5.325	114.0	17.0	16.6	1.76	1.76	1.40	66.2
4	2.000	4.312	113.9	17.2	16.4	3.52	3.52	1.14	80.2
5	2.700	5.735	116.9	14.7	16.2	7.04	7.04	2.00	118.0

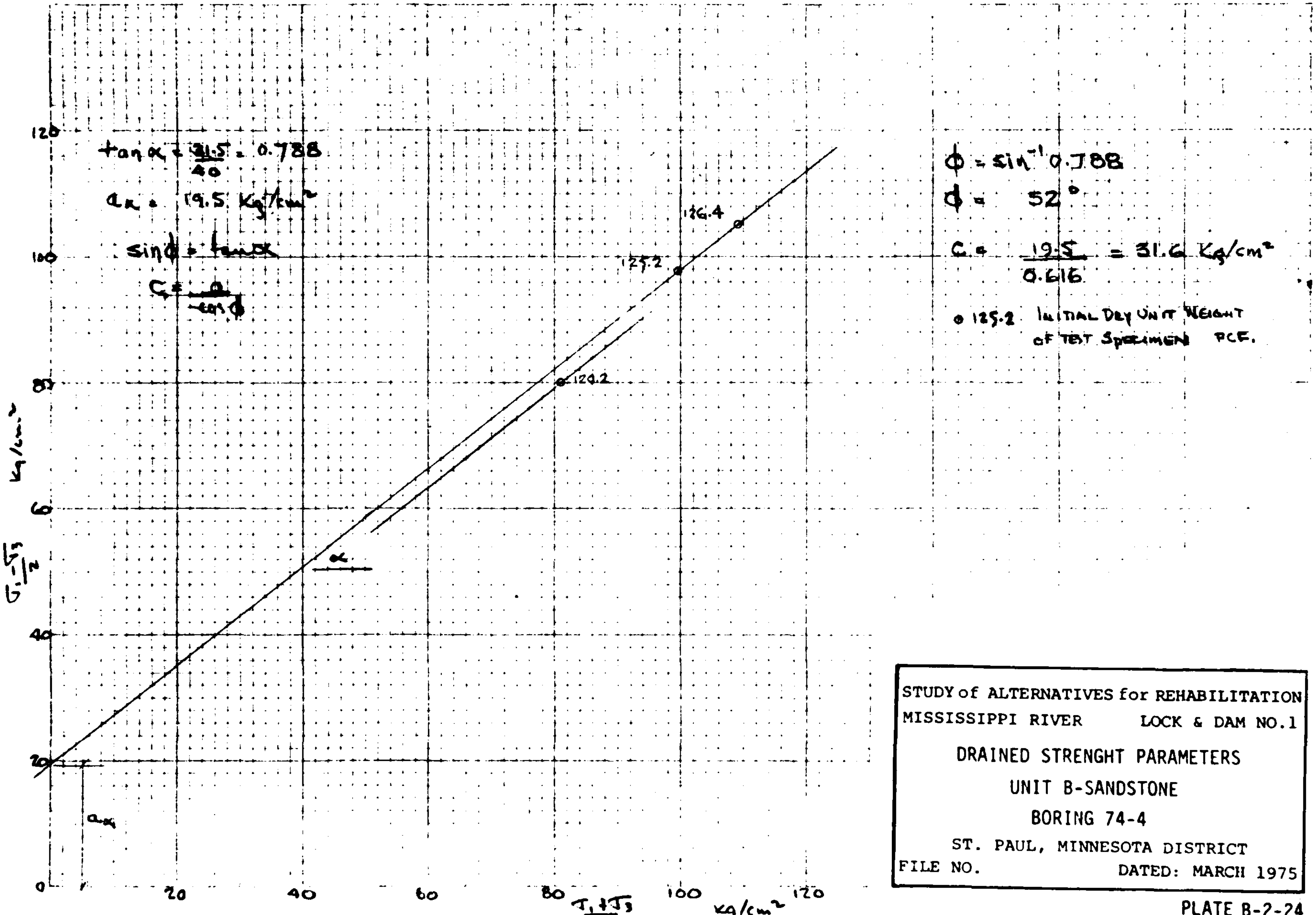
shear.
smaller size

SAMPLE : FRIABLE SAND STONE - POORLY CEMENTED
STRAIN RATE : 0.01 %/min.

BORING 74-36A

STUDY of ALTERNATIVES for REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO.1
CONSOLIDATED-DRAINED TRIAXIAL TESTS
SAMPLE DATA & STRESS-STRAIN CURVES
UNIT A-SANDSTONE
ST. PAUL, MINNESOTA DISTRICT
FILE NO. DATED: MARCH 1975

PLATE B-2-23

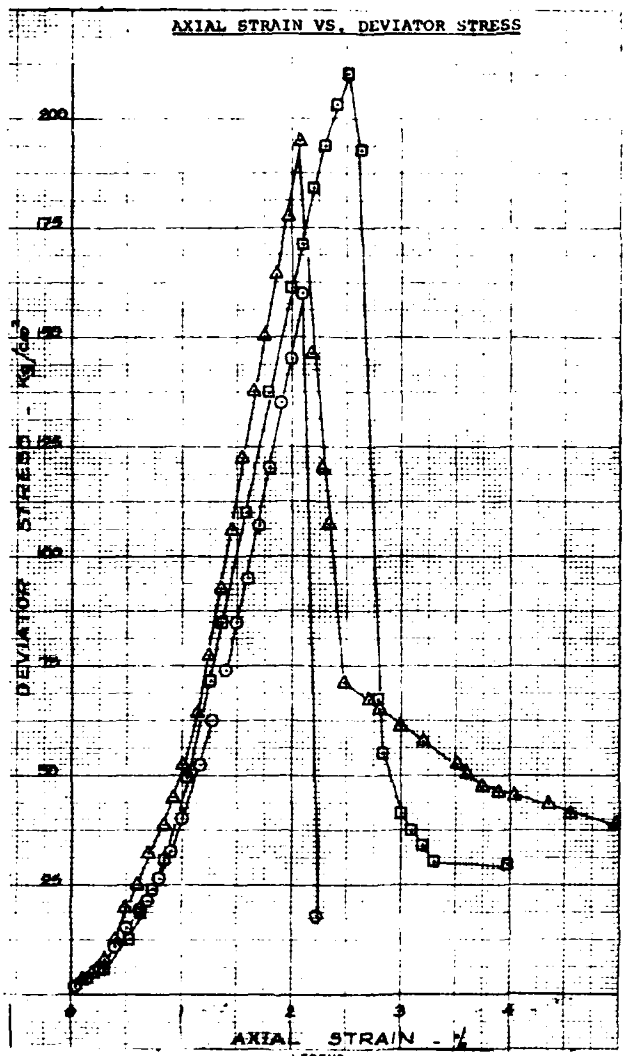


STUDY of ALTERNATIVES for REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO.1

DRAINED STRENGHT PARAMETERS
UNIT B-SANDSTONE
BORING 74-4

ST. PAUL, MINNESOTA DISTRICT
FILE NO. DATED: MARCH 1975

PLATE B-2-24



LEGEND

- ○—○ 1.05 kg/sq.cm Consolidation Pressure
- △—△ 2.10 kg/sq.cm Consolidation Pressure
- □—□ 4.20 kg/sq.cm Consolidation Pressure

INITIAL SAMPLE		DRY DENSITY pcf	WATER CONTENT %		CHAMBER PRESSURE	CONSOLIDATION PRESSURE	STRAIN AT MAX	MAX DEV STRESS
DIAMETER in	HEIGHT in		INITIAL	FINAL	kg/sq.cm	kg/sq.cm	($\sigma_1-\sigma_3$) %	kg/sq.cm
2.400	4.708	120.2	13.7	14.1	1.05	1.05	2.12	160
2.420	4.830	125.2	9.4	9.6	2.10	2.10	2.07	195
2.420	4.758	126.4	11.0	11.4	4.20	4.20	2.54	210

STRAIN RATE : 0.01 % /min.

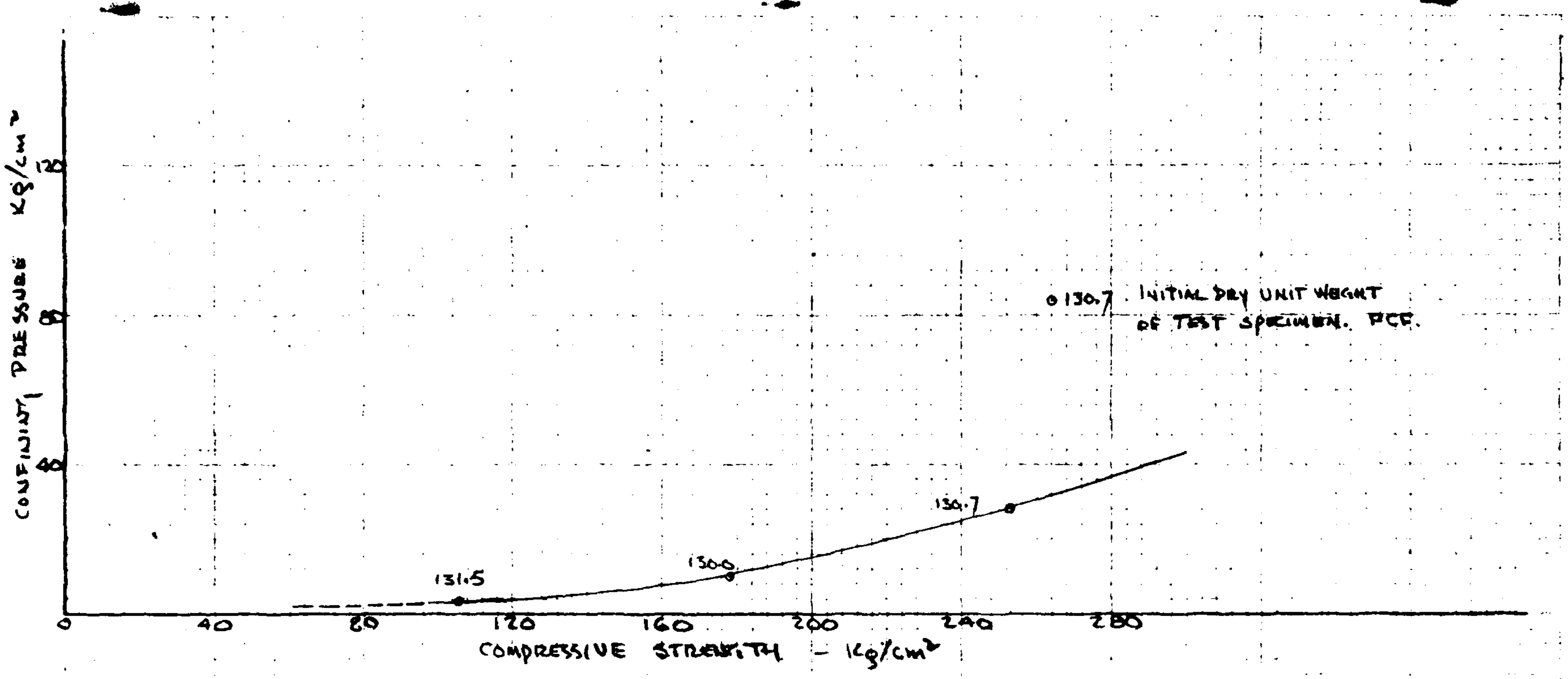
BORING 74-4
SA. 2 - 41.4' - 42.7'

STUDY of ALTERNATIVES for REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO. 1

CONSOLIDATED DRAINED TRIAXIAL TESTS
SAMPLE DATA & STRESS-STRAIN CURVES
UNIT B-SANDSTONE

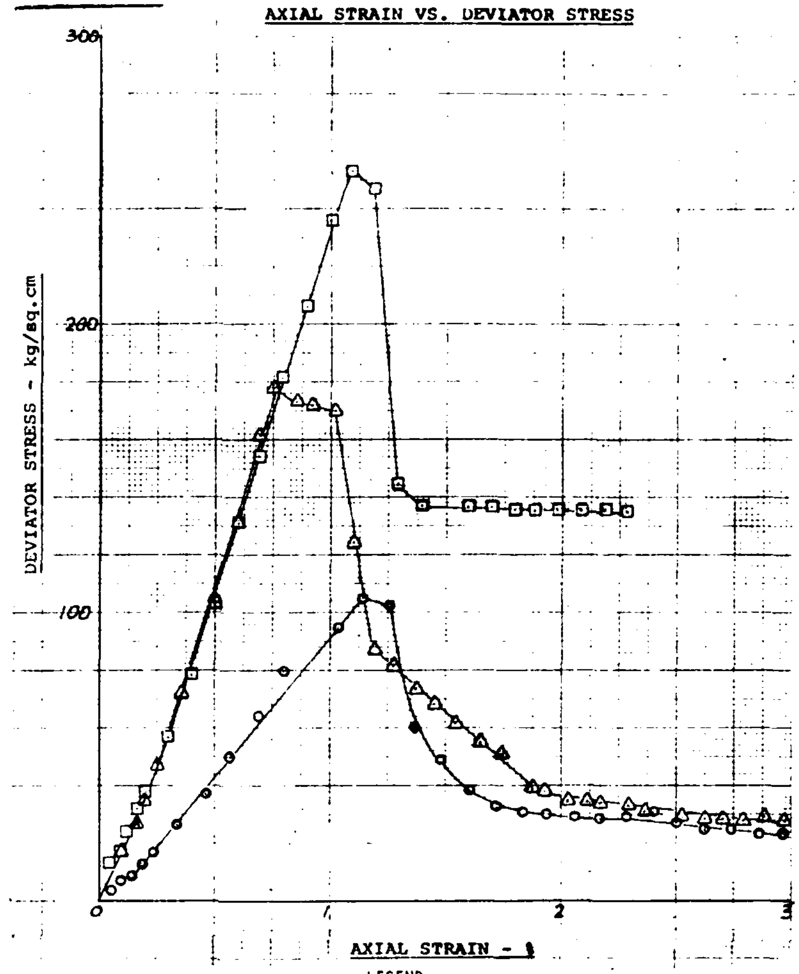
ST. PAUL, MINNESOTA DISTRICT
FILE NO. DATED: MARCH 1975

PLATE B-2-25



STUDY of ALTERNATIVES for REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO.1
UNCONSOLIDATED UNDRAINED STRENGTH PARAMETERS
UNIT B-SANDSTONE
BORING 74-16
ST. PAUL, MINNESOTA DISTRICT
FILE NO. DATED: MARCH 1975

PLATE B-2-26



LEGEND

○—○ 3.52 kg/sq.cm Consolidation Pressure
△—△ 10.56 kg/sq.cm Consolidation Pressure
□—□ 28.16 kg/sq.cm Consolidation Pressure

TEST NO.	BOX NO.	SAMPLE NO.	DEPTH FT.	HEIGHT IN.	DIA. IN.	W/C %	DRY γ_d PCF	σ_3 kg/sq.cm	MAX. ε %	MAX. σ_d kg/sq.cm
1	2	5	80	4.378	2.390	12.5	131.5	3.52	1.14	106
2	5	4	78	5.900	2.450	13.0	130.0	10.56	0.76	178
3	5	4	78	5.040	2.460	12.8	130.7	28.16	1.09	253

Strain Rate = 0.0050 in. per min.

BORING 74-16

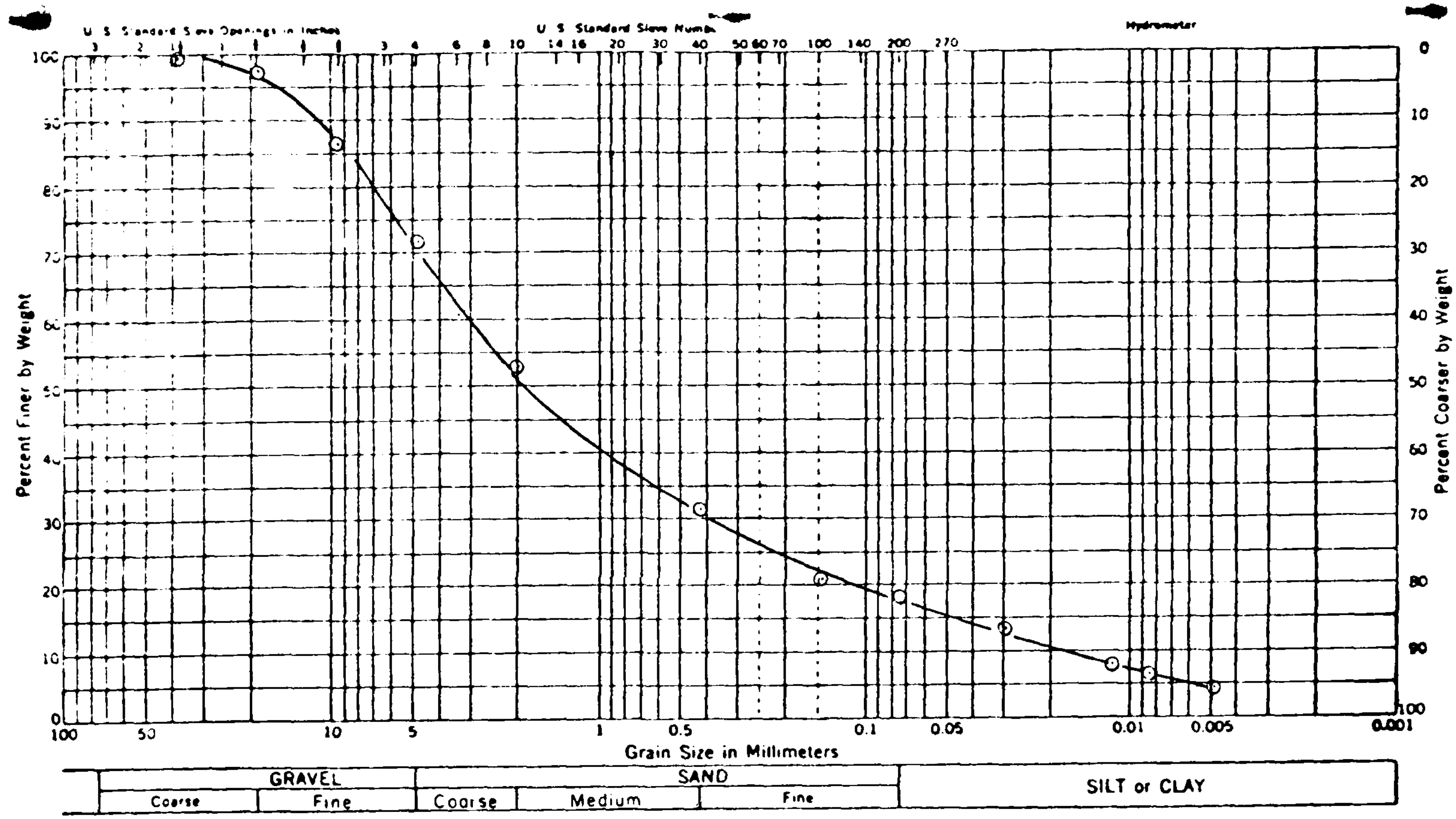
STUDY of ALTERNATIVES for REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO.1

UNCONSOLIDATED-UNDRAINED TRIAXIAL TESTS
SAMPLE DATA & STRESS-STRAIN CURVES
UNIT B-SANDSTONE

ST. PAUL, MINNESOTA DISTRICT
FILE NO. DATED: MARCH 1975

PLATE B-2-27

UNIFIED CLASSIFICATION SYSTEM



SAMPLE NO.	W.C.	LL	PL	PI	CLASSIFICATION
					Silty & Clayey Fine To Coarse Sand
					Some fine gravel - brown (SM-SC)
					Cu = D60/D10 = 160
					Sp.G. passing #4 sieve = 2.74
					Sp.G. retained on #4 sieve = [illegible]
					γd min = 110.5 pcf
					γd max = 146 pcf

STUDY of ALTERNATIVES
for REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO.1

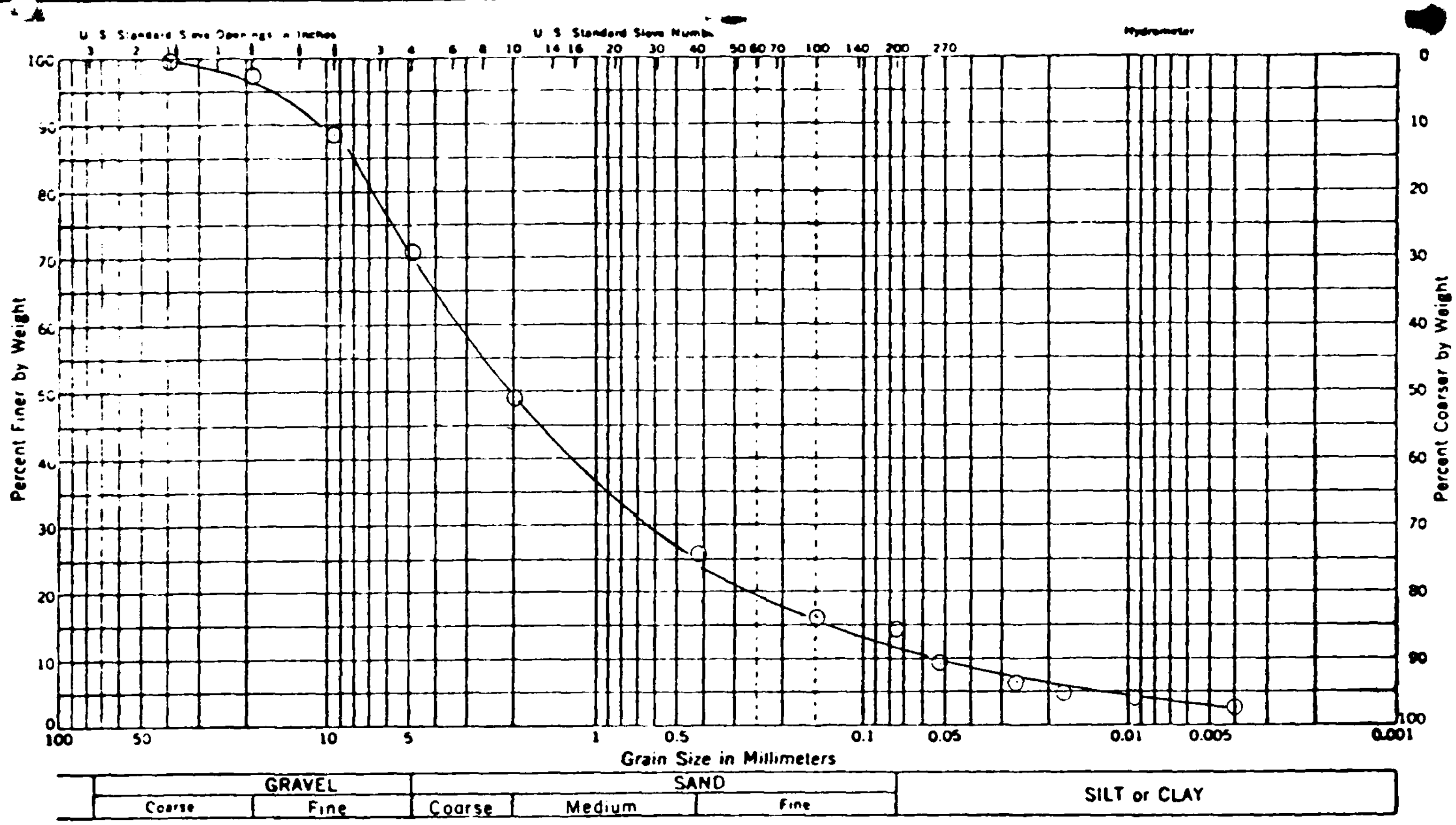
BORING 74-46A, RIVER WALL
COMBINED SAMPLES 16 TO 20

SOIL TESTING SERVICES, INC.
111 PFINGSTEN ROAD
NORTHBROOK ILLINOIS

DRAWN	APPROVED	DATE	JOB No.
SZ	JAC	1-11-75	1732-G

PLATE B-2-28

UNIFIED CLASSIFICATION SYSTEM



SAMPLE NO.	W.C.	LL	PL	PI	CLASSIFICATION
					[illegible] Fine to [illegible] Sand
					[illegible]
					$C_u = D_{60}/D_{10} = 60$
					Sp. G [illegible] #4 [illegible] = 2.75
					Sp. G [illegible] #4 [illegible] = 2.77
					[illegible] min = 110.6 pcf
					[illegible] max = 145 ([illegible])

= 153 (wet method)

STUDY of ALTERNATIVES
for REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO.1

BORING 74-82, RIVER ALLUVIUM
COMBINED SAMPLES 1 TO 4

SOIL TESTING SERVICES, INC.
111 PFINGSTEN ROAD
NORTHBROOK ILLINOIS

DRAWN	APPROVED	DATE	JOB No.
SZ	JAC	1-11-75	17326-

PLATE B-2-29

SUPPLEMENT A

APPENDIX B-2

RESULTS OF TESTING PROGRAM ON THREE HQ CORE SAMPLES FROM UNIT "C" - ST. PETER FORMATION MINNEAPOLIS, MINNESOTA

Prepared for

Harza Engineering Company

Chicago, Illinois

800-A

by

Alberto S. Nieto

January 6, 1975

N-40

1. Summary and Conclusions

1) A testing program was undertaken on three HQ core samples containing "soft" seams and occurring at the top of the Unit "C" siltstone, St. Peter Formation, Minneapolis, Minnesota. The testing was performed at the request of Mr. B. I. Maduke of Harza Engineering Company, Chicago. The aim of the program was to furnish shear strength parameters to be used in the reevaluation of the foundation conditions under Locks and Dam No. 1.

2) The testing program included drained direct shear tests, grain size analysis, Atterberg limits and x-ray analysis. The direct shear tests were performed in the University of Illinois direct shear apparatus -- a device capable of attaining shear displacements up to 3 in. without the uneven normal stress distributions common in conventional direct shear machines.

3) Tests were performed on the three "soft" seams, on ground-up siltstone, and on siltstone surfaces. Both the "soft" seams and the ground-up siltstone presented rather similar mineralogical and mechanical properties. Both of these materials can be characterized as CL soils, with about 50 % silt-sized particles and 30 % low-activity, clay-sized particles. The clay minerals are mostly kaolinite and illite. Individual values of residual friction angles varied from 18° to 24°; residual failure envelopes had values between 18° and 23°. A larger variation was observed between residual friction angles of "soft" seam material from two different samples than between the angles of "soft" seam material and ground-up siltstone from the same sample. Residual friction angles for rock (siltstone) surfaces also fell in the above range of values.

4) The writer submits that the "soft" seam consistently found at the top of Unit "C" is not a depositional feature but rather a mylonite formed by the shearing and grinding of the siltstone. This grinding is the result of displacements toward the center valley of the material overlying Unit "C". The similarity in mineralogy and residual resistance for the "soft" seam and the ground-up siltstone corroborates this interpretation.

5) It is recommended that a residual angle of friction of 23° be used in stability computations involving the "soft" seams, provided that the samples tested are representative of the conditions for the entire site. The choice of a residual friction angle close to the highest value tested allows for certain field conditions that are not met in laboratory tests. The recommended residual friction angle can be used in analysis for drained or undrained failure conditions.

2. Introduction

This report discusses the results of tests performed on three HQ samples from Unit "C", a siltstone member of the St. Peter formation, near Minneapolis-St. Paul, Minnesota. The samples were submitted to the writer by Mr. B. I. Maduke of Harza Engineering Company, Chicago. The results reported herein are to be used in the reevaluation of the foundation conditions under Locks and Dam No. 1 undertaken by Harza Engineering for the U. S. Corps of Engineers, Minneapolis-St. Paul District.

The main purpose of this testing program was the determination of relevant shear strength parameters for a "soft" seam that occurs in Unit "C", very close to the contact with the overlying sandstone member (Unit "B").

3. Sample Description and Testing Program

The three specimens arrived at the Engineering Geology Laboratory of the University of Illinois wrapped in aluminum foil and coated with wax. After examination the samples were placed in a humidity chamber.

Sample SR-1 74-34 was 8 in. long (depths 78.9 ft to 79.5 ft); the "soft" seam was approximately 2 in. thick and occurred at the top of the HQ core segment. Two zones could be discerned in the soft seam; the upper half was essentially a stiff/very stiff, gray clayey silt with a few small (1/16 in. - 1/4 in.) siltstone fragments, the lower half had a coarser "feel" and contained larger (1/4 in. - 1/16 in.) and more abundant siltstone fragments. The remaining core segment was a gray, moderately massive siltstone with a few thin, very fine-grained sandstone intercalations.

Sample S-7 74-16 was approximately 3 in. long (81.1 ft - 81.3 ft). The thickness of the soft seam could not be established because the entire sample was composed of a stiff, gray clayey silt matrix with over 60 % gray siltstone fragments of varying size.

Sample SR-1 74-88 was approximately 5 in. long (42.2 ft to 42.6 ft); the exact thickness of the "soft" seam was equally difficult to determine. In this case, however, it appeared that at least two thinner "soft" seams, of nature similar to the above, were present in the upper half of the core segment. Their combined thickness was about 1.5 in. and they also contained siltstone fragments of different sizes.

Table 1 shows the type and number of tests performed on each of the three samples. The testing program consisted of 15 drained direct shear tests, 5 Atterberg limits, 3 grain size analyses (hydrometer), and 4 x-ray analyses. Sample SR-1 74-34 was the more extensively tested. Drained direct shear tests were run on the upper, "clayey" zone of the "soft" seam; on the lower, "silty" zone of the seam; on ground-up siltstone; on lapped siltstone surfaces; and on induced fracture surfaces in the siltstone. Atterberg limits were obtained for the two seam zones and for the ground-up siltstone. Because of the amount of available material, grain size distribution was not determined for the seam material; however, a grain size distribution curve was obtained for the ground-up siltstone. Direct shear testing, Atterberg limits, and x-ray analysis were performed on the "soft" seams in samples S-7 74-16 and SR1 74-88. Both samples S7 74-16 and SR-1 74-88 yielded enough material to obtain a grain size distribution curves.

4. Testing Equipment and Procedure

4.1 General

The drained direct shear tests were performed in the University of Illinois direct shear device under the writer's supervision. The Atterberg limits and hydrometer tests were performed by Mr. A. K. Riggin at the Soil Mechanics Laboratories, Civil Engineering Department. The x-ray work was done by Mr. Harold C. Ganow at the Clay Mineralogy Laboratory, Geology Department. All testing, except for the direct shear testing, was performed essentially in accordance with ASTM designations.

The following sections deal specifically with the equipment and procedure employed in the direct shear tests.

4.2 Direct Shear Testing Equipment

Figure 1 shows an overall view of the University of Illinois direct shear device. Displacement rates ranging from 2.5×10^{2} in./min to 2.5×10^{-8} in./min can be attained with this machine. Another special feature of the device is a centralizing mechanism which maintains the normal load over the midpoint of the contact area undergoing shearing. A sample tested in a standard direct shear device, without this load centralizer, tends to develop large overturning moments after limited shear displacements. Figure 2(a) shows a standard arrangement with an eccentric normal load after a displacement δ. Figure 2(b) is a schematic diagram of the load centralizer and the shearing unit. A normal load, N, is applied to the loading head by means of dead weights. A horizontal shear force, S_A, develops as the lower half of the specimen is displaced with respect to the upper half which remains stationary. The loading head and upper half of the specimen are kept stationary by two linkages which are attached to the frame of the shear device at one end and to the loading head at the other end. The linkages are provided with load cells, accurate to 1 lb, which measure the reaction force, S_R, necessary to maintain the upper half of the specimen stationary while the lower half is displaced. The load centralizer consists of three small aluminum pumps whose net effect is to translate the point of load application a distance $\delta/2$ when the displacement of the lower half of the specimen is δ [see Fig. 2(a) and 2(b)]. In this manner, total displacements up to 3 in. can be attained without the uneven normal stress distributions common in conventional devices.

Four vertical potentiometers, accurate to 0.001 in., monitor vertical displacements of the sample, and a fifth potentiometer, accurate to 0.01 in., measures horizontal displacements. An xy plotter records a continuous force-horizontal-displacement curve as well as discrete vertical displacement points. Figure 3 is a typical test graph and corresponds to test SR-1 74-34 (N = 409 lb).

4.3 Direct Shear Testing Procedure

The details of the testing procedure varied depending on whether the seam material or the siltstone was tested.

When testing seam material, the specimen was prepared by separating manually the "soft" material from the siltstone fragments and, in most cases, adding small amounts of distilled water to facilitate reworking. A layer of reworked material approximately 1/8 in. thick was then spread on a Bedford limestone slab and another slab was placed on top of the layer. The limestone slabs had been previously lapped flat with #80 silicon carbide grit and were either 4 in. x 7 in. or 2 in. x 6 in. (The smaller slabs were used to obtain higher normal stresses). After controlling the layer of material for uniform thickness the specimen was placed in a water box and consolidated to the desired normal load. Doubling the load increments during consolidation usually results in an excessive squeeze of the sample between the limestone slabs. Thus, the specimen was loaded very gradually; at the beginning of the loading schedule the load increments were as little as 10 lb and were applied every 10 min.

The deformation rate for all "soft" material was 0.0025 in./min. For the present sample thickness (1/8 in.) this rate insures fully drained conditions in materials far less permeable than the silty "soft" seams being discussed (Nieto, 1974). Thus it is believed that in all cases the tests were run under fully drained conditions. Furthermore, it is well known that deformation rates many times faster than the one used for the soft material fail to affect the minimum shear resistance of even pure clay materials.

When testing siltstone surfaces, the core segment was either sawed or split parallel to the bedding. Each half of the sample was then mounted in Sulfaset[(1)] slabs in such manner that the rock surfaces were parallel to the base of the slabs. Figure 8 shows half of a sawn siltstone specimen (after testing) mounted in a Sulfaset slab. The specimen was immersed in the water box, and the assemblage was mounted in the direct shear device.

---

(1) Trademark for a gypsum-based, fast-setting, high-strength cement (Randustrial Corporation, Cleveland, Ohio).

Siltstone samples were sheared at a deformation rate of 0.025 in./min. Experience with rock samples similar to those presently reported indicates that both peak and residual shear resistances are not influenced significantly by displacement rates.

5. Test Results

5.1 General

Table 2 lists the reduced data for the direct shear tests and the Atterberg limits. The individual force-displacement curves are included as Appendix A. The two technical notes on the results of the x-ray analysis and the three grain size distribution curves are included as Appendix B.

5.2 Direct Shear Data Reduction

Figure 4 is a diagram of a typical force-displacement curve. Determination of the peak shear force, S_p, and horizontal displacement at peak, δ_p, are self-explanatory. Selection of the residual shear force and displacement at residual are not as straightforward. All tests were conducted up to a displacement such that the decrease rate in shear force was less than 2 lb per 0.5 in.[1] or, as was the case in a majority of tests, up to a displacement such that the shear force was ostensibly constant. In either case, the residual shear force was selected as the force corresponding to the lowest point, S_r, on the curve. The asymptotic approach of the curve to a minimum

---

(1) A drop of 2 lb was in all instances less than 2 % of the total shear force being measured.

value gives considerable latitude to the selection of a unique displacement point at which residual resistance was reached. Consequently, the following procedure was adopted. A horizontal line was traced on the graph corresponding to the minimum value, and a parallel line was drawn above the first one at a distance equivalent to 2 lb. This second line intersected the force-displacement curve while it was still decreasing and the intersection was fairly well defined; thus a displacement value, δ_n, could be obtained for that intersection. Because this displacement corresponded to a force 2 lb larger than the minimum force, an arbitrary distance of 0.250 in. was measured from δ_n. The value $\delta_r = \delta_n + 0.250$ in. was then assumed to be the displacement at which minimum resistance was reached (see Fig. 3).

5.3 Discussion of Test Results

5.3.1 General

The results of the testing program indicate that the residual shear resistance, Atterberg limits and mineralogy of the three samples of "soft" material are rather similar to one another and to the siltstone material. Discussed in the following sections are the individual results for the soft and rock materials with particular emphasis being placed on residual strength values.

5.3.2 Results of Tests on "Soft" Material

As shown in Table 2, three types of "soft" material were obtained from sample SR-1 74-34: a "clayey" portion designated SR-1 74-34 CL-1, a "silty" portion designated CL-2, and a portion obtained by grinding the siltstone and sieving it through a #70 sieve. This last portion, designated SR-1

74-34 GR, was mixed with distilled water and remolded for testing to a consistency similar to that of the "soft" seam material. Tests were performed on CL-1 and CL-2 at initial normal stresses of approximately 15, 30 and 60 psi. Two tests at initial stresses of 30 and 60 psi were performed on GR. The individual values of the residual friction angle, ϕ_r', varied from 19.8° to 23.6° for CL-1 and CL-2; an average ϕ_r = 22° was obtained for the ground-up siltstone. Typical displacements at residual for all three types of "soft" material were 1.0 ± 0.2 in. Figure 5 is a Coulomb plot (average shear stress on a horizontal plane vs average normal stress on same plane) of residual values for all direct shear tests on sample SR-1 74-34. The envelopes for CL-2 and GR have essentially no cohesion intercepts and their ϕ_r' angles are 23° and 21.5° respectively. The envelope for CL-1 has a ϕ_r' value of 22° and no cohesion intercept up to a normal stress of about 40 psi, from 40 psi to about 80 psi the envelope flattens to about 18° and presents a cohesion intercept of about 2.5 psi. The average peak values of shearing resistance for CL-1 and CL-2 were only 5 % greater than the residual values, for GR the average peak value was about 27 % greater than the residual value.

Table 2 also shows that CL-1 has somewhat higher liquid limit (L_w) and plasticity index (PI) than both CL-2 and GR. This may explain the lower ϕ_r' of CL-1 at higher stress levels. All three materials are classified as CL in the Unified Soil Classification system. Only GR yielded enough material to obtain a grain size distribution curve (see Appendix B). The curve indicated the following approximate percentages of grain sizes:

Fine Sand	14 %
Silt	54 %
Clay	32 %

X-ray analyses of CL-1 and GR showed no significant difference in the mineralogy of the two materials. Essentially, they consisted of about 50 % kaolinite and illite and approximately 50 % of a slightly expansive random mixture of illite and chlorite. Neither sample presented highly expansive clay minerals of the smectite (montmorillonite) type (see Appendix B).

Testing on samples S7 74-16 and SR-1 74-88 will be discussed next. Two direct shear tests were run on each sample with initial normal stresses of approximately 20 and 50 psi. Their residual envelopes have ϕ_r' values of 18° (S7 74-16) and 23° (SR-1 74-88) with no cohesion intercepts; they are shown in Figs. 6 and 7. The 18° value was the lowest for the samples of "soft" material The displacements to residual for these two samples were comparable to those of sample SR-1 74-34. Sample SR-1 74-88 presented an average peak shear resistance only 8% higher than the residual value. On the other hand, the average peak value for sample S7 74-16 was 30 % higher than the residual value. Another unique feature of this sample was that peak was reached at 0.04 in.--the smallest horizontal displacement for all samples tested.

Table 2 shows that the Atterberg limits are remarkably similar for these two samples; both are also classified as CL materials. Equally remarkable is the similarity between their grain size distribution curves (see Appendix B). The approximate percentages of grain sizes are:

Fine Sand	20 %
Silt	48 %
Clay	32 %

Finally, the x-ray analysis indicated that the mineralogy of both samples is virtually the same. As was the case with sample SR-1 74-34, they consist of approximately 50 % of kaolinite and illite and 50 % of a slightly expansive mixture of illite/chlorite. Again, no smectite was detected in either sample.

In brief, the results of the present testing program indicate that the "soft" seam material and the ground-up siltstone have similar properties. The materials can be collectively described as CL soils, with 50 % of silt-size particles and 30 % of low-activity, clay-size particles mostly of the kaolinite and illite type. Their residual friction angle varies between 18° and 23°; however, there seems to be less variation in ϕ_r' values between "soft" seam material and ground-up siltstone for a given sample (SR-1 74-34) than between "soft" seam materials from different samples (S-7 74-16 and SR-1 74-88).

5.3.3 Results of Direct Shear Tests on Siltstone Surfaces

Only three of the drained direct shear tests performed on siltstone surfaces are considered reliable and reported herein. Two of these tests were performed on lapped surfaces parallel to the axis of the HQ core (SR-1 74-34 LR). The initial normal stresses were 30 and 60 psi. The test at 30 psi did not display a peak value (see curve in Appendix A) and had a value of ϕ_r' = 24.1°. No residual value was reached in the test at 60 psi because the sample toppled and failed by crushing around the edges. Fig. 8 is a photograph of half of the failed specimen after a few days of drying under normal room conditions (notice that the siltstone shows a tendency to crack and flake). The third test was run at 30 psi on an induced fracture in the siltstone (SR-1 74-34 SR). This test did not present a peak value and had a ϕ_r' = 21.2°. After the test, the sample was observed to have disaggregate in part. (See Fig. 9).

6. Supplemental Notes

6.1 Geomechanical Interpretation of Available Data

The following interpretation is based on the limited data which is available to the writer, namely, the results of the present testing program and three geologic cross sections furnished by Mr. B. I. Maduke. One of those cross sections is reproduced here as Fig. 10.

The writer submits that the so-called "soft" seam (or seams) consistently found near the top of the Unit "C" siltstone is not a layer of material whose mechanical properties are due to inherent depositional conditions, but that it is a mylonite produced by the shearing of the siltstone of Unit "C". Shearing of the top of the siltstone unit was most probably caused by the displacement of all of the overlying material towards the center of the valley while the remaining portion of the unit stayed in place. The displacement of the overlying material was caused partly by a release of horizontal stresses in the rocks, as the river carved down the valley, and partly by the natural tendency of the valley slopes to spread laterally under the influence of gravity. Since the overlying Units "B" and "A'" are probably more competent than Unit "C" most of the horizontal displacements toward the center of the valley were accommodated in a narrow zone near the top of Unit "C". As a consequence, shearing and grinding of the top of the siltstone took place and the mylonite was formed.

The preceeding view is supported by two lines of evidence. First, the present testing program has indicated that the mineralogical and some of the mechanical properties of the "soft" seam material are very similar to those of the siltstone. Second, the mylonite occurs almost exactly at the same elevation as the bedrock valley bottom (the limit of river downcarving). This is precisely the position where mylonites of the type described above have been observed by engineering geologists (including the writer) in several valleys in Central U.S.A. and Canada (e.g., Ferguson, 1967; Brooker and

Anderson, 1970). These stress relief features along horizontal bedding planes are accompanied by another series of discontinuities either vertical or subparallel to the valley walls. The writer understands from his discussions with Mr. B. I. Maduke that these other stress relief joints also seem to have been disclosed by the exploration program at the present site. A third type of stress relief feature commonly associated with the preceeding ones is the buckling, thrusting and opening of the topmost beds in the center of the valley bottoms. This is caused by the horizontal compression induced by the valley walls on the valley bottom as the former move in toward the center of the valley. This last feature was not discussed with Mr. Maduke but its occurrence is conceivable at the present site.

6.2 Recommended Shear Strength Parameters for Stability Analysis.

It should be clearly understood that the following recommendations are based on the premise that the three samples tested are representative for the entire site under study. The writer is well aware that this premise may not be justifiable given the size of the project and the amount of testing performed. However he believes that the following considerations make results of this testing program much more useful.

The results of the drained direct shear tests indicate that two of the samples had ϕ_r' values of about 22°-23° and the third sample (S7 74-16) had a $\phi_r' = 18°$. A laboratory residual strength value of 18° is considered too conservative for the sliding stability analysis of the locks structures. It should be born in mind that the tests on the "soft" seam and ground-up siltstone were performed 1) on material which had all of the siltstone fragments removed and 2) on layers 1/8 in. thick. Thus, the minimum laboratory values were attained when under conditions in which all shear deformation is being accommodated in a very thin and flat layer of fine-grained material. Both of these conditions penalize the actual shear resistance available in the field. First, the presence of siltstone fragments in the "soft" seam interferes in the formation of the thin and flat layer needed for residual conditions. Second, it is understood that the actual thickness of the "soft" seam in the field is one to a few inches thick rather than 1/8 in. thick as the material was tested. It has been shown (Nieto, 1974) that the displacements necessary to attain peak and residual resistances in a clay seam increase exponentially with the thickness of the seam. Sample S7 74-16 reached a peak value of 24° at a horizontal displacement of 0.04 in. It is believed, however, that a 2-in. thick seam would reach its peak value at horizontal displacements of about 1.0 to 1.5 in. But since these horizontal displacements probably cannot be tolerated by the locks structures, it seems quite reasonable to use, in this particular case, the peak values of sample S7 74-16 for foundation stability studies. It is therefore recommended that, if the results of this limited testing program are representative of the conditions prevailing under the entire site, values of $\phi_r' = 23°$ and $C = 0$ be used in stability computations involving soft seam.

The above recommended values can be used assuming either drained or undrained failure conditions. The writer's experience with direct shear tests on similar clayey silts indicates that an increase in deformation rates by a few orders of magnitude above the values used in the present program, in fact, yields slightly higher values of shear resistance. This is particularly noticeable at residual conditions. This slight increase is believed to be a combination of positive pore water pressures and viscosity effects.

References

Brooker, E. W., and I. H. Anderson (1970), Rock Mechanics in Damsite Location, 5th Canadian Rock Mechs. Symp., Toronto; Mines Branch, Dept. Energy Mines and Resources, Ottawa, pp. 75-90.

Ferguson, H. F. (1967), Valley Stress Relief in the Allegheny Plateau, Eng. Geology, Vol. 4, n. 1, pp. 63-71.

Nieto, A. S. (1974), Experimental Study of the Shear Stress-Strain Behavior of Clay Seams in Rock Masses, Ph.D. Thesis, Department of Geology, Univ. of Ill., Urbana, Illinois.

TABLES AND ILLUSTRATIONS

TABLE 1 - TESTING PROGRAM

UNIT "C" - ST. PETER FORMATION

Sample	Direct Shear Test	Atterberg Limits	Grain Size Analysis	X-ray Analysis
SR-1 74-34 (seam)	6	2		1
SR-1 74-34 (ground rock)	2	1	1	1
SR-1 74-34 (rock surfaces)	3			
SR-7 74-16 (seam)	2	1	1	1
SR-1 74-88 (seam)	2	1	1	1

TABLE 2

DIRECT SHEAR TESTS - ST. PETER UNIT "C"

ST. PAUL-MINNEAPOLIS

FOR HARZA ENGINEERING COMPANY - CHICAGO

BY: ALBERTO S. NIETO
ENGINEERING GEOLOGIST
URBANA, ILLINOIS

Sample No.	Normal Load (lb)	Initial Dimensions (in.)	Shear Force Peak (lb)	Shear Force Residual (lb)	Horizontal Displacements Peak (in.)	Horizontal Displacements Residual (in.)	Normal Stress Peak (psi)	Normal Stress Residual (psi)	Shear Stress Peak (psi)	Shear Stress Residual (psi)	Stress Ratio Peak	Stress Ratio Residual	ϕ_p	ϕ_r	Displacement Rate 10^{-3} in./min.	Remarks
SR-1 74-34 CL-1	409	4 x 7	183	170	0.35	1.32	15.4	18.0	6.9	7.5	0.448	0.416	24.1	22.6	2.5	"Clayey" portion of seam--test run on 1/8-in. layer
(L_w = 46.9, PI = 27.8)	819	4 x 7	344	328	0.29	1.54	30.1	37.5	12.6	15.0	0.419	0.400	22.7	21.8	2.5	
	729	2 x 6	276	262	0.13	1.18	62.1	75.6	23.5	27.2	0.378	0.359	20.7	19.8	2.5	
SR-1 74-34 CL-2	409	4 x 7	186	179	0.13	0.87	14.9	16.7	6.8	7.3	0.456	0.437	24.5	23.6	2.5	"Silty" portion of seam--test run on 1/8-in. layer
(L_w = 40.4, PI = 23.1)	819	4 x 7	365	345	0.21	1.24	30.2	35.5	13.4	15.0	0.445	0.423	23.9	22.9	2.5	
	729	2 x 6	323	305	0.13	1.26	62.1	76.9	27.5	32.2	0.442	0.419	23.8	22.7	2.5	
SR-1 74-34 LR	129	D = 2.35	58(1)	58	0.10	0.10	31.5	31.5	14.1	14.1	0.448	0.448	24.1	24.1	25.0	Siltstone lapped with #80 grit
	271	D = 2.35	209	(2)	0.05	(2)	64.4	(2)	49.6	(2)	0.770	(2)	37.6	(2)	25.0	
SR-1 74-34 SR	129	D = 2.35	50(1)	50	0.10	0.10	31.5	31.5	12.2	12.2	0.387	0.387	21.2	21.2	25.0	Induced fracture in siltstone
SR-1 74-34 GR	225	2 x 6	120	92	0.07	1.05	18.9	22.7	10.1	9.3	0.535	0.409	28.2	22.3	2.5	Ground rock, passing #80 sieve--1/8-in. layer
(L_w = 39.7, PI = 18.8)	593	2 x 6	288	232	0.08	0.89	50.1	58.0	24.3	22.7	0.485	0.391	25.9	21.4	2.5	
S-7 74-16	547	4 x 7	245	183(3)	0.04	1.60(3)	19.6	25.3(3)	8.8	8.5(3)	0.449	0.335(3)	24.2	18.5(3)	2.5	Soft seam--1/8-in. layer
(L_w = 39.8, PI = 19.4)	593	2 x 6	252	197	0.04	0.73	49.7	56.7	21.1	18.7	0.425	0.331	23.0	18.3	2.5	
SR-1 74-88	547	4 x 7	262	240	0.10	0.84	19.8	22.2	9.5	9.7	0.477	0.438	25.6	23.7	2.5	Soft seam--1/8 in. layer
(L_w = 38.0, PI = 19.7)	593	2 x 6	274	254	0.08	0.99	50.1	59.2	23.1	25.3	0.461	0.428	24.8	23.2	2.5	

NOTES

(1) Force-displacement curve did not displayed peak value.

(2) Sample toppled and failed before residual was reached.

(3) Residual was not reached; actual value may be slightly (less than 1°) lower.

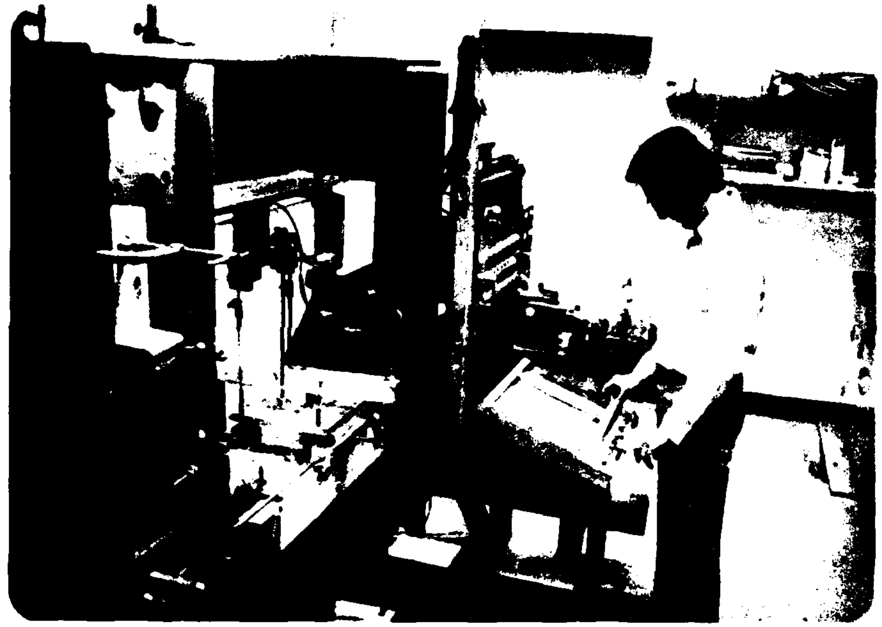


FIG. 1. OVERALL VIEW OF UNIVERSITY OF ILLINOIS DIRECT SHEAR DEVICE

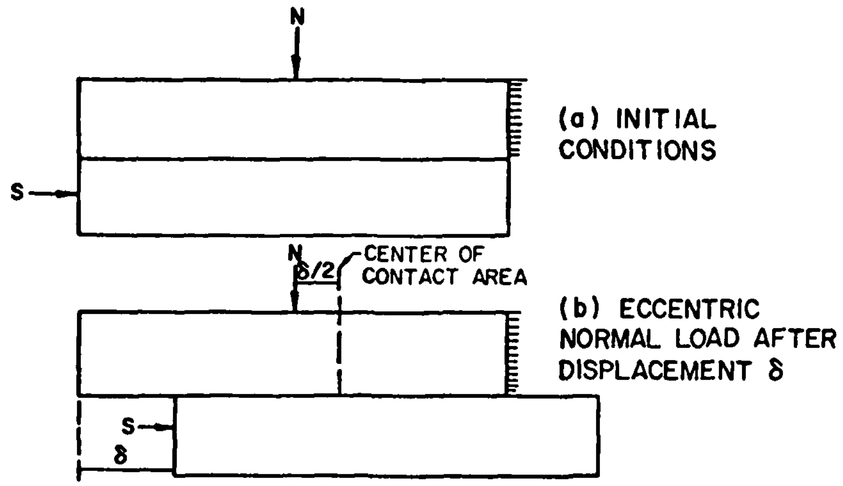
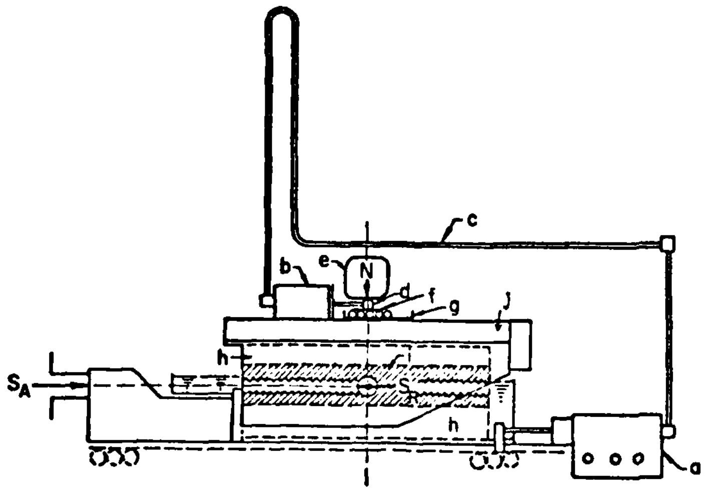


FIG. 2(a) STANDARD DIRECT SHEAR ARRANGEMENT WITH STATIONARY NORMAL LOAD SHOWING ECCENTRICITY OF NORMAL LOAD AFTER DISPLACEMENT δ



a) LATERAL PUMP, b) TOP PUMP, c) COPPER TUBING CONNECTION, d) BALL BEARING, e) HANGER, f) PLATE, g) SMALL ROLLER-BEARING BOX, h) SPACER, i) SAMPLE, j) HEAD

FIG. 2(b). SCHEMATIC DIAGRAM OF LOAD CENTRALIZER AND OTHER PARTS OF SHEARING UNIT

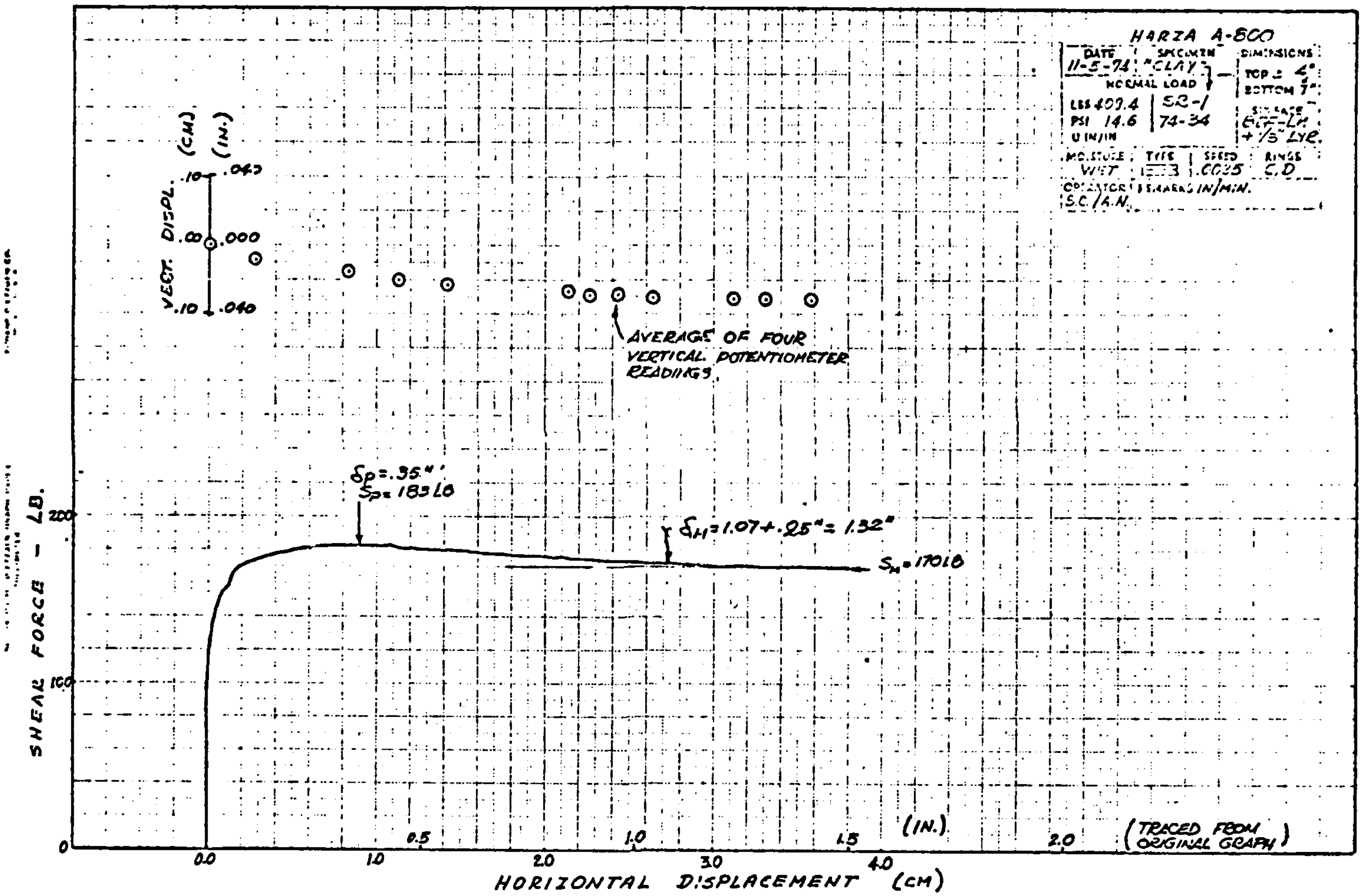


FIG. 3. EXAMPLE OF TEST GRAPH

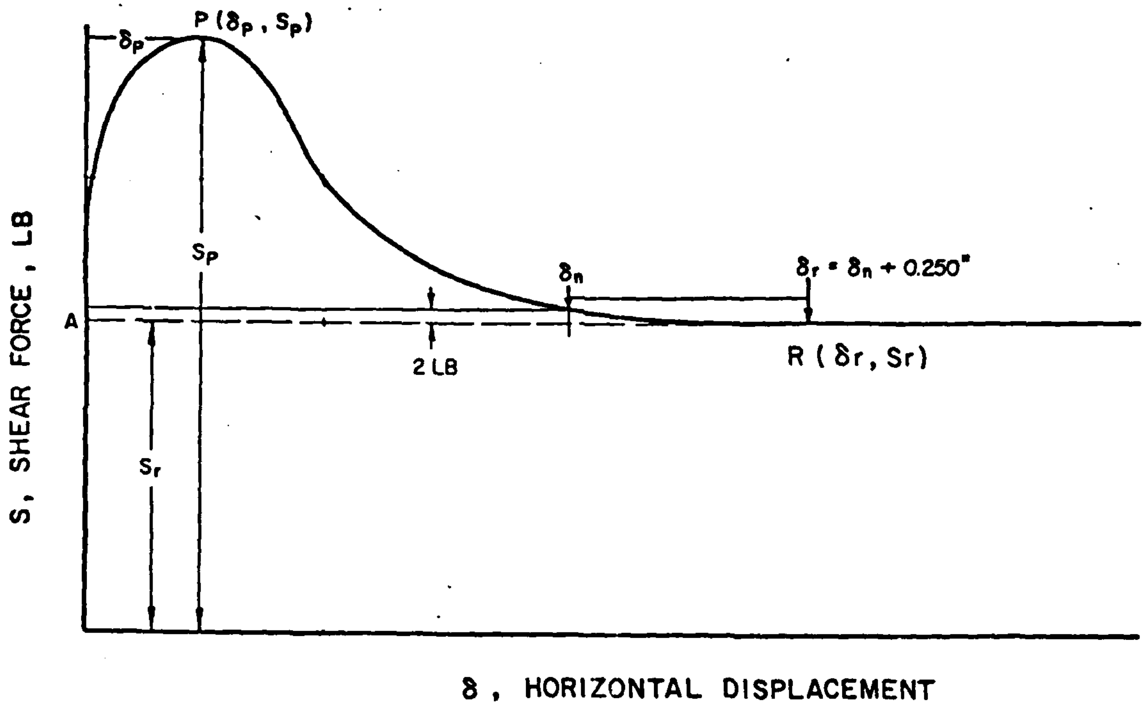


FIG. 4. DIAGRAM SHOWING ELEMENTS OF A FORCE-DISPLACEMENT CURVE

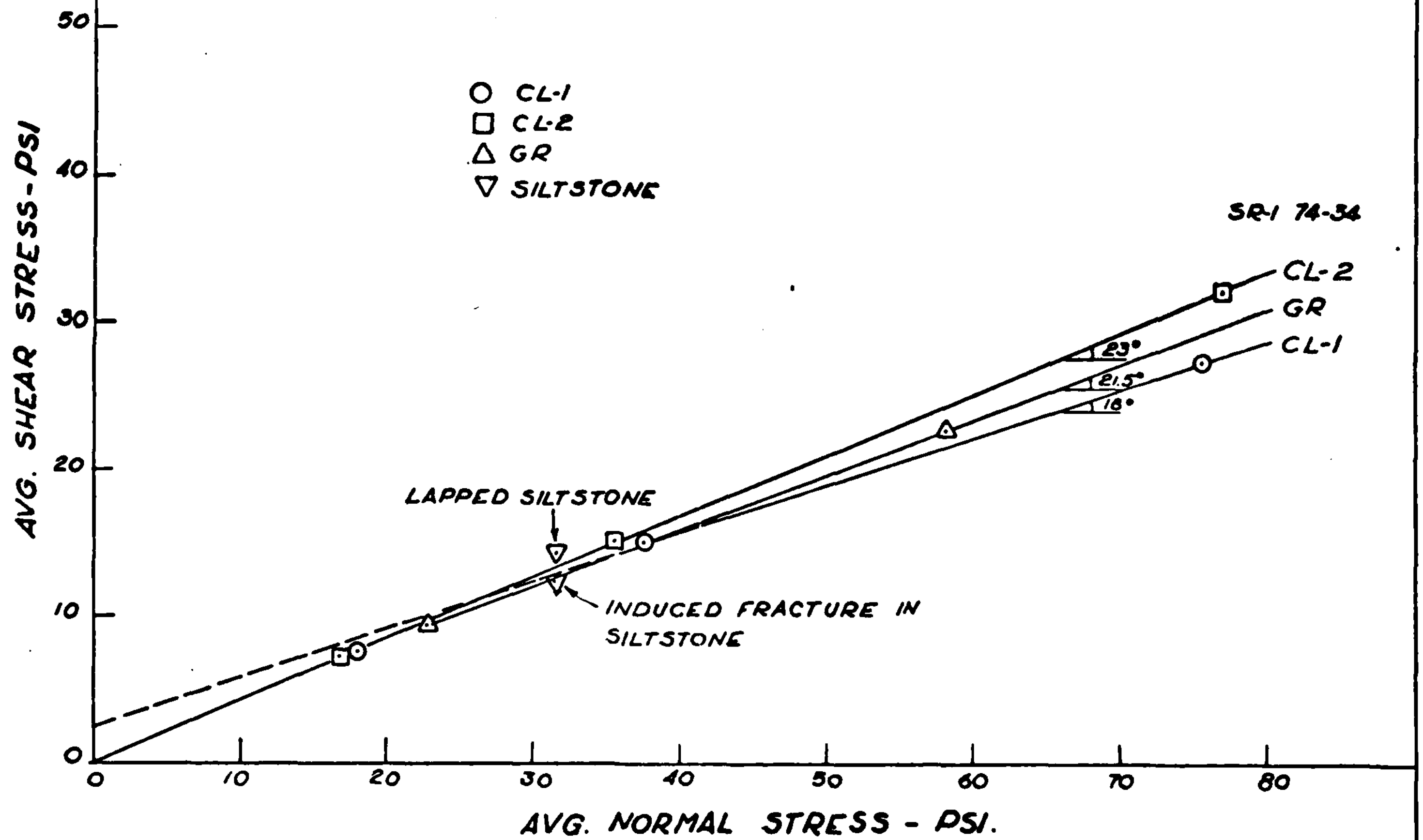


FIG. 5. COULOMB DIAGRAM - RESIDUAL SHEAR RESISTANCE ENVELOPES FOR SAMPLE SR-1 74-34

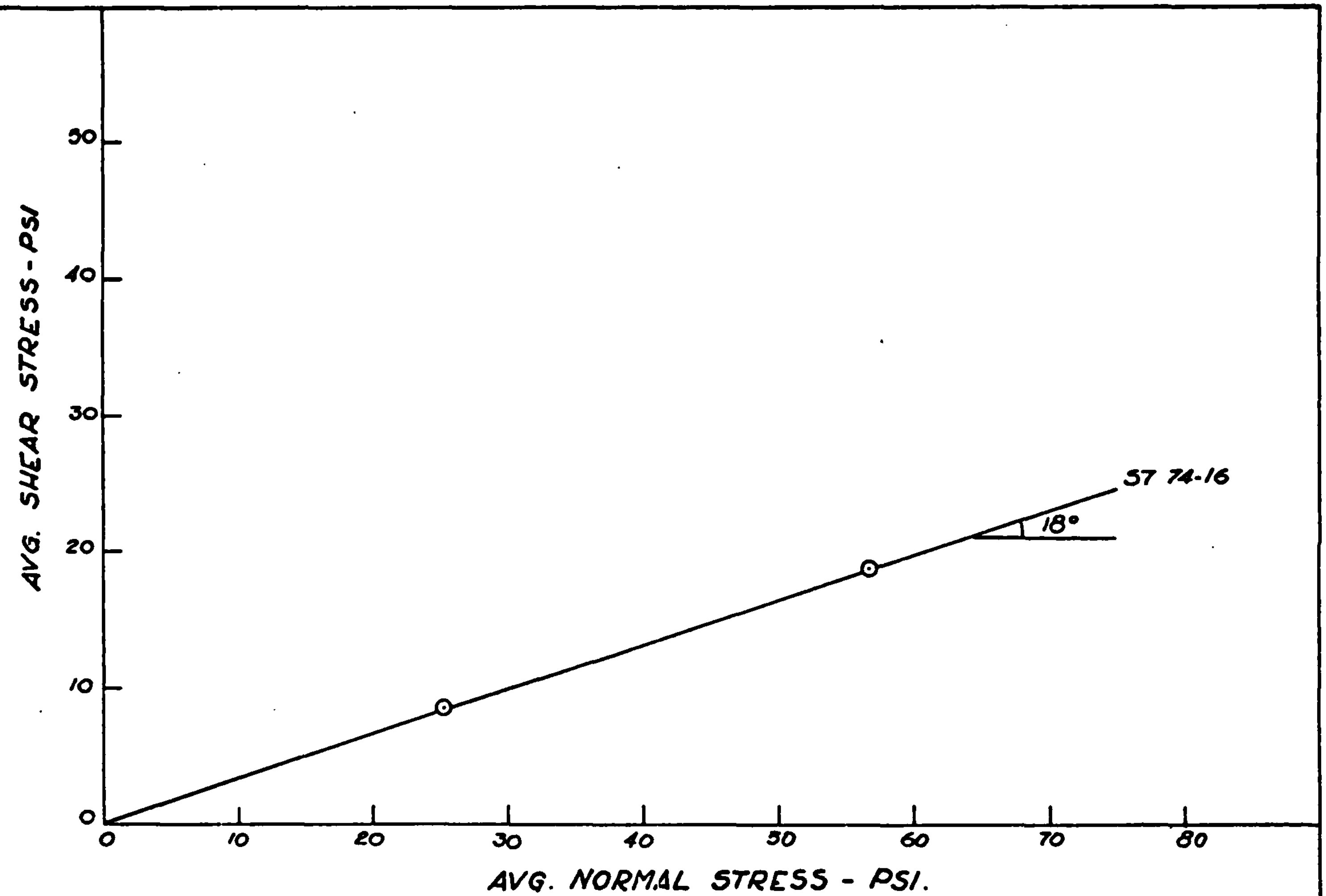


FIG. 6. COULOMB DIAGRAM - RESIDUAL SHEAR RESISTANCE ENVELOPE FOR SAMPLE S-7 74-16

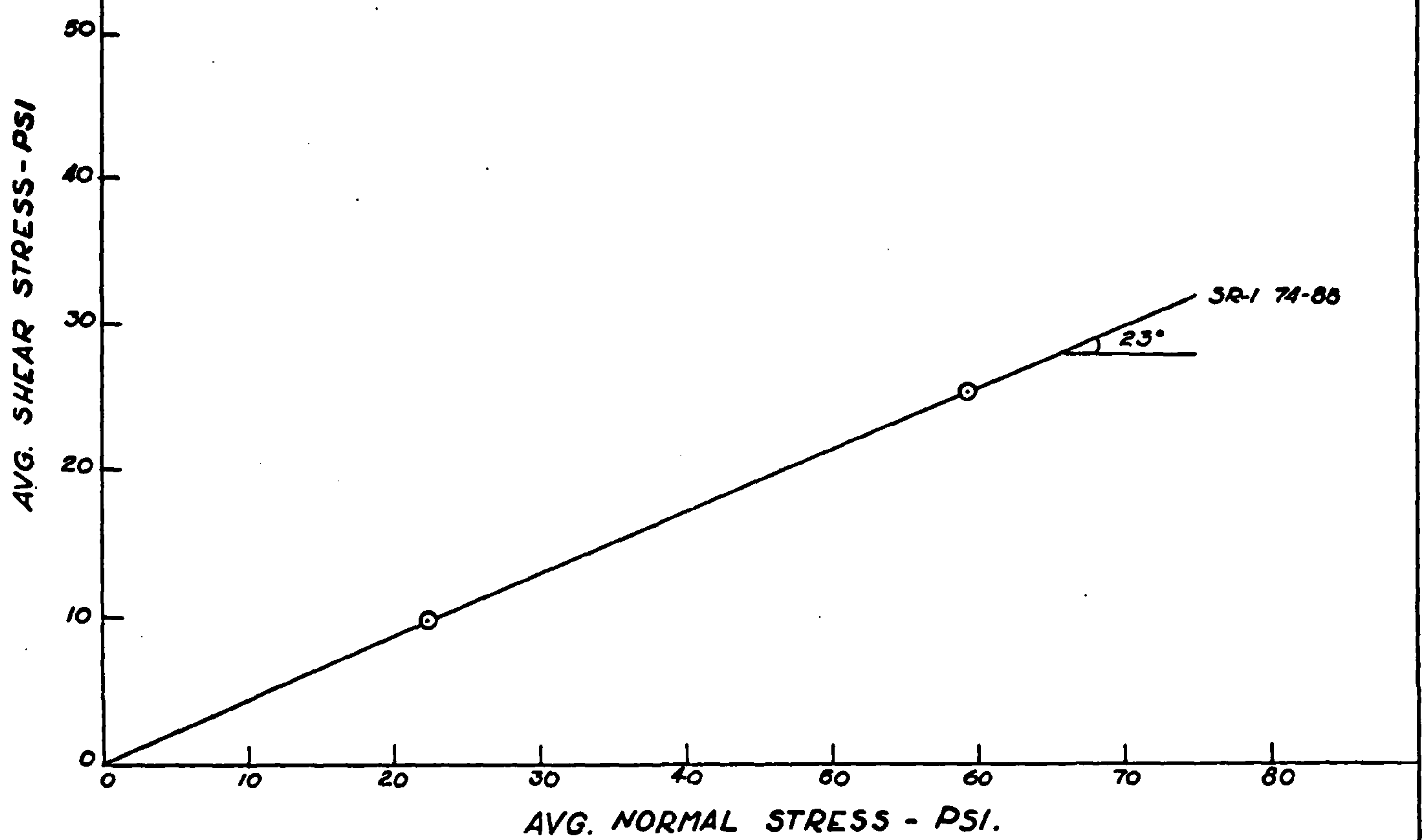


FIG. 7. COULOMB DIAGRAM - RESIDUAL SHEAR RESISTANCE ENVELOPES FOR SAMPLE SR-1 74-88

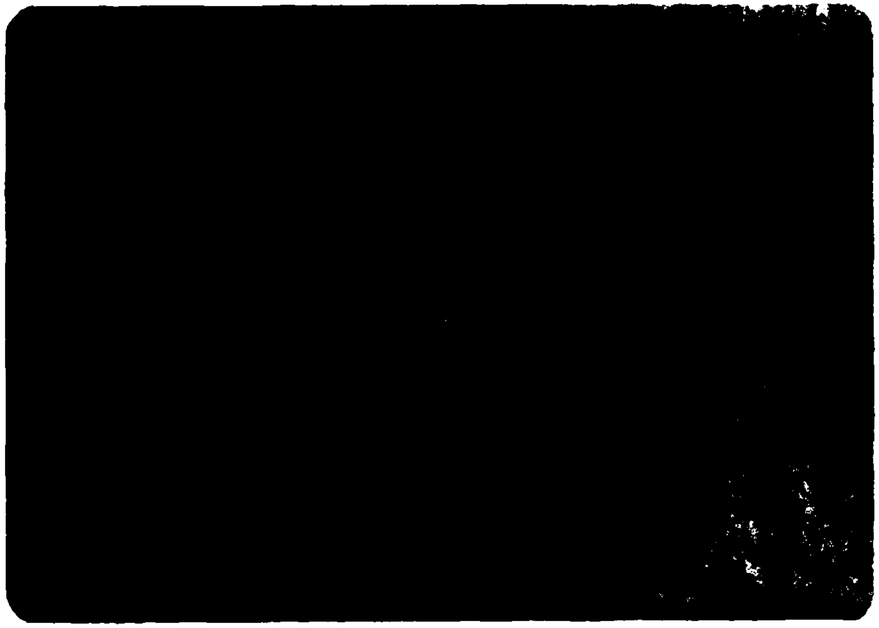


FIG. 8. HALF OF SPECIMEN SR-1 74-34 LR (σ = 60 psi) AFTER FAILURE

FIG. 9. HALF OF SPECIMEN SR-1 74-34 SR (σ = 30 psi) AFTER TESTING

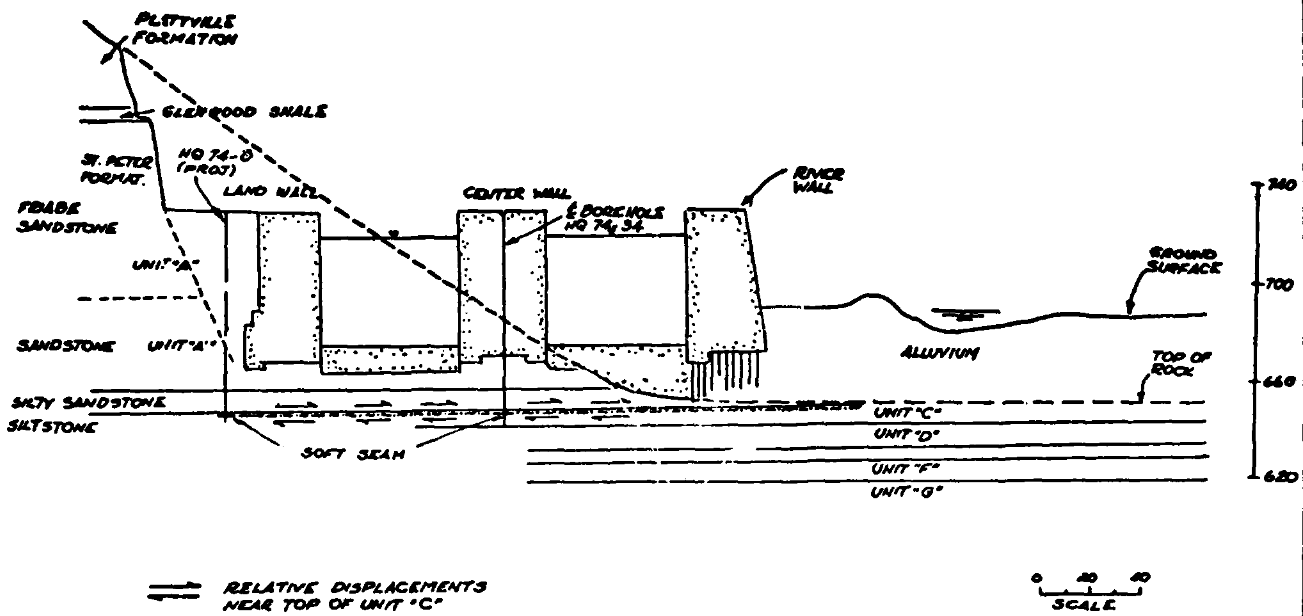
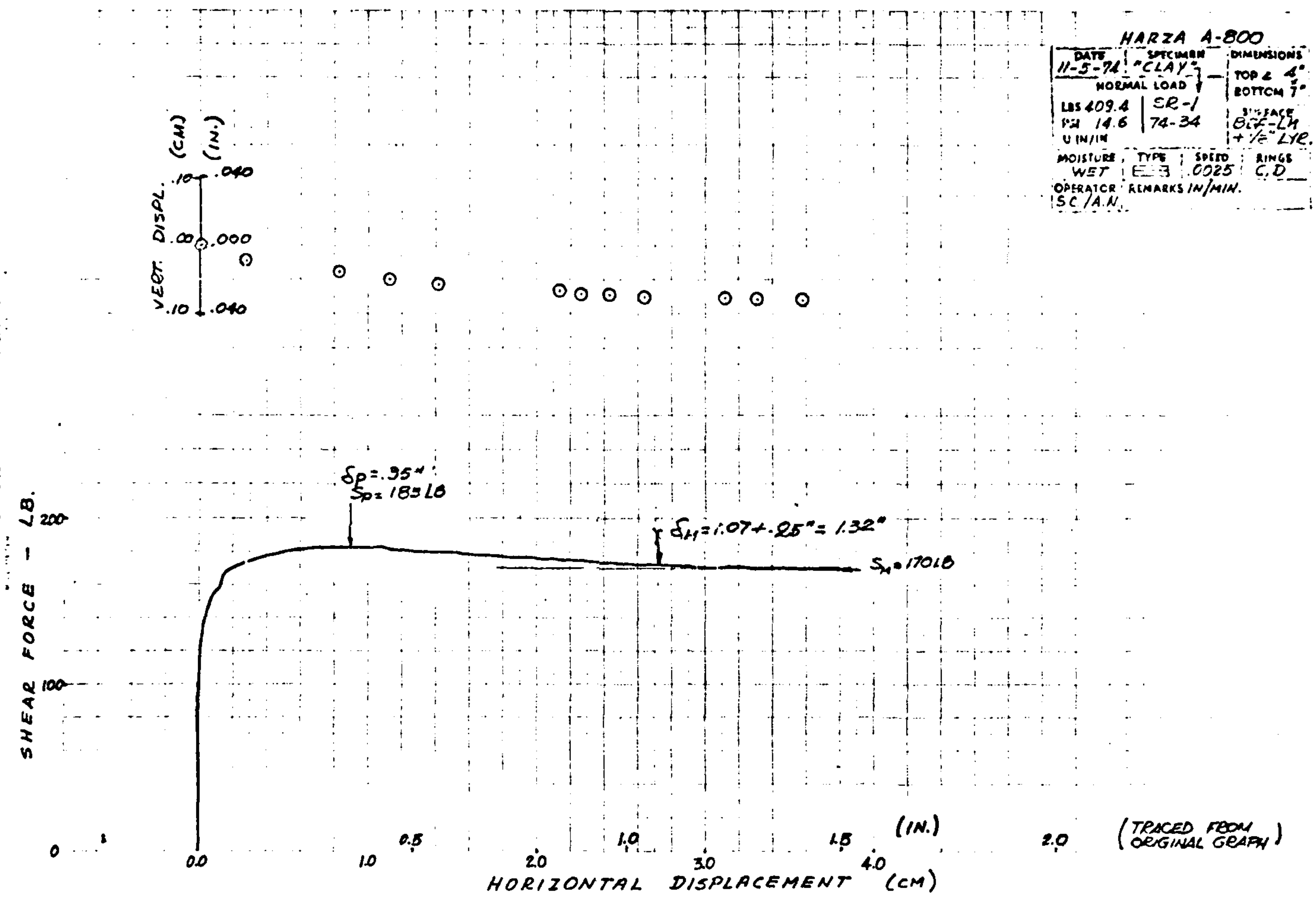
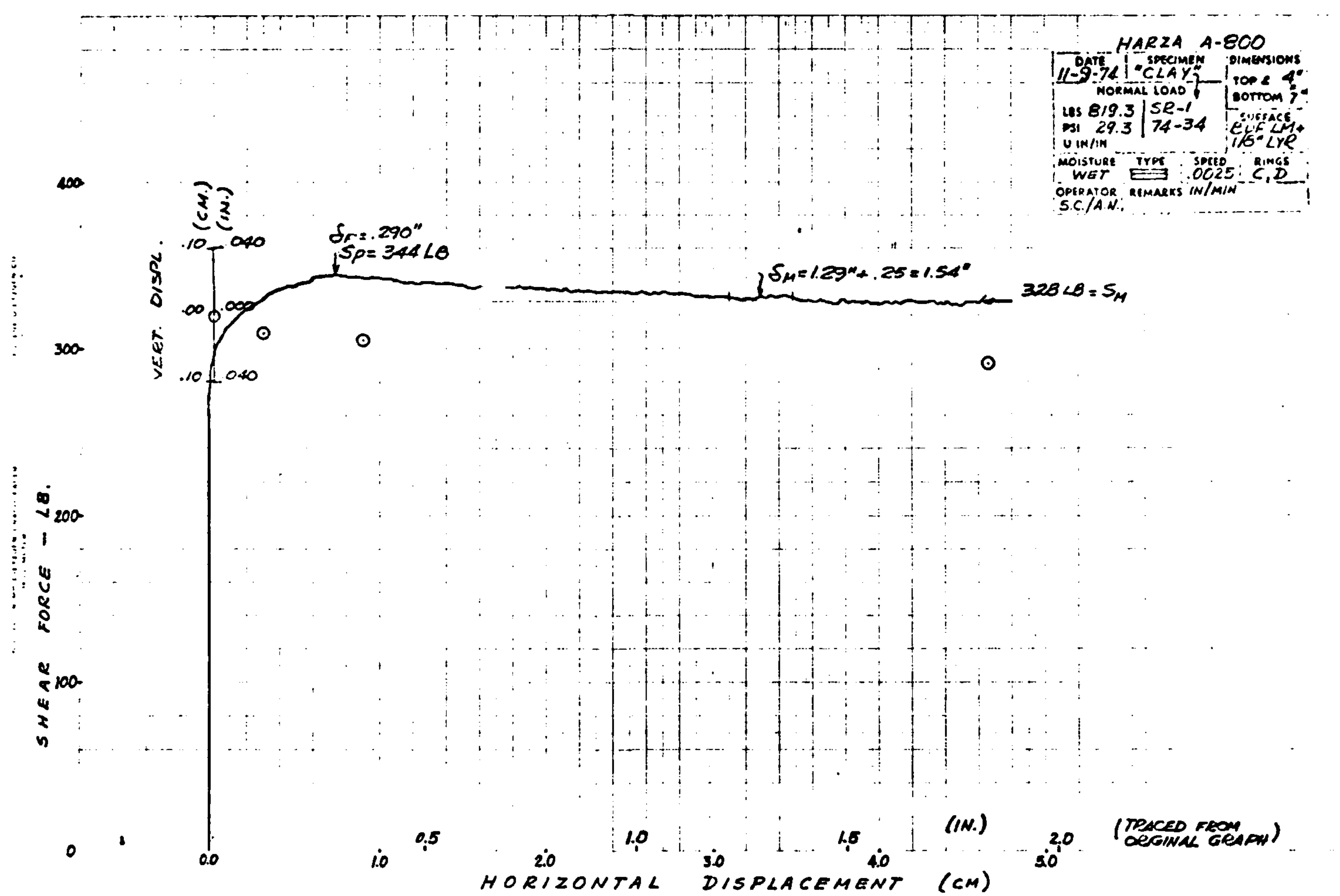


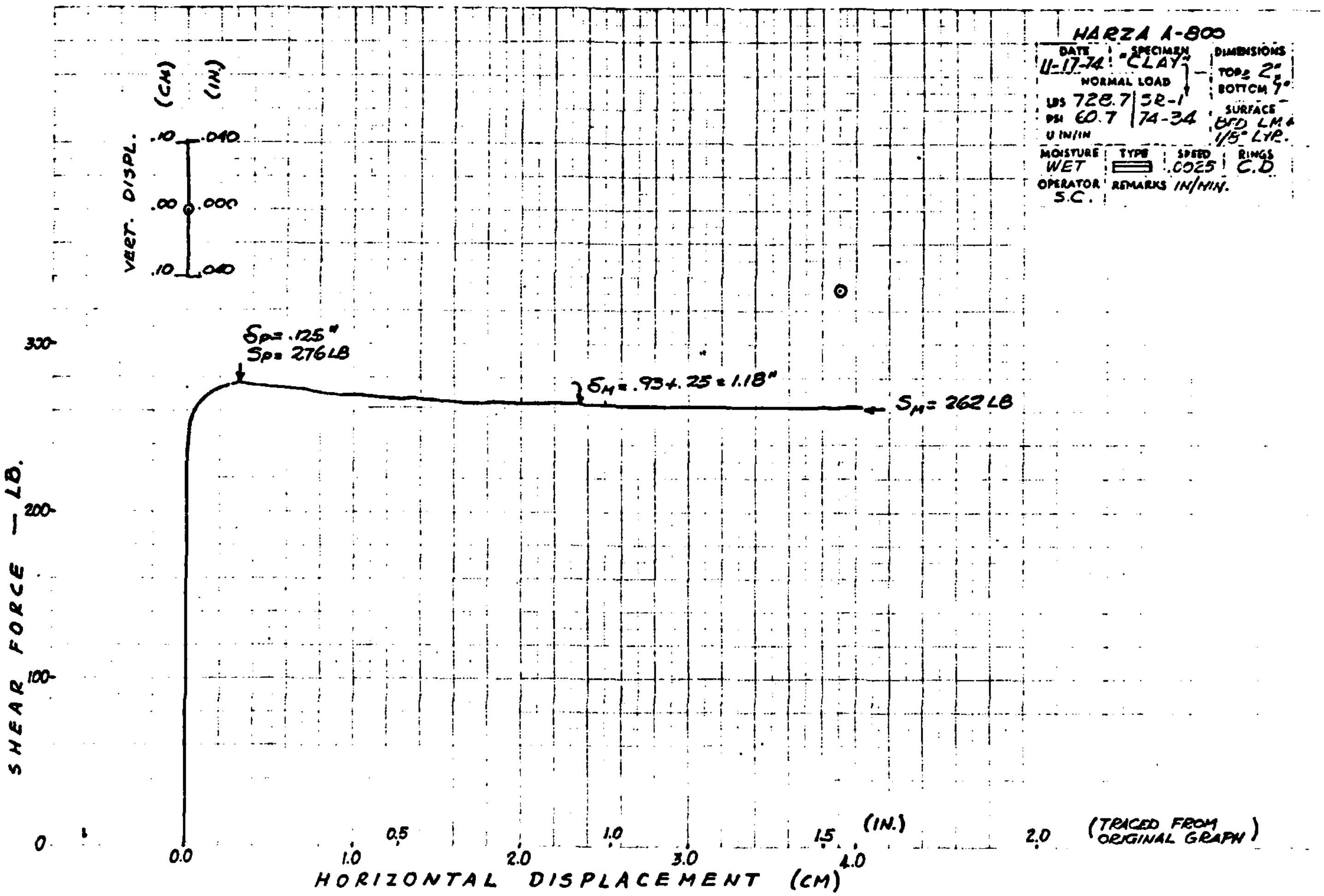
FIG. 10. GEOLOGIC SECTION SHOWING RELATIVE DISPLACEMENTS OF ROCK NEAR TOP OF UNIT "C"

APPENDIX A

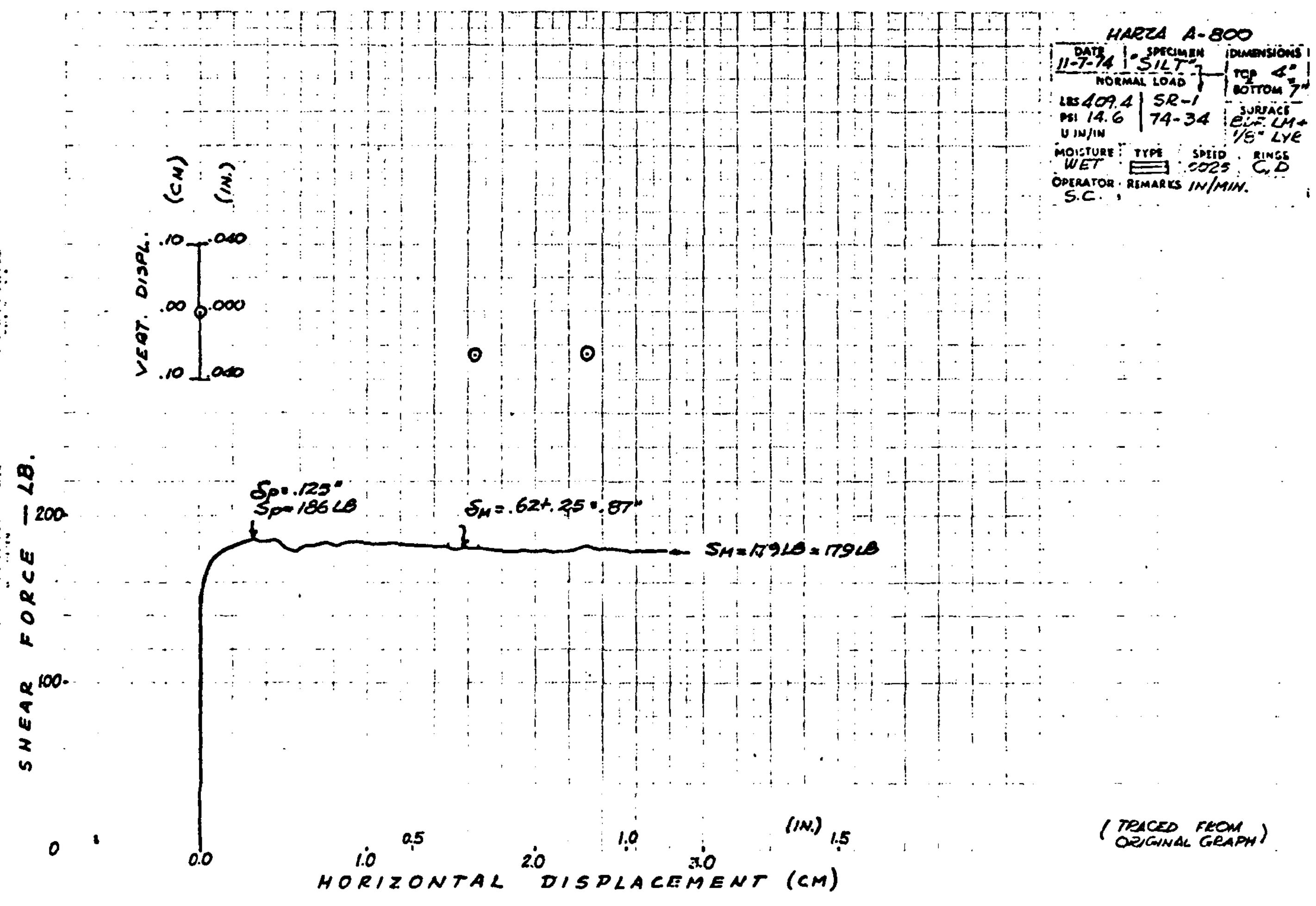


HARZA A-800
DATE 11-5-74
SPECIMEN "CLAY"
DIMENSIONS TOP ≥ 4" BOTTOM 7"
NORMAL LOAD
LBS 409.4
PSI 14.6
U IN/IN
SR-1
74-34
SURFACE BCF-LH + 1/2" LYR.
MOISTURE WET
TYPE
SPEED .0025 IN/MIN.
RINGS C.D
OPERATOR S.C./A.N.
REMARKS
VERT. DISPL. (CM) (IN.)
.10 .040
.00 .000
.10 .040
δp=.35"
Sp=185LB
δH=1.07+.25"=1.32"
SH=170LB
SHEAR FORCE - LB.
200
100
0
0.0
0.5
1.0
1.5
2.0
(IN.)
1.0
2.0
3.0
4.0
HORIZONTAL DISPLACEMENT (CM)
(TRACED FROM ORIGINAL GRAPH)

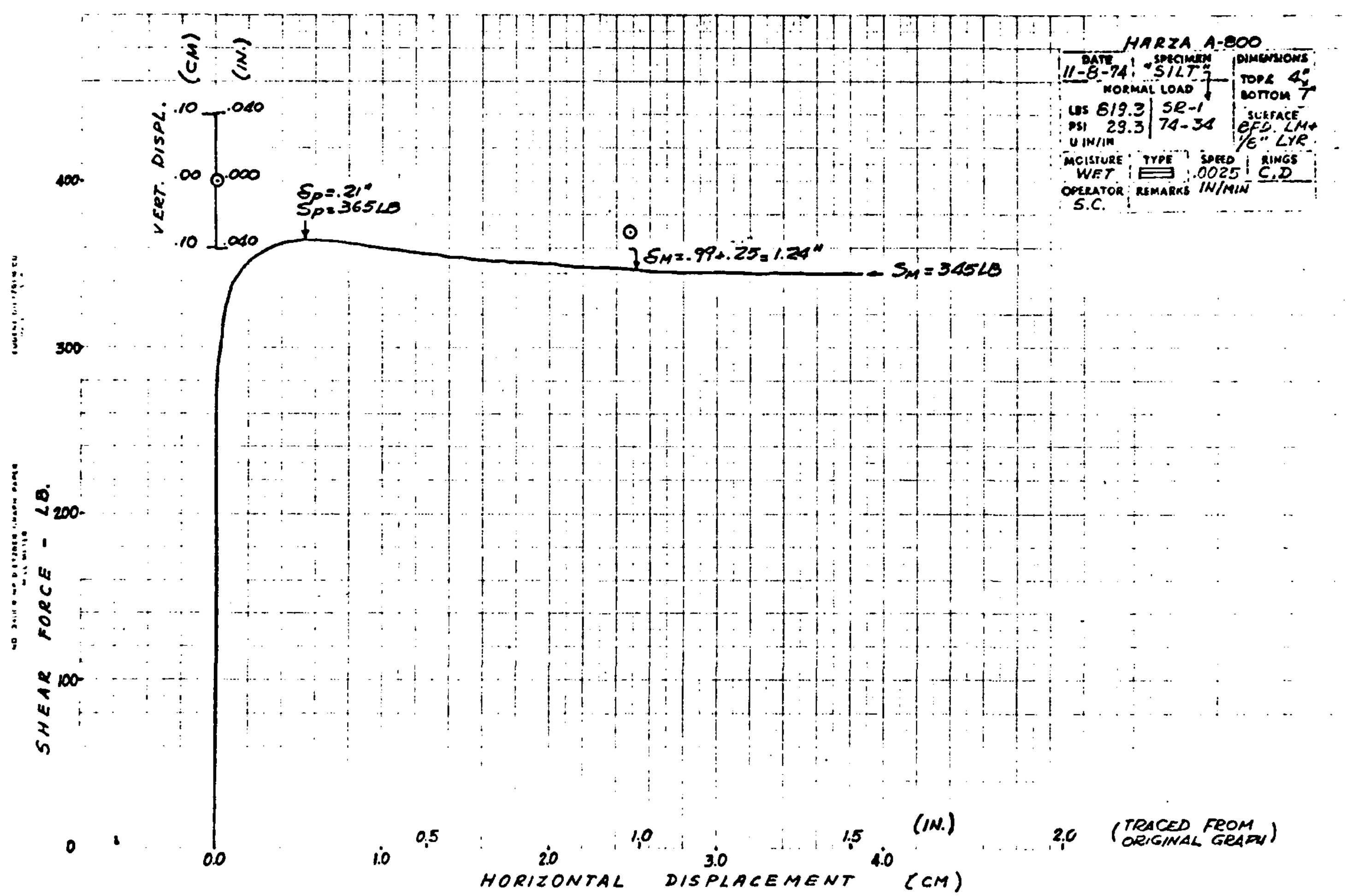

HARZA A-800
DATE 11-9-74
SPECIMEN "CLAY"
DIMENSIONS TOP & 4"
BOTTOM 7"
NORMAL LOAD
LBS 819.3
PSI 29.3
U IN/IN
SR-1
74-34
SURFACE
MOISTURE WET
TYPE
SPEED .0025 IN/MIN
RINGS C, D
OPERATOR S.C./A.N.
REMARKS
δF = .290"
Sp = 344 LB
δM = 1.29" + .25 = 1.54"
328 LB = SM
VERT. DISPL.
(CM.)
(IN.)
.10
.040
.00
.000
.10
.040
SHEAR FORCE — LB.
400
300
200
100
0
HORIZONTAL DISPLACEMENT (CM)
(IN.)
0.0
0.5
1.0
1.5
2.0
1.0
2.0
3.0
4.0
5.0
(TRACED FROM ORIGINAL GRAPH)



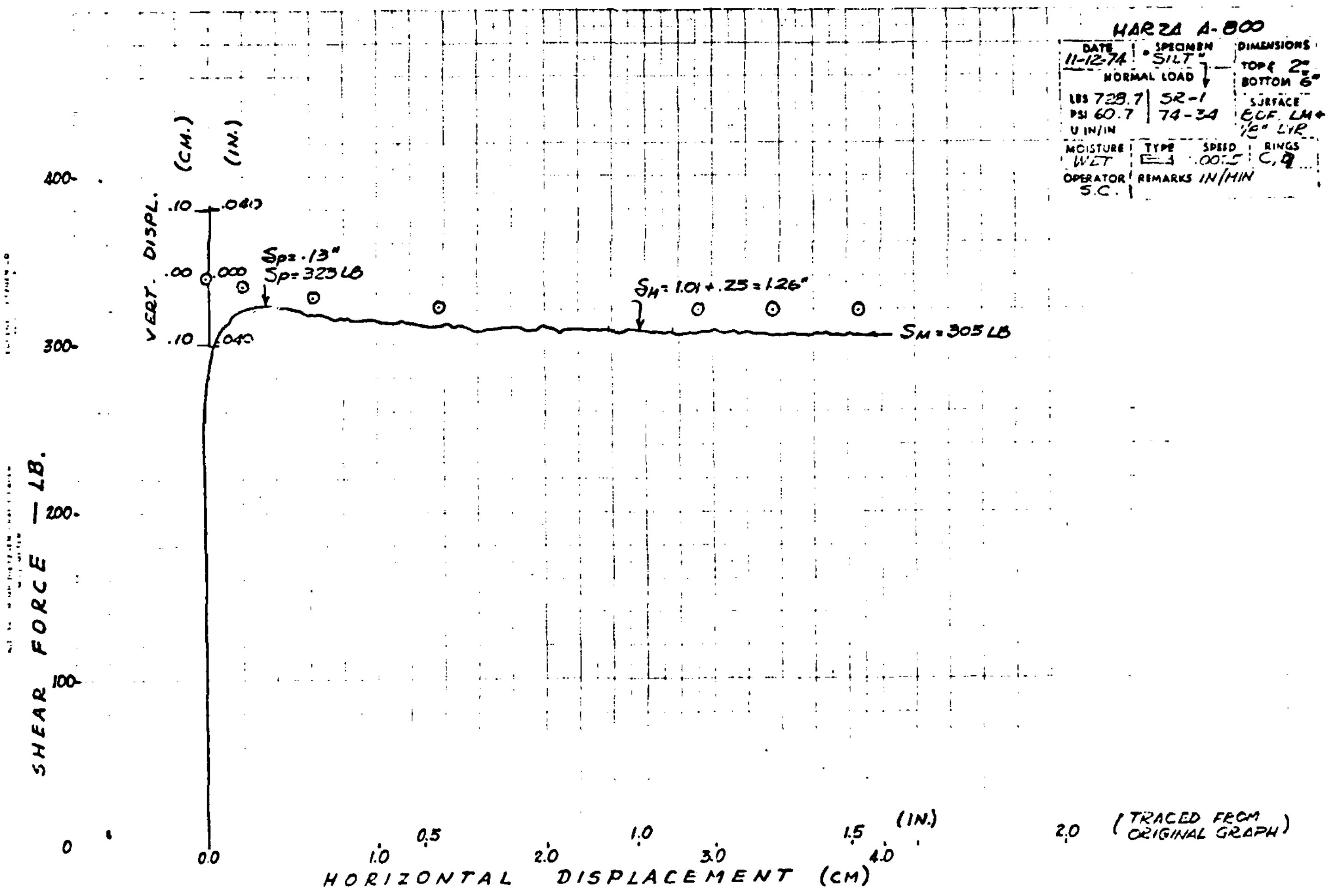
HARZA A-800
DATE 11-17-74
SPECIMEN "CLAY"
DIMENSIONS TOP 2" BOTTOM 7"
NORMAL LOAD
LBS 728.7 SR-1
PSI 60.7 74-34
U IN/IN
SURFACE BFD LM 4 1/5" LYR.
MOISTURE WET
TYPE
SPEED .0025
RINGS C.D
OPERATOR S.C.
REMARKS IN/MIN.
VERT. DISPL.
(CM) (IN)
.10 .040
.00 .000
.10 .040
δp= .125"
Sp= 276LB
δM= .93+.25 = 1.18"
SM= 262 LB
SHEAR FORCE — LB.
300
200
100
0
0.0
1.0
2.0
3.0
4.0
0.5
1.0
1.5
2.0
(IN.)
HORIZONTAL DISPLACEMENT (CM)
(TRACED FROM ORIGINAL GRAPH)



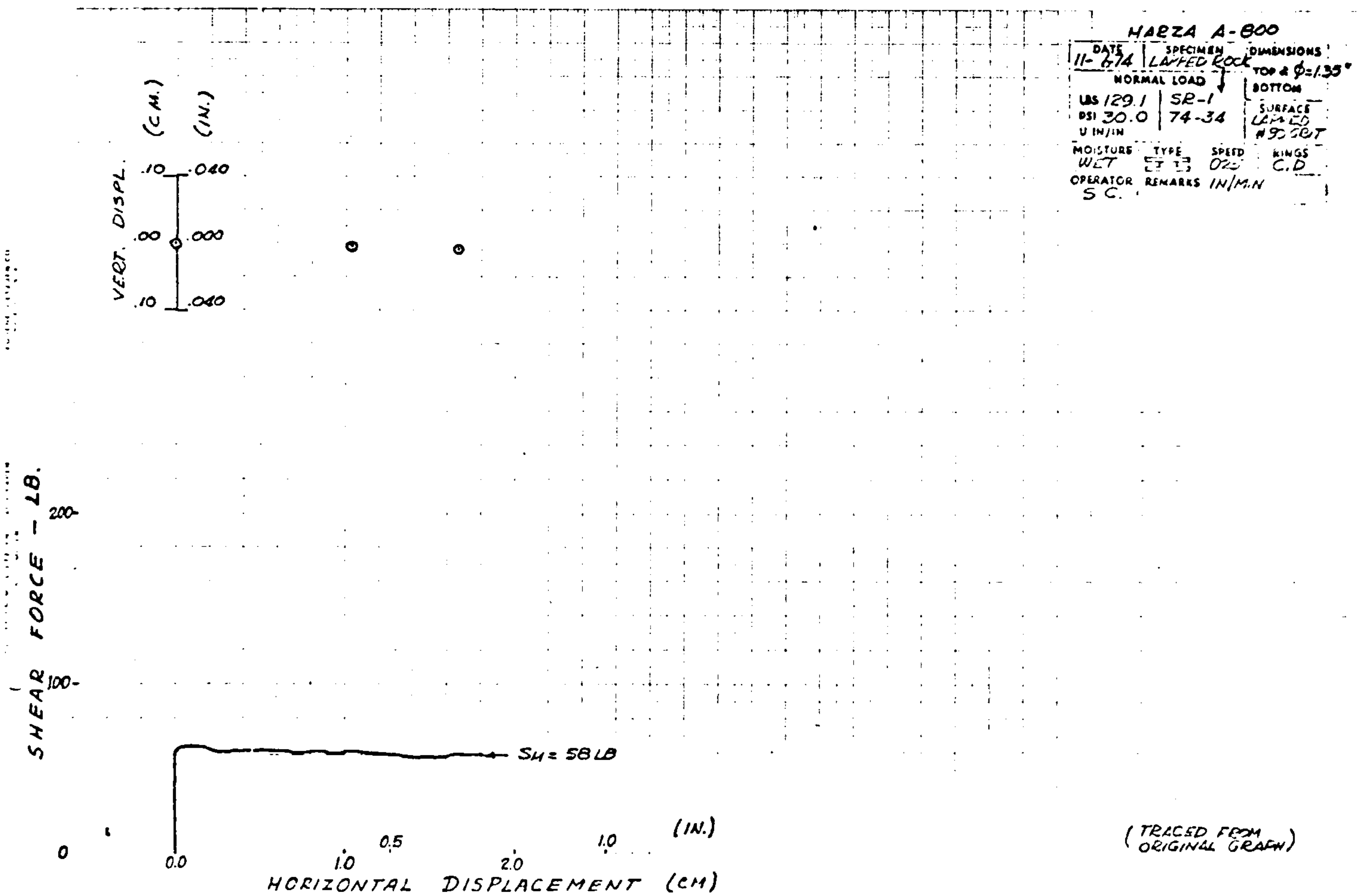
(TRACED FROM ORIGINAL GRAPH)

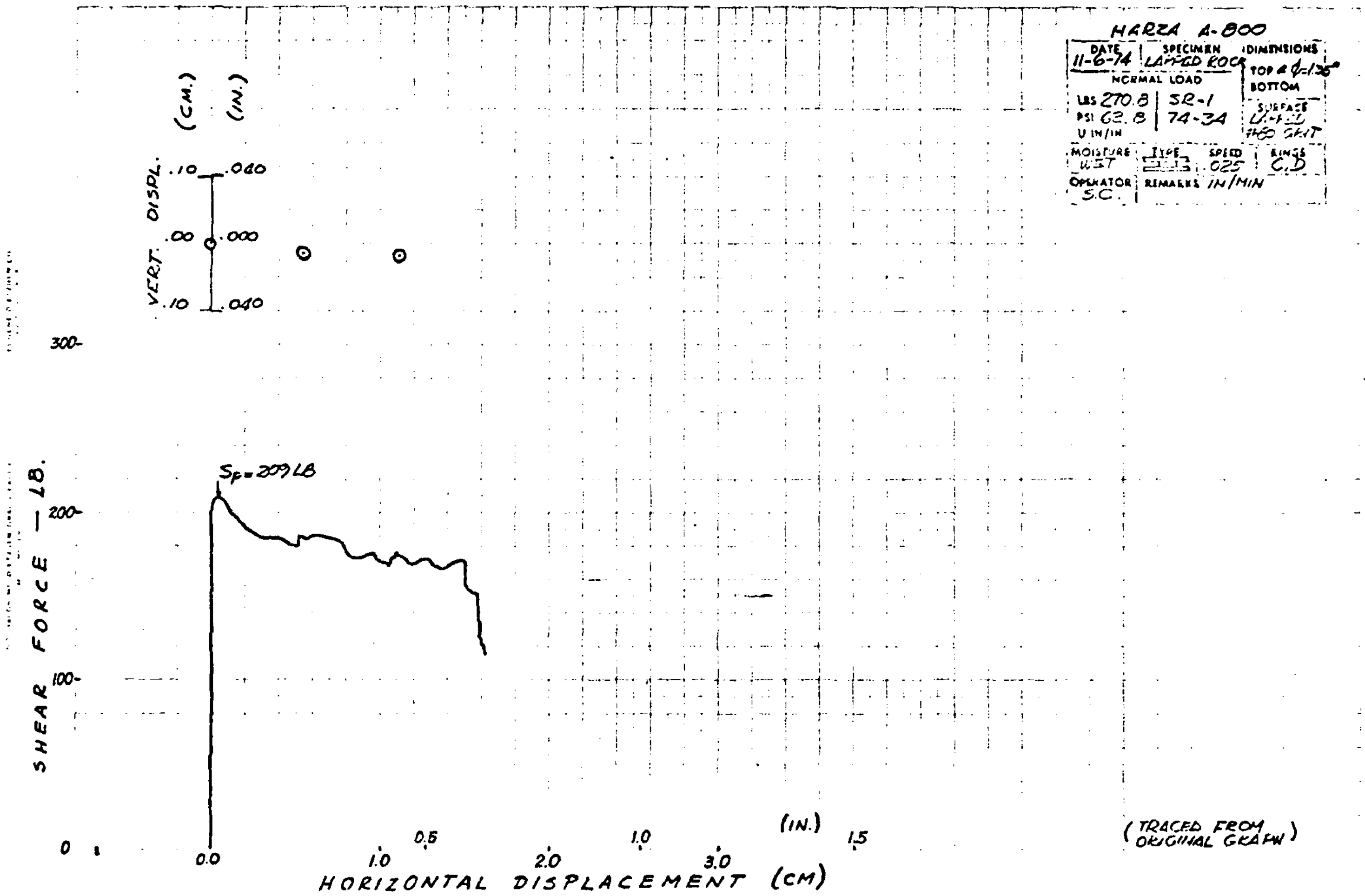


HARZA A-800
DATE 11-8-74
SPECIMEN "SILT"
NORMAL LOAD
LBS 619.3
PSI 29.3
U IN/IN
SR-1
74-34
DIMENSIONS
TOP & 4"
BOTTOM 7"
SURFACE
EFD. LM+
1/6" LYR.
MOISTURE WET
TYPE
SPEED .0025 IN/MIN
RINGS C.D
OPERATOR S.C.
REMARKS
VERT. DISPL. (CM) (IN.)
.10 .040
.00 .000
.10 .040
δ_p=.21"
S_p=365LB
δ_M=.99+.25=1.24"
S_M=345LB
SHEAR FORCE - LB.
400
300
200
100
0
0.0
0.5
1.0
1.5
2.0
(IN.)
1.0
2.0
3.0
4.0
HORIZONTAL DISPLACEMENT (CM)
(TRACED FROM ORIGINAL GRAPH)

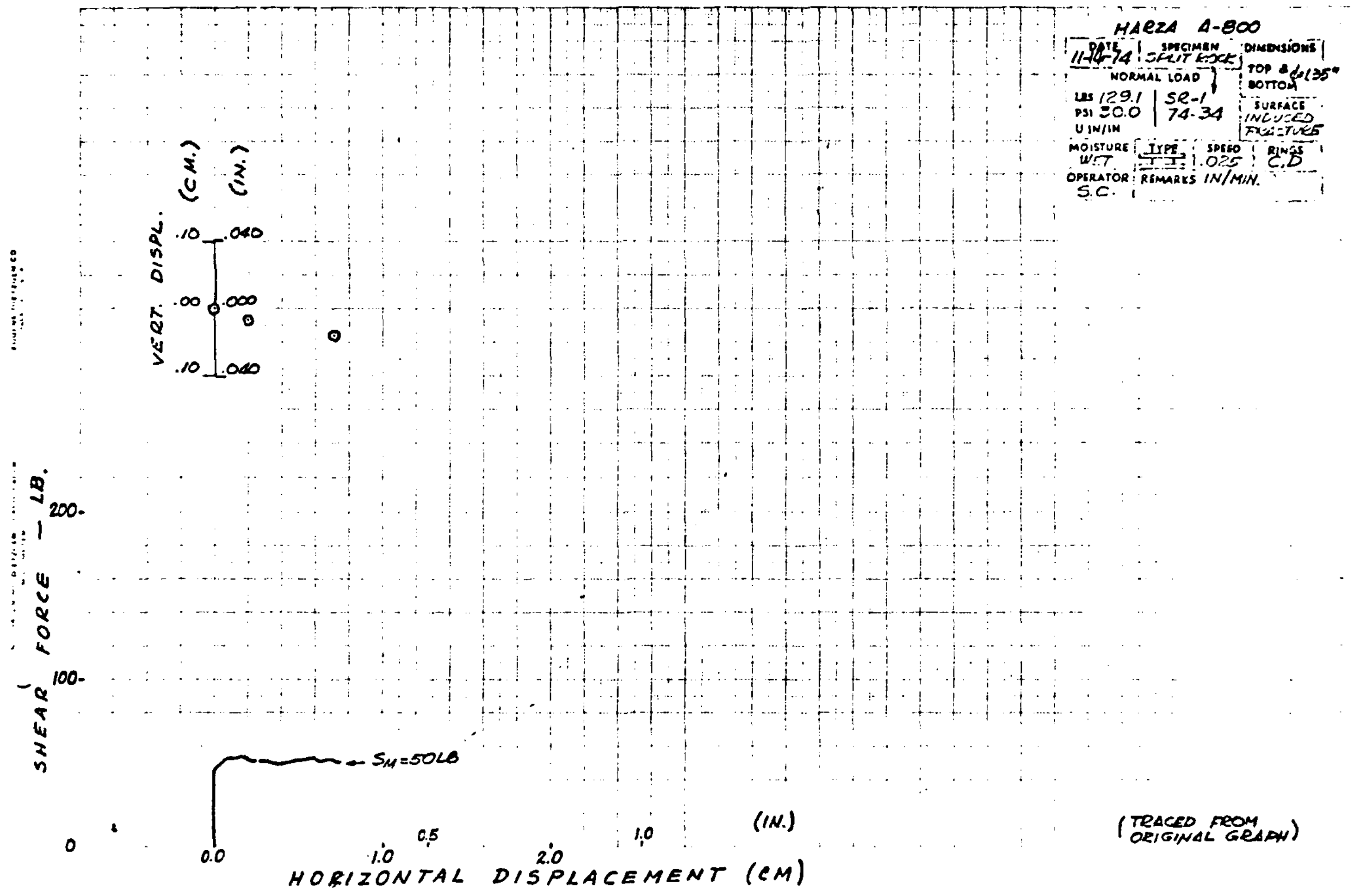


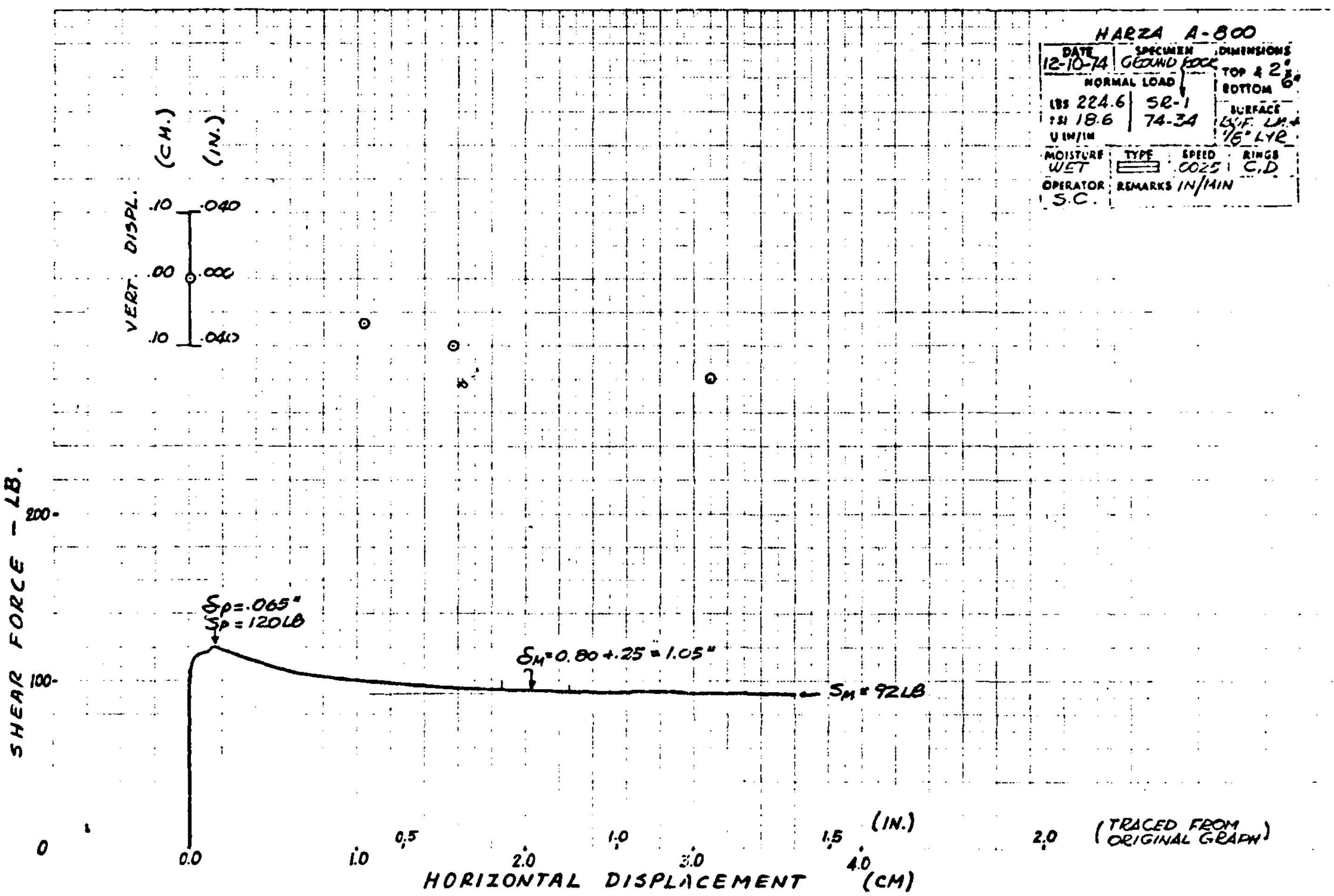
HARZA A-800
DATE 11-12-74
SPECIMEN "SILT"
DIMENSIONS
TOP 2"
BOTTOM 6"
NORMAL LOAD
LBS 728.7
PSI 60.7
U IN/IN
SR-1
74-34
SURFACE
EOF. LM + 1/2" LYR
MOISTURE WET
TYPE
SPEED .0015 IN/MIN
RINGS C, B
OPERATOR S.C.
REMARKS
VERT. DISPL.
(CM.)
(IN.)
.10
.040
.00
.000
.10
.040
Sp = .13"
Sp = 325 LB
SH = 1.01 + .25 = 1.26"
SM = 305 LB
SHEAR FORCE — LB.
400
300
200
100
0
0.0
0.5
1.0
1.5
2.0
(IN.)
1.0
2.0
3.0
4.0
HORIZONTAL DISPLACEMENT (CM)
(TRACED FROM ORIGINAL GRAPH)

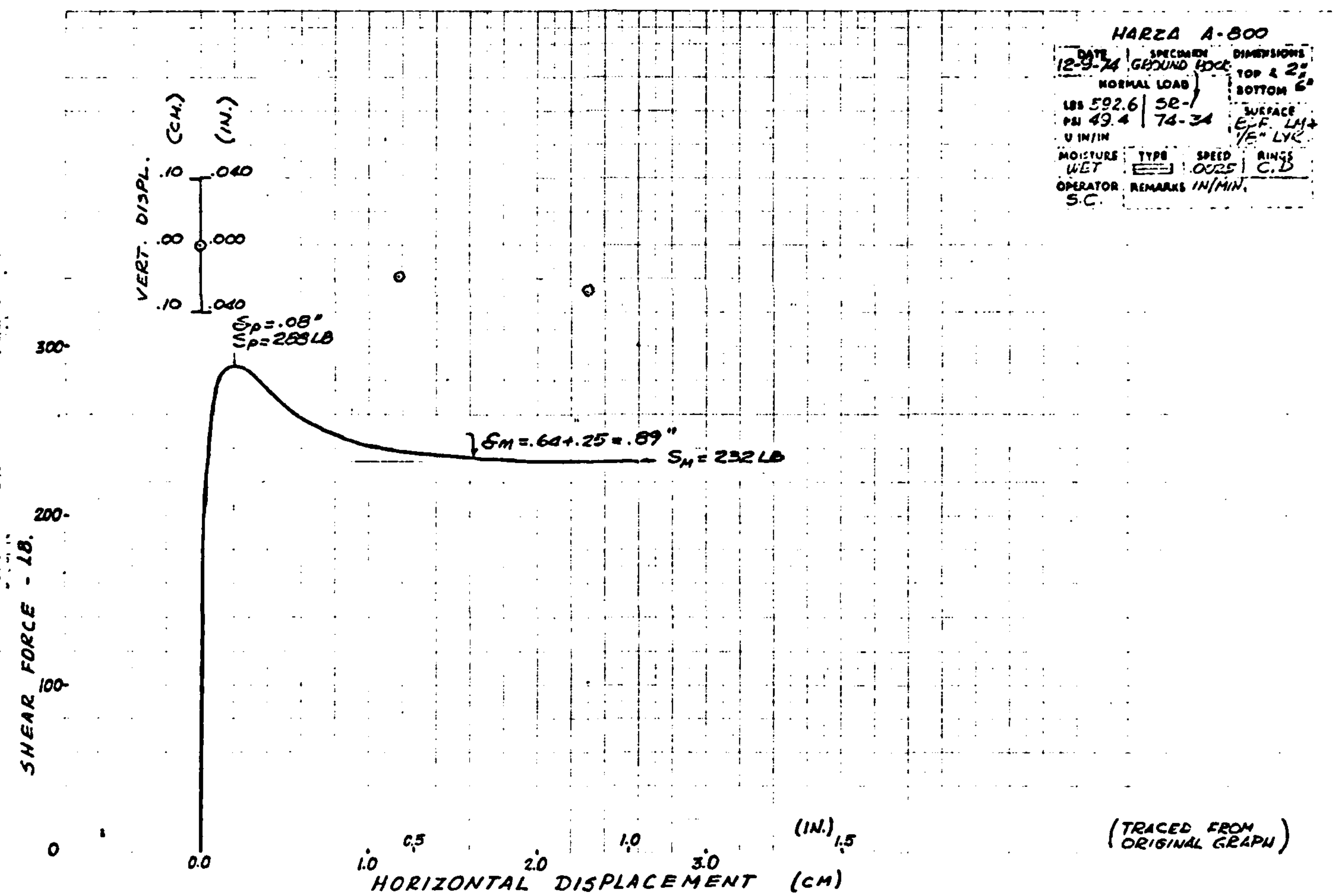

HARZA A-800
DATE 11-6-74
SPECIMEN LAPPED ROCK
DIMENSIONS TOP & BOTTOM φ=1.35"
NORMAL LOAD
LBS 129.1
PSI 30.0
U IN/IN
SR-1
74-34
SURFACE LAPPED #90 GRIT
MOISTURE WET
TYPE
SPEED .020
RINGS C.D
OPERATOR S.C.
REMARKS IN/MIN
VERT. DISPL. (CM.) (IN.)
.10 .040
.00 .000
.10 .040
SHEAR FORCE - LB.
200
100
0
SH = 58 LB
0.0
1.0
0.5
2.0
1.0
(IN.)
HORIZONTAL DISPLACEMENT (CM)
(TRACED FROM ORIGINAL GRAPH)

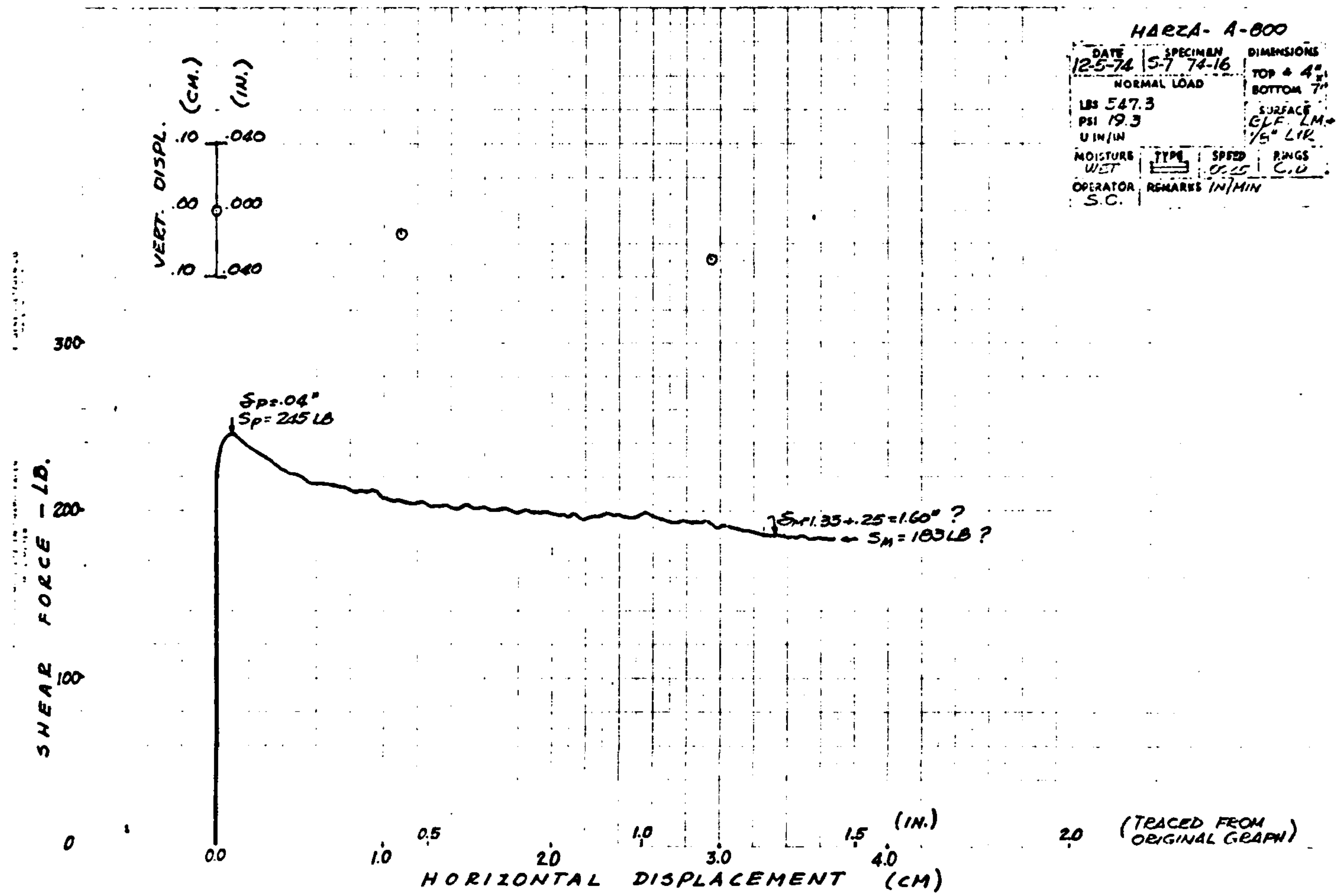


HARZA A-800
DATE 11-6-74
SPECIMEN LAPPED ROCK
DIMENSIONS TOP & ϕ=1.36"
BOTTOM
NORMAL LOAD
LBS 270.8
PSI 62.8
U IN/IN
SR-1
74-3A
SURFACE
MOISTURE WET
TYPE
SPEED .025
RINGS C.D
OPERATOR S.C.
REMARKS IN/MIN
VERT. DISPL. (CM.) (IN.)
.10 .040
.00 .000
.10 .040
300
200
100
0
Sp=209 LB
SHEAR FORCE — LB.
0.0
0.5
1.0
1.0
2.0
3.0
1.5
(IN.)
HORIZONTAL DISPLACEMENT (CM)
(TRACED FROM ORIGINAL GRAPH)

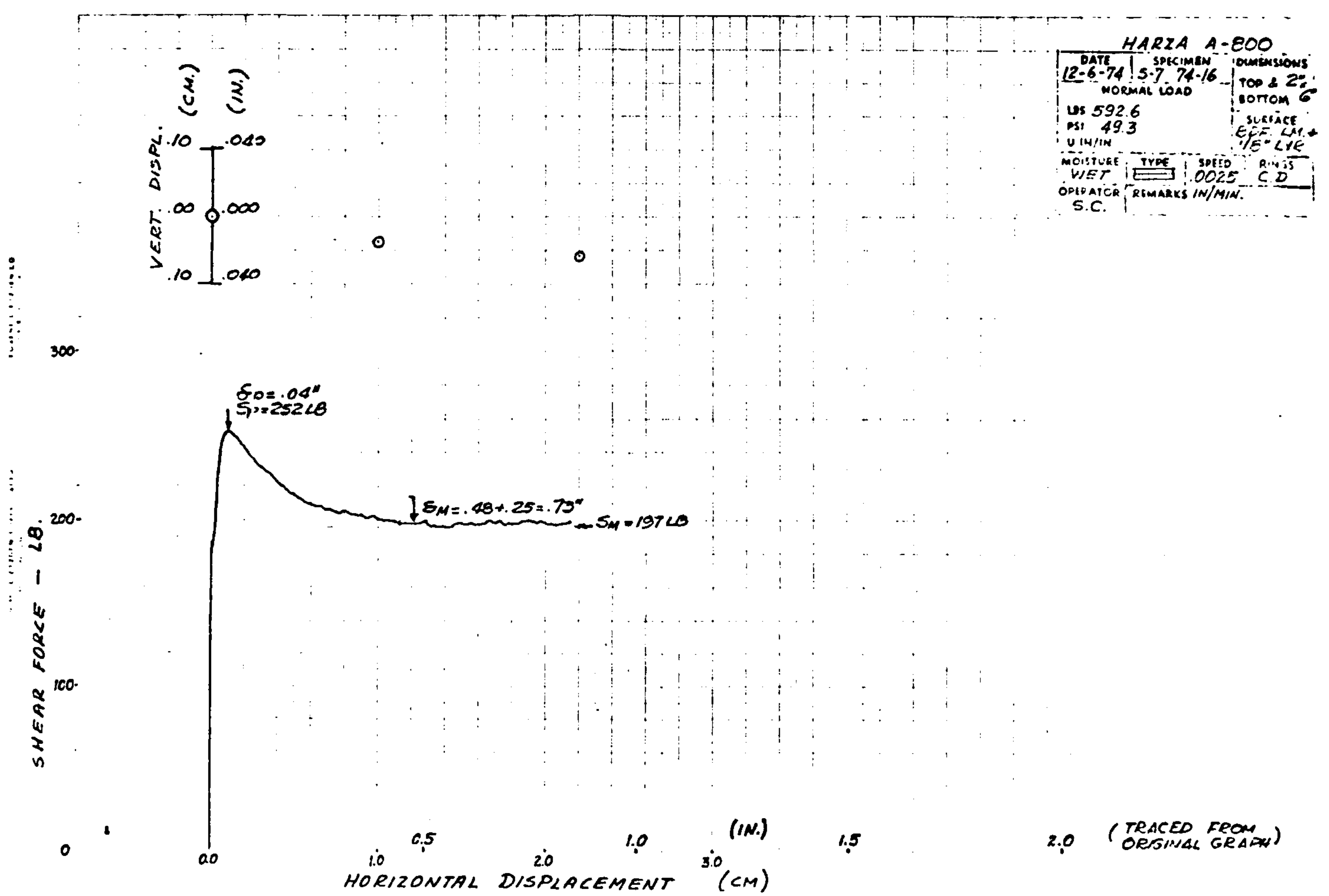

HARZA A-800
DATE 11-14-74
SPECIMEN SPLIT ROCK
DIMENSIONS
TOP & BOTTOM 4.135"
NORMAL LOAD
LBS 129.1
PSI 30.0
U IN/IN
SR-1
74-34
SURFACE INDUCED FRACTURE
MOISTURE WET
TYPE II
SPEED .025
RINGS C.D
OPERATOR S.C.
REMARKS IN/MIN.
VERT. DISPL. (CM.) (IN.)
.10 .040
.00 .000
.10 .040
SHEAR FORCE — LB.
200
100
0
S_M=50LB
0.0
1.0
0.5
2.0
1.0
(IN.)
HORIZONTAL DISPLACEMENT (CM)
(TRACED FROM ORIGINAL GRAPH)


HARZA A-800
DATE 12-10-74
SPECIMEN GROUND ROCK
DIMENSIONS TOP & BOTTOM
NORMAL LOAD
LBS 224.6
PSI 18.6
SR-1 74-34
SURFACE
MOISTURE WET
TYPE
SPEED .0025
RINGS C.D
OPERATOR S.C.
REMARKS IN/MIN
VERT. DISPL.
(CM.)
(IN.)
.10
.040
.00
.000
SHEAR FORCE - LB.
200
100
0
δ_P=.065"
S_P=120LB
δ_M=0.80+.25=1.05"
S_M=92LB
0.0
0.5
1.0
1.5
2.0
(IN.)
2.0
3.0
4.0
(CM)
HORIZONTAL DISPLACEMENT
(TRACED FROM ORIGINAL GRAPH)

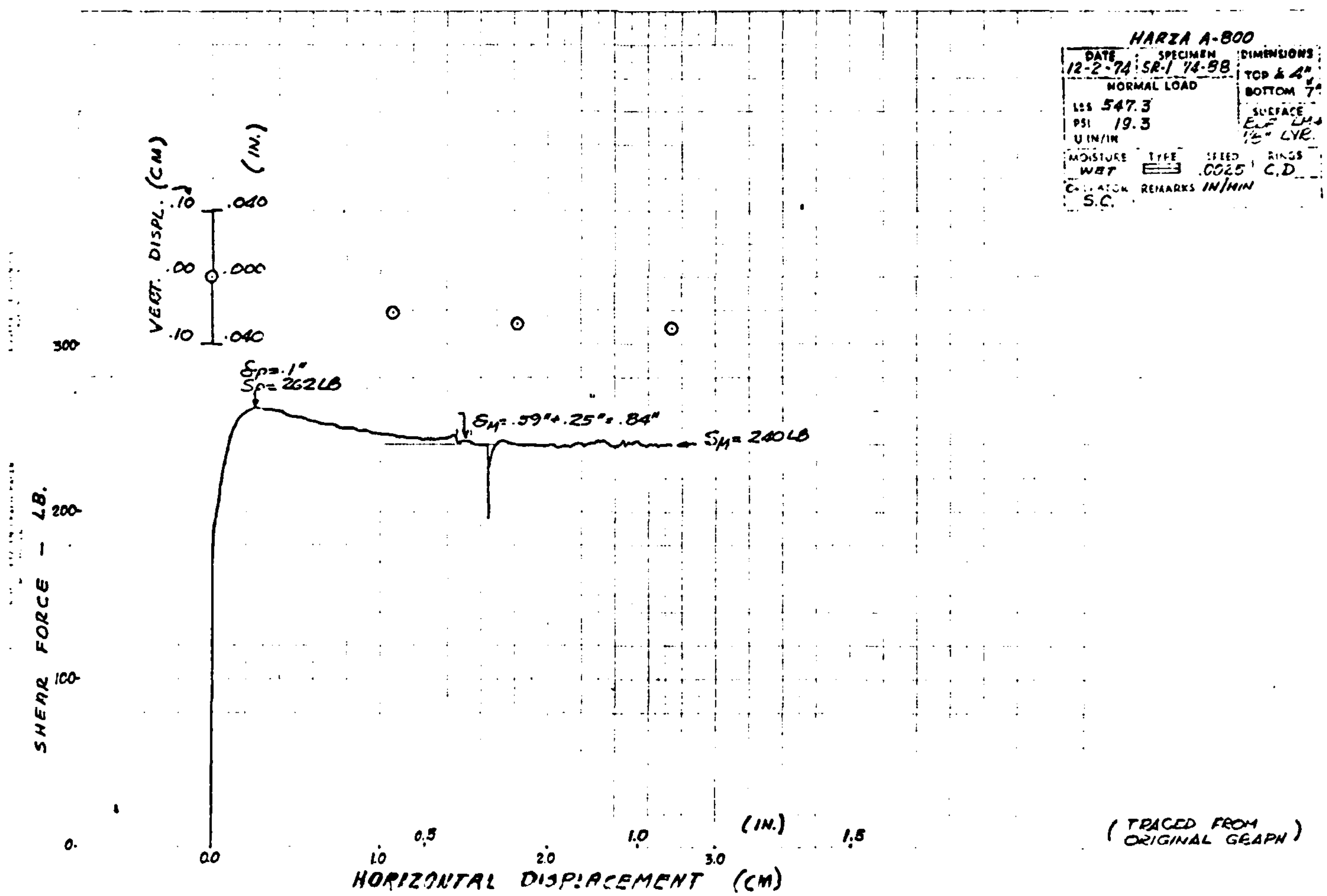

HARZA A-800
DATE 12-9-74
SPECIMEN GROUND ROCK
DIMENSIONS TOP & 2" BOTTOM 6"
NORMAL LOAD
LBS 592.6
PSI 49.4
U IN/IN
SR-1 74-34
SURFACE E-F. LM+ 1/E" LYC
MOISTURE WET
TYPE
SPEED .0025 IN/MIN.
RINGS C.D
OPERATOR S.C.
REMARKS
VERT. DISPL.
(CM.)
(IN.)
.10
.040
.00
.000
.10
.040
δ_P=.08"
S_P=288 LB
δ_M=.64+.25=.89"
S_M= 232 LB
SHEAR FORCE - LB.
300
200
100
0
0.0
0.5
1.0
1.0
2.0
3.0
(IN.)
1.5
HORIZONTAL DISPLACEMENT (CM)
(TRACED FROM ORIGINAL GRAPH)



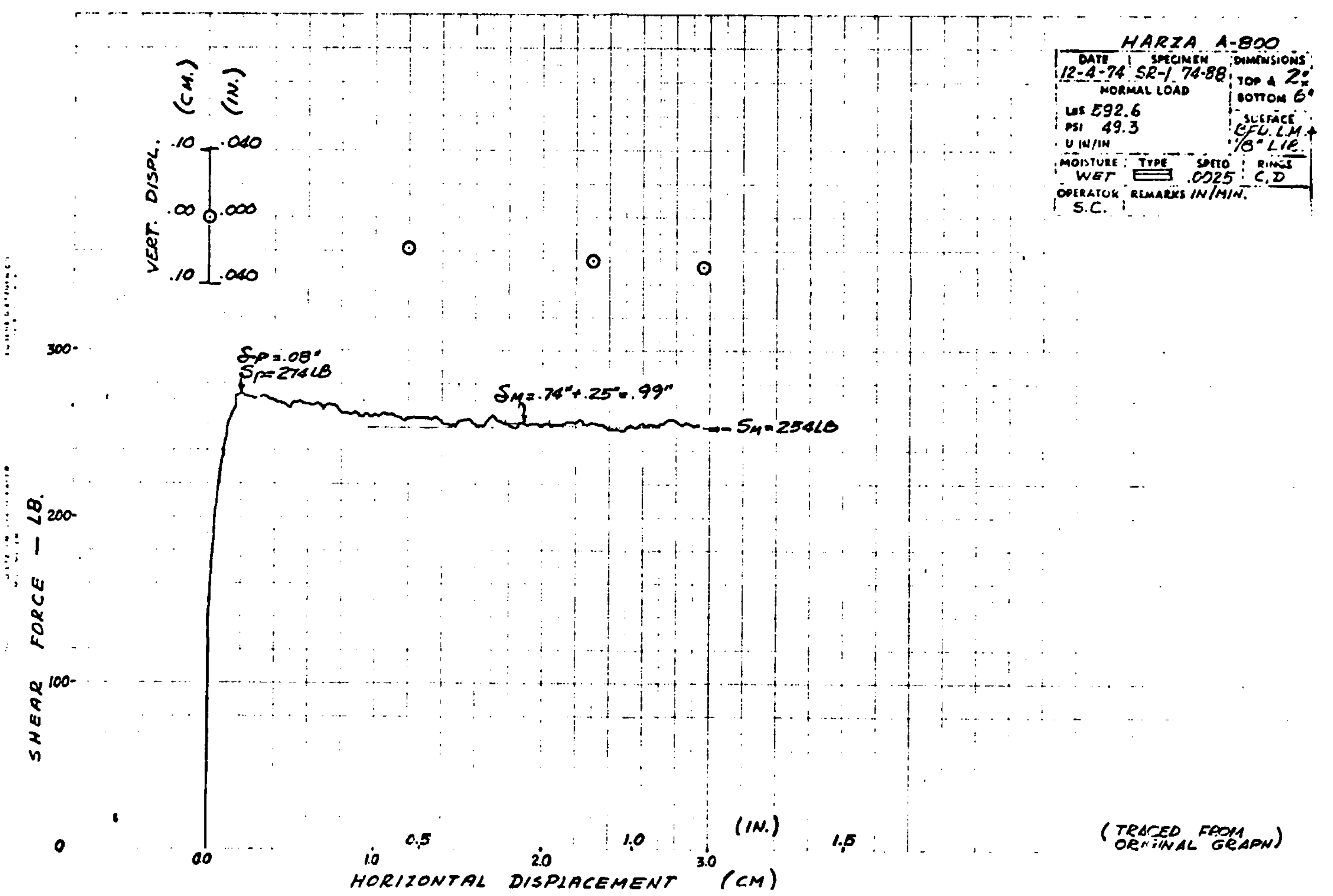
HARZA- A-800
DATE 12-5-74
SPECIMEN S-7 74-16
DIMENSIONS TOP 4 4" BOTTOM 7"
NORMAL LOAD
LBS 547.3
PSI 19.3
U IN/IN
SURFACE G.L.F. LM + 1/5" LIR
MOISTURE WET
TYPE
SPEED 0.25
RINGS C.D
OPERATOR S.C.
REMARKS IN/MIN
VERT. DISPL. (CM.) (IN.)
.10 .040
.00 .000
.10 .040
SHEAR FORCE - LB.
300
200
100
0
Sp=.04"
Sp= 245 LB
SM1.35+.25=1.60" ?
SM=183LB ?
0.5
1.0
1.5
2.0
(IN.)
0.0
1.0
2.0
3.0
4.0
HORIZONTAL DISPLACEMENT (CM)
(TRACED FROM ORIGINAL GRAPH)



HARZA A-800
DATE 12-6-74
SPECIMEN S-7 74-16
DIMENSIONS TOP & 2" BOTTOM 6"
NORMAL LOAD
LBS 592.6
PSI 49.3
U IN/IN
SURFACE
MOISTURE WET
TYPE
SPEED .0025 IN/MIN.
OPERATOR S.C.
REMARKS
VERT. DISPL. (CM.) (IN.)
.10 .040
.00 .000
.10 .040
δ_D = .04"
S_P = 252 LB
δ_M = .48 + .25 = .73"
S_M = 197 LB
SHEAR FORCE — LB.
300
200
100
0
0.0
0.5
1.0
1.5
2.0
1.0
2.0
3.0
(IN.)
(CM)
HORIZONTAL DISPLACEMENT
(TRACED FROM ORIGINAL GRAPH)



HARZA A-800
DATE 12-2-74
SPECIMEN SR-1 74-98
DIMENSIONS TOP 4" BOTTOM 7"
NORMAL LOAD
LBS 547.3
PSI 19.3
SURFACE
MOISTURE WET
TYPE
SPEED .0025
RINGS C.D.
REMARKS IN/MIN
S.C.
VERT. DISPL. (CM)
(IN.)
.10
.00
.10
.040
.000
.040
δ_P=.1"
S_P=262LB
δ_M=.59"+.25"=.84"
S_M=240LB
SHEAR FORCE — LB.
300
200
100
0
0.0
0.5
1.0
2.0
3.0
1.5
(IN.)
HORIZONTAL DISPLACEMENT (CM)
(TRACED FROM ORIGINAL GRAPH)



HARZA A-800
DATE 12-4-74
SPECIMEN SR-1 74-88
DIMENSIONS TOP & 2" BOTTOM 6"
NORMAL LOAD
LBS 592.6
PSI 49.3
U IN/IN
SURFACE CFU. LM + 1/8" LIE.
MOISTURE WET
TYPE
SPEED .0025
RINGS C, D
OPERATOR S.C.
REMARKS IN/MIN.
VERT. DISPL. (CM.) (IN.)
.10 .040
.00 .000
.10 .040
S_P=.08"
S_P=274LB
S_M=.74"+.25"=.99"
S_M=254LB
SHEAR FORCE — LB.
300
200
100
0
0.0
0.5
1.0
1.5
(IN.)
1.0
2.0
3.0
HORIZONTAL DISPLACEMENT (CM)
(TRACED FROM ORIGINAL GRAPH)

APPENDIX B

Report of Clay Mineral Analyses

At the request of Dr. A. S. Nieto-Pescetto, the writer has performed a clay mineral analysis on each of two rock core samples obtained from an HQ boring. The boring was made in conjunction with the foundation exploration program for the locks and dam No. 1 near Minneapolis, Minnesota by Harza Engineering of Chicago, Illinois. The samples are (1) a clay and (2) siltstone from the St. Peter Sandstone Formation (top portion of Unit C) of Ordovician age.

The less than two micron (spherical hydraulic equivalent) fraction of these samples was subjected to both x-ray diffraction (XRD) and differential thermal analysis (DTA) procedures. The following clay mineral phases were found to be present in the approximate percentages noted below. There is no significant difference in the clay mineralogical content of the two samples, nor is smectite (montmorillonite) present in either sample.

kaolinite	40%
illite	15%
randomly mixed layered chlorite-like mineral (expansive in glycerine to a minor degree)	45%

Harold C. Ganow

Harold C. Ganow

Report of Clay Mineral Analyses

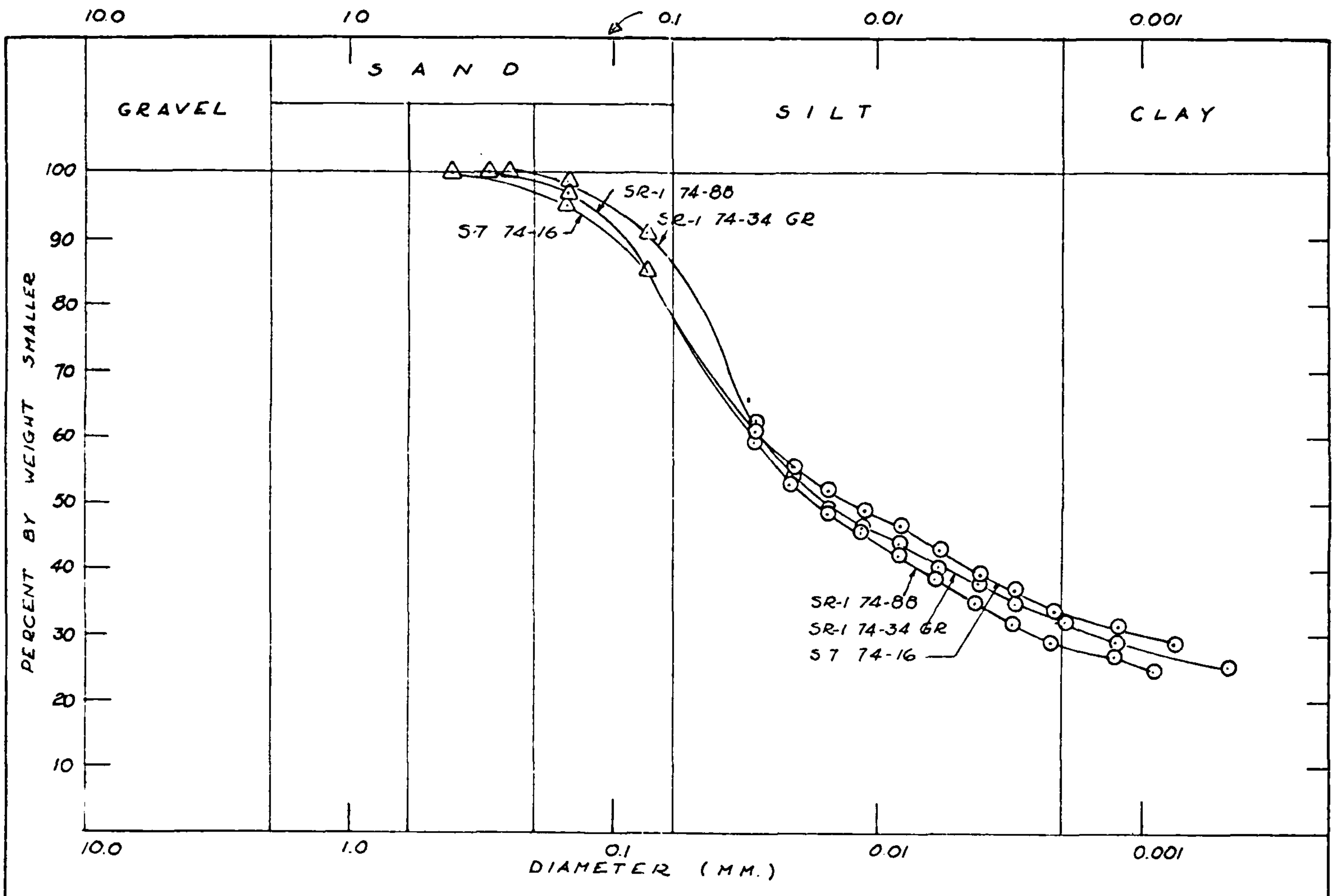
At the request of Dr. A. S. Nieto-Pescetto, the writer has performed two additional clay mineral analyses on two rock core samples obtained from HQ borings. The borings were made in conjunction with the foundation exploration program for the locks and dam No. 1 near Minneapolis, Minnesota by Harza Engineering of Chicago, Illinois. The samples (SR-1 74-16, depth 81.1'-81.3' and SR-7 74-88, depth 42.25'-42.6') are of clayey siltstones from the St. Peter Sandstone Formation (top portion of Unit C) of Ordovician age.

The less than two micron (spherical hydraulic equivalent) fraction of these samples was subjected to an X-ray Diffraction Analysis procedure which allowed the identification of the following clay mineral phases:

1. kaolinite	30%
2. illite	20%
3. a randomly interstratified illite/chlorite which is slightly expansive in glycerine	50%
4. chlorite ?	trace ?

The percentages given are only estimates based on the writer's experience in working with clay mineral mixtures, and may be in error by as much as 50% of the percentage reported. The two samples are not significantly different in their clay mineralogy. Although the illite/chlorite and to a lesser extent the illite are slightly expansive in a glycerine/water solution, they do not have the characteristics normally associated with a smectite (montmorillonite) clay mineral.

Harold C. Ganow
Harold C. Ganow
January 3, 1975



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MISSISSIPPI RIVER

STUDY OF ALTERNATIVES FOR REHABILITATION OF LOCK AND DAM NO.1

MINNEAPOLIS, MINNESOTA

APPENDIX B-3

GEOLOGY, FOUNDATIONS AND SOILS INVESTIGATION

SEEPAGE, UPLIFT AND PIPING

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

PLATES

APPENDIX B-3

SEEPAGE, UPLIFT AND PIPING

REVIEW OF PREVIOUS OBSERVATIONS
AND REMEDIAL MEASURES

1. Seepage Under Intermediate Wall in 1930. The land wall of the riverward lock (present intermediate wall of the twin locks) was built on sandstone without any steel cut-off piling. In 1930 seepage under the then land wall was noted, and to prevent loss of material, the floor of the lock was cut and a row of steel sheet piling was driven parallel and adjacent to the river face of the land wall (present intermediate wall) and then grouted.

2. Seepage Under Intermediate Wall in 1931. When excavating for the landward lock in 1931, seepage under the then existing landward wall (present intermediate wall) was noted and another row of steel sheet piling was driven into the sandstone parallel and adjacent to the landward face of the intermediate wall.

3. Leakage Between Lock Chambers in 1933. In 1933 leakage from one lock chamber to the other was noted. Grouting under the intermediate wall was undertaken to seal the leaks.

4. Scour Downstream of the Locks in 1935. Considerable scour was noted downstream of the locks and under the riverward guard wall in 1935. In the winter of 1935-36, 350 cubic yards of tremie concrete was placed under the riverward guard wall and 1,600 cubic yards of derrickstone paving was placed below the concrete apron. Movement of the lower guide wall on the right bank required removal of the backfill on the land side to near normal pool level.

5. Leaks Between Lock Chambers in 1941. In December 1941 and in April 1942 additional grouting was done to fill voids under landward lock floor and intermediate wall. Also voids under weep holes in the riverward lock were filled with graded sand and gravel filter.

6. Failure of Lower Riverward Guard Wall in 1950. Five monoliths of the lower riverward guard wall tipped over into the lower

approach channel in July 1950. To remove the damaged guard wall and to build a replacement rockfill guard wall, the lower approach was dewatered.

7. Foundation Grouting in 1950. Foundation grouting was done under the river wall near the lower miter gate in 1950. During the same year the lower landward guide wall, which had moved channelward, was stabilized by pressure grouting.

8. Settlement of River Wall Monoliths in 1959. Following indications of settlement of four monoliths of the riverward lock wall, an investigation was conducted to determine conditions under the wall. No extensive voids were noted but grouting was carried on to fill such voids as might exist. Some additional grouting under the riverward wall in other locations was undertaken also.

9. Repair of Joint Between Landward Lock and Intermediate Wall in 1960. To minimize leakage between the locks the joints in the landside of the intermediate wall and the connection between the existing steel sheet pile cut-off in the landward lock and the intermediate wall were repaired.

10. Piezometer Readings in 1964. During the fall of 1964 piezometer readings were carried out behind the land wall and along the intermediate wall.

FIELD INVESTIGATIONS OF SEEPAGE AND UPLIFT PRESSURES

11. Borehole Water Level Measurements. The purpose of the water level measurement program in the piezometers was twofold: 1) to determine apparent uplift pressures underneath the structures under operating conditions, and 2) to determine apparent seepage paths underneath the lock structures. Water level measurements were made in selected piezometers (Plate B-3-1) at the following conditions: 1) water maintained constant at upper pool level in both locks, 2) water maintained constant at lower pool level in both locks, 3) water maintained constant at upper pool level in landward lock and constant at lower pool level in riverward lock, and 4) water maintained constant at lower pool level in landward lock and constant at upper level in riverward lock. Water levels were measured frequently for a period of 2 hours under each set of conditions. The water level elevations in the piezometers

under each set of conditions were plotted on a series of graphs (Plates B-3-2 through B-3-7) from which the apparent uplift pressures were directly determined.

Relative water levels indicated movement of seepage towards a discharge area downstream of the landlock miter gate, ground-water movement directly down-gradient under the intermediate wall, and suggested some degree of hydraulic continuity between the lock structures. The hydraulic gradient along the land lock, sloping towards the tailwater, may also have been influenced by the longitudinal 12 inch tile drains which can still be open and in operating condition.

Some of the plotted points on Plates B-3-2 through B-3-7 could be reading or recording errors. The main water level trends, however, cannot be affected by a few erroneous values.

Some of the piezometers exhibited unusual behavior, not all of which is understood. Piezometer 74-8U did not respond to the change in lock levels, and the water level in it remained flat (Exhibit B-3-2). This may be due to plugging, or, since the piezometer was located in the fill above the bedrock, it may have been poorly connected with the drains and joint system in the bedrock. Piezometer 74-8L, although located in bedrock, was open only to Sandstone Units B and C - relatively low permeability aquitards which have a poor hydraulic communication with the more permeable sandstone units and the drains. This may explain the small amplitude of response to the changes in lock levels in this piezometer. Initial pressure buildup and subsequent declines noted in some piezometers, and other phenomena, are not well-enough understood to be discussed definitively.

12. Observation of Dye Movements - First Test. The purpose of the dye tests was to determine preferred seepage paths underneath the lock structures. Two tests were run. In the first test, completed over a three day period, dye was injected into piezometers on the river wall, the intermediate wall, the land wall and behind the land wall. Different colors were used to differentiate injection into the upper (Units A, A' and B) and the lower (Units D, E, F, G and H) sandstone. Inspections for dye shows were maintained during the daylight hours.

Dye injected into piezometers placed in the lower sandstone on the river wall did not emerge and is believed to have entered the ground-water body in the lower sandstone, ultimately discharging into the Mississippi River. On Tuesday, October 1, 1974, the river lock was maintained at upper pool level the entire day, while normal lockages occurred in the land lock. At 8:00 a.m. 200 grams of Rhodamine (red) dye was mixed with about 3 gallons of water and poured through a funnel into borehole 74-44, on the river wall, terminating in the lower sandstone. A hose was inserted into the borehole and the pressure adjusted with the valve so that a maximum head was put on the dye without spillage from the top of the casing. By 8:30 a.m. Fluoroscein (green) dye was similarly injected into borehole 74-48, terminating in the lower sandstone. For the remainder of the day, until about 4:30 p.m. the head was applied on the holes and all personnel were on the alert for dye shows. The channel area to the east was inspected in the morning and in the afternoon. No dye shows from these lower sandstone boreholes on the riverwall were observed.

Dye injected into the upper sandstone through piezometers on the intermediate wall came out less than one hour later downstream of the land lock miter gate. On Wednesday, October 2, the river lock was maintained at lower pool level all day, and the land lock at lower pool between lockages. Early Wednesday morning no shows were observed. By 9:30 a.m. 200 grams of Fluorescein was injected into intermediate wall boreholes 74-39 and 74-38 terminating in the upper sandstone. Water hoses were inserted in the boreholes to provide a driving head. At 10:05 a.m., a show of green dye downstream of the land lock miter gate on the I-wall side was observed. By 10:15, the show disappeared. No additional dye shows were noted that day.

Dye injected into upper sandstone piezometers on the land wall emerged less than one half hour later at the same discharge point. The dye-show discharge point is the end of a drain that runs beneath the land lock. Dye injected into the lower sandstone piezometers behind the landwall did not appear at or near the lock site. On Thursday, October 3, the locks were maintained at their normal position - river lock half full, with lockage through the land lock. Early Thursday morning, there were no dye shows. At 8:30 a.m. red dye was put in land wall borehole 74-58 terminating in the upper sandstone; at 9:00 injection of red dye in the land wall bore hole 74-52, also open to the upper sandstone was completed. Water hoses were inserted in each borehole. At 9:15 a.m., while the land lock was at upper pond, red dye was noted coming up in the same place as the green dye did the previous day. At 9:25 a.m. injection of red dye in upper and lower sandstone borehole 74-14 behind the land wall was completed. At 9:35 a.m. the land lock was lowered and the downstream miter gates were opened. The red dye was no longer visible. At 9:45 a.m., injection of the green dye in lower and upper sandstone borehole 74-16, behind the land wall, was completed. At 10:10 a.m., as the land lock was being raised, red dye showed again briefly downstream of the land lock miter gate. At 11:30 a.m., upon filling the land lock, there was a vague discoloration in the same spot for about ten minutes. No additional dye shows were observed on October 3 or 4, or any time afterwards.

13. Observation of Dye Movements - Second Test. In the second test on November 5, 1974, a diver injected dye into borehole 74-70 (in the river lock) which had exhibited suction[1/], and dye later appeared "upstream" in the land lock and downstream at the previous discharge point. With the river lock at full head and the land lock at low head, a diver inserted a 6-foot long steel pipe attached to the end of a 100-foot section of garden hose, into borehole 74-70, which was exhibiting suction. He inserted the steel rod and hose to a depth of 9 feet. The other end of the hose was attached to a stake in the grassy area of the intermediate wall. Two-hundred grams of fluorescein dye were mixed with 2 gallons of water and poured through a funnel into the hose at 2:08 p.m. A supply hose was attached to the garden hose to apply water pressure and to increase the velocity of dye movement. Within 25 minutes of dye injection into the hole a show was observed in the land lock opposite Monolith No. 7. The discharge point appears to be at the location of borehole 74-66. (See Plate B-1-1, Appendix B-1, Geology.) Within 40 minutes dye also appeared downstream of the land lock miter gate at the previous discharge point. These results indicate that there are openings beneath the intermediate wall that connect the land lock and the river lock, and that some of the boreholes are not properly sealed and allow communication between the underlying bedrock and the water in the locks.

14. Results of Observation of Dye Movements. The results of borehole water level measurements and dye tracer tests indicate a possible hydraulic connection between the upper sandstone and the drainage system beneath the land lock, and between the river lock and the land lock through the upper sandstone beneath the intermediate wall. The results also indicate that some of the boreholes are not effectively sealed.

15. Falling Head Permeability Tests. The purpose of the permeability tests was to determine the permeability of the material under the intermediate and river walls. Both the falling head

1/ Four weeks before the dye test, on October 9, 1974, the diver had felt suction while examining the lock floor.

and constant head methods were employed. In the falling head method a slug of known volume was injected into the well and the change in water level over a period of time was measured. The resultant data were used in the slug test formula[1/] to determine permeability. The falling head (slug) tests indicated a permeability of 3.7×10^{-4} cm/sec for the upper sandstone (borehole 74-38) and 4.6×10^{-4} for the lower sandstone (borehole 74-48). These results compare, in magnitude, with published values for fine-grained sandstone (2.4×10^{-4} cm/sec)[2/]; and reasonably well with published values of laboratory permeabilities for the St. Peter Sandstone in Illinois (8.4×10^{-4} cm/sec)[3/]. Two core samples of the upper sandstone (Unit A') at the Locks had an average laboratory permeability of 1.2×10^{-4} cm/sec.

16. <u>Constant Head Permeability Tests</u>. The constant head tests are not considered valid; first, because the constant head formula used in calculations is meant for a different type of construction than the gravel-packed piezometers, and second because of the low precision of the testing set-up (no flow meters or pressure gauging equipment) and lack of accurate control of the flow rate to maintain a constant head.

17. <u>Results of Permeability Tests</u>. The results of the field permeability tests indicate a coefficient of permeability of 3.7×10^{-4} cm/sec for the upper sandstone (Units A, A', and B) and 4.6×10^{-4} cm/sec for the lower sandstone (Units D, E, F, G and H). Two core samples of the upper sandstone (Unit A') had an average laboratory permeability of 1.2×10^{-4} cm/sec.

1/ Ferris, J.G., and others, 1962, Theory of Aquifer Tests, U.S. Geological Survey Water-Supply Paper 1536-E, p. 104-110 and

Mogg, J.L., 1962, Well Performance Shown by Simple Test, Johnson Drillers Journal, May-June, p. 1-3.

2/ Morris, D.A., and Johnson, A.I., 1967, Summary of Hydrologic and Physical Properties of Rock and Soil Materials, As Analyzed by the Hydrologic Laboratory of the U.S. Geological Survey, 1948-1960, U.S. Geol. Survey Water Supply Paper 1839-D.

3/ Davis, John C., 1973, Statistics and Data Analysis in Geology, John Wiley and Sons, New York, p. 112.

RECOMMENDED SCHEME FOR CONTROLLING SEEPAGE
AND ELIMINATING POSSIBLE PIPING

18. Downstream Cut-Off. The dye tests (see paragraphs 12 through 14) indicated there is a possibility of past movement and future piping of material in a downstream direction from beneath the floors of both locks and under the intermediate lock wall. Therefore, it seems imperative to construct a filter and drainage system into the modification of the emptying systems for the rehabilitation schemes which will minimize the possibility of the loss of material from under drains in the lock floors and intermediate wall at the downstream end of locks.

19. Gravel Drain. Plates D-2 and D-3 show the recommeded Hydraulic Improvements-Downstream Structures for Plans 1, 2, 3 and 4. For all of these emptying schemes a 1'-6" layer of pea gravel would be placed under the concrete. For the landward lock the pea gravel would lie directly on the sandstone foundation. For the landward lock the pea gravel drain would tie into the existing drainage system at the downstream miter sill. A two layer filter of 9 inches each may be required to meet filter criteria. If so, the top layer would be pea gravel.

20. Cut-Off in Sandstone. At the downstream end of both systems the drains would terminate at a header, a 24"ø perforated pipe under the downstream end of the emptying system. (See Section A-A Plate D-2). The header would drain through a series of pipes buried in the concrete sill. The exit of these pipes would be covered with a grate. The grate would be under the 4 feet thickness of riprap placed downstream of the emptying system. The header would be surrounded by gravel. A 2 feet thick filter layer would be placed between the gravel and the foundation rock to prevent possible piping of uncemented sandstone or river alluviums. The downstream sill would be excavated well into the sandstone to assure a positive cut-off and prevent any possibility of piping which could bypass the filters and drains.

21. Cut-Off in Alluvium. The foundation under the emptying system for the riverward lock is alluvium. The drainage and filter system is similar to that proposed for the landward lock except the end sill is extended to rock by a concrete cut-off wall constructed by the slurry-trench method. The cut-off wall would tie into the sill of the landward emptying system and would

extend all the way across the downstream end of the river side to join the existing riverward lock wall. The concrete cut-off would extend into the sandstone and would have a minimum thickness of 2'-6". The cut-off is shown on Sections B-B and C-C on Plate D-2.

22. Compaction Grouting Under Intermediate Wall and Downstream Portion of River Wall. The drainage system and filter described above (paragraphs 18 through 21) will give some added protection against possible piping from the floors of the locks and from the intermediate wall. If loss of material has occurred in the past, it would be desirable to fill these voids. Also the drilling indicated that there may be areas of unconsolidated material under the intermediate wall. Therefore it is desirable to grout in these areas to reduce the possibility of future movement of material and to improve locally the existing foundation conditions, that is, bearing capacity and strength. The grouting would also serve as an intensive exploration program. The compaction grouting is primarily to densify and strengthen the foundation by displacement. It will also fill any favored seepage paths. It is not intended that the alluvial materials will be grouted with cement grout. If the sediments are to be grouted a chemical grout should be used. In general low pressure groutings is anticipated.

23. Drains Under Landward Lock. It is important the drains under the landward lock be kept open and that they not be grouted up during the grouting operation. The drainage system underneath the land lock consists of six rows of 12-inch diameter standard unglazed tile drains, laid longitudinally at 10-foot intervals beneath the concrete floor slab. The pipe drains are placed in 18 by 14 inch trenches excavated in sandstone and filled with gravel, graded from 0.5 to one inch. At the downstream end the drains are carried through the concrete miter sill using 24 gauge corrugated iron pipes. At the low point at elevation 666.7 in the concrete sill the drains are interconnected by means of a 3 by 2 foot box drain. In addition to the above drainage system a single 12-inch corrugated iron pipe is carried from beneath the intermediate wall Monolith No. 16 through the concrete miter gate sill. This drain is connected to the other drains by the 3 by 2 foot box drain at elevation 666.7.

24. Location and Depth of Grouting. The grouting would be done from the floor of the filling conduits within both the intermediate lock wall and the river wall and from the floor of the two locks,

if required, with angle holes under the intermediate wall. It is anticipated that the depth of grouting would be from 6 to 12 feet under the intermediate wall and 12 to 24 feet under the downstream portion of the river wall. Some grout holes will be extended through Unit B at both the intermediate and river wall.

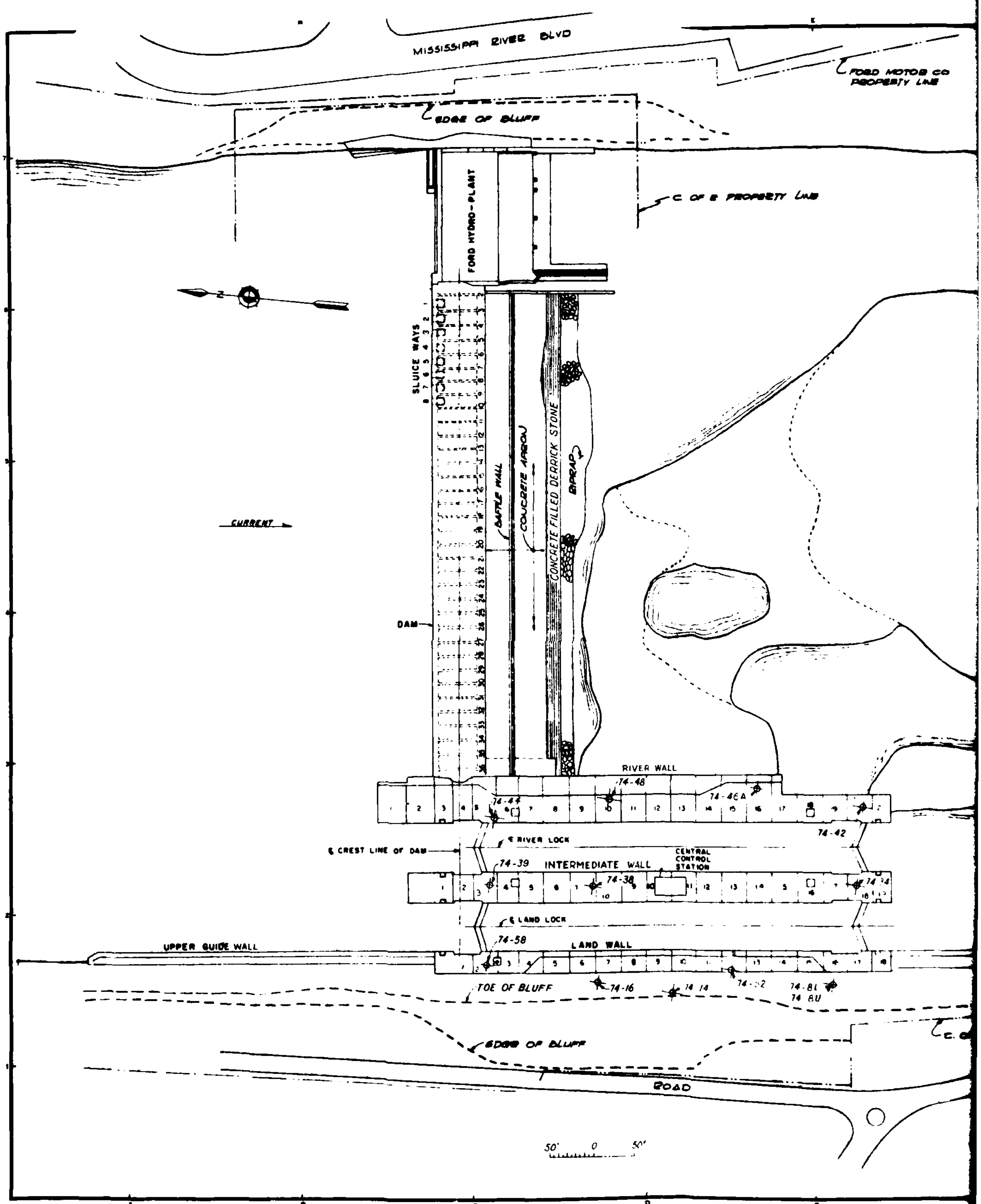
25. Type of Grout. The grout used in the unconsolidated material would be a low slump grout consisting of a portland cement-silty sand mix probably about 2 to 4 sacks of cement per cubic yard of silty sand. More conventional cement grout would be used directly under the structure with the volume of grout injected being limited by volume control to help prevent grouting of the floor drains. The purpose of this grouting is to fill voids at the concrete foundation contact and favored seepage paths. If grouting of sediments is required chemical grout will be used.

26. Amount of Grouting Beneath Intermediate Wall. With the exploratory nature of this grouting it is difficult to estimate the number of holes and quantity of grout required to improve the foundation conditions. The initial holes under the intermediate wall would be double row the full length of each conduit, approximately 400 feet. The holes would be spaced 12 feet on centers with the holes staggered. This would result in a total of 132 initial holes and a total length of grouted hole of 1200 feet. In addition there would be 800 feet of drilling through the concrete below the conduit. It is anticipated that two additional rows of holes per conduit could be required and that final spacing could be 6 feet on center on the outer rows instead of the initial 12 feet spacing. Therefore, it seems reasonable to assume that 330 holes in the intermediate wall with 3,000 feet of hole grouted and 2,000 feet of hole through concrete is a reasonable assumption of the treatment required. A grout take of one cubic foot per foot of hole would give a total grout take of 100 cubic yards.

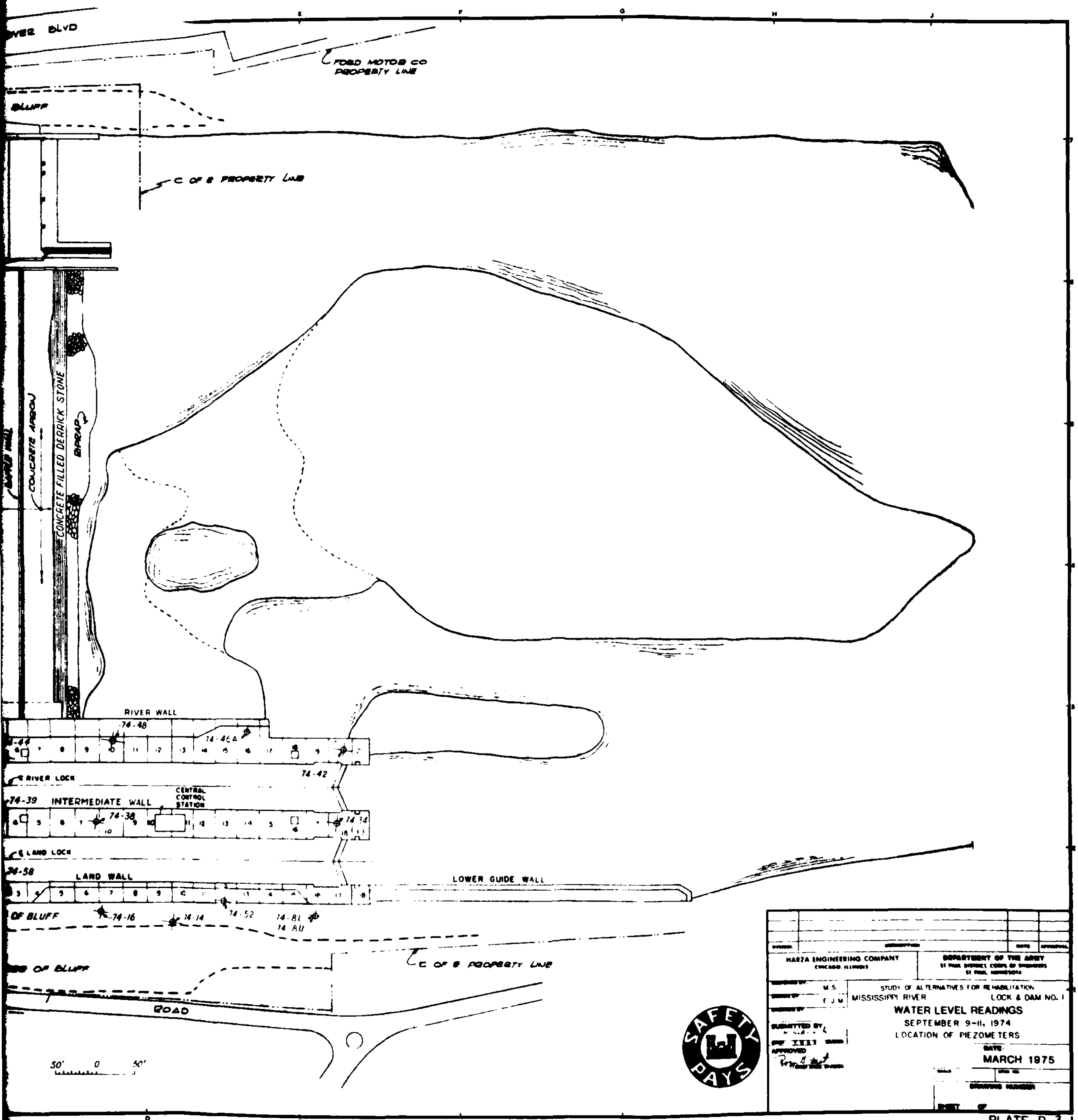
27. Amount of Grouting Beneath River Wall. For the downstream portion of the river wall the treatment will be limited to Monoliths 17 through 21. The depth of holes is assumed to be 24 feet deep. The width of these monoliths is about 32 feet. Assuming approximately 6-foot sapcing of holes would require 3 rows to cover the area. The length of each row would be about 140 feet. This would require about 25 holes per row for a total of 75 holes. The total length of grouted hole would be 1,800 feet and the length through concrete would be 450 feet. A grout take of one cubic foot per foot would give a total take of 50 cubic yards.

28. <u>Backfill Against Riverward Side of Downstream End of River Wall</u>. One recommended method (see Appendix C, Paragraph 57) of stabilizing monoliths 17 through 21 in the downstream end of the river wall is to place fill against the outside of the wall. This fill is shown in plan on Plate E-3. A section through the fill is shown on Section E-E on Plate E-4. The fill would be placed to El. 710 and would have an outside slope of 2.5 horizontal to 1.0 vertical. This outside slope would be protected by selected large rock. The fill would be underlain by a filter blanket to prevent possible piping from seepage under the lock wall. Also the alluvium beyond the blanket would be protected with an 18-inch thick filter to prevent piping of material toward the river.

29. <u>Selective Removal of Lock Basin Floor Slabs in Both Locks to Investigate Possibility of Piping and Existence of Seepage Channels and Proposed Methods of Correction</u>. The dye tests (Paragraphs 12 through 14) indicated that the possibility exists that the drains under the landward lock basin floor may be interconnected with pipes or existing seepage channels under the intermediate wall. These drains are 12" round pipes laid the length of the basin. It is not clear from the available information if these pipes are filtered or not. Selected slabs over the drains should be removed to determine the type and condition of the filter. There was no evidence of piping under the landward wall and the low groundwater level in both the bluff and the backfill behind the wall indicates that the presence of pipes or channels is much less likely. It is, nevertheless, a possibility which should be explored. The existence of at least one seepage channel under the concrete under the riverward lock basin floor was discovered by the divers. The entrance to this channel was located at drill hole 74-70. The floor of the riverward lock basin has 6 inch holes in the center of each 10 feet by 10 feet floor slab. There is no evidence that these holes are filtered. After the basin is dewatered during the construction period these holes should be examined and probed. Any slab containing a hole which is suspect of having piped should be removed for closer inspection. These holes provide pressure relief during downstream and, therefore should not be plugged. It may be desirable to plug some of these holes or provide a permeable cap and filter. The riverward lock wall is supported on piling, making it much more likely that in some areas the foundation may have settled away from the concrete base of the wall. It is recommended that the selected slabs be removed from both lock basin floors before the grouting program described in paragraphs 22 through 27 is initiated. The slabs would remain exposed during the grouting to allow observation of entry of the grout into the drains and to assist in determining the travel paths of the grout.

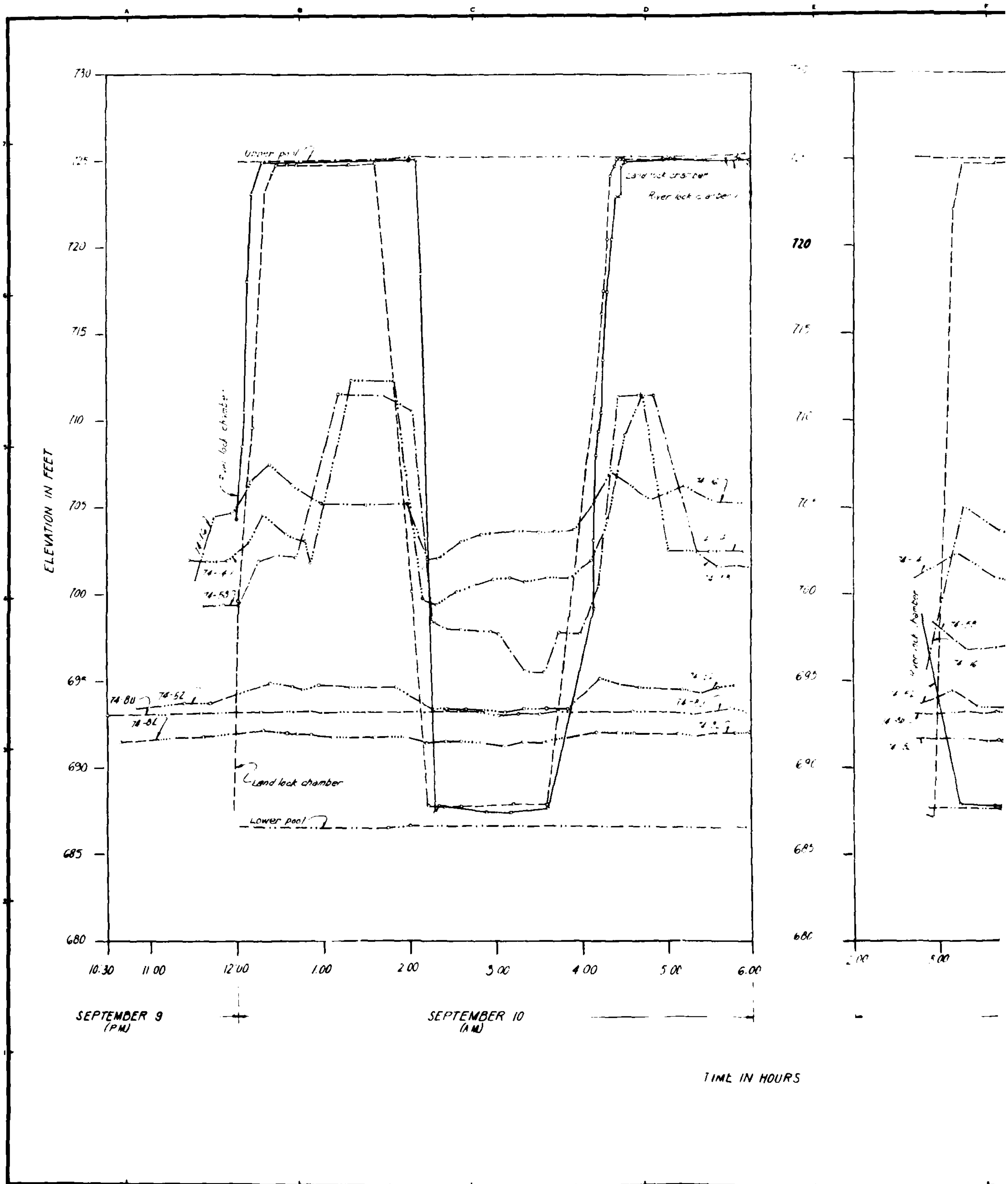


MISSISSIPPI RIVER BLVD
FORD MOTOR CO PROPERTY LINE
EDGE OF BLUFF
FORD HYDRO-PLANT
C OF E PROPERTY LINE
SLUICE WAYS
BAFFLE WALL
CONCRETE APRON
CONCRETE FILLED DERRICK STONE
RIPRAP
CURRENT
DAM
RIVER WALL
74-48
74-44
74-46A
74-42
℄ RIVER LOCK
℄ CREST LINE OF DAM
74-39
INTERMEDIATE WALL
CENTRAL CONTROL STATION
74-38
℄ LAND LOCK
74-58
UPPER GUIDE WALL
LAND WALL
TOE OF BLUFF
74-16
74-14
EDGE OF BLUFF
ROAD
50' 0 50'

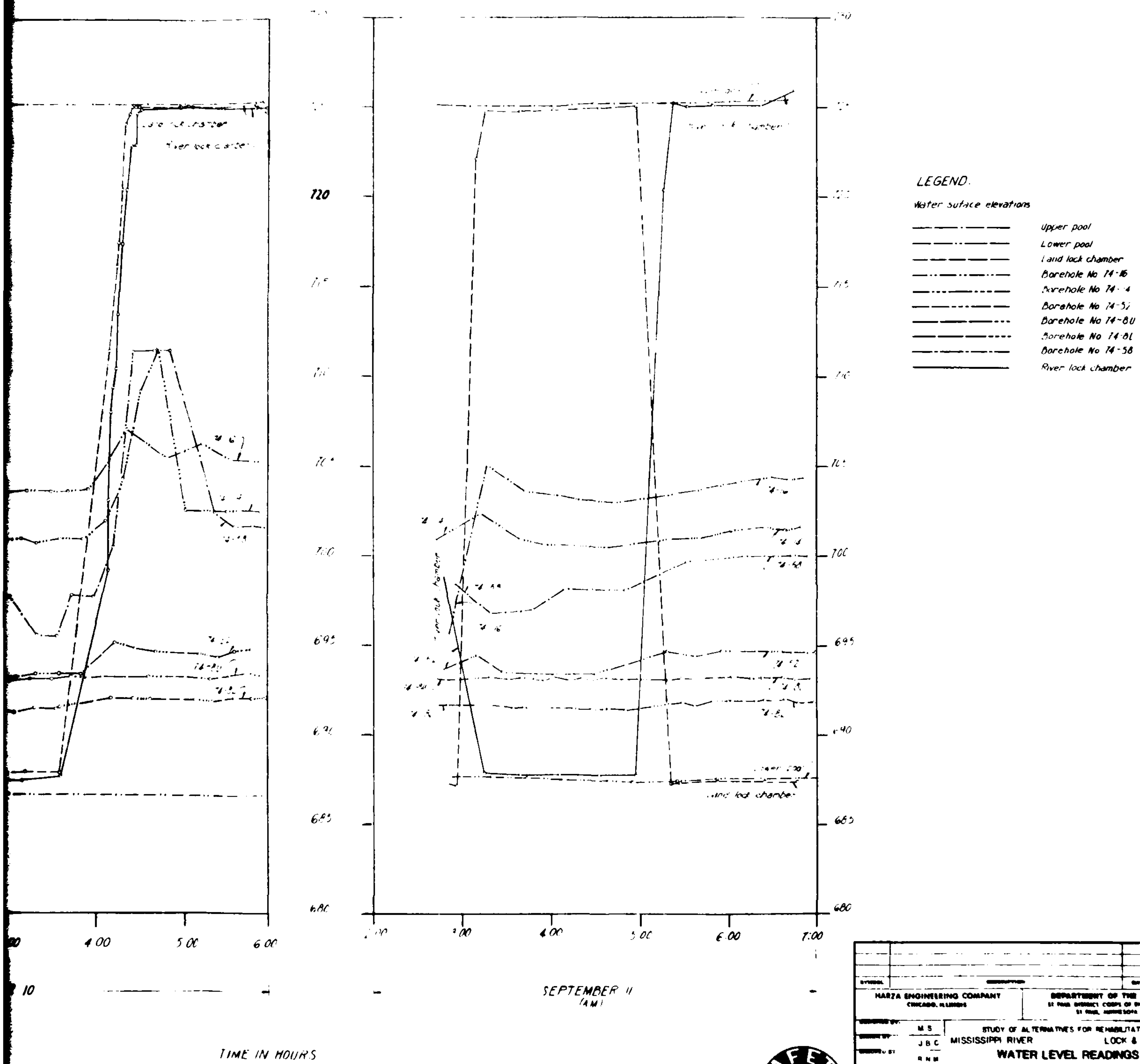


RIVER BLVD
FORD MOTOR CO PROPERTY LINE
BLUFF
C OF E PROPERTY LINE
CONCRETE APRON
CONCRETE FILLED DERRICK STONE
RIPRAP
RIVER WALL
74-48
74-46A
74-42
RIVER LOCK
74-39
INTERMEDIATE WALL
CENTRAL CONTROL STATION
74-38
74-34
LAND LOCK
74-58
LAND WALL
LOWER GUIDE WALL
OF BLUFF
74-16
74-14
74-52
74-8L
74-8U
C OF E PROPERTY LINE
OF BLUFF
ROAD
50' 0 50'
SAFETY PAYS
HARZA ENGINEERING COMPANY
DEPARTMENT OF THE ARMY
M S
F J M
STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER
LOCK & DAM NO. 1
WATER LEVEL READINGS
SEPTEMBER 9-11, 1974
LOCATION OF PIEZOMETERS
SUBMITTED BY
APPROVED
DATE:
MARCH 1975
DRAWING NUMBER
SHEET OF

PLATE B-3-1



ELEVATION IN FEET
730
725
720
715
710
705
700
695
690
685
680
Upper pool
Land lock chamber
River lock chamber
Land lock chamber
Lower pool
74-80
74-52
74-86
74-59
10:30
11:00
12:00
1:00
2:00
3:00
4:00
5:00
6:00
SEPTEMBER 9 (PM)
SEPTEMBER 10 (AM)
TIME IN HOURS



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ST. PAUL, MINNESOTA

M.S.
J.B.C.
R.N.M.

STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO.

WATER LEVEL READINGS
SEPTEMBER 9-11, 1974
LAND WALL

SUBMITTED BY:
APPROVED:

DATE
MARCH 1975

PLATE B-3-

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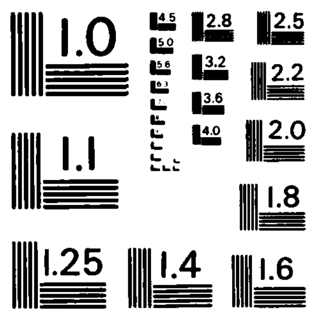
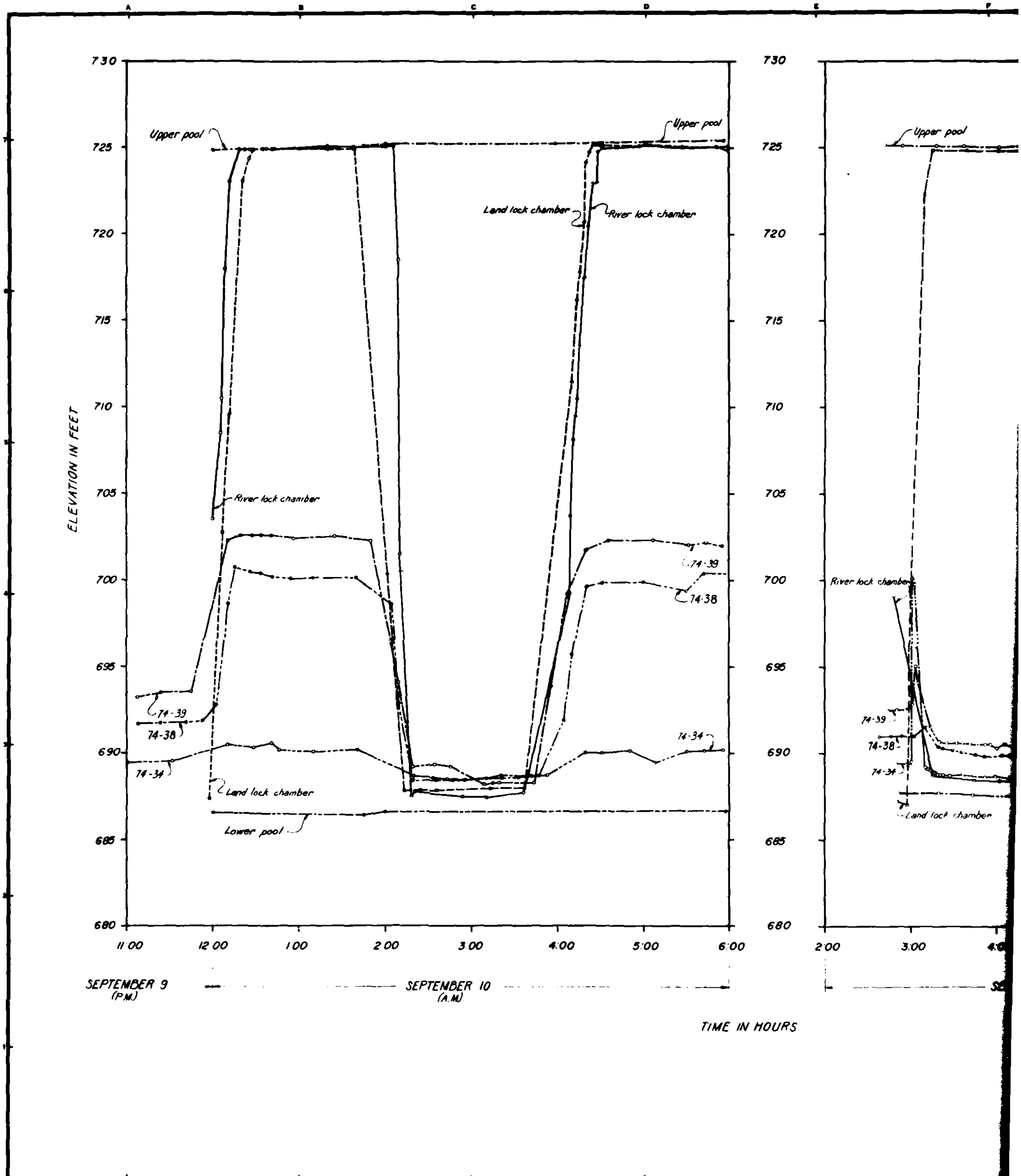
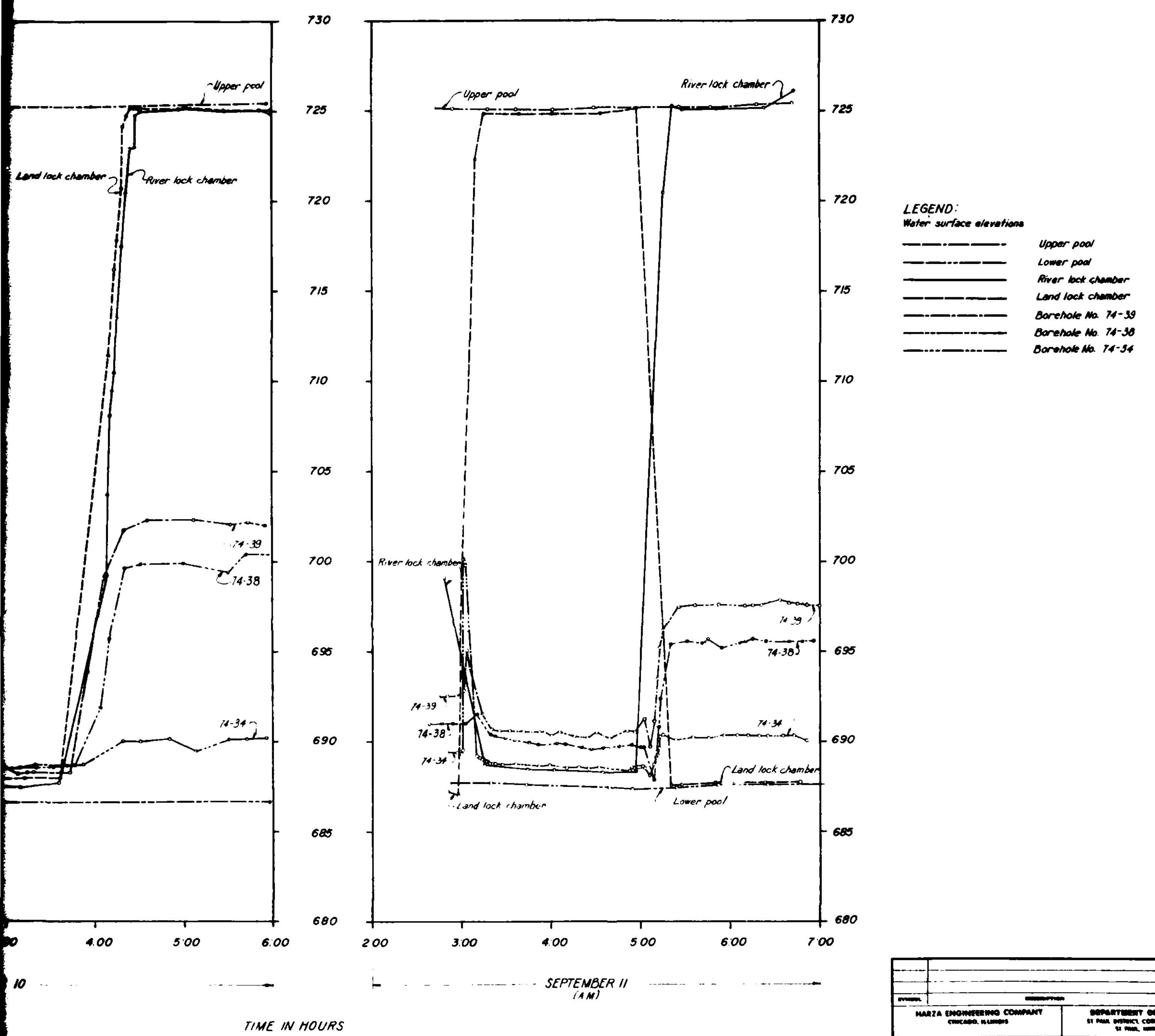


MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS - 1963 - A



730
725
720
715
710
705
700
695
690
685
680
ELEVATION IN FEET
Upper pool
Land lock chamber
River lock chamber
74-39
74-38
74-34
Lower pool
11:00
12:00
1:00
2:00
3:00
4:00
5:00
6:00
SEPTEMBER 9 (P.M.)
SEPTEMBER 10 (A.M.)
TIME IN HOURS



SAFETY PAYS

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ST. PAUL DISTRICT, CORPS OF ENGINEERS
ST. PAUL, MINNESOTA

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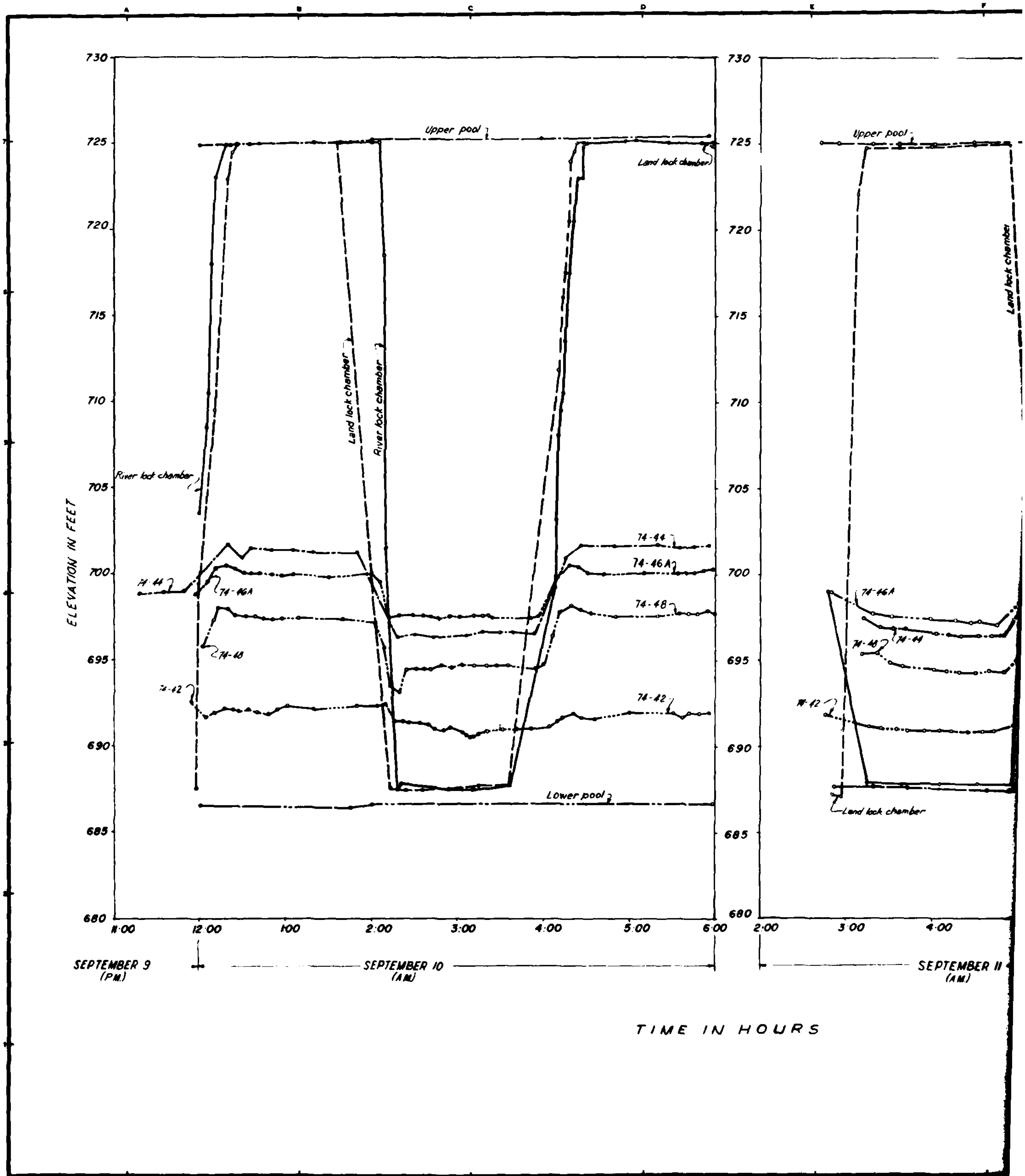
STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM

WATER LEVEL READINGS
SEPTEMBER 9-11, 1974
INTERMEDIATE WALL

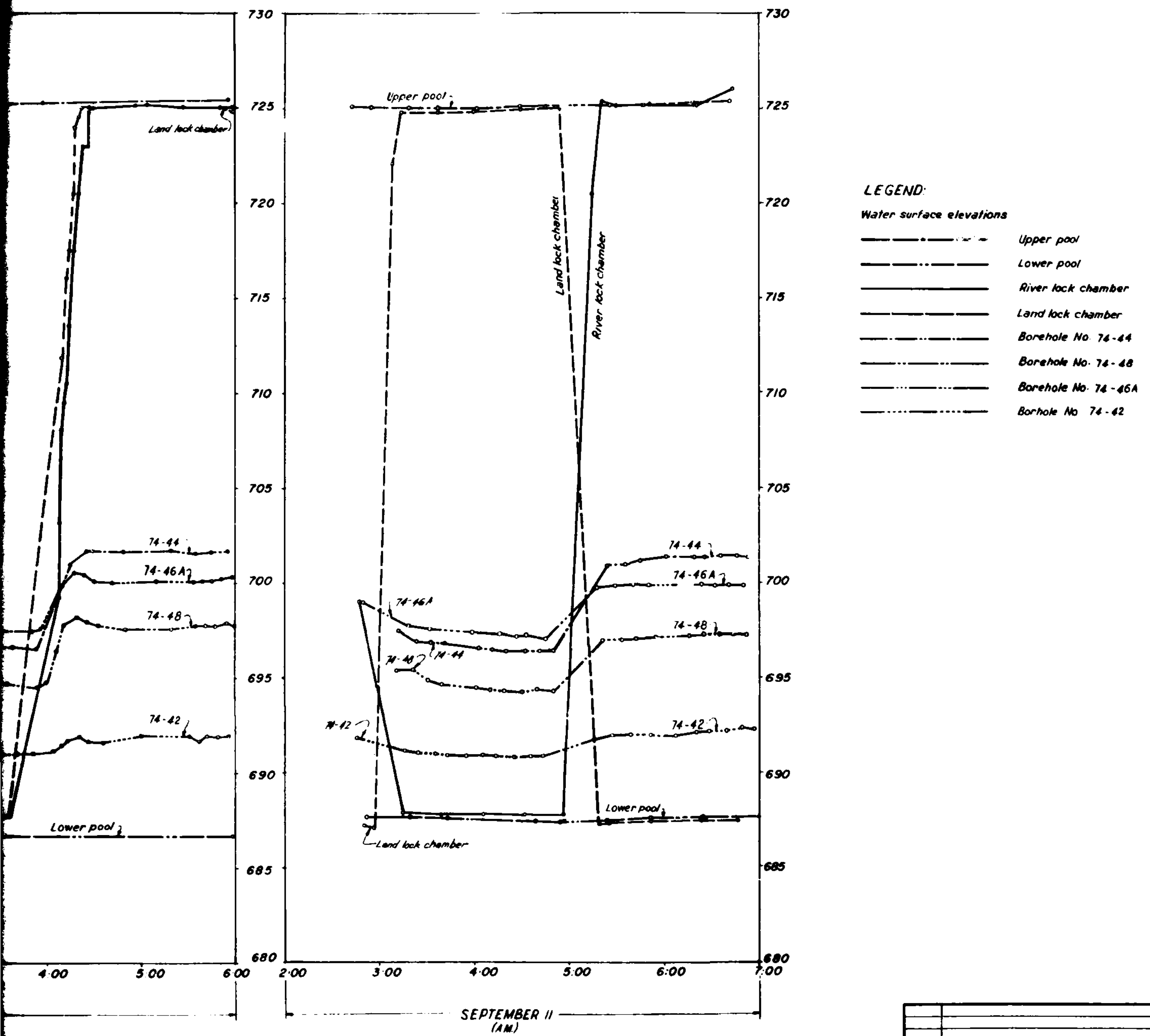
DATE:
MARCH 197

PLATE B-

1 1 2



730
725
720
715
710
705
700
695
690
685
680
ELEVATION IN FEET
Upper pool
Land lock chamber
River lock chamber
74-44
74-46A
74-48
74-42
Lower pool
11:00
12:00
1:00
2:00
3:00
4:00
5:00
6:00
SEPTEMBER 9 (PM)
SEPTEMBER 10 (AM)
3:00
4:00
SEPTEMBER 11 (AM)
TIME IN HOURS



TIME IN HOURS

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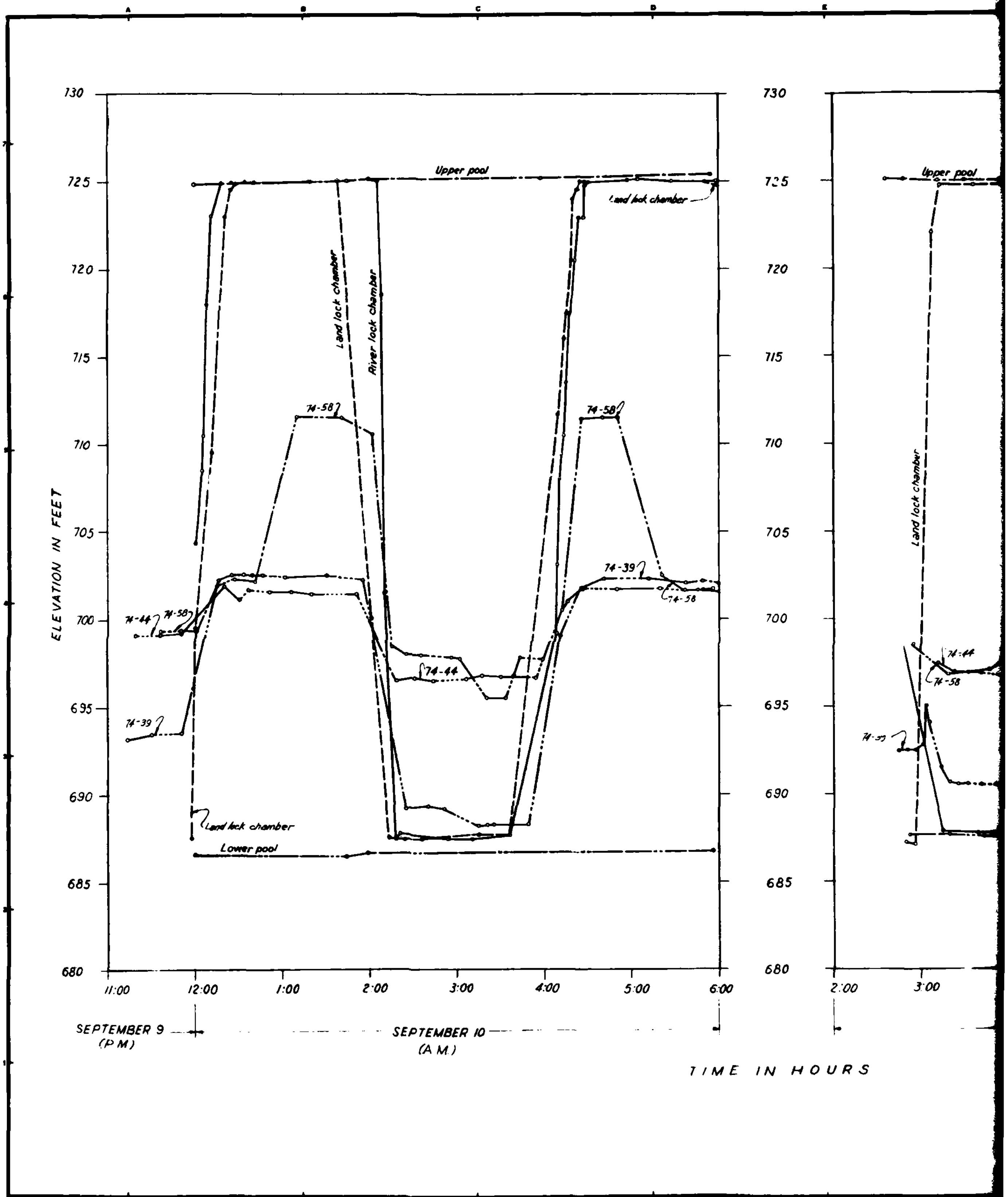
STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO. 1

WATER LEVEL READINGS
SEPTEMBER 9–11, 1974
RIVER WALL

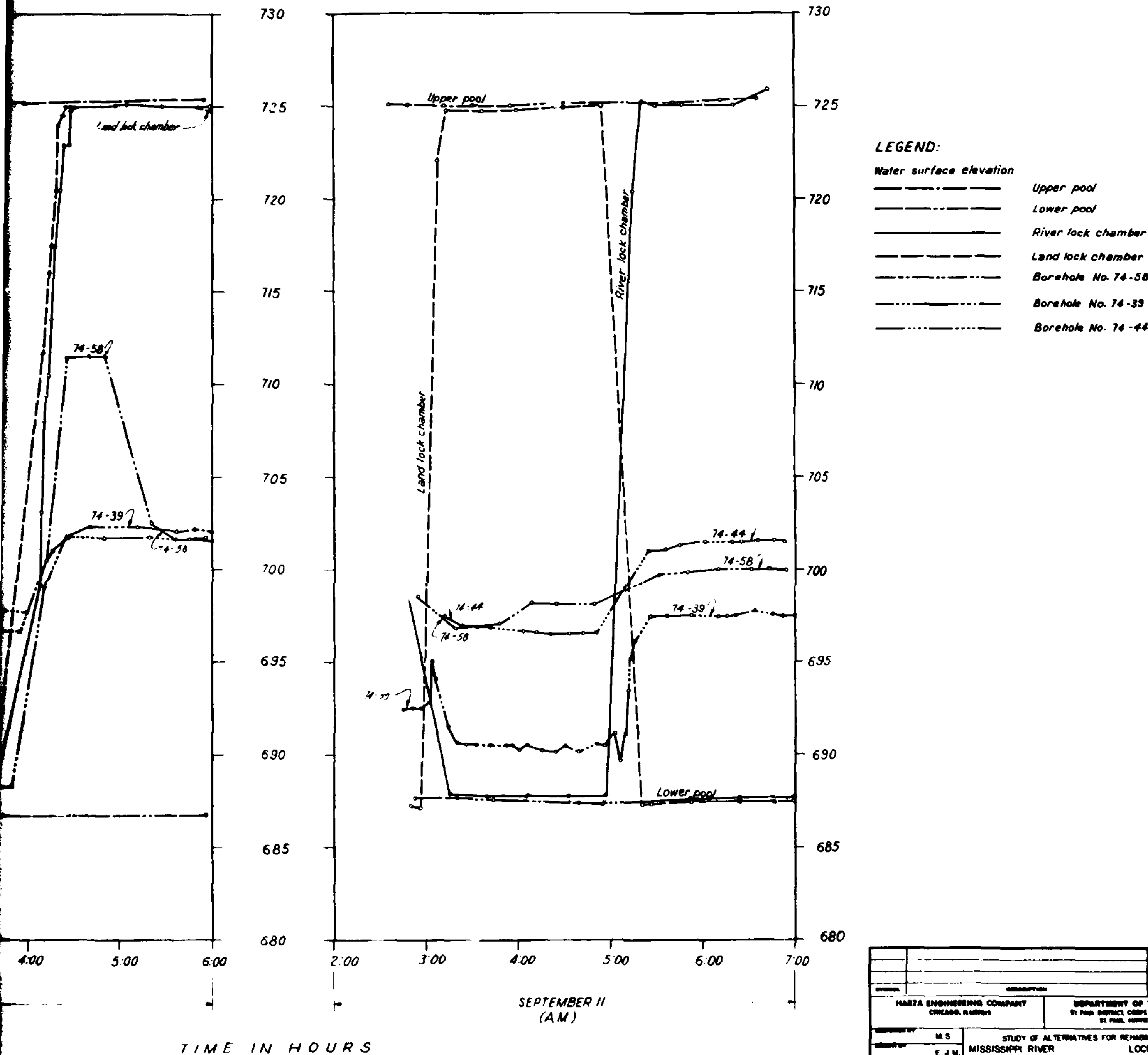
DATE: MARCH 1975

PLATE B-3-4

2



Upper pool
Land lock chamber
River lock chamber
Lower pool
74-58
74-44
74-39
ELEVATION IN FEET
730
725
720
715
710
705
700
695
690
685
680
11:00
12:00
1:00
2:00
3:00
4:00
5:00
6:00
SEPTEMBER 9 (PM)
SEPTEMBER 10 (AM)
TIME IN HOURS



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MISSISSIPPI RIVER LOCK & DAM NO. 1

WATER LEVEL READINGS

SEPTEMBER 9–11, 1974

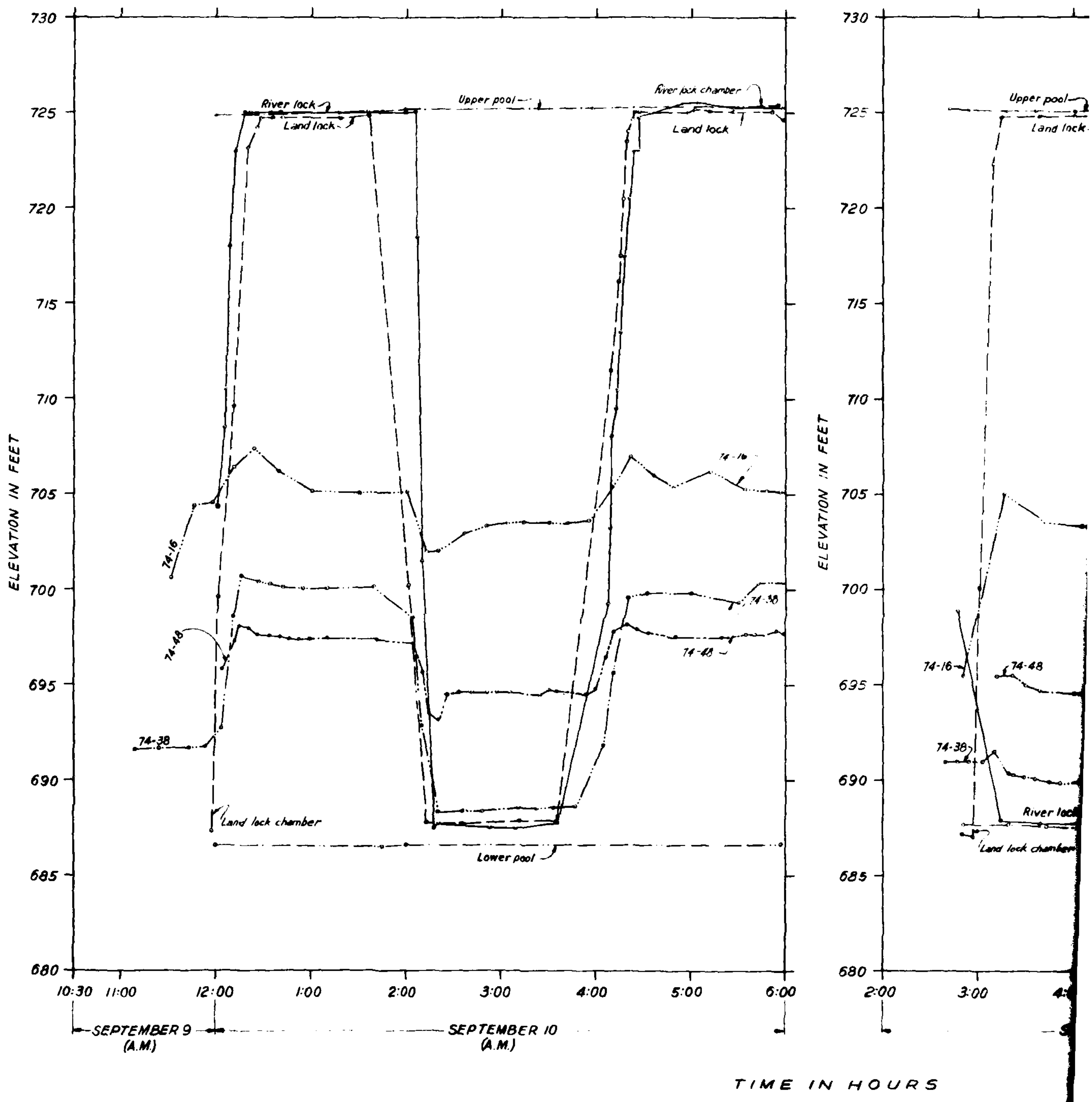
LANDWALL MONO. 2 TO RIVERWALL MONO. 5

DATE:
MARCH 1975

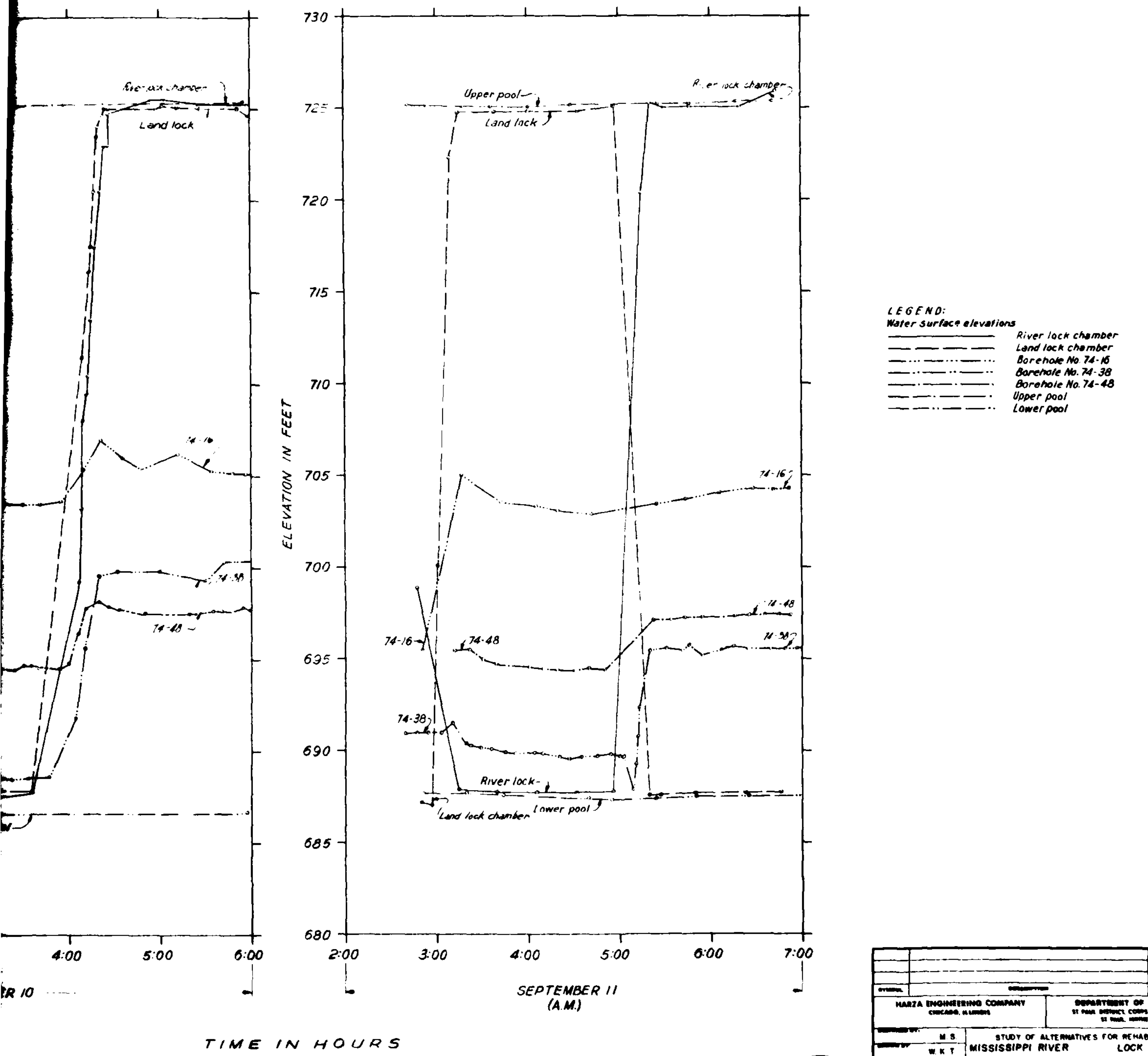
DRAWING NUMBER

SHEET OF

2



ELEVATION IN FEET
730
725
720
715
710
705
700
695
690
685
680
River lock
Land lock
Upper pool
River lock chamber
Land lock
74-16
74-38
74-48
Land lock chamber
Lower pool
10:30
11:00
12:00
1:00
2:00
3:00
4:00
5:00
6:00
SEPTEMBER 9 (A.M.)
SEPTEMBER 10 (A.M.)
Upper pool
Land lock
74-16
74-48
74-38
River lock
Land lock chamber
2:00
3:00
TIME IN HOURS



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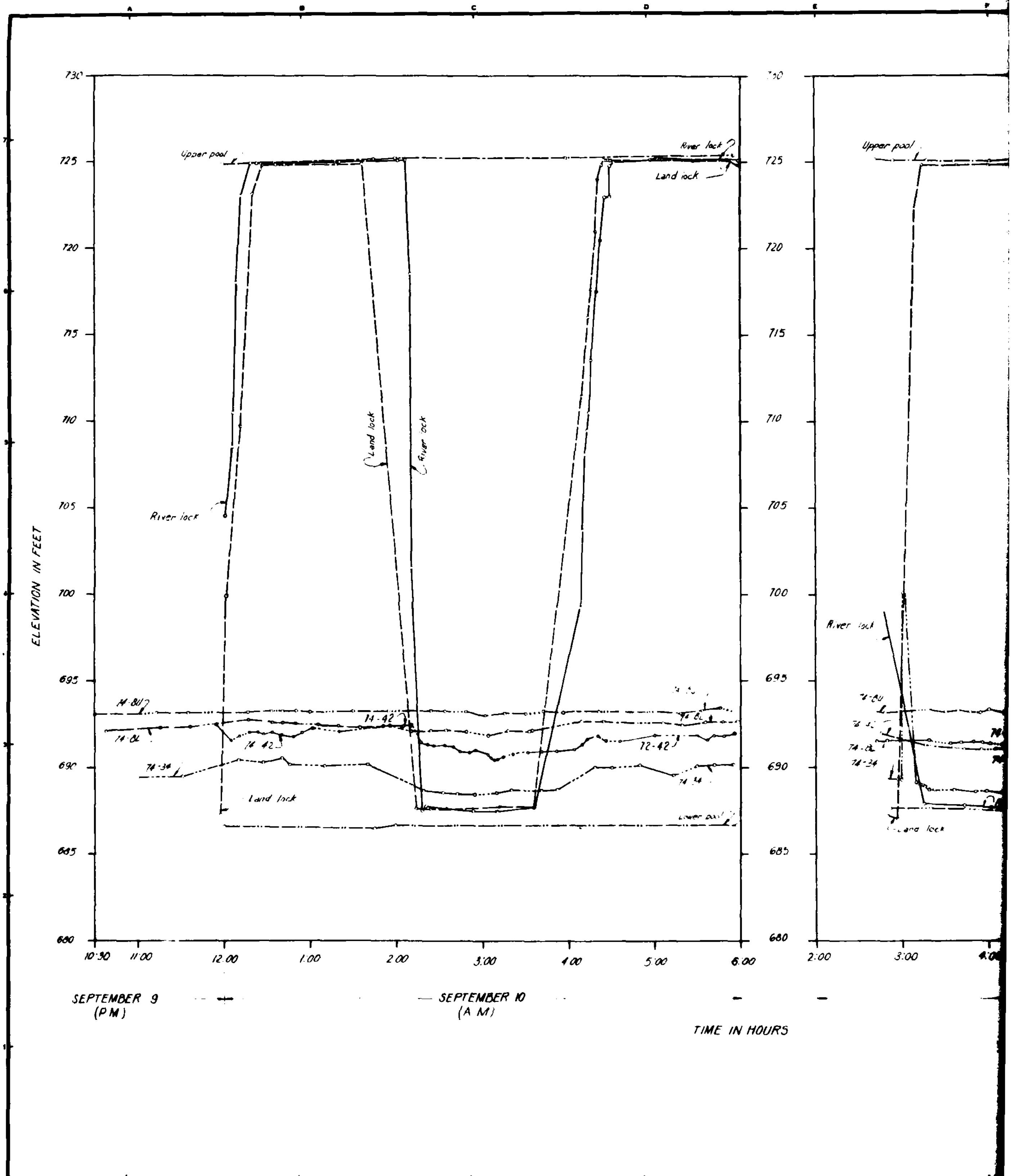
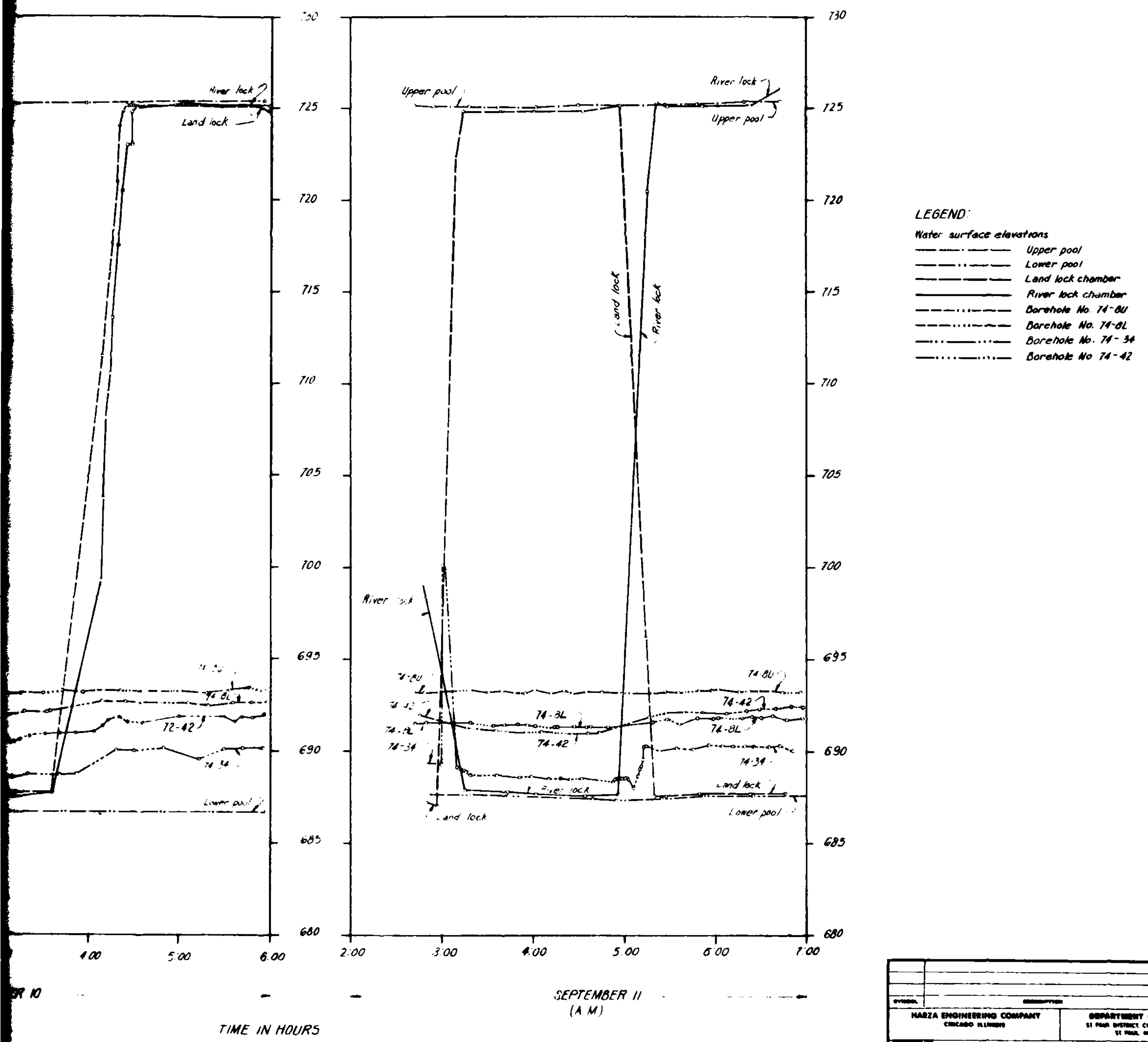
M. S. W. K. T. K. N. M. SUBMITTED BY: APPROVED:	STUDY OF ALTERNATIVES FOR REHABILITATION MISSISSIPPI RIVER LOCK & DAM NO 1 **WATER LEVEL READINGS.** SEPT 9-11, 1974 LANDWALL MONO 7 TO RIVERWALL MONO 10 DATE: MARCH 1975

PLATE B-3-6



ELEVATION IN FEET
Upper pool
River lock
Land lock
Lower pool
74-80
74-8L
74-42
72-42
74-34
SEPTEMBER 9 (PM)
SEPTEMBER 10 (A M)
TIME IN HOURS

1



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ST PAUL DISTRICT, CORPS OF ENGINEERS
ST PAUL, MINNESOTA

M.S
J B C
R N M

STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO 1
WATER LEVEL READINGS
SEPT 9-11, 1974
LANDWALL MONO 18 TO RIVERWALL MONO 20

SUBMITTED BY:
APPROVED:
DATE: MARCH 1975

PLATE B-3-7

DEPARTMENT OF THE ARMY
St. Paul District, Corps of Engineers
1210 U. S. Post Office & Custom House
St. Paul, Minnesota 55101

MISSISSIPPI RIVER

STUDY OF ALTERNATIVES FOR REHABILITATION OF LOCK AND DAM NO. 1

MINNEAPOLIS, MINNESOTA

APPENDIX C

STRUCTURAL INVESTIGATIONS

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APPENDIX C

STRUCTURAL INVESTIGATIONS

DESIGN CRITERIA

SOIL, ROCK AND CONCRETE PARAMETERS

1. The soil and rock parameters used in the structural analyses were derived from or confirmed by laboratory tests on samples recovered during the 1974 exploration program. The following parameters were utilized:

a) Land wall, Monoliths 1-7 Upper Guide Wall, and Monoliths 1-2 Lower Guide Wall.

Material	Unit Weight pcf	Angle of Internal Friction degrees	Cohesive Strength psf
Moist backfill	115	38	0
Saturated backfill	130	38	0
Submerged backfill	68	38	0

b) Monoliths 8-13 Upper Guide Wall and Monoliths 3-13 Lower Guide Wall.

Material	Unit Weight pcf	Angle of Internal Friction degrees	Cohesive Strength psf
Moist backfill	115	35	0
Saturated backfill	125	35	0
Submerged backfill	63	35	0

In the analyses the weight of concrete has been assumed to be equal to 150 pcf and the weight of water 62.5 pcf.

EARTH PRESSURES

2. The computed earth pressures, assumed to be near at rest condition, were based on Jaky's formula, $Ko = 1 - \sin \phi$. The land and guide wall monoliths have been considered to behave essentially as rigid structures. Therefore at-rest earth pressure coefficients have been modified by reduction factors considering the deformation history of the monoliths and the nature of the granular backfill materials. The resulting value of K used in the analyses is 0.36.

The stability analyses utilized the following lateral pressures to calculate earth loads:

Moist granular backfill 42 pcf/ft of depth
Submerged granular backfill 26 pcf/ft of depth

To determine the available passive resisting force due to river alluvium east of river wall monoliths, an earth pressure coefficient of 1.5 was used.

SLIDING

3. Angles of internal friction from 37° to 40° were obtained from laboratory test results for foundation materials underneath the lock walls. For sliding along the contact planes between the concrete (pile foundation underneath the river wall) and underlying soil an angle of internal friction equal to 32°, corresponding to a friction coefficient of 0.625 was used in the analyses. The river alluvium underneath the dam was estimated to have an angle of internal friction equal to 33°, corresponding to a friction coefficient of 0.65 in soil to soil contact. However, considering uncertainties regarding the nature of soils and the location and configuration of the sliding surface underneath the dam a more conservative friction coefficient of 0.55 was used in the computations. For all of the above cases a cohesive strength C = 0 psf was used. For sliding stability analyses involving soft, plastic seam in the upper layer of siltstone (unit "C") the values of ϕ= 23° and C = 0 psf were used.

WATER SURFACE ELEVATIONS

4. A review of the water level readings in observation wells installed during the 1974 investigations and the historical flow records for the Mississippi River at Lock and Dam No. 1 resulted in the following water level elevations used in various studies:

	Elevation in Feet M.S.L.Datum
Backfill, upstream end of land wall	704
Backfill, downstream end of land wall	700
Normal upper pool	725.2
Normal lower pool	687.2

The following peak flood elevations recorded during the years 1951 through 1972 were used in the analyses where applicable.

Maximum upper pool	734.7
Maximum lower pool	719.0

UPLIFT

5. The uplift pressure under the land wall is determined as hydrostatic pressure on the landward side plus additional pressure formed by hydraulic gradient, assumed to vary as a straight line up or down to hydrostatic pressure on the lock side. Linear variation of hydraulic gradient is used for both the intermediate and the river walls to determine the uplift. Flow net analysis is used to determine uplift under dam sections.

DESIGN MANUALS

6. General design criteria for lock structures have been outlined in the following Engineering Manuals:

EM 1110-2-1903	Bearing Capacity of Soils
EM 1110-2-2200	Gravity Dam Design
EM 1110-2-2502	Retaining Walls
EM 1110-2-2601	Navigation Locks
EM 1110-2-2602	Planning and Design of Navigation Lock Walls and Appurtenances
EM 1110-2-2606	Navigation Lock and Dam Design
EM 1110-345-147	Procedures for Foundation Design of Buildings and other structures (Except Hydraulic Structures).

PREVIOUS ANALYSES

7. Stability analyses were prepared for the lock walls in 1968-69 and for the dam in 1943.

FACTOR OF SAFETY AGAINST SLIDING

8. The field investigations and laboratory tests indicate that the foundation materials beneath the land and intermediate walls are more characteristic of a granular soil than a sound rock. Consequently the factors of safety against sliding were analyzed assuming that all structures are bearing on soil foundations. The factor of safety for these stability analyses was 1.5.

ALLOWABLE FOUNDATION PRESSURES

9. Calculations prepared for the existing conditions reveal that factors of safety for bearing capacity can be significantly below acceptable values. Rehabilitation concepts have been based on the criteria that the factor of safety for bearing capacity should not be less than 1.5. The maximum allowable foundation pressure is 20 ksf based on the foundation strengths determined from laboratory tests and Terzaghi's Bearing Capacity Factors, considering eccentric loading.

ALLOWABLE PILE LOADS

10. The existing wood piling was analyzed to determine the bearing capacity of the piles. The relatively short length of the piles limit their capacity to 8-12 kips per pile. The foundation exploration in the alluvium under the piling indicates that the foundation material is dense for the entire depth to top of rock. The presence of the limestone slabs in the alluvium would indicate that the possibility of damage to the piling during driving was very high. The maximum required piling capacities to carry the entire load, as piling is usually designed to take, are computed to be 57 kips per pile for Monoliths Nos. 6-16 for normal operating conditions. For Monolith No. 20 the maximum load is 294 kips per pile. There are no batter piles under the riverward lock wall. The computed average horizontal loading on the piles is 48 kips. This loading is about ten times a normal allowable load for wood piles where no field load tests were conducted. With the shallow piles, poor driving condition, low capacity compared to required loading for both vertical and horizontal loads, high foundation material strengths and relatively minor horizontal movement on loading, it is concluded that the existing piles do not function as separate load bearing members, but act together with soil material as a monolithic downward extension of the overlying river wall. Thus, the

effective base elevation of a given monolith can be assumed to occur at some depth below the monolith. At such depths the allowable foundation pressure is 20 ksf.

PRESENT CONDITIONS

UPPER GUIDE WALL

11. The 400 foot long upper guide wall, shown on Plate C-1, consists of two sections. The upstream section is built on rock filled timber cribs, while the downstream section rests directly on St. Peter Formation (Unit A) Sandstone.

12. Monoliths 1-7. Monoliths Nos. 1 through 7, which form the downstream section of the upper guide wall, are 27 feet high and 5 feet wide at the top. The base width of the monoliths is 14 feet and there is a 5 foot wide key beneath the landward side. The depth of the key varies from 1.5 to 3 feet. Monoliths Nos. 1 and 2 will be replaced by two 20 foot wide intake monoliths that will be designed to satisfy required stability criteria. During the construction period, also Monoliths Nos. 3 through 6 will be unwatered within the cofferdam area. Soldier beams and lagging, shown on Section C-C, Plate E-1 of Appendix E, Cofferdams, will serve as retaining wall during the construction of Monoliths Nos. 1 and 2. With the upper pool at elevation 725.2, typical monoliths were analyzed as shown on Plate C-4. Results of the analyses are summarized below.

	Operating Conditions Monoliths 1 through 7	Construction Condition Monoliths 1 through 6
Factor of safety against sliding	1.80	1.41
Factor of safety against overturning	1.48	1.57
Maximum foundation pressure	7.5 ksf	11.3 ksf
Resultant outside of middle third by	1.4 ft.	1.9 ft.

13. Monoliths 8-13. The upstream section of the upper guide wall, which consists of Monoliths Nos. 8 through 13, has the same top width and approximately the same total height as the downstream section. The base width of these monoliths is 18 feet and the lower portion is 15 feet high consisting of 11 feet of rock

filled cribs and 4 feet of concrete filled cribs. In 1963 the upper guide wall was inspected by divers. No voids at the base of the monoliths were discovered, but some settlement of the rock fill in the crib was noticed. An analysis of the upper guide wall Monoliths 8 through 13 at normal operating condition, is shown on Plate C-4 and gives the following results:

	Operating Condition Monoliths 8 through 13
Factor of safety against sliding	1.90
Factor of safety against overturning	2.06
Maximum foundation pressure	4.1 ksf
Resultant outside of middle third by	0.4 ft.

LAND WALL

14. The 531.5 foot long land wall consists of 18 monoliths. Plan and sections of the land wall are shown on Plates 3 through 7 of the Main Report. All its monoliths, except Monolith No. 1, are approximately 60 feet high and have a base width of 32 feet. All monoliths have an 8 foot wide by 3 foot deep key beneath the landward side. The foundation underneath the land wall is classified as Unit A' sandstone, characterized by a minimum degree of cementation. Backfill behind the wall is composed mainly of sand, sandstone and boulders. According to seepage tests, described in Appendix B-3, Seepage, Uplift and Piping, the landward groundwater level varies from elevation 704± near the upstream end to elevation 700± near the downstream end of the wall. Normal lower pool elevation 687.2 was used for stability analyses at normal operating condition. Frictional resistance, derived from the weight of the floor slab in the landward lock, was included in the analyses.

15. Monoliths 1 and 2. Monolith No. 1 and upper miter gate Monolith No. 2 are contiguous with the mass concrete sill. Because of this structural contact, earth and hydrostatic pressures have no adverse effect on the stability of these two monoliths.

16. Monoliths 3 and 4. At present Monolith No. 3 contains filling valve and bulkhead slots. Monolith No. 4 has 2 different cross sections, being the transition between Monoliths Nos. 3 and typical land wall Monoliths Nos. 5 through 15, which were both analyzed for stability at present normal operating conditions.

There will be a short period during construction when the stability of Monoliths 3 and 4 will be affected because of the removal of concrete for lowering of filling conduit and valve. Later, when the present upper conduit will be backfilled with concrete, the stability of these monoliths will return approximately to present state. Results of stability analyses for these two monoliths, shown on Plate C-4, are as follows:

	Monolith No. 3		Monolith No. 4	
	Operating Condition	Construction Condition	Operating Condition	Construction Condition
Factor of safety against sliding	1.48	1.43	1.40	1.52
Factor of safety against overturning	1.67	1.97	1.62	1.79
Maximum foundation pressure, ksf	16.1	13.4	16.0	13.4
Resultant outside of middle third by, ft.	2.4	2.1	2.6	2.1

17. Monoliths 5-16. Besides investigation of structural stability at operating conditions, Monoliths Nos. 5 through 15 were also analyzed for stability during maintenance conditions, i.e. landward lock chamber being dewatered. Monolith No. 16, being more massive than other monoliths and thus having higher factor of stability, was not actually investigated. Stability analyses of Monoliths Nos. 5 through 15, shown on Plate C-4, give the following results:

	Operating Condition	Construction and Maintenance Condition
Factor of safety against sliding	1.44	1.35
Factor of safety against overturning	1.57	1.50
Maximum foundation pressure	17.7 ksf	18.8 ksf
Resultant outside of middle third by	3.2 ft.	3.2 ft.

18. Monolith 17. Lower gate Monolith No. 17 is subject to thrust from the horizontally framed lower miter gate, a type of gate where no part of the load is carried by the sill. Considering the direction of the thrust vectors, the monolith was analyzed as

a 3-dimensional unit. The most critical loading case, however, occurs at low water level in the land lock when the gates are left hanging. At these conditions the stability of the monolith, as shown on Plate C-4, is as follows:

	Operating Condition
Factor of safety against sliding	1.38
Factor of safety against overturning	1.41
Maximum foundation pressure	26.0 ksf
Resultant outside of middle third by	4.7 ft.

19. Monolith 18. This most downstream monolith of the land wall is approximately the same as Monoliths Nos. 5 through 16 and it is subject to similar forces as the other monoliths. Since this monolith is laterally supported by a concrete sill beneath the downstream miter gate, and thus by inspection the sliding stability should be less critical than for the other monoliths, separate analyses were not carried out for Monolith No. 18.

20. Stabilization of Land Wall Monoliths. Since the factors of safety against sliding for most of the land wall monoliths do not meet acceptable values, alternative solutions to improve the stability of the land wall monoliths are presented in Paragraphs 42 through 49.

LOWER GUIDE WALL

21. The 400-foot long lower guide wall consists of 13 monoliths built in two sections. The upper section, consisting of Monoliths Nos. 1 and 2, is founded directly on St. Peter Formation Sandstone, Unit A. The downstream section, consisting of Monoliths Nos. 3 through 13, is constructed on top of intrusion-grouted rock-filled timber cribs. A plan and sections of the lower guide wall are shown on Plate C-2 of this Appendix. Alongside the lower guide wall there is a 20-foot wide concrete slab with steel sheet piling cut-off.

22. Monoliths 1 and 2. These monoliths are 18 feet wide at the base and 35.5 feet high. There is an extended retaining wall and steps on top of Monolith No. 1 up to elevation 732.7. Only Monolith No. 1 was analyzed for stability, since Monolith No. 2 has less backfill and is therefore more stable than the first monolith. Both monoliths will be replaced by new discharge structures, which will be designed to satisfy required stability criteria. As shown on Plate C-4, the stability of Monolith No. 1 is as follows at operating conditions:

Factor of safety against sliding	1.19
Factor of safety against overturning	1.26
Maximum foundation pressure	33.6 ksf
Resultant outside of middle third by	4.0 ft.

23. Monoliths 3-13. These eleven monoliths have a base width of 20 feet and a total height of 34 feet. The lower 10 feet of the monoliths consist of concrete and rock filled timber cribs. In July 1950, the rock filled section of the cribs was pressure grouted and in December of the same year the backfill behind the wall monoliths was lowered to about 12 to 15 feet below the top of wall. A typical section of these monoliths was analyzed at normal operating and construction conditions as shown on Plate C-4. The following results were obtained:

	Operating Condition	Construction Condition
Factor of safety against sliding	4.08	2.84
Factor of safety against overturning	5.8	3.8
Maximum foundation pressure	5.4 ksf	7.0 ksf
Resultant inside of middle third by	0.6 ft.	0.1 ft.

INTERMEDIATE WALL

24. The 564-foot long intermediate wall has a top width of 35 feet and an average height of 57 feet (Monoliths Nos. 2 through 19). Between the lock chambers the base of the monoliths has width of 40 feet and there is an 8 foot wide by 3 foot deep key beneath the wall next to the landward lock. Steel sheet piling on either side of the wall minimizes seepage between the lock chambers. Plan and sections of the intermediate wall are shown on Plates 3 through 7 of the Main Report. For normal operating conditions, water surface elevations 725.2 and 687.2, interchangeable in either lock chamber, were used. For construction loading condition, one lock was assumed to be empty and the other lock filled to elevation 725.2.

25. Monoliths 1-3. Since Monoliths Nos. 1 through 3 are between mass concrete sills on either side, no stability analyses were deemed necessary for this upstream section of the intermediate wall. To improve performance of the hydraulic filling system, it is proposed that the intake portals should be relocated from

their present location in Monoliths Nos. 2 and 3 to the new intake monolith located immediately upstream of Monolith No. 1.

26. Monoliths 4-17. The proposed hydraulic modifications require lowering and relocation of filling valves, affecting Monoliths Nos. 4 and 5. Plan 2 of rehabilitation allows the use of riverward lock for navigation when landward lock is being improved. Thus there will be a short period when both present and future conduits exist in the two monoliths, causing temporary reduction in the mass and stability of Monoliths Nos. 4 and 5. Consequently, in addition to operating condition, also temporary construction conditions of Monoliths Nos. 4 and 5, as described above and shown on Plate C-5, were included in stability analyses with the following results:

	Operating Condition	Construction Condition	
	Monoliths 4-17	Monolith 4	Monolith 5
Factor of safety against sliding	1.80	1.61	1.54
Factor of safety against overturning	1.94	1.89	1.90
Maximum foundation pressure, ksf	10.9	9.7	11.3
Resultant inside of middle third, ft.	-	0.6	-
Resultant outside of middle third, ft.	0.6	-	0.9

27. Monolith 18. Lower miter gate Monolith No. 18 is subjected to thrusts from horizontally framed miter gates. Due to the magnitude and direction of these forces, three-dimensional stability analyses with miter gate thrust on one side were carried out for Monolith No. 18, as shown on Plate C-5, with the following results:

	Operating Condition
Factor of safety against sliding	1.04
Factor of safety against overturning	1.03
Maximum foundation pressure, ksf	74.5
Resultant outside of kern	
Across wall monoliths, ft.	12.4
Along wall monoliths, ft.	7.9

28. Monolith 19. This most downstream monolith of the intermediate wall is outside of lock chambers. Since there are no significant overturning forces acting on it, no stability analyses were prepared for Monolith No. 19. The proposed hydraulic modifications include also new discharge structures to be constructed as downstream extension beyond Monolith No. 19 of the intermediate wall. Since this extension will not be subject to significant unbalanced forces, no stability analyses were prepared for the extension of the intermediate wall.

29. Stabilization of Intermediate Wall Monoliths. All intermediate wall monoliths, except lower miter gate Monolith No. 18, have adequate factors of safety. Solutions to improve the stability of Monolith No. 18 are presented in Paragraphs 50 through 54 of this Appendix.

RIVER WALL

30. For about 80 percent of its total length the existing new river wall is backed up by the original river wall. Both the new and original river walls are founded on wood piles driven into underlying river alluvium. Steel sheet piling cut-off walls have been constructed beneath both river walls to minimize leakage to or from the riverward lock. Plan and section of the river wall are shown on Plates 3 through 6 of the Main Report.

31. Monoliths 1 and 2. The 24-foot wide river wall monoliths are backed up by the original river wall monoliths of equal height. The width of the old wall monoliths is 20 feet at the top and 24 feet at the base. According to the proposed modifications to improve hydraulics of the filling system, Monoliths Nos. 1 and 2 will be rebuilt to house intake portals in case Plan 4 of rehabilitation will be adopted. The two original upstream river wall monoliths, which will be subject to unbalanced water loads, were found to be adequate to resist the hydrostatic loads corresponding to upper pool elevation 732.7 during the construction period.

32. Monoliths 3-5. These river wall monoliths are contiguous with the mass concrete sill of the riverward lock. Because of this contact no stability analyses were prepared for river wall Monoliths Nos. 3 through 5.

33. Monoliths 6-16. This reach of the river wall, consisting of original and new wall sections, has a maximum height of 57 feet and a combined base width of 54 feet. Both walls are founded on timber piles spaced at 3-foot centers each way. There are approximately 16 piles supporting a 3-foot wide strip of the combined walls. As analyzed for normal operating conditions and shown on Plate C-5, maximum bearing on extreme riverside pile of new river wall is 50 kips, corresponding to a maximum equivalent area loading of 5.5 ksf. At construction or maintenance loading condition, with riverward lock unwatered, maximum bearing on extreme lockside pile is 98 kips, corresponding to a maximum equivalent area loading of 9.6 ksf. At normal loading condition, average horizontal load per pile, caused by unbalanced hydrostatic loads, is 13 kips. At construction or maintenance condition, the average horizontal load per pile is 2 kips.

34. Monoliths 17-20. These river wall monoliths, which are not backed up by the original river wall monoliths, are 57 feet high, 24 feet wide at the top and have a bottom width of 32 feet. There is a 9 foot wide by 4 foot deep key beneath these monoliths along the riverward lock. The monoliths are founded on wood piles at 3-foot centers each way. Steel sheet pile cut-off wall underneath either side of the monoliths reduce seepage to and from the lock chamber and prevent loss of material due to piping underneath the wall. Backfill on the river side of the wall varies from elevation 706 behind Monolith No. 17 to elevation 681.2 (top of concrete slab) behind Monolith No. 20. Monoliths Nos. 18 through 20 were investigated for 2 different uplift conditions. "Lockside maximum uplift" was assumed to vary linearly from elevation 725.2 at the lock side to elevation 687.2 at the river side. "Lockside average uplift" was assumed as a linear pressure variation from elevation 706.2 at the lock side to lower pool elevation 687.2 at the river side. Because of thrusts from the lower miter gate, Monolith No. 20 was investigated using three-dimensional analyses. For Monoliths Nos. 19 and 20, the following results at normal operating conditions were obtained from the stability analyses shown on Plate C-5.

	Monolith 19		Monolith 20	
	Lockside Average Uplift	Lockside Maximum Uplift	Lockside Average Uplift	Lockside Maximum Uplift
Factor of safety against sliding	1.57	1.41	0.95	0.90
Factor of safety against overturning	1.84	1.55	-	-
Maximum foundation pressure, ksf	11.9	12.2	53.5	64.2
Resultant outside middle third, ft.	0.5	1.8	-	-
Resultant from center				
Across wall, ft.	-	-	8.6	9.3
Along wall, ft.	-	-	7.2	8.0
Bearing load per pile				
Maximum, kips	130	131	290	294
Minimum, kips	-10	-23	-151	-162
Horizontal load per pile, kips	21	21	40	48

35. Monolith 21. Since there are no significant overturning forces acting on Monolith No. 21, no stability analyses were prepared for it. If Plan 4 of rehabilitation will be adopted, then the emptying conduit terminating presently at Monolith No. 21 will be extended towards the river or in the downstream direction and be provided with energy dissipation structure. None of the possible alternative structural extensions will affect the stability of the monolith.

36. Stabilization of River Wall Monoliths. Since the factors of safety for river wall Monolith No. 20 are unsatisfactory, solutions for improvement are presented in Paragraphs 55 through 60.

OVERALL LOCK STRUCTURE

37. Close examination of all possible modes of failure reveals that sliding of the structure at the river wall along a soft plastic seam in the upper layer of siltstone (Unit C) would result in the least stable conditions. According to geological exploration program, the soft seam was found to be continuous under the structure approximately at elevation 650.5. Since the top of rock (Unit B) varies underneath the river wall, two alternative elevations, 655 and 660, were used in the analyses. The following results, as shown on Plate C-5, were obtained for overall sliding stability of the structure:

	Factor of Safety Against Sliding	
	Top of Rock Elevation at	
	655.0	660.0
Shear strength parameter $\phi = 23°$	1.70	1.75
Shear strength parameter $\phi = 22°$	1.68	1.72
Shear strength parameter $\phi = 18°$	1.36	1.58

Angle of internal friction $\phi = 23°$ is representative for the area of the structure. Two other values $\phi = 22°$ and $\phi = 18°$ were obtained in tests. The results involving these two values are shown for comparison purposes only.

BUTTRESS DAM

38. Overflow Section. The hollow Ambursen type concrete dam consists of 36 monoliths and has a total length of 574 feet. Its crest elevation is at 723.2 and it is topped by 2-foot high flashboards which are lowered during high floods and during the winter months. The dam is founded on river alluvium composed of sand, gravel, and limestone slabs. Buttresses 12 through 19 of the center portion of the dam are supported by timber piles. Beneath the upstream side of the dam there is a 6-foot deep concrete wall on top of steel sheet piling of unknown depth to provide a partial seepage cut-off. In 1952 the cavity of the dam was partially filled with sand to increase its weight and therefore increase the resistance to sliding. As measured in 1975 the average depth of sand in individual bays ranged from 7.7 to 9.1 feet with an average sand depth of 8.34 feet. Sections of the dam are shown on Plate C-3.

39. Apron. The downstream apron of the dam consists of two 40-foot slabs. In the west half of the apron, there is a row of inspection holes in each slab which serve also as relief holes for uplift pressures beneath each slab. The inspection holes were eliminated in the east half of the slab when it was resurfaced in 1953. During the same year, a baffle wall was added on the slab next to the dam in order to dissipate energy and cause hydraulic jump to form nearer to the dam.

40. Loadings and Design Criteria. The stability of the dam was analyzed applying generally the criteria as given in Engineering Manual EM 1110-2-2200, Gravity Dam Design. The following loading conditions were examined:

1. Normal operating condition
2. Flood discharge condition
   - 1965 flood
   - 1951 flood
3. Earthquake condition.

In all of the above cases, soil bearing pressure was checked at elevation 690.6 and sliding was considered to be most critical along an inclined plane starting from elevation 684.6 at the upstream face of dam and sloping upward at an angle of approximately 6° towards its intersection with extended downstream face of dam at elevation 690.6. The factor of safety against sliding along this inclined plane was estimated at a friction factor of 0.649 At flood conditions tailwater elevation of dam was determined applying hydraulic jump formulae and uplift pressures beneath the dam were obtained using flow net analyses.

41. Results of Analyses. The results obtained for the four loading conditions are listed in the following tabulation:

		Loading Condition		
	Normal Operating	1965 Flood	1951 Flood	Earthquake
Bearing pressure				
Maximum, ksf	2.5	1.4	1.5	2.3
Minimum, ksf	1.0	1.1	1.3	1.3
Resultant within middle third by ft.	5.8	9.2	9.4	7.3
Factor of sliding	0.28	0.21	0.44	0.32
Factor of safety against sliding	2.33	2.98	1.48	2.05

The above results indicate that bearing pressure is acceptable at all loading conditions investigated. The structure is safe against overturning since the resultant of forces remains within the middle third of base at all cases studied. At normal operating, 1965 flood, and earthquake loading conditions, the factor of safety against sliding remains satisfactory. At 1951 or lesser flood conditions, however, a hydraulic jump would form between the downstream face of the dam and the baffle wall. The lowering of tailwater depth will decrease the balancing horizontal water load acting in the upstream direction and also the amount of water filled in the cavity of dam, thus reducing resistance against sliding. Because of limited amount of data to determine with certainty at which flow conditions a hydraulic jump would form downstream of the dam, 1951 flood was used with a hydraulic jump for Phase A studies. More refined analyses, based on additional information, will be carried out in Phase B. The two flood loading conditions investigated are shown on Plate C-6.

STABILIZING LAND WALL

ALTERNATIVES STUDIED

42. Inclined Soil (Rock) Anchors. To increase the factors of safety against sliding and overturning along the wall, one solution would be to install inclined, pressure grouted rock anchors. In a typical installation procedure, recoverable casing would be driven using an expendable drive point. If it will be too difficult to drive the recoverable casing with the expendable point into the sandstone, rotary percussion or auger bits could be used to advance the holes. After the required depth is reached, anchor bars will be installed. As the grout is pumped into the hole, the casing is gradually withdrawn and high pressure grout zone will form at the end of the anchors. Using stressing anchorage and bearing plates at the lock face of the wall, the anchors will be prestressed to 125-150 percent of the design load, the total stress remaining within the elastic limit of the anchor material, and then set at the design load. For Monoliths Nos. 5 through 16, 3 different anchor sizes were assumed to stabilize the wall monoliths against overturning and sliding. The largest rock anchors used, 3-bar tendons of 1-1/4"Ø, are spaced at 15 feet centers. The maximum pretressing force of 356 kips per 3-bar anchor (450 kips initial force minus 94 kips prestressing loss), or equal to 23.7 kips per linear foot of wall, is equal to 79 percent of soil pressure of the backfill behind the land wall. The total lengths of the anchors vary from 90 to 100 feet. The analyses, shown on Plate C-4, give the following results at normal operating conditions:

Land Wall Monoliths 5-16

15ft	1-3/8" Ø Single Bar Tendon Per Anchor Spaced at 10 ft	2- 1-1/4" Ø Bar Tendons per Anchor Spaced at 15ft	3- 1-1/4" Ø Bar Tendons per Anchor Spaced at 15ft
Factor of safety against sliding	1.76	1.89	2.16
Factor of safety against overturning	2.37	2.45	3.27
Maximum foundation pressure, ksf	12.1	12.0	9.8
Resultant inside middle third, ft.	0.5	1.0	2.8

For lower miter gate Monolith No. 17, only stabilization with 1-3/8" Ø single anchor bars was investigated with the following results:

Land Wall Monolith 17

	1-3/8" Ø Single Bar Tendon Per Anchor Spaced at 10 ft
Factor of safety against sliding	1.67
Factor of safety against over turning	1.71
Maximum foundation pressure	18.7 ksf
Resultant outside middle third	1.0 ft.

43. Lowering Height of Backfill. Another way to stabilize the land wall is by lowering the backfill to about 10 feet below the top of the wall. Before lowering the backfill, the footing of the existing bluff protection crib wall should be lowered. This process would require temporary dismantling of the crib wall, lowering the foundation and rebuilding of the 46-foot long upstream section of the bluff protection crib wall. The computations for stability with backfill lowered, shown on Plate C-4, give the following results:

Backfill Lowered by 10 Feet Behind Land Wall	Monolith 3*	Monolith 4	Monoliths 5-16	Monolith 17
Factor of safety against sliding	1.85	1.86	1.74	1.79
Factor of safety against overturning	2.05	2.06	1.87	1.81
Maximum foundation pressure, ksf	12.6	11.6	12.3	16.4
Resultant outside middle third, ft.	0.3	0.1	1.3	1.9

44. Other Alternatives. Two other alternatives to stabilize the land wall were considered. These are:

a) Use of basin floor as a strut;
b) Replacing backfill with mass concrete.

The use of basin floor slab as a strut to stabilize the walls would not improve conditions as far as factor of safety for overturning or soil pressure values are concerned. The possibility to place mass concrete behind the wall by replacing backfill does not result in a practical solution structurally or costwise, since there is not much difference in weight between sand backfill and concrete.

COMPARATIVE RESULTS OF ALTERNATIVES

45. Factor of Safety Against Sliding. Installation of rock anchors improves factor of safety against sliding from 20 to 50 percent depending on size and number of anchors installed. Lowering of backfill by 10 feet will increase factor of safety against sliding by about 20 to 30 percent. The following results are summarized for comparison:

* 8 feet behind land wall Monolith No. 3

	Factor of Safety Against Sliding	
	Monoliths 5-16	Monolith 17
Existing conditions	1.44	1.38
Inclined rock anchors installed		
1- 1-3/8" Ø @ 10 ft	1.76	1.67
2- 1-1/4" 0 @ 15 ft	1.89	-
3- 1-1/4" Ø @ 15 ft	2.16	-
Backfill lowered by 10 ft	1.74	1.79

46. Foundation Pressures. The high foundation pressures prevailing beneath the land wall monoliths at present conditions can be reduced considerably by using various methods of stabilization. The maximum foundation pressures, and also minimum values as shown in parentheses, are listed below for conditions both before and after stabilization.

	Maximum (and Minimum) Foundation Pressures in ksf	
	Monoliths 5-16	Monolith 17
Existing conditions	17.7 (0.0)	26.0 (0.0)
Inclined rock anchors installed		
1- 1-3/8" Ø @ ft	12.1 (0.6)	18.7 (0.0)
2- 1-1/4" Ø @ 15 ft	12.0 (1.3)	-
3- 1-1/4" Ø @ 15 ft	9.8 (3.6)	-
Backfill lowered by 10 ft	12.3 (0.0)	16.4 (0.0)

47. Relative Costs. Stabilization on land wall Monoliths Nos. 3 through 18 would require 48 - 1-3/8 inch diameter anchors using 3 anchors per monolith at a spacing of 10 feet. The cost of these anchors is estimated to be approximately $110,000. Using 2- 1-1/4 inch or 3- 1-1/4 inch diameter anchors at a spacing of 15 feet (two anchors per monolith), the cost will be approximately $95,000, respectively $128,000. To the cost of anchors approximately $7000 must be added as incremental cost for excavation and backfill of inclined construction access starting from elevation 732.7 instead of elevation 723 behind the downstream

end of the land wall. The cost of lowering backfill by 10 feet behind the land wall will be approximately $27,000. Dismantling and reconstruction of the upstream 46-foot section of the bluff protection crib wall will cost approximately $30,000. Total cost of lowering of backfill and reconstruction of crib wall will be $57,000. The cost of removal of existing trees and structures on the land wall, planting of new trees, surfacing of access road and parking area, construction of central control station, elevator, etc., will have practically no influence on the relative costs of the two alternatives of land wall stabilization.

48. Advantages and Disadvantages of Lowering Backfill

Advantages

a) Excavated material can be used as random fill for access ramps to cofferdam areas.
b) Part of area must be lowered for construction of access road to cofferdam area.
c) Best solution if considering only the stability of the land wall.
d) Appears to be the more economical solution.
e) Possible operational or safety benefits.

Disadvantages

a) Decreases available work area. Reduction of useable area may result in increased costs.
b) Increases existing bluff erosion problems. If existing crib wall must be extended in the upstream direction to control bluff erosion, lowered backfill will increase the cost of crib wall. Also the extension of crib wall would probably result in the loss of useable space and working area.
c) Could create channel between land wall and bluff during high flows.
d) Will likely create operational, maintenance and safety problems.

49. Advantages and Disadvantages of Soil Anchors

Advantages

a) Personnel in esplanade area can see approaching tows, etc.
b) Shorter climb from esplanade to city streets.
c) Safer during floods.
d) If major revisions to venting system are not needed, existing trees could remain.
e) Possible operational or safety benefits.

Disadvantages

a) Higher cost (at least initially).
b) Slightly lower degree of confidence in final results of structural stability.
c) Anchors will eventually have to be replaced, say after 25 to 50 years. Probable cost approximately $100,000 at January 1975 price level.
d) Installation of anchors requires timing and scheduling considerations. Holes for the inclined anchors will be drilled with equipment placed on the top of intermediate wall. Because of possible interference with other construction activities and because of limited clearance it is preferable to install anchors when winter shelter is not in place.
e) More detailed plans and design of anchors may result in increased cost.

STABILIZING INTERMEDIATE WALL

ALTERNATIVES STUDIED

50. Soil (Rock) Anchors. Gate Monolith No. 18 is the only one in the intermediate wall that requires stabilization to improve the values of factor of safety against sliding, overturning and soil pressure. The first possibility investigated was the installation of vertical soil (rock) anchors. Within limited area at the top of the monolith when applying minimum spacing requirements between anchors (about 4.5 feet), it is possible to install 12

anchors consisting of 3 -1-1/4 inch diameter bars each. Installation procedure would be similar to the one described for stabilizing the land wall. A prestressing force up to 80 percent of yield strength with 25 ksi loss of prestress would result in usable anchor capacity of 350 kips. The resulting stability factors are shown on Plate C-5 and in Paragraph 52.

51. Shear Keys. Two alternatives were studied to stabilize Monolith No. 18 with shear keys:

a) Interconnect the monolith with downstream Monolith No. 19;

b) Interconnect it with Monoliths Nos. 19 and 17.

First solution does not yield factors of safety within acceptable limits. The second one, however, gives satisfactory results. In this solution, 2 vertical shear keys and four horizontal ones at each vertical contraction joint between Monoliths Nos. 17, 18 and 19 will be used. A vertical shear key is obtained by drilling a 2-foot diameter hole, from the top of the monolith, to a level approximately 2 feet above the crown of the emptying culvert, and thereafter filling it with 4000 psi concrete. Spiral and vertical reinforcement will be used to increase the strength of the shear key. A horizontal shear key would be 2 feet in diameter and 6 feet long. The horizontal shear keys will be located at elevations 690 and 720 on both faces of the intermediate wall. The results of both alternatives are summarized on Plate C-5 and in Paragraphs 52 through 54.

COMPARATIVE RESULTS OF ALTERNATIVES

52. Factor of Safety Against Sliding. The factor for existing conditions is close to unity. It is improved to 1.7 both for alternative with soil anchors and for alternative with shear keys between Monoliths Nos. 17, 18 and 19. The summary of alternatives is as follows:

	Factor of Safety Against Sliding
Existing conditions	1.04
Soil (rock) anchors provided	1.77
Shear keys between Monoliths Nos. 17, 18 and 19	1.68

53. Foundation Pressures. Very high maximum foundation pressure of 75 ksf under existing conditions can be reduced to about 48 ksf with the installation of soil anchors. This pressure is still too high to be acceptable. The soil anchors increase the vertical load but reduce the eccentricity and produce higher average pressures on the foundation. Satisfactory results for foundation pressures are achieved only using shear keys for interconnection of monoliths. The summarized results are as shown:

	Maximum and Minimum Foundation Pressure ksf		Maximum Vertical Deflection - inches
Existing conditions	74.5	(0.0)	2.0
Soil (rock) anchors provided	47.5	(0.0)	1.1
Shear keys between Monoliths Nos. 17, 18 and 19	13.8	(0.0)	0.2

54. Conclusions and Costs. Analyzing and comparing results in Paragraphs 50 and 51 indicates that the use of shear keys between Monoliths Nos. 17, 18 and 19 is the only solution for stabilization of intermediate wall gate Monolith No. 18 which meets the design criteria. The cost of the stabilization with shear keys is estimated to be $35,000.

STABILIZING RIVER WALL

ALTERNATIVES STUDIED

55. Soil (Rock) Anchors. Drawing conclusions from the investigation of intermediate wall, it was concluded that the stabilizing of the river wall by using soil (rock) anchors was also not feasible.

56. Shear Keys. To increase factor of safety against sliding and reduce pressures for gate Monolith No. 20, the use of shear keys is recommended. After considering several other alternatives, shear keys between Monoliths Nos. 19, 20 and 21 seem to be an effective way to stabilize the gate monolith. The size (2.0 ft diameter), the length and installation procedure is the same as for the stabilization of the intermediate wall. Summaries of design calculations are shown on Plate C-5 and in Paragraphs 58 and 59.

57. Backfill Behind Monoliths. The downstream Monoliths Nos. 17, 18 and 19 require some measures to increase the factor of safety against sliding. Backfilling up to elevation 710, or even up to elevation 715 would be a simple and inexpensive way to do it. Additional advantage of backfilling is increased protection against piping failures. River wall gate Monolith No. 20 could not be stabilized with acceptable factors of safety by backfilling alone. In this case, backfilling combined with shear keys at the adjoining monoliths offers the solution. The following section shows summarized results of the analysis arriving at the above conclusion.

COMPARATIVE RESULTS OF ALTERNATIVES

58. Sliding. The factor of safety against sliding is under unity for the gate monolith under existing conditions and about 1.5 for Monolith No. 19. Backfilling to elevation 710 satisfies the requirements for Monolith No. 19 and shear keys together with backfilling provide sufficient stability for gate Monolith No. 20. The summarized results are shown below:

	Factor of Safety Against Sliding	
	Monolith No. 19	Gate Monolith No. 20
Existing Conditions:		
Average uplift pressure at lockside	1.57	0.95
Maximum uplift pressure at lockside	1.41	0.90
Shear Keys:		
Average uplift pressure at lockside	-	1.42
Maximum uplift pressure at lockside	-	1.39
Backfilling to elevation 710 behind monoliths:		
Average uplift pressure at lockside	2.04	1.20
Maximum uplift pressure at lockside	1.83	1.15

	Factor of Safety Against Sliding	
	Monolith No. 19	Gate Monolith No. 20
Backfilling and shear keys:		
Average uplift pressure at lockside	-	1.74
Maximum uplift pressure at lockside	-	1.70

59. Foundation Pressures. Although most existing conditions were analyzed both for pile loading and foundation area pressures, the latter seems to be more realistic to use for final conclusions. Pile bearing capacity was checked using both point bearing and skin friction values. The resulting pile capacity of 8 to 12 kips supports the area loading approach. The results of the computations, shown on Plate C-5, are as follows for the various alternatives:

	Maximum Foundation Pressures	
	Monolith No. 19	Gate Monolith No. 20
Existing Conditions:		
Average uplift pressure at lockside	11.9 ksf	53.5 ksf
Maximum uplift pressure at lockside	12.2 ksf	64.2 ksf
Shear Keys:		
Average uplift pressures at lockside	-	14.2 ksf
Maximum uplift pressures at lockside	-	14.4 ksf
Backfilling to elevation 710 behind monoliths:		
Average uplift pressure at lockside	10.6 ksf	45.5 ksf
Maximum uplift pressure at lockside	10.7 ksf	52.4 ksf
Backfilling and shear keys:		
Average uplift pressure at lockside	-	13.7 ksf
Maximum uplift pressure at lockside	-	13.8 ksf

60. <u>Cost of Improvements</u>. The cost of backfilling will be about $90,000, and the cost of stabilization using shear keys will be the same as for the intermediate wall or about $35,000. Thus the total cost of stabilizing the downstream river wall monoliths will be approximately $125,000.

STABILIZING BUTTRESS DAM

61. Of the various loading conditions investigated, as described in Paragraph 40 above, a too low factor of safety of 1.49 against sliding was obtained at 1951 flood condition. To increase the factor of safety to 1.5 it was found that the fill in the dam should be raised to elevation 701.25$\pm$. The following results were obtained after the raising of fill in the dam:

Foundation pressure (max)	1.5 ksf
Resultant within middle third by	9.7 ft.
Factor of sliding	0.433
Factor of safety against sliding	1.50

The analysis of stabilizing the buttress dam is presented graphically on Plate C-6. To facilitate the placing of fill through the limited access openings it is recommended that a mixture of sand and water be pumped by means of a pipeline starting on the draft tube deck of the Ford hydro plant. A small amount of fill (200 cubic yards) will be needed at a cost of approximately $10,000.

IMPROVEMENT OF GUIDE WALLS

UPPER GUIDE WALL

62. Although present stability factors are within acceptable limits, pressure grouting of the cribs is recommended to avoid any excessive movements due to future settlement in cribs. The cost of drilling and pressure grouting is estimated to be approximately $45,000.

LOWER GUIDE WALL

63. Monoliths of the lower guide wall meet the design criteria for sliding and overturning, except Monoliths Nos. 1 and 2.

Since a new hydraulic emptying system is proposed for all alternative plans of rehabilitation, the first 2 or 3 monoliths, depending on the plan used, will be replaced by a new discharge structure designed to satisfy the required criteria of stability. The cost of the replacement structures is included in Appendix A.

Table C-1

STUDY OF ALTERNATIVES FOR REHABILITATION OF LOCK AND DAM NO. 1

SUMMARY OF STRUCTURAL INVESTIGATIONS

	Factor of Safety		Maximum Foundation Pressure	Resultant Outside Middle Third	Resultant Inside Middle Third
	Sliding	Overturning	ksf	feet	feet
UPPER GUIDE WALL					
Monoliths 1-7					
Operating condition	1.80	1.48	7.5	1.4	-
Monoliths 1-5					
Const. condition	1.41	1.57	11.3	1.9	-
Monoliths 8-13					
Operating condition	1.90	2.06	4.1	0.4	-
LAND WALL					
Monoliths 1-2[1/]	-	-	-	-	-
Monolith 3					
Operating condition	1.48	1.67	16.1	2.4	-
Const. condition	1.43	1.97	13.4	2.1	-
Monolith 4					
Operating condition	1.40	1.62	16.0	2.6	-
Const. condition	1.52	1.79	13.4	2.1	-
Monoliths 5-16					
Operating condition	1.44	1.57	17.7	3.2	-
Const. condition	1.35	1.50	18.8	3.2	-
Operating condition after improvement					
3- 1-3/8" Ø single anchor bars per monolith	1.76	-	12.1	-	0.5
2- 1-1/4" Ø double anchor bars per monolith	1.89	-	12.0	-	1.0
2- 1-1/4" Ø triple anchor bars per monolith	2.16	-	9.8	-	2.8
Backfill lowered by 10 feet	1.74	1.87	12.3	1.3	-
Monolith 17					
Operating condition	1.38	1.41	26.0	4.7	-
Operating condition after improvement					
3- 1-3/8" Ø single anchor bars per monolith	1.67	1.71	18.7	1.0	-
Backfill lowered by 10 feet	1.79	1.81	16.4	1.9	-
Monolith 18[2/]					

Table C-1 (Continued)

STUDY OF ALTERNATIVES FOR REHABILITATION OF LOCK AND DAM NO. 1

SUMMARY OF STRUCTURAL INVESTIGATIONS

	Factor of Safety		Maximum Foundation Pressure	Resultant Outside Middle Third	Resultant Inside Middle Third
	Sliding	Overturning	ksf	feet	feet
LOWER GUIDE WALL					
Monolith 1[3/]					
Operating condition	1.35	1.26	33.6	4.0	-
Monolith 2[3/]	-	-	-	-	-
Monoliths 3-13					
Operating condition	4.07	5.8	5.4	-	0.6
Const. condition	2.84	3.8	7.0	-	0.1
INTERMEDIATE WALL					
Monoliths 1-3[1/]	-	-	-	-	-
Monoliths 4-17					
Operating condition	1.8	1.94	10.9	0.6	-
Monolith 4					
Const. condition	1.61	1.89	9.7	-	0.6
Monolith 5					
Const. condition	1.54	1.90	11.3	0.9	-
Monolith 18					
Operating condition	1.04	1.03	74.5	-	-
Improved operating conditions:					
Vert. anchors	1.77	-	47.5	-	-
Connected with shear keys to Mono. 17 and 19	1.66	-	13.8	-	-
Monolith 19[4/]	-	-	-	-	-
RIVER WALL					
Monoliths 1-2[5/]	-	-	-	-	-
Monoliths 3-5[1/]	-	-	-	-	-
Monoliths 6-16[6/]	-	-	-	-	-
Monolith 17[7/]	-	-	-	-	-
Monolith 18[8/]	-	-	-	-	-
Monolith 19					
Operating condition					
Lock.avg. uplift	1.57	1.84	11.9	0.5	-
Lock.max. uplift	1.41	1.55	12.2	1.8	-

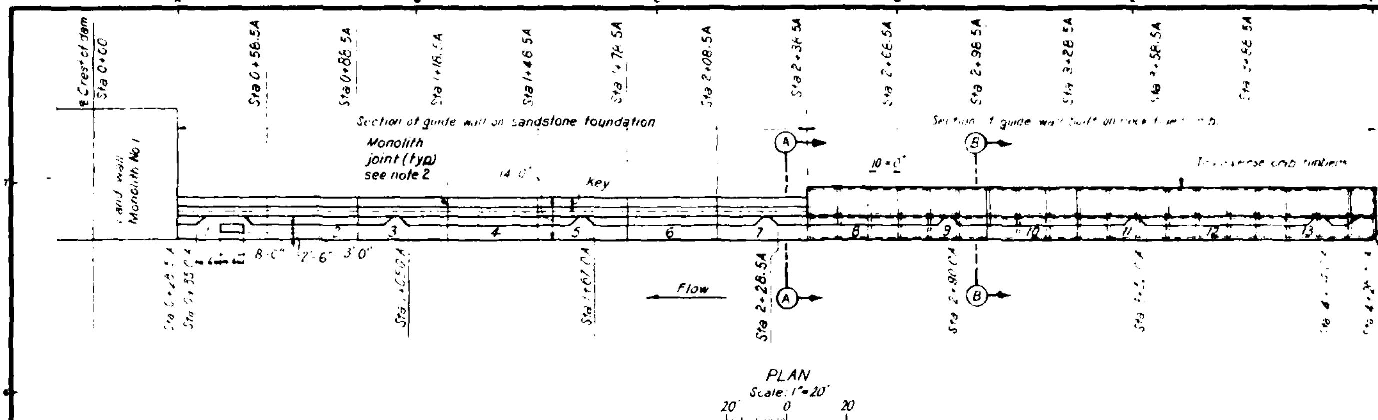
Table C-1 (Continued)

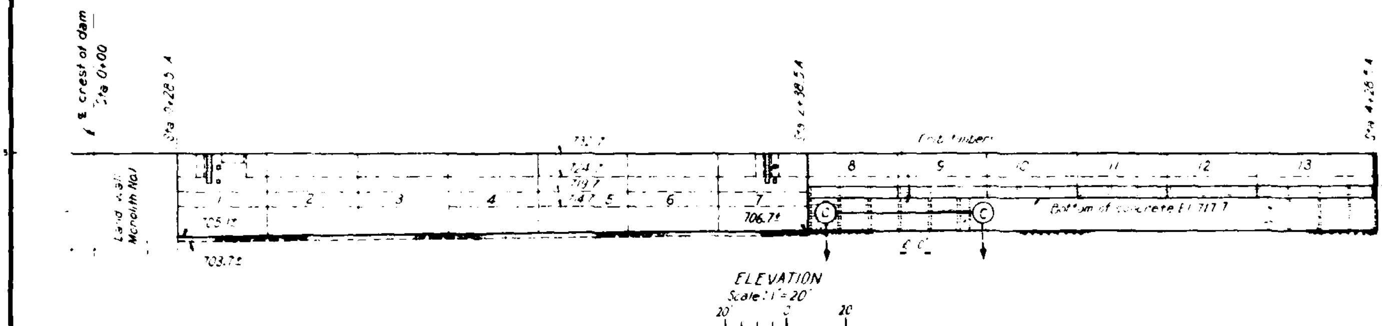
STUDY OF ALTERNATIVES FOR REHABILITATION OF LOCK AND DAM NO. 1

SUMMARY OF STRUCTURAL INVESTIGATIONS

	Factor of Safety		Maximum Foundation Pressure	Resultant Outside Middle Third	Resultant Inside Middle Third
	Sliding	Overturning	ksf	feet	feet
RIVER WALL (Continued)					
Operating condition after improvement Backfill to El. 710					
Lock.avg. uplift	2.04	1.93	10.6	-	0.6
Lock.max. uplift	1.83	1.63	10.7	0.5	-
Monolith 20					
Operating condition					
Lock.avg. uplift	0.95	-	53.5	-	-
Lock.max. uplift	0.90	-	64.2	-	-
Operating condition after improvement Connected with shear keys to Mono. 19 and 21					
Lock.avg. uplift	1.42	-	14.2	-	-
Lock.max. uplift	1.39	-	14.4	-	-
Backfill to El.710					
Lock.avg. uplift	1.20	-	45.5	-	-
Lock.max. uplift	1.15	-	52.4	-	-
Backfill to El.710 and connected with shear keys to Mono. 19 and 21					
Lock.avg. uplift	1.74	-	13.7	-	-
Lock.max. uplift	1.70	-	13.8	-	-
Monolith 21[9]	-	-	-	-	-
BUTTRESS DAM					
Normal operating condition	2.33	-	2.5	-	5.8
1965 flood condition	2.98	-	1.4	-	9.2
1951 flood condition	1.48	-	1.5	-	9.4
Improved 1951 flood condition					
Sand fill raised to El. 701.25	2.03	-	1.7	-	9.7
Earthquake condition	2.05	-	2.3	-	7.3

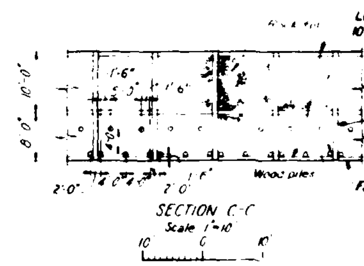
1/ Restrained by mass concrete sill.
2/ Similar to Monoliths 5-16.
3/ Will be replaced by new outlet monolith.
4/ Not subject to unbalanced lateral loads.
5/ Will be rebuilt to house new intake.
6/ Double monoliths consisting of old and new river walls.
7/ Partially double monolith.
8/ Similar to Monolith 19.
9/ Subject to insignificant unbalanced lateral loads.

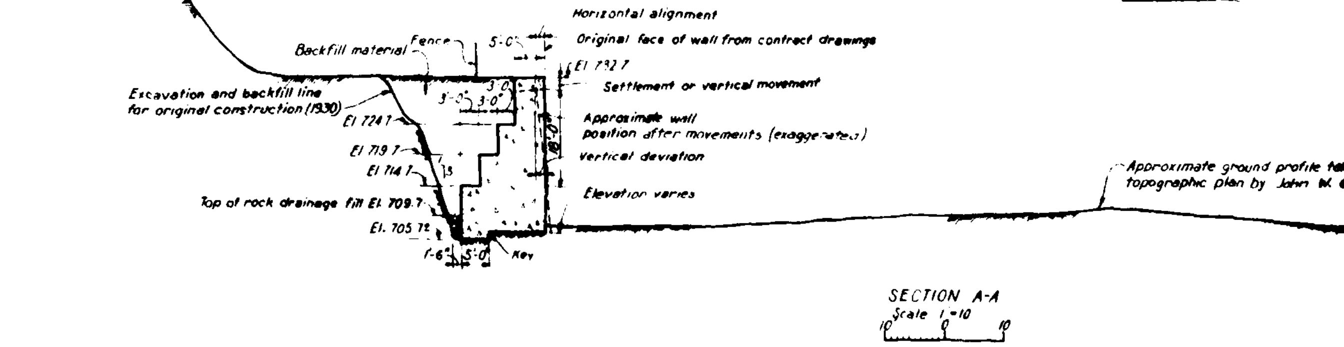


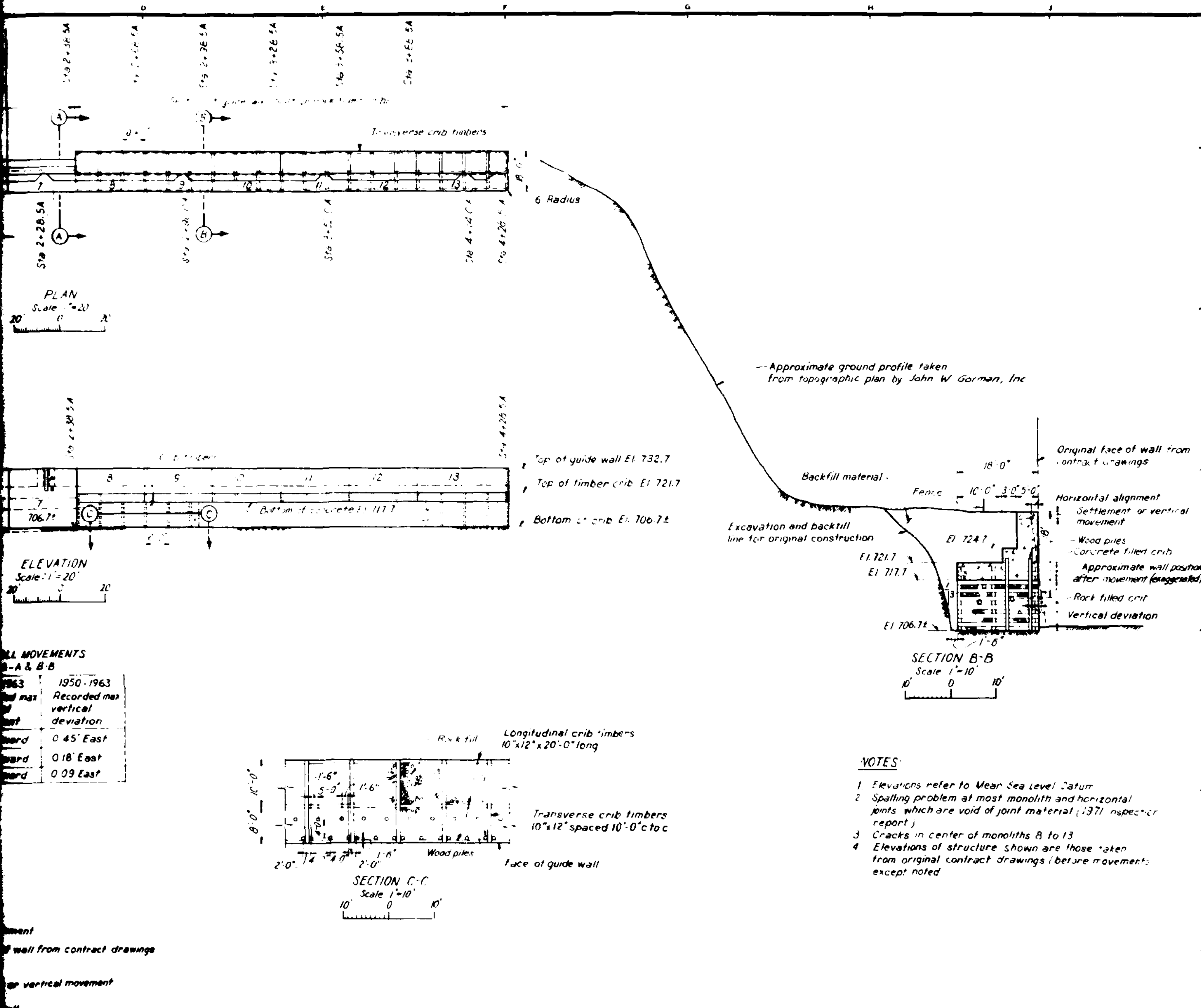


TABULATION OF WALL MOVEMENTS
See sections A-A & B-B

Station	1934-1963 Recorded max horizontal movement	1938-1963 Recorded max vertical movement	1950-1963 Recorded max vertical deviation
4+23.6A	0.14' West	0.02' upward	0.45' East
+89.0A	0.12' West	0.0[illegible] upward	0.18' East
0+71.0A	0.12' West	0.02 upward	0.09 East







NOTES

1. Elevations refer to Mean Sea Level Datum
2. Spalling problem at most monolith and horizontal joints which are void of joint material (1971 inspection report)
3. Cracks in center of monoliths 8 to 13
4. Elevations of structure shown are those taken from original contract drawings (before movement) except noted

HARZA ENGINEERING COMPANY CHICAGO, ILLINOIS	DEPARTMENT OF THE ARMY ST PAUL DISTRICT CORPS OF ENGINEERS ST PAUL MINNESOTA

DRAWN BY J B C
CHECKED BY R N M
SUBMITTED BY
APPROVED

STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER — LOCK & DAM NO 1

EXISTING STRUCTURES

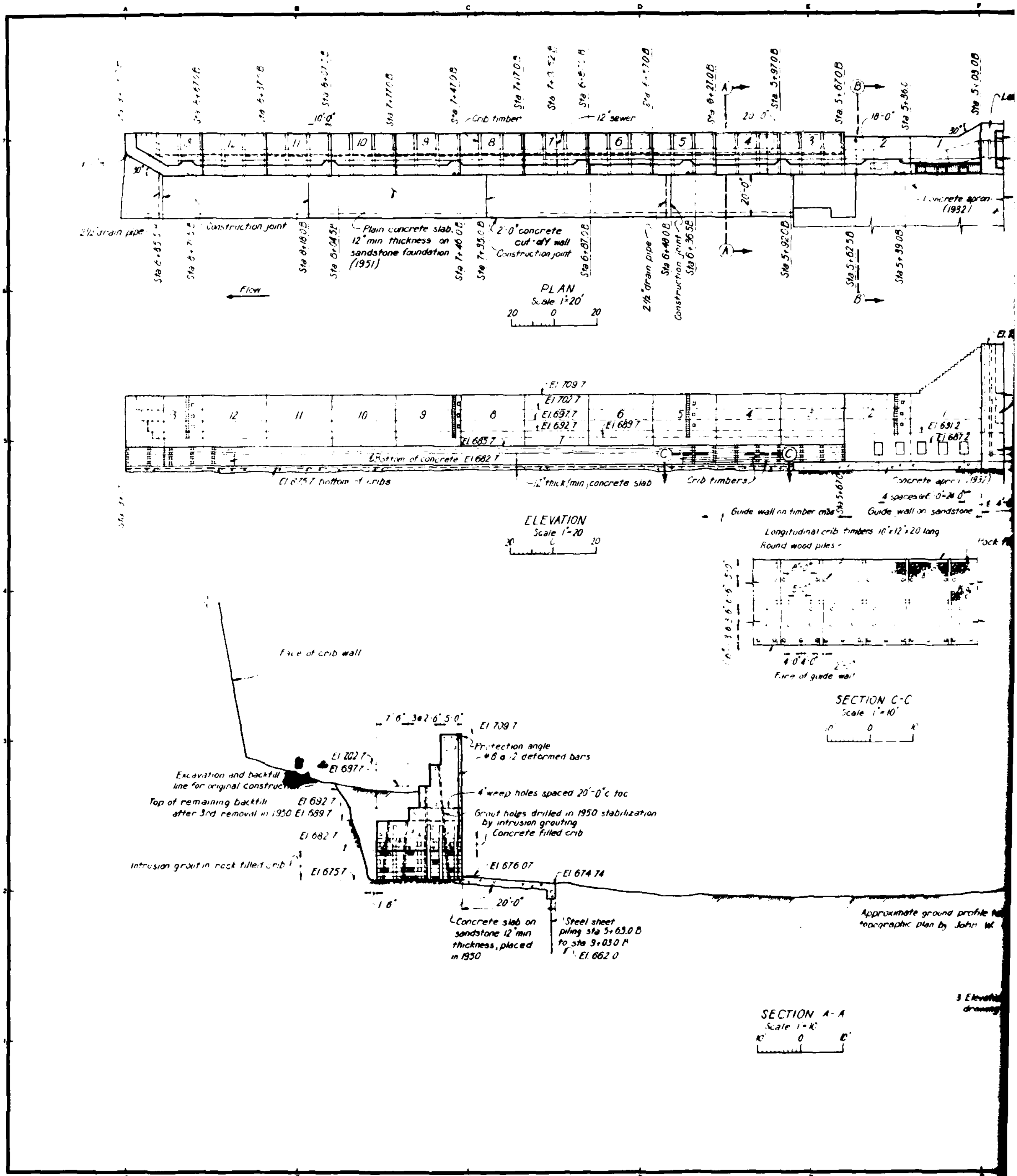
UPPER GUIDE WALL

DATE MARCH 1975

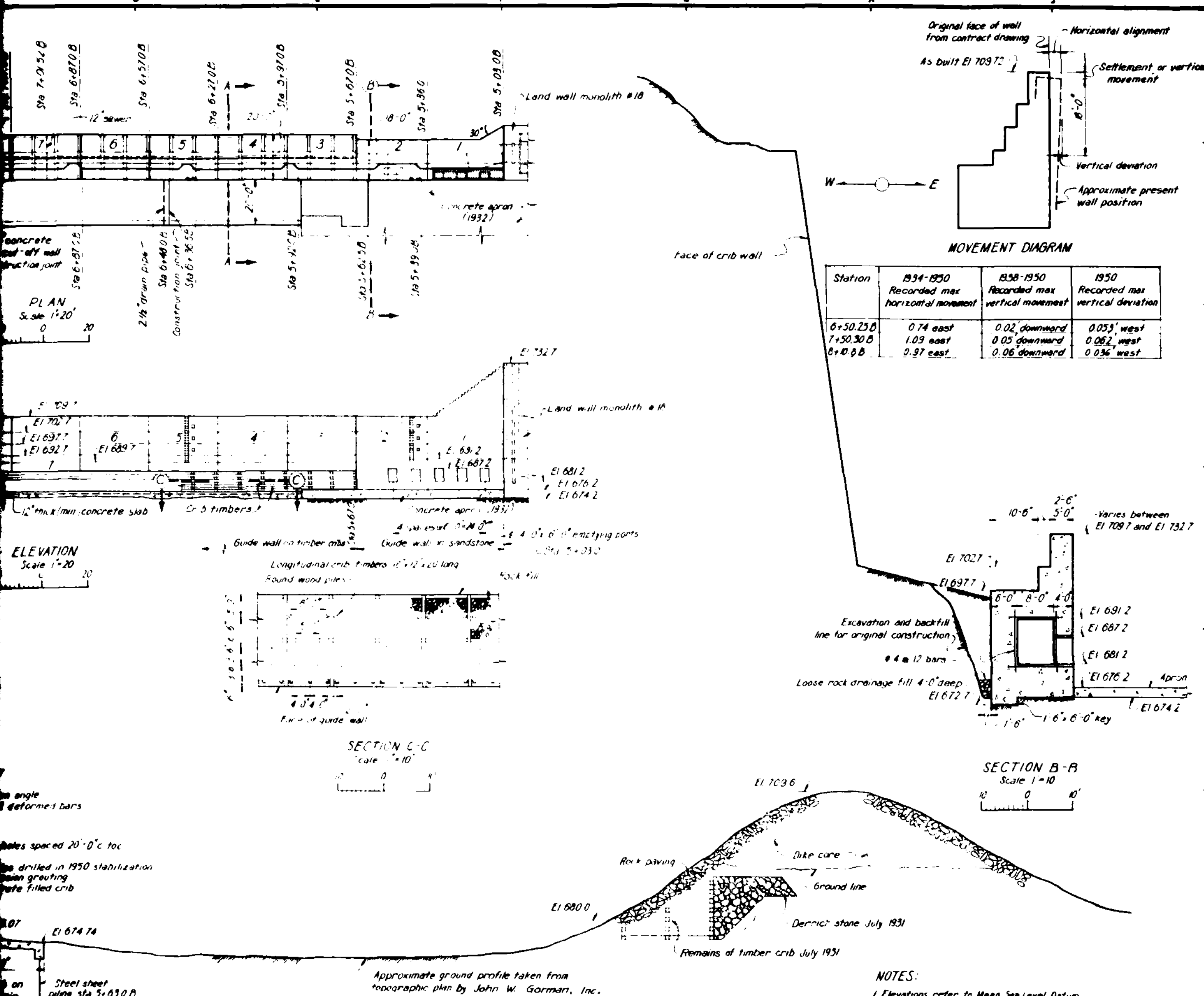
DRAWING NUMBER

SHEET OF

PLATE C-1



PLAN
Scale 1"=20'
Flow
Crib timber
12" sewer
Construction joint
Plain concrete slab, 12" min thickness on sandstone foundation (1951)
2'-0" concrete cut-off wall
Construction joint
2½" drain pipe
Concrete apron (1932)
ELEVATION
Scale 1"=20'
Bottom of concrete El 682.7
El 675.7 bottom of cribs
12" thick (min) concrete slab
Crib timbers
Guide wall on timber cribs
Guide wall on sandstone
Longitudinal crib timbers 16"x12"x20 long
Round wood piles
Face of guide wall
SECTION C-C
Scale 1"=10'
Face of crib wall
El 709.7
Protection angle
#6 @ 12 deformed bars
El 702.7
El 697.7
Excavation and backfill line for original construction
Top of remaining backfill after 3rd removal in 1950 El 692.7 El 689.7
4" weep holes spaced 20'-0" c toc
Grout holes drilled in 1950 stabilization by intrusion grouting
Concrete filled crib
El 682.7
Intrusion grout in rock filled crib
El 675.7
El 676.07
El 674.74
Concrete slab on sandstone 12" min thickness, placed in 1950
Steel sheet piling sta 5+63.0 B to sta 9+03.0 B
El 662.0
Approximate ground profile
SECTION A-A
Scale 1"=10'



MOVEMENT DIAGRAM

Station	1934-1950 Recorded max horizontal movement	1938-1950 Recorded max vertical movement	1950 Recorded max vertical deviation
6+50.25 B	0.74 east	0.02' downward	0.053' west
7+50.30 B	1.09 east	0.05' downward	0.062' west
8+10.8 B	0.97 east	0.06' downward	0.036' west

NOTES:

1 Elevations refer to Mean Sea Level Datum

2 Spalling problem at most monolith and horizontal joints which are void of joint material

3 Elevations of structure are those taken from original contract drawing except noted

HARZA ENGINEERING COMPANY
CHICAGO ILLINOIS

DEPARTMENT OF THE ARMY
ST PAUL DISTRICT CORPS OF ENGINEERS
ST PAUL MINNESOTA

STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO 1

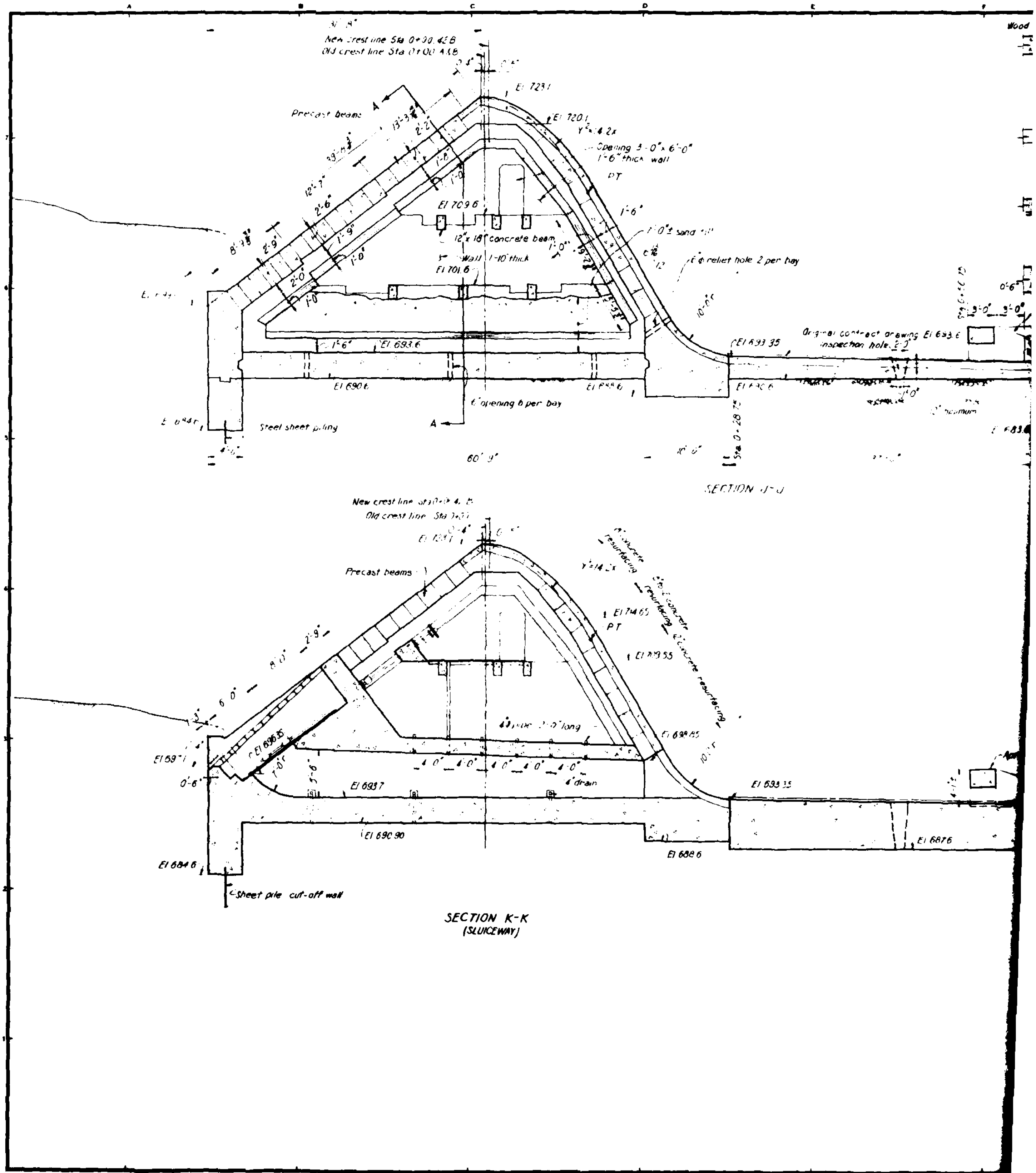
EXISTING STRUCTURES
LOWER GUIDE WALL

J B C
R N M
SUBMITTED BY
APPROVED

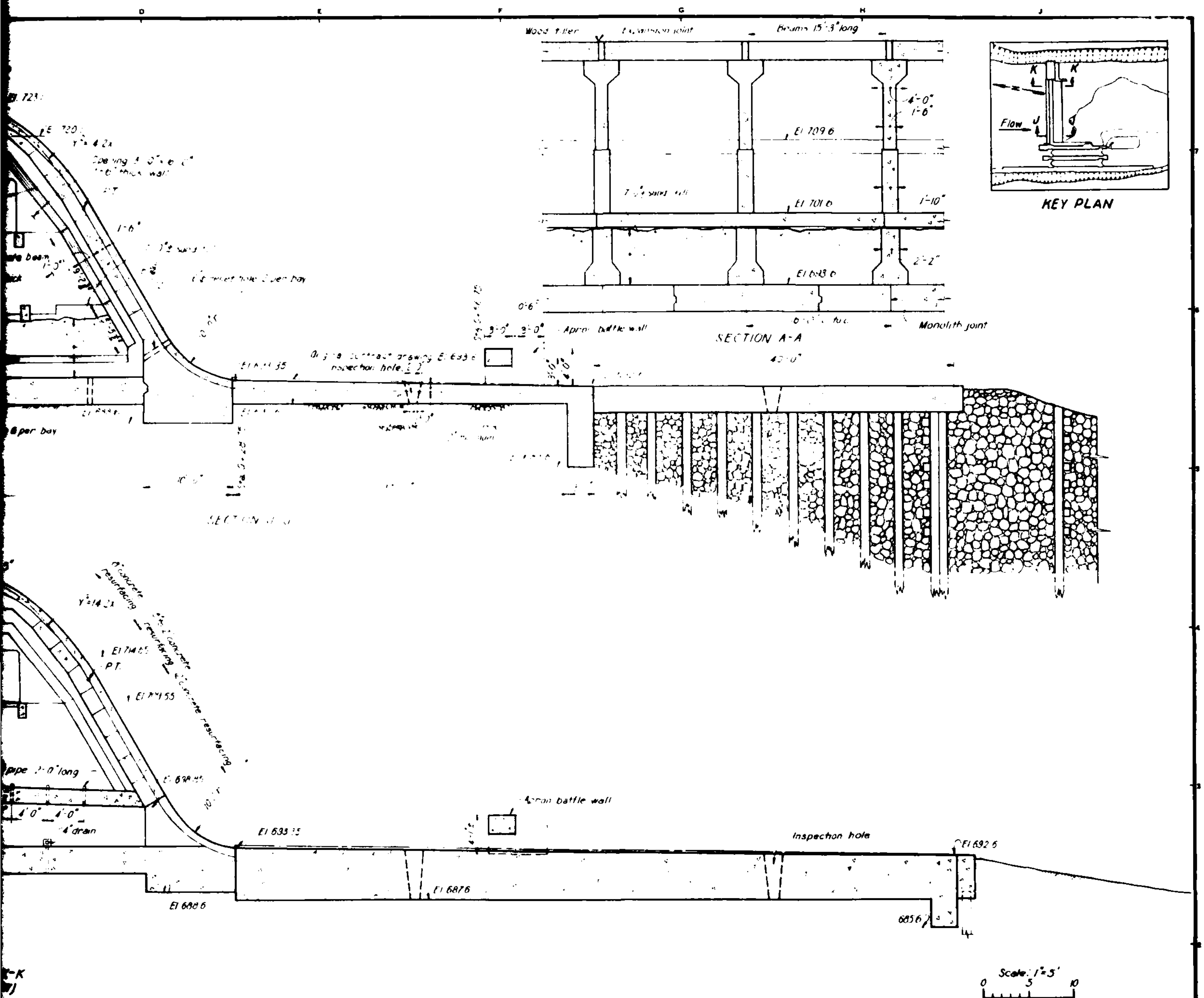
DATE
MARCH 1975

DRAWING NUMBER
SHEET OF

PLATE C-2



SECTION K-K
(SLUICEWAY)

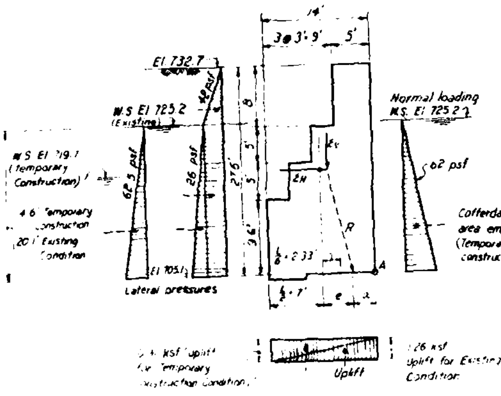


HARZA ENGINEERING COMPANY CHICAGO, ILLINOIS		DEPARTMENT OF THE ARMY ST. PAUL DISTRICT CORPS OF ENGINEERS ST. PAUL, MINNESOTA	
DRAWN BY	J B C	STUDY OF ALTERNATIVES FOR REHABILITATION	
CHECKED BY	R N M	MISSISSIPPI RIVER	LOCK & DAM NO 1
SUBMITTED BY:		EXISTING STRUCTURES	
APPROVED:		DAM	
		TRANSVERSE SECTIONS	
		DATE	MARCH 1975
		SCALE 1"=5'	
		DRAWING NUMBER	
		SHEET OF	

2

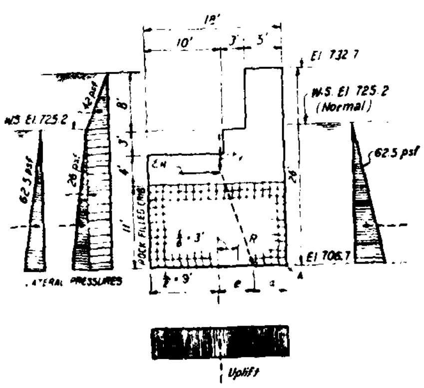
UPPER GUIDE WALL MONOLITHS

UPPER GUIDE WALL MONOLITHS 1-7 EXISTING AND CONSTRUCTION CONDITIONS



	Monoliths 1-7 Existing, Normal Loading condition	Monoliths 1-5 Temporary Construction condition
(1) Resultant: e	3.?? ft	4.22 ft
$\pm(\frac{L}{6}-e)$	-1.39 ft	-1.89 ft
(2) f_{SOIL}	7.46 ksf	11.34 ksf
(3) EH/EV	.35	.44
(4) FSS	1.80	1.4
(5) FSO.T	1.48	1.57

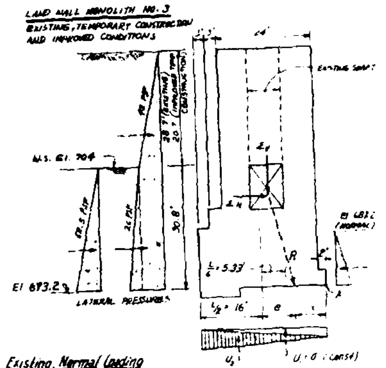
UPPER GUIDE WALL MONOLITHS #8-13 EXISTING, NORMAL LOADING CONDITION



(1) Resultant: e	3.40 ft
$\pm(\frac{L}{6}-e)$	-0.40 ft
(2) f_{SOIL}	4.13 ksf
(3) EH/EV	.77
(4) FSS	1.90
(5) FS.OT	2.06

LAND WALL MONOLITHS

LAND WALL MONOLITH NO. 3 EXISTING, TEMPORARY CONSTRUCTION AND IMPROVED CONDITIONS



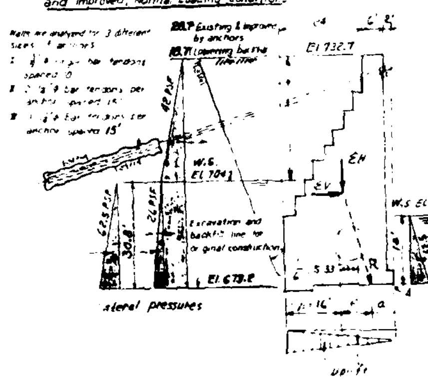
I Existing, Normal Loading (Without stabilization) Conduits lowered, and original conduit and shaft openings filled with conc.

II Temporary Construction Lock empty (All Rehab. Plans) Conduit lowered, but no conc. fill in original conduit & shaft openings. Elev. of backfill 724.7 ft

III Improved Stability Same as "Existing" (I) except for lowering of backfill by 8 ft

	I Existing, Normal Loading	II Temporary Construction Loading	III Improved, Normal Loading
(1) Resultant: e	7.?? ft	7.47 ft	5.61 ft
$\pm(\frac{L}{6}-e)$	-2.4?	-2.14 ft	-0.28 ft
(2) f_{soil}	16.10 ksf	13.39 ksf	12.60 ksf
(3) EH/EV	.42	.44	.34
(4) FSS	1.48	1.4?	1.5?
(5) FSOT	1.67	1.97	2.05

LAND WALL MONOLITHS 5-15 - Existing, Temporary construction and improved, Normal Loading Conditions

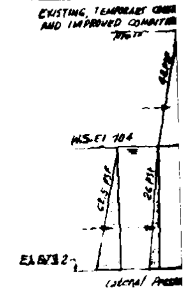


	Existing Normal Loading	Temporary Construction Loading	Improved stability by 10' Lowering of Backfill, Normal loading
(1) Resultant: e	8.56 ft	8.50 ft	?.59 ft
$\pm(\frac{L}{6}-e)$	-3.23 ft	-3.20 ft	-.26 ft
(2) f_{SOIL} (max)	17.74 ksf	18.84 ksf	12.33 ksf
(3) EH/EV	43	44	36
(4) F.S.S.	1.44	1.35	1.74
(5) F.S.O.T	1.57	1.50	1.87

IMPROVED STABILITY BY ROCK ANCHORS

	I $1\frac{3}{8}$" ø single bar tendons spaced 10'	II 2-$1\frac{1}{4}$" ø bar tendons/anchor spaced 15'	III 3-$1\frac{1}{4}$" ø bar tendons/anchor spaced 15'
(1) Resultant: e	4.86 ft	4.90 ft	2.50 ft
$\pm(\frac{L}{6}-e)$	+0.47 ft	1.00 ft	+2.83 ft
(2) f_{SOIL} (max)	12.10 ksf	12.00 ksf	9.80 ksf
(3) $\frac{EH}{EV}$	.355	33	29
(4) FSS	1.76	1.?9	2.?6
(5) FSOT	2.37	2.45	3.27

LAND WALL MONOLITH NO...
EXISTING, TEMPORARY ... AND IMPROVED CONDITI...



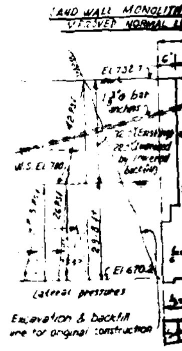
I Existing, Normal Loading (without stabilization) Conduits lowered 2 new gate shafts Concrete fill in original conduit opening & for uniform monolith sect.

II Temporary Construction Lock empty (All Rehab. Plans) Conduit lowered but no conc. fill Elevation of backfill ...

III Improved Stability Same as "Existing" (I) except for lowering of backfill by 10 ft

(1) Resultant
(2) f_{soil}
(3) EH/EV
(4) F.S.S.
(5) F.S....

LAND WALL MONOLITH... IMPROVED NORMAL L...



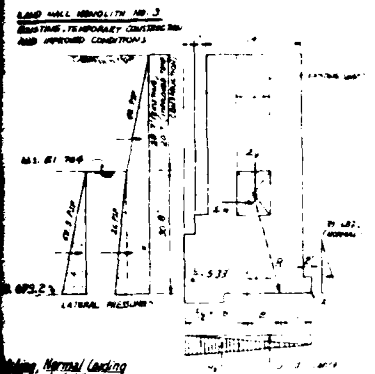
Excavation & backfill line for original construction

1) Resultant: $\pm(\frac{L}{6}-$...
2) f_{soil} (max)
3) EH/EV
4) F.S.S
(5) F.S.O.T.

1) Resultant: e
$\pm(\frac{L}{6}-e)$
2) f_{soil} (max)
3) EH/EV
4) F.S.S.
5) F.S.O.T.

LAND WALL MONOLITHS

LAND WALL MONOLITH NO. 3 — EXISTING, TEMPORARY CONSTRUCTION AND IMPROVED CONDITIONS



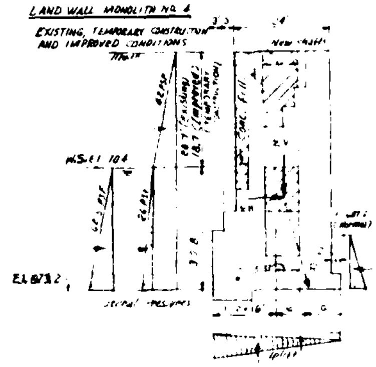
	I Existing Normal Loading	II Temporary Construction Loading	III Improved Normal Loading
(1) Resultant e	7.18 ft	7.47 ft	5.61 ft
$\pm(\frac{b}{6}-e)$	-1.40	-1.4 ft	0.4 ft
(2) f_{soil}	16.10 ksf	13.39 ksf	12.60 ksf
(3) EH/EV	.42	.44	.34
(4) F.S.S	1.45	1.43	1.55
(5) F.S.O.T	1.67	1.97	2.05

I Existing, Normal Loading without stabilization bars, conduits lowered, and conduit and shaft openings filled with conc.

II Temporary Construction — empty (All Rehab. Plans); conduit lowered, but no fill in original conduit openings; of backfill El 674.7 ft

III Improved Stability — same as "Existing (I)" except for lowering of backfill by 8 ft (El 696)

LAND WALL MONOLITH NO. 4 — EXISTING, TEMPORARY CONSTRUCTION AND IMPROVED CONDITIONS



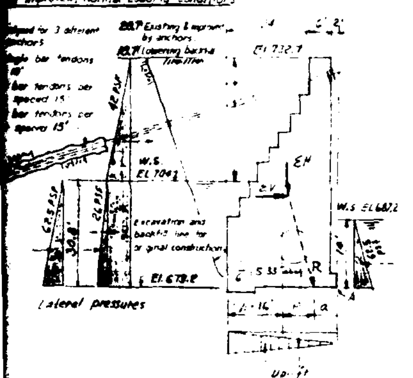
	I Existing Normal Loading	II Temporary Construction Loading	III Improved Normal Loading
(1) Resultant e	8.04 ft	7.44 ft	5.38 ft
$\pm(\frac{b}{6}-e)$	[illegible]	[illegible]	0.25 ft
(2) f_{soil}	6.36 ksf	13.40 ksf	11.58 ksf
(3) EH/EV	.445	[illegible]	.336
(4) F.S.S	1.4	[illegible]	[illegible]
(5) F.S.O.T	1.62	1.79	2.06

I Existing, Normal Loading without stabilization, conduits lowered, 2 new gate shafts, conduits in the original shaft filled with conc.

II Temporary Construction (All Rehab. Plans) — Conduit lowered, but no conc. fill; Elevation of backfill

III Improved Stability — Same as "Existing (I)" except for lowering of backfill by 8 ft

LAND WALL MONOLITHS 5-15 — Existing, Temporary Construction and Improved, Normal Loading Conditions

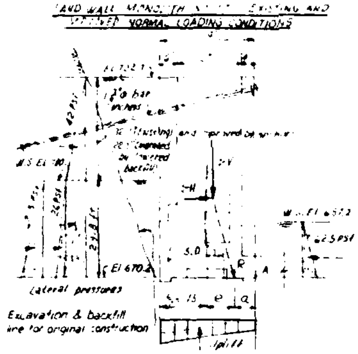


	Existing Normal Loading	Temporary Construction Loading	Improved stability by 10' Lowering of Backfill, Normal loading
Resultant: e	8.56 ft	8.50 ft	6.59 ft
$\pm(\frac{b}{6}-e)$	-3.23 ft	-3.20 ft	-1.26 ft
f_{soil} (max)	17.74 ksf	18.84 ksf	12.33 ksf
EH/EV	43	46	36
F.S.S.	1.44	1.35	1.74
F.S.O.T	1.57	1.50	1.87

IMPROVED STABILITY BY ROCK ANCHORS

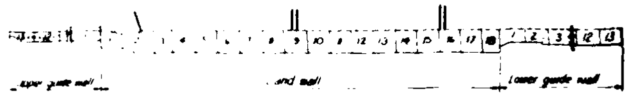
	I $1\frac{3}{8}$" Ø single bar tendons spaced 10'	II 2-$1\frac{1}{4}$" Ø bar tendons/anchor spaced 15'	III 3-$1\frac{3}{8}$" Ø bar tendons/anchor spaced 15'
Resultant e	4.86 ft	4.30 ft	2.50 ft
$\pm(\frac{b}{6}-e)$	+0.47 ft	1.00 ft	+2.83 ft
f_{soil} (max)	12.10 ksf	12.30 ksf	[illegible] ksf
EH/EV	.355	33	29
F.S.S	1.76	[illegible]	[illegible]
F.S.O.T.	2.37	2.45	[illegible]

LAND WALL MONOLITH — EXISTING AND IMPROVED NORMAL LOADING CONDITIONS



	Existing Condition	Improved Stability by 10' lowering of backfill
1) Resultant: e	9.70 ft	6.90 ft
$\pm(\frac{b}{6}-e)$	-4.70 ft	-1.90 ft
2) f_{soil} (max)	26.0 ksf	16.44 ksf
3) EH/EV	.455	.350
4) F.S.S	1.38	1.79
(5) F.S.O.T.	1.41	1.81

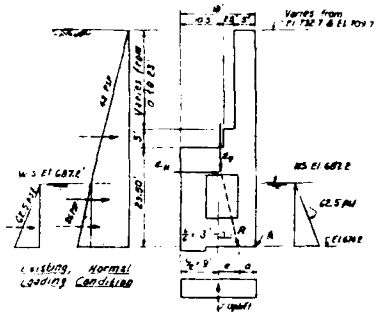
	Improved Stability by $1\frac{3}{8}$" Ø anchors spaced 10 feet	Improved stability by both $1\frac{3}{8}$" Ø anchors and 10' backfill lowering
1) Resultant e	6.00 ft	3.20 ft
$\pm(\frac{b}{6}-e)$	[illegible]	1.80 ft
2) f_{soil} (max)	15.70 ksf	11.10 ksf
3) EH/EV	[illegible]	.27
4) F.S.S.	[illegible]	2.34
5) F.S.O.T	[illegible]	2.23



KEY PLAN

LOWER GUIDE WALL MONOLITHS

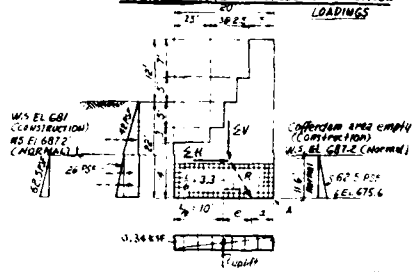
LOWER GUIDE WALL MONOLITH #1



Existing, Normal Loading Condition

(1) Resultant e	7.00 ft
$\pm(\frac{b}{6}-e)$	4.00 ft
(2) f_{soil}	3.348 ft
(3) EH/EV	.463
(4) F.S.S	[illegible]
(5) F.S.O.T	1.34

LOWER GUIDE WALL MONOLITHS 3-13 NORMAL AND CONSTRUCTION CONDITION LOADINGS



	Existing Condition (Normal Loading)	Monoliths 6-12 Construction Condition
(1) Resultant e	2.70 ft	3.22 ft
$\pm(\frac{b}{6}-e)$	0.63 ft	0.11 ft
(2) f_{soil}	5.39 ksf	6.95 ksf
(3) EH/EV	.15	0.22
(4) F.S.S	4.07	2.64
(5) F.S.O.T	5.75	3.78

NOTES:

1. Location of Resultant:
   Outside of middle $\frac{1}{3}$ by $(-)(\frac{b}{6}-e)$
   Within the middle $\frac{1}{3}$ by $(+)(\frac{b}{6}-e)$

2. Abreviations:
   f_{soil} = Foundation pressure, ksf
   E_H/E_V = Factor of sliding
   F.S.S. = Factor of safety against sliding
   F.S.O.T. = Factor of safety against overturning

SAFETY PAYS

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT CORPS OF ENGINEERS
ST. PAUL, MINNESOTA

DESIGNED BY: R.N.M., M.J.
DRAWN BY: R.N.M., M.J.
CHECKED BY: M.J., R.N.M.
SUBMITTED BY:
APPROVED:

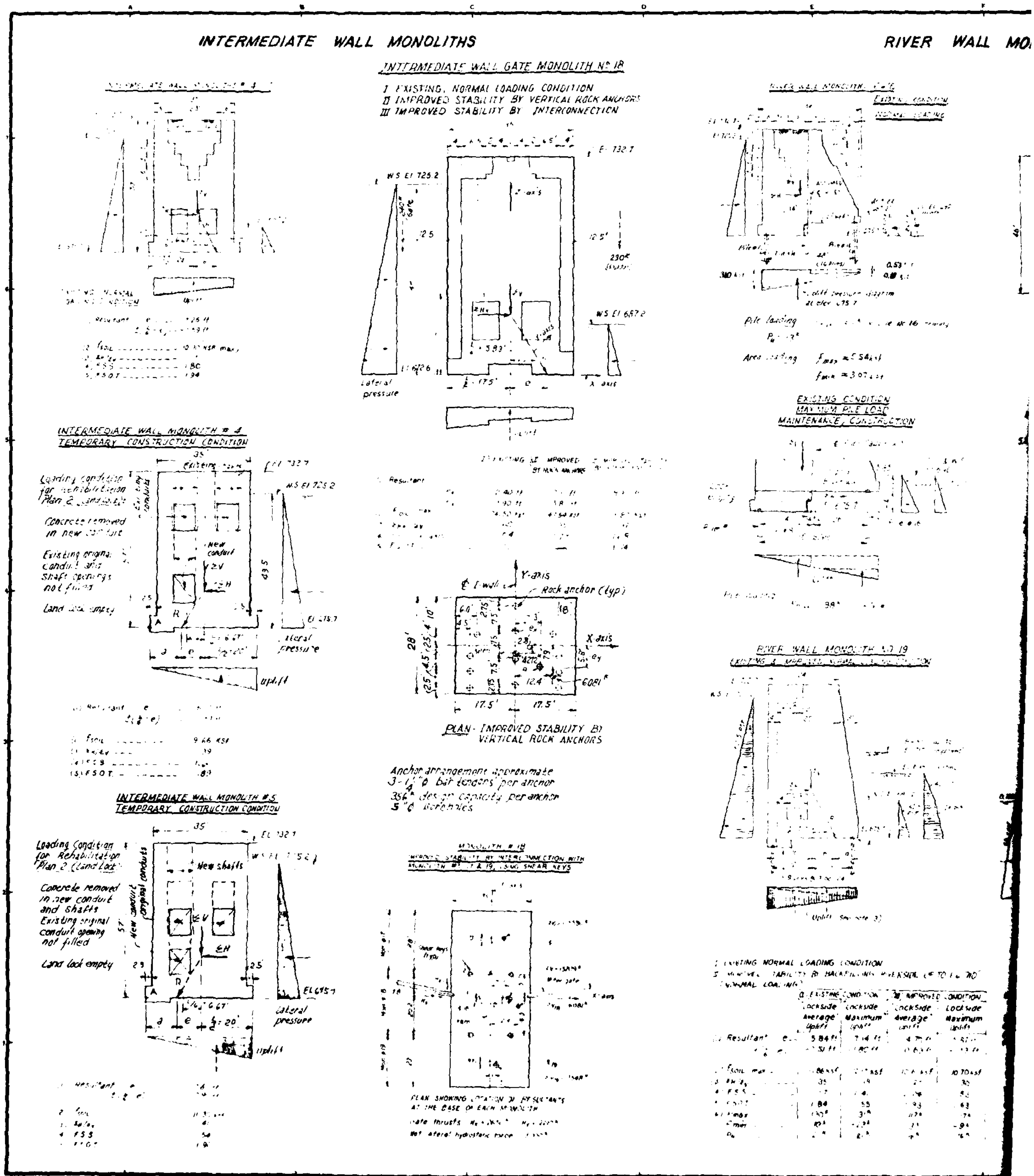
STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO. 1
STRUCTURAL INVESTIGATIONS
STABILITY ANALYSES OF LOCK WALLS
SHEET 1

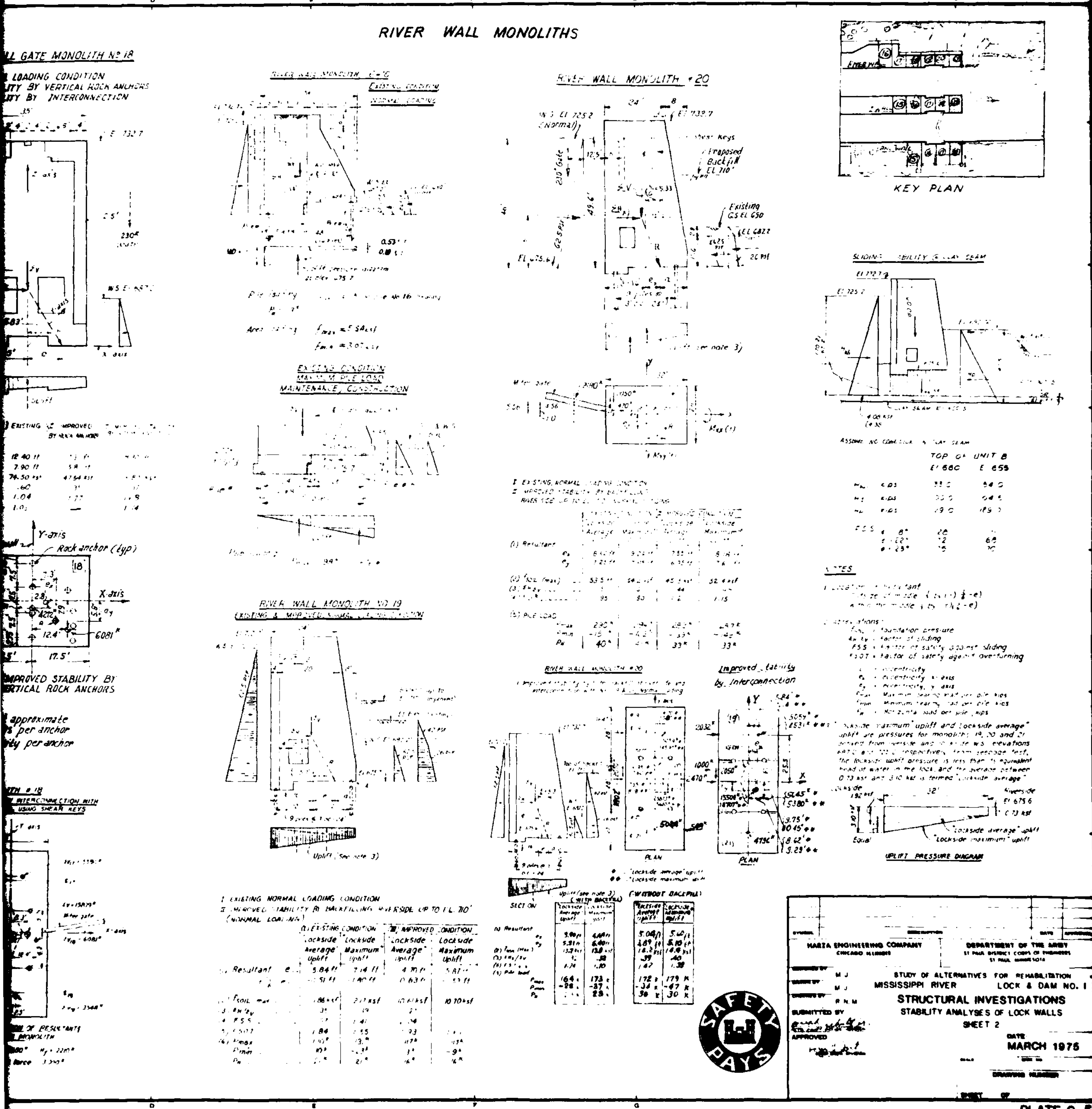
DATE: MARCH 1975

DRAWING NUMBER
SHEET OF

PLATE C-4

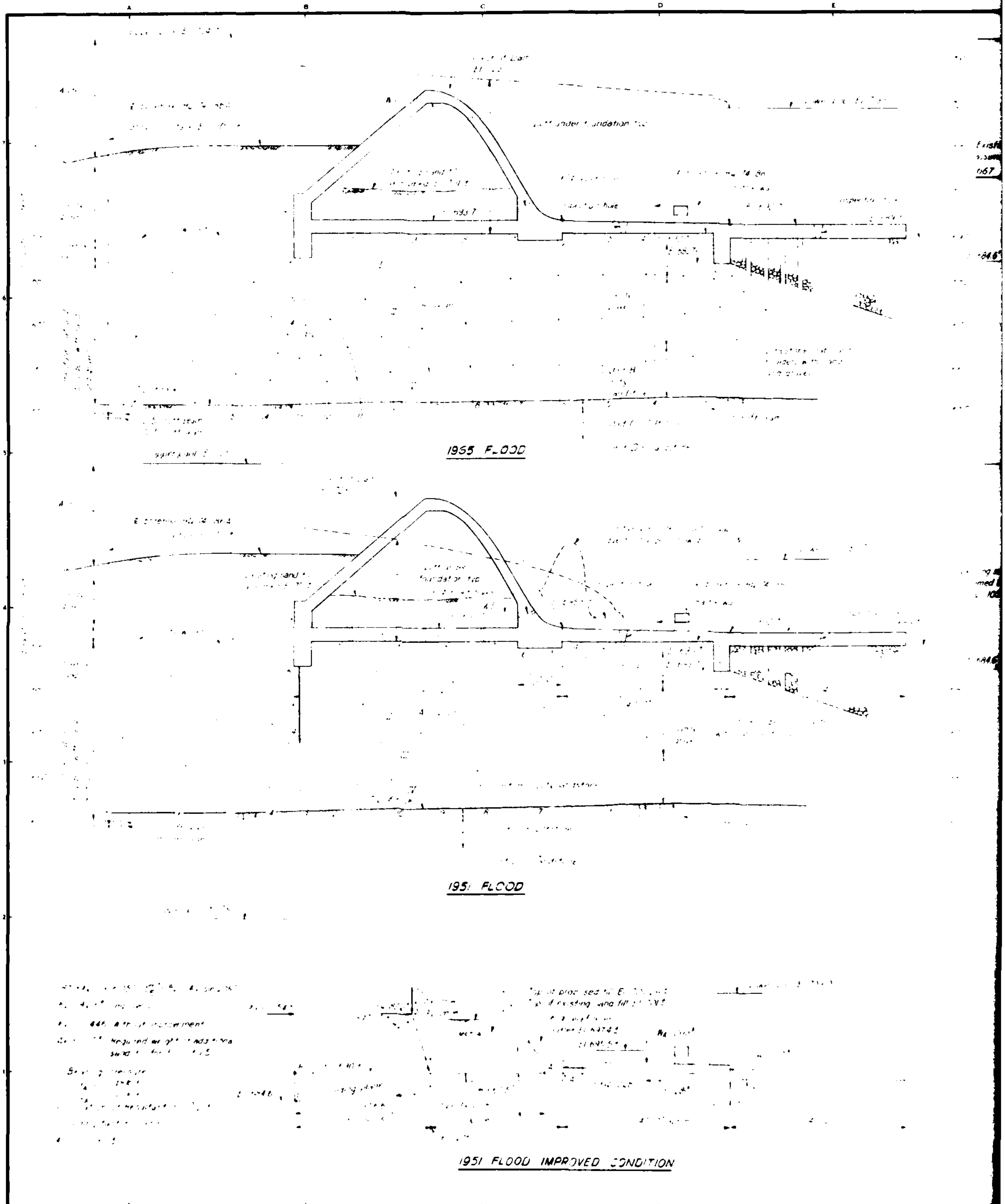
2


INTERMEDIATE WALL MONOLITHS
RIVER WALL MO
INTERMEDIATE WALL GATE MONOLITH Nº 18
I EXISTING, NORMAL LOADING CONDITION
II IMPROVED STABILITY BY VERTICAL ROCK ANCHORS
III IMPROVED STABILITY BY INTERCONNECTION
W.S. El 725.2
El 732.7
W.S. El 657.2
lateral pressure
INTERMEDIATE WALL MONOLITH # 4
TEMPORARY CONSTRUCTION CONDITION
Loading condition for rehabilitation Plan 2
Concrete removed in new conduit
Existing original conduit and shaft openings not filled
Land lock empty
lateral pressure
uplift
Y-axis
Rock anchor (typ)
X axis
PLAN - IMPROVED STABILITY BY VERTICAL ROCK ANCHORS
Anchor arrangement approximate
3-1¼"φ bar tendons per anchor
356k design capacity per anchor
5"φ boreholes
INTERMEDIATE WALL MONOLITH # 5
TEMPORARY CONSTRUCTION CONDITION
Loading Condition for Rehabilitation Plan 2 (Land Lock)
Concrete removed in new conduit and shafts
Existing original conduit opening not filled
Land lock empty
New shafts
lateral pressure
uplift
MONOLITH # 18
PLAN SHOWING LOCATION OF RESULTANTS AT THE BASE OF EACH MONOLITH
EXISTING CONDITION
MAXIMUM PILE LOAD
MAINTENANCE, CONSTRUCTION
RIVER WALL MONOLITH NO 19
Pile loading
Area loading
Uplift
I EXISTING NORMAL LOADING CONDITION
Lockside Average Uplift
Lockside Maximum Uplift
Resultant

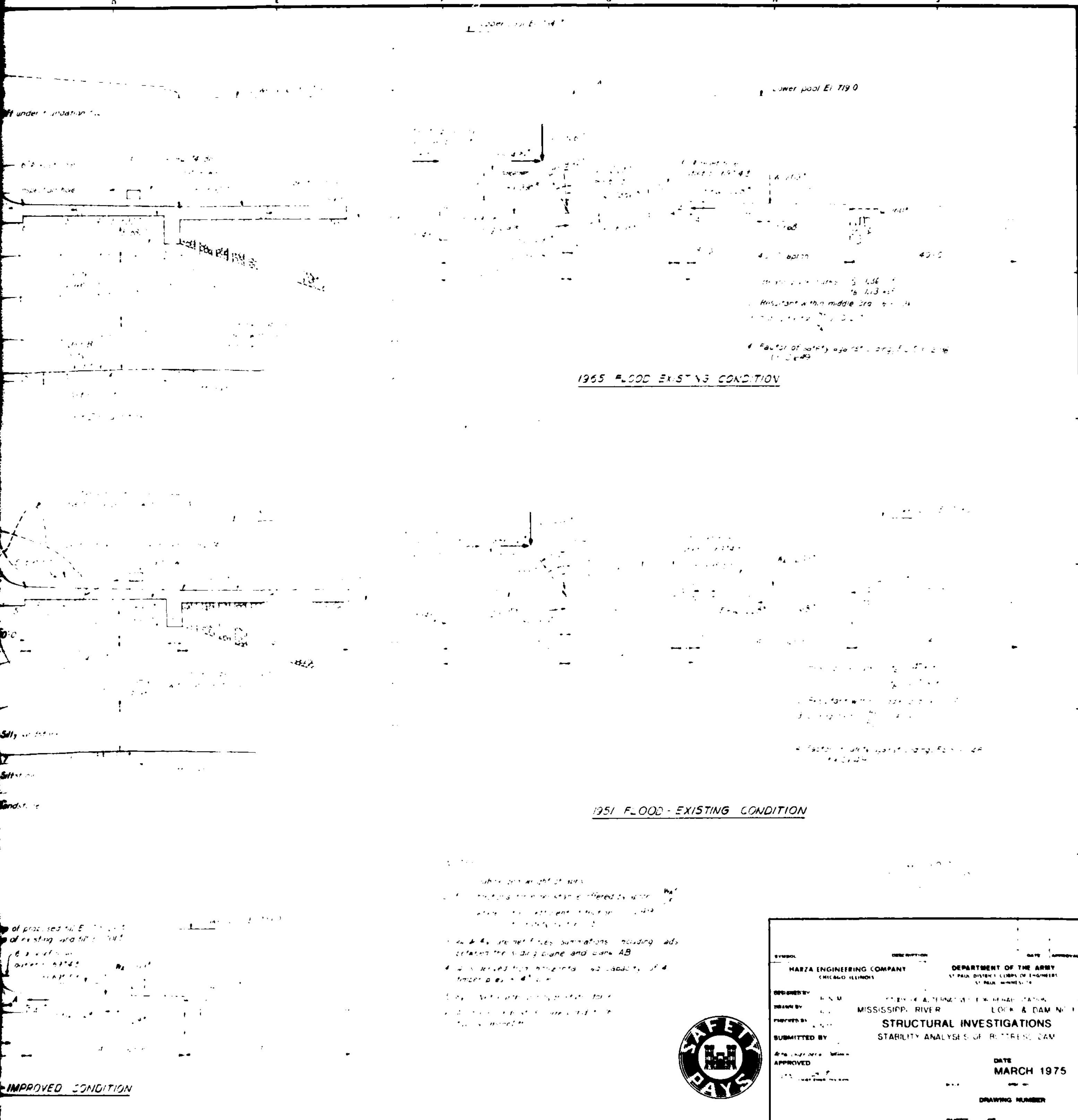

RIVER WALL MONOLITHS
RIVER WALL MONOLITH #20
RIVER WALL MONOLITH NO 19
EXISTING CONDITION
MAINTENANCE, CONSTRUCTION
KEY PLAN
IMPROVED STABILITY BY VERTICAL ROCK ANCHORS
Rock anchor (typ)
Y-axis
X-axis
Improved stability by Interconnection
PLAN
SECTION
UPLIFT PRESSURE DIAGRAM
NOTES
I EXISTING NORMAL LOADING CONDITION
Lockside Average Uplift
Lockside Maximum Uplift
Existing GS EL 650
Proposed Backfill EL 710'
MARTA ENGINEERING COMPANY
CHICAGO ILLINOIS
DEPARTMENT OF THE ARMY
ST PAUL DISTRICT CORPS OF ENGINEERS
ST PAUL, MINNESOTA
STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO. 1
STRUCTURAL INVESTIGATIONS
STABILITY ANALYSES OF LOCK WALLS
SHEET 2
MARCH 1975
SAFETY PAYS
PLATE C-5

1

2



1955 FLOOD
1951 FLOOD
1951 FLOOD IMPROVED CONDITION



Lower pool El 719.0
Resultant within middle third
Factor of safety against sliding
1965 FLOOD EXISTING CONDITION
1951 FLOOD - EXISTING CONDITION
IMPROVED CONDITION
HARZA ENGINEERING COMPANY
CHICAGO ILLINOIS
DEPARTMENT OF THE ARMY
MISSISSIPPI RIVER
LOCK & DAM
STRUCTURAL INVESTIGATIONS
STABILITY ANALYSES OF BUTTRESS DAM
SUBMITTED BY
APPROVED
DATE
MARCH 1975
DRAWING NUMBER
SHEET
OF
SAFETY PAYS

PLATE C–6

DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT, CORPS OF ENGINEERS
1210 U.S. Post Office & Custom House
St. Paul, Minnesota 55101

MISSISSIPPI RIVER

STUDY OF ALTERNATIVES FOR REHABILITATION OF LOCK AND DAM NO.1

MINNEAPOLIS, MINNESOTA

APPENDIX D

FILLING, VENTING AND EMPTYING SYSTEMS

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

TABLES

PLATES

SUPPLEMENT

Report Entitled:

Hydraulic Problems and Recommended Solutions
at Lock and Dam No. 1, Mississippi River

prepared by
Martin E. Nelson

for

Department of the Army
St. Paul District, Corps of Engineers
1210 U.S. Post Office and Custom House
St. Paul, Minnesota 55101

January 1974

Contract No. DACW37-74-C-0038

Appendix D

FILLING, VENTING AND EMPTYING SYSTEMS

Problems Associated with the Existing System

1. Inadequacy of the System. The existing hydraulic filling, emptying and venting system is inadequate and causes turbulence, hazardous to navigation during locking operations.

2. Turbulence in Lock Chambers. Excessive turbulence, hazardous to small boats and craft, occurs in lock chambers during filling operations. These undesirable conditions are caused by insufficient submergence of the ports and by entrained air in water flowing from culverts into lock chambers. The violent action in the locks, especially during the beginning of the filling cycle, is further compounded by the direct opposite location of the ports.

3. Turbulence Downstream of Lock. Downstream of the lock structure the concentrated discharge emerging from the culverts are not baffled and generate large waves which easily swamp boats located in the navigation channel. Water emerging from the culvert in the river wall is discharged towards a center island and could endanger people and craft located in the area.

4. Air Entrapment in Culverts. Since the crowns of the culverts are above normal lower pool elevation, air is present in the culverts. During the filling and emptying operation the air gets entrapped in the culverts because of the inadequacy of the present venting system. During the filling cycle the inrushing water will compress the air in the culverts and cause pulsating pressures on the downstream valves. Since the culverts between the intakes and the filling valves are located at a higher level than the culverts along the ports, the air entrainment at the beginning of the filling cycle is accentuated by the air in the sloping section of the culvert between the filling valve and the tailwater level.

5. Vortices at Culvert Intakes. Upstream of the locks, near the culvert intakes in the upper miter gate recesses, vortices are formed during the filling operations. Ice and floating

debris drawn into the lock entrances may become lodged in the miter gate recesses and thus obstruct full opening of the miter gate for navigation passages.

Filling and Emptying Operations

6. The time needed for filling and emptying of land lock at existing conditions is shown on Plate D-1. Emptying of river lock through river culvert is also given on the same plate. Discharge rates during the filling and emptying operations of the land lock are presented on Plate D-2. For the emptying of river lock the discharge rate is given when using the riverward culvert only.

Suggested Hydraulic Improvements

7. Solutions Recommended in Contract DACW37-74-C-0038. The hydraulic problems associated with the present filling and emptying systems are described in detail by Martin E. Nelson in a report titled "Hydraulic Problems and Recommended Solutions at Lock and Dam No. 1, Mississippi River" (Contract DACW37-74-C-0038). M.E. Nelson proposed to improve its operation and to reduce hazards to navigation. The suggested improvements, described in detail in the attached report, are as follows:

a) Relocate the filling valves to the lower culvert level with invert at elevation 681.2. Air entrainment and surges in the culverts will be reduced by this modification.

b) Fill the culvert crowns with concrete above elevation 687.2 to exclude air in the culverts at low tailwater elevations.

c) Lower the ports in lock walls to discharge flush with lock chamber floors to increase the submergence of the jets emerging from the ports during the filling cycle.

d) Enlarge the venting system to facilitate the escape of air from the culverts.

e) Provide energy dissipation by constructing bottom lateral discharge manifolds downstream of each lock chamber.

f) Construct new intake manifolds upstream of the miter gate recesses to reduce tendency for floating debris to interfere with opening of miter gates.

g) Replace the present stoney gate filling valves with new reversed tainter gate, vertical lift gate, or butterfly valves.

h) If damaged beyond repair, replace the present emptying valves with new reversed tainter gates, vertical lift gates, or butterfly valves.

8. Alternative Solutions for River Wall Outlet. Subsequent to his report, M. E. Nelson submitted the following 3 alternative proposals for discharges from the river wall culvert:

Alt. A: Covered conduit under backfill and hydraulic jump stilling basin for energy dissipation east of the river wall.

Alt. B: Covered conduit under backfill with 90 degree bend and 5 discharge ports east of the river wall.

Alt. C: Energy dissipating and flow diffusing lateral discharge manifolds downstream of the river lock, combined into one system with staggered laterals discharging from the intermediate wall.

Alternatives A and B of these proposals are presented on Plate D-4 and Alternative C is incorporated in the system shown on Plate D-3. Alternative C would be more favorable than Alternatives A or B, since it would keep the discharge in one area where it can be observed instead of in an area where boaters or fishermen might not be observed. Alternative B would be better than Alternative A as it would direct the flow into a safer area.

Intake Manifolds

9. General. All new intake manifolds will be constructed within an area enclosed by a cellular cofferdam. The proposed intakes are shown on Plate D-3 and a plan and sections of the cofferdam are given on Plate E-1 of Appendix E.

10. Land Wall. To construct new intakes to the land wall filling culvert, the existing monoliths Nos. 1 and 2 of the upper guide wall must be removed. Since the only vehicular access to the locks is immediately behind the upper guide wall, a retaining wall consisting of soldier beams and lagging should be placed before excavating for the new intake monoliths. This retaining wall could serve also as formwork when pouring concrete for the intake monoliths. To provide adequate submergence for the intake ports, the present foundation of the guide wall monoliths must be lowered from elevation 706± to elevation 704.7. For construction purposes the intakes will be located so that the contraction joint between the two monoliths will be located centrally across the pier between the 3rd and 4th intake openings.

11. Intermediate Wall. New intakes for the intermediate wall culverts will be placed at the site of the present most upstream of Sta. 0+28.5A. In case of rehabilitation of landward lock only, the culvert for filling of the landward lock will have new intakes. The intakes to the riverward lock culvert in the intermediate wall will remain unchanged in the miter gate recess. For the construction of the intake ports the existing concrete will be removed from the deck level down to elevation 704.7. After completing the intakes the monolith will be built up to elevation 732.7, i.e. to the deck elevation of the intermediate wall. If both locks will be rehabilitated, separate intakes could be provided for each culvert in the intermediate wall as shown on Plate D-1.

12. River Wall. According to M. E. Nelson's proposal, the manifold in the river wall would have intake orifices on the river-ward side of the two most upstream river wall monoliths. To facilitate construction of the upstream cofferdam and to reduce the amount of concrete work, however, it is advisable to relocate the intake orifices to the lock side. The proposed 6 intake openings will be distributed equally between Monoliths Nos. 1 and 2 of the river wall. Existing concrete in the monoliths will be cut down to elevation 704.7, and after completion of the intake manifold, the monoliths will be built up to elevation 732.7. The proposed intake manifold is shown on Plate D-3.

13. Concrete Apron. To prevent erosion a concrete apron with top elevation at 706.7, i.e. 2 feet below the invert elevation of the intake manifolds will be constructed in front of the intake structures.

Discharge Manifolds

14. Landward Lock. For the rehabilitation of the landward lock only, lateral manifold systems will be constructed to discharge water through the land and intermediate walls downstream of the lock chambers. To construct discharge manifolds, the existing concrete slab downstream of both lock chambers and Monoliths Nos. 1 through 3 of the lower guide wall must be removed. Since the access to the downstream construction area is behind the lower guide wall, steel sheet piling or soldier beam and lagging retaining wall will be placed behind the existing guide wall monoliths. The retaining wall could also serve as formwork for the new discharge monoliths. The construction area will be excavated down to elevation 666.7 at the lower guide wall and downstream of the landward lock chamber, and down to elevation 668.7 downstream of the intermediate wall and riverward lock chamber. Plan and sections of the discharge manifolds are shown on Plate D-4.

15. Both Locks. The scheme of rehabilitation of both locks depends on the treatment of discharges from the river wall culvert. As a solution, the discharge manifold system proposed for the rehabilitation of the landward lock could be used for the rehabilitation of both locks, with the addition of an hydraulic jump stilling basin east of the river wall to dissipate energy of the flow emerging from the river wall culvert (Alt. A on Plate D-5). A sub-alternative of this scheme would be to provide the emptying conduit from the river wall with second 90 degree bend and then discharge through several orifices in the downstream direction with or without a stilling basin (Alt. B on Plate D-5). Since Monoliths 1, 2, and 3 of the river wall lower guide wall are in need of replacement because of downstream movement and unsafe stairway, it is assumed that these monoliths will be replaced if the above Alternatives A or B will be selected.

Another scheme for the rehabilitation of both locks consists of bottom lateral discharge manifolds as shown on Plate D-6. This scheme provides almost complete energy dissipation and flow dispersion downstream of the locks. Before construction of the manifold system, the existing concrete apron downstream of the lock chambers, Monoliths Nos. 1, 2, and 3 of the lower guide wall, and part of the crib and rockfill guide wall downstream of the river wall must be removed.

Filling Valve Location and Setting

16. To reduce the amount of air entrapped in the filling culverts, the filling valves will be lowered from the present upper conduit invert elevation 708.7 to new lower conduit invert elevation 678.7. The new location of the filling valves will be in Monolith No. 4 in the land wall and in Monolith No. 5 in the intermediate wall for the rehabilitation of the landward lock. For the rehabilitation of the riverward lock, the new filling valves will be located in Monolith No. 5 of the intermediate wall and in Monolith No. 7 of the river wall. New downstream bulkhead gates for these filling valves will be located in adjacent monoliths immediately downstream of the filling valve monoliths. The bulkhead gates upstream of the filling valves will remain at the present locations.

Filling and Emptying Ports

17. The filling and emptying ports in the lock walls (see Plate D-7 and Drawing M-R1-5/32 of the attached Report by M. E. Nelson), which are too high for safe lock operation, will be lowered to maintain the inverts flush with the floor slabs. To improve hydraulic conditions in the lock chambers, the ports in the lock walls should not be opposite each other but staggered. Since the cross-sections of the ports are too small for convenient construction activities, considerable amount of overbreak is required to provide easy access to place reinforcing steel, forms, and concrete. When pouring concrete around the new ports, the old ports will be filled simultaneously.

Miscellaneous Modifications

18. General. To determine the effectiveness of the lowering of culverts and/or the installation of larger vents, hydraulic model studies will be necessary. Results of the hydraulic tests will determine the extent of these modifications.

19. Lowering of Culverts. To fill the culvert crowns with concrete the roofs of the culverts should be roughened and seats must be cut in the existing concrete. In the intermediate and river walls, where compaction grouting of the wall foundation is required, the grouting work should be carried out from the filling and emptying culverts before starting any new construction in the culverts. Vent and grout pipes, reinforcing steel and forms should be placed into the crown of the culverts before filling the crown sections with pumped concrete.

20. Venting System. The need, size and number of additional vents should be determined by hydraulic model testing.

Cost of Hydraulic Modifications

21. According to the detailed cost estimates (January 1975 price levels) presented in Appendix A for the Alternative Plans of Rehabilitation, the subtotal direct costs of the hydraulic modifications and improvements are as follows:

PLAN 1	2,685,000
PLAN 2	2,588,000
PLAN 3	2,539,000
PLAN 4	
Alt. A. With straight discharge conduit and stilling basin east of river wall	4,370,000
Alt. B. With bent discharge conduit east of river wall	4,446,000
Alt. C. With extension of river wall and staggered bottom discharge laterals downstream of both locks	4,926,000

Of the various schemes for Plan 4, Alt. C offers almost complete energy dissipation and flow dispersion downstream of both locks. In case of Alt. A, energy dissipation for discharges from the river wall will be handled by the stilling basin. The dimensions and performance of the stilling basin, however, need further study in order to be certain of its satisfactory functioning at various operating conditions. In Alt. B, flows from the river wall are conveyed into a narrow channel southeast of the lock. Also this alternative requires additional studies to ascertain its satisfactory performance. Costs of the various elements are given in Table D-1.

Hydraulic Model Testing

22. On 16 June 1975 an in-house District meeting was held to summarize and firm up the District's recommendations for items to be tested in the model study at WES. The comments and opinions given by personnel from WES and by Mr. John Davis (OCE, retired) during their visit to the District on 3-4 June 1975, were considered. All model tests should be directed toward the goal of the shortest filling time consistent with minimum turbulence in the lock chamber and an acceptable discharge condition below the locks. It was decided that the following items should be tested:

a) Using a square (or rectangular) culvert shape with the same area as that of the existing culverts, begin tests with the culvert crown one foot below minimum tailwater of 687.2. This would place the culvert invert about two feet above the bottom of the lock wall, which would be possible to construct in the prototype.

b) If the lock hydraulic performance is not significantly improved by the one-foot submergence of the culvert crown, test the condition of the culvert crown submerged three feet below minimum tailwater, as suggested by OCE. WES personnel and Mr. John Davis felt that at least this amount of submergence and preferably more would be needed to cushion the culvert flow and to produce acceptable hydraulic performance. However, it would be very costly and possibly infeasible to lower the lock culverts to get three feet of submergence. Nevertheless, it is necessary to know if this much submergence is needed to significantly reduce the turbulence problem in the lock chamber during filling.

c) Venting system - Mr. Davis suggested eliminating the venting system altogether. However, it is suggested that WES determine whether or not a venting system would be needed at the lock. It is recognized that air flow requirements of a venting system cannot be accurately scaled so that the model can give only an indication of the possible need for vents.

d) Gates - Harza Engineering Company's recommendation of using slide gates throughout was rejected because of the anticipated wear of the slides due to sediment and because of the lack of use of slide gates in the past in locks with lifts as large as that at Locks and Dam No. 1. Instead, the model should be planned, constructed and tested for installation of 1) wheel gates throughout, or 2) reversed tainter gates throughout. The gates selected for further studies, however, could also be a combination of reversed tainter gates for the upstream filling end and wheel gates for the downstream emptying end of the culvert. The upstream filling gates should be relocated from the upper to the lower conduit level.

e) Lock model - It will be sufficient to build and test a model consisting of only one lock. However, both the landward and the riverward locks should be reproduced and tested in this single model. The floor elevations of the two locks are not the same. It is assumed that both locks will be rehabilitated.

f) Intakes - Each lock should be capable of operating independently of the other. The intakes should be of the manifold type and should be located on the sides of the land wall, I-wall, and river wall, not on the front of the I-wall bullnose.

g) Outlets - The outlet system for each lock should be capable of independent operation. Culvert outlets in the I-wall should therefore not be combined. The following outlet discharge systems should be tested:

1) Landwall lock - combined manifold system below lock for outflow from land wall and I-wall culverts.

2) Riverward lock - combined manifold system below lock for outflow from I-wall and river wall culverts.

3) Riverward lock - single manifold below lock for outflow from I-wall culvert, plus stilling basin on riverward side of lock for outflow from river wall culvert.

h) Stilling basin, river wall culvert discharge - Test the two alternatives proposed in Martin Nelson's report. Consideration should be given to determining the effects of the stilling basin discharge on the stage near the center island in the river.

i) Gate speed - The speed of the model gates should be variable so that optimum filling time (compatible with lock chamber turbulence) and acceptable emptying time can be determined.

j) Ports - Test design proposed by WES in 2nd Indorsement letter of 7 May 1975 (3.55 square feet in cross-sectional area, 19 ports in each culvert, spaced 13 feet center to center, staggered in opposite walls, and shaped as WES recommends). Also test Martin Nelson's recommended design (12 square feet in cross-sectional area, 10 ports in each culvert, spaced 26 feet center to center, staggered in opposite walls and shaped according to Martin Nelson's drawings Nos. M-R1-5/38 and M-R1-5/39, attached to his report).

COSTS OF VARIOUS HYDRAULIC MODIFIC

(January 1975 price levels)

ITEM	PLAN NO. 1	PLAN NO. 2	PL
Upstream Structures			
Upper guide wall	$ 171,000	$ 163,000	$
Land wall	39,000	38,000	
Intermediate wall	233,000	222,000	
River wall	—	—	
Concrete apron	93,000	84,000	
Downstream Structures			
Removal of concrete slab and apron	72,000	72,000	
Lower guide wall	242,000	227,000	[illegible]
Bottom laterals downstream of landward lock	201,000	195,000	[illegible]
Extension of intermediate wall	298,000	272,000	[illegible]
Bottom laterals downstream of riverward lock	210,000	204,000	[illegible]
Extension of river wall	—	—	
Hydraulic jump stilling basin east of river wall	—	—	
Bent discharge structure east of river wall	—	—	
Miscellaneous Modifications			
Lowering of filling culverts	398,000	391,000	3
Valves, bulkheads and embedded parts	254,000	254,000	2
New ports in lock walls	118,000	116,000	1
Lowering of culvert crown	178,000	173,000	1
New vents (5 - 3 ft. vents per culvert)	178,000	178,000	1
Subtotal Direct Cost	$2,685,000	$2,589,000[5]	$2,

1/ Discharge from the River Wall across the River.
2/ Discharge from the River Wall into a natural channel between the Center Island and Rockfill Dike.
3/ Discharge from the River Wall through the laterals downstream of River Lock.

4/ For incremental cost of downstream cof

5/ For incremental cost of downstream cof

TABLE D-1

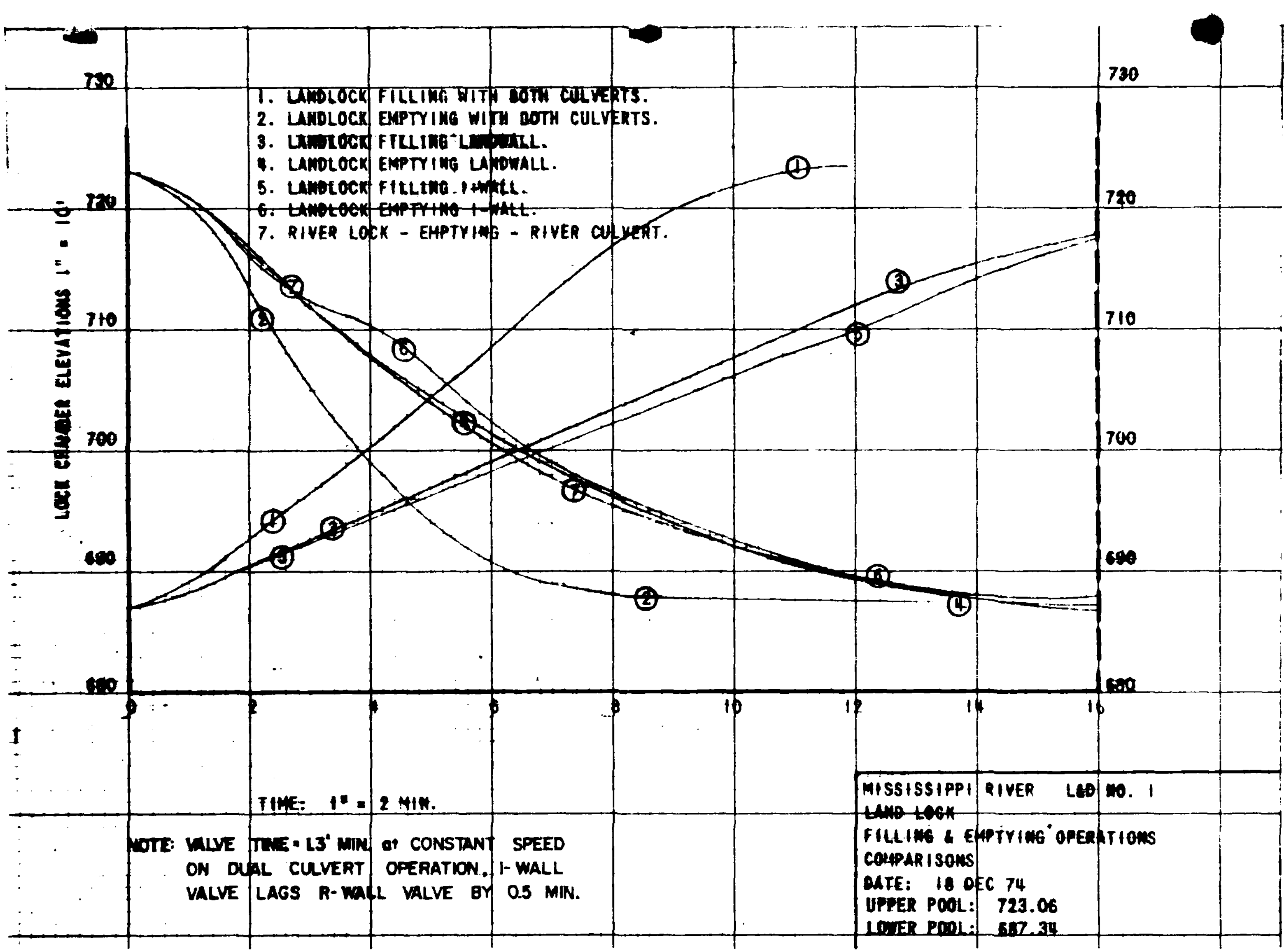
COSTS OF VARIOUS HYDRAULIC MODIFICATIONS
(January 1975 price levels)

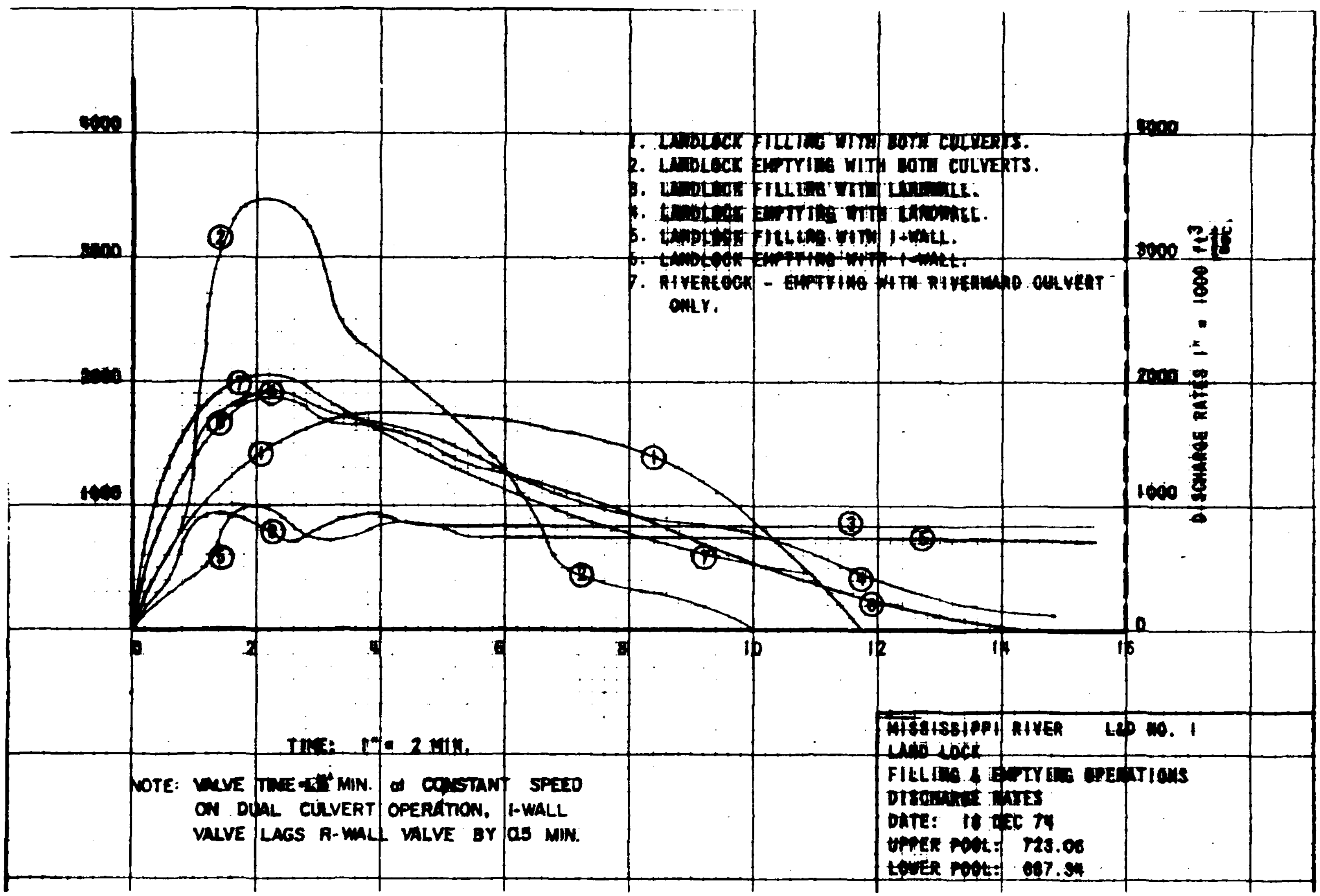
	PLAN NO.1	PLAN NO.2	PLAN NO.3	PLAN NO.4A 1/	PLAN NO.4B 2/	PLAN NO.4C 3/
	$ 171,000	$ 163,000	$ 158,000	$ 175,000	$ 175,000	$ 175,000
	39,000	38,000	38,000	39,000	39,000	39,000
	233,000	222,000	217,000	273,000	273,000	273,000
	—	—	—	237,000	237,000	237,000
	93,000	84,000	84,000	93,000	93,000	93,000
	72,000	72,000	72,000	72,000	72,000	72,000
	242,000	227,000	219,000	249,000	249,000	341,000
ck	201,000	195,000	189,000	207,000	207,000	261,000
	298,000	272,000	259,000	311,000	311,000	519,000
ock	210,000	204,000	199,000	215,000	215,000	325,000
	—	—	—	135,000	135,000	362,000
er wall	—	—	—	133,000	—	—
ll	—	—	—	—	209,000	—
	398,000	391,000	389,000	767,000	767,000	767,000
	254,000	254,000	254,000	509,000	509,000	509,000
	118,000	116,000	116,000	248,000	248,000	248,000
	178,000	173,000	168,000	364,000	364,000	364,000
	178,000	178,000	178,000	342,000	342,000	342,000
t Cost	$2,685,000	$2,589,000 5/	$2,540,000	$4,369,000 4/	$4,445,000 4/	$4,927,000 5/

4/ For incremental cost of downstream cofferdam add - $183,000.
5/ For incremental cost of downstream cofferdam add - $144,000.

annel

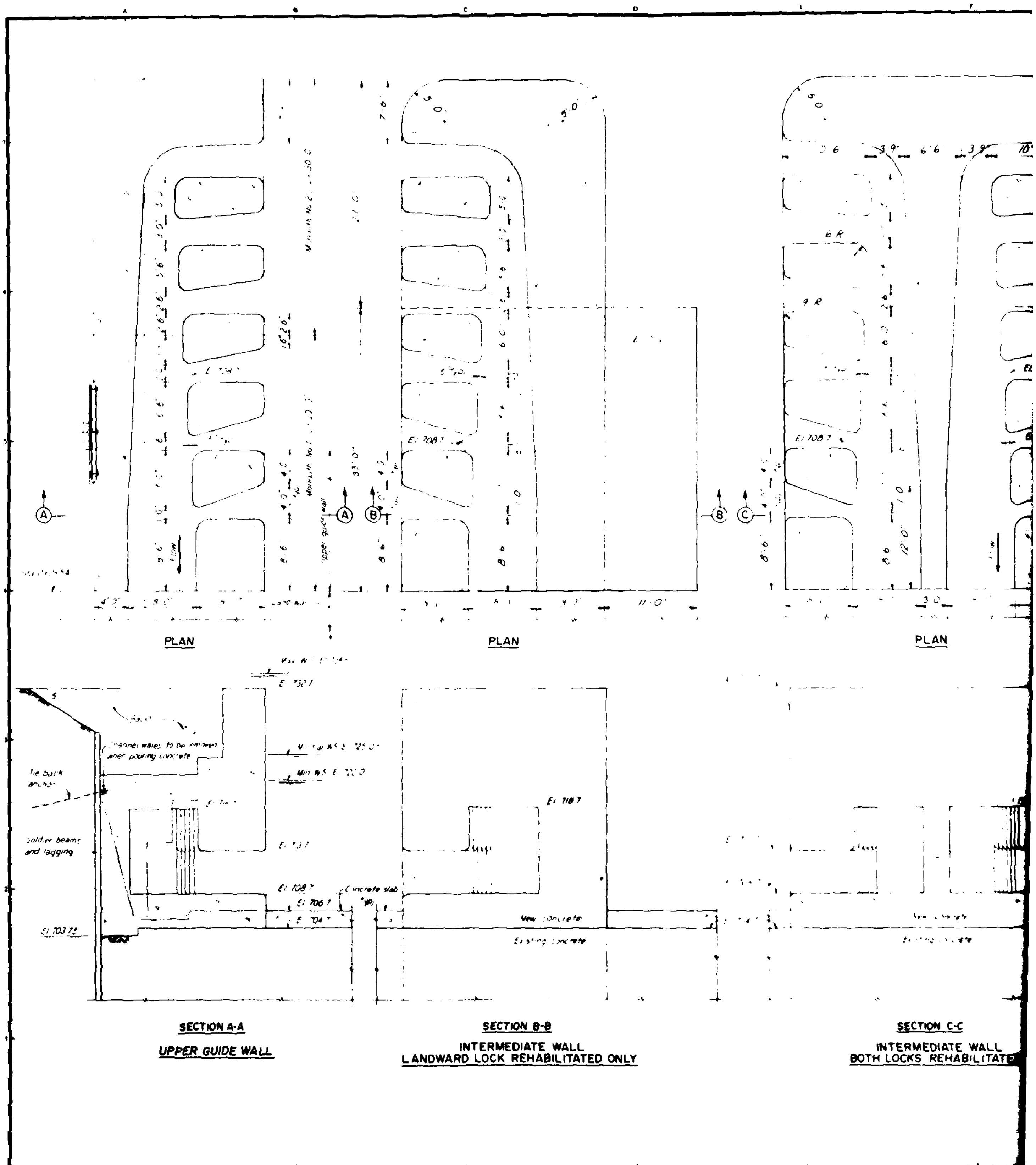
als


1. LANDLOCK FILLING WITH BOTH CULVERTS.
2. LANDLOCK EMPTYING WITH BOTH CULVERTS.
3. LANDLOCK FILLING LANDWALL.
4. LANDLOCK EMPTYING LANDWALL.
5. LANDLOCK FILLING I-WALL.
6. LANDLOCK EMPTYING I-WALL.
7. RIVER LOCK - EMPTYING - RIVER CULVERT.
LOCK CHAMBER ELEVATIONS 1" = 10'
730
720
710
700
690
680
0
2
4
6
8
10
12
14
16
TIME: 1" = 2 MIN.
NOTE: VALVE TIME = 1.3' MIN. at CONSTANT SPEED
ON DUAL CULVERT OPERATION, I-WALL
VALVE LAGS R-WALL VALVE BY 0.5 MIN.
MISSISSIPPI RIVER L&D NO. 1
LAND LOCK
FILLING & EMPTYING OPERATIONS
COMPARISONS
DATE: 18 DEC 74
UPPER POOL: 723.06
LOWER POOL: 687.34

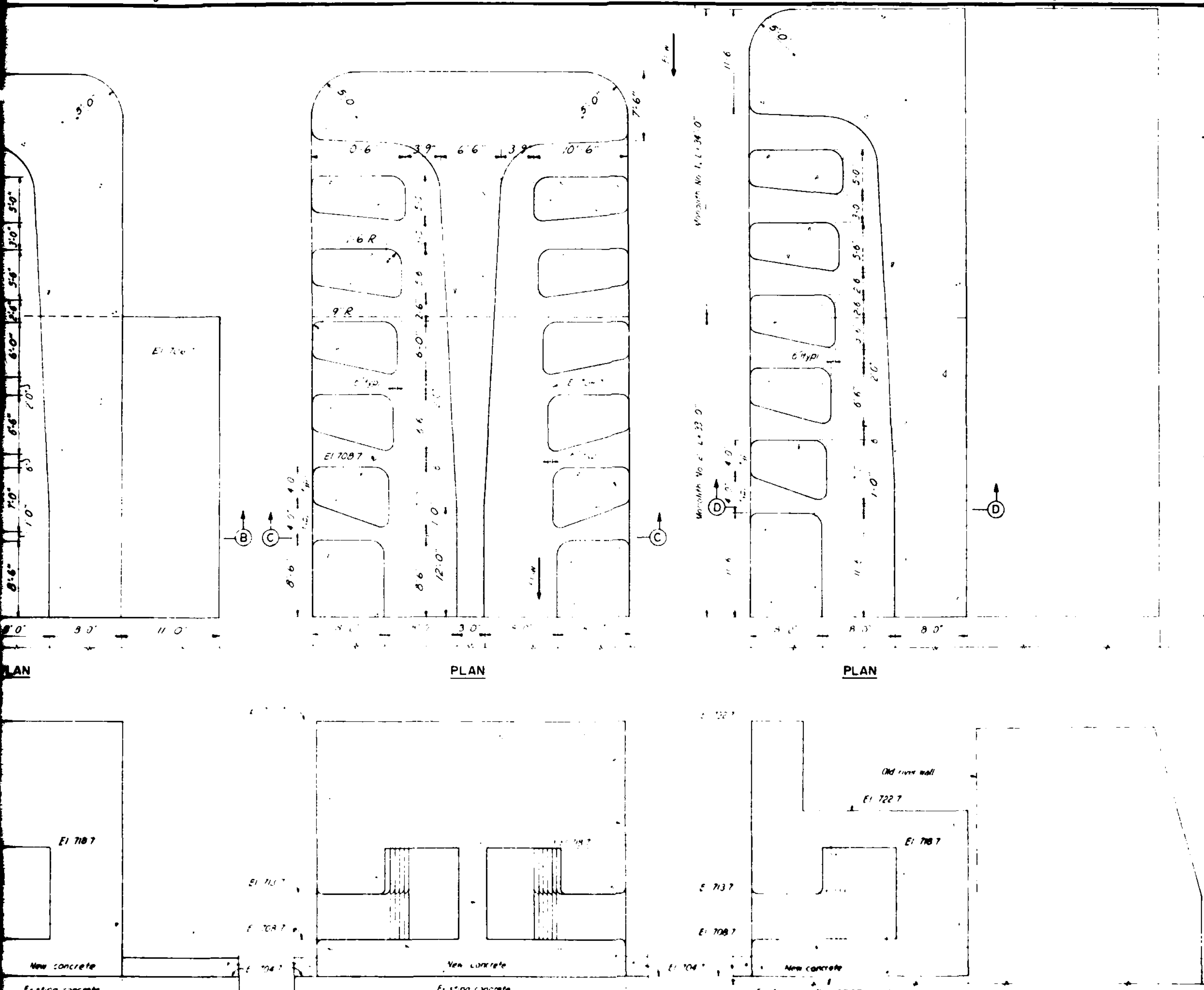


4000
3000
2000
1000
0
DISCHARGE RATES 1" = 1000 ft³/sec
0
2
4
6
8
10
12
14
16
1. LANDLOCK FILLING WITH BOTH CULVERTS.
2. LANDLOCK EMPTYING WITH BOTH CULVERTS.
3. LANDLOCK FILLING WITH LANDWALL.
4. LANDLOCK EMPTYING WITH LANDWALL.
5. LANDLOCK FILLING WITH I-WALL.
6. LANDLOCK EMPTYING WITH I-WALL.
7. RIVERLOCK - EMPTYING WITH RIVERWARD CULVERT ONLY.
TIME: 1" = 2 MIN.
NOTE: VALVE TIME = MIN. at CONSTANT SPEED ON DUAL CULVERT OPERATION, I-WALL VALVE LAGS R-WALL VALVE BY 0.5 MIN.
MISSISSIPPI RIVER L&D NO. 1
LAND LOCK
FILLING & EMPTYING OPERATIONS
DISCHARGE RATES
DATE: 18 DEC 74
UPPER POOL: 723.06
LOWER POOL: 687.34

PLATE D-2



PLAN
PLAN
PLAN
Channel wales to be removed when pouring concrete
Tie back anchor
Soldier beams and lagging
El. 703.7±
El. 708.7
El. 706.7
Concrete slab
El. 118.7
New concrete
Existing concrete
SECTION A-A
UPPER GUIDE WALL
SECTION B-B
INTERMEDIATE WALL
LANDWARD LOCK REHABILITATED ONLY
SECTION C-C
INTERMEDIATE WALL
BOTH LOCKS REHABILITATE



PLAN

PLAN

PLAN

SECTION D-D

RIVER WALL

Scale 1" = 5' 0"

0 5 10

...TION B-B

...EDIATE WALL
...K REHABILITATED ONLY

SECTION C-C

INTERMEDIATE WALL
BOTH LOCKS REHABILITATED

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT CORPS OF ENGINEERS
ST. PAUL, MINNESOTA

M.E.N., V.T.
R.E.
R.N.M.

STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO. 1

HYDRAULIC IMPROVEMENTS
INTAKE MANIFOLDS

SUBMITTED BY

APPROVED

DATE
MARCH 1975

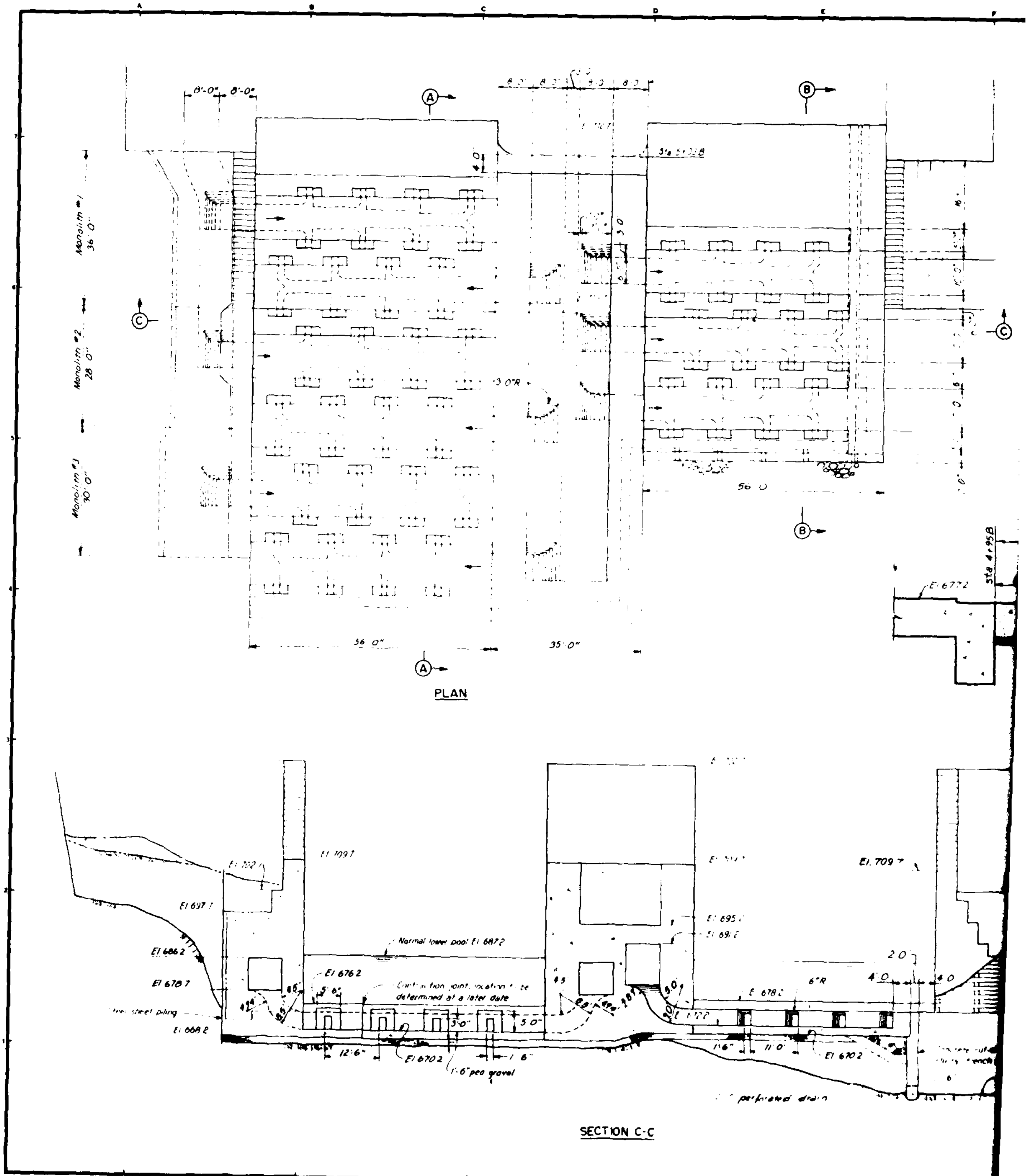
SCALE 1" = 5' 0"

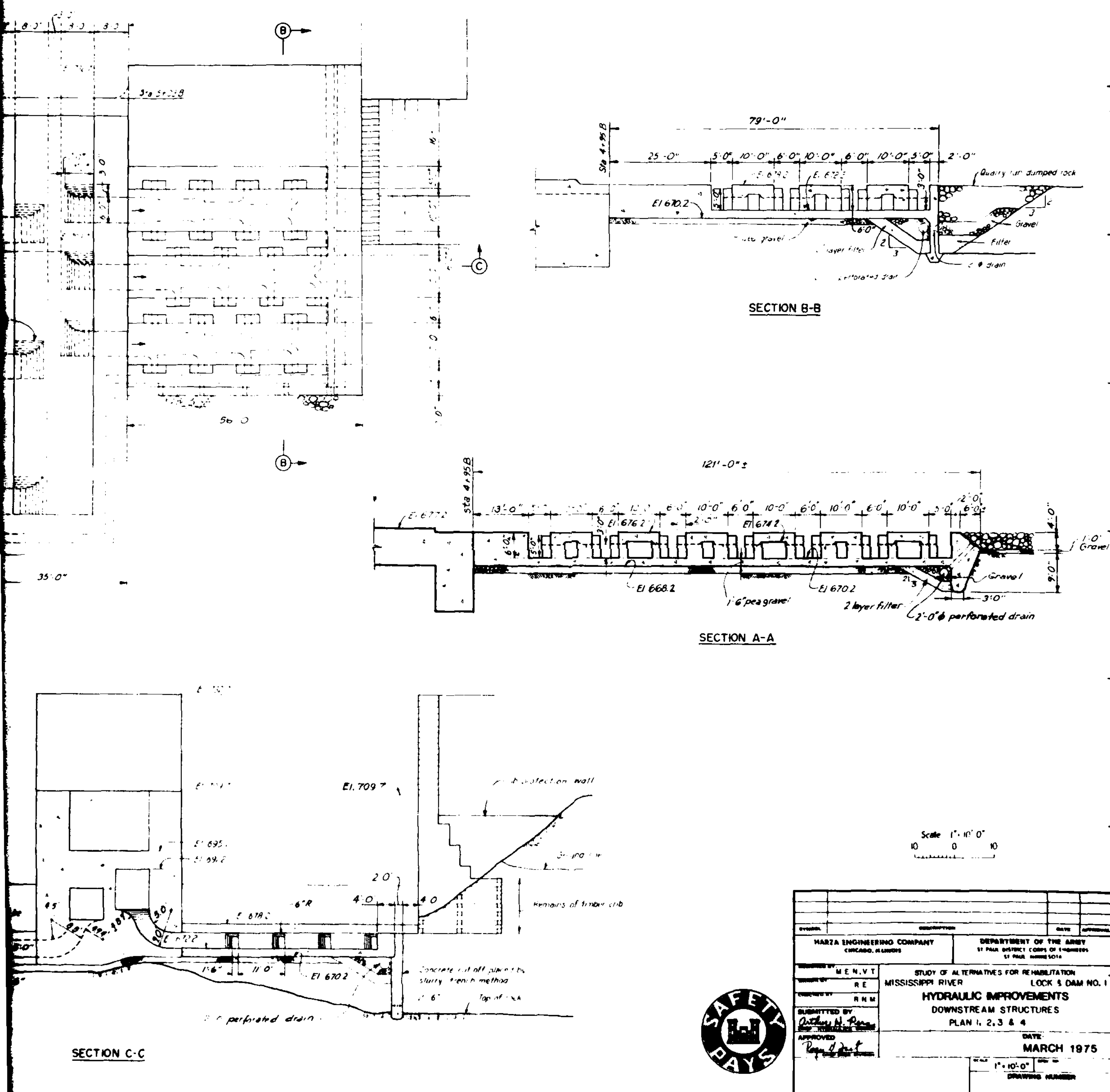
DRAWING NUMBER

SHEET OF

PLATE D-3

2


8'-0"
8'-0"
Monolith #1 36'0"
Monolith #2 28'0"
Monolith #3 30'0"
4'0
3'0"R
56'0"
35'0"
56'0
Sta 4+95B
El 677.2
PLAN
El 709.7
El 702
El 697.7
El 686.2
El 678.7
El 668.8
Normal lower pool El 687.2
El 676.2
Construction joint, location to be determined at a later date
El 695
El 678
El 670.2
1'-6" pea gravel
12'-6"
5'-6"
3'-0"
5'0"
1'6"
11'0"
4'0
2'0
6"R
perforated drain
SECTION C-C



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CHICAGO, ILLINOIS

DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT CORPS OF ENGINEERS
ST. PAUL, MINNESOTA

STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER — LOCK & DAM NO. 1
HYDRAULIC IMPROVEMENTS
DOWNSTREAM STRUCTURES
PLAN 1, 2, 3 & 4

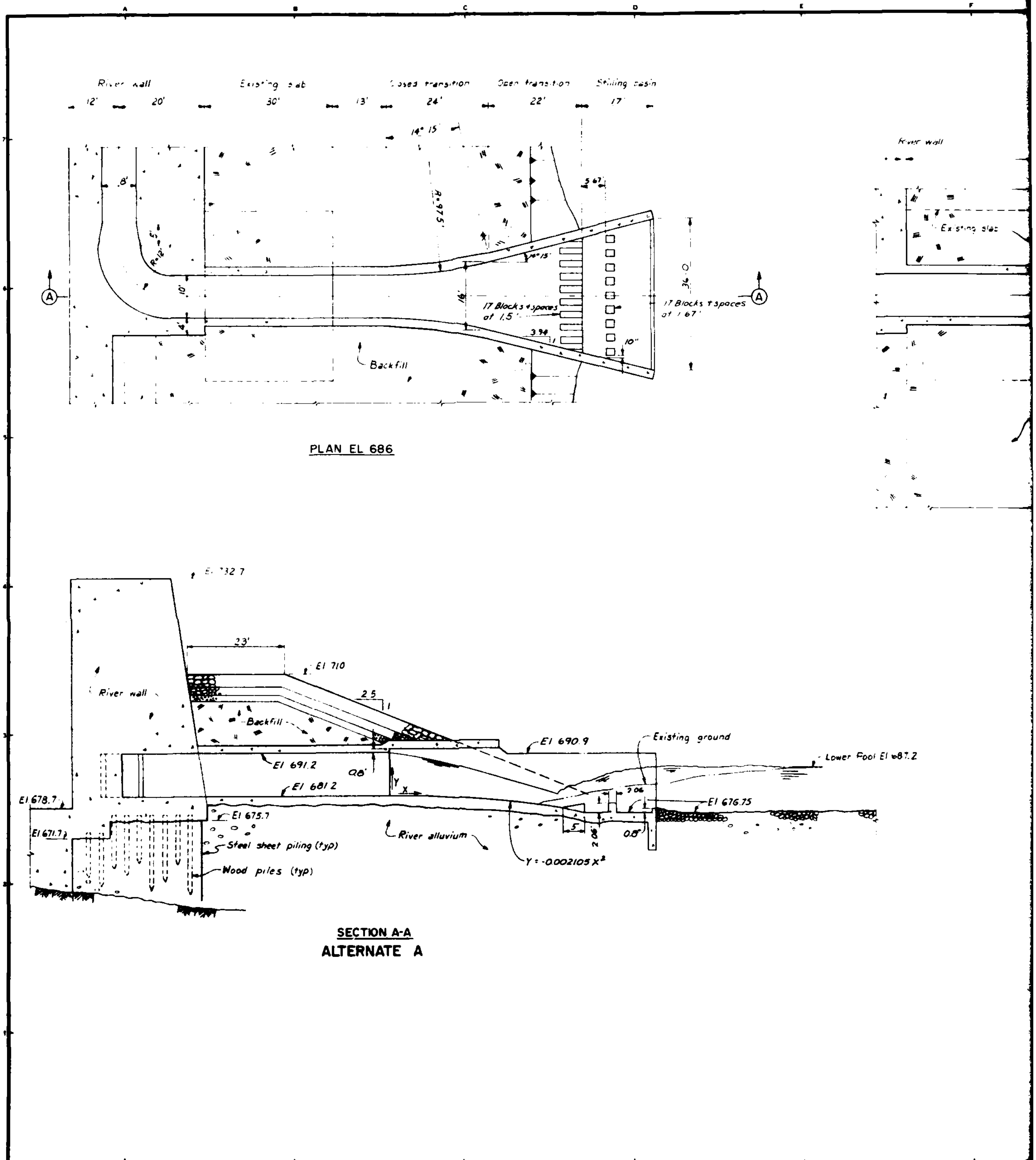
DATE: MARCH 1975

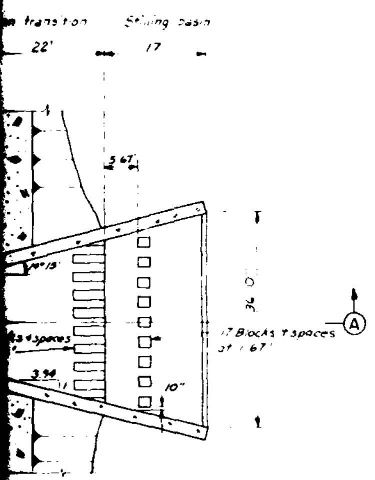
SCALE: 1" = 10'-0"

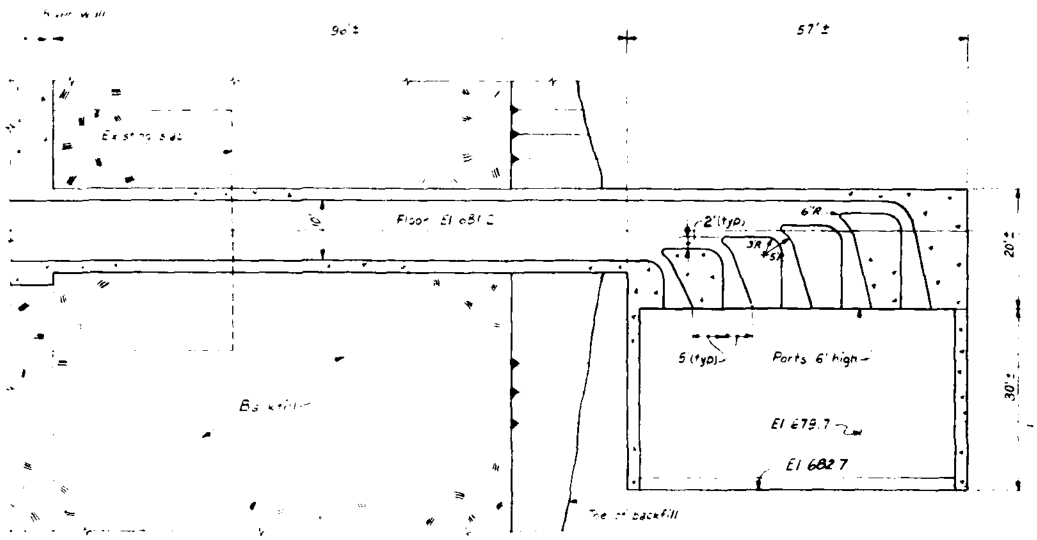
DRAWING NUMBER

SHEET OF

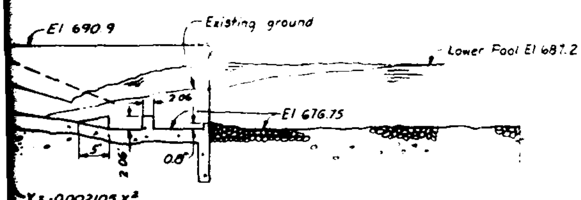
PLATE D-4


River wall
Existing slab
Closed transition
Open transition
Stilling basin
12'
20'
30'
13'
24'
22'
17'
14° 15'
R=97.5'
5.67'
36'-0"
8'
R=12'
10'
16'
17 Blocks + spaces at 1.5'
17 Blocks + spaces at 1.67'
3.94
10"
Backfill
Existing slab
River wall
PLAN EL 686
El 732.7
23'
El 710
2.5
1
River wall
Backfill
El 690.9
Existing ground
Lower Pool El 687.2
El 691.2
0.8'
El 681.2
El 678.7
El 675.7
El 671.7
El 676.75
Steel sheet piling (typ)
Wood piles (typ)
River alluvium
Y = -0.002105 X²
0.8'
SECTION A-A
ALTERNATE A





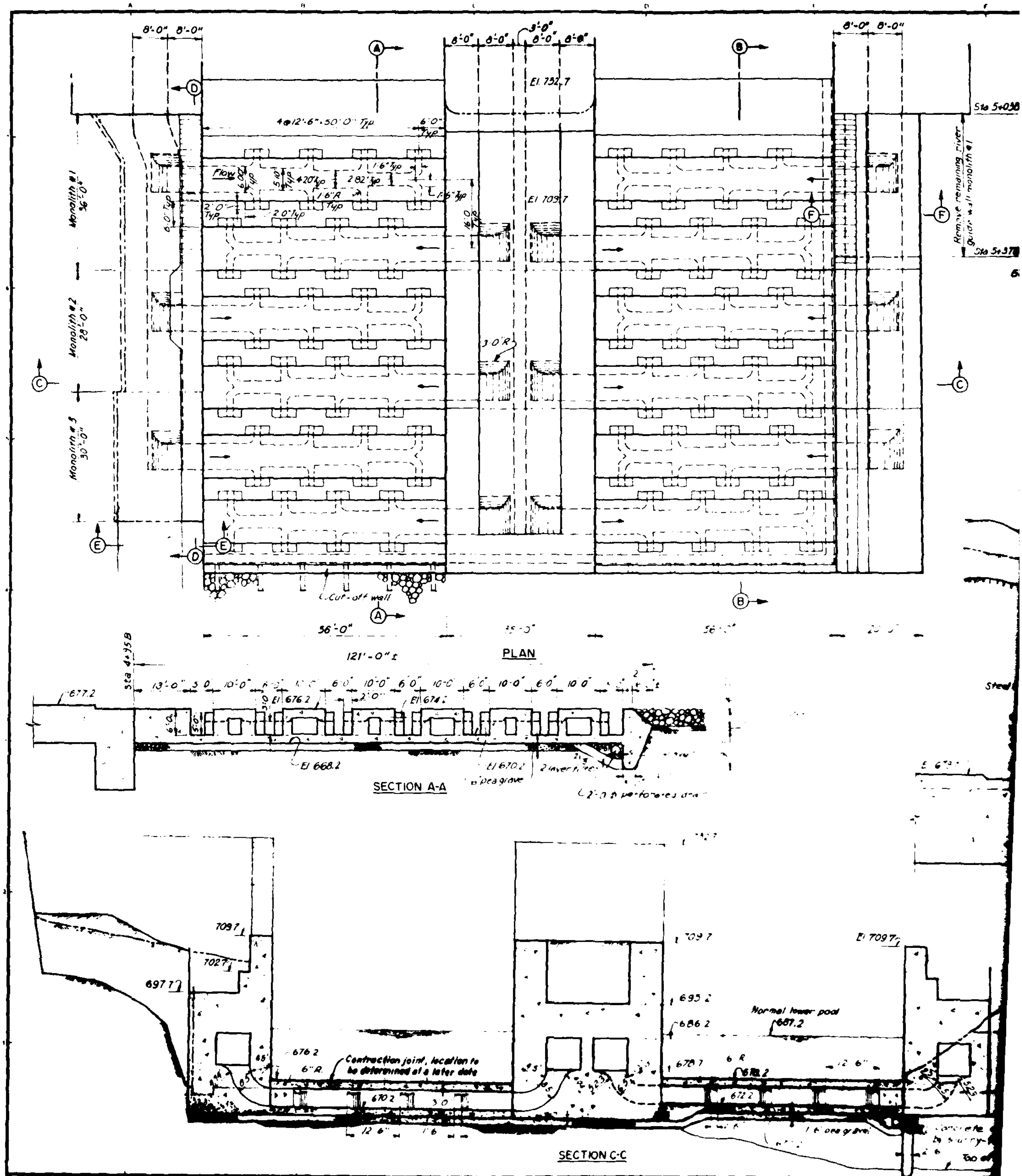
PLAN - EL 686
ALTERNATE B



M E N
T E G
V T

STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER
LOCK & DAM NO 1
HYDRAULIC IMPROVEMENTS
RIVER WALL OUTLET STRUCTURES
PLAN 4
MARCH 1975

PLATE D-5



8'-0" 8'-0"
8'-0" 8'-0" 3'-0" 8'-0" 8'-0"
8'-0" 8'-0"
El 732.7
Sta 5+038
4@12'-6"=50'-0" Typ
6'-0" Typ
Flow
El 709.7
Monolith #1 56'-0"
Monolith #2 28'-0"
Monolith #3 30'-0"
Remove remaining river guide wall monolith #1
3'-0"R
Cut-off wall
56'-0"
121'-0"±
PLAN
Sta 4+95B
677.2
El 676.2
El 674.2
El 668.2
El 670.2
2 layer filter
pea gravel
SECTION A-A
709.7
702.7
697.7
676.2
Contraction joint, location to be determined at a later date
670.2
709.7
695.2
686.2
678.7
Normal lower pool 687.2
El 709.7
672.2
pea gravel
SECTION C-C

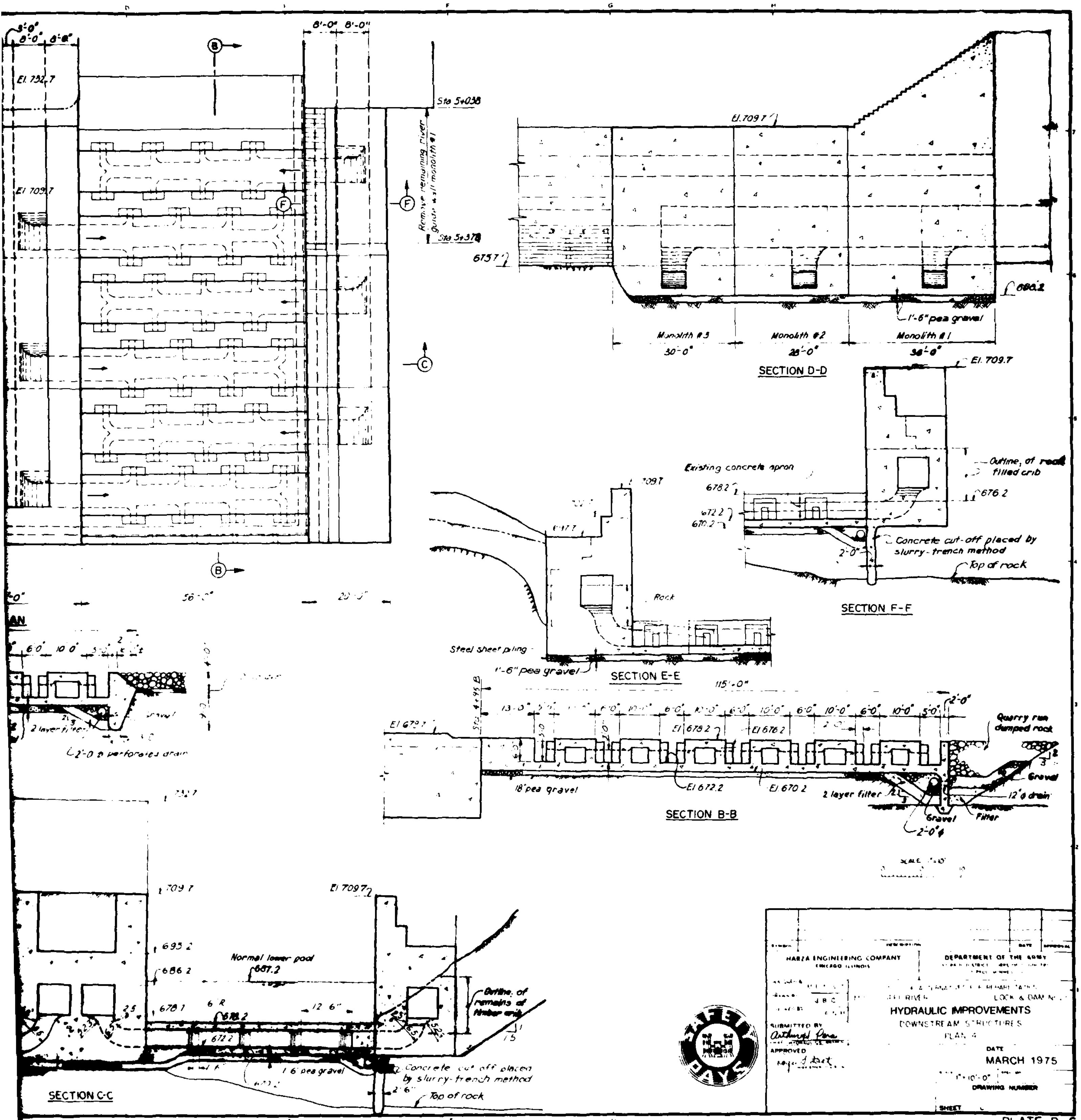
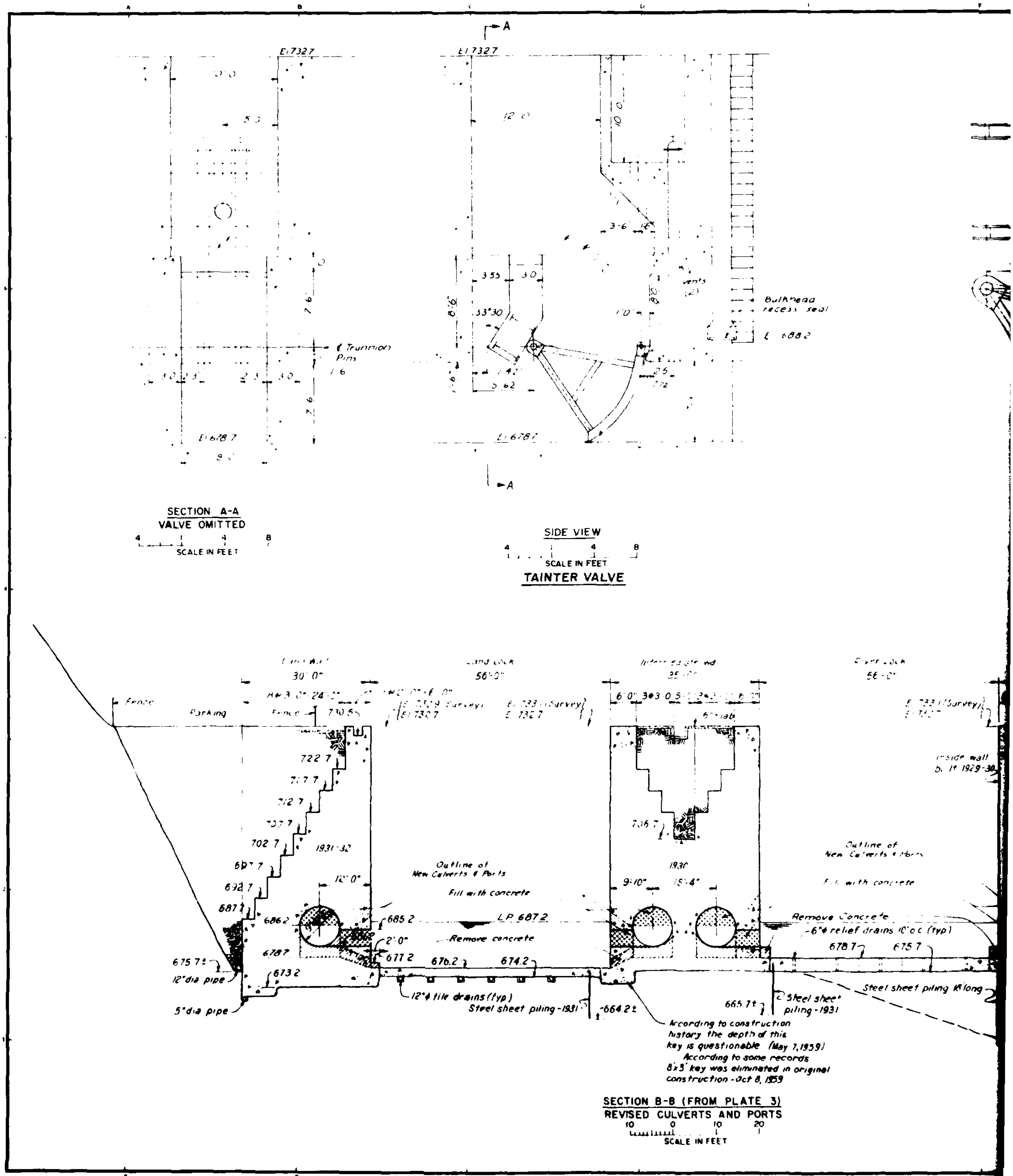

SECTION D-D
Monolith #3
Monolith #2
Monolith #1
1'-6" pea gravel
SECTION F-F
Existing concrete apron
Outline of rock filled crib
Concrete cut-off placed by slurry-trench method
Top of rock
SECTION E-E
Steel sheet piling
1'-6" pea gravel
Rock
SECTION B-B
Quarry run dumped rock
18" pea gravel
2 layer filter
Gravel
Filter
12" drain
SECTION C-C
Normal lower pool
Outline of remains of timber crib
1'6" pea gravel
Concrete cut off placed by slurry-trench method
Top of rock
Remove remaining river guide wall monolith #1
HARZA ENGINEERING COMPANY
DEPARTMENT OF THE ARMY
HYDRAULIC IMPROVEMENTS
DOWNSTREAM STRUCTURES
SUBMITTED BY
APPROVED
MARCH 1975
DRAWING NUMBER

PLATE D-6

1

2



SECTION A-A
VALVE OMITTED
SCALE IN FEET
SIDE VIEW
SCALE IN FEET
TAINTER VALVE
Bulkhead recess seal
Fence
Parking
Outline of New Culverts & Ports
Fill with concrete
Remove concrete
Remove Concrete
6"ϕ relief drains 10' o.c. (typ)
12" dia pipe
5" dia pipe
12"ϕ tile drains (typ)
Steel sheet piling - 1931
Steel sheet piling - 1931
Steel sheet piling 18' long
According to construction history the depth of this key is questionable (May 7, 1959)
According to some records 8'x3' key was eliminated in original construction - Oct 8, 1959
SECTION B-B (FROM PLATE 3)
REVISED CULVERTS AND PORTS
SCALE IN FEET

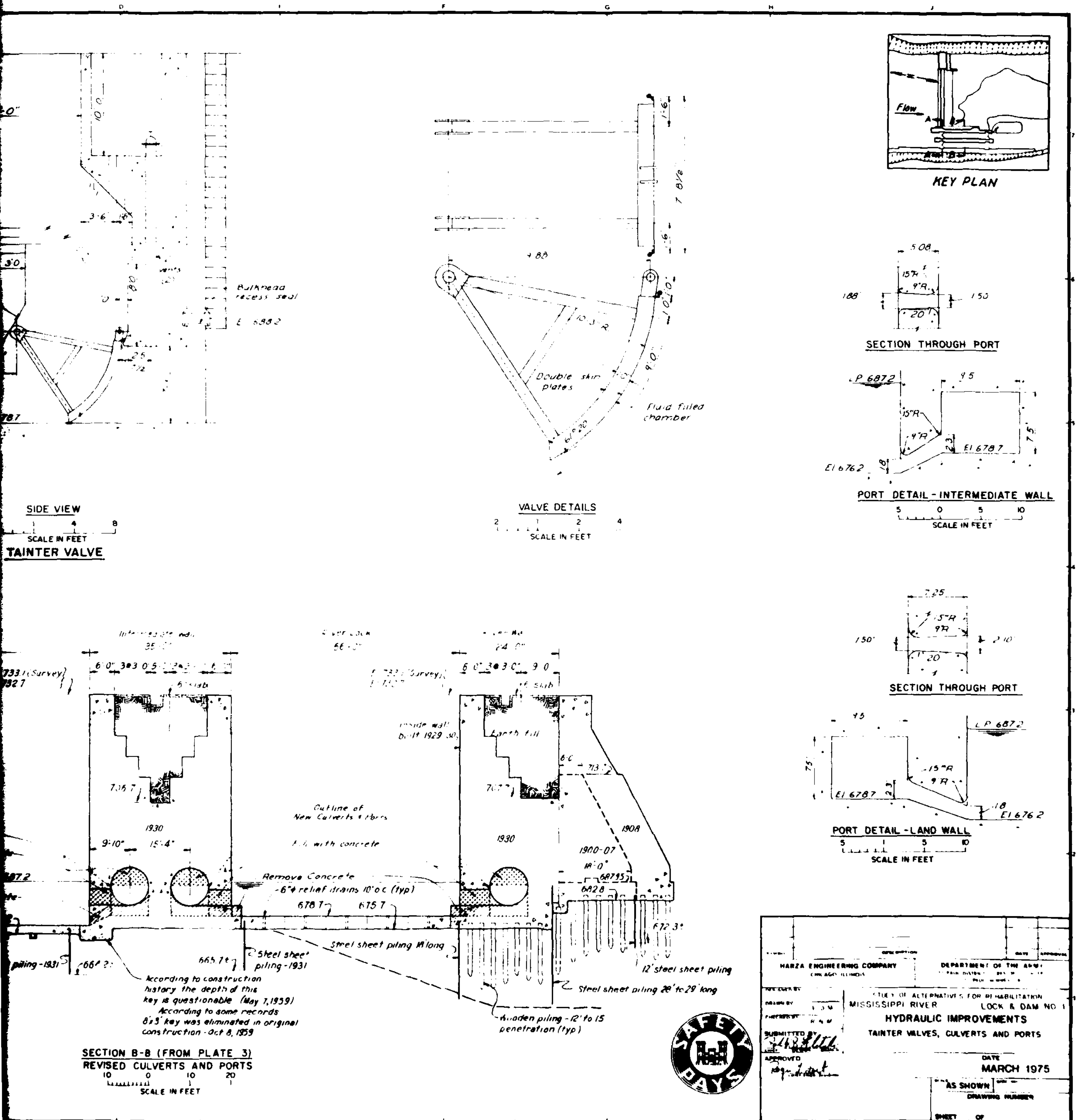

KEY PLAN
Flow
SECTION THROUGH PORT
PORT DETAIL - INTERMEDIATE WALL
SCALE IN FEET
SECTION THROUGH PORT
PORT DETAIL - LAND WALL
SCALE IN FEET
SIDE VIEW
SCALE IN FEET
TAINTER VALVE
Bulkhead recess seal
VALVE DETAILS
SCALE IN FEET
Double skin plates
Fluid filled chamber
Outline of New Culverts & Ports
Fill with concrete
Remove Concrete
6"φ relief drains 10' o.c. (typ)
Steel sheet piling 16' long
Steel sheet piling - 1931
According to construction history the depth of this key is questionable (May 7, 1959)
According to some records 8x3' key was eliminated in original construction - Oct 8, 1959
Inside wall built 1929-30
Earth fill
12' steel sheet piling
Steel sheet piling 28' to 29' long
Wooden piling - 12' to 15' penetration (typ)
SECTION B-B (FROM PLATE 3)
REVISED CULVERTS AND PORTS
SCALE IN FEET
HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS
DEPARTMENT OF THE ARMY
STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER
LOCK & DAM NO. 1
HYDRAULIC IMPROVEMENTS
TAINTER VALVES, CULVERTS AND PORTS
SUBMITTED BY
APPROVED
DATE
MARCH 1975
AS SHOWN
DRAWING NUMBER
SHEET OF
SAFETY PAYS

PLATE D-7

SUPPLEMENT TO APPENDIX D

FILLING, VENTING AND EMPTYING SYSTEMS

Hydraulic Problems and Recommended
Solutions at Lock and Dam No. 1,
Mississippi River

prepared by
Martin E. Nelson

for

Department of the Army
St. Paul District, Corps of Engineers
1210 U. S. Post Office and Custom House
St. Paul, Minnesota 55101
January 1974

Contract No. DACW37-74-C-0038

INTRODUCTION

Three major hydraulic problems exist at Lock and Dam No. 1; air entrapment in the culverts of the filling and emptying system, excessive turbulence in the lock chamber during filling, and hazardous conditions downstream of the locks during emptying operations.

The crown of the filling and emptying culverts in the walls of the lock chambers is located 3.5 feet above normal lower pool level, elevation 687.2. This elevation and all elevations to follow in this report are referenced to 1912 MSL datum. When lower pool level is below the crown of the culverts, elevation 690.7, a layer of air will be resident in the culverts, along the chamber as well as in the drop section of the culverts immediately downstream of the filling valves.

When the upstream valves are opened, the gush of water compresses the air in the culvert causing destructive forces on the downstream valves. Damage occurs to the roller train rollers and the roller train guide plates. Loud noises can be heard and vibration of the lock walls can be felt. Air also enters the tunnels through the upper bulkhead recesses. The existing venting system is inadequate. This system extracts air from near the middle of the tunnels and exhausts it in the downstream valve pits. The spray settles on the machinery and freezes during the winter. Also, the downstream valves do not seal properly when closed against a head. The roller trains and guide rails are worn out.

When the lock chamber is being filled, jets issuing from the filling ports flow directly across the lock chamber. During the initial part of filling, these jets exert strong forces tending to move barges away from the mooring wall resuling in large hawser forces. To prevent this, the lockmaster must begin filling the lock chamber from the opposite wall. When the water level is sufficiently above the ports, he can begin admitting water from the side to which the barge is moored. This operation significantly increases locking time. When pleasure craft are locked, turbulence is so strong that they must be moored at the upper end of the lock chamber to reduce the danger of capsizing.

During the emptying operations the discharge from the culverts in the I-wall flows directly into the approach channel downstream from the lock. This discharge is not baffled and generates large waves. Consequently, boats must remain more than 400 feet downstream to avoid being swamped.

The contractor has studied these hydraulic problems and his recommended solutions are presented in the remainder of this report.

THE AIR ENTRAINMENT PROBLEM

One of the principle causes of severe damage to the valves in Lock No. 1 and excessive turbulence in the chambers during filling operations is the release of air entrained by the flow in the culvert system. Two faults in the design of the system contribute to this problem:

1. The roof of the culverts is located at elevation 690.7, 3.5 feet above normal lower pool level, elevation 687.2. Thus, when the lock is empty and tailwater level is below elevation 690.7, a pocket of air resident along the roof of the culverts will be trapped and compressed during filling operations.

2. Between the intakes and the valves the culverts are located at a higher elevation (elevation 708.7) than along the port manifold section (681.2). This drop of 27.5 feet occurs on a slope of about 60° immediately downstream from the valve. At the beginning of filling operations, this section of the culvert is also filled with air between tailwater level and the valve, and a great deal of air is entrained by the hydraulic jump which forms in this section.

Venting System

During filling operations the resident air in the culvert will be compressed into one or more large bubbles which migrate back and forth along the roof of the culvert, locally restricting the flow, until an exit is found at a vent, a bulkhead recess or a valve shaft. The existing vents, being too small to expel the entire bubble, allow only a small amount of air to escape each time a bubble passes by. Blasts of air and water are released periodically, during nearly the entire filling operation. Only a small part of the resident air, if any, will be entrained by the flow and released into the lock chambers. To be completely effective the vent system should be enlarged. Several vertical vents should be connected to a larger gallery or header where the air and water can separate, the water returning to the culvert through inactive vents and the air expelled to the atmosphere through hooded vertical stacks or into the bulkhead recesses at either end of the culvert, but not into the valve recesses.

Lowering Conduit Roof

The air trapped in the culverts, except in the drop sections, could be excluded by filling the void with concrete down to normal lower pool level (elevation 687.2). This would, of course, reduce the area of each culvert from 70.8 square feet to 48.1 square feet, a reduction of 22.7 square feet. Without compensating improvements in the system, the filling time would then by correspondingly increased. Means of compensating for the loss of flow area in the culverts, if this revision is adopted, will be discussed in a later section of this report.

In modern locks it is customary to submerge the culverts as far below the lower pool level as practical in order to avoid negative pressures in the area immediately below the filling valves. It would be impractical to submerge the culverts in Lock No. 1 sufficiently to provide this protection and some negative pressure and slight air entrainment during the valve opening period may have to be tolerated.

If the roof of the culverts is lowered sufficiently to exclude the layer of air trapped above normal lower pool level as recommended, the air problem will not necessarily be completely eliminated and the present venting system should not be abandoned. Because of the scant submergence of the culvert, low pressures will prevail immediately below the filling valves during filling operations, tending to draw air through any unsealed openings nearby, such as at the upper valve seal and adjacent bulkhead recess. It appears that the present system does not facilitate separation of the air and water except when both are blown out at a common outlet. Therefore, it is recommended that additional risers be provided, particularly near the valves and that the air be released from the header at intervals through vertical hooded stacks, as well as at the present terminals in the bulkhead recesses.

Should it be deemed impractical or inadvisable to lower the crown of the wall culverts by means of a concrete filler 3.5 feet deep, the resident air will have to be removed by a more efficient venting system than that presently in use. A suggested modification is shown in Drawing 5/38. Since adequate design data for such a device are not available the dimensions of risers and headers, as shown, are only approximate. Air can escape from the culvert only when a compressed bubble glides past the lower opening of a riser. The air rising in the stack will carry with it the column of water above. Obviously, the volume of air exhausted will be proportional to the cross sectional area of the stack, but so will also the volume of water simultaneously pumped out. The purpose of the header is to allow separation of the water and air and to permit the water to return to the culvert through inactive risers, while the dry air is released to the atmosphere through the smaller hooded vertical vents.

Culvert Drop Below Existing Valves

The problem created in the vertical drop below the valves cannot be eliminated by venting. When a valve is being opened to make a filling operation, a hydraulic jump occurs in the drop and, as in all instances when this phenomenon takes place, a large amount of air is entrained in the flow in the form of tiny bubbles. Very little of this air will be released in the culverts, but it will be carried into the lock chamber where it appears as a milky mixture of air and water which augments the turbulence caused by the submerged jets issuing from the wall ports. The hydraulic jump in the culvert is objectionable for another important reason. The jump breaks the flow column and creates two independent flow systems, until the water level in the lock chamber rises sufficiently to drown the jump out. Flow between the intakes and the valves is generated by the effective head between these two points, while flow in the main culvert and through the port manifold is generated by the pressure difference between the culvert and the lock chamber. A substantial part of the apparent head between the upper pool and lock chamber water level is lost in the flow discontinuity caused by the hydraulic jump.

Hydraulic Tests

In preliminary studies of the hydraulic system of Lock No. 1, it was found that the average discharge coefficient was only about 0.45 based on data obtained from lock operation personnel. For clarification of this unusually low coefficient, in comparison with that of other similar systems, three filling tests were made 8 December 1973 under the following conditions.

Test No.	Upper Pool	Lock Chamber	Head feet	Filling Time Minutes	Ave. C
1	722.7	687.8	34.9	10.1	.41
2	722.7	691.1	31.6	8.9	.45
3	722.7	694.5	28.2	8.2	.46

It will be noted that the initial water levels in the lock chamber relative to the top of the culvert (690.7) were respectively; -2.9 ft., +0.4 ft., and +3.8 ft. In tests 2 and 3, readings were taken on the water level gage in the lock chamber at 1 minute time intervals. From this data, filling curves were constructed and instantaneous rates of discharge and coefficients of discharge were computed. The equation used in computing overall lock coefficients is as follows:

$$T - kt_V = \frac{2 A_L (\sqrt{H + d} - \sqrt{d})}{C A_C \sqrt{2g}}$$

T = total filling time, seconds
t_V = valve opening time, seconds
k = valve factor (0.5)
A_L = area of water surface in lock, square feet
H = lift, feet
d = overtravel, ft. (1.0')
A_C = cross sectional area of culverts, square feet
C = lock coefficient
g = 32.2 ft/sec^2

The results of these computations are shown in Figure 1 (see Drawing 5/39). The slope of the filling curve is precisely the same in both tests and the rate of filling is identical until the water level in the lock chamber reaches approximately elevation 713.6. This indicates that the flow in this part of the filling operation is controlled solely by the culvert system upstream from the drop section. As the increasing back pressure supresses the jump, the curve assumes the characteristic shape of usual filling curves.

As indicated by the graph, the coefficient of discharge rises gradually from 0.31 and 0.32 to approximate constant values of 0.55 and 0.60, respectively, when the jump has been completely drowned out. In observing conditions in the lock chamber it appeared that the inflow of entrained air remained constant until the transition from dual control to continuous flow, indicating that the jump had disappeared.

For comparison of the filling characteristics of Lock No. 1 with those of a typical modern lock, data similar to those shown in Figure 1 obtained in a model test of the Eisenhower Lock are shown in Figure 2 (see Drawing 5/39). It will be noted that the hydraulic coefficient for the filling system after the valves are fully open in the Eisenhower Lock remains practically constant at 0.75 throughout the rest of the operation. Also the rate of filling decreases uniformly after the valves are fully open, indicating that the hydraulic system is operating under a gradually falling head. In tests 2 and 3 at Lock No. 1, the filling rate remained constant several minutes due to the discontinuity in flow caused by the hydraulic jump in the drop section.

Relocation of Filling Valves

It appears that the problem of air entrainment and poor hydraulic efficiency attributable to the presence of the hydraulic jump in the drop section of the culvert can be eliminated only by locating the filling valves downstream of the drop. This solution is recommended and a suggested modification is shown on Section A-A of Drawing 5/38. Another possibility might be to relocate the filling valves at the upstream end of the horizontal part of the culvert in the existinq system. This would eliminate the need for having to reconstruct the vertical part of the conduit. However, the valve position should probably be determined by a cost analysis. Tainter valves, as shown, are believed to be operationally most satisfactory, but there may be compelling reasons for adopting vertical lift or butterfly valves and these types should also be given consideration.

If the filling valves are lowered to the bottom of the main culvert (elevation 681.2) thus eliminating the excessive head loss induced by the hydraulic jump which now occurs in the drop section, the overall lock coefficient should increase from 0.45, as found in tests 2 and 3, to about 0.60, or possibly to 0.65 if the port entrances in the culvert system are rounded. These increases in the hydraulic coefficient will result in substantial improvement in the rate of filling and would compensate to a large degree for the loss of capacity from reduction of the culvert area if the roof is lowered to normal lower pool level. The following tabulation indicates the effect of these two major changes in the filling system under consideration in respect to the operation characteristics observed in prototype tests of Lock No. 1 on 8 December 1973.

		Valve at Elev 708.7				Valve at Elev 681.2			
		Crown El. 690.7		Crown El. 687.2		Crown El. 690.7		Crown El. 687.2	
Test No.	Lift ft.	C	Fill time min.	C	Fill time min.	C	Fill time min.	C	Fill time min.
1	34.9	.41	10.1	.41	14.9	.60	7.25	.65	9.5
2	31.6	.45	8.9	.45	13.1	.60	6.9	.65	9.0
3	28.2	.46	8.2	.46	12.1	.60	6.6	.65	8.5

If it is determined preferable to locate the filling valves downstream of the drop section in the culvert, it will be necessary to close one or two ports at the upstream end of the lock chamber manifold and to add compensating ports at the downstream end of the manifold at Sta 3+89.7 in order to preserve the present capacity of the filling system.

Emptying Valves

It is reported that large forces are exerted on the lower valves due to compression of the air in the culverts during filling operations, causing a great deal of wear and damage to the valves. The compression of the air per se probably is not the cause of these forces. A compressed bubble may suddenly be shifted or escape through a vent and a slug of water rushing into the cavity produces an impact accompanied by a loud bang analogous to a hammer blow that is transmitted as a shock wave impinging against the valve. The resulting water hammer effect can be very destructive. It is believed that this condition will be largely corrected if the resident air is eliminated by lowering the upstream valves and lowering the roof of the main culvert or by improving the capacity and functioning of the vent system.

Obviously, the downstream valves are now badly in need of repair or replacement. If the valves can not be made operational by extensive repair, at a reasonable cost, and with reasonable assurance of several years of continued service, they will have to be replaced. Tainter valves are used almost exclusively in modern locks because experience has demonstrated that they provide dependable service, simplicity of operation and ease of maintenance. In lock hydraulic systems the reversed tainter valve has one advantage not generally attainable with butterfly or vertical slide valves, in that the valve recess serves as a surge chamber where pressure waves due to sudden closure or other causes can be dissipated, thus avoiding severe impact forces on the valve. However, the cost of construction and maintenance should probably be the governing criterion in selecting the type of valve to be adopted for the emptying system. Should tainter valves be selected the design and installation would be practically the same as for the filling valves.

LOCK CHAMBER TURBULENCE

Existing Ports

The excessive turbulence which takes place in the lock chamber during filling operations could be reduced appreciably by the elimination of entrained air in the flow. However, the turbulence results to a large extent also from inadequate dissipation of the energy inherent in the jets of water issuing from the lock chamber ports in the filling system. At normal lower pool level the ports are submerged only 2.1 feet below the water surface. Furthermore, the ports in one lock wall are located directly opposite those in the other wall. The opposing jets block each other at the centerline of the lock, creating strong boils and intense turbulence throughout the mixing area. In narrow locks, such as No. 1, the turbulence is more severe than in wider locks under otherwise similar circumstances. Turbulence of this nature is undoubtedly more hazardous to traffic, particularly to small craft, than that caused by entrained air. Because of their shallow submergence, the jets impinge directly against the sides of barges or other craft with a draft of 2 feet or more, causing mooring difficulties and excessive hawser stresses during filling operations.

Port Redesign

The turbulence can be reduced appreciably by revising the ports so that the jets issue into the lock chamber flush with the lock floor and by staggering the ports in one wall with respect to those in the opposite wall. With the deeper water cushion the jets will not break through the water surface with as great intensity and the jets flowing across the entire lock chamber will diverge rapidly and strike the opposite wall with greatly reduced velocity. Furthermore, the jets will not impinge directly against the sides of craft in the lock.

Suggested revisions in the port design are shown in Figure 4 (see Drawing 5/39). The port entrances have been rounded to improve the hydraulic efficiency of the filling system. Ten ports will be required in each wall as at present. The existing ports in one wall can probably be revised in accordance with the suggested design but those in the opposite wall should be plugged and new ports constructed midway between present locations.

TURBULENCE IN LOWER APPROACH

The hazardous waves and currents which occur in the downstream approach to Lock No. 1 are due to concentration and lack of diffusion of the outflow from the discharge systems, particularly from those located in the intermediate wall. The outlet from the river wall discharges into a stilling basin which distributes the flow to some extent in an open channel isolated from the navigation approach and probably does not disturb traffic. If by inspection the stilling basin is found to be structurally sound, it would appear that no modifications in the riverward outlet will be required.

The outflow from the land wall culvert and from the two culverts in the intermediate wall should be distributed and diffused as much as possible in the immediate area in order to reduce the currents and eliminate the presently hazardous conditions downstream. A suggested modification of the outlets from these three culverts is shown in Figure 5 (see Drawing 5/39). The two culverts in the intermediate wall are joined and will discharge into a common lateral diffuser located below the miter gate of the riverward lock. It is presumed that both locks will not be emptied at the same time. The culvert in the landwall will discharge into a similar lateral diffuser located below the miter gate of the landward lock. Thus, the outflow from the landward lock will be distributed across both lock entrances. The lateral diffusers will localize the dissipation of energy within a short distance below the miter gates where no traffic should be allowed during emptying operations and the approach channel downstream should be free of excessive turbulence and currents that might endanger traffic waiting for lockage clearance.

INTAKE MANIFOLDS

The intake manifolds for the lock filling systems are located in the miter gate recesses. During filling operations the flow from the upper pool toward the intake ports will draw ice and other floating debris into the lock entrances. Frequently debris becomes lodged in the gate recesses and obstructs complete opening of the gate until it is removed. Lockages are delayed and normal removal of the debris under some conditions can be a hazardous operation.

In more modern locks the intake ports are located outside of the lock entrances if structurally feasible. If this is not possible, the intakes will be located in the face of the lock walls upstream of the miter gate recesses. Thus, the current subsides upstream of the miter gates and the problem of trash obstruction and removal is greatly reduced. Either of these solutions can be only approximated in Lock No. 1.

An extension of the intermediate wall upstream, permitting construction of dual intake manifolds, as shown in Figure 3 of Drawing 5/39, will provide flow to respective culverts in either lock, whichever is to be filled. By drawing water from both sides of the I-wall the current into the lock entrance will be substantially reduced. A typical intake manifold for the landward guide wall is shown in Figure 6. The port areas are graduated to provide uniform flow, as nearly as possible, into all the ports. A similar intake can be constructed in the riverward guide wall. By facing the ports toward the river, drift will not be carried into the lock entrance by flow approaching this intake.

SUMMARY OF RECOMMENDED MODIFICATIONS

1. Replace the present stoney gate filling valves with reversed tainter valves.

2. Lower the filling valves to the level of the main culvert with floor elevation at 681.2 as indicated in Section A-A of Drawing 5/38 or relocate just below the vertical drop of the existing culverts.

3. Revise the lock chamber ports to discharge flush with the lock floor and as shown in Figure 4 of Drawing 5/39. Round the entrance corners for improved efficiency and stagger ports in one wall with respect to those in the opposite wall to reduce turbulence.

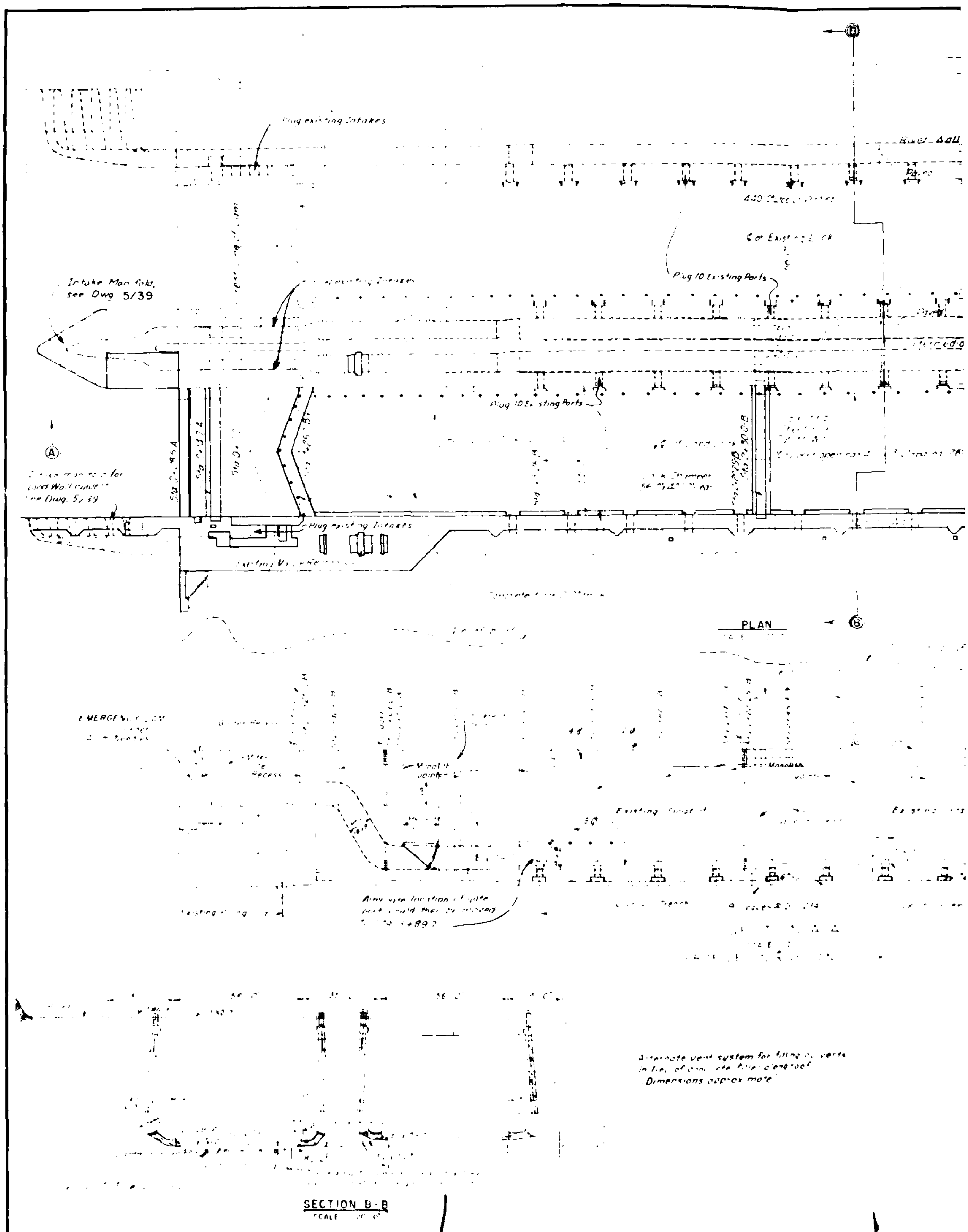
4. Fill the culvert crowns with concrete to elevation 687.2 to exclude air resident in the culverts when tailwater level is below elevation 690.7. This recommendation is based on the assumption that the filling valves will be lowered as indicated in (2) above. The proposed concrete filler along the roof of the culvert will reduce the combined area of two culverts from 141.7 square feet to 96.2 square feet, a reduction of 45.5 square feet. The filling time will be increased inproportion to the reduction in area if the valves remain at their present location. However, if the valves are lowered the hydraulic efficiency of the filling system will be increased sufficiently to compensate for the reduction in area of the culverts. Should it be deemed impractical to lower the valves, then it might be preferable to eliminate the filling along the roof and rely on an improved venting system to remove the trapped air from the culverts. Air entrained in the hydraulic jump and released in the lock chamber would then have to be tolerated as only a very small amount of the entrained air would escape through the vents.

5. Provide additional exits for escape of air in the present venting system in conjunction with revision 4. Without revision 4 the venting system must be substantially enlarged as shown on Drawing 5/38.

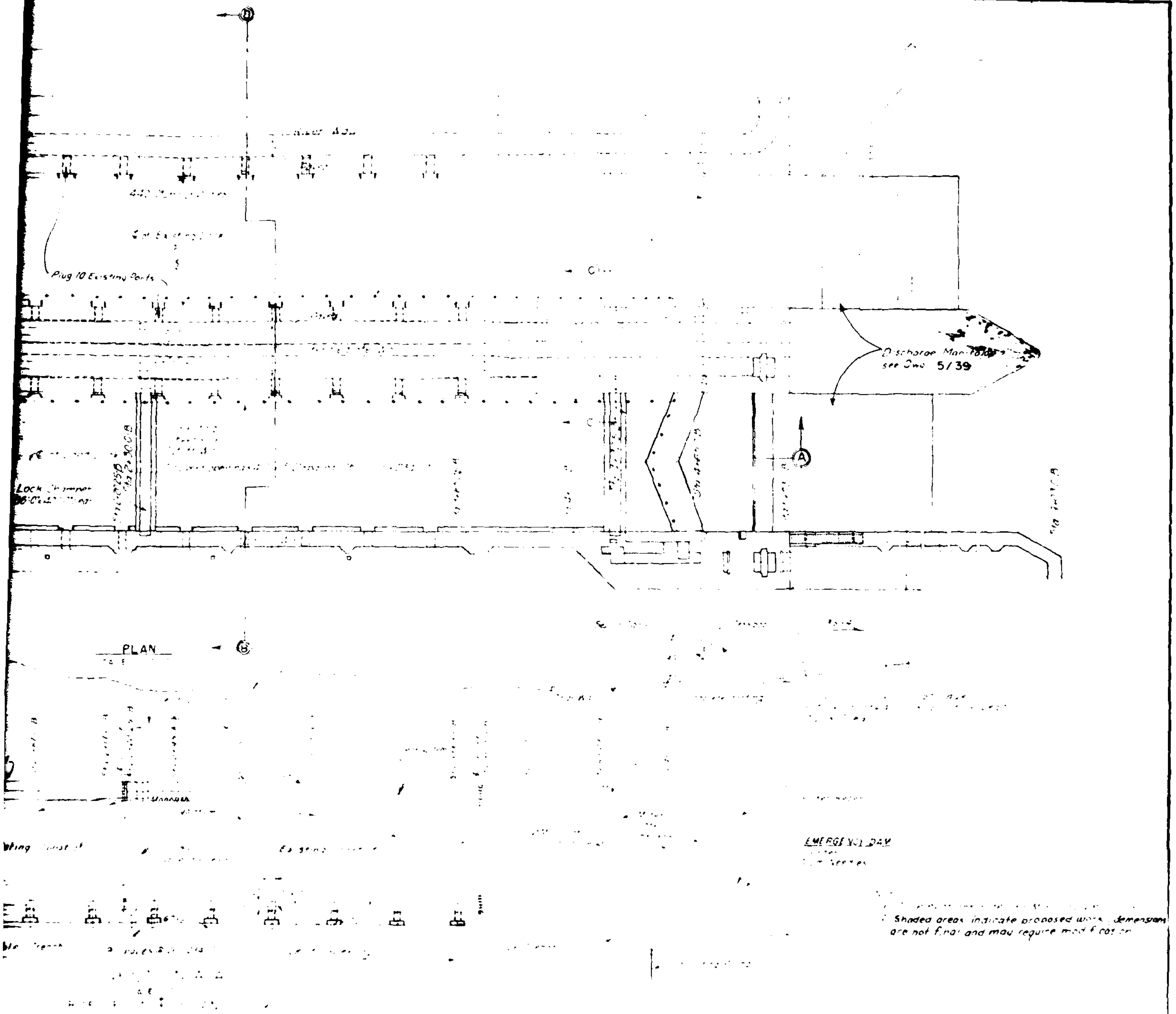
6. Construct lateral diffusers below the downstream miter gates as shown in Figure 5 of Drawing 5/39 to dissipate the energy of discharge during emptying operations in order to minimize hazardous currents and disturbances in the lower lock approach.

7. If the present emptying valves are damaged beyond repair, they should be replaced preferably with reversed tainter valves but butterfly or vertical slide valves would also be satisfactory if cost is appreciably more favorable.

8. Constant new intake manifolds upstream of the miter gate recesses to reduce tendency for floating debris to interfere with opening of miter gates. Suggested intake revisions are shown on Drawing 5/39.



Plug existing Intakes
Intake Man fold, see Dwg. 5/39
Plug 10 Existing Ports
Plug 10 Existing Ports
Plug existing Intakes
PLAN
Existing
Alternate vent system for filling culverts in lieu of concrete filled end roof
Dimensions approximate
SECTION B-B



Alternate vent system for filling culverts
In lieu of concrete filled roof
(Dimensions approximate)

SYMBOL	DESCRIPTION	DATE	APPROVAL

DEPARTMENT OF THE ARMY
ST PAUL DISTRICT, CORPS OF ENGINEERS
ST PAUL, MINNESOTA

DESIGNED BY M N
DRAWN BY
CHECKED BY
SUBMITTED BY
APPROVED

GENERAL DESIGN MEMORANDUM
MISSISSIPPI RIVER
LOCK AND DAM NO 1 REHABILITATION

RECOMMENDED SOLUTIONS FOR HYDRAULIC PROBLEMS IN LOCK OPERATION

PLAN & SECTIONS

DATE

SCALE 1" = 20'

DRAWING NUMBER
M-R1-5/38
SHEET 1 OF 2

AD-A134 023 MISSISSIPPI RIVER: STUDY OF ALTERNATIVES FOR REHABILITATION OF LOCK AND D..(U) CORPS OF ENGINEERS ST PAUL MN ST PAUL DISTRICT APR 76 4/4

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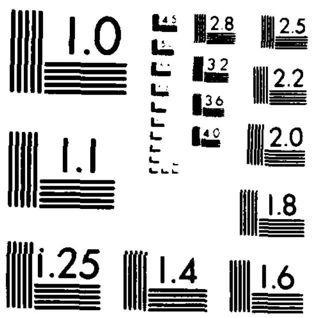
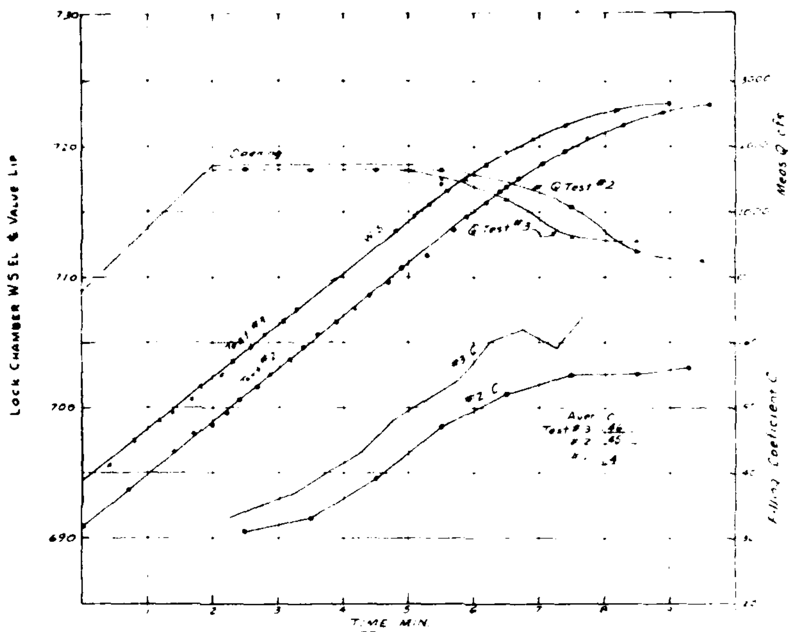


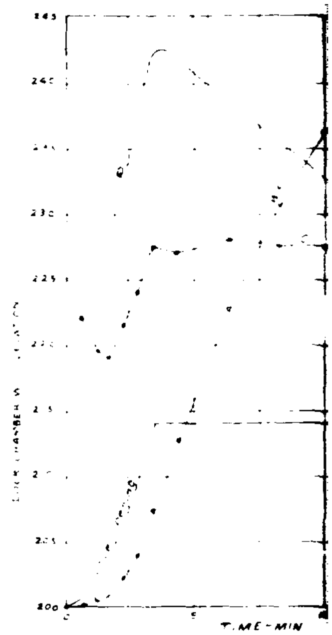
MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS 1963-A

CORPS OF ENGINEERS



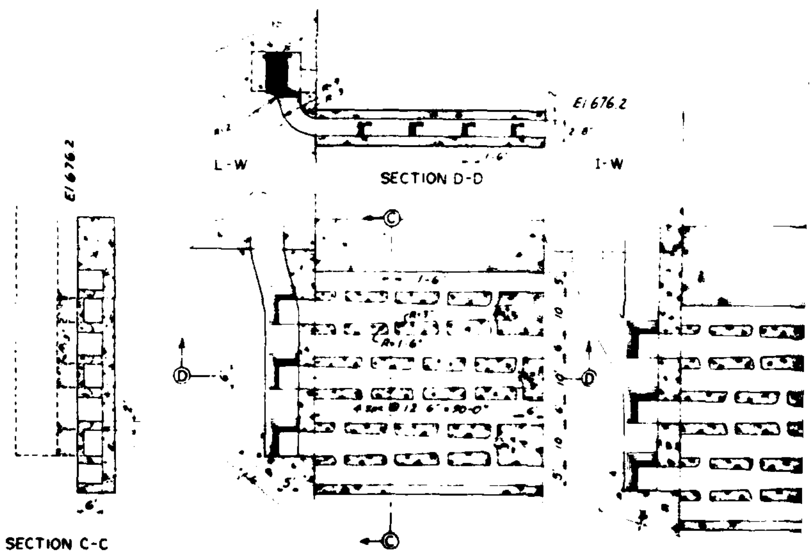
LOCK #1 TESTS 8 DEC 1973

Figure 1



EISENHOWER LOCK MODEL 5(

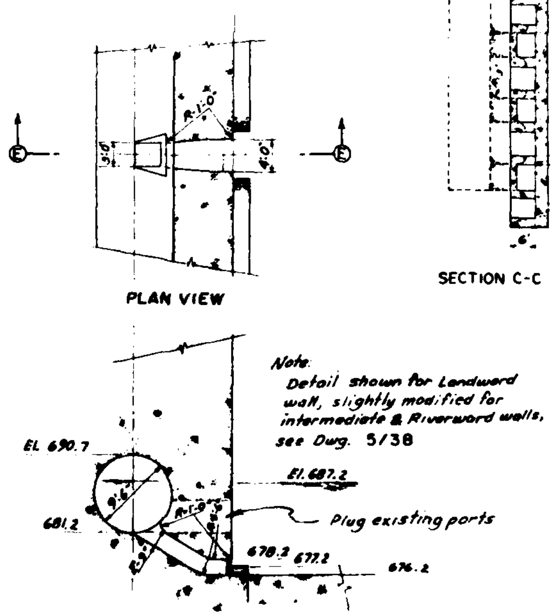
Figure



SECTION AT ELEVATION 67.5
LOCK NO 1 DISCHARGE MANIFOLDS

Note
Concrete cover over Manifolds not shown in Plan view

Figure 5



Note:
Detail shown for Landward wall, slightly modified for intermediate & Riverward walls, see Dwg. 5/38

SECTION E-E
PORTS - 10 PER WALL

Figure 4

U S ARMY

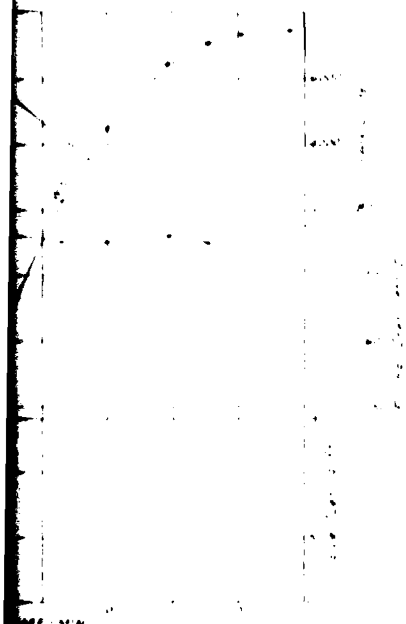
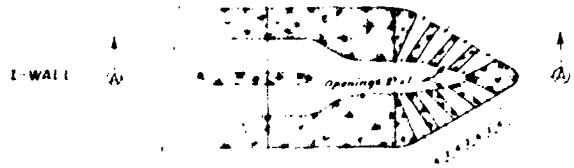


Figure 2

SECTION A A

Current

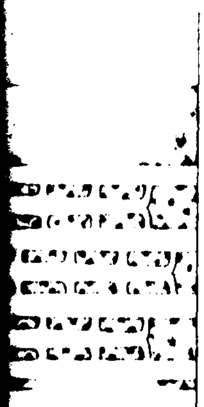


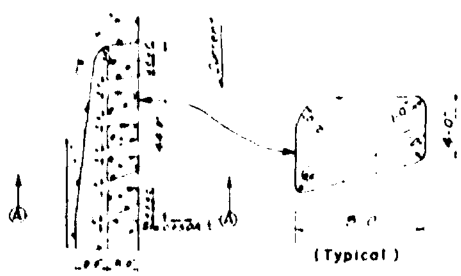
SECTION AT ELEVATION 710
LOCK NO 1 PROPOSED INTAKE MANIFOLD-I-WALL

Figure 5

SECTION A A

Intake Manifold F... Landwall Culvert, intake fo... Riverwall will be similar and facing in the same direction, ie, toward the river





SECTION AT ELEVATION 710
Figure 6

DEPARTMENT OF THE ARMY
ST PAUL DISTRICT CORPS OF ENGINEERS
ST PAUL MINNESOTA

GENERAL DESIGN MEMORANDUM
MISSISSIPPI RIVER
LOCK AND DAM NO.1 REHABILITATION
RECOMMENDED SOLUTIONS FOR HYDRAULIC PROBLEMS IN LOCK OPERATION
CHARTS & DETAILS

DESIGNED BY	RR
DRAWN BY	
CHECKED BY	DCL
SUBMITTED BY	
APPROVED	

DATE JANUARY 1974

SCALE 1" = 16'

DRAWING NUMBER M-RI-5/39

SHEET 2 OF 2

DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT, CORPS OF ENGINEERS
1210 U.S. Post Office & Custom House
St. Paul, Minnesota 55101

MISSISSIPPI RIVER

STUDY OF ALTERNATIVES FOR REHABILITATION OF LOCK AND DAM NO. 1

MINNEAPOLIS, MINNESOTA

APPENDIX E

COFFERDAMS

TABLE OF CONTENTS

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PLATES

Appendix E

COFFERDAMS

Structural Type

1. Cofferdam Crest Elevations. Recorded upstream and downstream water surface elevations at the project site during the years from 1951 through 1972 reveal that the highest water levels occur during the month of April. To reduce the cost of cofferdams and to avoid the necessity for incurring a high risk of overtopping during the month of April, the cofferdams are designed for closure between November 1 and April 1. Based on the above water level recordings, crest levels of 733 and 705 were selected for the upstream and downstream cofferdams, respectively. For both cofferdams, a freeboard of 3 feet is provided and complete protection against overtopping is obtained during November through March according to 1951-1972 records.

During the month of April the maximum headwater stage recorded was 734.5 on April 17, 1965. This is 1.8 feet above the top of lock walls. The accompanying maximum tailwater elevation registered was 719 or 14 feet above the proposed crest elevation of the downstream cofferdam. Obviously, by including the month of April in the dewatered period it would require that the top of the downstream cofferdam be substantially raised to minimize the risk that the Contractor must incur. At first glance it would appear that the top of the downstream cofferdam should be raised to at least elevation 712.0 to take care of all of the April flood peaks of record except for the three exceptionally highest floods recorded. However, this would result in a substantial increase in the cost of the sheet pile cofferdam and would still require the Contractor to assume a risk of about one in eight of being overtopped in the month of April. By avoiding the month of April, i.e., not require that the area behind the cofferdam be dewatered through April, the top of cofferdam elevations as selected will protect against ALL flood peaks observed during the months of August through March, inclusive of the period of 1951-1972. This certainly minimizes the Contractor's risk for Alternative Schemes of Rehabilitation 1, 2 and 4 where the compressed time schedule could not tolerate overtopping. Alternative Scheme of Rehabilitation 3 requires that the area behind the cofferdams be dewatered from December 1 of the first year through to October 1 of the second year, thus requiring the Contractor to assume some added risk of overtopping during the month of April. The scheme as depicted on the drawings would expose the Contractor to about a 1 in 6, or 17% chance of being overtopped during the month of April. However, Alternative 3 does not have a compressed time schedule and, in the case of an imminent overtopping, the earth embankment tie-in section could be carefully breached to inundate the working area. Subsequent cleanup and repairs to the tie-in section could take up to two months without affecting the overall schedule.

2. Space Limitations. The structural type of cofferdam selected was based on space limitations, height requirements, speed of construction, resistance to scour, and effect of its construction on existing environmental conditions. The limited space available both upstream and downstream of the locks, the need for relatively rapid, staged construction and the ability to resist scour for the flow conditions to be experienced during construction led to the adoption of cellular, interconnected steel, sheet pile cofferdams.

3. Cellular Steel Sheet Pile Cofferdams. Stability computations indicated that a 28 foot diameter cell should be used. With a 28 foot diameter, each cell is stable against overturning and sliding and the tension in the interlocks is not excessive. The cellular structures will have an actual diameter of 28.65 feet, with cells 33.87 feet on centers using U.S. Steel steel sheet piling section P.S. 28 or equal. The stability computations ignored any benefit from penetration of the sheet piles into the alluvium. Average penetration in soil will be in the order of 15 feet. This length is necessary to limit the expected under-seepage to a reasonable value when the work area is dewatered. In addition, a berm will be constructed against most of the inside perimeter of both the upstream and downstream cofferdams. If seepage exceeds the anticipated value, the loading at the inside toe of the cofferdam can be increased by extending the berm created by the access road and/or raising the level of the access road. This would increase the seepage path and should reduce seepage accordingly. Also the possibility of a piping failure would be reduced.

An [illegible] miter gate lies submerged and partially buried approximately [illegible] feet away from the river wall discharge opening. It is within the proposed foundation area for the downstream cellular sheet pile cofferdam and will have to be removed prior to the cofferdam installation. In addition, the alluvium material overlying the top of rock in the area of the cofferdam foundations contains weathered limestone cobbles and blocks ranging in size from about 1 to 3 feet in diameter. Special care must be exercised in obtaining the required penetration in this material. The use of a hairpin jet, with a jet on both the inside and outside of the cell should be considered to get the sheets down with the least amount of bind. When a limestone block is met the block should be carefully probed with easy driving on the sheet or sheets obstructed and the adjacent sheets. It may be possible to penetrate or push aside the block by forcing down the sheets affected with careful, light driving and in small increments if the block is not too large or is highly weathered. If, however, the affected sheets do not move, driving should stop and the obstructing block should be dug out or chopped out from under the sheets. It may even be necessary to pull some of the piles in the cell to obtain greater freedom for digging.

Upstream Cofferdam

4. General Arrangement. An upstream cofferdam is required to permit rehabilitation and modifications to the lock bulkhead slots and modifications to the intakes of the lock hydraulic filling system. Since some laydown area is available just upstream and adjacent to the cofferdam along the landward wall only a limited amount of area is necessary below at elevation 708±. The cofferdam configuration shown in Plate E-1 results in the minimum length of cofferdam commensurate with access, storage and working area requirements. Modifications to the intakes of the filling system will require removal of existing concrete down to elevation 704.7. This material can be loaded on trucks and hauled out by way of the access ramp to a waste site. Subsequent replacement by new concrete can be effected by driving ready mix trucks down to the working area at elevation 708±. A bucket handled by a mobile truck crane can be used there to transfer the concrete to the placement area within the forms.

5. Closure Sections. The cofferdam configuration shown requires a closure section against monoliths 7 and 8 of the landward guide wall and against the upstream face of the old existing concrete gravity lock wall adjacent to the riverward wall of the riverward lock chamber. In compliance with an early suggestion to have the

intake for the hydraulic filling system of the river wall opening out to the spillway side of the unused lock wall, another cofferdam configuration had been studied. To provide adequate working room, that cofferdam would have had to pass around the upstream end of the old unused lock wall and close back against that same wall just upstream of the spillway structure. Such a configuration would have several adverse features that led to its rejection and subsequent modification to the suggested intake location. For example: the cofferdam cells would block off part of the effective spillway length; the river bottom adjacent to the outermost cells would be subjected to higher scour velocities; and during placement of the last cells, the spillway structure could be subject to possible damage from collision with the pile driving barge during maneuvering of the barge due to the lack of free space.

6. Construction of Cells. The majority of the cells would be placed during the fall navigation season prior to winter closure of the navigation locks. Those cells and connecting diaphragms on the riverward side of the temporary navigation opening indicated on Plate E-1, would be constructed with a barge mounted crane and pile driver and also filled with granular material from a barge. The lone cell against the landward guide wall can be placed and filled from the landward wall.

7. Construction of Closures. Between the last cells and the concrete face of the guide wall and the upstream face of the old unused riverward lock wall closure will have to be effected by utilizing tremie placed concrete to prevent failure resulting from piping and loss of the cell fill material due to an inadequate seal between the sheet piles and the concrete surface. Unbalanced hydrostatic pressure on the connecting section will cause it to be wedged against the contiguous sheet pile cell and the concrete river wall or guide wall. At both ends, after the closing units of sheet piling have been driven up against the concrete faces, tie members will be placed within the last section to hold the sheet piling in place during the time that concrete is placed and to insure that the last sheet pile is tight against the concrete face. Existing erosion along the nose of the old unused riverward lock wall will require special forming prior to concrete placement. A diver could be used to position wood blocking conforming to the outline of the eroded face. By placing the tremie concrete at a rate of one foot per hour or less, the lateral force on the cross tied sheet piling will be minimized while a good closure is effected against the existing concrete face.

8. Temporary Navigation Opening. The temporary navigation opening provided is based on a size estimated necessary for passing barge traffic. Its actual size and location in the line of the cofferdam will be checked using buoys prior to the initiation of construction, and the final size and location will depend on the ability of tow operators to pass various buoy arrangements. The construction schedule requires that the temporary opening be closed in a period of two weeks. This can easily be done working from the land side off the top of the initial cell placed earlier.

9. Unwatering and Access. After closure is effected, the working area will be unwatered by pumping over the cofferdam. When the area is dewatered, it will be cleaned up and provided with settling basin and drainage pumps. A collecting ditch will be excavated and maintained during the period of construction. The access ramp construction will proceed as soon as the permanent construction drainage system is operable.

Downstream Cofferdam

10. Space Requirement. The principal access to the lock basins and the area involved with modifications to the hydraulic emptying system will be obtained from within the confines of the downstream cofferdam configuration. This area will provide ample laydown space for storage of mobile equipment and trucks, formwork, and construction materials, outside of the areas to be affected by removal and construction of wall and slab structures.

11. Selected Alignment. Alternative cofferdam configurations were considered before the final arrangements shown on Plates E-2, E-3, and E-4 were established. The configurations shown on Plates E-2 and E-3 are essentially the same and only differ in the point of closure to the riverward wall of the riverward lock, which of course is dependent on the scheme of rehabilitation considered. The sheet pile cofferdam configuration shown on Plate E-3 will not accommodate the riverwall outlet structures shown on Plate D-4. Calculations indicate that the cofferdam length would have to be increased by one circular cell and one connecting arch to enclose either of the proposed discharge structures. The cost of this additional length of cofferdam is included in estimates for Plans No. 4A and 4B.

12. Alternative Alignments. The alternative cofferdam configurations which were rejected, all resulted in less laydown area and

had closure problems against the downstream landward guide wall and the permeable riverward guide wall constructed essentially of stone and smaller granular material. The downstream portion of the landward guide wall is constructed on rock filled timber cribbing, which is exposed above the bottom of the bed of the channel. It would be difficult to close off seepage around the cofferdam closure on either structure due to seepage through the base of the existing structures. The cost of removing and then replacing part of the riverward stone covered guide wall would be very expensive. Also the time required for removal would interfere with contractors ability to accomplish the work in the short time available for this work in the construction schedules.

13. Construction of Cells. To complete the construction of the downstream cofferdam cells in the time available, the Contractor will need two placing units. As shown on Plates E-2 and E-3, a portion of the cofferdam cells must be constructed from a barge mounted crane and pile driver. The remaining portion must be constructed from a land operated crane and pile driving rig. Placing the cofferdam piling entirely from a barge would be impractical. The water is normally too shallow in the area designated for cell construction from a land based crane, and it would be necessary to first dredge a channel for access. This would be uneconomical and would also have some effect on the island. In order to facilitate his work by land based operations, the Contractor will have to develop a temporary staging area on the adjacent existing island. This will require the construction of a temporary barge unloading dock on the downstream end of the island. Before constructing the land based portion of the cofferdam, the flow in the shallow existing channel between the island and the riverward lock guide wall would be stopped by filling the channel to elevation 690.0 at its upper end. The small amount of turbulent seepage that could be expected to pass through the closure material could be tolerated during the subsequent construction of the sheet pile cells in the shallow channel which can then be temporarily filled in with granular material to provide a base for operation of the crane and pile driving unit.

14. Closure at River Wall. To effectively close against the sloping face of the riverward wall of the riverward lock, an earth embankment transition section will be used with a wraparound enclosing the first pile cell. The Contractor can construct a sloping ramp up to the top of the transition section to permit filling the cells with trucks. The trucks could be filled from barges at the temporary dock constructed on the island. The

upstream portion of the impervious embankment section will be protected with filter and rip-rap.

15. Closure at Crib Wall. The closure of the downstream cofferdam against the crib wall protecting the existing bluff will require special treatment. To keep from disturbing the foundation of the crib wall and to eliminate damage to the face of crib wall, an earth embankment transition section will be used to tie the cellular sheet pile cofferdam to this wall. The earth embankment transition section will also tie into the earth fill behind the landward guide wall. This will prevent seepage from passing around through the granular backfill behind the crib wall facing into the unwatered work area. Since seepage through the cribwall backfill occurs at tailwater elevations between 700 and 705, the intent of the scheme depicted is to provide positive protection against tailwater to elevation 702 during the time that the construction area is dewatered. For the resulting head of 2' on the roadway section the effective weighted creep-head ratio is in the order of 4.7 which is adequate to eliminate the possibility of excessive seepage and piping through the cribwall backfill material that is equal to or larger than fine gravel. Should the tailwater elevation rise above elevation 702, a decision would have to be made to carefully breach the earth embankment section so as to fill the dewatered area behind the cofferdams before the cofferdams themselves would be overtopped, to avoid any damage that could negate their continued use. A decision to carefully breach the cofferdam and flood the working area would, of course, have to be made on the basis of a rising river stage and the known effect of upstream inflows, which would lead to the inference of a possible cofferdam overtopping.

16. Temporary Navigation Opening. A temporary navigation opening is provided in the line of the cellular sheet pile cofferdam as shown on Plates E-2 and E-3. The final opening width and orientation will depend on tests made with barge tows passing marker buoys prior to the initiation of construction. The navigation opening would be closed off in a period of two weeks immediately after the locks are closed for the winter. The closure can be effected by a crane and pile driving unit working off the top of the two land side cells placed previously. During low water surface levels in the downstream pool, the flow velocity through the temporary navigation opening will reach a maximum average velocity of approximately 3 feet per second when the landward lock basin alone is lowered to lower pool level. The amount of scour to be expected would be minimal and would not jeopardize

the stability of the freestanding cofferdam cells at either side of the temporary opening.

17. Access. Access to the construction site would be over the access road to be constructed on the existing fill between the landward guide wall and the crib wall protecting the face of the bluff. A very minimum amount of earthwork must be done to obtain access by this route. The limiting clearance of the access road is the 10-foot wide gap between the bluff and the lower guidewall monolith No. 13. The dump trucks entering and leaving the lower construction area have an overall width of 8 feet and are expected to negotiate the turns below the wall without difficulties. However, a small modification to the end wall could be easily and quickly accomplished if necessary without affecting the stability or esthetics of the wall. Other modifications than the removal of 2 or 3 feet of the end closure section above elevation 700 would not be necessary at the lower guide wall. To support the construction access road when replacing the first two or three monoliths of the lower guide wall, an anchored steel sheet pile wall should be used as shown in plan on Plates E-2 and E-3 and in Sections A-A, B-B, and C-C on Plate E-4. The existing crib wall in the vicinity of the proposed steel sheet pile wall, as shown on the above sections, is founded on sandstone. The existing backfill gives some minor support to the crib wall. However, the crib wall was constructed before the fill was placed. Vibrations from driving the steel sheet pile wall could cause some densification of the backfill material. These vibrations should have no effect on the actual sandstone foundation of the crib wall. A minor settlement of the backfill would have no effect on the stability of the crib wall. If a sonic vibrator were used the vibrations would be in resonance with the piles. The impulse of the vibration would be in phase with the elastic compression wave that travels down the pile and the energy would be used most effectively in overcoming friction and point resistance. Thus, the noise and shock nuisance would be less than for impact hammers.

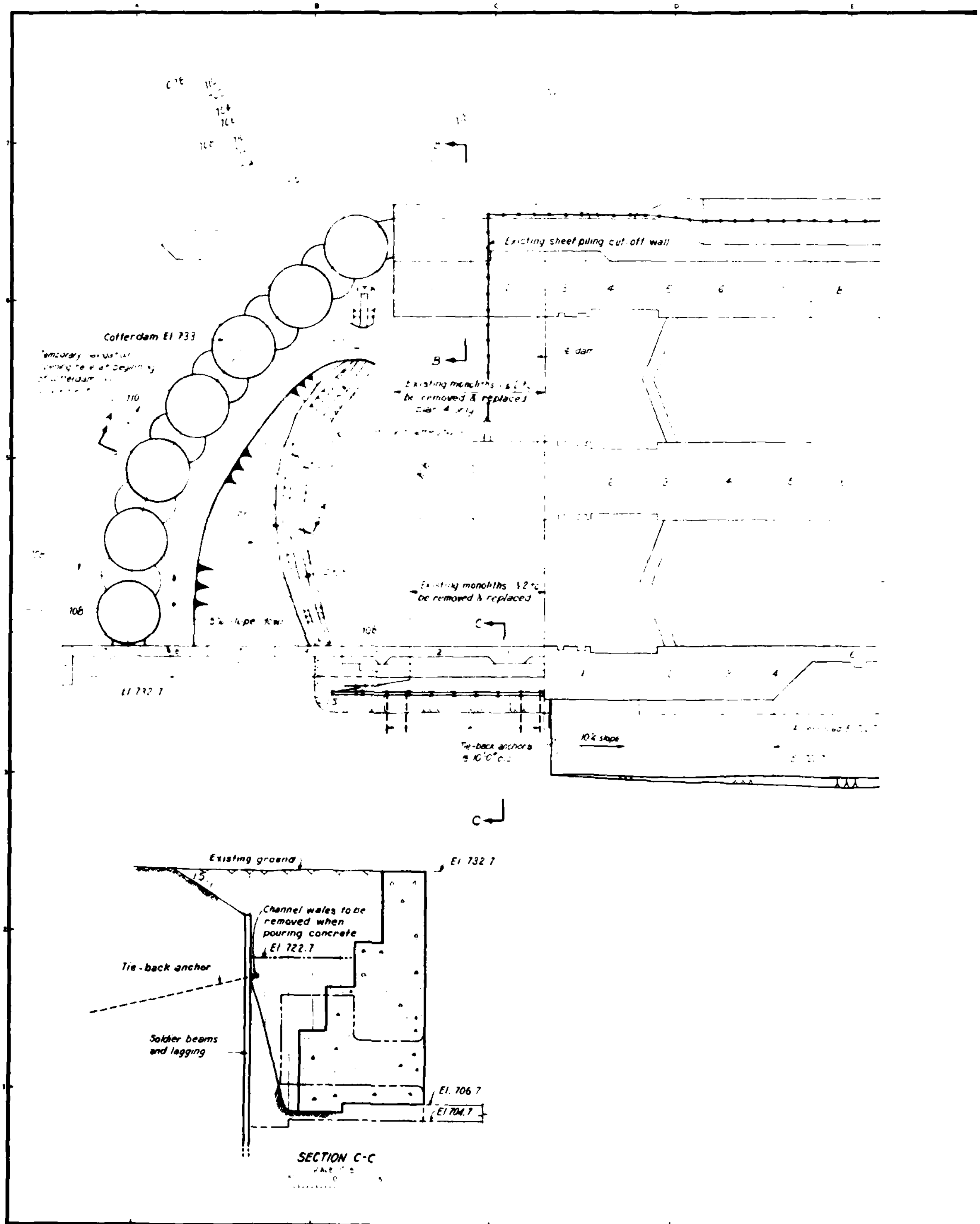
The groundwater level in the backfill is reflected in Borehole 74-4, measured at elevation 692.4 on July 23, 1974 and in Borehole 74-8, measured at elevation 698.9 on May 31, 1974. To reduce the pressure against the wall and to prevent the piping of material in front of the embedded piling, the backfill must be drained through windows in the sheet pile wall or by dewatering with wells. The cost of removing this water has been included in the cost of dewatering the work area during construction.

18. Unwatering. As soon as closure is effected, the working area will be unwatered by pumping over the cofferdams to tailwater. Clean up operations, construction of a drainage sump and seepage collection ditch, and construction of a sloping access ramp down into the working area will proceed similar to the operation explained for the upstream cofferdam. The sump collection system will include an adequately sized settling pond to settle out suspended solids prior to pump-out.

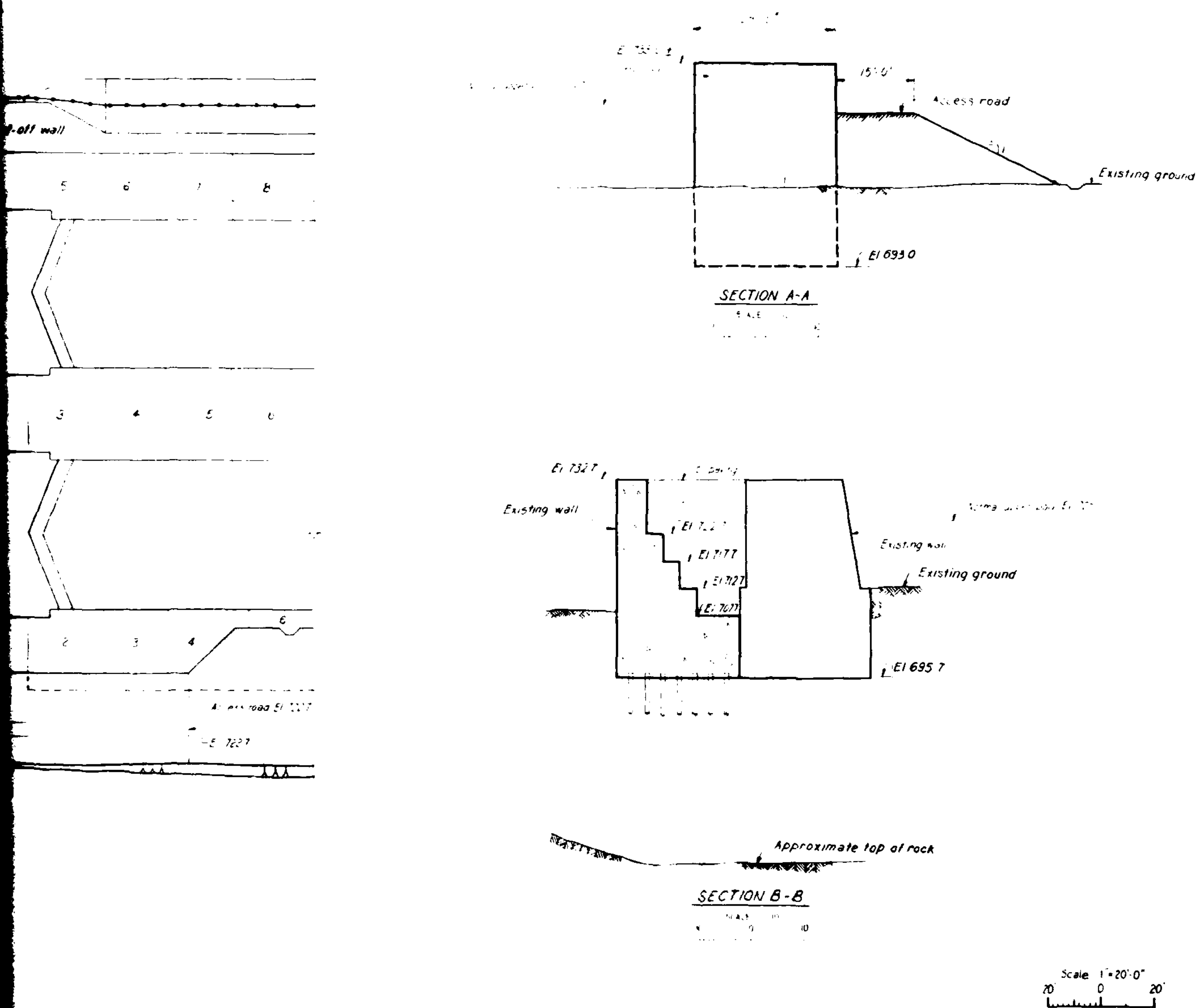
Cofferdam Removal

19. Temporary Navigation Opening. It is recognized as essential that overall construction proceeds so that the temporary navigation openings can be opened in the first week of April prior to the beginning of the high flow season. This could be done using a crane mounted on top of the land side cells, offloading materials into barges. The working areas would be flooded by pumping to permit the opening to be effected against balanced head.

20. Removal and Clean-Up. The remaining portions of the cellular sheet pile cofferdams can then be removed after the high flow season is over with a minimum of risk to the Contractor. Removal of the cofferdams will essentially follow their construction in reverse. The Contractor will be required to effect their removal in such a manner as to cause the minimum possible disturbance to the river ecology. The staging area on the island will be cleaned up and the island returned to a configuration and ecological state acceptably similar to the conditions encountered at the initiation of construction.



Cofferdam El 733
Existing sheet piling cut-off wall
Existing monoliths to be removed & replaced
Existing monoliths to be removed & replaced
Tie-back anchors
10% slope
El 732.7
Existing ground
El 732.7
Channel wales to be removed when pouring concrete
El 722.7
Tie-back anchor
Soldier beams and lagging
El. 706.7
El 704.7
SECTION C-C



HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT CORPS OF ENGINEERS
ST. PAUL, MINNESOTA

M.J.
E.J.M.
R.N.M.

SUBMITTED BY

APPROVED

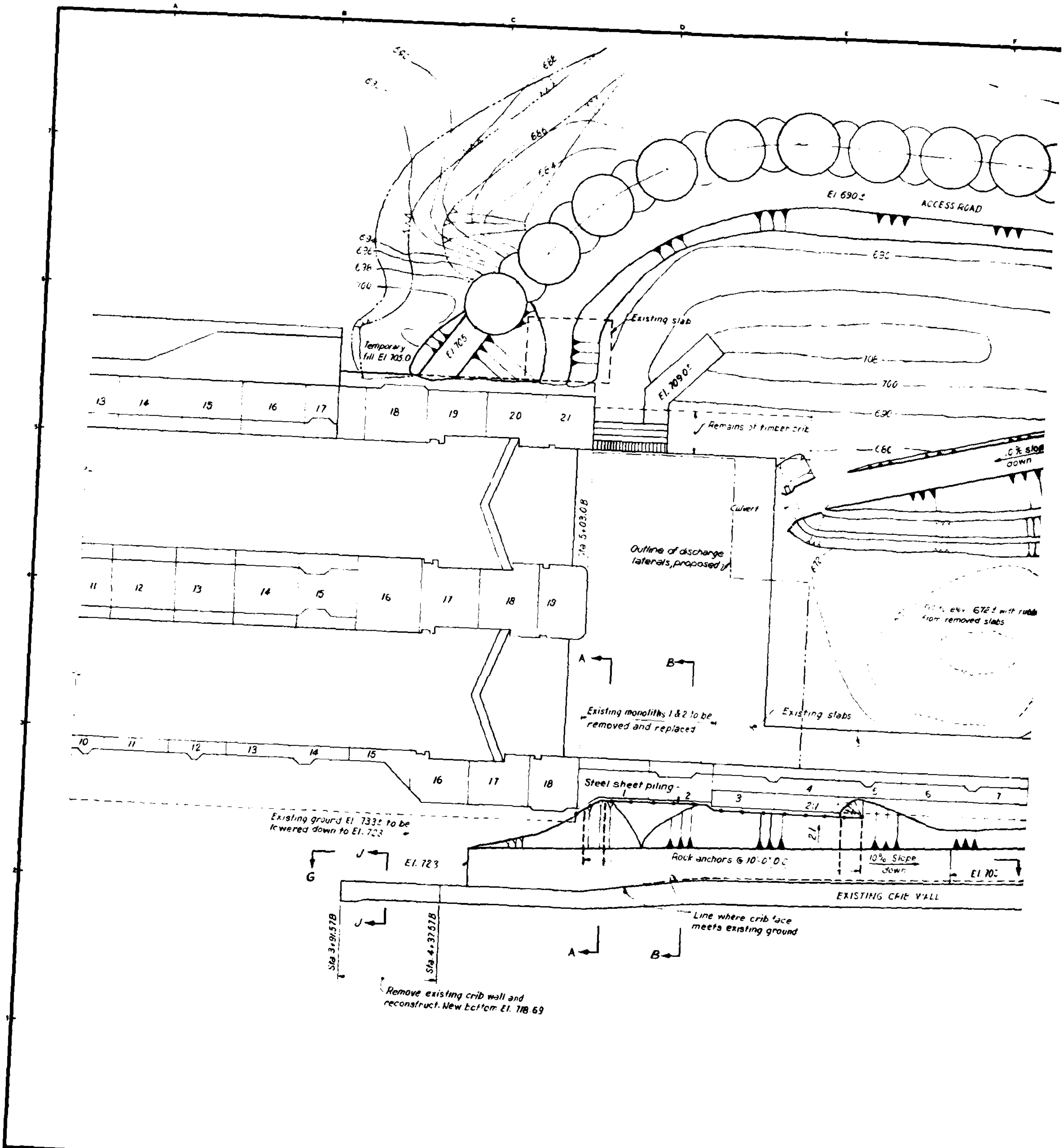
STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO. 1
UPSTREAM COFFERDAM

DATE:
MARCH 1975

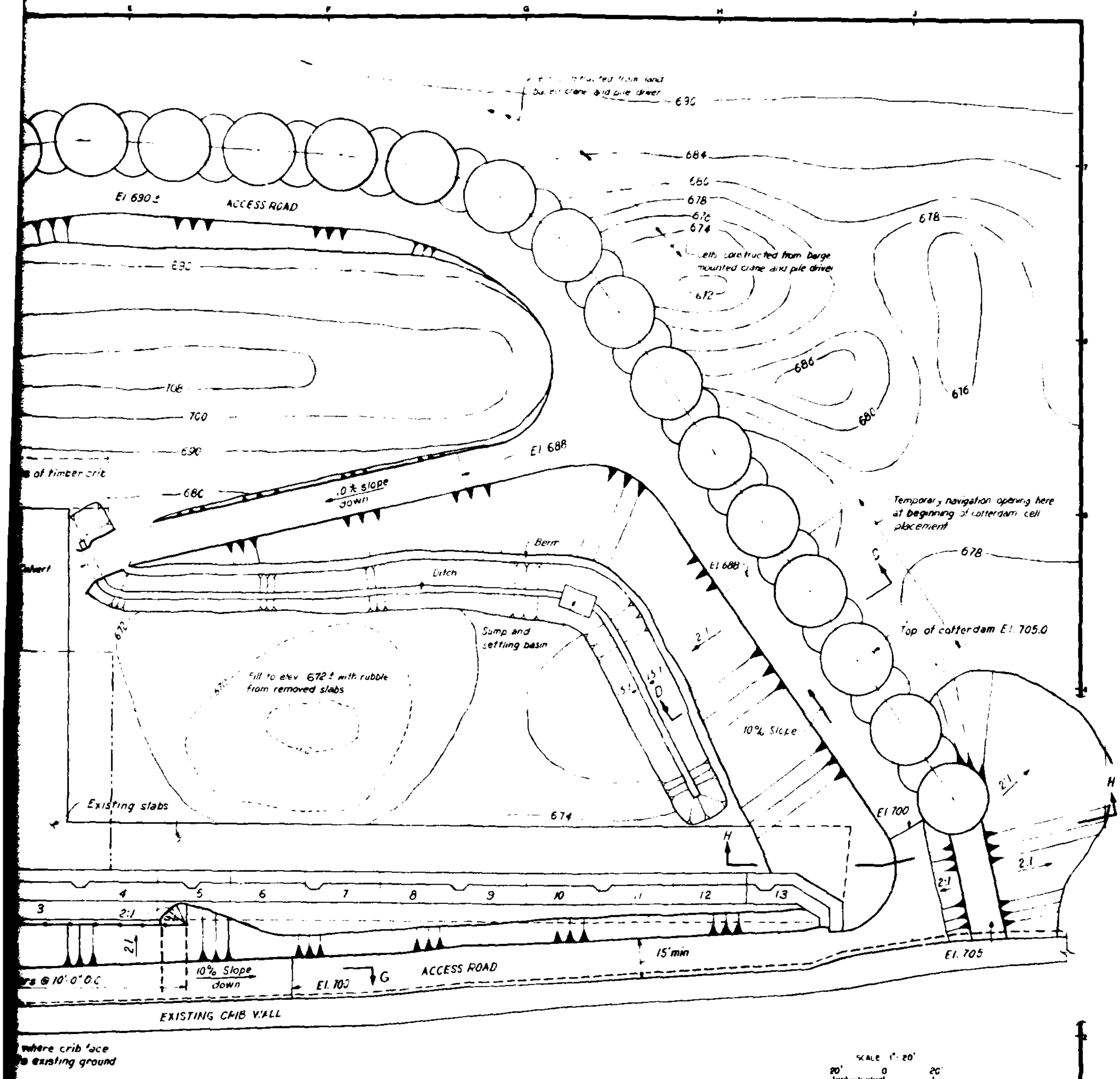
DRAWING NUMBER

SHEET OF

PLATE E-1



ACCESS ROAD
El 690±
Existing slab
Temporary fill El 705.0
El 705
El 709.0±
Remains of timber crib
Culvert
Outline of discharge laterals, proposed
Sta 5+03.08
Existing monoliths 1 & 2 to be removed and replaced
Existing slabs
Steel sheet piling
Existing ground El 733± to be lowered down to El 723
El 723
Rock anchors @ 10'-0" O.C.
10% Slope down
El 700
EXISTING CRIB WALL
Line where crib face meets existing ground
Sta 3+91.57B
Sta 4+37.57B
Remove existing crib wall and reconstruct. New bottom El 718.69



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ST PAUL MINNESOTA

DESIGNED BY R. N. M.
DRAWN BY E. J. M.
CHECKED BY R. N. M.

STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO. 1

DOWNSTREAM COFFERDAM

PLANS 1 AND 3

SUBMITTED BY

APPROVED:

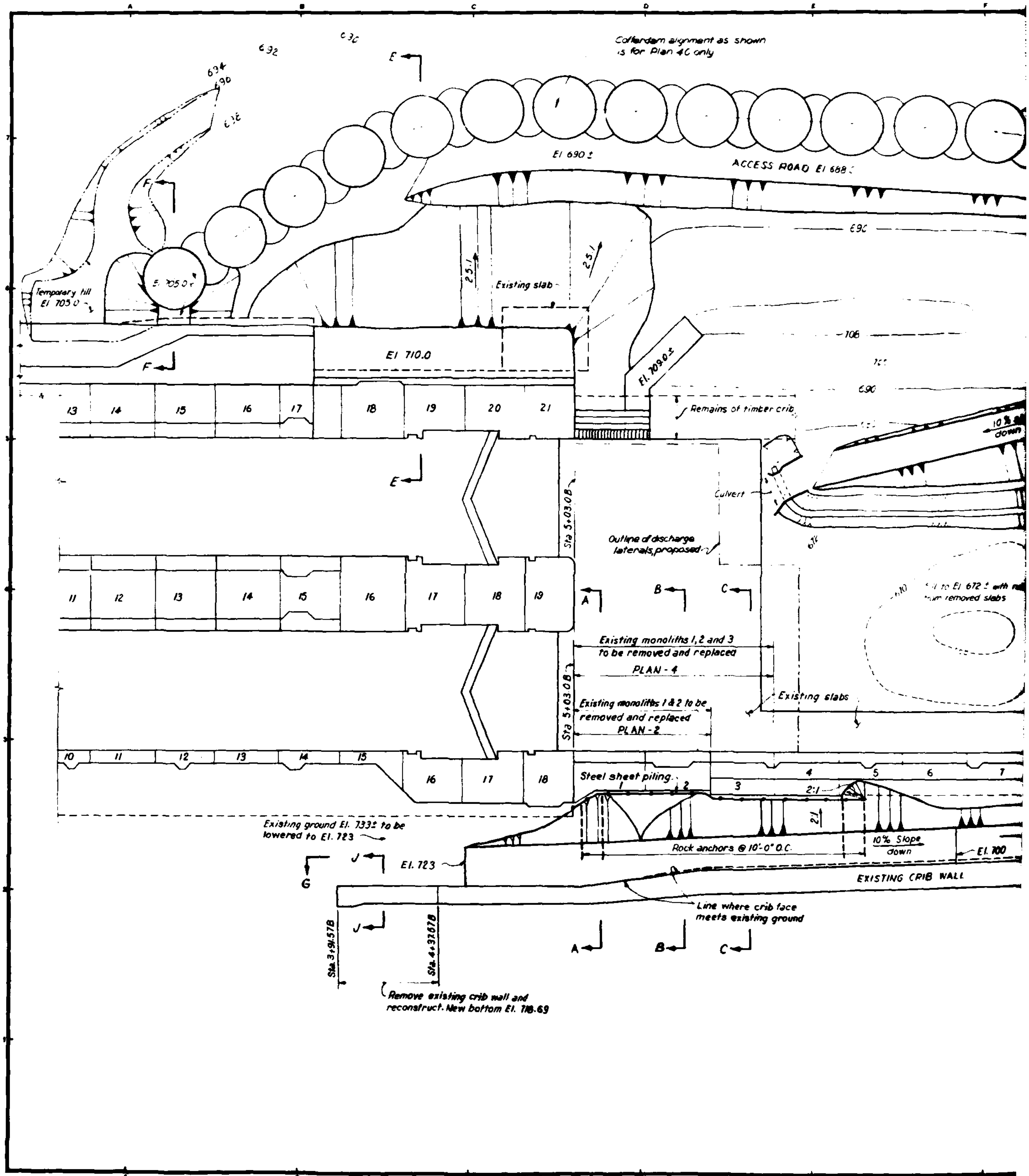
DATE: MARCH 1975

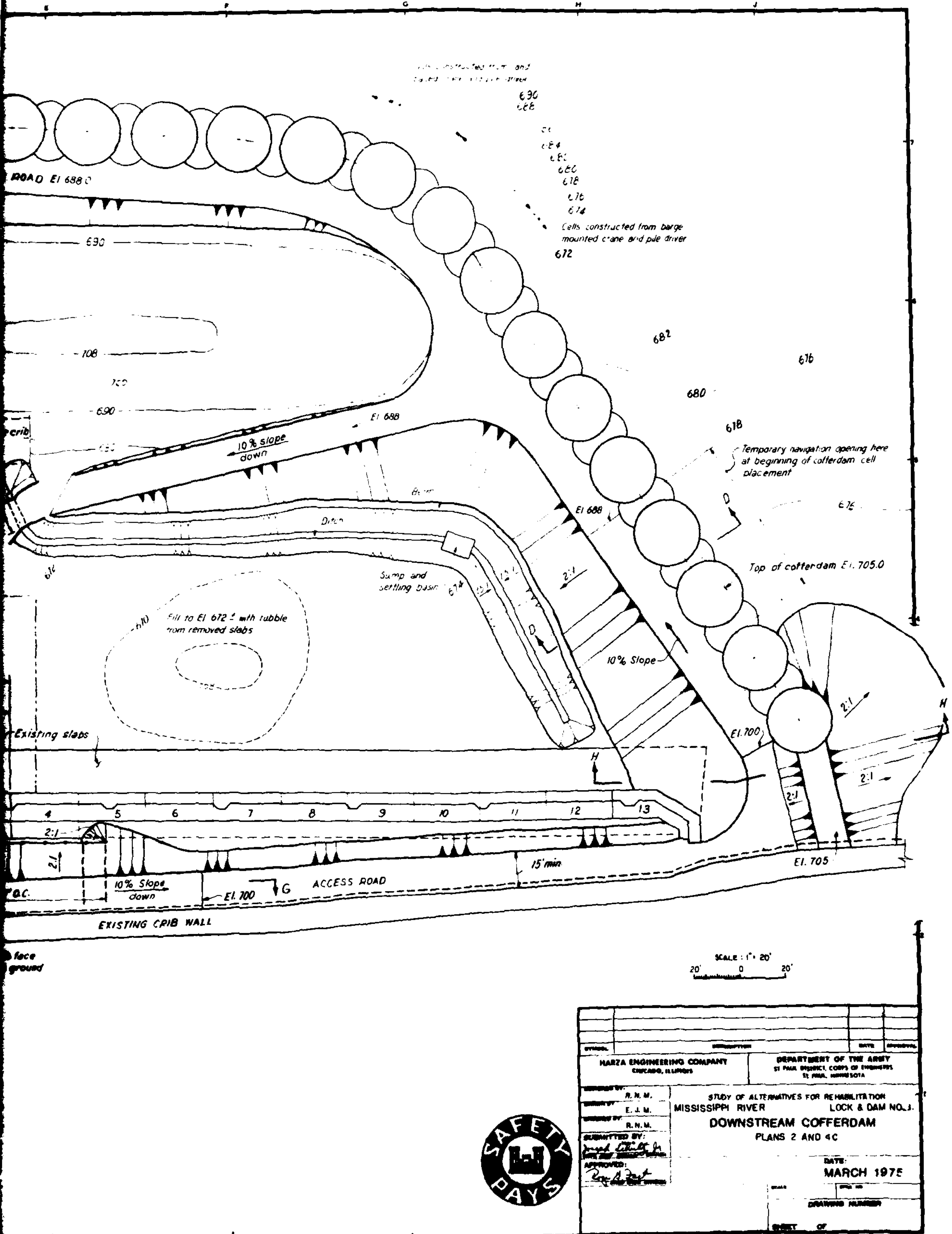
DRAWING NUMBER

SHEET OF

PLATE E-2

2


Cofferdam alignment as shown is for Plan 4C only
ACCESS ROAD El 688±
El 690±
Temporary fill El 705.0±
El 705.0±
Existing slab
El 710.0
El 709.0±
Remains of timber crib
Culvert
Sta 5+03.0B
Outline of discharge laterals, proposed
Existing monoliths 1, 2 and 3 to be removed and replaced
PLAN - 4
Existing slabs
Existing monoliths 1 & 2 to be removed and replaced
PLAN - 2
Steel sheet piling
Existing ground El. 733± to be lowered to El. 723
El. 723
Rock anchors @ 10'-0" O.C.
10% Slope down
El. 700
EXISTING CRIB WALL
Line where crib face meets existing ground
Sta 3+91.57B
Sta 4+32.87B
Remove existing crib wall and reconstruct. New bottom El. 718.69



ROAD El 688.0
Cells constructed from barge mounted crane and pile driver
Temporary navigation opening here at beginning of cofferdam cell placement
Top of cofferdam El. 705.0
10% slope down
Sump and settling basin
Fill to El 672 ± with rubble from removed slabs
Existing slabs
10% Slope
ACCESS ROAD
EXISTING CRIB WALL
El. 700
El. 705
15' min
SCALE : 1" = 20'
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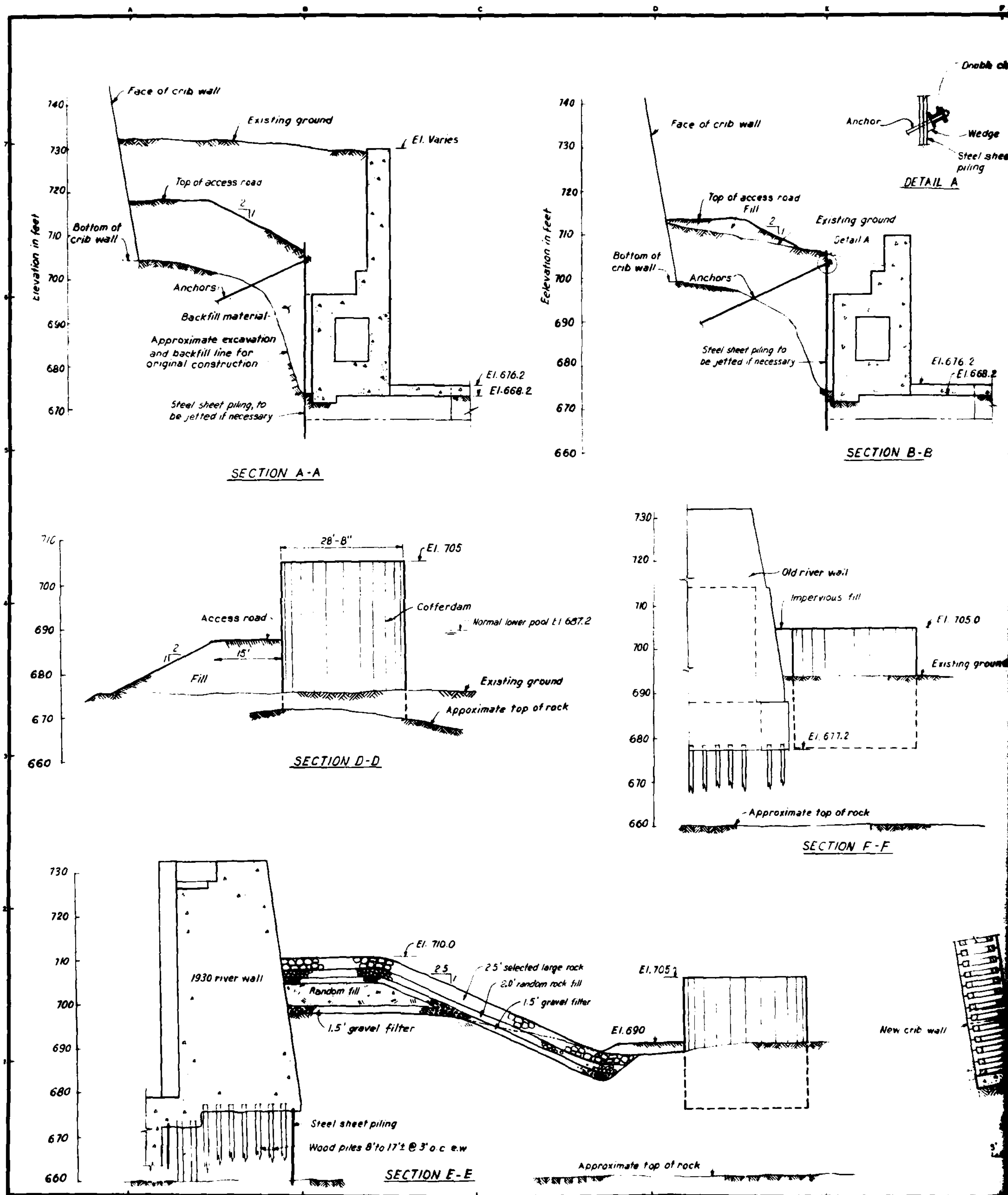
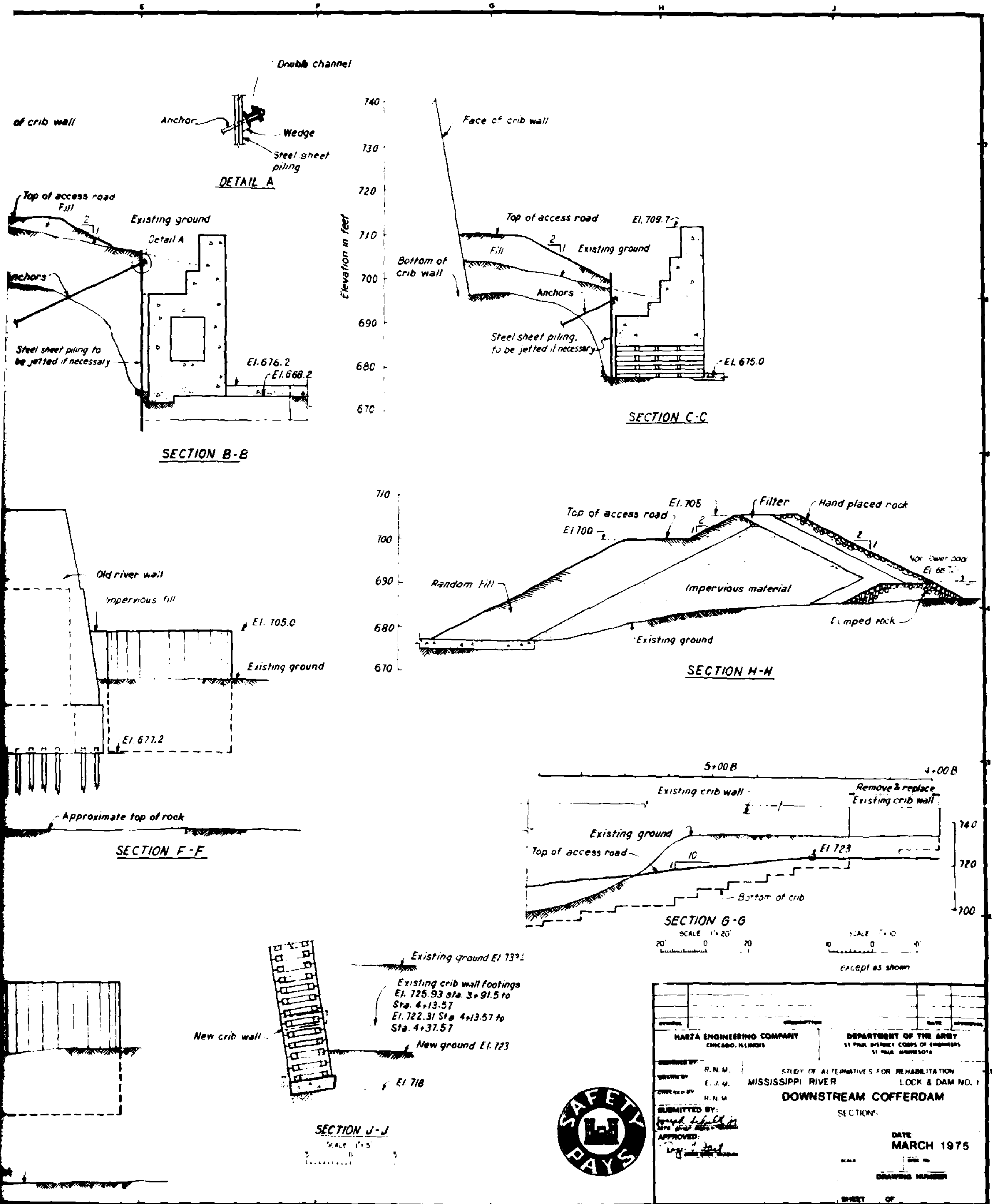
ST. PAUL, MINNESOTA
STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER
LOCK & DAM NO. 3
DOWNSTREAM COFFERDAM
PLANS 2 AND 4C
DATE: MARCH 1975
SAFETY PAYS

PLATE E-3



Face of crib wall
Existing ground
El. Varies
Top of access road
Bottom of crib wall
Elevation in feet
Anchors
Backfill material
Approximate excavation and backfill line for original construction
Steel sheet piling, to be jetted if necessary
El. 676.2
El. 668.2
SECTION A-A
Face of crib wall
Top of access road
Fill
Existing ground
Detail A
Bottom of crib wall
Anchors
Eelevation in feet
Steel sheet piling to be jetted if necessary
El. 676.2
El. 668.2
SECTION B-B
Anchor
Wedge
Steel sheet piling
DETAIL A
28'-8"
El. 705
Cofferdam
Normal lower pool El 687.2
Access road
15'
Fill
Existing ground
Appoximate top of rock
SECTION D-D
Old river wall
Impervious fill
El. 705.0
Existing ground
El. 677.2
Approximate top of rock
SECTION F-F
El. 710.0
2.5' selected large rock
2.0' random rock fill
1.5' gravel filter
1930 river wall
Random fill
1.5' gravel filter
El. 705
El. 690
New crib wall
Steel sheet piling
Wood piles 8' to 17'± @ 3' o.c. e.w.
Approximate top of rock
SECTION E-E



Double channel
Anchor
Wedge
Steel sheet piling
DETAIL A
of crib wall
Top of access road
Fill
Existing ground
Detail A
Anchors
Steel sheet piling to be jetted if necessary
El. 676.2
El. 668.2
SECTION B-B
Face of crib wall
Top of access road
El. 709.7
Fill
Existing ground
Bottom of crib wall
Anchors
Steel sheet piling, to be jetted if necessary
El. 675.0
Elevation in feet
SECTION C-C
El. 705
Filter
Hand placed rock
Top of access road
El 700
Random Fill
Impervious material
Existing ground
SECTION H-H
Old river wall
Impervious fill
El. 705.0
Existing ground
El. 677.2
Approximate top of rock
SECTION F-F
5+00B
4+00B
Existing crib wall
Remove & replace Existing crib wall
Existing ground
Top of access road
El 723
Bottom of crib
SECTION G-G
except as shown
Existing ground El 739±
Existing crib wall footings El. 725.93 sta. 3+91.5 to Sta. 4+13.57 El. 722.31 Sta 4+13.57 to Sta. 4+37.57
New crib wall
New ground El. 723
El 718
SECTION J-J
SAFETY PAYS
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DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT CORPS OF ENGINEERS
ST. PAUL, MINNESOTA
R.N.M.
E.J.M.
R.N.M.
SUBMITTED BY:
APPROVED:
STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO. 1
DOWNSTREAM COFFERDAM
SECTIONS
DATE
MARCH 1975
DRAWING NUMBER
SHEET OF

PLATE E-4

DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT, CORPS OF ENGINEERS
1210 U.S. Post Office & Custom House
St. Paul, Minnesota 55101

MISSISSIPPI RIVER
STUDY OF ALTERNATIVES FOR REHABILITATION OF LOCK AND DAM NO.1
MINNEAPOLIS, MINNESOTA

APPENDIX F
CONCRETE EVALUATION STUDY

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TABLE OF CONTENTS (Continued)

PLATES (Continued)

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REPORTS (Continued)

2 R. C. Mielenz, Petrographic Examination of Concrete Core Specimens, Lock and Dam No. 1, Mississippi River, October 15, 1974, Introduction and Conclusions.

3 R. C. Mielenz, Petrographic Examination of Concrete Core Specimens, Lock and Dam No. 1, Mississippi River, December 9, 1974, Introduction and Conclusions.

4 Letter dated December 16, 1974 with attached laboratory data on "Glas-Con."

Appendix F

CONCRETE EVALUATION STUDY

I. Programs of Evaluation

A. Core Drilling. Core drilling of the concrete lock structure was performed in conjunction with a portion of the geologic and foundation exploration program. A total of 14 vertical holes were drilled through the lock monoliths, and 11 vertical holes were drilled through the land lock and river lock floor slabs. Each drill hole was logged with detailed descriptions of the concrete core taken. The locations of drill holes are shown on Plate 1. The geologic log of each drill hole can be found in Appendix C, *Geology, Foundation and Soils Investigation Program*.

B. Compressive Strength Tests.1/ 2/ Compressive strength tests were made in two stages on selected concrete core specimens. In Stage I, four drill holes were selected representative of the various portions of the structure and age of the concrete. A section of NX or HQ core was taken from the upper, center, and lower part of the monolith at each location. Two core specimens were cut and tested from each section or interval. The drill holes, relative locations, period of construction, and depth intervals tested are listed below:

Drill Hole No.	Location	Period of Construction	Depth Interval, ft.
74-35	Intermediate Wall	(1930)	3.4-4.5
			24.5-26.0
			58.0-59.3

1/ "In-place Concrete Strength Evaluation - A Recommended Practice," National Ready Mixed Concrete Association, Publication No. 133.

2/ ACI Report Title No. 65-67, Guide for Making a Condition Survey of Concrete in Service, ACI Manual of Concrete Practices, Part I, p. 201-39.

Drill Hole No.	Location	Period of Construction	Depth Interval, ft.
74-42	River Wall	(1930)	7.5-8.7
			20.4-22.0
			53.6-55.0
74-48	River Wall	(1900-1908)	0.2-1.9
			25.5-26.5
			48.1-49.9
74-50	Land Wall	(1931-1932)	2.0-3.5
			21.7-23.3
			41.5-42.6

The results of compressive strength tests for Stage I are contained on Plate 2.

Stage II involved selection and testing of NX and HQ core sections from areas which were described as drummy, and possibly unsound, in the geologic drill hole logs. Two specimens were tested from each suspected area. In addition, three 6-inch core specimens were tested for correlation with the NX and HQ compressive strengths. The drill holes, locations, depth, and core size are listed below:

Drill Hole No.	Location	Depth, ft.	Core Size
74-34	Intermediate Wall	0.7	6 inch
		2.85	6 inch
		4.7	6 inch
74-35	Intermediate Wall	3.4	NX
		6.2	HQ
		7.4	HQ
		8.5	HQ
74-46	River Wall	10.4	NX
		11.5	NX
		25.7	NX
		26.2	NX

Drill Hole No.	Location	Depth, ft.	Core Size
74-48	River Wall	33.1	NX
		35.0	NX
74-50	Land Wall	33.4	NX
		34.8	NX
		58.0	NX
		59.1	NX

The results of compressive strength tests for Stage II are contained on Plate 3.

C. Petrographic Examination of Core Specimens. Petrographic examination was performed on random core specimens from three vertical drill holes and on individual core specimens taken by portable concrete core machine from the vertical faces of the land lock walls. All specimens were submitted to Richard C. Mielenz, Geologist and Petrographer, located in Gates Milles, Ohio. Specimens were examined visually and under stereoscopic microscope. Thin sections were prepared on portions of the specimens and examined by petrographic microscope. Composition, degree of cement hydration, alkali-aggregate reactivity, effects of bleeding, microcracking, chemical attack, and soundness were noted. A total of three reports were written by Mr. Mielenz. The introduction and conclusion of each report has been attached to this appendix.

Initially, a total of 26 - 2 to 3 inch random NX core specimens from drillholes 74-48 (River Wall - 1900 to 1908), 74-39 (Intermediate Wall 1930), and 74-50 (Land Wall - 1931 and 1932) were submitted for examination of interior concrete. A sufficient number of specimens were examined to represent the full thickness of concrete at each location. The results are contained in Report No. 1 dated August 31, 1974, attached.

Four 6-inch specimens were then drilled horizontally by portable concrete core machine from the vertical face of the land lock walls to examine the exterior concrete surfaces. These were submitted to determine the cause of visually observed deterioration and the depth to sound concrete in the land lock chamber. Three of the core specimens were taken from areas of deterioration along the land wall. The fourth was taken for comparison in an area which appeared sound in the intermediate wall. Locations of the horizontal core holes are shown on Plates 4 and 7. The results of examination are contained in Report No. 2, dated October 15, 1974, attached.

Following a review of Report No. 2, an additional ten 4-1/4 inch horizontal core specimens were drilled into the land lock walls in Monolith No. 12, land wall, and Monolith No. 13, intermediate wall, to further investigate deterioration of the land lock walls. These cores were submitted for examination to determine if surface deterioration tended to increase from top to bottom of the walls. Five cores were taken in each monolith at distances from the top of the wall of 7.0, 15.5, 24.0, 32.5, and 41.0 feet. The locations of the core holes are shown on Plates 5 and 8. The results of the examination are contained in Report No. 3, dated December 9, 1974, attached.

D. Superficial Condition Survey. A condition survey was made on the exposed surfaces of the locks including vertical and sloping faces, tops of walls, and upper and lower guide walls during the fall of 1974. A visual inspection of the interior and exterior of the dam was also made during the same period and the findings are contained in Appendix K, Dam Inspection. The condition survey of the locks consisted of mapping cracking, spalling and scaling, repair work, and other observations along with a written assessment of each monolith. Two methods were used in completing the survey: 1) the top surfaces were visually inspected, mapped and described, and 2) the mapping and descriptions of vertical and sloping surfaces were made from close-up 8" x 10" photographs of each monolith. This survey updates the periodic inspection conducted last by the Corps of Engineers in August, 1967.[3/] The results of the survey are shown on Plates 4 through 9.

In addition to the mapping and assessment of the surface conditions, Swiss Impact Hammer or rebound hammer readings were taken in several locations to obtain values of comparative compressive strength. Twelve impact hammer readings were taken on the surface at each test location. All readings were obtained with the instrument held in a horizontal position. In general, the

3/ Periodic Inspection Program - Lock and Dam No. 1, U.S. Corps of Engineers, August 1967, 7 drawings.

area selected for testing was a smooth formed surface free of scaling or spalling. The highest and lowest value of the twelve readings were discarded and the remaining readings were averaged. The compressive strength for each location was determined using the average reading and the calibration curve that was furnished with the instrument. First, the vertical surface where each gate anchor is embedded, was tested. The results are shown on Plate 12. Second, the face of Monoliths No. 12 and No. 13, land wall, was tested at several locations horizontally and vertically. The results in the areas tested are shown on Plate 13.

II. Appraisal of Concrete in Lock Walls

A. Compressive Strength. The overall compressive strength for the 38, NX and HQ, core specimens tested averaged 5251 psi and ranged from 2595 psi to 9420 psi. The compressive strength tended to vary between portions of the structure depending on the period construction took place. The river wall constructed during the period of 1900 to 1908, average 4468 psi/12 cores. The adjacent river wall section and intermediate wall constructed in 1930, averaged 5888 psi/6 cores and 5892 psi/10 cores, respectively. The land wall constructed during 1931-1932, averaged 5166 psi/10 cores. As far as individual drill holes, Drill hole 74-46 (river wall) had the lowest average compressive strength at 3389 psi/4 cores.

A limited compressive strength comparison was made between large diameter 6 inch core, and the smaller diameter NX and HQ core, used more exclusively during exploration. Three 6-inch core specimens from Drill hole 74-34 were tested and compared with one NX and two HQ core specimens from Drill hole 74-35. Both drill holes were within the same monolith and the specimens were taken at the same approximate elevation. The 6 inch core averaged 6415 psi, and the NX and HQ core averaged 5577 psi or approximately 13 percent lower.

In general, the compressive strength results obtained from the core tests indicate existing strength levels far above that assumed for the original design or required for modification of the structure.

B. Interior Concrete. Petrographic examination along with compressive strength results on selected samples indicate the interior concrete in all walls is sound and in good structural condition.4/ Cement-aggregate reactivity was found in nearly all core specimens checked; however, the reaction is essentially complete. Microcracking of the concrete matrix from the formation of potentially expansive reaction products was rare. Cement hydration was essentially complete and no inherent weakness in the cement-aggregate bond was observed. Some significant bleeding was found in core specimens from the river wall (1900-1908) which resulted in variation in porosity and firmness of the cement paste matrix, but no significant cracking, sulfate attack or leaching was found. Therefore, no evidence was apparent in the interior of the structure which might suggest any concrete distress.

C. Exterior Concrete. The exterior surfaces of the lock walls generally show the effects of many years of weathering. Surface distress and deterioration can be readily seen in most areas which have been subjected to saturation and many cycles of freezing and thawing. Unfortunately, the structure was constructed prior to the discovery that entrainment of air in the concrete improved its resistance to freeze-thaw damage. The surfaces most seriously affected are the top and edges of the walls and slabs. These have been maintained by replacing concrete in local areas on a continual basis. Many of the repairs are unbonded and also showing signs of distress. Repair work and its current condition have been noted to some extent on Plates 4 through 11.

The depth of deterioration on exposed horizontal surfaces appears variable between monoliths, probably due to such factors as variation in concrete quality, exposure, and maintenance. Some indication of depth can be seen along the top edge of walls where longitudinal cracking or "D-cracking" and leaching varies between monoliths from minor to severe. In local areas, longitudinal or "D-cracking" extends 1.5 to 2.0 feet down from the top of the wall. Vertical drill holes did not show significant evidence of deep deterioration but in a few locations did indicate laminar cracking at depths exceeding one foot. For estimating purposes, the average depth of deterioration was assumed at 3 inches with an allowance of one inch for overbreak.

4/ Mielenz, R. C., "Diagnosing Concrete Failures," Stanton Walker Lecture Series on the Materials Sciences, University of Maryland, November, 1964.

Freeze-thaw damage is also evident on the vertical and sloping faces of the lock walls where spalling along joints between monoliths, and intermediate scaling, can be readily observed at low water level. Damage appears more severe in the land lock than the river lock. An estimated 3700 square feet of spalling and scaling has occurred in the land lock chamber as compared to 770 square feet in the river lock chamber. These estimates were based on planimetering the actual areas mapped on the condition survey. Generally speaking, the vertical faces of monoliths 2, 3, 4, 9, and 10 of the land wall and monoliths 6 and 10 of the intermediate wall - land lock showed the most noticeable increase in surface scaling between the 1967 condition survey and the current survey. Greater damage can be expected in the land lock since it is more frequently used. Deterioration in the form of laminar cracking probably exists over a considerably larger area but has not progressed to the point of surface scaling. This fact was evident from petrographic examination of horizontal cores taken from the lock walls.

Freeze-thaw damage is more pronounced toward lower pool level. This can be expected since the surfaces in these areas are saturated for longer periods. Rebound hammer readings shown on Plate 11 tend to confirm a decrease in the quality of the concrete surface from the top to bottom of the walls.

A total of 14 horizontal cores were taken in the land lock chamber to investigate the depth of deterioration. Results indicated a wide variation in depth of deterioration which does not necessarily relate to surface appearance. Five core specimens showed no deterioration other than surface crazing. The remaining 9 specimens showed deterioration varying from 11/16" to 10-1/2". In general, the depth of deterioration was most severe in core speciments taken adjacent to spalled areas and in the lower portion of the lock chamber. Furthermore and noteworthy, only 1 of the 14 cores was drilled in an area where scaling had already occurred. The depth of deterioration here was found to be only approximately 11/16".

Sulfate attack was observed in practically all specimens, evident by the presence of ettringite and secondary deposits of gypsum. This suggests the concrete was exposed at some time to fairly high concentrations of sulfate in the river water which chemically attacked and altered the portland cement matrix. It was believed that the sulfate attack was a secondary effect occurring by means of increased surface porosity in the areas damaged by freezethaw cycles. Chemical attack was not found in sound concrete. No water quality samples were taken nor was any water quality information obtained and evaluated during this investigation. The river water should be examined for potential chemical attack on concrete in any future work. Consideration should be given to the use of more sulfate resistant Type II or Type V portland cement for any repair work which may be in contact with sulfate bearing waters.

Neglecting minor surface crazing, the average depth to sound concrete on the specimens examined was 4.8 inches. For estimating purposes, the depth to sound concrete over all vertical surfaces was assumed to be 4.4 inches and 1 inch was added for overbreak.

D. Cracking. In addition to the longitudinal cracking or "D-cracking" observed in conjunction with freeze-thaw damage, vertical cracking has occurred primarily in the river lock on the face of the river wall. This cracking probably relates to differential shrinkage between the original river wall (1900-1908) and the more recent section added in 1930. Vertical cracking is more common in the lower portion of the lock chamber where the construction joint was made with the original concrete. Vertical joints between monoliths were not consistent with the original joint pattern which may have aggravated the differential shrinkage and induced additional stress. Vertical and random cracking measured approximately 55 linear feet in the land lock and approximately 550 linear feet in the river lock. The cracking does not appear to present any serious problem, but rehabilitation should include epoxy grouting of cracking to assure monolithic action and sealing of the structure.

Minor hairline transverse shrinkage cracks were observed at regular intervals on the top surface of the land wall. These did not appear to extend to any depth and were not apparent along the wall face.

E. Joints. Vertical joints between monoliths which can be visually inspected appear tight for the most part. The exceptions are in the river wall; the joint between Monoliths R-1 and R-2 is open about 1" at the top; the joint between Monoliths R-13 and R-14 is open about 3/8" at the top, and the joint between Monoliths R-17 and R-18 is open about 3/8" at the top. Other work has shown leakage is occurring through the vertical joints in the intermediate wall during the filling operation for the land lock. Also, if the joint is open water loss may be occurring between the intermediate wall and the river lock slab. There was no evidence of any sealing material in joints. Therefore, to control water loss and seepage, rehabilitation work should include effective sealing of all vertical joints between monoliths, the joints and cracks in the lock slabs, and joints in the interior culverts between monoliths.

Minor seepage, as indicated by leaching, can be seen at a few horizontal construction joints. The conditions are not serious but these joints should be treated as cracks and grouted with epoxy to eliminate any further deterioration.

III. Recommendation for Rehabilitation of the Locks

A. Recommended Repairs and Reconditioning

1. Resurfacing. The vertical and horizontal surface deterioration should be removed to sound concrete and the surfaces replaced in order to check deterioration and extend the service life of the existing structure. Furthermore, the remaining surfaces, while not currently showing distress, may not have satisfactory resistance for extended freeze-thaw action. Therefore, these surfaces should receive a coating or resurfacing to provide protection from progressive weathering. Overall resurfacing will not only provide durable surfaces but will result in an esthetically pleasing appearance.

Since the exposed sloping face of the river wall is not critical with regard to traffic, abrasion resistance, or progressive deterioriation, removal of existing deterioration should not be required. A concrete slab on a pea gravel bedding placed directly over a clean sloping wall surface would provide satisfactory protection and appearance. The capacity of the pea gravel bedding to drain quickly should keep drain holes clean and eliminate any potential frost problem under the slabs. Precast panels could be used in place of cast-in-place slabs to further reduce costs. Plate 14 shows a general scheme for covering the exposed river wall. Each slab panel can be anchored to sound concrete for support and for installing to line and grade.

2. Grouting Cracks and Joints. Vertical cracks and horizontal construction joints which show leaching should be

pressure grouted with epoxy resin to restore integrity of the monoliths and seal cracked surfaces from moisture. Shrinkage, for all practical purposes, is complete; therefore, progressive cracking of the existing concrete is unlikely. Provisions for expansion and contraction should be maintained where necessary.

3. Sealing Joints. Water loss and seepage should be contained by sealing of the vertical joints on the wall face between monoliths, the vertical joints and cracks in the lock floor slabs, and the joints in interior culverts between monoliths. An elastomeric sealing compound, marine grade two-component polysulfide, can be easily installed in formed grooves in the vertical joints and a routed groove in the lock slabs. The sealing compound is available in both self-leveling and non-sag types. Backing behind the sealing compound should be preformed sponge rubber or cork joint filler material. The resistance to aging and weathering of polysulfide joint sealing compound is high in comparison with other field-molded sealing materials available. However, the service life will depend on the service conditions and whether any defects are built in at the time of sealing. Plate 15 shows typical details of the joint sections recommended.

B. Cost Estimates

1. Alternate Studies. Cost estimates were prepared from the recommendations for repairs and reconditioning. Four alternates were studied considering either partial or complete rehabilitation of the structure. These are listed below:

Alternates	Description
I. Rehabilitation of Land Lock Only - Plan I, II and III	
"A"	Provide a concrete slab over the exposed face of the river wall. Resurface the vertical surfaces of the land lock and the top surfaces of all walls. Grout cracks in the land lock only. Seal vertical joints in the land wall and intermediate wall. Seal horizontal joints and cracks in the land lock floor slab and adjacent to the intermediate wall in the river lock slab. Seal

conduit joints in the land wall and intermediate wall land lock conduit.

"B" Resurface the vertical surfaces of the land lock and the top surfaces of all walls. Grout cracks in the land lock only. Seal vertical joints in the land wall and intermediate wall. Seal horizontal joints and cracks in the land lock floor slab and adjacent to the intermediate wall in the river lock slab. Seal conduit joints in the land wall and intermediate wall land lock conduit.

"C" Resurface the vertical surfaces of the land lock and the top surfaces of the land wall and intermediate wall only. Grout cracks in the land lock only. Seal vertical joints in the land wall and intermediate wall. Seal horizontal joints and cracks in the land lock floor slab and adjacent to the intermediate wall in the river lock slab. Seal conduit joints in the land wall and intermediate wall land lock conduit.

II. Rehabilitation of Both Locks - Plan IV

"D" Provide a concrete slab over the exposed face of the river wall. Resurface the vertical surfaces of the land lock and river lock, and the top surfaces of all walls. Grout cracks in the land lock and river lock. Seal vertical joints in the land wall, intermediate wall and river wall. Seal horizontal joints and cracks in the land lock floor slab and adjacent to the intermediate wall in the river lock slab. Seal conduit joints in the land wall, intermediate wall, and river wall.

2. Resurfacing Materials. In addition to the alternates, two methods of resurfacing vertical faces were compared. The methods were resurfacing by conventional shotcrete and resurfacing by glass fiber reinforced, latex modified, portland cement mortar.[5/]

Shotcrete has been extensively used for repair of deteriorated concrete in hydraulic structures and is an accepted repair method.[6/] Shotcrete has excellent bond properties when the surfaces have been properly prepared. Shotcrete has good strength and durability due to the low water-cement ratio generally used, but does have slightly higher shrinkage values than low-slump concrete. Good curing practices and a minimum of reinforcement are generally required to control shrinkage and shrinkage cracking. The assumptions made for resurfacing vertical surfaces of Lock and Dam No. 1 by shotcrete are listed below:

a. Shotcrete repair would extend from slightly above the filling and emptying ports in the lock walls to 1.5 feet from the top of the walls. No surface repair is anticipated for concrete continuously submerged below the top of the ports and in the lock chamber slab which has not been exposed to weathering.

b. The average depth of concrete removal and replacement over all vertical surfaces would be 5.4 inches, including a one inch overbreak and a minimum depth of removal of 3 inches.

c. Shotcrete would be reinforced with 4x4-6/6 welded wire fabric, held in position by dowels anchored in the old concrete.

d. The repaired surface would be screeded and would receive a float finish.

e. Top surfaces would be removed and replaced with conventional concrete as shown on Plate 14.

5/ ACI Report Title No. 59-57, Durability of Concrete in Service, ACI Manual of Concrete Practices, Part I, p. 201-1.

6/ ACI 506-66, Recommended Practice for Shotcreting, ACI Manual of Concrete Practices, Part III, p. 506-1.

Glass fiber reinforced latex modified portland cement mortar is a product referred to as "Glas-Con." The materials are furnished and applied by Glas-Crete Corporation in Bloomington, Minnesota. The product has no history of use in hydraulic structures but has recognized potential application in this area. The attached Report No. 4 contains information and certified laboratory test data on "Glas-Con." Latex modified portland cement mortars have a proven history of excellent bonding characteristics and performance. The addition of 5% by volume of the glass fibers improve the strength and impact resistance of the material. However, "Glas-Con" may have a tendency to seal moisture within the concrete which could result in loss of bond when the entrapped moisture freezes. This problem has been experienced with epoxy coatings due to lack of flexibility of the coating material.[1/] The flexibility of latex mortar is substantially better than epoxy mortars, therefore, durability should be greatly improved. A laboratory study would be necessary to confirm the durability, should "Glas-Con" be incorporated into the work. Providing the concrete surface is sound, bond should not be influenced by the age, type or existing moisture condition of the concrete on which "Glas-Con" will be applied. Requirements for surface treatment, and the actual preparation of surfaces, will be the most important single item in achieving the necessary bond and a durable coating.

The estimated cost of a 3/8" thick application of "Glas-Con" is $3.25/s.f. which equals approximately $2,830 per cubic yard. Therefore, full replacement of deterioration up to 10-1/2 inches in depth could not be economically justified. In lieu of replacement, the cost estimate is based on repairing only the deteriorated areas with shotcrete applied to bring the surfaces to line and grade. Then, a 3/8" thick protective coating of "Glas-Con" could be applied uniformly over the vertical surfaces. The assumptions made for repairing deterioration with shotcrete and coating the vertical surfaces with "Glas-Con" are listed below:

a. Shotcrete repair would consist of replacing deterioration over an area three times that of mapped areas of spalling and scaling. This area was selected because shallow horizontal core specimens were examined from the land lock walls only, where deterioration appears most severe, and because the deepest deterioration occurred adjacent to the locations of scaling and spalling.

1/ Strassburger, A.G., "Determination of Concrete Properties in Old Dams," ASCE Proceedings, Inspection, Maintenance and Rehabilitation of Old Dams, Pacific Grove, California, September, 1973, p. 477.

b. The average depth of concrete removed and replaced with shotcrete would be 6.0 inches, 3 inch minimum removal, since removal and replacement will be localized in areas of deterioration only.

c. Shotcrete would be reinforced with 4x4-6/6 welded wire fabric held in position by dowels anchored in the old concrete.

d. A 3/8" thickness of "Glas-Con" would be applied to all vertical surfaces extending from slightly above the filling and emptying ports in the lock walls to the top of the walls. No surface repair is anticipated for concrete continuously submerged such as below the top of the ports and in the lock chamber slab which have not been exposed to weathering.

e. All surfaces to receive "Glas-Con" would be prepared by sand blasting.

f. Top surfaces would be removed and replaced with conventional concrete as shown on Plate 16.

A third method of resurfacing the vertical walls, that of placing conventional concrete by slipform was checked. The rate of concrete placement, as controlled by the setting time of the concrete, was 0.7 cy/hr. for a 6 inch thickness, 30 feet wide. No cost estimate was prepared since the placing rate would be far below that of shotcrete, and no other benefits could be realized.

3. Results. The results of cost analysis are summarized below:

	Estimated Cost	
Alternates	Shotcrete	Glas-Con
"A"	$589,340.00	$614,800.00
"B"	$506,750.00	$537,740.00
"C"	$416,900.00	$479,360.00
"D"	$852,795.00	$876,825.00

Note: Joint sealing not included

The following table was prepared to show the cost differential for Alternate D, recognizing that deterioration may be more extensive than assumed for the "Glas-Con" estimate. These results are as follows:

	Alternate D	
Deterioration with Respect to Existing Areas of Scaling & Spalling 1/	Differential Cost of "Glas-Con" vs. Shotcrete	Percent Cost Increase
3 times	+$ 24,030.00	2.8
6 times	+$ 86,030.00	10.1
9 times	+$148,030.00	17.4
12 times	+$210,030.00	24.6

1/ As the area of deterioration is increased the quantities of concrete removal and shotcrete replacement, prior to "Glas-Con" application, are increased.

Calculated quantities and itemized cost estimates are contained on Plate 17 (Shotcrete) and Plate 18 (Glas-Con). The final estimate for repair of concrete surfaces can be found on page 16, Item 10, of Appendix A, and reflects the use of shotcrete as the most economical method and material for resurfacing of vertical faces.

4. Wall Armor. The final cost estimate was prepared covering the installation of wall armor similar to that used at St. Anthony Falls, Lower Lock and Dam on the Mississippi River. Areas protected included the upper and lower guidewalls, and the entrance to and exit from the lock chambers. The total increased estimated cost for furnishing and installing wall armor in Plans 1, 2, and 3 was $478,000, and in Plan 4 was $655,000, as shown on Plates 17 and 18. These costs generally reflect the difficulty of installing and anchoring the wall armor on an existing structure as opposed to lower costs if the installation was done during initial construction. The increased costs include removal of 16 inches of concrete for anchorage of the armor, additional anchors into old concrete, formwork, and concrete replacement, in addition to the cost of the armor and armor anchors. The wall armor does

not appear economically justifiable, considering the minimal visual evidence of serious damage caused by barges hitting the walls weighing the cost, but has been included in the final estimate for rehabilitation.

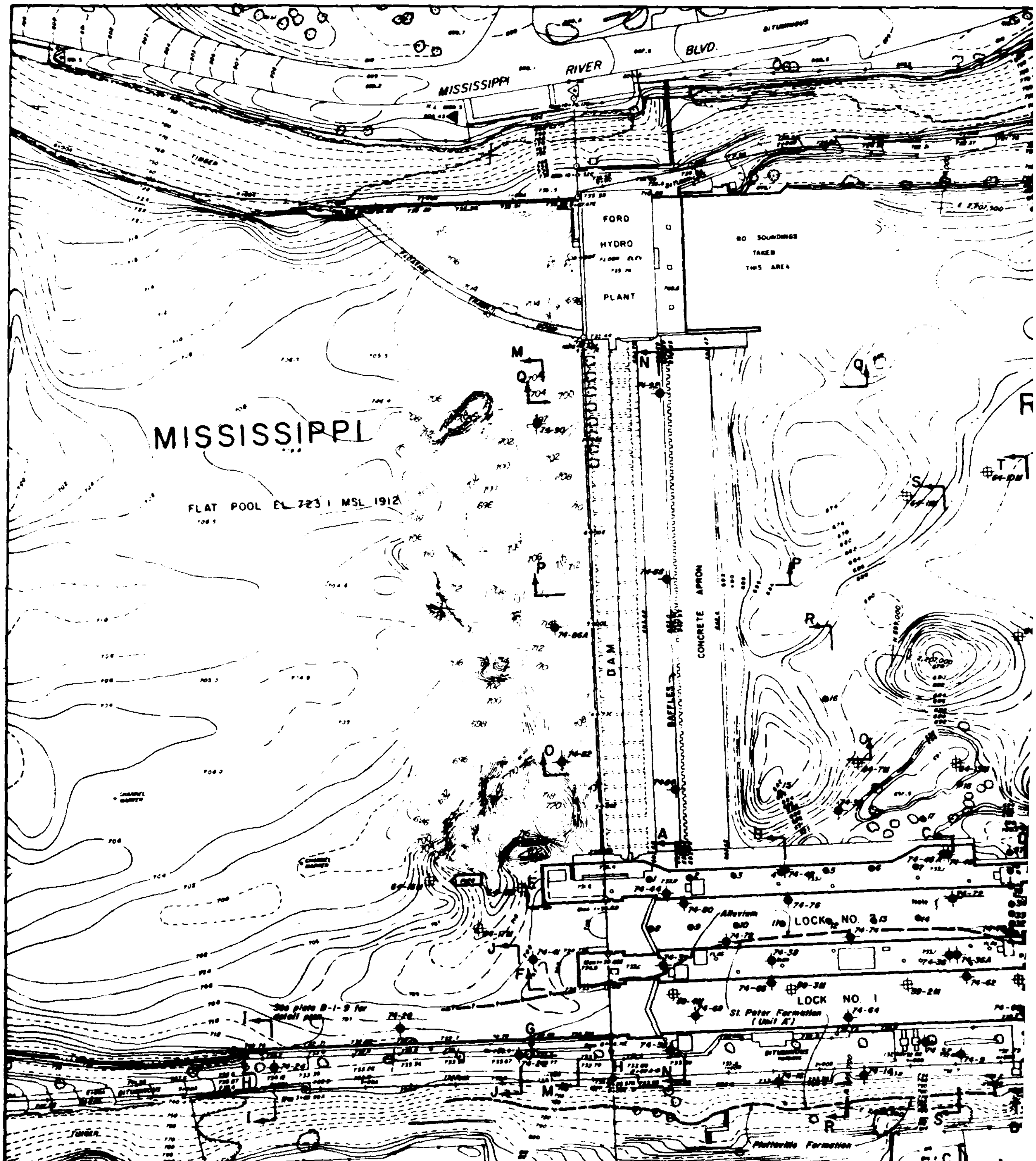
IV. Recommendations for Futher Study

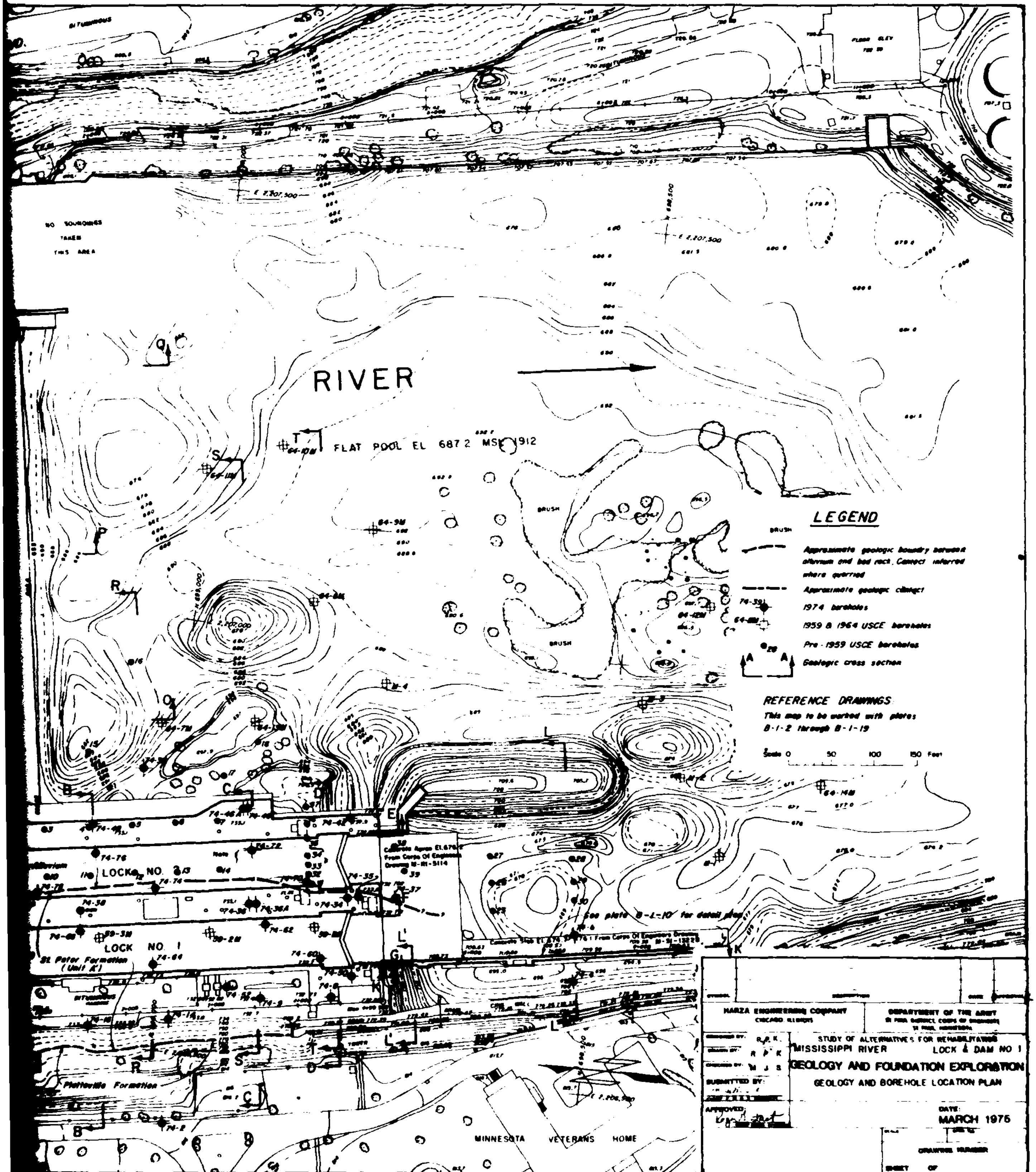
1. Further study is needed to more accurately define the depth of surface deterioration. The concrete evaluation study has shown that removal and resurfacing of exposed concrete will be the major item of concrete work required in rehabilitating the structure. The values for depth of concrete removal and replacement, applied to the overall structure for cost estimating were based on a study of core specimens taken from the land lock walls, only. Visual observations suggest that deterioration may vary greatly between walls. Therefore, the drilling and petrographic examination of sixteen additional core specimens is recommended to further define the depth of deterioration in various areas. The suggested general locations and number of core specimens to be taken from each location are listed below:

Upper Guide Wall	- 2
Lower Guide Wall	- 2
Top of Land Wall	- 2
Top of Intermediate Wall	- 2
Top of River Wall	- 4
River Lock Chamber	- 4
Total	-16

2. River water quality should be studied for potential chemical attack on concrete. Water analysis would indicate what provisions may be required to assure resistant concrete.

3. Future developments in the repair of concrete should be studied for potential application for this project.


MISSISSIPPI RIVER BLVD.
FORD HYDRO PLANT
NO SOUNDINGS TAKEN THIS AREA
MISSISSIPPI
FLAT POOL EL 723.1 MSL 1912
DAM
BAFFLES
CONCRETE APRON
Alluvium
LOCK NO. 2
LOCK NO. 1
St. Peter Formation (Unit A')
See plate B-1-9 for detail plan.
Platteville Formation
MINNEHAHA PARK



RIVER
FLAT POOL EL 687.2 MSL 1912
NO SOUNDINGS TAKEN THIS AREA
BRUSH
LEGEND
Approximate geologic boundary between alluvium and bed rock. Contact inferred where queried
Approximate geologic contact
1974 boreholes
1959 & 1964 USCE boreholes
Pre-1959 USCE boreholes
Geologic cross section
REFERENCE DRAWINGS
This map to be worked with plates B-1-2 through B-1-19
Scale 0 50 100 150 Feet
LOCK NO. 2
LOCK NO. 1
St. Peter Formation (Unit K)
Platteville Formation
MINNESOTA VETERANS HOME
HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS
DEPARTMENT OF THE ARMY
STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER
LOCK & DAM NO 1
GEOLOGY AND FOUNDATION EXPLORATION
GEOLOGY AND BOREHOLE LOCATION PLAN
R.P.K.
R P K
M J S
SUBMITTED BY:
APPROVED:
DATE:
MARCH 1975
DRAWING NUMBER
SHEET OF

PLATE F-1

1

Concrete Compression Test

Report To HARZA ENGINEERING COMPANY Date [illegible]
Address 150 SOUTH WACKER DRIVE, CHICAGO, ILLINOIS [illegible]
Project LOCK & DAM #1, ST. PAUL, MINNESOTA
Contractor ______________ File No. [illegible]

LABORATORY NUMBER:					
LOCATION OF POUR:		SC-1 Top	SC-1 Bot.	SC-2 Top	SC-2 Bot
SAMPLE IDENTIFICATION NUMBER:		1	2	3	4
BORING:		74-48	74-48	74-48	74-48
DEPTH:		0'-2 1/2" to 1'-11"		25'-5 1/2" to 26'-5"	
DATE TESTED:		7-23-74	7-23-74	7-23-74	7-23-74
ELEVATION TOP OF SAMPLE:		732.5 Ft.		707.25 Ft.	
DIAMETER (INCHES):	MINIMUM	2.03	2.03	2.03	2.03
	MAXIMUM	2.03	2.03	2.03	2.03
HEIGHT (INCHES):	BEFORE CAPPED	4.34	4.03	4.22	4.00
	CAPPED	4.56	4.39	4.48	4.36
L/D RATIO (% OF STANDARD):		2.25	2.16	2.21	2.15
CROSS-SECTION AREA (SQ. IN.):		3.23	3.23	3.23	3.23
LOAD AT FAILURE (LBS.):		19,925	22,050	16,050	12,500
COMPRESSIVE STRENGTH (PSI):		6,170	6,825	4,970	3,870
REQUIRED STRENGTH (PSI)					
UNIT WEIGHT	WT. GRAMS	542.30	497.30	545.52	513.61
	UNIT WT.	146.6	144.9	151.9	150.8

SLUMP: AIR CONTENT:
MIX DESIGN: ADMIXTURE:
CONCRETE SUPPLIER: SAMPLED BY:
METHOD OF LABORATORY CURING: Immersed in saturated lime water. ASTM C192-69, Sec. 6.3
REMARKS:

LABORATORY NUMBER:					
LOCATION OF POUR:		SC-3 Top	SC-3 Bot.	SC-4 Top	SC-4 Bot
SAMPLE IDENTIFICATION NUMBER:		5	6	7	8
BORING:		74-48	74-48	74-42	74-42
DEPTH:		48'-3" to 49'-10"		7'-6" to 8'-9"	
DATE TESTED:		7-23-74	7-23-74	7-23-74	7-23-74
ELEVATION TOP OF SAMPLE:		684.6 Ft.		725.2 Ft.	
DIAMETER (INCHES):	MINIMUM	2.11	2.11	2.47	2.47
	MAXIMUM	2.13	2.13	2.47	2.47
HEIGHT (INCHES):	BEFORE CAPPED	4.23	4.22	4.97	5.19
	CAPPED	4.58	4.72	5.25	5.5[illegible]
L/D RATIO (% OF STANDARD):		2.17	2.24	2.13	2.23
CROSS-SECTION AREA (SQ. IN.):		3.50	3.50	4.7[illegible]	4.79
LOAD AT FAILURE (LBS.):		16,500	15,100	29,20[illegible]	27,700
COMPRESSIVE STRENGTH (PSI):		4,715	4,315	6,095	5,780
REQUIRED STRENGTH (PSI)					
UNIT WEIGHT	WT. GRAMS	607.82	610.72	972.66	959.66
	UNIT WT.	156.3	157.7	155.7	153.1

SLUMP: AIR CONTENT:
MIX DESIGN: ADMIXTURE
CONCRETE SUPPLIER: SAMPLED BY
METHOD OF LABORATORY CURING: Immersed in saturated lime water. ASTM C192-69 Sec. 6.3
REMARKS: Sample #5 small pocket of green decomposed shale on top end of core

LABORATORY NUMBER:					
LOCATION OF POUR:		SC-7 Top	SC-7 Bot.	SC-8 Bot.	SC-8 Top
SAMPLE IDENTIFICATION NUMBER:		13	14	15	16
BORING:		74-35	74-35	74-35	74-35
DEPTH:		3'-4 1/2" to 4'-6 1/2"		24'-5" to 25'-11"	
DATE TESTED:		7-23-74	7-23-74	7-23-74	7-23-74
ELEVATION TOP OF SAMPLE:		729.3 Ft.		708.3 Ft.	
DIAMETER (INCHES):	MINIMUM	2.13	2.13	2.47	2.47
	MAXIMUM	2.13	2.13	2.47	2.47
HEIGHT (INCHES):	BEFORE CAPPED	4.41	4.41	5.06	4.91
	CAPPED	4.75	4.75	5.31	5.06
L/D RATIO (% OF STANDARD):		2.24	2.24	2.15	2.05
CROSS-SECTION AREA (SQ. IN.):		3.56	3.56	4.79	4.79
LOAD AT FAILURE (LBS.):		21,600	23,400	29,000	35,500
COMPRESSIVE STRENGTH (PSI):		6,065	6,575	6,055	7,410
REQUIRED STRENGTH (PSI)					
UNIT WEIGHT	WT. GRAMS	639.68	630.62	978.99	964.55
	UNIT WT.	156.8	153.6	153.8	156.3

SLUMP: AIR CONTENT
MIX DESIGN: ADMIXTURE:
CONCRETE SUPPLIER: SAMPLED BY:
METHOD OF LABORATORY CURING: Immersed in saturated lime water. ASTM C192-69 Sec. 6.3
REMARKS: Sample #14 has pocket of green decomposed shale.

LABORATORY NUMBER:					
LOCATION OF POUR:		SC-9 Top	SC-9 Bot.	SC-10 Top	SC-10 Bot
SAMPLE IDENTIFICATION NUMBER:		17	18	19	20
BORING:		74-35	74-35	74-50	74-50
DEPTH:		58'-0" to 59'-4"		2'-0" to 3'-6"	
DATE TESTED:		7-23-74	7-23-74	7-23-74	7-23-74
ELEVATION TOP OF SAMPLE:		676.7 Ft.		730.7 Ft.	
DIAMETER (INCHES):	MINIMUM	2.47	2.46	2.13	2.13
	MAXIMUM	2.47	2.47	2.13	2.13
HEIGHT (INCHES):	BEFORE CAPPED	5.18	5.12	4.31	4.09
	CAPPED	5.37	5.20	4.62	4.43
L/D RATIO (% OF STANDARD):		2.18	2.15	2.18	2.29
CROSS-SECTION AREA (SQ. IN.):		4.79	4.67	3.56	3.56
LOAD AT FAILURE (LBS.):		36,000	44,000	14,000	12,150
COMPRESSIVE STRENGTH (PSI):		7,515	9,420	3,930	3,410
REQUIRED STRENGTH (PSI)					
UNIT WEIGHT	WT. GRAMS	1027.14	1009.75	611.4[illegible]	576.[illegible]
	UNIT WT.	157.4	160.2	152.2	151.2

SLUMP: AIR CONTENT
MIX DESIGN: ADMIXTURE
CONCRETE SUPPLIER: SAMPLED BY
METHOD OF LABORATORY CURING: Immersed in saturated lime water. ASTM C192-69 Sec. 6.3
REMARKS:

TESTED by

SOIL & MATERIALS ENGINEERS, INC.
7900 77th AVENUE NORTH MINNEAPOLIS MINNESOTA 55445

By [signature]

Concrete Compression Test

Report To HARZA ENGINEERING COMPANY Date [illegible]
Address 150 SOUTH WACKER DRIVE, CHICAGO, ILLINOIS [illegible]
Project LOCK & DAM, #1, ST. PAUL, MINNESOTA
Contractor File No. [illegible]

LABORATORY NUMBER:					
LOCATION OF POUR:		SC-3 Top	SC-3 Bot.	SC-4 Top	SC-4 Bot
SAMPLE IDENTIFICATION NUMBER:		5	6	7	8
BORING:		74-48	74-48	74-42	74-42
DEPTH:		48'-1" to 49'-10"		7'-6" to 8'-9"	
DATE TESTED:		7-23-74	7-23-74	7-23-74	7-23-74
ELEVATION TOP OF SAMPLE:		684.6 ft.		725.2 ft.	
DIAMETER (INCHES):	MINIMUM	2.11	2.11	2.47	2.47
	MAXIMUM	2.13	2.13	2.47	2.47
HEIGHT (INCHES):	BEFORE CAPPED	4.23	4.22	4.97	5.19
	CAPPED	4.58	4.72	5.25	5.5
L/D RATIO (% OF STANDARD):		2.17	2.24	2.13	2.23
CROSS-SECTION AREA (SQ. IN.):		3.50	3.50	4.79	4.79
LOAD AT FAILURE (LBS.):		16,500	15,100	29,200	27,700
COMPRESSIVE STRENGTH (PSI):		4,715	4,315	6,095	5,780
REQUIRED STRENGTH (PSI)					
UNIT WEIGHT	WT. GRAMS	607.82	610.77	972.66	998.66
	UNIT WT.	156.3	157.7	155.7	153.1

SLUMP: AIR CONTENT:
MIX DESIGN: ADMIXTURE:
CONCRETE SUPPLIER: SAMPLED BY:
METHOD OF LABORATORY CURING: Immersed in saturated lime water. ASTM C192-69. Sec. 6.3
REMARKS: *Sample #5 small pocket of green decomposed shale on top end of core*

LABORATORY NUMBER.					
LOCATION OF POUR		SC-5 Top	SC-5 Bot.	SC-6 Top	SC-6 Bot
SAMPLE IDENTIFICATION NUMBER:		9	10	11	12
BORING:		74-42	74-42	74-42	74-42
DEPTH:		20'-5" to 22'-0"		53'-8" to 55'-0"	
DATE TESTED:		7-23-74	7-23-74	7-23-74	7-23-74
ELEVATION TOP OF SAMPLE		712.3 ft		679.05 ft.	
DIAMETER (INCHES):	MINIMUM	2.47	2.47	2.47	2.47
	MAXIMUM	2.47	2.47	2.47	2.47
HEIGHT (INCHES):	BEFORE CAPPED	5.37	5.31	5.19	5.06
	CAPPED	5.62	5.50	5.44	5.25
L/D RATIO (% OF STANDARD):		2.28	2.23	2.20	2.13
CROSS-SECTION AREA (SQ. IN.):		4.79	4.79	4.79	4.79
LOAD AT FAILURE (LBS.):		23,600	22,750	22,000	44,000
COMPRESSIVE STRENGTH (PSI):		4,925	4,750	4,590	9,185
REQUIRED STRENGTH (PSI)					
UNIT WEIGHT	WT. GRAMS	1051.99	1046.88	1031.13	1007.41
	UNIT WT.	155.6	156.7	158.0	158.2

SLUMP: AIR CONTENT:
MIX DESIGN: ADMIXTURE:
CONCRETE SUPPLIER: SAMPLED BY:
METHOD OF LABORATORY CURING: Immersed in saturated lime water. ASTM C192-69. Sec. 6.3
REMARKS:

LABORATORY NUMBER:					
LOCATION OF POUR:		SC-9 Top	SC-9 Bot.	SC-10 Top	SC-10 Bot.
SAMPLE IDENTIFICATION NUMBER:		17	18	19	20
BORING:		74-35	74-35	74-50	74-50
DEPTH:		58'-0" to 59'-4"		2'-0" to 3'-6"	
DATE TESTED:		7-23-74	7-23-74	7-23-74	7-23-74
ELEVATION TOP OF SAMPLE:		674.7 ft.		730.7 ft.	
DIAMETER (INCHES):	MINIMUM	2.47	2.46	2.13	2.13
	MAXIMUM	2.47	2.47	2.13	2.13
HEIGHT (INCHES):	BEFORE CAPPED	5.18	5.12	4.31	4.09
	CAPPED	5.37	5.20	4.62	4.43
L/D RATIO (% OF STANDARD):		2.18	2.15	2.18	2.09
CROSS-SECTION AREA (SQ. IN.):		4.79	4.67	3.56	3.56
LOAD AT FAILURE (LBS.):		36,000	44,000	14,000	12,150
COMPRESSIVE STRENGTH (PSI):		7,515	9,420	3,930	3,410
REQUIRED STRENGTH (PSI)					
UNIT WEIGHT	WT. GRAMS	1027.14	1009.75	611.40	576.77
	UNIT WT.	157.4	160.7	152.2	151.7

SLUMP: AIR CONTENT:
MIX DESIGN: ADMIXTURE:
CONCRETE SUPPLIER: SAMPLED BY:
METHOD OF LABORATORY CURING: Immersed in saturated lime water. ASTM C192-69. Sec. 6.3
REMARKS:

LABORATORY NUMBER:					
LOCATION OF POUR:		[illegible]	SC-11 Bot.	SC-12 Top	SC-12 Bot.
SAMPLE IDENTIFICATION NUMBER:		21	22	23	24
BORING:		74-50	74-50	74-50	74-50
DEPTH:		21'-8" to 23'-4"		41'-6" to 42'-7"	
DATE TESTED:		7-23-74	7-23-74	7-23-74	7-23-74
ELEVATION TOP OF SAMPLE:		711.05 ft.		691.2 ft.	
DIAMETER (INCHES):	MINIMUM	2.13	2.13	2.13	2.13
	MAXIMUM	2.13	2.13	2.13	2.13
HEIGHT (INCHES):	BEFORE CAPPED	4.41	4.21	4.25	4.31
	CAPPED	4.75	4.47	4.62	4.56
L/D RATIO (% OF STANDARD):		2.24	2.10	2.18	2.15
CROSS-SECTION AREA (SQ. IN.):		3.56	3.56	3.56	3.56
LOAD AT FAILURE (LBS.):		20,600	20,400	22,500	17,500
COMPRESSIVE STRENGTH (PSI):		5,785	5,730	6,320	4,915
REQUIRED STRENGTH (PSI)					
UNIT WEIGHT	WT. GRAMS	654.19	639.43	646.83	626.33
	UNIT WT.	159.3	159.1	163.3	155.0

SLUMP: AIR CONTENT:
MIX DESIGN: ADMIXTURE:
CONCRETE SUPPLIER: SAMPLED BY:
METHOD OF LABORATORY CURING: Immersed in saturated lime water. ASTM C192-69. Sec. 6.3
REMARKS:

TESTED by

SOIL & MATERIALS ENGINEERS, INC.

By [signature]

SAFETY PAYS

SYMBOL	DESCRIPTION	DATE	APPROVAL

HARZA ENGINEERING COMPANY
CHICAGO ILLINOIS

DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT CORPS OF ENGINEERS
ST. PAUL, MINNESOTA

DESIGNED BY R M C/G R M
DRAWN BY R M C
CHECKED BY G R M
SUBMITTED BY [signature]
APPROVED [signature]

STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO. 1
CONCRETE CONDITION SURVEY
SUMMARY OF COMPRESSION TEST RESULTS
FALL 1974

DATE
MARCH 1975

DRAWING NUMBER

SHEET 1 OF 2

PLATE F-2

1

2

Concrete Compression Test

Report To: HARZA ENGINEERING COMPANY   Date: 9-5-74
Address: 150 SOUTH WACKER DRIVE, CHICAGO, ILLINOIS 60606
Project: LOCK & DAM #1, ST. PAUL, MINNESOTA
Contractor: ______   File No.: 90360

LABORATORY NUMBER:				
LOCATION OF POUR:	SC-13 Top	SC-13 Bot.	SC-14 Top	SC-14 Bot.
SAMPLE IDENTIFICATION NUMBER:	25	26	27	28
BORING:	74-46	74-46	74-48	74-48
DEPTH:	25.7'	26.2'	33.1'	35.0'
DATE TESTED:	9-4-74	9-4-74	9-4-74	9-4-74
ELEVATION TOP OF SAMPLE:				
DIAMETER (INCHES):	2.09"	2.09"	2.03"	2.03"
HEIGHT (INCHES): BEFORE CAPPED	4.1"	4.2"	4.15"	4.15"
HEIGHT (INCHES): AFTER CAPPED	4.35"	4.5"	4.4"	4.4"
L/D RATIO (% OF STANDARD):	1.96	2.01	2.04	2.04
CROSS-SECTION AREA (SQ. IN.):	3.429	3.429	3.235	3.235
LOAD AT FAILURE (LBS.):	8,900 lbs.	11,500 lbs	14,850 lbs.	14,900 lbs.
COMPRESSIVE STRENGTH (PSI):	2,595 psi	3,355 psi	4,590 psi	4,605 psi
REQUIRED STRENGTH (PSI)				
UNIT WEIGHT: WT. GRAMS	547.75	563.10	517.64	527.83
UNIT WEIGHT: UNIT WT.	148.11 pcf	148.6 pcf	146.6 pcf	149.5 pcf

SLUMP: AIR CONTENT:
MIX DESIGN: ADMIXTURE:
CONCRETE SUPPLIER: SAMPLED BY:
METHOD OF LABORATORY CURING: Immersed in saturated lime water. ASTM C192-69. Sec. 6.3
REMARKS:

LABORATORY NUMBER:				
LOCATION OF POUR:	SC-15 Top	SC-15 Bot.	SC-16 Top	SC-16 Bot
SAMPLE IDENTIFICATION NUMBER:	29	30	31	32
BORING:	74-46	74-46	74-50	74-50
DEPTH:	10.4'	11.5'	58.0'	59.1'
DATE TESTED:	9-4-74	9-4-74	9-4-74	9-4-74
ELEVATION TOP OF SAMPLE:				
DIAMETER (INCHES):	2.03"	2.03"	2.03"	2.03"
HEIGHT (INCHES): BEFORE CAPPED	4.15"	3.95"	4.175"	4.15"
HEIGHT (INCHES): AFTER CAPPED	4.65"	4.3"	4.5"	4.5'
L/D RATIO (% OF STANDARD):	2.04	1.94	2.06	2.04
CROSS-SECTION AREA (SQ. IN.):	3.235	3.235	3.235	3.235
LOAD AT FAILURE (LBS.):	11,100 lbs.	13,500 lbs.	16,700 lbs.	17,700 lbs
COMPRESSIVE STRENGTH (PSI):	3,430 psi	4,175 psi	5,160 psi	5,470 psi
REQUIRED STRENGTH (PSI)				
UNIT WEIGHT: WT. GRAMS	525.19	509.41	548.13	535.97
UNIT WEIGHT: UNIT WT.	148.7 pcf	151.6 pcf	154.3 pcf	151.8 pcf

SLUMP: AIR CONTENT:
MIX DESIGN: ADMIXTURE:
CONCRETE SUPPLIER: SAMPLED BY:
METHOD OF LABORATORY CURING: Immersed in saturated lime water ASTM C192-69. Sec. 6.3
REMARKS:

LABORATORY NUMBER:				
LOCATION OF POUR:				
SAMPLE IDENTIFICATION NUMBER:	35	36	37	38
BORING:	74-35	74-35	74-35	74-35
DEPTH:	3.4'	6.2'	7.4'	8.5'
DATE TESTED:	9-11-74	9-11-74	9-11-74	9-11-74
ELEVATION TOP OF SAMPLE:				
DIAMETER (INCHES):	2.12"	2.47"	2.47"	2.47"
HEIGHT (INCHES): BEFORE CAPPED	4.20"	4.91"	4.94"	4.91"
HEIGHT (INCHES): AFTER CAPPED	4.62"	5.44"	5.5"	5.5"
L/D RATIO (% OF STANDARD):	2.0	1.99	2.0	1.99
CROSS-SECTION AREA (SQ. IN.):	3.54	4.79	4.79	4.79
LOAD AT FAILURE (LBS.):	14,500 lbs.	18,500 lbs.	21,500 lbs	16,500 lbs.
COMPRESSIVE STRENGTH (PSI):	4,090 psi	3,860 psi	4,490 psi	3,445 psi
REQUIRED STRENGTH (PSI)				
UNIT WEIGHT: WT. GRAMS	612.2	926.08	959.16	931.33
UNIT WEIGHT: UNIT WT.	153.3 pcf	149.7 pcf	154.1 pcf	150.6 pcf

SLUMP: AIR CONTENT:
MIX DESIGN: ADMIXTURE:
CONCRETE SUPPLIER: SAMPLED BY:
METHOD OF LABORATORY CURING: Immersed in saturated lime water. ASTM C192-69. Sec. 6.3
REMARKS:

LABORATORY NUMBER:
LOCATION OF POUR:
SAMPLE IDENTIFICATION NUMBER:
BORING:
DEPTH:
DATE TESTED:
ELEVATION TOP OF SAMPLE:
DIAMETER (INCHES):
HEIGHT (INCHES): BEFORE CAPPED / AFTER CAPPED
L/D RATIO (% OF STANDARD):
CROSS-SECTION AREA (SQ. IN.):
LOAD AT FAILURE (LBS.):
COMPRESSIVE STRENGTH (PSI):
REQUIRED STRENGTH (PSI)
UNIT WEIGHT: WT. GRAMS / UNIT WT.
SLUMP:
MIX DESIGN:
CONCRETE SUPPLIER:
METHOD OF LABORATORY CURING: Immersed ... Sec. 6.3
REMARKS:

TESTED by

SOIL & MATERIALS ENGINEERS, INC

By [signature]

ete Compression Test

[illegible] ENGINEERING COMPANY Date 9-5-74
[illegible] WACKER DRIVE, CHICAGO, ILLINOIS 60606
[illegible] DAM #1, ST. PAUL, MINNESOTA
File No. 30160

	SC-15 Top	SC-15 Bot.	SC-16 Top	SC-16 Bot.
[illegible]:	29	30	31	32
	74-46	74-46	74-50	74-50
	[illegible].4'	11.5'	58.0'	59.1'
	9-4-74	9-4-74	9-4-74	9-4-74
	2.03"	2.03"	2.03"	2.03"
[illegible]	4.15" 4.40"	3.95" 4.3"	4.175" 4.5"	4.15" 4.5"
[illegible]):	2.04	1.94	2.06	2.04
[illegible].):	3.235	3.235	3.235	3.235
	11,100 lbs.	13,500 lbs.	16,700 lbs.	17,700 lbs.
[illegible]:	3,430 psi	4,175 psi	5,160 psi	5,470 psi
[illegible]	525.19 148.7 pcf	509.41 151.6 pcf	548.13 154.3 pcf	535.97 151.8 pcf

AIR CONTENT:

ADMIXTURE:

SAMPLED BY:

[illegible]: Immersed in saturated lime water. ASTM C192-69. Sec. 6.3

LABORATORY NUMBER:		
LOCATION OF POUR:	SC-17 Top	SC-17 Bot.
SAMPLE IDENTIFICATION NUMBER:	33	34
BORING:	74-50	74-50
DEPTH:	33.4'	34.8'
DATE TESTED:	9-4-74	9-4-74
ELEVATION TOP OF SAMPLE:		
DIAMETER (INCHES):	2.12"	2.12"
HEIGHT (INCHES): BEFORE CAPPED / AFTER CAPPED	4.19" 4.5"	4.20" 4.5"
L/D RATIO (% OF STANDARD):	1.95	1.97
CROSS-SECTION AREA (SQ. IN.):	3.545	3.545
LOAD AT FAILURE (LBS.):	12,600 lbs.	26,200 lbs.
COMPRESSIVE STRENGTH (PSI):	3,555 psi	7,390 psi
REQUIRED STRENGTH (PSI)		
UNIT WEIGHT: WT. GRAMS / UNIT WT.	603.69 156.4 pcf	607.37 155.1 pcf

SLUMP: AIR CONTENT:

MIX DESIGN: ADMIXTURE:

CONCRETE SUPPLIER: SAMPLED BY:

METHOD OF LABORATORY CURING: Immersed in saturated lime water. ASTM C192-69. Sec. 6.3

REMARKS:

LABORATORY NUMBER:			
LOCATION OF POUR:			
SAMPLE IDENTIFICATION NUMBER:	39	40	41
BORING:	74-34	74-34	74-34
DEPTH:	~~0.7~~ 1.7	~~2.85~~ 3.85	~~4.7~~ 5.8
DATE TESTED:	9-11-74	9-11-74	9-11-74
ELEVATION TOP OF SAMPLE:			
DIAMETER (INCHES):	5.94"	5.94"	5.94"
HEIGHT (INCHES): BEFORE CAPPED / AFTER CAPPED	11.94" 12.25"	11.97" 12.37"	11.00" 11.37"
L/D RATIO (% OF STANDARD):	2.0	2.0	1.85
CROSS-SECTION AREA (SQ. IN.):	27.7	27.7	27.7
LOAD AT FAILURE (LBS.):	168,000 lbs.	184,000 lbs.	181,000 lbs.
COMPRESSIVE STRENGTH (PSI):	6,065 psi	6,645 psi	6,535 psi
REQUIRED STRENGTH (PSI)			
UNIT WEIGHT: WT. GRAMS / UNIT WT.	13,350 153.6 pcf	13,570 155.6 pcf	12,035 150.2 pcf

SLUMP: AIR CONTENT:

MIX DESIGN: ADMIXTURE:

CONCRETE SUPPLIER: SAMPLED BY:

METHOD OF LABORATORY CURING: Immersed in saturated lime water. ASTM C192-69. Sec. 6.3

REMARKS:

TESTED by

[illegible] & [illegible]ERIALS [illegible]NEERS, INC.

By [illegible signature]

SAFETY PAYS

SYMBOL	DESCRIPTION	DATE	APPROVAL

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT CORPS OF ENGINEERS
ST. PAUL, MINNESOTA

DESIGNED BY: G R M
DRAWN BY: R M C
CHECKED BY: G R M
SUBMITTED BY: [signature]
APPROVED: [signature]

STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO 1

CONCRETE CONDITION SURVEY

SUMMARY OF COMPRESSION TEST RESULTS
FALL 1974

DATE: MARCH 1975

SCALE

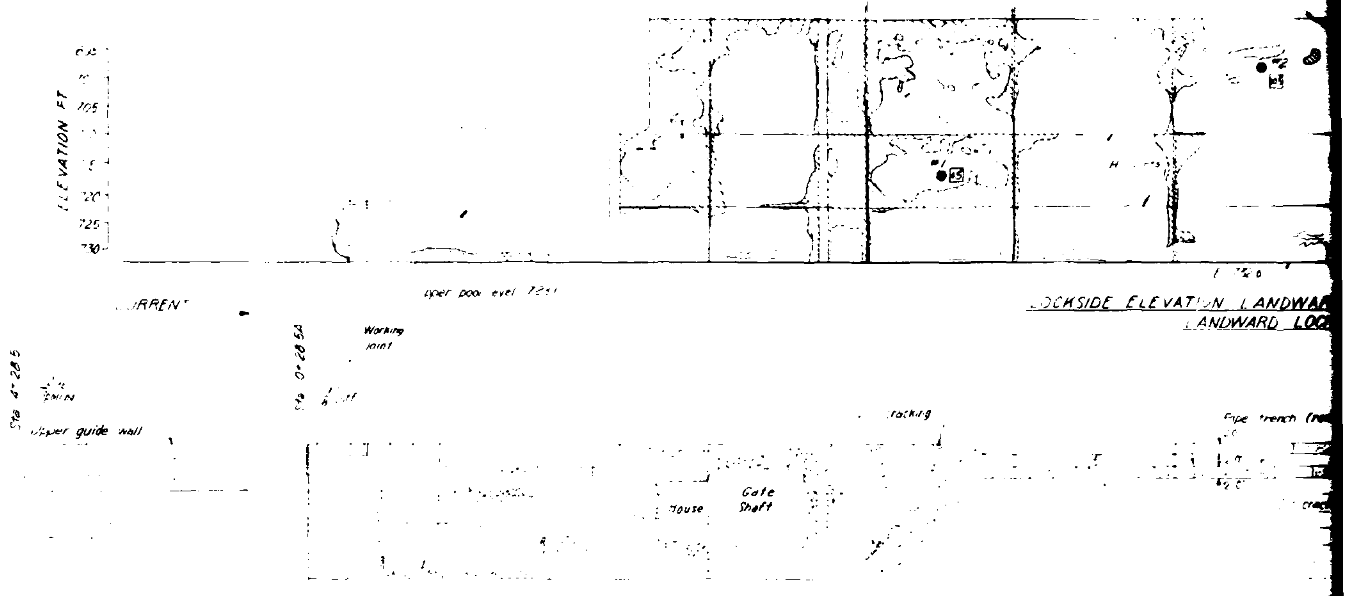
DRAWING NUMBER

SHEET 2 OF 2

PLATE F-3

1

2



CURRENT

LOCKSIDE ELEVATION LANDWARD
LANDWARD LOCK

Top of wall el 730

LANDWARD ELEVATION - LANDWARD

CONCRETE CONDITION	UPPER GUIDE WALL	MONOLITH NO 1	2	3	4	5	6
CRACKING	Minor transverse and hairline	None	None	Longitudinal hairline cracks on the top face of the wall	D-cracking on top surface along edge. Longitudinal cracks along the edge of wall	[illegible] Minor transverse [illegible] surface	Moderate [illegible] joint
SCALING AND SPALLING	Very light to medium on horizontal surfaces	Light scaling	Severe scaling and spalling along downstream vertical joint edge. 60% of top surface has been repaired	Severe scaling along the vertical joints and edges with medium scaling at lowest horizontal construction joint. 65% of top surface repaired	Severe scaling along the vertical joints and edges with medium scaling at lowest horizontal construction joint. 65% of the top surface repaired	Moderate scaling [illegible] vertical [illegible] 55% of the top surface [illegible] along top surface edge unrepaired area	Severe scaling along vertical [illegible] lower portion of the wall. 70% of top surface repaired. [illegible] replaced [illegible] light scaling in other [illegible]
JOINTS (VERTICAL)	Generally good condition	Leakage along horizontal construction joint	See above. Vertical joints tight	See above. Vertical joints tight	See above. Vertical joints tight	See above. Vertical joints tight	See above. Vertical joints tight.
OTHER	Fractured hole in Monolith #8 at waterline appears to be approximately 2 feet in depth	Top river edge repaired along with edges of utility raceway.	Minor leaching on lower portion of face wall in small individual areas	Leaching along upper edge of wall and in local areas at the bottom of wall. Top surface drummy in unrepaired areas	Considerable leaching [illegible] of the wall. Some leaching from horizontal cracking [illegible] upper edge of the wall	Moderate leaching [illegible] longitudinal [illegible] edge of the wall. Minor leaching [illegible] areas along the bottom of wall	Considerable leaching from longitudinal cracks below [illegible] and in local areas at the bottom of wall

LEGEND

[illegible]

Replaced Concrete

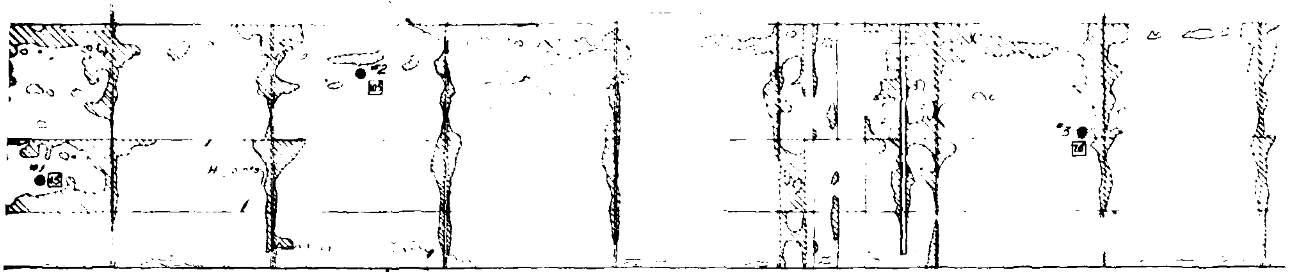
[illegible]

● Core hole

[#] [illegible] designation

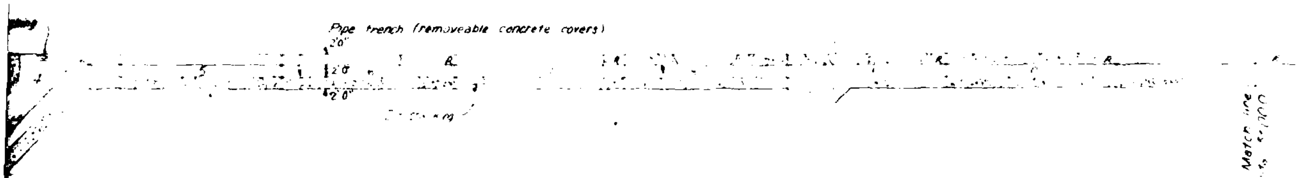
PLATE	F-4	LANDWARD WALL - LANDWARD LOCK	MONOLITHS 1 THRU 11 & UPPER GUIDE WALL
"	F-5	LANDWARD WALL - LANDWARD LOCK	MONOLITHS 12 THRU 18 & LOWER GUIDE WALL
"	F-6	INTERMEDIATE WALL	MONOLITHS 1 THRU 8
"	F-7	INTERMEDIATE WALL	MONOLITHS 9 THRU 12
"	F-8	INTERMEDIATE WALL	MONOLITHS 13 THRU 19
"	F-9	RIVERWARD WALL - RIVERWARD LOCK	MONOLITHS 1 THRU 8
"	F-10	RIVERWARD WALL - RIVERWARD LOCK	MONOLITHS 9 THRU 13
"	F-11	RIVERWARD WALL - RIVERWARD LOCK	MONOLITHS 14 THRU 22

Lower pool level 688.6



LOCKSIDE ELEVATION LANDWARD WALL (INVERTED)
LANDWARD LOCK

Pipe trench (removeable concrete covers)



Top of wall el. 732.6

LANDWARD ELEVATION - LANDWARD WALL - (NON-EXPOSED)

4	5	6	7	8	9	10	11
[illegible]ing on top [illegible] along edge [illegible]dinal cracks [illegible] top edge of wall	Longitudinal cracking along top edge of wall. Minor transverse hairline cracks in replacement concrete on the top surface.	Moderate longitudinal cracking below cold joint	Minor longitudinal cracking [illegible] surface. [illegible] wall.	Moderate longitudinal cracking along the top edge of the wall.	Moderate longitudinal cracking along the top edge of the wall. A few hairline transverse cracks on the top surface.	Minor longitudinal cracking at top edge of wall. A few hairline transverse cracks on top surface.	Moderate longitudinal cracking at top edge of wall. A few hairline transverse cracks on top surface.
[illegible]ing along the [illegible] joints and edges [illegible] scaling at lowest [illegible] construction [illegible] of the top [illegible] repaired.	Moderate spalling along vertical joints. 55% of the top surface repaired. Drummy along top surface edge in unrepaired area.	Severe scaling along vertical joints and in lower portion of the wall. 70% of top surface repaired. Popouts in replacement concrete. Light scaling in [illegible] concrete.	Top surface generally in [illegible] condition. 5% surface repaired adjacent to vertical [illegible] between Mon. #7 and #8. Moderate spalling along vertical joints and scaling of lower wall.	Moderate spalling and scaling along vertical joints. Moderate scaling in the [illegible] of the wall. Top surface 80% repaired and generally sound.	Severe spalling and scaling along vertical joints, ladder, and water level staff gage. Most severe in lower portion of wall. 90% top surface repaired.	Moderate spalling and scaling along vertical joints, scaling more severe at lower portion of the wall. Top surface 100% repaired but contains minor popouts and light scaling.	Moderate spalling along vertical joints. Moderate scaling near lower [illegible]. Water line at the bottom of wall. Top surface 100% repaired and [illegible]
See above [illegible] joints tight	See above. Vertical joints tight.	See above. Vertical joints tight.	See above. Vertical joints tight.	See above. Vertical joints tight.	See above. Vertical joints tight.	See above. Vertical joints tight.	See above. Vertical joints tight.
[illegible]able leaching, [illegible]ly in lower [illegible] of the wall. Some [illegible] from horizontal [illegible] on upper edges [illegible] wall.	Moderate leaching from longitudinal cracks along the top edge of the wall. Minor leaching in local areas along the bottom of wall.	Considerable leaching from longitudinal cracks below cold joint and in local areas at the bottom of wall.	Moderate leaching from areas near the bottom of the wall.	Moderate leaching from longitudinal cracks at top of wall [illegible] edge of scaling at the bottom of the wall.	Moderate leaching from longitudinal cracks at top of the wall. Severe leaching lower portion of staff gage.	Minor leaching at top of wall to moderate in local area towards the bottom.	Moderate leaching at top of wall from longitudinal cracks. Minor [illegible] leaching at the bottom of the wall.

LEGEND

Cracking

Replaced Concrete

Spalling

Core holes

[10] Depth of Deterioration

SYMBOL	DESCRIPTION	DATE	APPROVAL

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT, CORPS OF ENGINEERS
ST. PAUL, MINNESOTA

DESIGNED BY G.R.M.
DRAWN BY D.I.P.
CHECKED BY G.R.M.
SUBMITTED BY [signature]
APPROVED: [signature]

STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO. 1

CONCRETE CONDITION SURVEY
UPSTREAM LAND WALL
FALL 1974

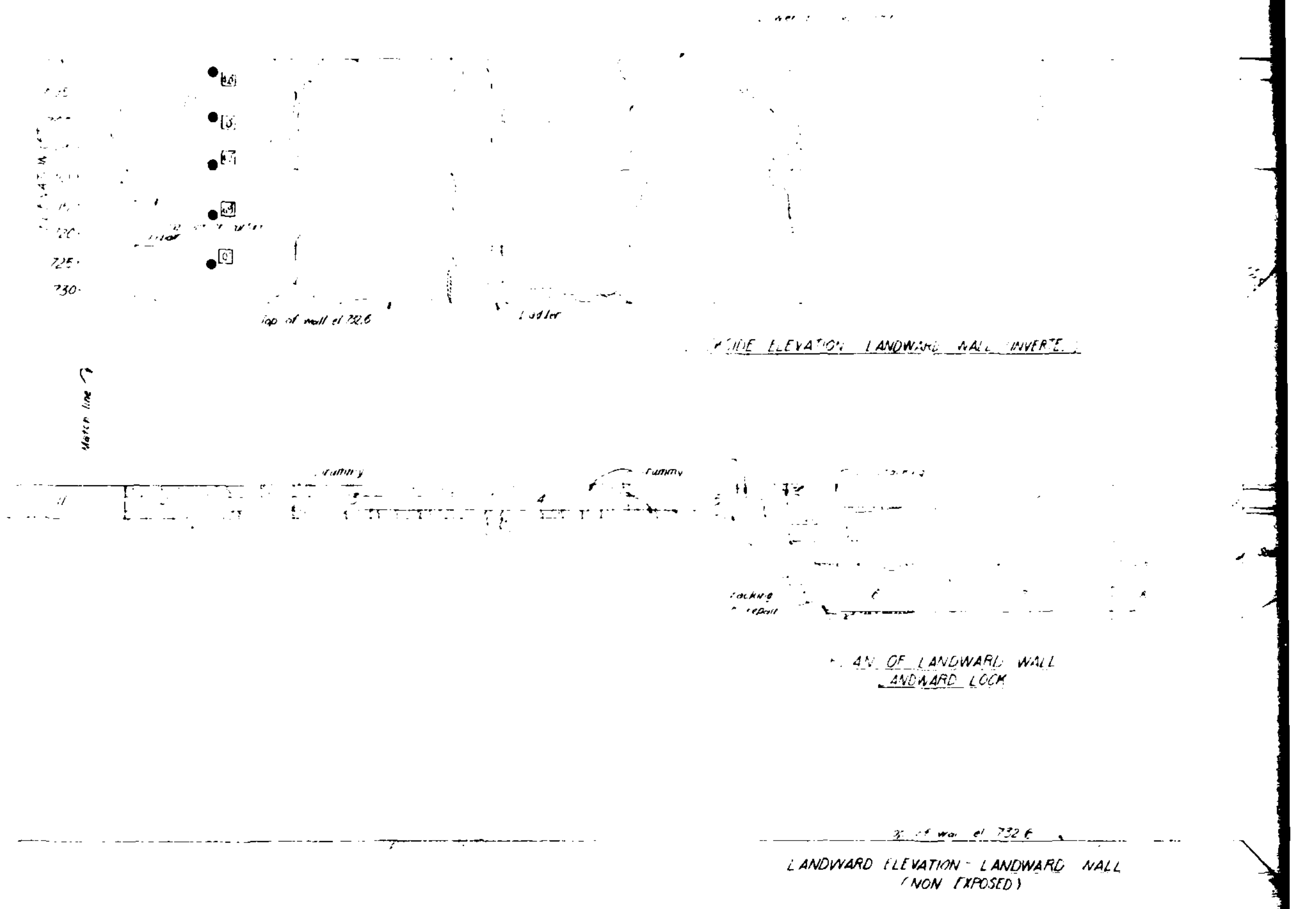
DATE: MARCH 1975

DRAWING NUMBER

SHEET OF

SAFETY PAYS

PLATE F-4



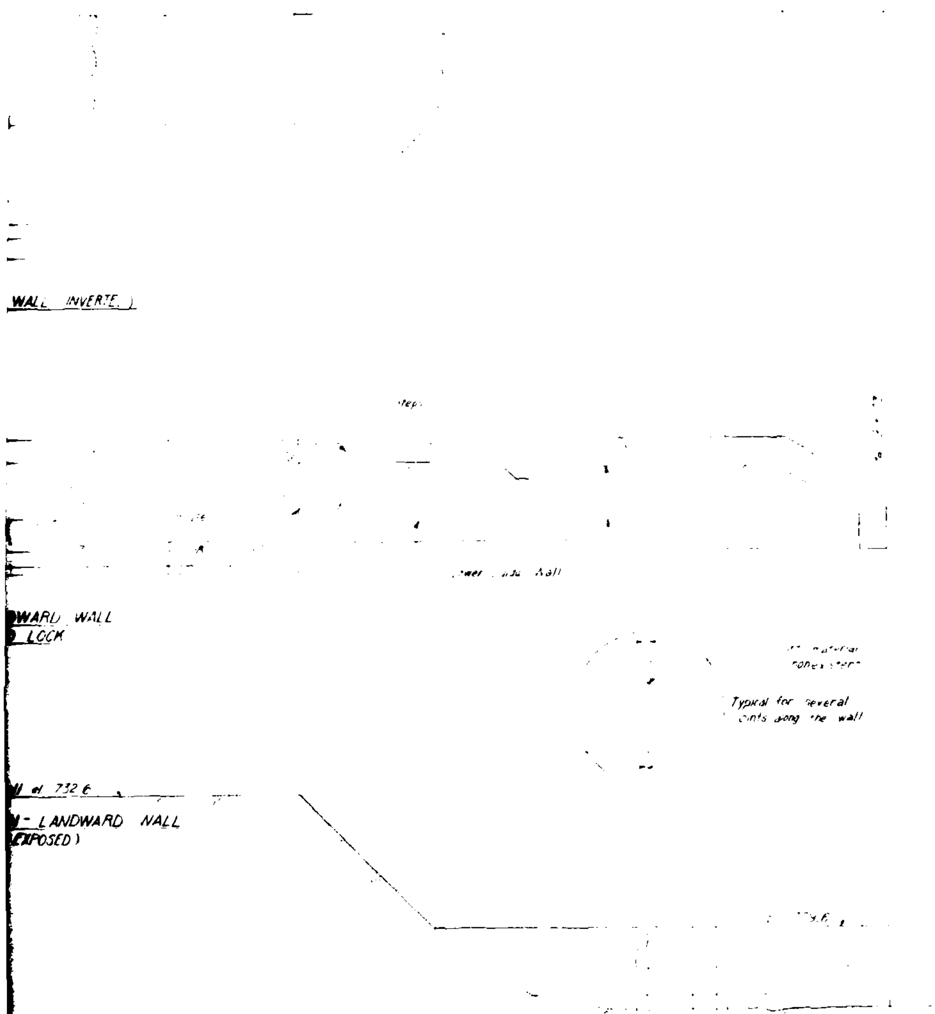
SIDE ELEVATION LANDWARD WALL (INVERTED)

PLAN OF LANDWARD WALL
LANDWARD LOCK

LANDWARD ELEVATION - LANDWARD WALL
(NON EXPOSED)

MONOLITH NO. →

CONCRETE CONDITION	12	13	14	15	16	17	18
CRACKING	Moderate longitudinal cracking at the top wall. A few transverse cracks in top surface.	Minor longitudinal cracking at the top of the wall. A few transverse cracks in the top surface.	Moderate to severe longitudinal cracking at top of wall. Minor hairline transverse cracking at top surface.	A few hairline transverse cracks in the top surface repair concrete	Longitudinal or D cracking in local areas on the top surface and along top edge of wall a few transverse cracks along top surface.	Minor cracking at [illegible] repairs of top surface	Minor cracks [illegible] Minor to moderate longitudinal cracking along top edge of wall
SCALING AND SPALLING	Moderate spalling along vertical joints. Moderate scaling in lower portions of the wall. 95% of the top surface has been repaired and is in good condition.	Moderate spalling along upstream vertical joint. Moderate scaling in lower portion of wall. 85% top surface repaired and generally in good shape.	Slight spalling along vertical joints on walls to moderate along ladder edges. Slight scaling on wall surface. 100% top surface repaired and generally in good shape.	Moderate spalling along vertical joints. Moderate scaling on lower portion of wall. 65% top surface repaired and generally sound.	Moderate spalling along vertical joints. Slight spalling along top surface edges with intermediate popouts. 30% top surface repaired and generally sound.	Slightly spalling along ladder edge. Slight scaling on wall surface. 50% top surface repaired. Repaired area sounds drummy.	Slight scaling on wall surface. 60% top surface repaired but sounds drummy.
JOINTS (VERTICAL)	See above Vertical joints tight.	See above Vertical joints tight.	See above Vertical joints tight.	See above Vertical joints tight.	See above Vertical joints tight.	Vertical joints tight.	Vertical joints tight.
OTHER	Moderate leaching from longitudinal cracks and in local areas at lower portion of the wall from scaled surface. Appears upper 15' of the wall was repaired. (May be cold joint).	Slight leaching from longitudinal cracks and in local areas of wall scaling. Top surface repair unbonded at upstream vertical joint.	Moderate to severe leaching from longitudinal cracks. Local areas of unbonded repair on top surface.	Local areas of unbonded repair on top surface	Slight leaching from longitudinal cracks at top of wall. Local areas of unbonded repair on top surface	Local areas of unbonded repair on top surface	Slight leaching from longitudinal cracks along top edge of wall. Local areas of unbonded repair on top surface



17	18	LOWER GUIDE WALL
Minor cracking around repairs of top surface	Minor cracking in old area of top surface. Minor to moderate longitudinal cracking along top edge of wall.	Minor longitudinal cracking along top edge of the transition monolith leading to lower guide wall, and in local areas at top of lower guide wall.
Slightly spalling along ladder edge. Slight scaling on wall surface. 80% top surface repaired. Repaired area sounds drummy.	Slight scaling on wall surface. 60% top surface repaired but sounds drummy.	Slight to moderate spalling along vertical joints between a few monoliths. Slight scaling on wall surface. Spall areas at water line are evident. All monoliths are badly weathered on top surface.
Vertical joints tight.	Vertical joints tight.	See above A few vertical joints loosen up.
Local areas of unbonded repair on top surface.	Slight leaching from longitudinal cracks along top edge of wall. Local areas of unbonded repair on top surface.	Slight leaching from longitudinal cracks in local areas along top of wall.

● Core Holes

Depth of Deterioration

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

DEPARTMENT OF THE ARMY
ST PAUL DISTRICT, CORPS OF ENGINEERS
ST PAUL, MINNESOTA

DESIGNED BY: G. R. M.
DRAWN BY: D. I. P.
CHECKED BY: G. R. M.
SUBMITTED BY:
APPROVED:

STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO 1
CONCRETE CONDITION SURVEY
DOWNSTREAM LAND WALL
FALL 1974

DATE: MARCH 1975

SCALE

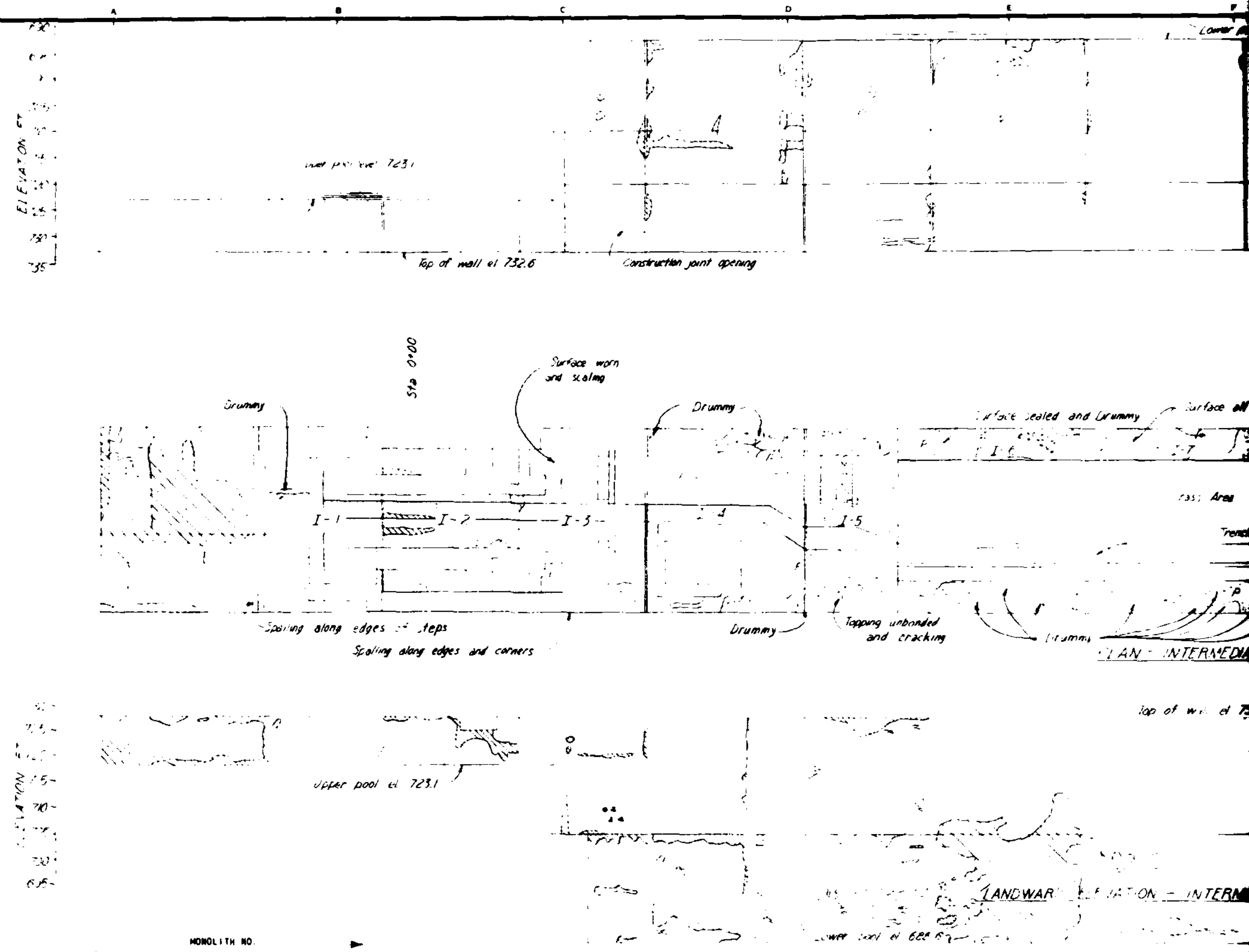
DRAWING NUMBER

SHEET OF

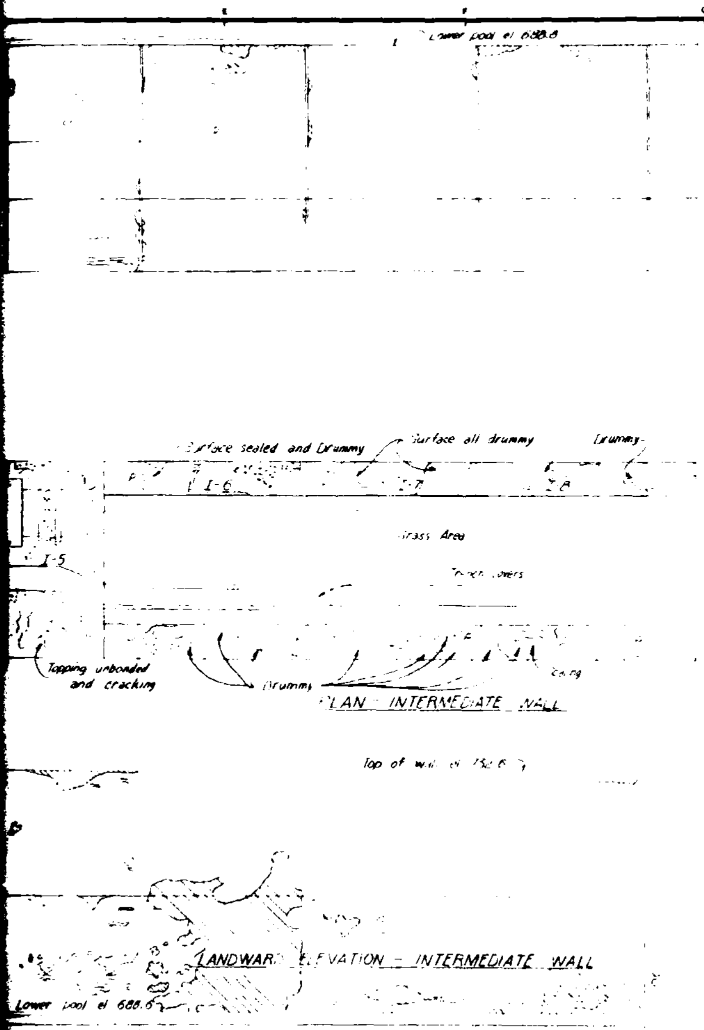
PLATE F-5

1

2



CONCRETE CONDITION		1	2	3	4	5	6	7
CRACKING	Badly cracked on top surface. One prominent crack on the wall at landlock side and on top surface as shown	Minor longitudinal cracks on the top surface.	Minor cracks on top surface and on wall at landlock side in local areas.	Slight cracking on top of wall at riverlock side.	Map cracking on top surface. Slight cracking at bottom of the wall on landlock side.	Severe map cracking on top surface. Moderate to severe longitudinal cracking at upper and lower portion of the riverside wall and local areas of landlock side wall	Min[illegible] at [illegible] sides	M[illegible] at upper edge [illegible] of wall and [illegible] areas of land- [illegible] wall
SCALING AND SPALLING	Large spall areas and badly weathered on top surface. Severe spalling on wall surface along vertical joints of landlock side. Slight to moderate scaling.	Surface weathering and minor spalling on top surface. 2% of top surface repaired. Slight spalling and scaling on walls and along vertical joints.	Spalling along edge on top surface. 95% top surface repaired. Moderate to severe spalling on wall at landlock side.	Moderate spalling along vertical & horizontal joints. Slight spalling on wall surface in local areas. Moderate to severe scaling on upper portions of wall at landlock side. Top surface scaling and spalling at edges and corners. Top surface 5% repaired.	Moderate to severe spalling along vertical joints. Slight spalling on wall of riverlock side. Severe spalling on lower portion of wall at landlock side. Slight scaling on riverlock wall, moderate at landlock side. Scaling on old concrete pavement at top surface. (38% repaired)	Slight spalling along vertical joints and slight to moderate spalling [illegible] upper portion of landlock wall. Slight scaling on wall surface. Top surface drummy over most areas. 85% top surface repaired.	Slight spalling at [illegible] wall [illegible] & along vertical joints. Severe [illegible] on most areas [illegible] Slight to moderate [illegible] walls. Top surface [illegible] and moderate scaling. 60% top surface repaired.	S[illegible] joints. Slight [illegible] of wall [illegible] side. S[illegible] Top surface drummy. 50% [illegible]
JOINTS	See above Vertical joints tight.	See above Vertical joints	See above Vertical joints tight	See above Vertical joints tight. A few horizontal construction joints opened up.	See above Vertical joints tight.	See above Vertical joints tight	See above Vertical [illegible] tight	See above Vertical [illegible] tight
OTHER	Badly weathered on top surface	Drummy in local areas on the top surface.	Drummy in local areas on top surface. Slight leaching from crack	Slight leaching from cracks & horizontal joints of riverlock wall. Water seeps from horizontal joint at lower portion of wall on riverlock side. Local areas of unbonded repair on the top surface	Slight leaching from cracks at bottom of wall on landlock side. Local areas of unbonded repair on top surface	Moderate to severe leaching from longitudinal cracks	Slight [illegible] cracks [illegible] unbonded repair [illegible] surface	Slight [illegible] from cracks



5	6	7	8
...ere map cracking on top ...rface. Moderate to ...ere longitudinal crack...g at upper and lower ...rtion of the riverside ...ll and local areas of ...dlock side wall	Minor cracks on lower portion of wall at both sides	Min... cracks at upper edge of riverlock side of wall and in local areas of land-lock side wall.	D... cracking ... surface Minor cracks at lower portion of wall ... land-lock side
...ight spalling along ...rtical joints and slight ...moderate spalling on ...per portion of landlock ...ll. Slight scaling on ...ll surface. Top surface ...mmy over most areas. ... top surface repaired.	Slight spalling at bottom of wall in riverlock side & along vertical joints. Severe spalling on most areas of landlock wall. Slight to moderate scaling on walls. Top surface drummy and moderate scaling. 60% top surface repaired.	Slight spalling along vertical joints. Slight spalling in lower portion of wall on landlock side. Slight scaling on walls. Top surface drummy. 50% repaired.	Slight spalling at ... edge of landlock side wall. Moderate to severe spalling in portions of walls above waterline. Slight spalling on walls. 50% top surface repaired. Repair ... surface drummy and scaling
See above ...rtical joints tight.	See above Vertical joints tight.	See above Vertical joints tight.	See above Vertical joints tight.
...derate to severe ...ching from long...dinal cracks.	Slight leaching from cracks. Local areas of unbonded repair on top surface	Slight leaching from cracks	Slight leaching from cracks. Local areas of unbonded repair on top surface.

● [illegible]
[illegible] Detail [illegible]

SAFETY PAYS

SYMBOL	DESCRIPTION	DATE	APPROVAL

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT, CORPS OF ENGINEERS
ST. PAUL, MINNESOTA

DESIGNED BY G R M
DRAWN BY D I P
CHECKED BY G R M
SUBMITTED BY
APPROVED

STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO 1
CONCRETE CONDITION SURVEY
UPSTREAM INTERMEDIATE WALL
FALL 1974

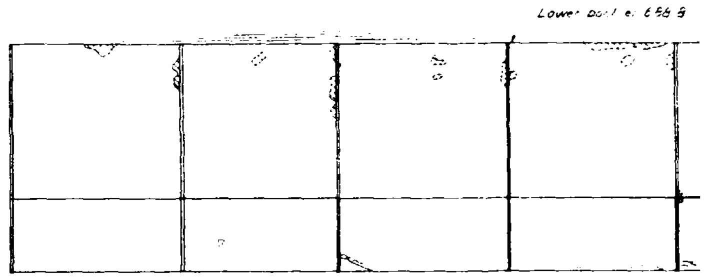
DATE
MARCH 1975

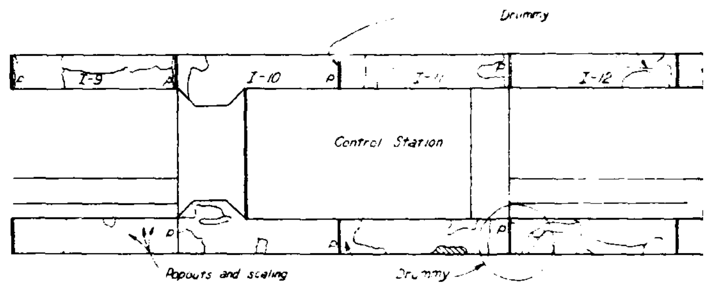
DRAWING NUMBER

SHEET ___ OF ___

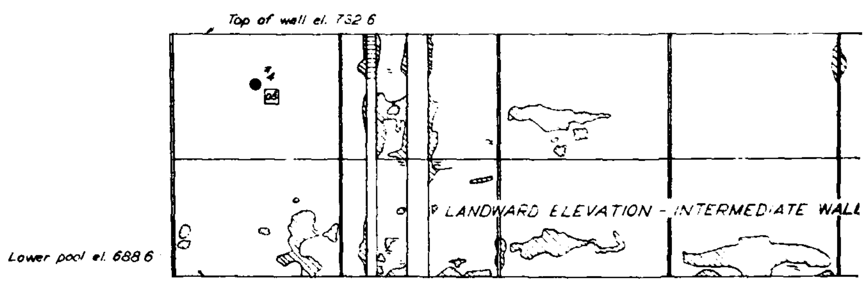
PLATE F-6

2





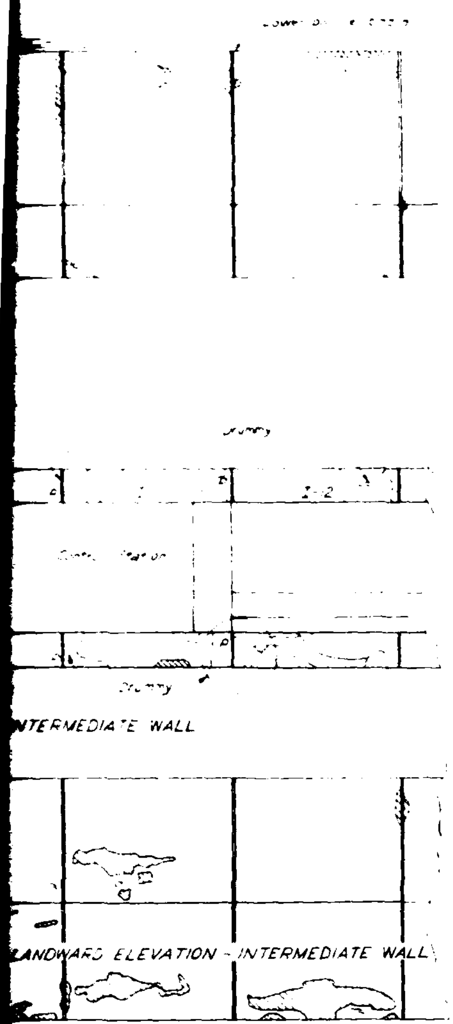
PLAN - INTERMEDIATE WALL



LANDWARD ELEVATION - INTERMEDIATE WALL

(CONTINUED)

CONCRETE CONDITION	MONOLITH NO 9	10	11	12
CRACKING	Hairline cracks on top surface. Minor cracks at lower portion of wall on landlock side	Craze cracking and cracking along repairs on top surface. Minor cracks in local areas on wall at landlock side	Diagonal cracking at [illegible] Minor cracks [illegible] landlock side [illegible] areas	[illegible]
SCALING AND SPALLING	Slight spalling along vertical joints. Slight to moderate spalling on lower portion of walls. Slight to moderate scaling on walls. Top surface weathered. 20% repaired.	Old concrete pavement on top surface weathered. Moderate spalling along vertical joints. Slight spalling on walls in local areas. Slight to moderate scaling on both walls. 10% top surface repaired	Moderate scaling on top surface. Slight spalling along vertical joints and on wall [illegible] side. Moderate to severe spalling on wall [illegible] side. Slight to moderate scaling on both walls. 15% top surface repaired	[illegible] Slight to moderate [illegible] 6% top surface repaired
JOINTS	See above Vertical joints tight	See above Vertical joints tight	See above Vertical joints tight	See above [illegible]
OTHER	Slight leaching from cracks. Local areas of unbonded repair on top surface	Slight leaching from cracks in wall of landlock side. Local areas of unbonded repair on top surface	Slight leaching from cracks [illegible]	[illegible]



	11	12
[illegible]	[illegible] areas	[illegible] repair [illegible] surface areas [illegible] landside [illegible] water line
[illegible]	Moderate scaling [illegible] surface. Slight spalling along vertical joints and [illegible] Moderate to severe spalling [illegible] Slight to moderate scaling [illegible] surface repaired	[illegible] spalling [illegible] surface. Slight spalling along vertical joints. Slight to moderate spalling in both walls. Slight to moderate. Scaling on both walls. 65% of surface repaired
Slight	See above. Vertical [illegible]	See above. Vertical [illegible] Slight. Horizontal [illegible] joint between surface pavement and top of wall [illegible] riverward side [illegible]
[illegible]	Slight leaching [illegible]	Slight to moderate leaching from cracks at [illegible] joints [illegible] horizontal construction joints between surface pavement and top of wall [illegible]

● Chip Holes

Concrete Deterioration

SYMBOL	DESCRIPTION	DATE	APPROVAL

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

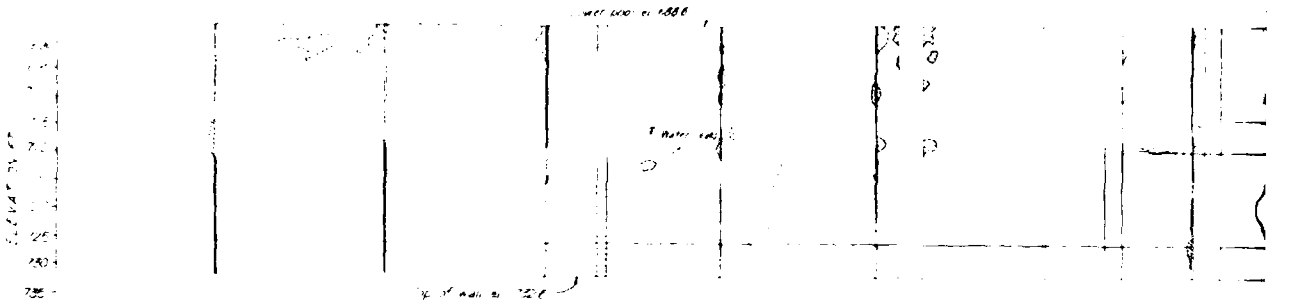
DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT, CORPS OF ENGINEERS
ST. PAUL, MINNESOTA

DESIGNED BY G.R.M.
DRAWN BY D.I.P.
CHECKED BY G.R.M.
SUBMITTED BY:
APPROVED:

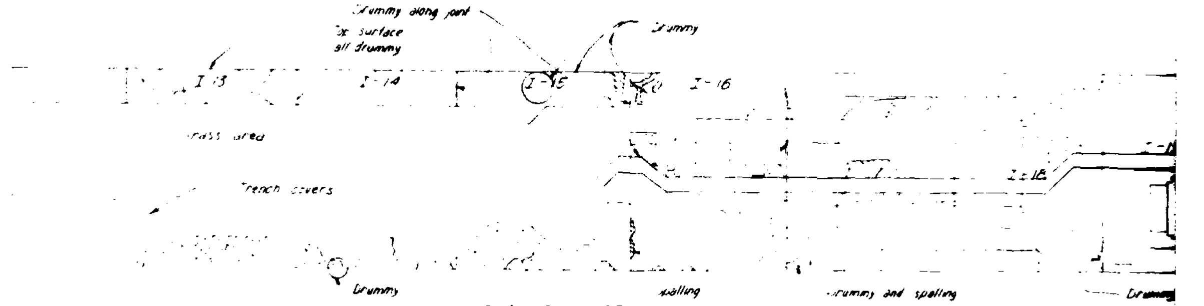
STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO. 1
CONCRETE CONDITION SURVEY
MIDDLE INTERMEDIATE WALL
FALL 1974

DATE: MARCH 1975
SCALE
SPEC. NO.
DRAWING NUMBER
SHEET OF

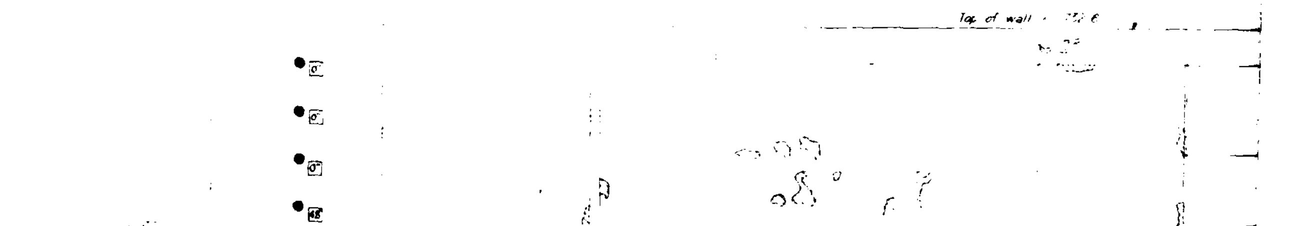
PLATE F-7



RIVERSIDE ELEVATION INTERMEDIATE WALL (INVERTED)

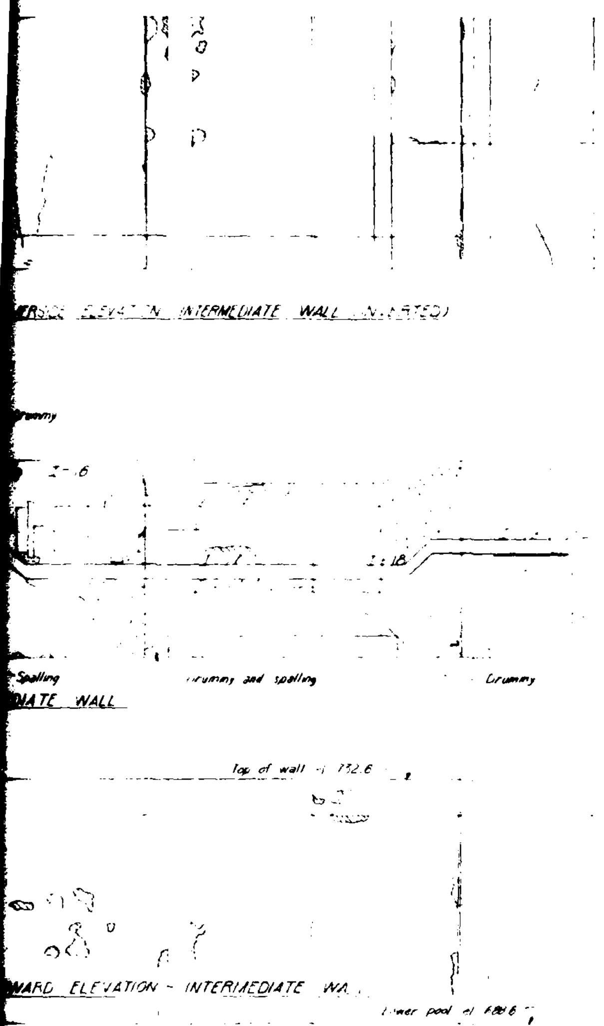


PLAN OF INTERMEDIATE WALL



LANDWARD ELEVATION - INTERMEDIATE WA[illegible]

CONCRETE CONDITION	13	14	15	16	17	18	
CRACKING	Major cracking & transverse hairline cracks on top surface. Minor cracks at upper corner at wall next to mon lith #12 at riverside side. Minor longitudinal cracks on lower portion of wall on both sides.	Minor craze cracks and cracks around repair on top surface. Minor cracks at lower portion of wall on riverside side.	Map cracking in repair concrete on top surface.	Map [illegible] cracking [illegible] surface [illegible] wall at landside side.	Minor [illegible] at upper portion of wall [illegible] landside side.	Slight [illegible]	Some large [illegible] surface, [illegible] vertical [illegible] side.
SCALING AND SPALLING	Slight spalling along vertical joints. Moderate spalling and slight scaling on both walls. 95% of top surface repaired.	Minor spalling along joints and edges on top surface. Slight spalling along vertical joints. Slight to moderate scaling on both walls. 10% of top surface repaired.	Minor scaling on top surface. Slight spalling along vertical joints and on both walls. Slight to moderate scaling on both walls. Top surface 30% repaired.	Slight scaling along vertical joints. Slight to moderate scaling on walls. Severe scaling on wall at landside side. 25% top surface repaired.	Slight to moderate spalling along vertical joints. Slight scaling on walls. 15% top surface repaired.	Slight to moderate [illegible] 40% top surface repaired.	Slight [illegible]
JOINTS	See above Vertical joints tight.	See above Vertical joints tight.	See above Vertical joints tight.	See above Vertical joints tight. One horizontal [illegible] landside side.	See above Vertical joints tight. One horizontal [illegible]	See above Vertical joints between [illegible] #18 and #19 [illegible]	Vertical [illegible] except at [illegible]
OTHER	Top surface at riverside side drummy. Slight to moderate leaching from cracks at upper corners of riverside side wall and lower portion of landside side wall. Water seeps out of cracks at lower portion of riverside side wall.	Local areas drummy on top surface. Water seeps out of cracks at lower portion of wall at riverside side.	Local area drummy on top surface.	Top surface weathered and drummy in some areas. Minor leaching from [illegible] Water seeps [illegible]	Top surface weathered and drummy in some areas. Minor leaching from cracks and [illegible] horizontal [illegible]	[illegible] weathered [illegible] Moderate [illegible] severe leaching [illegible] Water [illegible]	Top surface [illegible] Moderate [illegible] vertical [illegible]



16	17	18	19
...p and [illegible] cracking [illegible] surface. ...p [illegible] vertical cracks [illegible] at river lock side. Minor [illegible] lower portion of wall at landside side	Min[illegible] at upper portion of wall [illegible] landside side	Slight [illegible] moderate [illegible] cracking [illegible] walls	Some large [illegible] surface. A few [illegible] vertical [illegible] river [illegible] side of wall
Slight spalling along vertical joints. Slight to moderate spalling [illegible] walls. Severe scaling on wall at landside side. 25% top surface repaired	Slight to moderate spalling along vertical joints. Slight scaling [illegible] walls. 15% top surface repaired.	Slight to moderate spalling along vertical joints on both walls and [illegible] areas on wall at landside side. Slight scaling on walls. 40% top surface repaired	Slight scaling [illegible] walls
See above. Vertical joints tight. One horizontal construction joint [illegible] at landside side	See above. Vertical joints tight. One horizontal construction joint at [illegible] side wall [illegible]	See above. Vertical joints between monoliths #18 and #19 open. Two horizontal joints (one on each side) [illegible]	Vertical [illegible] tight except at lower [illegible] of joints between monoliths #18 and #19. Some horizontal joints [illegible] at riverside side
Top surface weathered and drummy in some areas. Minor leaching from cracks [illegible] landside side wall [illegible] vertical cracks [illegible] side [illegible]. Water seeps out [illegible] horizontal construction [illegible] riverside side wall	Top surface weathered and drummy in some area. Minor leaching from cracks and loose horizontal construction [illegible]	Top surface mostly weathered in old concrete areas. Moderate to severe leaching from longitudinal cracks and from horizontal construction joints. Water seeps from open vertical joint	Top surface mostly weathered. Moderate to severe leaching from vertical cracks and loose horizontal [illegible] joints. Water seeps [illegible] vertical joint at [illegible] waterline

● [illegible]
[illegible] Deterioration

SYMBOL	DESCRIPTION	DATE	APPROVAL

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT, CORPS OF ENGINEERS
ST. PAUL, MINNESOTA

DESIGNED BY G R M
DRAWN BY D I P
CHECKED BY G R M

STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO. 1
CONCRETE CONDITION SURVEY
DOWNSTREAM INTERMEDIATE WALL
FALL 1974

SUBMITTED BY
[signature]
APPROVED
[signature]

DATE
MARCH 1975

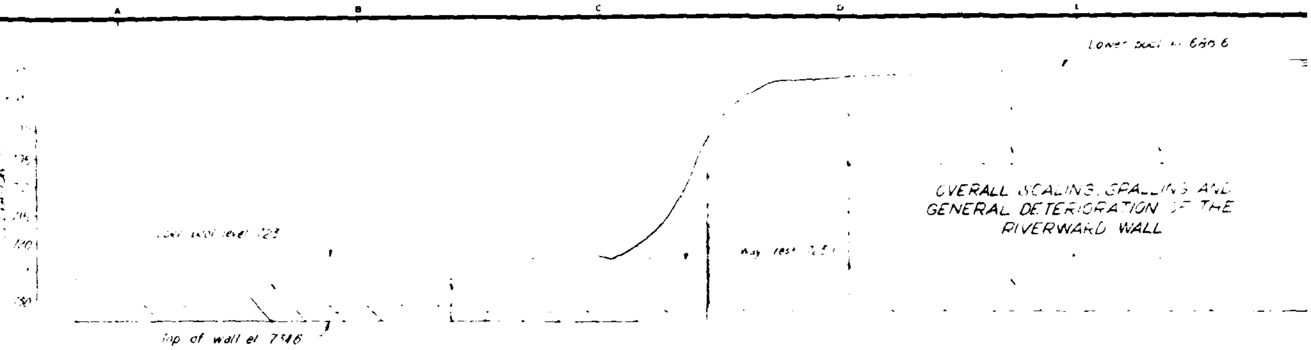
DRAWING NUMBER

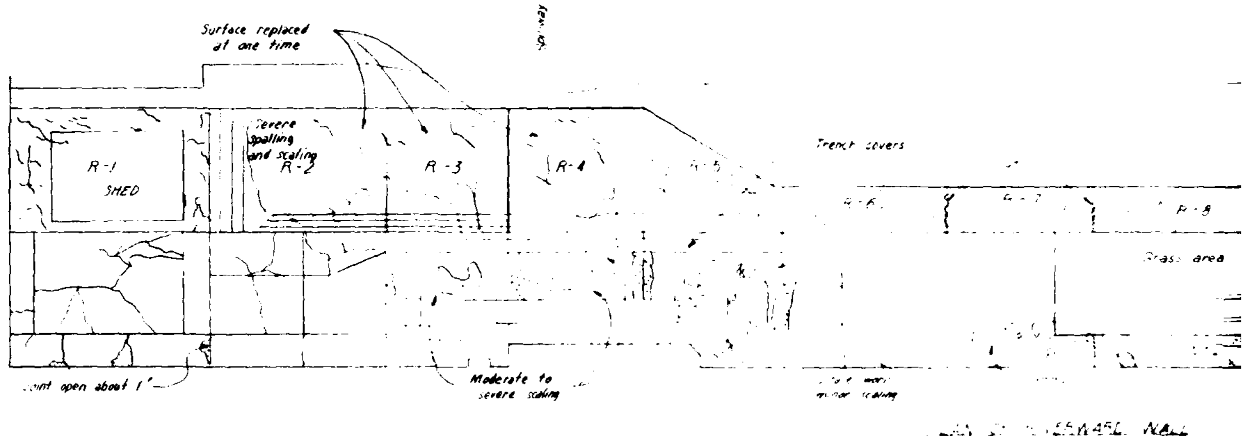
SHEET OF

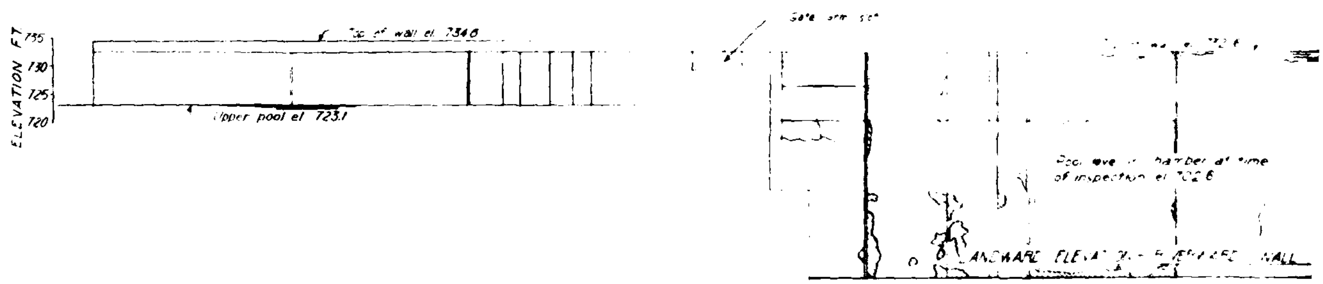
SAFETY PAYS

PLATE F-8

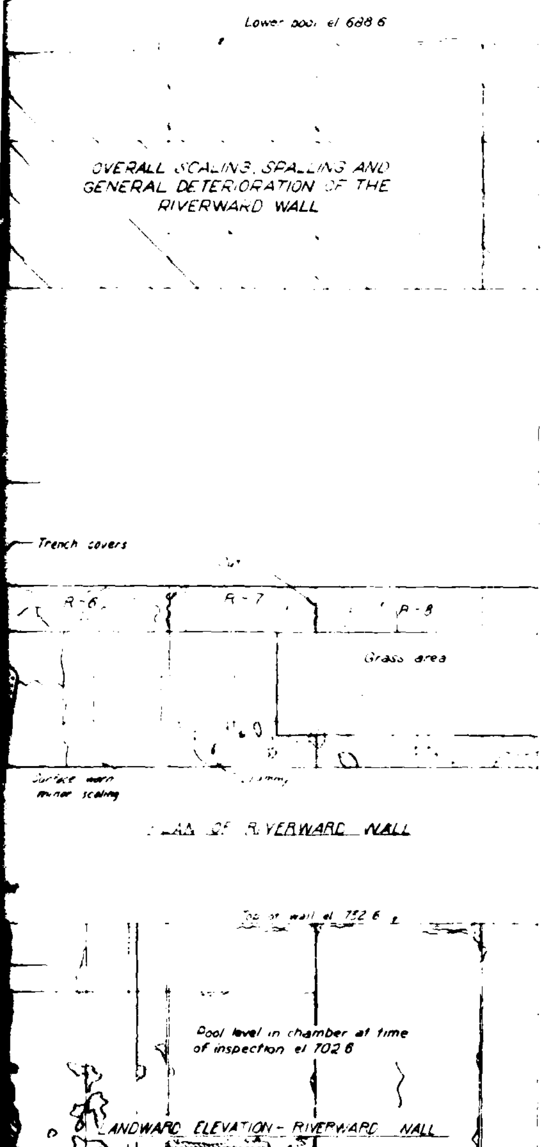
2







	MONOLITH NO.							
CONCRETE CONDITION	1	2	3	4	5	6	[illegible]	8
CRACKING	Severe random cracking and pattern cracking of the top surface.	Severe random cracking of the top surface.	Random cracking of top surface, old concrete (riverwall).	Random cracking of top surface, old concrete (riverwall)	Random cracking of the surface, old concrete (riverwall). Horizontal and vertical cracking of riverlock wall.	[illegible]	[illegible]	[illegible]
SCALING AND SPALLING	Upstream face severely scaled & deteriorated; increase to water level.	Severe scaling of the top surface and old concrete wall (riverside). Spalling along corners & edges.	50% of concrete surface repaired at one time. Moderate to severe scaling of top surface and riverwall.	50% of concrete surface repaired at one time. Local areas drummy and unbonded. Moderate to severe scaling of top surface and riverwall	75% of concrete top surface repaired at one time. Minor scaling on riverlock wall. Severe scaling of riverwall	30% [illegible]	[illegible]	[illegible]
JOINTS	Downstream vertical joint open about 1" on top surface.	Vertical joints tight	Vertical joints tight.	Vertical joints tight.	Vertical joints tight but spalling along top surface joint. Leakage through horizontal construction joints	Moderate [illegible]	[illegible]	[illegible]
OTHER	Considerable distortion in the top slab	Considerable distortion in the top surface			Slight leaching mostly from cracking in the wall and along horizontal construction joints	Minor leaching from vertical cracking [illegible] riverlock wall	[illegible]	Moderate leaching [illegible]



6	7	8
Random cracking of top surface. Short vertical crack in riverface wall.	Minor random cracking in top surface. Longitudinal cracking.	Medium random cracking in top surface. Longitudinal cracking at top of riverface wall.
30% of top surface, old concrete repaired at one time. Top surface weathered with minor scaling. Moderate scaling riverface wall to severe scaling riverwall.	Slight scaling top surface. Moderate scaling near low water level on riverface wall. Severe scaling in face of riverwall (old concrete).	Minor scaling on top surface. Severe scaling on face of riverwall. 10% top surface [illegible]
Moderate spalling along joints, but joints tight.	Minor leaching along top horizontal construction joints in riverface wall. Minor spalling along vertical joints.	Riverface wall vertical joints have very little spalling and are tight.
Minor leaching from vertical cracking in riverface wall.	Leaching from longitudinal cracks at top of riverface wall. Local areas of unsound top surface.	Moderate leaching from longitudinal cracking in riverface wall.

SYMBOL | DESCRIPTION | DATE | APPROVAL

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

DEPARTMENT OF THE ARMY
ST PAUL DISTRICT CORPS OF ENGINEERS
ST PAUL MINNESOTA

DESIGNED BY G.R.M.
DRAWN BY B.I.P.
CHECKED BY G.R.M.
SUBMITTED BY
Joseph [illegible]
APPROVED
[illegible]

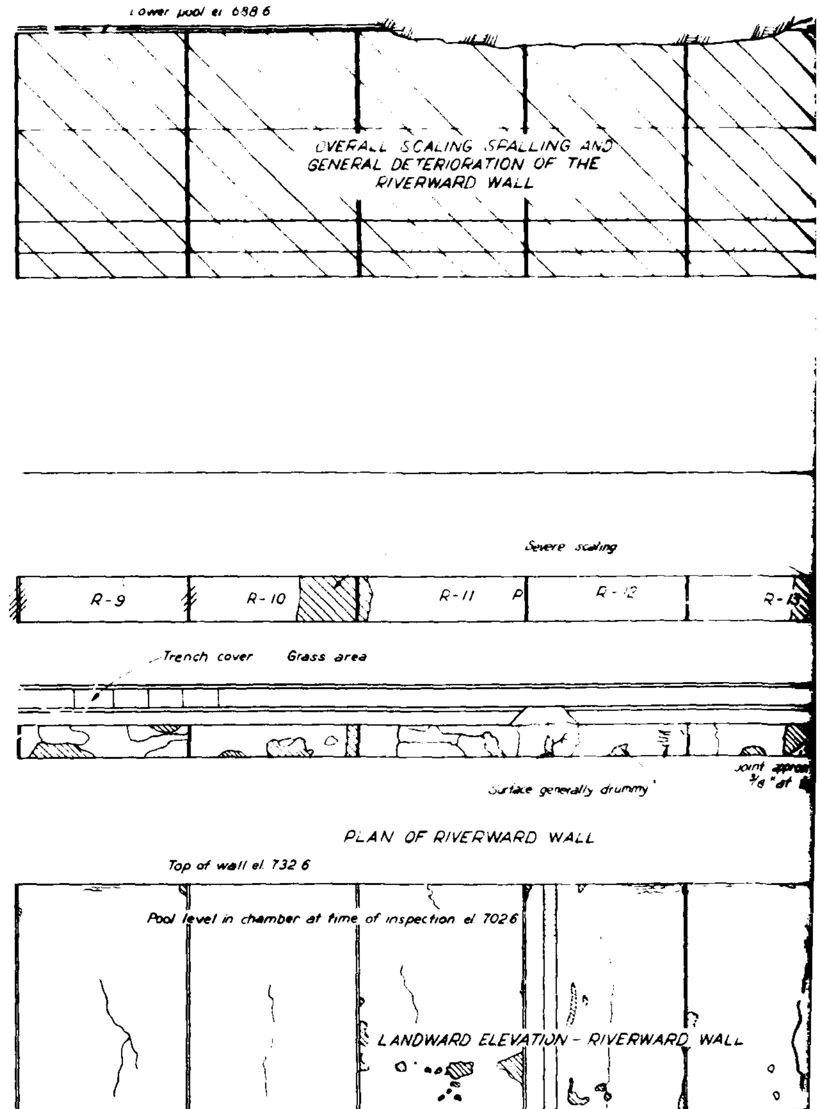
STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO 1
CONCRETE CONDITION SURVEY
UPSTREAM RIVER WALL
FALL 1974

SAFETY PAYS

DATE
MARCH 1975

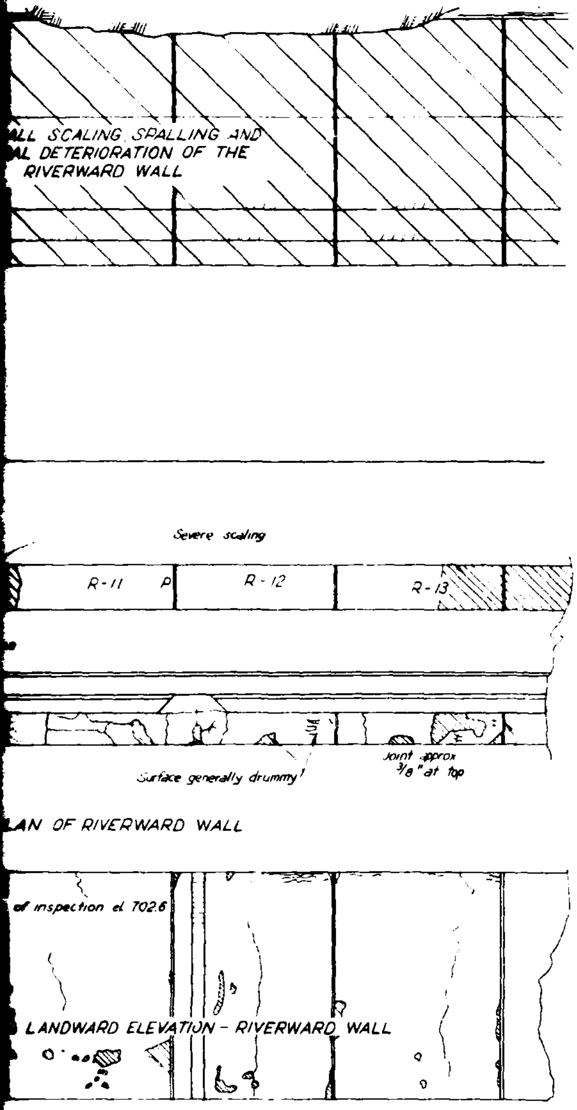
SCALE
SPEC NO
DRAWING NUMBER
SHEET OF

PLATE F-9



(CONTINUED) MONOLITH NO

CONCRETE CONDITION	9	10	11	12	13
CRACKING	Minor longitudinal cracking at top on riverlock wall. Vertical crack in face of riverlock wall	[illegible]	[illegible]	[illegible]	[illegible]
SCALING AND SPALLING	Moderate scaling of top surface to severe on face of riverwall. Very slight spalling along vertical joints riverlock wall	Moderate to severe [illegible]	Moderate [illegible]	[illegible]	Moderate to severe [illegible]
JOINTS	Vertical joints tight	[illegible]	[illegible]	[illegible]	[illegible]
OTHER	Minor leaching along longitudinal cracks at [illegible] 20% [illegible] top surface repaired	40% [illegible]	[illegible]	[illegible]	[illegible]



11	12	13
Minor [illegible] top surface. [illegible] vertical cracks [illegible]	[illegible] cracking [illegible] surface. Minor longitudinal cracking near top of the river lock wall. Vertical [illegible] riverlock wall.	Minor longitudinal cracking at top of river lock wall. Several vertical cracks in the riverlock wall.
Moderate [illegible] and near [illegible] water [illegible] riverlock wall to severe face of [illegible] wall.	[illegible] areas of scaling and [illegible] surface. Severe scaling on face of the river wall. Moderate [illegible] riverlock wall.	Moderate to severe scaling of the top surface and severe scaling on face of the river wall. Minor to moderate spalling along vertical joints on riverlock wall.
Vert[illegible]	Vertical [illegible] tight.	Vertical joint between mono. #13 and mono. #14 open 3/8" at the top. Also see spalling above.
20% [illegible] repa[illegible] of [illegible]	Minor leaching from [illegible] cracks [illegible]	Minor leaching from longitudinal cracks at top of riverlock wall.

SYMBOL	DESCRIPTION	DATE	APPROVAL

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT, CORPS OF ENGINEERS
ST. PAUL, MINNESOTA

DESIGNED BY G.R.M.
DRAWN BY D.J.P.
CHECKED BY G.R.M.

STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO. 1

CONCRETE CONDITION SURVEY
MIDDLE RIVER WALL
FALL 1974

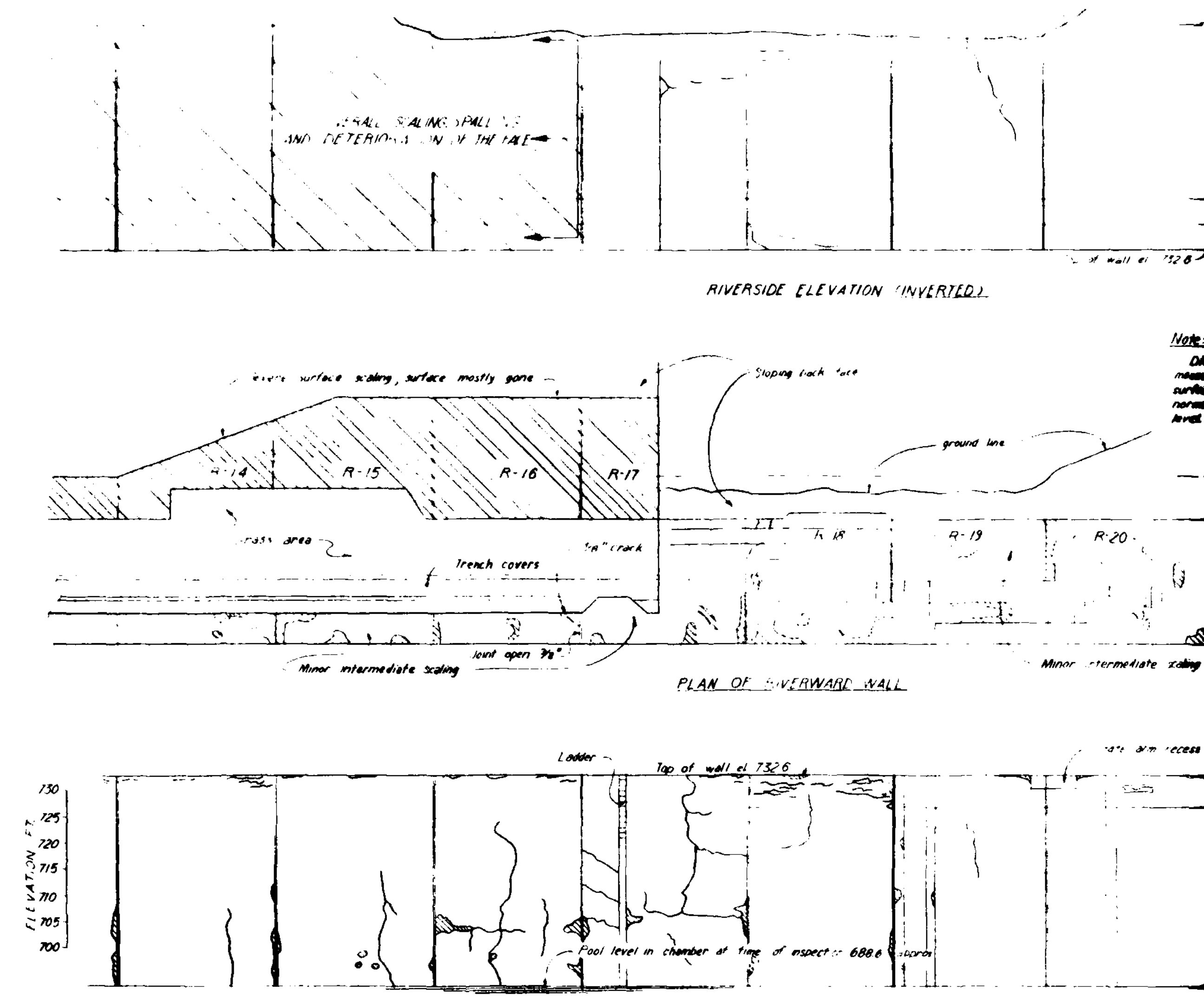
SUBMITTED BY
APPROVED

DATE
MARCH 1975

DRAWING NUMBER

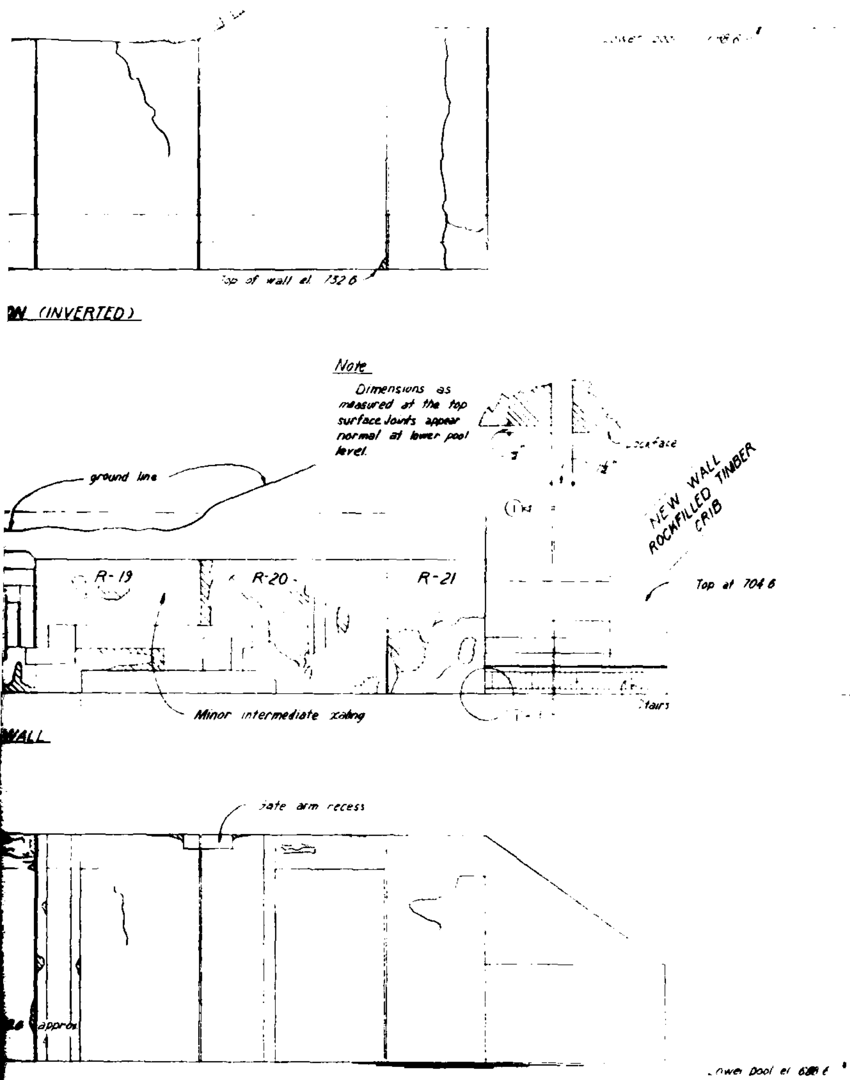
SHEET OF

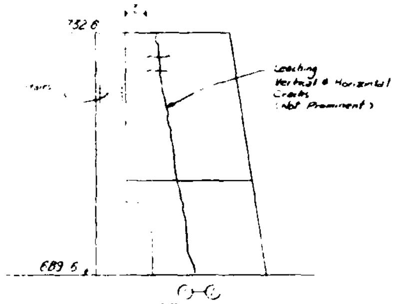
PLATE F-10



MONOLITH NO. →

CONCRETE CONDITION	14	15	16	17	18	19	20
CRACKING	Minor to moderate longitudinal cracking at top of riverlock wall. Vertical crack in riverlock wall. Minor random cracking in local areas of top surface.	Minor longitudinal cracking at top of riverlock wall. Vertical crack in riverlock wall	Minor longitudinal cracking at top of riverlock wall. Several vertical cracks in riverlock wall.	Vertical and horizontal cracking in riverlock wall. Minor longitudinal cracks	Severe longitudinal cracking at top of riverlock wall. Vertical crack in riverlock wall	Some vertical cracks	Minor longitudinal cracking at top of riverlock wall.
SCALING AND SPALLING	Severe scaling on riverwall face and top of riverwall. Minor scaling on top of riverlock wall	Severe scaling on riverwall face and top of riverwall. Moderate weathering and scaling on top of riverlock wall	Several scaling on riverwall face and top of riverwall. Minor surface scaling on top and face of riverlock wall.	Severe scaling on top of riverwall. Riverwall face weathered. Minor scaling and weathering on top, and moderate scaling & spalling on face of riverlock wall	Moderate scaling of top surface. Minor spalling and scaling on riverlock wall.	Minor weathering with some scaling on top surface, some spalling along edges of riverlock wall	Minor spalling along vertical edges. Moderate scaling on top surface
JOINTS	Vertical joints between mono #13 and mono #14. Open 3/8" at the top	Minor spalling along vertical joints in riverlock wall. Vertical joints tight	Vertical joint between mono #16 and mono #17. Open 3/8" at top.	Vertical joint between mono #16 and mono #17. Open 3/8" at top.	Vertical joints tight	Vertical joints tight. Leaching from horizontal construction joints on riverwall.	Vertical joints tight. Leaching from horizontal construction joints on riverwall.
OTHER	Moderate leaching from longitudinal cracks at top of riverlock wall	Slight leaching from top of riverlock wall.	Minor leaching from longitudinal cracks at top of riverlock wall.	Minor to moderate leaching for longitudinal cracks and from vertical cracks in riverlock wall.	Severe leaching from longitudinal cracking to moderate along vertical crack.	Minor leaching from vertical cracks	Minor leaching from longitudinal cracks.





SECTION
DOWNSTREAM FACE OF MONO

	19	20	21	
...al ...all.	Some vertical cracks.	Minor longitudinal cracking of top of riverlock wall.	Random cracking of top surface, vertical cracking in walls.	
... on	Minor weathering with some scaling on top surface, some spalling along edges on riverlock wall.	Minor spalling along vertical edges. Moderate scaling on top surface.	Moderate scaling on top surface.	
...t.	Vertical joints tight. Leaching from horizontal construction joints on riverwall.	Vertical joints tight. Leaching from horizontal construction joints on riverwall.	Moderate scaling on top surface.	
... to ...cal	Minor leaching from vertical cracks.	Minor leaching from longitudinal cracks.	Minor leaching from vertical cracks	Construction joints generally rough.

SAFETY PAYS

SYMBOL	DESCRIPTION	DATE	APPROVAL

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

DEPARTMENT OF THE ARMY
ST. PAUL DISTRICT, CORPS OF ENGINEERS
ST. PAUL, MINNESOTA

DESIGNED BY C.R.M.
DRAWN BY D.I.P.
CHECKED BY E.R.B.
SUBMITTED BY
APPROVED

STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO. 1
CONCRETE CONDITION SURVEY
DOWNSTREAM RIVER WALL
FALL 1974

DATE
MARCH 1975

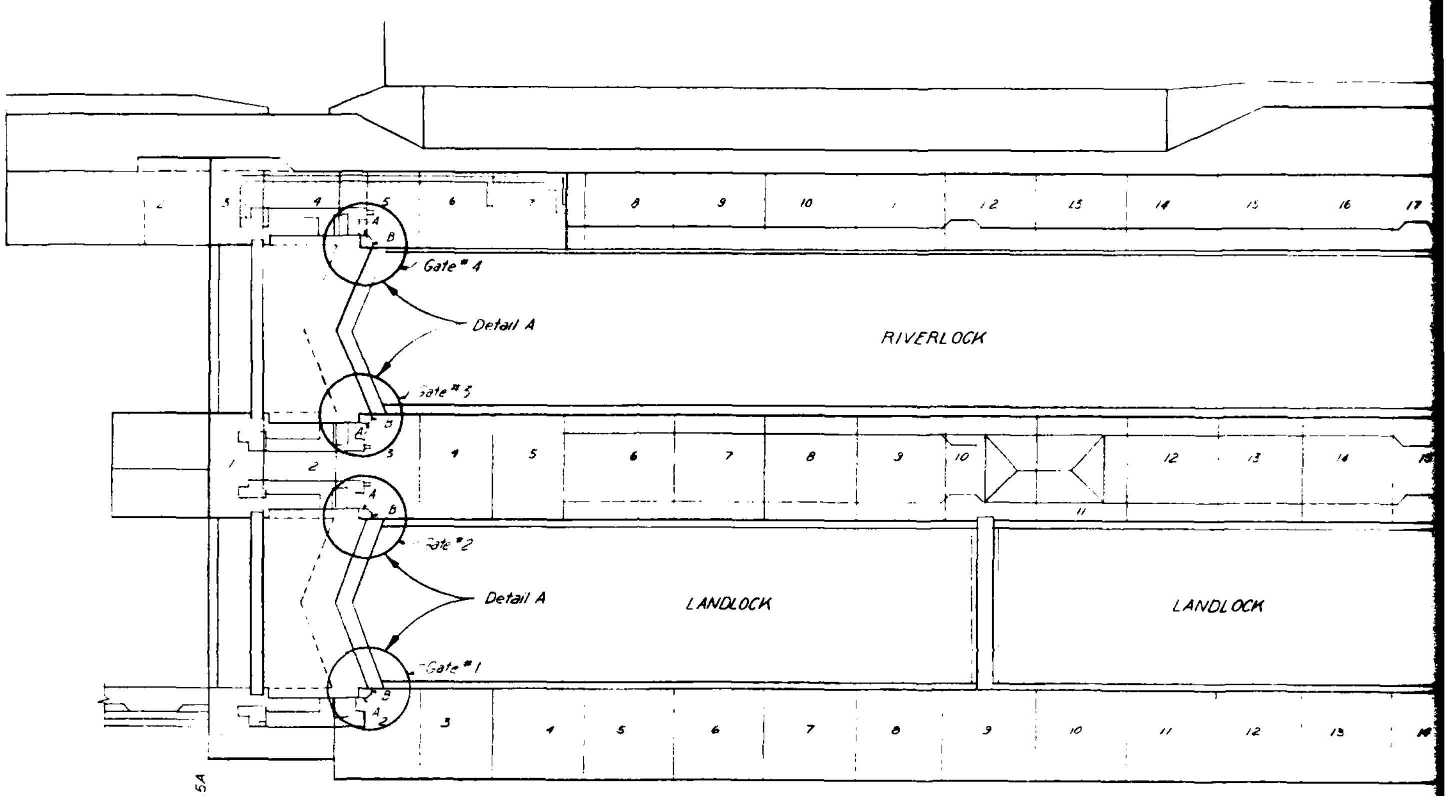
DRAWING NUMBER

SHEET OF

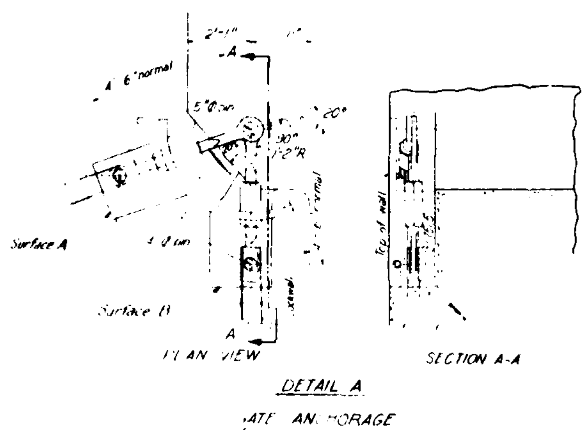
PLATE F-11

1

2



PLAN



DETAIL A

GATE ANCHORAGE
(As shown for Landwall
opposite hand for Riverwall)

CHART SHOWING COMPRESSIVE STRENGTHS
FOR GATE ANCHORS

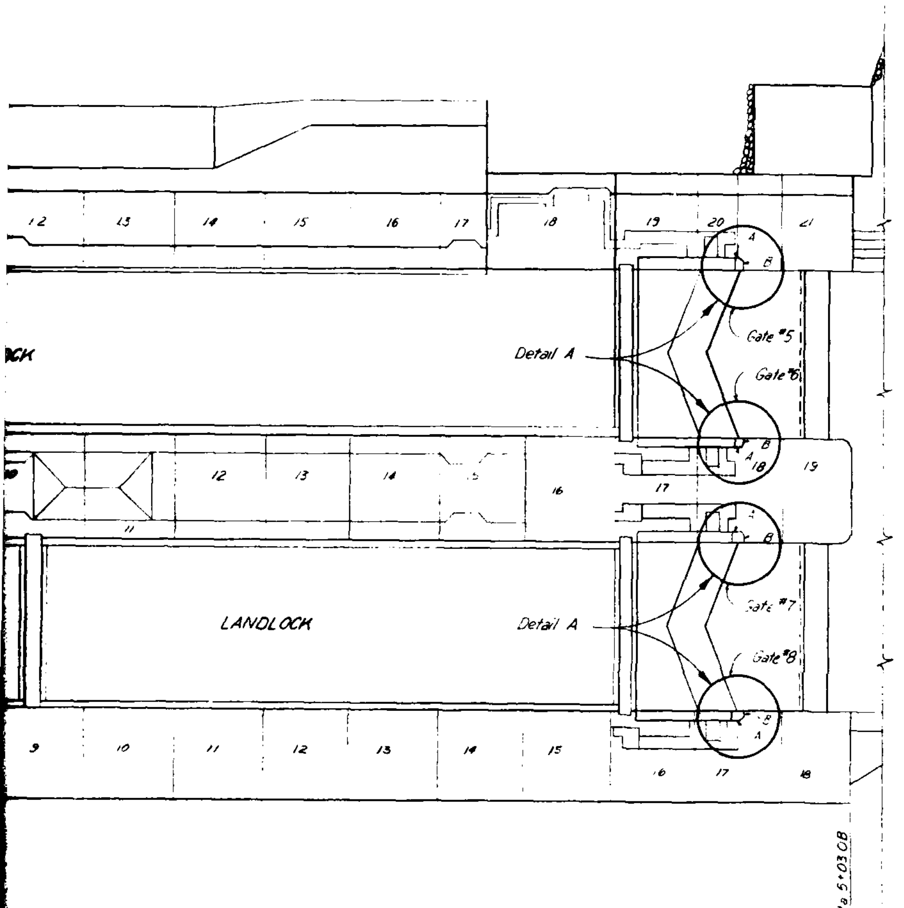
GATE NO.	SURFACE A 1)	SURFACE B 1)
1	4200 psi	3350 psi 2)
2	5200 psi	6250 psi
3	3400 psi 2)	4000 psi
4	3400 psi	4950 psi
5	3500 psi 3)	2950 psi 3)
6	4800 psi	3300 psi
7	5800 psi 2)	6500 psi
8	6000 psi	5500 psi

Notes

1) The values of compressive strengths were obtained by Swiss Impact Hammer with an average value from 10 readings

2) Surface scaling around anchor

3) D-cracking and leaching



...MPRESSIVE STRENGTHS
...TE ANCHORS

...RFACE A 1/	SURFACE B 1/
4200 psi	3350 psi 2/
5200 psi	6250 psi
3400 psi 2/	4000 psi
3400 psi	4950 psi
3500 psi 2/	2950 psi 3/
4[illegible]00 psi	3300 psi
5800 psi 2/	6500 psi
6000 psi	5500 psi

...es of compressive strengths
...ed by Swiss Impact Hammer
...erage value from 10 readings.

...aling around anchor

...g and leaching

SYMBOL	DESCRIPTION	DATE	APPROVAL

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

DEPARTMENT OF THE ARMY
ST PAUL DISTRICT CORPS OF ENGINEERS
ST PAUL MINNESOTA

DESIGNED BY G R M
DRAWN BY D I P
CHECKED BY G R M

SUBMITTED BY

APPROVED

STUDY OF ALTERNATIVES FOR REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO 1

CONCRETE CONDITION SURVEY

SURFACE COMPRESSIVE STRENGTH
AT GATE ANCHORS

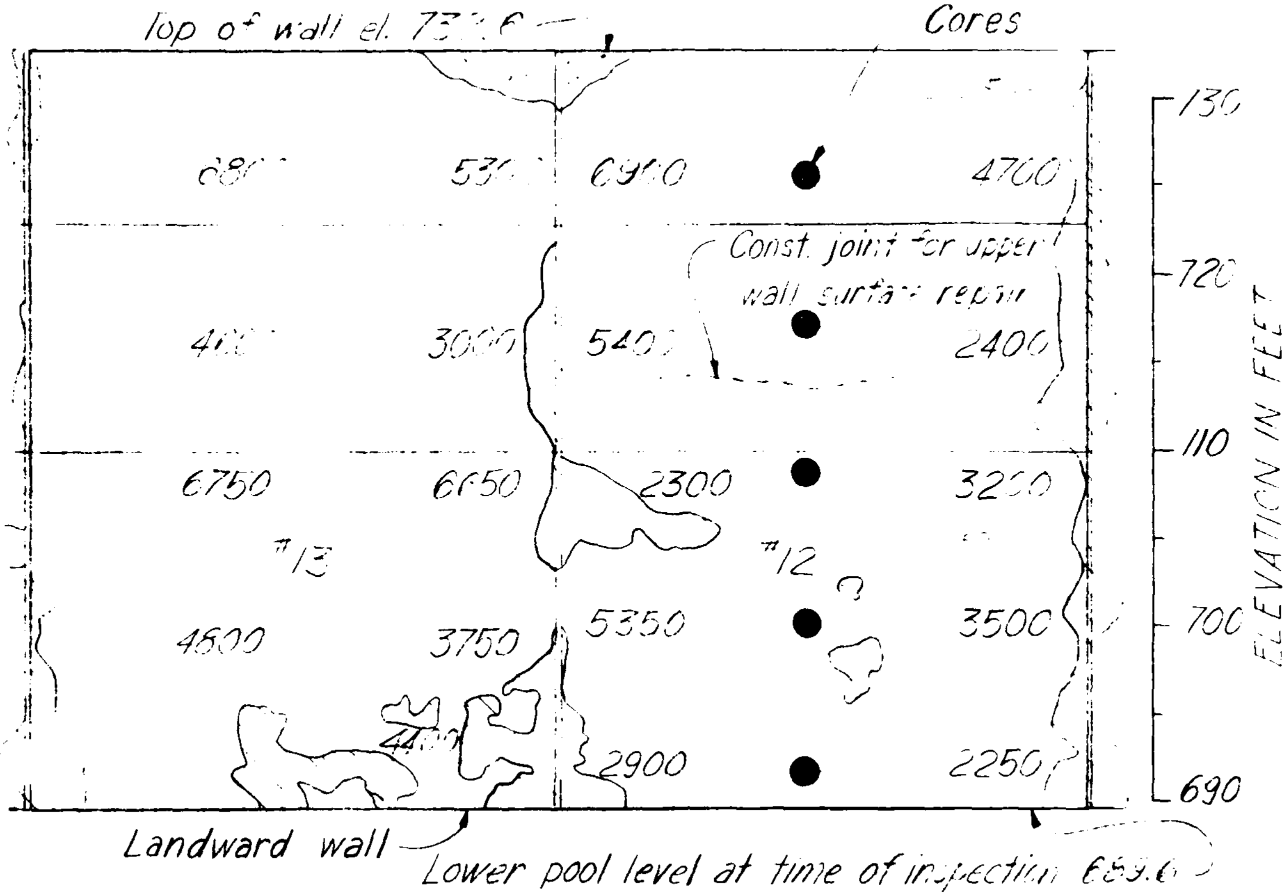
DATE
MARCH 1975

SCALE

DRAWING NUMBER

SHEET OF

PLATE F-12



Notes:

1.) Compressive strengths are in-place strengths at the location shown.

2.) All readings are in psi, taken from an average of 10 Swiss hammer readings.

3.) Horizontal cores are 11" deep

Scale 10 0 10

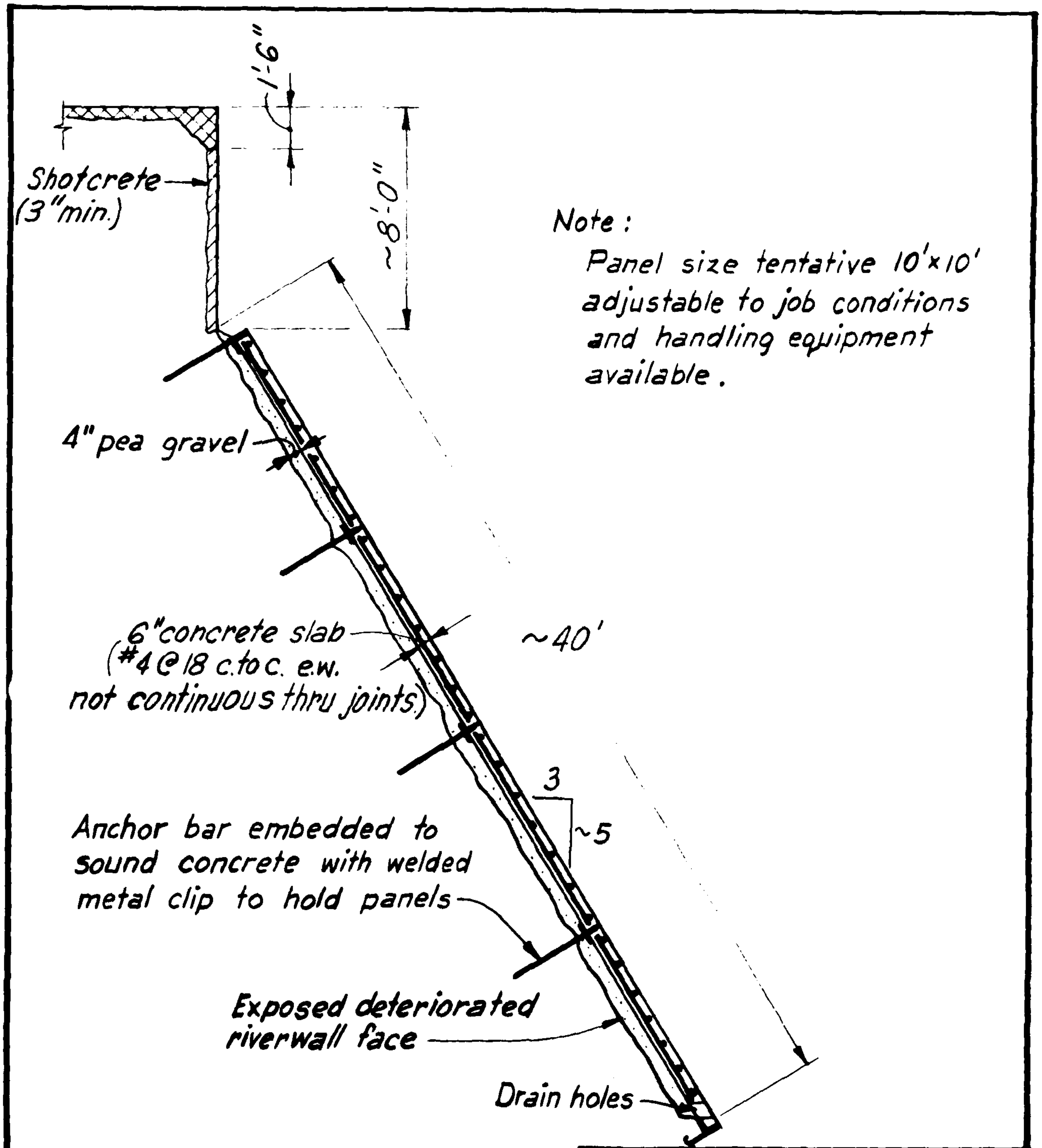
Scale: 1" = 10'

STUDY of ALTERNATIVES for REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO. 1

CONCRETE CONDITION SURVEY
SURFACE COMPRESSIVE STRENGTH

ST. PAUL, MINNESOTA DISTRICT
FILE NO. DATED: MARCH 1975

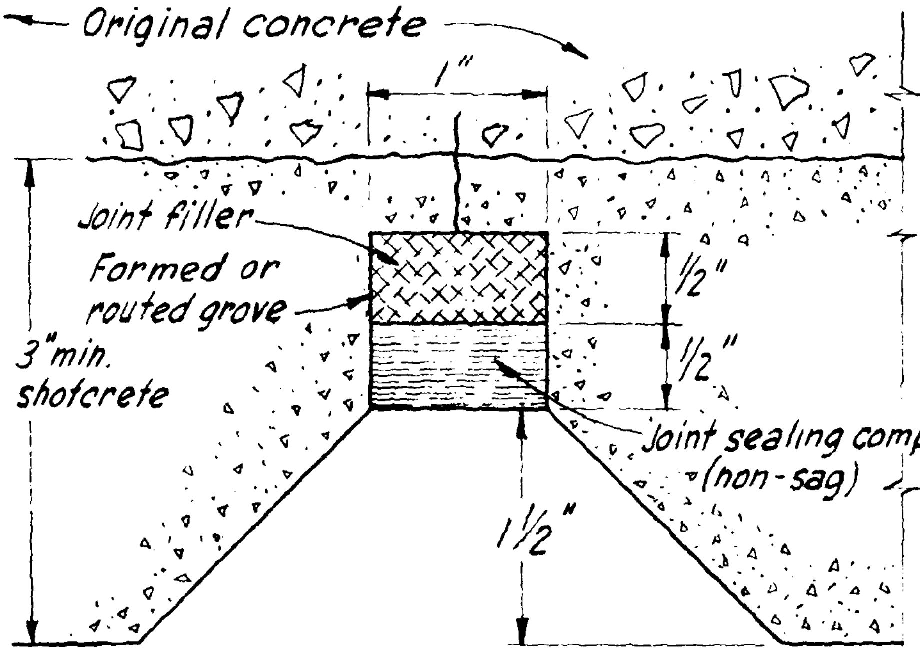
PLATE F-13



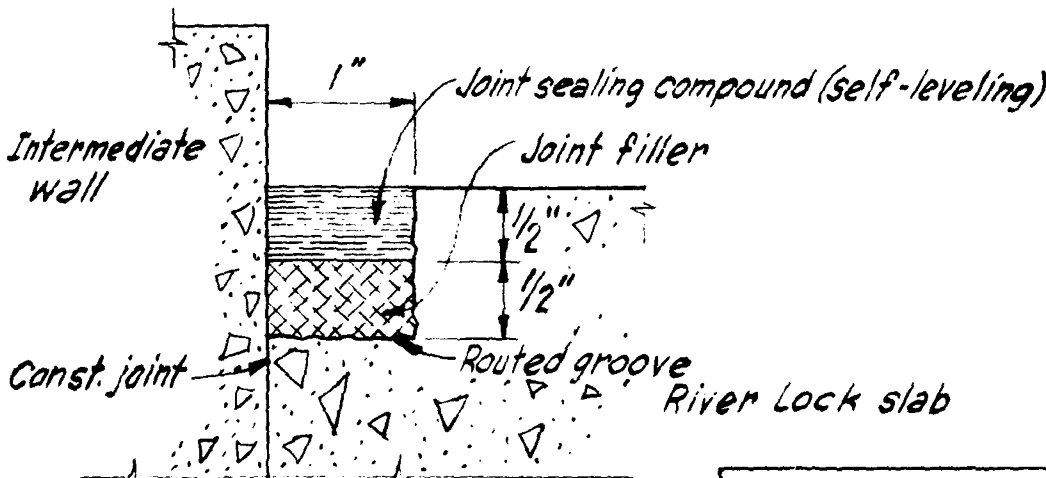
STUDY of ALTERNATIVES for REHABILITATION
MISSISSIPPI RIVER LOCK & DAM NO.1

CONCRETE REPAIR
EXPOSED RIVER WALL

ST. PAUL, MINNESOTA DISTRICT
FILE NO. DATED: MARCH 1975



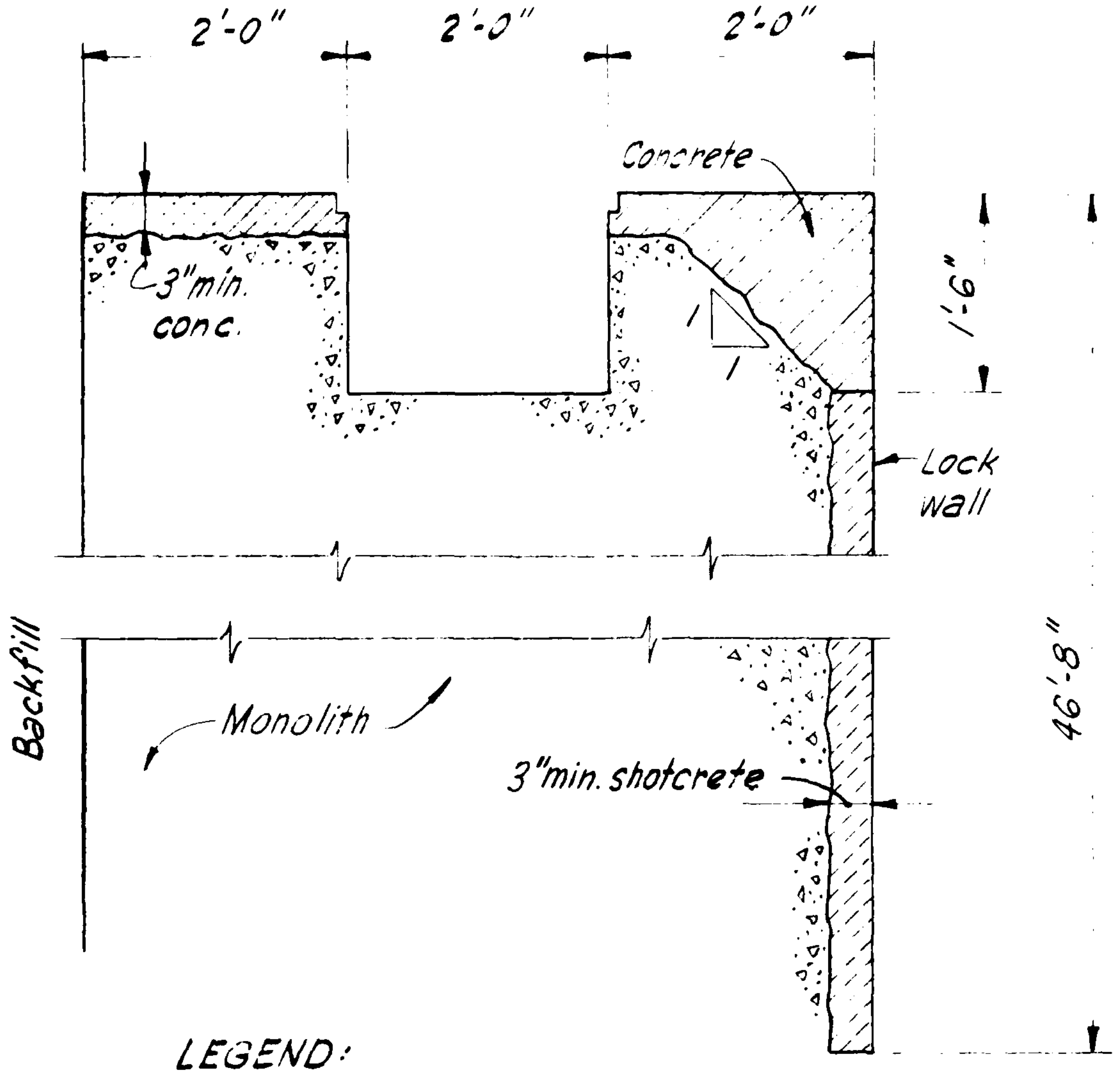
DETAIL OF VERTICAL JOINT SEALING



DETAIL - FLOOR SLAB SEAL

STUDY of ALTERNATIVES for REHABILITATION	
MISSISSIPPI RIVER	LOCK & DAM NO.1
CONCRETE REPAIR JOINT SEALING DETAILS	
ST. PAUL, MINNESOTA DISTRICT	
FILE NO.	DATED: MARCH 1975

PLATE F-15



LEGEND:

Remove existing concrete and replace with conventional concrete.

Remove existing concrete and replace with shotcrete

NOTE:

1. Reinforcement - 4X4-6/6 mesh
2. All vertical contraction joints between monoliths will be maintained.

STUDY of ALTERNATIVES for REHABILITATION	
MISSISSIPPI RIVER	LOCK & DAM NO. 1
CONCRETE REPAIR LOCKS	
ST. PAUL, MINNESOTA DISTRICT	
FILE NO.	DATED: MARCH 1975

Shotcrete Repairs

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

ESTIMATE

Project Lock & Dam No. 1 - 600A **Date** January 1975 **Page** [illegible] **of** 1

Structure Locks - Concrete Repair **Estimated by** [illegible] **Checked by**

-Land Lock Only-

Item No.	ITEM	Plan I Quantity	Plan I Unit Price	Plan I Amount		Plan II Quantity	Plan II Unit Price
	Alt "A" Remove Concrete	2,000 c.y.	100.00	200	000		
	Shotcrete	1,380 c.y.	150.00	207	000		
	Concrete	900 c.y.	120.00	108	000		
	Sand Bedding	350 tons	5.00	1	750		
	Repair Cracks & Joints	240 l.f.	4.00		960		
	Epoxy Grout	30 gals.	25.00		750		
	Portland Cement	760 tons	45.00	34	200		
	Reinforcing Steel	91,700 lbs.	0.40	36	680		
	Subtotal			589	340		
	Alt "B" Remove Concrete	1,860 c.y.	100.00	186	000		
	Shotcrete	1,260 c.y.	150.00	189	000		
	Concrete	600 c.y.	120.00	72	000		
	Repair Cracks & Joints	240 l.f.	4.00		960		
	Epoxy Grout	30 gals.	25.00		760		
	Portland Cement	640 tons	45.00	28	800		
	Reinforcing Steel	73,100 lbs.	0.40	29	240		
				506	750		
	Alt "C" Remove Concrete	1,500 c.y.	100.00	150	000		
	Shotcrete	1,260 c.y.	150.00	189	000		
	Concrete	240 c.y.	120.00	28	800		
	Repair Cracks & Joints	240 l.f.	4.00		960		
	Epoxy Grout	30 gals.	25.00		750		
	Portland Cement	550 tons	45.00	24	750		
	Reinforcing Steel	56,600 lbs.	0.40	22	640		
	Subtotal			416	900		

Page 1 of 1 Pages

Checked by

ly-

-Both Locks-

Plan II Unit Price	Plan II Amount	Plan III Quantity	Plan III Unit Price	Plan III Amount	Plan IV Quantity	Plan IV Unit Price	Plan IV Amount
					Alt "D" 2,900 c.y.	100.00	290 000
					2,300 c.y.	150.00	345 000
					900 c.y.	120.00	108 000
					350 tons	5.00	1 750
					1,100 l.f.	4.00	4 400
					135 gals	25.00	3 375
					1,110 tons	45.00	49 950
					125,800 lbs	0.40	50 320
	589 340			589 340			589 340
	506 750			506 750			
	416 900			416 900			

Plate F-17

2

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

ESTIMATE

"Glas-Con" Coating

Project Lock & Dam No. 1 - 800A **Date** January 1975 **Page** 1 **of** 1

Structure Locks - Concrete Repair **Estimated by** RM **Checked by**

-Land Lock Only-

Item No.	ITEM	Plan I Quantity	Plan I Unit Price	Plan I Amount	Plan II Quantity	Plan II Unit Price
	Alt."A" Remove Concrete	810 c.y.	100.00	81 000		
	Shotcrete	210 c.y.	150.00	31 500		
	Sandblasting	84,000 s.f.	1.00	84 000		
	Glas-Con Coating	84,000 s.f.	3.25	273 000		
	Sand Bedding	350 tons	5.00	1 750		
	Repair Cracks & Joints	240 l.f.	4.00	960		
	Epoxy Grout	30 gals.	25.00	750		
	Portland Cement	320 tons	45.00	14 400		
	Steel Reinforcemnt	48,600 lbs.	0.40	19 440		
	Concrete	900 c.y.	120.00	108 000		
	Subtotal			614 800		
	Alt."B" Remove Concrete	810 c.y	100.00	81 000		
	Shotcrete	210 c.y.	150.00	31 500		
	Sandblasting	77,100 s.f.	1.00	77 100		
	Glas-Con Coating	77,100 s.f.	3.25	250 575		
	Repair Cracks & Joints	240 l.f.	4.00	960		
	Epoxy Grout	30 gals.	25.00	750		
	Portland Cement	235 tons	45.00	10 575		
	Steel Reinforcement	33,200 lbs.	0.40	13 280		
	Concrete	600 c.y.	120.00	72 000		
	Subtotal			537 740		
	Alt. "C" Remove Concrete	570 c.y.	100.00	57 000		
	Shotcrete	210 c.y.	150.00	31 500		
	Sandblasting	77,100 s.f.	1.00	77 100		
	Glas-Con Coating	77,100 s.f.	3.25	250 575		
	Repair Cracks & Joints	240 l.f.	4.00	960		
	Epoxy Grout	30 gals	25.00	750		
	Portland Cement	175 tons	45.00	7 875		
	Steel Reinforcement	23,000 lbs.	0.40	9 200		
	Concrete	370 c.y.	120.00	44 400		
	Subtotal			479 360		

PANY

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ed by GRM Checked by

Lock Only-						-Both Locks-		
Plan II			Plan III			Plan IV		
Quantity	Unit Price	Amount	Quantity	Unit Price	Amount	Quantity	Unit Price	Amount
						Alt. "D" 850 c.y.	100.00	85 000
						248 c.y.	150.00	37 200
						141,800 s.t.	1.00	141 800
						141,800 s.f.	3.25	460 850
						350 tons	[illegible].00	1 [illegible]50
						1,100 l.f.	4.00	4 400
						135 gals.	25.00	3 3[illegible]
						330 tons	45.00	14 850
						49,000 lbs.	0.40	19 [illegible]00
						900 c.y.	120 00	108 000
		614 800			614 800			876 825
		537 740			537 740			
		479 360			479 360			

Plate F-18

2

1

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

ESTIMATE

Project Rehabilitation of Lock & Dam No. 1 Date March 1975 Page 43 of

Structure Repair of Concrete Surfaces Estimated by JAT Checked

Item No.	ITEM	Plan No. 1 Quantity	Unit Price	Amount		Plan No. 2 Quantity	Unit Price
	Repair of Concrete Surfaces						
	(without wall armor)						
	Concrete Removal	1650 c.y.	65.00	107	250		
	Shotcrete	1150 c.y.	175 00	201	250		
	Concrete (w/o Reinf. Steel)	500	80.00	40	000		
	Precast Panels	220 c.y.	200.00	44	000		
	Formwork	9000 Sq. Ft.	3.00	27	000		
	Bedding (Pea Gravel)	150 c.y.	10.50	1	575		
	Repair Cracks	250 Lin. Ft.	4.00	1	000		
	Epoxy Grout	50 Gal	25.00	1	250		
	Rock Bolts For Mesh	3120 Lin. Ft	6.00	18	720		
	Reinforcement (Rebar and Mesh)	75.000 lbs.	0.40	30	000		
	Upper Protection Angle	126 000 lbs	1.00	126	000		
	Wall Armor						
	Anchor Bars 4 ft.	3,100 lbs.	1.75	5	425		
	Anchors 3/4" Dia x ft.	3,500	2.00	7	000		
	Joint Sealer	11.000 Lin. Ft.	3.00	33	000		
				643	470		
			USE	643	000		

PANY

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Plan No. 2 Quantity	Unit Price	Amount	Plan No. 3 Quantity	Unit Price	Amount	Plan No. 4 Quantity	Unit Price	Amount
						2.385 c y	65.00	155 025
						1,885 c y.	175.00	329 825
						500 c.y.	85 00	42 500
						220 c y	200.00	44 000
						12,500 sq. ft.	3.00	3[illegible] 500
						150 c y	10 50	1 575
						1,100 lin. ft.	4.00	4 400
						135 gal	25.00	3 375
						5,250 lin ft	[illegible].00	31 500
						98,000 lbs.	0 40	39 200
						126,000 lbs.	1 00	126 000
						3,100 lbs	1 75	5 425
						3,500 lbs	2 00	7 000
						11 000 lin. ft	3 00	33 000
		643 470			643 470			860 375
		643 000			643 000			960 000

Plate F-17

2

HARZA ENGINEERING COMPANY
CHICAGO, ILLINOIS

ESTIMATE

Project Rehabilitation of Lock & Dam No.1 Date March 1975 Page 18 of
Structure Repair of Concrete Surfaces Estimated by JAT Checked

Item No.	ITEM	Plan No. 1 Quantity	Unit Price	Amount			Plan No. 2 Quantity	Unit Price
13	Repair of Concrete Surfaces							
	(with wall armor)							
	Concrete removal	2,200 c.y	65.00		143	000		
	Shotcrete	960 c y.	175.00		168	000		
	Concrete (w/o steel reinf.)	1,170 c.y.	80 00		93	600		
	Precast Panels	220 c.y.	200.00		44	000		
	Formwork	19,000 sq. ft.	3.00		57	000		
	Bedding,Pea gravel	150 c.y.	10.50		1	575		
	Repair Cracks and Joints	250 lin.ft.	4.00		1	000		
	Epoxy Grout	30 Gal	25.00			750		
	Rockbolts for mesh	2,500 lin.ft.	6.00		15	000		
	Steel Reinforcement (bars and mesh)	68,000 lbs.	0 40		27	200		
	Upper Protection steel angle	126 000 lbs.	1.00		126	000		
	Wall armor, steel	379,000	1.00		379	000		
	Anchor bats 4 ft.	8.600 lbs.	1.75		15	050		
	Anchors 3/4" dia x 1 ft.	8,200 lbs.	2.00		16	400		
	Joint sealer	11 000 lin.ft.	3.00		33	000		
				1	120	575		
			USE	1	121	000		

ANY

1975 Page 18 of 20 Pages

d by JAT Checked by GJK

Plan No. 2 Quantity	Unit Price	Amount	Plan No. 3 Quantity	Unit Price	Amount	Plan No. 4 Quantity	Unit Price	Amount
						2,900	65.00	188 500
						1,660 c.y.	175.00	290 500
						1,260 c.y.	85.00	107 100
						220 c.y.	200.00	44 000
						22,000 sq. ft.	3.00	66 000
						150 c.y.	10.50	1 575
						1,100 lin ft	4.00	4 400
						135 Gal	25.00	3 375
						3,550 lin.ft.	6.00	21 300
						96,000 lbs.	0 40	38 400
						126,000 lbs.	1.00	126 000
						553,000 lbs	1.00	553 000
						9,500 lbs.	1 75	16 625
						10,800 lbs.	2.00	21 600
						11,000 lin.ft.	3.00	33 000
		1 120 575			1 120 575			1 515 375
		1 121 000			1 121 000		USE	1 515 000

Plate F-20

2

PETROGRAPHIC EXAMINATION OF SAMPLES OF CONCRETE, LOCK AND DAM NO. 1, ST. PAUL, MINNESOTA

INTRODUCTION

In accordance with the letter of transmittal dated August 6, 1974, from Mr. Gary R. Mass, Concrete and Materials Engineer, Harza Engineering Company, Chicago, Illinois, I have examined by petrographic methods concrete core specimens that were received on August 14, 1974, by United Parcel Service.

The samples were identified as portions of NX core, 2 to 3 in. in length, taken at random intervals from three drill holes. The project was identified as Lock and Dam No. 1, Harza Engineering Company Project No. 800A, St. Paul, Minnesota, located on the Mississippi River. The samples were identified as follows:

Drill Hole No.	74-48	74-39	74-50
Location	River Wall	Center Wall	Land Wall
Date Placed	1900-08	1930	1931-32
Sample Depth, ft.	2.4	0.6	1.6
	9.2	6.8	9.5
	17.7	13.2	16.2
	23.8	21.2	27.3
	31.7	29.9	35.2
	37.1	37.2	42.7
	44.4	44.0	43.7*
	48.1*	52.0	52.4
		56.5	59.0

* Not listed in the letter of transmittal.

It was requested that the samples be examined to determine the overall condition and integrity of the concrete in each drill hole. Each of the specimens was examined visually and in detail under the stereoscopic microscope. For each drill hole, a selection of specimens was prepared by sawing and lapping so as to permit detailed examination of the internal structure of the concrete in an undisturbed condition when viewed at high magnification. Such sections facilitate the detection of microcracking within the cement paste and mortar matrix, in aggregate particles, or at the boundary of aggregate particles; evidences of cement-aggregate reactions; the occurrence of secondary chemical deposits with cracks, air voids, or at aggregate boundaries; variations of texture and composition within the cement paste; and identification of aggregate constituents without destruction of the concrete specimen. Selected portions were examined in immersion oils under the petrographic microscope.

CONCLUSIONS

River Wall, Drill Hole No. 74-48

1. The concrete includes crushed dolomite coarse aggregate and a natural sand fine aggregate. No finely divided mineral admixture is present in the matrix of essentially completely hydrated portland cement.

2. The concrete is relatively weak at a depth of 2.4 ft. but is moderately hard at greater depth. The fresh concrete was subject to significant bleeding (water gain), which resulted in separations beneath particles of coarse aggregate and variability of firmness and porosity of the cement paste matrix because of variation of water-cement ratio and accumulation of aggregate fines. The concrete represented by the samples is free from significant cracking, except for isolated microcracking specifically related to particles of aggregate involved in the alkali-silica reaction (see below). No evidence of progressive deterioration of the cement paste matrix, such as sulfate attack or leaching was found.

3. The dolomite coarse aggregate was subject to a moderate degree of the so-called alkali-silica reaction and possibly to the alkali-carbonate rock reaction as well. These reactions involve the attack of the alkalies sodium and potassium, primarily originating the cement, upon constituents of the aggregate. In the case of the alkali-silica reaction, the susceptible constituents in the dolomite were not identified specifically, but are probably very finely divided quartz and microcrystalline quartz that is present in silty and clayey fractions within the matrix of the dolomite. Some writers studying somewhat similar rock types have concluded that illitic clay is susceptible of deleterious alkali-silica reactions in concrete; illite is present also within the matrix of the dolomite.

In any event, the examination of the submitted samples reveals the occurrence of moderately extensive alkali-silica reaction, the effects being formation of peripheral reaction rims on affected particles, minor microcracking within the rock particles, very rare microcracking of the adjacent cement paste matrix of the concrete, and production of moderate to copious amounts of alkalic silica gel, most of which has now been converted by reaction with calcium hydroxide into limited swelling calcium-alkali-silica gels.

4. The sand fine aggregate includes a variety of types of chalcedonic cherts which show evidences of the alkali-silica reaction in the form of softening, darkened rims, and stained of the adjacent cement paste matrix as a result of outward permeation of alkalic silica gel. However, except for occasional particles of a green chalcedonic chert, which occurs in pebbles as much as one inch across, no distress of the concrete was found to relate to alkali reactivity of the cherts. The green cherts are totally reacted, that is, converted to calcium-alkali-silica gels that are waxy in consistency and internally cracked and occupy the original volume of the aggregate particle. Except for particles of green chert whose minimum dimension exceeds about 7 mm (occurring in samples at 31.7, 37.1, and 48.1 ft.), no visible cracking of the adjacent matrix was found. However, in these three instances, cracks extend from the reacted chert particle into the surrounding matrix. In the occurrences at 31.7 and 48.1 ft., alkalic silica gel in copious amounts has spread outwardly and has created extensive cracking of the mortar matrix.

Although the larger particles of the highly reactive green chert were found at only the three depths, there is no reason to believe that they are not distributed sparsely throughout the entire mass of concrete. Their presence in the particular samples is the result of random chance.

5. In my opinion, the concrete represented by the sample of the drilled core is not in a condition of distress. The examination indicates that the alkali-silica reaction involving the dolomite coarse aggregate has terminated in the production of calcium-alkali-silica gels of limited swelling capacity, not unlike the calcium silicate hydrates that form the cement paste matrix of any portland cement concrete. Upon 48 hr. of soaking of the concrete, no gel exudations related to the dolomite aggregate were developed, whereas other concretes affected by progressive alkali-silica reaction produce obvious exudations and pop-outs during similar immersion as a result of rapid osmotic

swelling of alkalic silica gels already present in the aggregate or in cracks and voids.

As noted above (Conclusion 4), active alkalic silica reaction is present in the relatively rare, large particles of the green chalcedonic chert. Such particles constitute only a fraction of one percent of the sand fine aggregate, according to the sampling represented by the concrete submitted. In the particles of green chert which are in a size range less than about 7 mm in diameter the chalcedonic matrix is converted entirely to limited swelling calcium-alkali-silica gels. Only in the very rare larger particles has the residue of alkalic silica gel within the core of the particle been preserved where it is subject to development of high swelling pressure upon wetting. However, even here, considering that the potential swelling pressure is only on the order of 600 psi, potential distress is to be expected only where the confining pressure is less than this amount.

6. The potential for residual expansion of the concrete can be determined by laboratory testing of core specimens that are fitted with gauge points and are maintained in a moist atmosphere for an extended period, during which length change measurements are made.

REC'D.-HARZA ENGINEERING CO. OCT 21 1974

RICHARD C. MIELENZ — GEOLOGIST AND PETROGRAPHER

PETROGRAPHIC EXAMINATION OF SAMPLES OF CONCRETE, LOCK AND DAM NO. 1 MISSISSIPPI RIVER

INTRODUCTION

In accordance with the letter of transmittal dated September 10, 1974, from Mr. E. T. Moore, Project Manager, Harza Engineering Company, Chicago, Illinois, I have examined by petrographic methods four samples of portland-cement concrete that were received by truck freight on September 21, 1974.

The shipment consisted of four sections of nominally 6-in. diameter drilled cores designated as Nos. 1 through 4, respectively. The samples were secured in connection with proposals for rehabilitation of concrete structures of Lock and Dam No. 1 - Mississippi River. Three of the samples were said to have been taken from the Land Wall of the locks, which shows most severe damage in the form of extensive spalling at various locations. It was requested that the samples be examined to provide an assessment of the surface condition and soundness of the concrete.

One of the core was taken from the land side of the Center Wall and was submitted for examination and comparison. The concrete in this location appears to be sound.

Unfortunately, the identification of the source of the respective cores was not indicated. However, from the appearance of the samples, it is presumed that Core No. 4 was taken from the Center Wall.

The samples were examined visually and in detail under the stereoscopic microscope. For each of the cores a section parallel to the length was prepared by sawing and lapping so as to allow examination of the concrete in an undisturbed condition. Selected portions of the concrete were examined in immersion oils under the petrographic microscope.

CONCLUSIONS

1. The four samples of concrete are alike in the quality and composition of the coarse and fine aggregates and the original quality of the concrete as placed. The aggregates are natural gravel and sand, the apparent nominal maximum size

Report No. 2
Introduction and Conclusions
Dated October 15, 1974

being 1-1/2 in. In all instances, the concrete was well compacted and cured and shows no evidence of significant bleeding or segregation before setting. As would be expected of concrete of the age of this construction, the concrete is not air entrained.

2. Cores No. 1, 2, and 3 show varying degrees of distress as the result of the alkali-silica reaction and the service exposure. These cores show the concrete to be so severely damaged by cracking and, in the outer portions of Cores Nos. 1 and 3, by chemical deterioration of the matrix to require consideration of removal to the following depths in preparation for restoration:

Core No. 1	1-1/4 to 4-1/2 in. from the existing surface
Core No. 2	At least 10-1/2 in. (representing the depth penetrated)
Core No. 3	Approximately 7 in. from the formed surface.

3. Three destructive agents have been effective in causing the observed distress of the concrete, namely, (1) alkali-silica reaction, primarily involving certain constituents of the coarse aggregate, (2) sulfate attack upon the cement paste matrix, and (3) freezing and thawing while the concrete was saturated or nearly saturated by water.

a. The alkali-silica reaction is evident in all of the cores, including Core No. 4, which was submitted for comparison and shows no significant distress. This reaction is most well developed in Core No. 1, in which particles of graywackes, red siltstones, and chalcedonic cherts are affected and have caused minor cracking of the matrix. This reaction is only incipiently developed in the remaining three cores and, in these three latter cores only the cherts and siltstones are involved. In my opinion, the alkali-silica reaction has essentially terminated through the development of calcium-alkali-silica gels of limited swelling capacity.

b. Cores No. 1 and 3 display severe deterioration of the cement paste matrix adjacent to the exterior surface of the wall, with formation of copious secondary chemical deposits, cracking of the near-surface concrete, and spalling and general disintegration. The deposits include abundant amounts of the high-sulfate calcium sulfoaluminate (ettringite) and gypsum (calcium sulfate dihydrate) as a result of introduction of sulfate from an external source and its attack upon constituents, mainly the aluminates, that are a normal portion of portland cement hydration products. The attack has produced severe damage only to the depths noted above, but the presence of the secondary sulfates was detected throughout the length of the core sections submitted.

Core No. 2 showed minor effects of the sulfate attack but much less than that evident in Cores 1 and 3. No gypsum was found in Core No. 2, gypsum being a product of more extreme sulfate attack.

c. Although the evidences of its effects are indirect, freezing and thawing undoubtedly produced disintegration that is superimposed upon the distress resulting from sulfate attack and the alkali-silica reaction. Cracking resulting from these processes permits entry of water into the interior of the concrete, thus increasing the degree of saturation of the cement paste matrix and of porous, unsound particles of coarse aggregate. The cement paste matrix is susceptible of damage in freezing and thawing while in a wetted condition because of the lack of air entrainment and the fact that the water-cement ratio is in an intermediate range. As is noted in the text, a particle of white, highly porous chert lying 6 cm from the exterior surface had been disrupted internally and had produced subradiate cracks that extend into the surrounding mortar matrix; this phenomenon is common in concrete containing highly porous particles of this type when the particles are above a critical size, such as 3/8 to 1/2 in. for porous cherts. In such cases, failure is within the particle, rather than in the cement paste matrix.

In my opinion, freezing and thawing is the primary cause of the internal cracking of the concrete represented by Core No. 2. The repeated cracking oriented more or less parallel to the exposed face of the constructions is characteristic of the action of freezing and thawing of non-air-entrained concrete in a saturated or near-saturated condition. In this instance, the alkali-silica reaction and sulfate attack are subordinate but probably contributory agents.

4. The examination indicates that the restoration should be carried out with air-entrained concrete containing Type II cement, preferably in combination with a suitable fly ash admixture in an amount about 15 per cent by weight of the cement content to assure proper sulfate resistance. Use of aggregates not susceptible to the alkali-silica or alkali-carbonate rock

reaction is strongly recommended because of the possible availability of alkalies (sodium and potassium) from an external source in these constructions.

REC'D.-HARZA ENGINEERING CO. DEC 13 1974

RICHARD C. MIELENZ — GEOLOGIST AND PETROGRAPHER

PETROGRAPHIC EXAMINATION OF CONCRETE CORE SPECIMENS, LOCK AND DAM NO. 1, MISSISSIPPI RIVER

INTRODUCTION

In accordance with the letter of transmittal dated November 13, 1974, from Mr. E. T. Moore, Project Manager, Harza Engineering Company, Chicago, Illinois, I have examined by petrographic methods ten concrete core specimens that were received by truck freight on November 27, 1974.

The shipment consisted of ten sections of drilled cores of portland cement concrete from Lock and Dam No. 1, Mississippi River. According to the letter of transmittal, five of the specimens are from the Landward Wall, Monolith #12, at distances down from the top of the wall as follows: 7, 15.5, 24, 32.5, and 41 ft. Note that the cores were marked "West Wall, Monolith #12." The other five cores were reported to have been drilled from the opposite landward side of the Center Wall, Monolith #I-13, at the same respective elevations. The cores were marked "East Wall, Monolith #I-13." It was requested that the specimens be examined by petrographic methods to determine the surface condition of the concrete with details regarding the depth of deterioration or to sound concrete, and factors responsible for deterioration observed.

The specimens were examined visually and in detail under the stereoscopic microscope. Each of the specimens was sawed longitudinally through the entire depth in which cracking or other evidences of deterioration could be detected. The sections so obtained were prepared by lapping so as to permit detailed examination of the internal structure of the concrete in an undisturbed condition. Selected portions of the concrete and secondary deposits were examined in immersion oils under the petrographic microscope.

CONCLUSIONS

1. The examination indicates that the concrete represented by the ten samples was originally very similar in composition, degree of consolidation, lack of significant bleeding or segregation of the fresh concrete, and quality and composition of the coarse and fine aggregates. The aggregates are natural sand and gravel, the apparent nominal maximum size being 1-1/2 in. In all instances, the concrete was well cured. As would be expected of concrete of the age of these constructions, the concrete is not air entrained.

Report No. 3
Introduction and Conclusions
Dated December 9, 1974

2. Several of the cores include varying degrees and kinds of cracking, as follows: (1) Laminar cracking which is cracking more or less parallel to the external surface of the wall, occurring mainly within the mortar matrix but commonly intersecting particles of aggregate, and being sufficiently closely spaced to produce a laminated structure in the affected concrete; (2) longitudinal cracking, that is, cracks that pass into the concrete approximately perpendicular to the outer surface of the wall to depths of 7 or 8-1/4 in.; (3) microcracking spatially related to individual particles of coarse aggregate that were significantly involved in the alkali-silica reaction; and (4) microcracks originating at specific particles of highly porous chert or argillaceous dolomites in the outermost 4-1/2 in. or so of the section, evidently as a result of bursting of the particle as a consequence of freezing while the particle was saturated or nearly saturated by water.

The condition of the specimens in these respects is summarized in the following tabulation:

Summary of Condition of the Specimens

Core No.	Depth of Laminar Cracking, in.	Remarks
1	None	Exterior mortar repair
2	6-15/16	Exterior mortar repair; cracking of Types 3 and 4, above
3	4-1/4	
4	None	Cracking of Type 3, above
5	4	Cracking of Type 3, above
6	None	Cracking of Type 2, above, to depth of 7 in.
7	None	Cracking of Type 2, above, to depth of 8-1/4 in.; also cracking of Types 3 and 4
8	None	
9	4-3/4	
10	11/16	

3. In my opinion consideration should be given to removal of the laminated concrete as shown in Cores 2, 3, 5, 9, and 10 to the indicated depths or to such depths as are indicated by scaling operations at the site. The significance of the longitudinal cracking exhibited by Cores No. 6 and 7 cannot be assessed because of the limited area represented by the cores.

4. The microcracking classified as Types 3 and 4, above, probably is not significant. although it contributes to general deterioration, primarily by increasing the ease with which water can penetrate the concrete and maintain a high degree of saturation, thus increasing susceptibility of the concrete to the effects of freezing and of freezing and thawing.

5. Sulfate attack upon the cement paste matrix of the concrete is indicated by the widespread development of ettringite (high-sulfate calcium sulfoaluminate) in voids, fractures, and openings in aggregate sockets, and also in most of the cores within the cement paste matrix itself. In Cores No. 2, 7, and 9, gypsum (calcium sulfate dihydrate) was found within secondary deposits in voids and cracks, representing a somewhat more extensive exposure to sulfate-bearing waters. However, the examination indicates that sulfate attack is not a primary cause of the observed distress.

6. All of the specimens show evidence of incipient or moderate alkali-silica reaction involving specific lithologic constituents of the coarse and fine aggregate, mainly graywacke sandstones, red quartzose siltstones, olive green cherts, and white chalcedonic cherts. In general, the extent of the reaction and its physical effects are not significant. Microcracking from this cause was found in Cores No. 2, 4, 5, and 7.

7. In my opinion, the primary cause for the distress of the concrete showing laminar cracking is freezing and thawing occurring while the concrete was in a condition of saturation or near-saturation by water. The effects of freezing and thawing would be aggravated by any factors that increase the absorptivity of the concrete, such as microcracking as a consequence of the alkali-silica reaction, sulfate attack on the cement paste, or disruption of highly porous particles of aggregate in the near-surface zone.

8. In all instances, the submitted specimens appear to have penetrated completely any distressed zones in the concrete at the location of the drill hole.

GLAS-CRETE
A MINNESOTA CORPORATION
FIBERGLAS REINFORCED CONCRETE PRODUCTS

December 16, 1974

Corps of Engineers
Room 1305 U.S. Post Office & Custom House
St. Paul, Minnesota 55101

Attention: Mr. Starkey Grove Jr.
Chief Maintenance Branch

Gentlemen:

This letter is a follow up on our telephone conversation of last week wherein we discussed the possible application of our product, GLAS-CON, in patching surfaces of mass concrete.

Enclosed herewith are some samples of the product and testing lab report. The most significant properties for patching are

1.	Absorption	2.5%
2.	Compressive Strength	9,440 psi (average)
3.	Tensile Strength	1,804 psi (average)

The ingredients in this sample are Hi-early cement, Silica sand, Dow latex 460, water, water reducing agent and fiberglas chopped to 4" length.

The Dow latex provides properties that are particularly important in this proposed patching application. It greatly increases the bond to the old concrete and it increases the flexibility of the cement matrix. Latex-modified mortars and concrete mixes are commonly used in patching industrial floors. The addition of glass fiber gives this material a tensile strength never before achievable.

A typical repair of a mass concrete surface would proceed as follows:

1. Break-out deteriorated concrete.
2. Sand-blast remaining surfaces.
3. Apply a layer of GLAS-CON to a minimum 3/8" thickness.
4. Float or trowel finish the surface.

The application of GLAS-CON is by spraying and roller compacting. It can be placed to 3/8" thickness at a rate of 100 to 200 SF / Hr. It should definitely be much less expensive than existing repair methods.

Report No. 4
"Glas-Con"

PHONE 612/645-3601

TWIN CITY TESTING AND ENGINEERING LABORATORY, INC.

ENGINEERS AND CHEMISTS

662 Cromwell Avenue - St. Paul, Minnesota 55114

REPORT OF: TESTS OF GLASS-CRETE

PROJECT: MATERIAL CHECK

DATE: October 18, 1974

REPORTED TO: Gunnar I Johnson & Son Inc
9320 James Ave So
Minneapolis, MN 55431
Attn: Iver Johnson

(4)

FURNISHED BY:

COPIES TO:

LABORATORY No. 6-13560

COMPRESSIVE STRENGTH:

Method of Test: ASTM C97, Air Cured Condition
Type of Specimen: 4" x 4" x 4" cubes
Date Cast: September 11, 1974

Sample Number:	1	2	3	4
Load at Failure (lb)	158,000	149,250	138,000	173,250
Area Tested (sq in.)	17.0	16.8	15.5	16.0
Unit Stress (psi)	9290	8880	8900	10,770
Orientation to Plant of Fibers	Parallel	Parallel	Perpendicular	Perpendicu
Age at Test (days)	7	28	7	28

FLEXURAL STRENGTH:

Method of Test: ASTM C99, Air Cured Condition
Type of Specimen: 4" x 8" x 1/2" Slabs (Approximate)
Date Cast: September 7, 1974

Sample Number:	1	2	3
Load at Failure (lb)	425	645	725
Thickness (inches)	0.542	0.689	0.769
Test Width (inches)	2.98	4.125	4.125
Modulus of Rupture (psi)	5100	3460	3130
Age at Test (days)	11	28	28

TENSILE STRENGTH:

Type of Test: Direct Tension, Air Cured Condition
Type of Specimen: 4" x 16" x 3/4" reduced to 3" x 3/4" at center
Date Cast: September 16, 1974

Sample Number	1	2	3	4
Load at Failure (lb)	6000	4200	4850	3100
Test Area	2.51	2.73	2.41	2.43
Unit Stress (psi)	2390	1540	2010	1275
Age at Test (days)	8	8	28	28

PHONE 612/645-3601

TWIN CITY TESTING AND ENGINEERING LABORATORY, INC.

ENGINEERS AND CHEMISTS

662 Cromwell Avenue - St. Paul, Minnesota 55114

REPORT OF: TESTS OF GLAS-CRETE

DATE: October 18, 1974

ABORATORY No. 6-13560

PAGE: Two

ABSORPTION AND DENSITY:

Method of Test:	ASTM C97
Type of Specimen:	4"x8"x1/2" Slab
Bulk Specific Gravity, Oven Dry	2.01
Bulk Density, Oven Dry (pcf)	125.4
Absorption, 24 hr Immersion (%)	2.5

MOISTURE EXPANSION:

Initial Condition:	Oven Dry
Final Condition:	Saturation by water immersion at 700 F for 24 hours
Moisture Expansion	0.022%

COEFFICIENT OF THERMAL EXPANSION: (in./in./F)

Coefficient (-45 F to 126 F)	5.7×10^{-6}

REMARKS:

The 50 cycle freezing and thawing test is also in progress. At the end of 15 cycles neither the panel having sealed edges nor the panel having unsealed edges shows any signs of deterioration.

This report is an addendum to our report of October 1, 1974.

Twin City Testing and Engineering Laboratory, Inc.

By:

645-3601

TWIN CITY TESTING AND ENGINEERING LABORATORY, INC.

ENGINEERS AND CHEMISTS

662 Cromwell Avenue - St. Paul, Minn. 55114

REPORT OF: FREEZING AND THAWING TEST

PROJECT: GLAS-CRETE

DATE: December 19, 1974

REPORTED TO: Gunnar I Johnson & Son Inc
9320 James Ave So
Minneapolis, MN 55431
Attn: Iver Johnson

FURNISHED BY:

COPIES TO:

LABORATORY No. 6-13689

GENERAL:

Freezing and thawing tests were conducted on two samples of Glas-Crete approximately 4" x 7 3/4" x 3/4" thick prepared from the sample slabs submitted to the laboratory for testing on September 13, 1974. One of the samples had the edges sealed with an epoxy coating before testing while the edges remained unsealed on the other sample. The unsealed sample was identified as #1 while the sealed sample was identified as #2.

The test specimens were subjected to 50 cycles of freezing and thawing. The cycle consisted of 16 hours in a freezer at 0 F and 8 hours in water at room temperature, approximately 70 F The samples were saturated in water for 24 hours at the start of the cycling. During interruption of the cycling procedure on week-ends, the test samples were stored in the frozen condition.

After completion of the 50 cycles, the samples were examined visually for deterioration. The samples were then subjected to flexural strength tests at the moisture condition existing in the specimens.

VISUAL EXAMINATION AFTER TEST:

The test specimens did not show any evidence of cracking or spalling at completion of the freezing and thawing cycling. No evidence of delamination or swelling was visible These observations apply both to the specimens having sealed and unsealed edges.

FLEXURAL STRENGTH TESTS: (ASTM C67, Center Point Loading)

Sample Number	1	2
Edge Condition	Unsealed	Sealed
Load at FAilure (pounds)	760	510
Span Length (inches)	7.0	7.0
Width (inches)	4.05	3.75
Thickness (inches)	0.755	0.718
Modulus of Rupture (psi)	3460	2770
Test Moisture Content (%)	4.7%	4.3%

REMARKS:

These test specimens were prepared from the samples submitted under your letter of September 13, 1974. Other properties determined on these samples were reported under our laboratory number 6-13560, report dated October 18, 1974.

Twin City Testing and Engineering Laboratory, Inc.

By

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