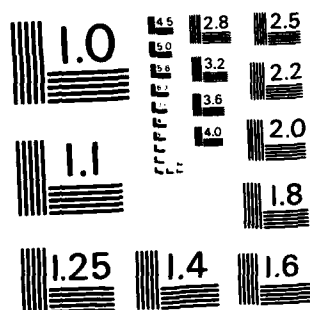UNCLASSIFIED   F/G 13/13   NL

MICROCOPY RESOLUTION TEST CHART

NATIONAL BUREAU OF STANDARDS – 1963 – A

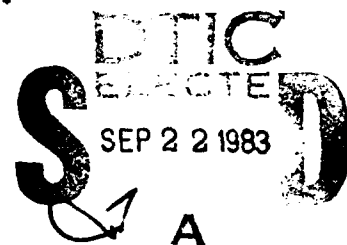# EMBANKMENT CRITERIA AND PERFORMANCE REPORT

MARCH 1983

MISSOURI RIVER

FORT RANDALL DAM-
LAKE FRANCIS CASE

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**US Army Corps
of Engineers**
Omaha District

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number)
Foundation features, earthfill design, construction, instrumentation, and performance history of the Fort Randall Dam embankment are summarized. The 165-foot high embankment was constructed in six earthwork stages, from 1947 through 1955, and is composed of approx. 50,000,000 cubic yards of rolled earthfill and chalkfill materials. It is founded on pervious alluvial soils in the valley and loess and glacial till in the abutments. Relief wells aid in controlling seepage. Instrumentation includes piezometers, settlement gages, crest and slope movement markers, tiltmeters, and strong motion accelerographs.

MISSOURI RIVER
FORT RANDALL DAM - LAKE FRANCIS CASE
SOUTH DAKOTA

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

TABLE OF CONTENTS

TABLE OF CONTENTS (CONT'D)

APPENDIX A - DRAWINGS

A-1

APPENDIX A - DRAWINGS (CONT'D)

APPENDIX B - PHOTOS

APPENDIX B - PHOTOS (CONT'D)

Plate No.	Photo No. and Description
B-11	Photo No. 20 - Construction of upstream chalk berm, looking S.W. towards right abutment. 4 Aug 53.
	Photo No. 21 - Aerial view of construction during Earthwork Stage V. 16 Nov 53.
B-12	Photo No. 22 - View of embankment construction looking along dam axis towards the right abutment. Earthwork Stage V. 25 May 54.
	Photo No. 23 - Fill placement between west spillway abutment and previously completed embankment. Top of fill is approximately El. 1,360. 25 Jun 54.
B-13	Photo No. 24 - Aerial veiw of completed embankment, looking from right abutment. 30 Jun 55.
	Photo No. 25 - Aerial view of project, looking S.W. 29 Mar 55.

MISSOURI RIVER
FORT RANDALL DAM - LAKE FRANCIS CASE
SOUTH DAKOTA

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

PERTINENT DATA

1. EMBANKMENT

Type	Rolled Earth and Chalk Fill
Height Above Stream Bed	165 Feet
Height Above Flood Plain	140 Feet
Length	10,700 Feet
Crest Elevation	1,395 Feet, m.s.l.
Crest Width	60 Feet
Volume	50,000,000 Cubic Yards
Closure Date	20 July 1952

2. SPILLWAY

Type	Concrete-lined Chute w/Gated Weir
Width	1,000 Feet
Weir Crest Elevation	1,346 Feet, m.s.l.
Gates, Type	Radial Tainter
Gates, Number and Size	21-40 Feet by 29 Feet
Elevation, Top of Gates	1,375 Feet, m.s.l.
Design Discharge Capacity at Elevation 1379.2	629,000 c.f.s.

3. OUTLET WORKS

Type	12 Concrete-lined Tunnels With Control In Intake Structure
Number, Diameter, and Length of Flood Control Tunnels	Four, 22-Foot Diameter, 873 Feet Long
Number, Diameter, and Length of Power Tunnels	Eight, 22-Foot Diameter, 873 Feet Long
Type, Number, and Size of Intake Gates	Bulkhead, 24 - 11 Feet by 23 Feet
Invert Elevation of Intake	1,229 Feet, m.s.l.
Discharge Capacity of Flood Control Tunnels	128,000 c.f.s.

4. POWERHOUSE

Length	561 Feet
Width	78 Feet
Number of Generating Units	8

Generating Capacity, Each Unit	42,105 kVA
Total Installed Capacity	320,000 Kilowatts
Power on Line	March, 1954

5. RESERVOIR

Drainage Area Above Dam	263,480 Square Miles
Drainage Area, Fort Randall Dam to Big Bend Dam	14,150 Square Miles
Storage Capacity at Maximum Pool (Elev. 1375)	5,600,000 Acre-Feet
Storage Capacity at Maximum Normal Operating Pool (Elev. 1365)	4,620,000 Acre-Feet
Flood Control Reserve (Elev. 1365 to Elev. 1375)	980,000 Acre-Feet
Annual Flood Control and Multiple-Purpose (Elev. 1350 to Elev. 1365)	1,300,000 Acre-Feet
Carry-Over Multiple-Purpose (Elev. 1320 to Elev. 1350)	1,730,000 Acre-Feet
Dead Storage, Elev. 1320	1,600,000 Acre-Feet
Length of Pool at Elev. 1365	107 Miles
Maximum Normal Operating Pool Elevation	1,365 Feet, m.s.l.
Base, Seasonal Flood Control Pool	1,350 Feet, m.s.l.

MISSOURI RIVER
FORT RANDALL DAM - LAKE FRANCIS CASE
SOUTH DAKOTA

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

1. INTRODUCTION.

1.1 **Purpose and Scope of Report.** This report provides a summary record of significant design, construction, and operational data on the Fort Randall Dam embankment. It was prepared in accordance with ER 1110-2-1901, "Embankment Criteria and Performance Report," dated 31 December 1981 and is for use by engineers to familiarize themselves with the project, reevaluate the embankment when needed, and for guidance in designing comparable future projects.

The report presents a general description of the foundation conditions, the type of material and placement methods of the various sections of the embankment, the design considerations on stability and seepage control, instrumentation, significant operational events, and an evaluation of the condition of the embankment. Pertinent drawings, design and construction data, and photos are included. A more detailed description of the foundation conditions is contained in the Construction Foundation Report prepared in 1980.

1.2 **Brief Description and Purpose of Project.** The Fort Randall Dam-Lake Francis Case Project is one of six multipurpose dam projects on the Missouri River for flood control, irrigation, navigation, power, and recreational purposes. The project is operated and maintained by the U.S. Army Corps of Engineers, Omaha District. The dam is located about 105 river miles downstream of Big Bend Dam and about 70 river miles upstream of Gavins Point Dam. As shown on the location map on Plate A-1, it is approximately 6 miles south of Lake Andes, South Dakota, and is 880 river miles (1960 adjustment) above the mouth of the Missouri River.

The project consists of an earth embankment, a multigated concrete spillway, an intake structure, multitunneled outlet works, and a hydro-electric power generating plant. The embankment is about 165 feet high above

the streambed and extends about 10,700 feet from the right abutment to the spillway structure located in the left abutment. A plan and typical sections of the project structures are shown on Plate A-2. Photo No. 1 shows an aerial view of the project.

1.3 **Authorization of Dam Project.** The Fort Randall Dam and Reservoir project was authorized by the Flood Control Act, approved 22 December 1944 (Public Law 534, 78th Congress, 2nd Session).

1.4 **Design and Construction of Project.** The project was designed by the U.S. Army Corps of Engineers, Omaha District. Members of the Board of Consultants were Dr. Arthur Casagrande, Mr. L. F. Harza, Dr. L. C. Glenn, Mr. J. D. Justin, Mr. S. O. Harper, and Mr. W. H. McAlpine.

The embankment was constructed in six earthwork contract stages as listed below.

Embankment Construction Stage	Contract No.	Contractor	Date Started
Foundation Preparation Left Bank Chute	W-25-066- ENG-1350	Peter Kiewit Sons Co.	Sep 47
Initial Earthwork	W-25-066- ENG-1434	Western Contracting Corp.	Feb 48
Earthwork Stage II	W-25-066- ENG-1837	Western Contracting Corp.	Feb 49
Earthwork Stage III	DA-25-066- ENG-363	Western Contracting Corp.	Jun 50
Earthwork Stage IV	DA-25-066- ENG-2139	Western Contracting Corp.	Nov 52
Earthwork Stage V	DA-25-066- ENG-2531	List and Clark Construction Co.	Jul 53

Work on the outlet works tunnel started in December 1948 and on the spillway, in April 1951. The river closure and diversion was made on 20 July 1952 during earthwork Stage III and the embankment was essentially completed by June 1955. The embankment pressure relief well system was installed by Bassett Drilling Company during 1953 and 1954 under Contract No. DA-25-066-ENG-2571. Some of the wells were extended by the Stage V earthwork contractor. All of the contracts were administered by the Corps of Engineers, Omaha District. Field supervision was by personnel of the Fort Randall area office which was located in Pickstown, a town constructed to accommodate the large number of people who were involved in the construction of the dam project.

1.5 **Significant Operational Events.** The reservoir was initially raised to Elev. 1,340 in 1954 and to Elev. 1,350 in 1957. Since 1971, it has fluctuated annually from a low of about Elev. 1,340 in the winter to about Elev. 1,355 to Elev. 1,360 in the summer. It attained its highest level of Elev. 1,366.5 on 23 June 1967.

Erosion of the upstream chalk berm by wave action since 1957 required the placement of riprap protection in 1981 along the erosion scarp which extended over the full length of the berm.

1.6 **Reference Project Publications.** Detailed information on the constructed dam foundation, evaluation of relief wells, project maintenance, and periodic inspections are included in the following Omaha district manual and reports:

a. Construction Foundation Report (September 1980).
b. Embankment Relief Well Study (August 1982).
c. Operation and Maintenance Manual (1982).
d. Periodic Inspection Reports Nos. 1, 2, 3, and 4, the latest of which is dated June 1981.

2. GEOLOGY.

2.1   General. Fort Randall Dam is located in the eastward sloping Missouri Plateau Section of the Great Plains physiographic province. The dam lies across an entrenched glacial melt-water channel at the southwestern margin of glaciated eastern South Dakota. During the mid-Pleistocene Period melt water ran along the front of the Illinoian Glacier which had advanced into South Dakota. When the glacier retreated the melt-water channel had entrenched enough to remain as the infant Missouri River. The river trench was partly filled during the later Wisconsin glaciation, but it retrenched its valley in the Recent Period.

The dam is located on generally flat lying, well compacted, sedimentary formations deposited by continental seas during the Cretaceous Period. Although the formations may be locally considered horizontal, they display an undulating trend on a larger scale. This is due to minor folding or differential compaction over the underlying Precambrian basement structure. The Precambrian basement consists predominantly of a NW-SE trending quartzite ridge in eastern South Dakota giving way in east-central South Dakota to a broad, flat structural platform which slopes gently westward.

The valley at the damsite is approximately 8,000 feet wide with a low flood plain elevation of about 1,250. The left abutment is formed by a promontory which rises between two deep tributary ravines to 160 feet above the flood plain; the ground then rises gently for a distance of about 3,000 feet to an elevation of 1,440. The left flood plain is 250 feet wide at the dam axis and increases in width upstream to about 2,000 feet at the head of the spillway approach channel. The right flood plain was just a few feet wide on the dam axis, but increased rapidly in width downstream. The right abutment is on a 60-foot high promontory formed by an eroded river terrace. The terrace rises to the west with a 1V on 40H slope for a distance of about 2,600 feet; it then rises on a 1V on 20H slope for about 1,400 more feet. Prior to construction of the dam, the river at the site was divided by an

4

alluvial island into a left 1,100-foot-wide channel and a right 1,500-foot-wide channel. The island was about 1-1/4 mile long, 1,400 feet wide, and had a surface elevation of about 1,245 feet.

2.2   Subsurface Explorations. After the present damsite was selected, 11 rotary and churn drill holes were made in 1941. An additional 51 borings and three test pits were made in 1942. From 1946 and throughout the design and construction period, over 450 additional borings and 28 test pits were made to determine the foundation conditions. The borings included 3-inch core borings, 6-inch core borings, 12-inch auger holes, and three 36-inch diameter calyx holes. Boring locations are shown on Plate A-8. The drilling program and other foundation investigations such as bedrock contouring, pumping tests to determine the permeability of the valley sands, and geologic mapping of excavations during construction are described in the referenced Construction Foundation Report.

2.3   Ground Water. Ground water levels were recorded in numerous exploratory drill holes prior to construction. The water table in the river valley ·and was as much as 12.0 feet below ground surface, but it only varied in elevation from about 1,237 to 1,240, the river surface elevation. In the left abutment, the water table ranged in depth from about 73 to 142 feet at elevations between 1,304 to 1,360 with an average of about 1,318. The water table was found in both overburden and bedrock, but for the most part, it was in or near the overburden-bedrock contact. In the right abutment, the water table generally occurred in overburden between 15 to 75 feet below the ground surface, and the elevations varied from 1,237 to 1,288 with an average of about 1,240.

A number of artesian springs and wells were found in the Missouri River Valley upstream and downstream from the Fort Randall damsite. One of these springs was located at about Sta. 80+00 near the axis of the embankment. The spring water was tested and found to originate from the Dakota sandstone at about Elev. 750. Apparently, the river had eroded enough of the overlying sediments to allow the artesian pressured Dakota water to force its way to

the surface through fracture zones. The presence of this and similar springs in the reservoir was not considered a critical problem.

2.4   **Overburden.** The overburden at the damsite consists predominantly of fluvial sands in the river valley, clays and silts in the river terrace of the right abutment, loess overlying the river terrace, and glacial till in the upper left abutment. A geologic profile along the axis of the embankment is shown on Plate A-6.

2.4.1   **Fluvial Valley Sands.** The upper portion of the valley fill consists of alluvial sand which ranges from fine to medium in grain size. The fine-to-medium sizes predominate throughout this fill, but with increasing depth the sand becomes more coarse and in part, gravelly. The thickness of the alluvium varied across the river valley from a relatively shallow depth adjacent to the abutments to a maximum thickness of 175 feet in the vicinity of the midstream island. Some thin layers of gravel occurred in this material, but they appeared to be restricted lenses with no great lateral extent. There were also occasional small seams of clay, silty sand, lime, and lignite float. The lower deposits are probably of glacial origin.

Gradation curves for the sand are shown on Plate A-24. Well pumping tests indicated a permeability from 0.001 to 0.004 ft/sec in the upper medium-grained sands; however, the permeability for the deep sands containing more gravel was found to be about 0.001 ft/sec (86.4 ft./day). This suggested that the gravelly sand contained fines which reduced its permeability. The coefficient of permeability for remolded samples of the sand ranged between 3 and 10 feet per day. The coefficients of internal friction for remolded samples of the sand varied between 0.6 and 0.7.

2.4.2   **River Terrace Clays.** An extensive river terrace formed the right abutment of the dam. It consisted of poorly compacted alluvial clays and silts more than 100 feet in thickness. The material was classified as a lean to fat clay with approximately 77 to 98 percent finer than the No. 200 sieve size as shown by the gradation curves on Plate A-26. The in-place

6

dry density of the clay averaged about 90 pounds per cubic foot (pcf) and the moisture content varied between 20 and 35 percent. The consistency of the material ranged from friable to stiff above the ground water table, but varied from stiff to soft at greater depths. The Atterberg Limits for the material varied widely, with liquid limits varying from 32 to 90 percent and plasticity indexes ranging from 10 to 58 percent. Direct shear and triaxial compression test results indicated a design shear strength of tan $\emptyset$ = 0.22 and cohesion = 0.4 tons per square foot. Sample data sheets and summaries of the shear tests are presented on Plates A-28 through A-32.

2.4.3 **Loess.** A blanket of loess covered the right abutment terrace clay deposit. It varied in thickness from about 50 feet at the river bluff to just a few feet in the upper gently sloping terrace area. The upper several feet of weathered zone did not have the low moisture content which is normally characteristic of loess, and it was somewhat denser than the unweathered material. In its natural state the loess has low density, ranging between 75 and 85 pounds per cubic foot, with up to 50 percent voids. It has a liquid limit of about 33 percent and a plasticity index of about 10. Its natural moisture content is about 10 to 15 percent. A shear strength of tan $\emptyset$ = 0.65 and cohesion = 0 was determined by direct shear tests and was selected for design. The material has a uniform grain size which is restricted almost entirely to silt and fine sand sizes as shown in the gradation curves on Plate A-25. The silt and sand grains are characteristically bonded by interstitial clay particles which give dry loess the strength to stand in vertical bluffs.

2.4.4 **Glacial Till.** Glacial till formed the overburden in the left abutment with a maximum thickness of 90 feet in the embankment foundation area. It consisted of a heterogeneous mixture of clay, silt, sand, gravel, and boulders with some fragmented shale and chalk. The range in gradation is shown on Plate A-27. The material is predominately a sandy clay, with an average liquid limit of about 40 to 45 and average plasticity index of about 20 to 25. Tests on 18 undisturbed samples indicated that the in-place dry density ranged from about 90 to 114 pounds per cubic foot (pcf)

7

and averaged about 99 pcf. Tests on a few of the larger samples, however, showed an average density of about 102 pcf. Moisture content averaged about 20 percent. Consolidation tests on the undisturbed glacial drift material, in which the specimen was loaded to its natural overburden load and then immersed, did not show any tendency of the material to slump or rapidly consolidate.

2.5 **Bedrock.** The rock formations at Fort Randall Dam are well indurated, compact sedimentary marine deposits of the upper Cretaceous Period. The stratigraphic sequence in ascending order is Greenhorn limestone, Carlile shale, Niobrara chalk, and Pierre shale. Plate A-6 shows a geologic column and geologic profile along the dam axis. Carlile shale and the Niobrara Formation constitute most of the bedrock foundation at the dam site. Formations at the dam site are flat-lying with a gentle dip of a few feet per mile to the northwest. A bedrock contour map is presented on Plate A-7. Bedding planes vary from being inches apart to as much as 5 feet apart in some portions of the Niobrara Formation. They often contain thin layers of gypsum, calcite, or clay. Nearly vertical, randomly oriented joints are common in rock exposures, but less common at depth. There is occasional faulting throughout the area, but usually with no more than a few feet of displacement and relatively short lateral extent. The faulting at the damsite was comparatively minor, but not uncomplicated. Both jointing and faulting in the chalky Niobrara Formation were relatively tight and prohibitive to solution; no caves, caverns, or solution channels were found in the area.

2.5.1 **Greenhorn Limestone.** This formation is generally recognized as a hard fossiliferous limestone. The predominant constituent is crystalline calcium carbonate, but it also has a considerable amount of clay and fine sand. Drill holes in the right abutment penetrated about 45 feet of Greenhorn limestone below Elev. 910.

2.5.2 **Carlile Shale.** The Carlile shale is mostly sandy to clayey shale with some interbedded sandstone. It is about 265 feet thick,

but the upper 100+ feet beneath the valley had eroded and filled with alluvium. It is in the river valley that the Carlile shale forms the flooring bedrock and creates the most concern as a foundation material. There is a relatively weak contact plane between the eroded Carlile shale and the overlying valley sands. Consolidated direct shear tests on samples of the shale indicated a design shear strength of tan ϕ = 0.3 and cohesion = 0.1 ton per square foot. The top of the formation under the left abutment and the right terrace is at about Elev. 1,175, approximately 65 feet below the river level.

2.5.3 **Niobrara Formation.** The Niobrara Formation is the upper-most formation of the Colorado Group, and is the oldest exposed bedrock in the area. It is a dark gray, argillaceous, soft but firm chalk and chalky shale which contains many microscopic shells of Foraminifera and Ostracoda. The color changes to a buff or light gray when the formation is weathered. The chalk is a porous rock, but the voids are so poorly interconnected that the rock is relatively impervious. It is a massive, coherent rock, and although it is soft enough to be scratched with a fingernail, it is also tough and resilient enough to resist fracturing. Fresh exposures of the chalk withstand repeated cycles of wetting and drying without appreciable deterioration. Thin layers of bentonitic clay with thicknesses up to 2 inches occur throughout the formation, but they are more concentrated in the upper 20 feet. Tests have revealed that after being thoroughly dried these clays, unlike pure bentonite, will not swell greatly when saturated.

The Niobrara Formation is the predominant sedimentary material at the damsite. It extends for about 145 feet in thickness from Elev. 1,175 to Elev. 1,320; this is about 65 feet below the river level to an average height of 80 feet above the river level. It is the most stable bedrock in the Fort Randall area and is the foundation for the outlet works, the spillway, and the powerplant. It was also the major rock encountered in construction excavations. Dry unit weight of the chalk varied from 85 to 114 pcf, and moisture content ranged from 20 to 30 percent.

9

2.5.4   Pierre Shale. The Pierre shale is the uppermost formation at the damsite, and its contact with the underlying Niobrara Formation occurs sharply at about Elev. 1,320. The formation is susceptible to landsliding, and it may be generally described as a noncalcareous to highly calcareous, gray, green, brown, or black, tough, gummy, marine shale with zones of bentonite seams and iron-manganese concretions. It is commonly divided into eight members; however, erosion has removed all but the lower member and therefore, the shale had only a minor influence on the dam construction. Excavation for the spillway encountered about 30 feet of Pierre shale, but no part of the structure was founded on the formation. A thin remnant of the shale, however, formed the bedrock beneath a small portion of the dam where the embankment is relatively low.

3.   EMBANKMENT SECTION. A typical section of the embankment is shown on Plate A-2 and sections at various locations along the embankment are shown on Plates A-4 and A-5. The embankment has a maximum height of about 165 feet above the river bed and has an average height of about 140 feet over the flood plain. The crest of the embankment is 60 feet wide and is at Elev. 1,395 feet, mean sea level. The embankment section consists mainly of a central wide-based compacted impervious earth fill section and dumped chalk fill outer berm sections. An upstream impervious fill blanket adjacent to the central impervious section reduces uplift pressures beneath the embankment by lengthening the seepage path. In addition to the stability provided by the downstream chalk berm, the relief wells along the toe of the central impervious section provide assurance against the development of excessive underseepage pressures. The flat 1V on 15H surface of the upstream chalk berm was considered an acceptable alternative to the more expensive conventional riprap protection. Except for the areas inundated by the reservoir and the riprapped slope in the vicinity of the outlet works intake structure, the entire embankment is grassed for protection against surface erosion.

4.   CONSTRUCTION STAGES. The embankment was constructed in six earthwork stages under separate contracts. The river was diverted through the power

and outlet works structures and tunnels during Earthwork Stage III. Diversion of the river, therefore, controlled the extent and sequence of construction for the various earthwork stages. Work covered under each stage is summarized below.

4.1   **Foundation Preparation - Left Bank Chute. (September, 1947 - January, 1948).** This preparatory work was required to provide a dry foundation for the embankment from the left bank of the river to the toe of the left abutment. It was completed during the fall and early winter months in preparation for the initial embankment earthwork stage that was scheduled to start the following spring. Approximately 390,000 cubic yards of compacted fill up to a maximum thickness of about 10 feet, to about Elev. 1,252 were placed under this contract.

4.2   **Initial Earthwork (February 1948 - December 1948).** Work under this stage included partial embankment construction from the river to the left abutment. It also included excavation in the upstream and downstream portal areas of the power and outlet works tunnels and partial excavation in the spillway approach channel. Two test embankments, one of shale and the other of excavated chalk material, were constructed during this phase and are discussed in subparagraph 6.5, Chalk Berm Fill. Approximately 6,725,000 cubic yards of compacted earth fill were placed to elevations varying from Elev. 1,271.5 over the left bank to Elev. 1,365 over the left abutment. The fill material was predominately glacial drift overburden excavated from the outlet works and spillway areas. Approximately 4,510,000 cubic yards of chalk and shale were also excavated from these areas and were used primarily in the construction of the upstream and downstream embankment chalk berms.

4.3   **Earthwork Stage II (February 1949 - October 1949).** Work under this contract was entirely on the left side of the river and was essentially a continuation of the activities performed under the previous earthwork contract. Approximately 4,380,000 cubic yards of compacted earthfill and 147,000 cubic yards of pervious channel fill were placed. The compacted

11

embankment over the left bank varied up to Elev. 1,315 under this construction phase. Approximately 4,000,000 cubic yards of chalk and shale were excavated from the outlet works and spillway areas and placed in the upstream and downstream embankment berms.

4.4   Earthwork Stage III (June 1950 - March 1953).   Stage III work included construction of the left bank embankment to its full height, placement of riprap on the upstream slope of the embankment in the vicinity of the intake structure, construction of an upstream cutoff trench along the right bank of the river, removal of right bank loess material beneath the embankment and backfilling with compacted impervious fill, diversion of the Missouri River through the outlet works, construction of the embankment in the closure section and over the right bank area to a maximum elevation of 1,325, and placement of the upstream impervious blanket in the closure section and over the right bank area. In addition, the approach and discharge channels of the outlet works were completed and a portion of the spillway channel was excavated. The outlet works structures, including the tunnels, intake structures, downstream outlet structure and powerhouse substructure *were under a separate* contract and were sufficiently complete to allow diversion of the river. Approximately 7,350,000 cubic yards of compacted fill, 2,060,000 cubic yards of hydraulic pervious fill in the closure foundation area, and 194,000 cubic yards of hydraulic chalk fill in the diversion weir structure were placed during this construction stage. In addition, approximately 3,800,000 cubic yards of chalk and shale were excavated from the outlet works and spillway areas and were placed in the upstream and downstream chalk berms. Photos No. 5 through No. 18 were taken during Earthwork Stage III.

4.5   Earthwork Stage IV (November 1952 - September 1953).   The compacted impervious and upstream chalk berm sections of the embankment were placed to Elev. 1,355 in the right bank and closure areas under the Stage IV construction. Also included was the 5-foot thick pervious drainage fill blanket beneath the right bank downstream compacted section of the impervious embankment. Approximately 2,740,000 cubic yards of impervious fill and

12

220,000 cubic yards of pervious fill were placed in the compacted earthfill portions of the embankment. About 3,590,000 cubic yards of chalk and shale were excavated from the spillway area and were placed mostly in the upstream chalk berm section of the embankment. Photos Nos. 19 and 20 were taken during Earthwork Stage IV.

4.6   Earthwork Stage V (July 1953 - November 1955). Earthwork Stage V was the final embankment construction stage. The embankment over the right bank and closure section was raised to its design crest elevation of 1,395. The left bank portion of the embankment had previously been constructed to Elev. 1,395 under Earthwork Stage III. The entire length of the embankment crest was overbuilt to heights up to 3.5 feet, as indicated on Plate A19, to compensate for post construction settlement. The embankment at both abutments of the spillway structure was constructed to the final crest elevation of 1,395. The left and right bank embankment toe drains were installed and 14 of the 36 previously installed relief wells were raised in conjunction with raising of the chalk berm in the downstream closure area. Excavation of the spillway discharge channel was also completed under this earthwork contract. Approximately 4,330,000 cubic yards of impervious fill and 27,000 cubic yards of pervious drain fill were placed in the compacted embankment. In addition, about 4,560,000 cubic yards of chalk and shale were excavated from the spillway area and placed in the upstream and downstream berms of the embankment. Photos No. 21 through No. 25 were taken during Earthwork Stage V.

5.   FOUNDATION PREPARATION. All areas upon which embankment material were placed, plus at least a 10-foot contiguous strip, were cleared of all brush, trees, structures, trash, debris, and other unsuitable foundation material. Roots larger than 1-1/2 inches in diameter were removed to a minimum depth of 3 feet below the ground surface. Thin surface layers containing sod, humus, and other undesirable material were stripped and wasted. Prior to placement of embankment, the foundation was loosened to a depth of 12 inches by scarifying, plowing, or harrowing, cleared of loosened roots and debris, then compacted as for impervious fill. Foundation areas and conditions requiring

13

specific treatment, such as low left bank areas, left bank springs, right bank loess deposit, and the river closure area are discussed below.

5.1 **Left Bank Preparation.** Low ponded areas on the left bank were filled with pervious alluvial sands to assure a dry foundation for subsequent embankment fill operations. Placement of the sand fill was done by end-dumping and dozing the material in place, an operation that pushed water and muck away from the foundation area. When internal drainage of the fill material was impeded, the filling operation was temporarily stopped to allow dissipation of the pore pressures.

During the initial earthwork stage, the natural spring near the center-line of the dam about 400 feet from the left bank of the river was excavated of muck and provided with a drainage channel to the river. The spring area was about 145 feet in diameter. The excavation extended 5 to 8 feet in depth into sound foundation sand and was backfilled with relatively clean pervious sand and gravel.

5.2 **Removal of Right Bank Loess.** The surface loess material was removed from a major portion of the right bank embankment foundation. A loess excavation plan and sections are shown on Plate A-20 and additional profiles are on Plate A-21. The loess was removed to increase stability, reduce settlement, and minimize possible cracking of the embankment from uneven foundation settlement. The excavated material was then reused to fill the excavation as compacted impervious fill. The entire right bank excavation involved the removal of approximately 2,450,000 cubic yards of predominantly loess material.

5.3 **River Closure Area.** Immediately after diversion of the river, the embankment foundation within the river channel was prepared by filling with pervious sand. Prior to placement of the material, all chalk that was used for river bank protection within the embankment area was removed with a dragline and later placed in the upstream chalk berm. The foundation for the

14

upstream end of the impervious blanket was placed by hauling pervious material which was stockpiled from material excavated from the outlet works approach channel. The material was dozed into the river channel to at least Elev. 1,242, above the water level. Photo No. 11 shows placement of the sand. The pervious foundation fill in the major portion of the channel, between the upstream dumped pervious foundation and the downstream diversion dike, was placed hydraulically by dredging sand from the downstream river bed. This operation is pictured in Photo No. 14. Filling started at the usptream end and progressed downstream. The closure plan and sections are shown on Plate A-23. Approximately 1,630,000 cubic yards of pervious foundation material were placed in the river channel.

6. **MATERIALS AND MATERIALS PLACEMENT.** The embankment was constructed of material obtained primarily from required excavations from the outlet works and spillway areas. Construction operations were generally in two 10-hour shifts. Impervious glacial till material and chalk were the predominant types of material used to construct the compacted embankment and berms, respectively. Pervious sand fill was used primarily as river chute fill and as pervious drainage blanket beneath the downstream section of the embankment. Data on field compaction tests and on embankment construction were obtained from construction reports that were prepared by the Area Engineer and his staff during and immediately after construction of the various earthwork stages. Information was also extracted from laboratory test records on approximately 70 undisturbed box samples that were taken during construction of the compacted impervious embankment. Tests on the record samples included classification, moisture contents, specific gravity, and density. In addition, some of the samples were tested to determine shear strength and consolidation characteristics. Properties and placement of the embankment material are discussed below. Material types include pervious river chute fill, rolled embankment fill, impervious blanket fill, pervious drain fill, and chalk berm fill.

15

6.1 **Pervious River Chute Fill.** River chute filling is described in paragraph 5, Foundation Preparation. Material placed in the closure chute consisted primarily of dredged pervious alluvial sand from the riverbed located downstream of the embankment area. The hydraulically placed material was assumed to be in a similar state of compaction as the existing natural alluvial foundation. Photos No. 11 and No. 14 show placement of the river chute fill.

6.2 **Rolled Embankment Fill.** The central wide-based main embankment section was constructed of predominately impervious material and was designated as the "rolled embankment" section through Earthwork Stage III and is shown as such on Plates A-2, A-4, and A-5. In Earthwork Stages IV and V, this section was designated as "impervious fill" section. The material requirements for the section, however, were the same in all stages. Except for a relatively small quantity of excavated and recompacted right abutment terrace clay and loess, the rolled embankment section was constructed primarily of glacial till material excavated from the left abutment. Approximately 90 percent of the rolled embankment section was constructed of till material which consisted predominately of impervious sandy clays. Small quantities of relatively pervious material that were present in the excavated till were placed in the downstream one third of the rolled embankment section.

Through Earthwork Stage III, the rolled embankment material was placed, as specified, in 8-inch loose lifts and compacted by 6 passes of a sheepsfoot tamping roller. Scarifying, blading, and watering operations are pictured in Photos Nos. 2, 3, and 4 and compaction by sheepsfoot is shown by Photos Nos. 6, 12, and 13. During Earthwork Stage III, the contractor was allowed by contract modification to place the material in 12-inch lifts with compaction provided by at least 3 passes of a 50-ton pneumatic roller. Both compaction methods were allowed in Earthwork Stages IV and V. Except in very few instances, adequate compaction was obtained by the specified minimum number of compaction coverages. Both types of rollers resulted in comparable densities; however, the pneumatic roller was generally preferred by the

16

Contractor because of the lower number of required coverages. Moisture content was maintained near optimum and was controlled by visual inspection by experienced Corps of Engineers inspectors who had immediate access to the results of numerous ongoing field control tests, such as classification, moisture content, compaction, and density. Moisture contents were not specified in the earlier construction stages; however, in Earthwork Stages IV and V, they were specified to be not less than 2 percent below optimum and not more than that required for proper excavating, hauling, placing, and compacting without causing excessive deformation.

Numerous field tests were made during construction of the rolled embankment. For example, during Earthwork Stage II, an average of 20 field density tests were made during each 10-hour shift. In addition, an average of one sample was taken for each 12,000 cubic yards of compacted fill for determining classifications, moisture contents, densities, and air voids. Only a partial record of field control tests exists and, therefore, the total number of field control tests conducted for the project is not known. Available data on at least 1,100 tests made during Earthwork Stages III and V show that the average dry density of the rolled embankment material was about 73°.5 pounds per cubic foot (pcf) and the average moisture content was about 18.5 percent. The material was predominantly sandy clay (CL) and compaction averaged approximately 100 percent of maximum standard density. The tests conducted during the initial Earthwork and Earthwork Stage II construction showed an average dry density of 101.9 pcf and an average moisture content of 17.7 percent.

Fifty-seven undisturbed box samples were taken of the compacted fill during the Initial, Stage II, and Stage III earthwork contracts. This averaged about one sample for each 250,000 cubic yards of compacted fill. For Stages IV and V, a total of 13 box samples were taken for an average of approximately one sample for each 550,000 cubic yards of compacted fill. All samples were shipped to the Missouri River Division Laboratory in Omaha, Nebraska for testing. The test results for the 70 record box samples are tabulated on Plate A-41. For comparison with the results of the field

17

control tests given above, the tests on the undisturbed samples revealed a slightly lower average dry density of 99.5 pcf and a slightly higher average moisture content of 19.9 percent. Atterberg limits are plotted on Plate A-42 and moisture - density plots are shown on Plate A-43. The direct shear test results, shown on Plate A-44, show that only one of the 44 test envelopes fell entirely below the design strength envelope. Also, the tan ϕ values of all the tests ranged from 0.36 to 0.73, all higher than the 0.35 design value. These results indicate that the adopted design shear strength parameters, tan ϕ = 0.35 and cohesion, C = 0.35 tons per square foot were conservatively selected.

6.3 **Impervious Blanket Fill.** The impervious blanket adjoins the upstream toe of the rolled embankment section. It consists of material similar to that used in the rolled embankment section, except that pervious material was not allowed. Impervious material that was unsuitable for the rolled section, due to excessive moisture or presence of pieces of weathered chalk, however, was allowed in the impervious blanket. The material was placed in 12-inch thick layers and was compacted by 3 complete coverages of a crawler type tractor weighing not less than 10 tons. Field compaction test records on this material are not available; however, the construction reports indicated that the impervious blanket was well compacted.

6.4 **Pervious Drain Fill.** Pervious fill was used in the downstream section of the embankment on both abutments.

6.4.1 **Left Abutment.** A 10-foot thick pervious fill blanket drains a natural basal sand stratum which overlies a small portion of the chalk bedrock in the left abutment. The pervious fill, consisting of pit-run sand and gravel free from overburden soils was placed during the initial earthwork stage at the location shown on Plate A-15. The blanket lies between Elev. 1,320 and Elev. 1,330 and extends from Sta. 105+90 to Sta. 107+90. It outlets at the downstream toe of the embankment into a collector drain which was installed under Earthwork Stage V. The material was placed in 12-inch thick layers and compacted with a crawler-type tractor weighing at least 10 tons.

18

6.4.2 Right Abutment. A 5-foot thick pervious blanket was constructed beneath the downstream embankment section. It was constructed during Earthwork Stage IV and extends from Range 5,050 (50 feet downstream of the dam axis) to the downstream toe of the embankment and from Sta. 29+00 to Sta. 57+15. The material consisted of clean, free-draining sand, and sand and gravel containing not more than 10 percent of particles finer than a standard No. 200 sieve. It was obtained from onsite stockpiled excavated sand. The specifications required a lift thickness not exceeding 12 inches for compaction by 6 passes of a tamping roller and not exceeding 18 inches for compaction by 3 passes of a rubber-tired roller. If adequate compaction was not obtained by these methods, the contractor was required to make 3 complete coverages with a 10-ton crawler type tractor. Daily summary records of field compaction tests indicated that final compaction was made with a crawler tractor. The dry density of the surface layer averaged about 109.5 pcf and that of the underlying layer, at least a foot below the surface, averaged about 112.5 pcf. This reflects the additional compaction received by the underlying layer through surface rolling. Moisture contents generally were between 6 and 10 percent. A toe drain collector pipe was later installed along the downstream end of the pervious blanket during earthwork Stage V.

6.5 Chalk Berm Fill. Chalk and shale excavated from the outlet works and spillway were used to form the massive upstream and downstream chalk berms. Berm construction continued through all of the earthwork construction stages. The excavated chalk was generally blocky and varied in size from about 5 feet to a fine granular particle. The total chalk-shale excavation included relatively small quantities of shale. This was due to shale being encountered immediately below the overburden in the upper left abutment, mostly upstream of the spillway and in the approach channel. Most of the shale that was excavated in the initial earthwork stages was placed in the downstream chalk spoil area that formed part of the right bank area of the outlet works discharge channel. In later earthwork stages, the shale was placed primarily in the upstream chalk berm. The chalk berms, therefore,

were constructed predominately of chalk material, which is more resistant to weathering than shale. The berms were constructed generally in lifts not exceeding 10 foot in thickness, with no moisture control, and compaction only by the hauling and grading equipment. Later exposure of the upper 10 foot of the upstream chalk berm by wave erosion revealed that the material was moderately compact with no visible voids. The upstream chalk berm, with an outer slope of 1V on 15H, was considered during project design to be an adequate wave protection alternative to the more expensive conventional riprap protection. Wave erosion scarps, however, developed and required stone protection, as described in paragraph 7, Wave Protection.

Two test embankments, one of shale and the other of chalk, were constructed under Modification No. 4 of the initial earthwork contract. The shale test embankment was constructed in the upstream blanket area between Sta. 86+25 and Sta. 90+25 and Range 3,925 and Range 4,175. The chalk test embankment was made in the chalk spoil area on the right bank of the outlet works discharge channel. The test embankments were constructed to determine the compaction characteristics of shale and chalk which were then being considered for possible use in the rolled embankment section. This possibility was prompted by an expected decrease in overburden excavation due to a revision in the spillway excavation plan. The test fills showed that compaction of these materials into an impervious mass required a rather long and expensive process. The process involved breaking the large chunks by several passes of a specially built spike-toothed roller, then compacting with a sheepsfoot roller. Although density and unconfined compression tests were performed on samples from the compacted fill, additional tests were desired but not performed to determine the effects of saturation and long term weathering of the materials. For these reasons, the tests were considered inconclusive and the use of shale and chalk was not considered further for use in the rolled embankment section. As it later turned out, ample quantity of overburden was available for the construction of the rolled impervious section. The results of the test embankments were later supplemented by additional tests and were used in the design and construction of Gavins Point Dam.

20

7. **WAVE PROTECTION.** Riprap protection on the upstream slope of the embankment was originally placed only in the vicinity of the outlet works intake structure and spillway. Later placement of stones on the embankment upstream chalk berm was made only after many years of observation and monitoring strongly indicated that the wave erosion scarps would continue to increase in height and eventually cut into and adversely affect the stability of the main embankment.

7.1 **Riprap at Intake Structure and Spillway.** The entire upstream slope of the embankment in the vicinity of the outlet works intake structure is protected with riprap. The riprap extends from Sta. 103+00 to the right spillway wall, at approximately Sta. 121+40. The riprapped slope from Sta. 103+00 to Sta. 106+00 extends beneath the northeast end of the upstream chalk berm. This was done for protection of the rolled embankment as some erosion of the chalk berm was expected to occur in this area. Riprap also protects the left bank of the spillway approach channel. The riprap near the intake structure was placed during Earthwork Stage III and those adjacent to both spillway walls were placed later during Earthwork Stage V. The riprap section near the intake structure and right spillway wall consists of a 3-foot layer of riprap, a 1-foot layer of spalls, and a 2-foot layer of filter blanket as indicated on Plates A5 and A-16. The riprap at the left spillway wall is similar, except that a 1-foot filter blanket is used. The riprapped slopes are in good condition.

7.1.1 **Riprap.** Riprap consists of glacial boulders that were stockpiled from required left abutment excavations. The stones were reasonably well-graded from a 3-foot maximum size to a 5-inch minimum size.

7.1.2 **Spalls.** The spall layer was constructed by a subcontractor who obtained the material from a locally owned gravel pit. The stones were reasonably well-graded between the 6- and 2-inch sizes.

7.1.3 Filter Blanket. The filter blanket underlying the spalls was also constructed by the spalls subcontractor. The processed material was obtained from a locally owned gravel pit and was required to meet the following gradation.

Sieve Size U.S. Std. Sq. Mesh.	Percent by Weight Passing
3-1/2-inch	100
3-inch	95-100
2-inch	90-100
3/4-inch	75-100
3/8-inch	60-100
No. 4	40-80
No. 10	25-60
No. 16	20-50
No. 100	0-15

7.2 Stone Protection on Chalk Berm. The flat 1V on 15H upstream chalk berm was assumed to provide adequate resistance against wave action. However, the berm started to erode when the reservoir was initially raised to Elev. 1,355 in 1957. Erosion continued into the 1V on 7H chalk slope above the flatter berm and progressed toward the main embankment section at an average rate of about 6 feet per year. By 1973, it had advanced to about range 4,850, approximately 150 feet from the axis of the dam. Scarps up to about 10 feet in height formed along practically the entire length of the unprotected embankment slope.

In 1977, the 8,000-foot long erosion scarp was graded to a 1V on 3H slope, but stone protection was placed only along a 400-foot reach adjacent to the existing riprap in the vicinity of the intake structure. The stone protection consisted of dumped field boulders having a maximum size of 250 pounds and a median size of 50 to 100 pounds and placed at a rate of 2.5 tons per lineal foot of scarp. The unprotected graded slope was severely eroded by waves in 1978 which led to the placement of riprap over the entire length of the erosion scarp in 1981. Riprap composed of field boulders was dumped at an average rate of 2.5 tons per lineal foot of scarp length and consisted of stones having a maximum size of 400 pounds and a median size of 70 to 150 pounds. These stones are about 50 percent larger than those used in 1977, and were selected for use after excessive stone displacements were noted in the existing riprap.

8. **DIVERSION AND CLOSURE.** Prior to construction of the embankment closure section, the Missouri River was fully diverted through the outlet works on 20 July 1952 as part of the Earthwork Stage III construction. Photo No. 15 shows an aerial view of the diverted river. Diversion was effected by dredging chalk directly into the river. Location and typical sections of the diversion structure are shown on Plate A-22. As shown in Photo No. 8, the chalk was dredged from the chalk spoil area located downstream of the dam and riverward of the outlet works discharge channel. The diversion plan was developed by the contractor as an alternative to the plan shown on the contract drawings which required the use of lumber mattresses, timber cribs, and an open deck pile trestle.

The diversion dam consisted of a lower, wide-based chalk blanket, a dredged chalk weir section, and an upper dumped chalk dike. Stage construction of the dam is indicated by the sections shown on Plate A-22. The chalk blanket was placed to Elev. 1230 approximately 10 feet below the river level, from 29 April 1952 through 22 May 1952. Construction of the dam was temporarily suspended until 7 July 1952 while the outlet works approach and discharge channel plugs were being removed. The diversion weir section was then constructed from 8 July 1952 through 20 July 1952 to Elev. 1244, about 2 feet above the headwater level, to complete the river diversion. By 29 July 1952, the diversion structure was completed by end dumping chalk directly onto the weir section to Elev. 1265. Photo No. 9 shows placement of dredged chalk for the construction of the diversion weir and Photo No. 10 shows end dumping operations in building the dike over the weir section.

Immediately following the diversion operations and prior to placement of fill in the embankment closure section, a dike was first constructed along the upstream end of the impervious blanket foundation. The dike was constructed by end dumping sand that was previously stockpiled from the outlet works approach channel excavation. This operation is shown in Photo No. 11. The foundation for the upstream impervious blanket and the main embankment was then hydraulically filled with sand dredged from the downstream riverbed. Hydraulic fill operations are shown in Photo No. 14.

23

Following placement of the sand foundation to above water level, the impervious blanket was constructed to Elev. 1,265 and the rolled embankment was constructed to Elev. 1,325. The closure plan and section are shown on Plate A-23. The rolled embankment section over the right abutment and closure area was subsequently raised to Elev. 1,355 under Earthwork Stage IV and brought up to final crest, Elev. 1,395, under Earthwork Stage V, the final embankment construction stage.

9. SEEPAGE CONTROL. The embankment is founded on a deep deposit of alluvial sand in the valley and primarily on natural or recompacted clay over-burden in the abutments. Seepage through and beneath the valley embankment section is controlled primarily by the massive embankment and berm sections and by pressure relief wells along the downstream toe of the compacted embankment. Cutoff walls and trenches were considered for underseepage control across the valley section, but were rejected primarily because of the high cost of these methods. Seepage control on the right abutment included removal and recompaction of surface loess deposits, construction of an upstream cutoff trench, and construction of a downstream pervious drain blanket. Control on the left abutment included treatment of a pervious glacial stratum and blanketing of pervious exposures in the approach channel.

9.1 Valley. Seepage control in the valley is provided by the massive embankment section and by pressure relief wells.

9.1.1 Embankment Section. The massive embankment section, including the central compacted impervious section, the upstream impervious blanket, and the upstream and downstream chalk berms, provides the necessary seepage resistance to keep hydrostatic uplift pressures to below the levels assumed during design. The downstream chalk berm provides stability of the area downstream of the compacted embankment section.

9.1.2 Relief Wells. Thirty-six relief wells were installed along the downstream toe of the impervious embankment section to control uplift pressures that develop in the alluvial foundation, especially the

higher pressures that develop at lower depths. The wells were designed to discharge flows directly through the screens and into the chalk fill berm. The berm was assumed to contain sufficient voids to accommodate the discharges. The location of the wells are shown on Plate A-11. The wells are spaced at 60 to 115 feet, but are mostly at 100-foot intervals. They are fully penetrating, except in the center of the valley where bedrock is at a considerable depth. In this deeper section, the well screens were set at about Elev. 1,150 resulting in a well penetration of approximately 60 percent. Relief wells RW-1 through RW-14 were extended by the Earthwork Stage V contractor in conjunction with raising of the downstream chalk berm in the embankment closure section. The riser and screen sections of the relief wells consist of 8-inch inside diameter wire-wrapped wood stave pipes. Each well includes a 36-inch diameter corrugated metal pipe well pit. A tabulation of the relief wells and relief well piezometers and a soils profile along the line of wells are shown on Plates A-12 and A-13. Relief well and piezometer details are on Plate A-14. Relief well design computations are presented on Plates A-38 and A-39 and the gradation curves of the gravel packs are on Plate A-40.

Twenty-one well point type piezometers were installed between and in line with the relief wells to monitor the uplift pressures at the line of wells. These piezometers are described in "Instrumentation," below. The water level in the wells and the piezometers have always remained below the top of the screen which is at about Elev. 1,270. Distortion of the riser and screen by settlement of the chalk fill prevented flow measurements from being made in 24 of the 36 wells, including all of the wells in the closure area. In wells where the flow meter could be used, only insignificant or no flows were indicated. Inspection of the interior of the pipes with the aid of a mirror revealed the bulging of the pipes and indicated the water surface to be generally still and without ripples. Water levels in the wells and piezometers fluctuate with changes in pool levels, but the average readings have varied little over the years. Detailed description and evaluation of the relief well system are presented in the referenced "Embankment Relief Well Study" report.

9.2   **Right Abutment.** Seepage control measures at the right bank included excavation of the natural loess and backfilling with impervious material, construction of a cutoff trench along the right bank, and construction of a pervious drain blanket beneath the downstream section of the embankment.

9.2.1   **Loess Excavation.** Loess excavation is described in paragraph 5, Foundation Preparation. In addition to preventing excessive settlement of the overlying embankment, removal of the loess and its replacement with compacted impervious material resulted in a stable impervious foundation.

9.2.2   **Cutoff Trench.** A cutoff trench was excavated through the sand and gravel layer that overlies the Niobrara Chalk along the right bank. The location and details of the trench are shown on Plates A-20 and A-21. Dewatering was required in the deeper excavations and the trench was backfilled with compacted impervious material. The loess covered area between the cutoff trench and the main embankment was excavated to a depth of 5 feet and was then backfilled with the recompacted excavated material to form a continuous relatively impervious blanket.

9.2.3   **Pervious Drain Blanket.** The 5-foot thick pervious drain blanket beneath the downstream embankment section on the left abutment is described in paragraph 6.4.2. Its purpose is to increase the stability of the embankment by providing drainage for both the overlying embankment and underlying foundation. The blanket is provided with a toe drain collector pipe along the downstream toe of the embankment. Details of the toe drain are shown on Plates A-18 and A-19. The toe drain pipe outlets into the massive chalk berm through a perforated end section. The toe drain has shown very little or no seepage flows.

9.3   **Left Abutment.** Seepage control at the left bank consisted of seepage cutoffs and drainage of the pervious glacial stratum and also impervious blanketing of the pervious sand exposures along the outlet works approach channel.

26

9.3.1 Pervious Glacial Stratum. Treatment of the pervious stratum on the left abutment is detailed on Plate A-15. The glacial drift and basal sand overburden beneath the upstream section of the embankment was removed to the top of the chalk formation and was replaced with rolled impervious embankment material. A shallow cutoff trench was excavated into the top of the chalk formation. Drainage of the basal sand is through the pervious blanket and is described in paragraph 6.4.1. A 12-inch diameter perforated CMP at the downstream toe of the embankment collects and conducts drain water from the pervious blanket. The perforated toe drain pipe extends a distance of about 1,460 feet, from manhole No. 8 at the right spillway wall to manhole No. 5. From manhole No. 5, the drain outlets through a series of solid CMP, drop inlets, and manholes into the surface drainage system at Sta. 104+95 and Range 5,304. A plan and details of the toe drain are presented on Plate A-17.

9.3.2 Blanketing of Approach Channel. Sand and gravel exposures along the outlet works approach channel were excavated, then covered with a 15-foot thick compacted impervious blanket. The locations of these exposures are indicated on Plate A-3 and a typical blanket section is depicted by section H-II shown on Plate A-5. Seepage analyses during design revealed that exposure of the sand layer in the approach channel would create the shortest underseepage path beneath the embankment. Water entering the sand would seep around the riverside of the outlet works tunnels and into the discharge channel. Relief wells and drains located riverward of the outlet works and discharge channel provide additional underseepage control in this area.

10. EMBANKMENT STABILITY. The stability of the embankment over the right bank terrace clay, the valley alluvium, and the left bank glacial material were analyzed during the project design stage.

10.1 Stability of Embankment Over Right Bank Terrace Clay. A "wedge" method of stability analysis was performed for the embankment founded over

27

the thick clay deposit on the right bank terrace. The analysis is graphically shown on Plate A-35. With impervious layers above and below the saturated clay, drainage would be greatly impeded and the pore pressure due to the fill load would be dissipated very slowly. For the stability analysis, very little consolidation of the clay foundation was assumed to occur during construction. The minimum factor of safety obtained was 2.2 which was believed to be adequate for the method of analysis used. A reanalysis in 1952 during construction of the project indicated a reduced factor of safety of 1.4, primarily due to the use of a higher ground water level and also to minor changes in the embankment section. The ground water level was raised from Elev. 1,245 to Elev. 1,275. The toe of the embankment was changed to a 1V on 3H slope instead of a feathered slope, and the pervious drain blanket did not extend as far upstream as in the original embankment section analyzed.

10.2 **Stability of Embankment Over Valley Alluvium.** The embankment section overlying the valley alluvium was analyzed during the initial design stage by the "wedge" method, as graphically presented on Plate A-36. The section did not include the chalk berms as the extent of the berms were not known at that time. When the analysis was made, test results were not available on the strength of the shale foundation. The required shear strengths computed for safety factors of 1.5 and 2.0, assuming zero cohesion, were tan $\emptyset$ = 0.189 and tan $\emptyset$ = 0.358, respectively. Consolidated direct shear tests later indicated a tan $\emptyset$ = 0.3 and cohesion = 0.1 tons per square foot for the shale. On this basis, a factor of safety of about 1.8 was assumed for the analysis. The safety factor is actually much higher due to the presence of the chalk berms.

The same section was also analyzed using a modification of the Fort Peck Elastic Theory method, the results of which are shown on Plate A-37. In this method, the horizontal shearing forces along the shale-sand contact were computed for a depth of 75 feet below the structure. These were determined assuming that no stress discontinuity occurs at the contact plane or that the

28

shale might act as a rigid boundary. It is believed that actual conditions are somewhere intermediate between these assumptions. Using the rigid boundary assumption, the minimum safety factor against horizontal sliding along the contact at any point was found to be 1.7. On the other hand, the infinite depth assumption gave a minimum point safety factor of 2.5.

10.3 Stability of Embankment Over Left Bank Glacial Till. Details on the stability analysis for the embankment over the left abutment glacial material are not available. However, from the minutes of the 9 October 1947 Board of Consultants meeting, no great concern was expressed on the embankment stability over the left abutment. It was noted that the relatively compact material was stable and that based on extremely conservative assumptions, stability studies indicated a safety factor of 1.5. It was also stated that a safety factor of 1.3 was obtained for the sudden drawdown condition.

11. SETTLEMENT. To compensate for expected settlement, the crest of the embankment was overbuilt during Earthwork Stage V, the last earthwork contract stage. Overbuild profiles and details are shown on Plate A-19. The profile reflects the settlement estimated for the different foundation conditions beneath the right bank terrace, closure, and left bank sections of the embankment. The settlement of the right bank terrace clay was estimated primarily on data from consolidation tests run on the clay samples. It was estimated that about 20 percent of the total settlement would occur during construction. For the closure and left bank sections, the settlement of the alluvial foundation was based on data from settlement gages that were installed during the earlier earthwork construction stages. The data indicated that 60 to 75 percent of the settlement would occur during construction.

Plots of settlement gage readings indicate that all foundation settlement have essentially stabilized and that the post construction settlement very closely followed those estimated. Consolidation of the right bank

29

terrace clay, however, occurred at a faster rate than predicted. By the end of construction, approximately 60 percent of the settlement of the clay foundation had occurred. The alluvial foundation settled at a rate slightly higher than predicted. Approximately 75 to 85 percent of the settlement beneath the closure and left bank section occurred during construction.

12. INSTRUMENTATION. Instrumentation of the Fort Randall embankment consists of embankment piezometers, settlement gage piezometers, relief well piezometers, settlement gages, crest and slope movement markers, tiltmeters (slope indicators) and strong motion accelerographs.

12.1 Piezometers. A general tabulation of all the piezometers for the project, except for the settlement gage piezometers, is shown on Plates A-9 and A-10. Sixty well-point type piezometers and 34 open pipe combination piezometer - settlement gages are used to monitor the hydrostatic uplift pressures beneath the embankment. The piezometers that are located beneath the main rolled impervious embankment section are described below under "embankment" piezometers. The piezometers that are located between and in line with the relief wells are discussed under "relief well" piezometers. The few piezometers that are located in the downstream area of the chalk berm are discussed under "downstream" piezometers. Details of a well-point piezometer are shown on Plate A-14.

12.1.1 Embankment Piezometers. There are 32 well-point type piezometers and 34 open-pipe settlement gage piezometers that extend beneath the rolled embankment section. Twenty-four of the well-point piezometers are located in seven piezometer lines, A, B, C, D, E, F, and L across the embankment, as shown on Plate A-11. The piezometers measure seepage pressures in the downstream pervious drain in both abutments and underseepage pressures in the valley alluvial sands. The abutment piezometers also give an indication of the effectiveness of the upstream cutoff trenches in both abutments. Typical piezometer plots are presented on Plates A-47 through A-52.

30

Eight piezometers, FR 79-1 and FR 79-3 through FR 79-9, are located beneath the downstream slope of the valley embankment section and are used to measure the uplift pressures in the alluvial sand foundation. These piezometers were placed in holes that were drilled to obtain information for a seismic evaluation of the embankment foundation.

The 34 settlement gage open-pipe piezometers are located at the settlement gage locations shown on Plate A-53. All of the settlement gage plates are set on top of the embankment foundation. The settlement gage pipe is used as an open-pipe piezometer by allowing entrance of water through small perforations in the lower 2-foot length of the 2.5-inch diameter pipe. Pervious sand was placed around the perforated pipe to at least 3 feet laterally and to at least 1 foot above the top of the perforations.

12.1.2 **Relief Well Piezometers.** Twenty-one well-point type piezometers are located between and in line with the relief wells to monitor the effectiveness of the relief well system along the downstream toe of the rolled embankment. The piezometer locations are shown on Plate A-11 and profiles of the piezometers and wells are presented on Plates A-12 and A-13. Typical plots of relief well and piezometer readings are shown on Plates A-45 and A-46.

12.1.3 **Downstream Piezometers.** Seven piezometers were installed through the downstream portion of the chalk berm to monitor uplift pressures in the alluvial sand near the toe of the chalk berm.

12.2 **Settlement Gages.** Thirty-nine settlement gages were installed during construction of the embankment. Thirty-four are still operational. The five gages at Range 3,735 were inundated by the reservoir and have been abandoned. Location and elevation data on the active gages are tabulated on Plate A-53. Each settlement gage consists of a 6-foot diameter, 1/2-inch thick steel base plate and a vertical 2-1/2-inch diameter steel pipe which

31

extends from the base plate about 3 feet above the embankment surface. The pipe was extended inside of a 4-inch diameter protective, steel pipe from about 6 feet above the base plate and both pipes are capped above the embankment surface. The base plate is founded on 12 inches of a sand levelling layer about 3 feet below the top of the foundation. The lower 2-foot length of the 2-1/2-inch diameter pipe is perforated and backfilled with sand to allow the pipe to act as an open-pipe piezometer. Settlement gage readings have been taken at regular intervals and typical plots are shown on Plate A-54 for the gages at Sta. 22+50, 30+00, and 40+00 or the right abutment and on Plate A-55 for gages at Sta. 82+00 and 90+00 in the valley section.

12.3 Crest and Slope Movement Markers. The locations of 28 crest and slope movement markers are shown on Plate A-56. Initially, the markers consisted of concrete monuments extending approximately 5 feet into the embankment. Concrete monuments were also set in the abutments for survey reference points. All of the markers were replaced with deeper markers in 1979. The new markers are of two types. One type consists of an 11-foot long, 2-inch diameter pipe set in a 10-inch diameter, 10-foot deep augered hole. It is centered in an 8-inch diameter casing which is set about 3 feet above the bottom of the pipe. The uncased pipe and the lower 1.5 foot of casing is embedded in concrete and the top of the casing is capped with a removable cover. The second type of marker consists of a 1-inch diameter, 10-foot long, steel rod driven to a depth of 9 feet below ground surface through a 4-inch diameter, 3-foot deep cased auger hole. The top of the casing extends about a foot above the ground and is provided with a removable cap. Typical plots of movement marker readings are shown on Plates A-57 and A-58.

12.4 Tiltmeters. Two tiltmeter (slope indicator) wells were installed in 1978 and 1979 through the embankment and underlying clay foundation in the right abutment. Tiltmeter Well T 40/503, located at Sta. 40+00 and Range 5,030, is 303.4 feet deep and extends approximately 93 feet into bedrock.

Tiltmeter Well T 40/517, located at Sta. 40+00 and Range 5,170, is 221.5 feet deep and is embedded approximately 61 feet into bedrock. Each tiltmeter well consists of a 3-inch I.D. grooved tiltmeter casing set with an 18-inch stickup in a sand or grout filled vertically drilled hole. These wells are for the purpose of monitoring subsurface displacement by allowing measurement of the change in well casing tilt with a tiltmeter. The tiltmeter casing and tiltmeter (Digitilt TM) were purchased from the Slope Indicator Company. Typical computer plots of tiltmeter readings for T 40/503 are shown on Plate A-59.

   12.5   **Strong Motion Accelerographs.** Fort Randall Dam is in Zone 1, a low seismic activity region outlined in the seismic probability map, Figure 6, EM 1110-2-1902. Three Kinemetrics SMA-1 strong motion accelerographs were installed at the project in 1976. One instrument is located off the crest of the dam at about Sta. 60+00, at about the maximum section of the embankment. Another is in the downstream area near the old Fort Randall chapel, and the third is located in the west end of the spillway gallery. All of the instruments were installed and are maintained by the U.S. Geological Survey.

13. **OPERATIONS AND INSPECTIONS.** The Fort Randall Dam - Lake Francis Case project is operated and maintained by the U.S. Army Corps of Engineers, Omaha District. The project office is located in the powerhouse complex and is staffed by permanently assigned operations and maintenance personnel. Annual inspections of the project are conducted by personnel of the district office and periodic in-depth inspections are made jointly by members of the Omaha District and the Missouri River Division, and occasionally the Office of the Chief of Engineers. These inspections are made to assure the structural and operational soundness of this multipurpose dam project. Periodic inspections are made in accordance with the requirements of ER 1110-2-100 and to date, such inspections have been successfully conducted in 1967, 1971, 1976, and 1981. Results of the inspections are included in the referenced periodic inspection reports.

14. **EVALUATION.** The Fort Randall Dam embankment is in good structural condition. In over 28 years of operation, no serious stability problems have occurred. Instrumentation readings indicate that settlement of the embankment foundation has essentially stabilized, that no unusual embankment deformations are occurring and that hydrostatic uplift pressures are lower than those assumed during design of the project. Daily surveillance by project personnel and annual and periodic inspections by members of the District and Division offices assure that the performance of the dam is adequately monitored and evaluated.

APPENDIX A
DRAWINGS

LOCATION MAP

MONTANA   NORTH   DAKOTA

SOUTH   DAKOTA

WYOMING

LAKE FRANCIS CASE

NEBRASKA

COLORADO
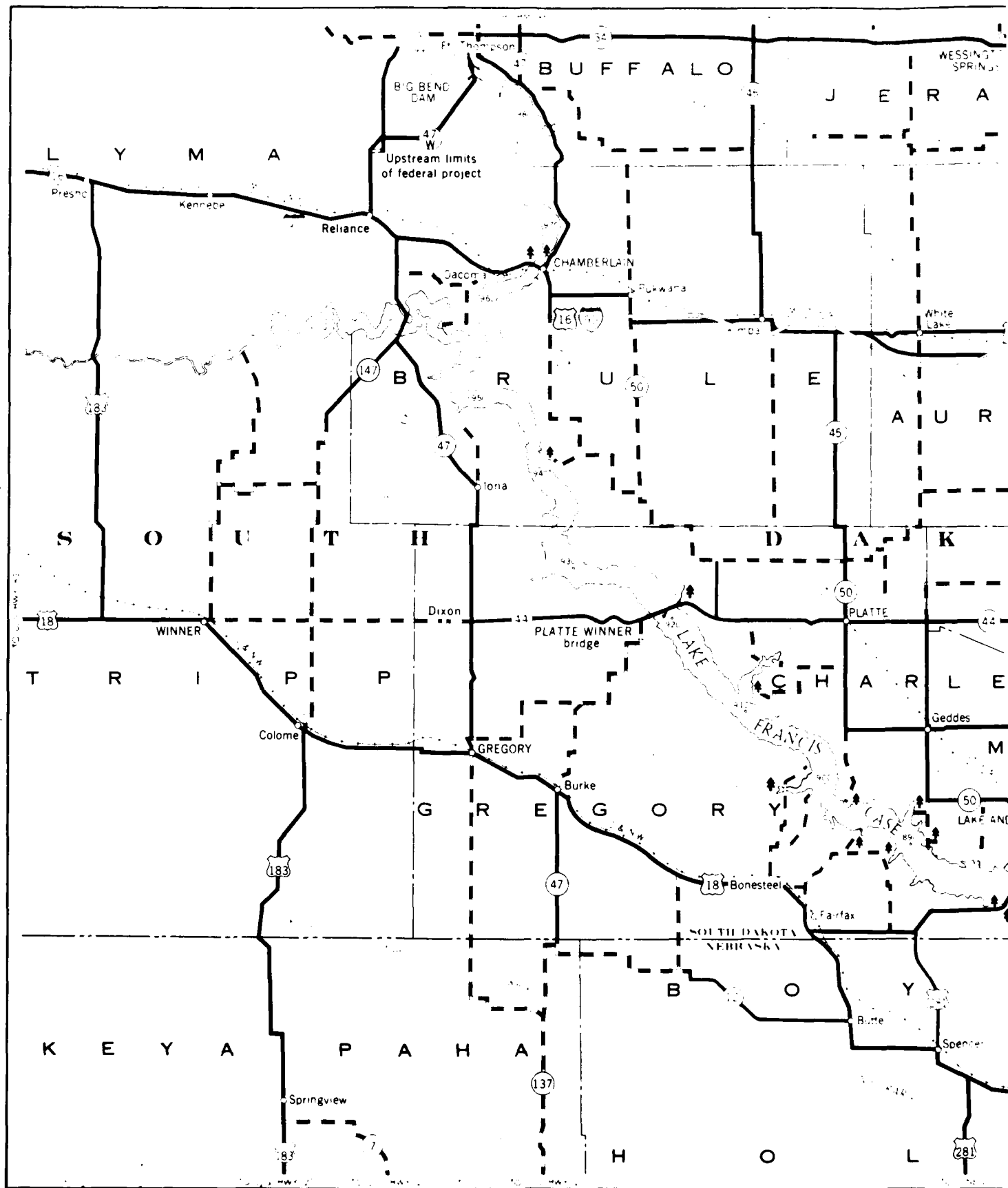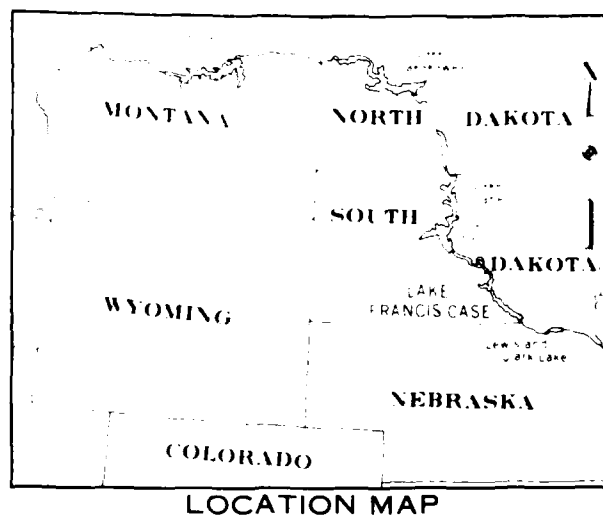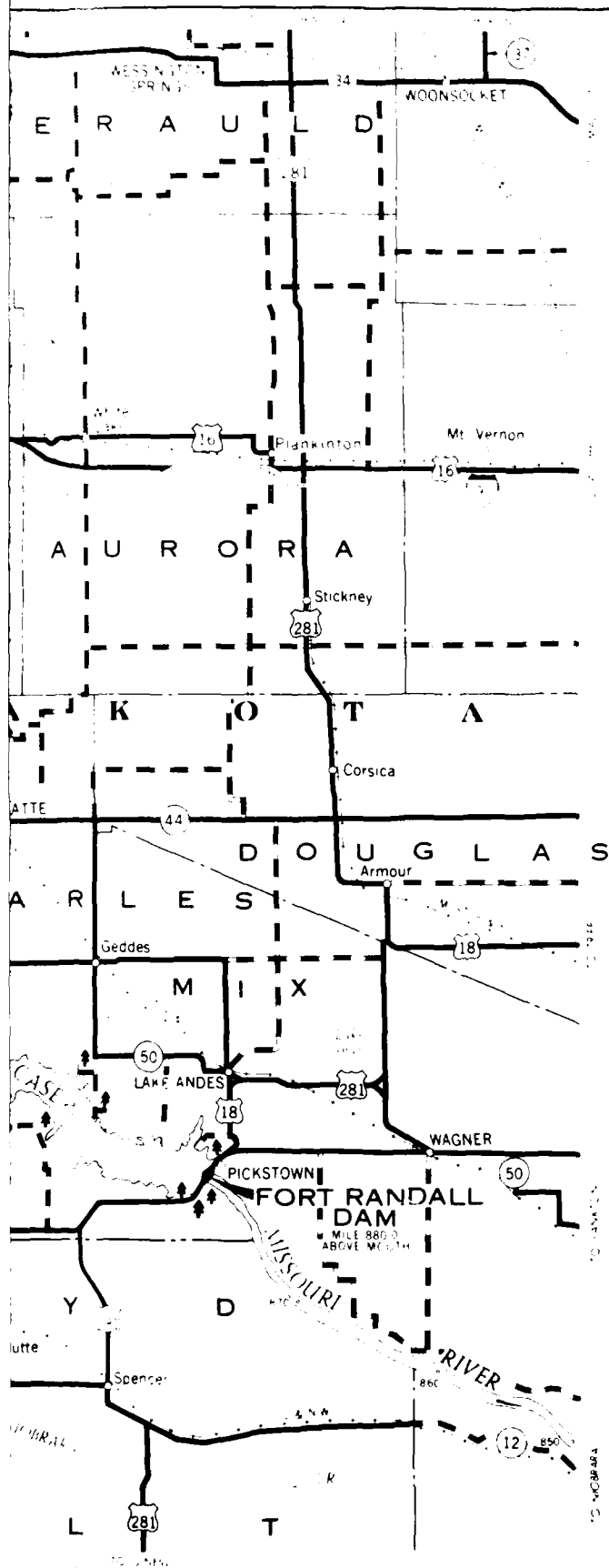
♣ Recreation area

RESERVOIR CAPACITY 5 700 000 ACRE FEET EL 13 5

TOTAL UNITED STATES LAND ACQUIRED TO DATE
115 045 0 ACRES

SCALE IN MILES
5   0   5   10

FLOOD CONTROL PROJECT
FORT RANDALL DAM
LAKE FRANCIS CASE
MISSOURI RIVER BASIN
SOUTH DAKOTA
U S ARMY ENGINEER DISTRICT OMAHA
CORPS OF ENGINEERS
OMAHA NEBRASKA

PLAN

Maximum normal operating pool, El 1365.0

Intake structure

Approach channel

LAKE FRANCIS CASE

Powerhouse

Spillway

Pickstown

Outlet works

Switchyard

Discharge channel

AXIS OF DAM

+ MILE 880.0

MISSOURI RIVER

Site of old Fort Randall

· 1960 Adjusted mileage above mouth

Camping area

U. S Hwys 18 & 281

PLAN

Max. oper. pool, El 1375.0

El. 1360.0

1 on 3

El. 1325.0

1 on 2

El 1346

El 1285

Max. oper. pool, El. 1375.0

M.N.O.P. El. 1365.0

Min mult purpose pool, El. 1310.0

El. 1255.0

1 on 3

El. 1265.0

Slope 1.0%

Impervious blanket

IV ON 7H

El 1330.0

1 on 15

Chalk berm

1 on 7

1 on 2.5
El. 1385.0

El 1355.0

1 on 7

1 on 3.5

El. 1395.0

1 on 3

EL 1370

El 1370.0

EL 1305

Rolled embankment

1 on 3.5
El. 1320.0

El 1303.0

1 on 7

AXIS OF DAM

RELIEF WELL

Slope

Pervious fill

Valley alluvial sand

Bedrock (Carlile formation)

EMBANKMENT SECTION

Gantry crane

El. 1400.0

El. 1380.0

Max. oper. pool, El. 1375.0

M.N.O.P. El. 1365.0

Min. multi-pur. pool El. 1310.0

Approach slab, El. 1227.0

AXIS OF DAM

El. 1395. 0

L Surge tanks

El 1396.0

El 1320.0

El. 1230.0

El 12

Penstock

Intake Structure

Power tunnel

Powerhouse

Tunnel terminal structure

OUTLET WORKS PROFILE

SPILLWAY PROFILE

EL. 1325.0

EL. 1360.0
1 on 2

1 on 3

£ Roadway
EL. 1395.0

1 on 3

EL. 1346.0
EL. 1285.0

EL. 1370.0

1 on 3

EL. 1370.0

1 on 3

1 on 1

EL. 1198.0   EL. 1218.0

EL. 1330.5

EL. 1273.0

EL. 1266.0

Max. design
tailwater,
EL. 1258.0

RELIEF WELLS
Slope 0.78%

EL. 1303.0

Chalk berm

Dredged chalk fill

EL. 1288.0

Penstock

EL. 1230.0

Normal tailwater,
EL. 1237.0

Min. tailwater,
EL. 1220.0

is fill

edrock (Carlile formation)

Draft tube

EL. 1188.0

EL. 1180.0

POWERHOUSE SECTION

e tanks

EL. 1267.0

EL. 1250.0

0.0

EL. 1209.0

EL. 1211.0

Powerhouse
Tunnel terminal structure

Tailrace

LAKE FRANCIS CASE

Normal Operating Pool Level

Impervious Blanket

Cut-off trench

El 300 Upstream Chalk Berm

El 1330

AXIS OF DAM
El 1355

POWERHOUSE

El 1370
El 1320
El 1300

Transition

Fort Randall Cemetery

Fort Randall Chapel Ruins

SWITCHYARD

OUTLET WORKS

Site of Old Fort Randall

Right River Bank (Old)

FLOW
Discharge Channel

Left River Bank

MISSOURI RIVER

CASE

El 1300
Upstream Chalk Berm

AXIS OF DAM
El 1355

POWERHOUSE

APPROACH CHANNEL

FLOW

Transition

Limits of Impervious Blanket

Transition

El 1330

70+00   80+00   90+00   100+00

El 1370   El 1320

El 1300

INTAKE STRUCTURE

FLOW

Approach Channel
El 1205.0

Berm
El 1320

SWITCHYARD

Tailrace

Stilling
Basin

OUTLET WORKS

FLOW

SPILLWAY

Slope 4%

Slope 20%

El 1180.0

FLOW

Discharge Channel

El 1180

MISSOURI RIVER

Left
River Bank (Old)

Pumphouse Access Road

NOTES
1. For location of Sections A, B, C, D and
   E see Plate A4
2. For location of Sections F, G, and H see
   Plate A6

THIS DRAWING HAS BEEN REDUCED TO
THREE-QUARTERS THE ORIGINAL SCALE

SCALE 1 INCH = 400 FEET
400'   0   400'

DATE	DESCRIPTION		MARK	APPD

REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM-LAKE FRANCIS CASE
RESERVOIR AND POWERHOUSE

GENERAL PLAN

SCALE AS SHOWN

THIS PLAN ACCOMPANIES CONTRACT NO.
MODIFICATION NO.

BASE

SECTION A-A: STA. 20+25

SECTION B-B: STA. 40+00

SECTION C-C: STA. 55+00

SECTION D-D: STA. 68+00

SECTION E-E: STA. 86+00

Upstream   Downstream

El 1385 / R 4945
El 1395 / R 4970
El 1385 / R 4920
El 1370 / R 49075
El 1395 / R 5030
El 1370 / R 5105
1 on 7
1 on 25
1 on 3
1 on 3.5
Chalk berm
1 on 3.5
Clay
Loess
Rolled embankment
Bedrock (Niobrara Chalk)

Upstream
El 1330 / R 4335
El 1355 / R 4710
El 1385 / R 4945
El 1385 / R 4920
El 1370 / R 49075
El 1395 / R 4970
El 1395 / R 5030
El 1370 / R 5105
1 on 7
1 on 15
Chalk berm
1 on 7
1 on 25
1 on 3
Rolled embankment
Loess
Clay
Settlement gages
Bedrock (Niobrara Chalk)

Upstream   Downstream
Variable, see Plate A-3
Impervious blanket
El 1355 / R 4710
El 1304 / R 4720
El 1303 / R 4675
El 1330 / R 4335
El 1385 / R 4945
El 1385 / R 4920
El 1395 / R 4970
El 1395 / R 5030
El 1370 / R 5105
1 on 7
1 on 150
El 1265
1 on 15
Chalk berm
1 on 7
1 on 25
1 on 3
Impervious blanket
Loess
Rolled embankment
Variable slope
Pervious fill
Cutoff trench, see Plate A-3 for location
interbedded strata of clays and sands
Bedrock (Niobrara chalk)

Upstream
El 1255 / R 2905
El 1265 / R 3900
El 1330 / R 4335
El 1355 / R 4710
El 1305 / R 4680
El 1385 / R 4945
El 1385 / R 4920
El 1370 / R 49075
El 1395 / R 4970
El 1395 / R 5030
El 1370 / R 5105
1 on 3
Slope = 10% ±
El 1242 ±
Impervious blanket
El 1265
Chalk berm
1 on 7
1 on 15
1 on 7
1 on 25
Rolled embankment
1 on 3
1 on 3.5
El 1260
Pervious fill
Valley alluvial sand
Bedrock (Carlile formation)

Upstream   Downstream
Upstream limit of berm varies, see Plate A-3
El 1255 / R 2905
El 1310 / R 4200
El 1330 / R 4335
El 1355 / R 4710
El 1300 / R 4660
El 1385 / R 4945
El 1385 / R 4920
El 1370 / R 49075
El 1395 / R 4970
El 1395 / R 5030
El 1370 / R 5105
El 1320 / R 5280
El 1265 / R 4400
1 on 3
Impervious blanket
1 on 7
Chalk berm
1 on 15
1 on 7
1 on 3.5
1 on 25
1 on 3
1 on 3.5
1 on 2
Powerhouse road
Rolled embankment
Natural surface blanket of clay & silts
Slope = 0% ±
1 on
Relief well
Valley alluvial sand
Bedrock (Carlile formation)

R 4700   R 5000   R 5300
R 4500   R 5000
R 3000   R 4000   R 4500   R 5500
R 3000   R 4000   R 4500   R 5000   R 5500
R 3000   R 4500   R 5000   R 5500

1400   1300   1200
1400   1300   1200   1100

1

SECTION B-B STA. 40+00

SECTION C-C STA 55+00

SECTION D-D STA 68+00

STA 86+00

NOTES
1. For location of sections, see Plate
2. All elevations shown refer to feet, M.S.L. 1929 Gen Adj

EMBANKMENT CRITERIA AND PERFORMANCE REPORT   PLATE A4

2

Upstream — Downstream

Intake Structure
Riprap
Impervious blanket
Pervious glacial strata
Rolled emb.
Pervious fill
Surge tank berm
SIMILAR TO DETAIL "A"
Power structure
Niobrara chalk
Power Tunnel No.1
5 Deep cut-off trench

El 1395 / R4970
El 1370 / R4900
1 on 2.5
1 on 2.2
El 1395 / R5030
El 1365 / R5225
1 on 3
1 on 3.5

8" impervious film
6" top soil
Exist toe of embank
El 1325 42
El 1325 42
Stone protection
Gravel backfill
Approx top of chalk
3" sec on chalk surface
Glacial drain collector

SECTION F-F: STA.107+15

Upstream — Downstream

Intake Structure
Impervious blanket
Rolled embankment
Pervious glacial strata
Surge tank berm
SEE DETAIL "A"
Niobrara chalk
Outlet Works Tunnel No.10
5 Deep cut-off trench

El 1384.2 / R4959.2
El 1370 / R49075
El 1365 / R4911
El 1320½
1 on 3.5
1 on 2.2
1 on 2.5
El 1395 / R4970
1 on 1
1 on 3
El 1395 / R5030
El 1365 / R5225
1 on 3.5

SECTION G-G: STA.113+45

Impervious blanket
Original ground line
Limits of excavation
Silt
Valley alluvial sand
1 on 3
1 on 3
El 1227
Niobrara chalk

SECTION H-H
APPROACH CHANNEL SLOPE

1

DETAIL "A"

GEOLOGICAL PROFILE ON AXIS OF DAM

SCALE
HORIZ 1" = 400'
VERT 1" = 80'

TOP OF DAM EL. 1395

PIERRE SHALE

NIOBRARA CHALK

CLAY

W.T.

CLAY

LOESS

GRAVEL

NIOBRARA CHALK

WATER SURFACE EL. 1240 ±

CLAY SILT

W.T.

SAND

SAND

CARLILE SHALE

CODELL SANDSTONE MEMBER

CARLILE SHALE

ELEVATION IN FEET ABOVE MSL

1500
1400
1300
1200
1100
1000

10+00 20+00 30+00 40+00 50+00 60+00 70+00 80+00 90+00

GEOLOGICAL COLUMN
SCALE 1" = 40'

AGE	FORMATION	DESCRIPTION

ELEVATION IN FEET ABOVE M.S.L.

RECENT — ALLUVIUM
PLIESTOCENE — LOESS — GLACIAL DRIFT
PIERRE — SHARON SPRINGS
NIOBRARA — SMOKY HILL — FT. HAYS
CARLILE — CODELL — BLUE HILL

CRETACEOUS

AGE FORMATION

1000
900
800
700
600
500

CRETACEOUS — GRANEROS — DAKOTA — FUSON — LAKOTA
GREENHORN — FAIRPORT — BLUE HILL

PRE-CAMBRIAN — SIOUX QUARTZITE

ELEVATION IN FEET ABOVE M.S.L.

GEOLOGICAL PROFILE ON AXIS OF DAM

Labels within profile:
- TOP OF DAM EL 1395
- GLACIAL TILL
- PIERRE SHALE
- POWER AND O.W. TUNNELS
- CLAY SILT
- NIOBRARA CHALK
- WT
- SAND
- SAND
- CARLILE SHALE
- CODELL SANDSTONE MEMBER
- CARLILE SHALE
- EN SURFACE EL 1240 ±

Elevation scale (right): 1500, 1400, 1300, 1200, 1100, 1000 — ELEVATION IN FEET ABOVE M S L

Station scale (bottom): 70+00, 80+00, 90+00, 100+00, 110+00, 120+00, 130+00, 140+00

SCALE
HORIZ 1" = 400'
VERT 1" = 80'

GEOLOGICAL COLUMN (CONTINUED)

AGE   FORMATION   DESCRIPTION

Elevation scale (left): 1000, 900, 800, 700, 600, 500 — ELEVATION IN FEET ABOVE M S L

CRETACEOUS:
- FAIRPORT | BLUE HILL
- GREENHORN
- GRANEROS
- DAKOTA
- FUSON
- LAKOTA

PRE-CAMBRIAN:
- SIOUX QUARTZITE

THIS DRAWING HAS BEEN REDUCED TO
THREE-EIGHTHS THE ORIGINAL SCALE.

FORT RANDALL RESERVOIR
MISSOURI RIVER BASIN
SOUTH DAKOTA

GEOLOGICAL PROFILE ON AXIS OF DAM
AND GEOLOGICAL COLUMN

IN   SHEETS   SCALE AS SHOWN   SHEET NO.
FORT RANDALL AREA, U.S.ENGINEER OFFICE, PICKSTOWN, SO. DAK.

SUBMITTED	RECOMMENDED	APPROVED
GEOL & EXPLORATIONS BRANCH	ENGINEERING DIVISION	AREA ENGINEER

| DRAWN BY CET | TRACED BY CET | CHECKED BY | DATE MAY 4 1948 |

EMBANKMENT CRITERIA AND PERFORMANCE REPORT   **PLATE A6**

MISSOURI

AXIS OF DAM

RIVER

EMB

MISSOUR: RIVER
FORT RANDALL DAM
SOUTH DAKOTA

BEDROCK CONTOURS

U S ENGINEER OFFICE OMAHA, NEBRASKA   JANUARY 1947

R-15G-1A/17

EMBANKMENT CRITERIA AND PERFORMANCE REPORT   PLATE A7

LEGEND
• COMPLETED HOLES

CORPS OF ENGINEERS, U.S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM – LAKE FRANCIS CASE

EMBANKMENT

PLAN OF
SUBSURFACE EXPLORATIONS

SAFETY PAYS

PIEZOMETER TUBE AND OBSERVATION WELL DATA

Group	Piezometer or Well Number	Location Station	Range	Elevation Top of Pipe	Elevation Bottom of Screen	Type	Purpose of Piezometer Tube or Observation Well
1	A5.2	48+00	5200	1350.06	1303.04		To record seepage pressures in the downstream pervious drain.
	A5.5	48+00	5500	1318.38	1300.58		
	B4.95	56+00	4950	1388.62	1242.19		To record seepage pressures in terrace substrata and to determine the effectiveness of the upstream cutoff trench.
	B5.2	56+00	5200	1345.49	1291.49		To record seepage pressures in the downstream pervious drain.
	B5.26	55+98	5259	1329.88	1298.18		
	B5.5	56+00	5500	1303.02	1247.9		To record seepage pressures in terrace substrata and to determine the effectiveness of the upstream cutoff trench.
	C4.95	70+00	4950	1389.33	1224.09	Well Point	To record underseepage pressures in the valley alluvial sands.
	C5.2	70+00	5200	1345.19	1224.29		
	C5.45	70+00	5450	1305.04	1224.64		
	C5.75	70+00	5750	1302.70	1224.60		
	D4.95	81+50	4950	1384.78	1224.68		
	D5.2	81+50	5200	1343.56	1221.56		
	D5.45	81+50	5450	1307.62	1219.32		
	D5.75	81+50	5750	1301.78	1213.0		
	D6.2	81+50	6200	1297.66	1214.96		
	E4.95	95+00	4950	1387.22	1219.52		
	E5.2	95+00	5200	1343.70	1225.70		
	E5.45	95+00	5450	1304.75	1223.55		
	E5.75	95+00	5750	1285.68	1226.88		
	F4.9	109+25	4900	1368.10	1318.10		To record seepage pressures in the pervious strata overlaying the chalk formation.
	F4.96	109+25	4960	1396.29	1329.49		
	F5.04	109+25	5040	1395.66	1308.88		To record seepage pressures in the downstream pervious drain.
	F5.12	109+25	5217	1338.90	1312.90		
	H5.03S	108+90	5030	1396.46	1229.86	Casagrande	To record the hydrostatic pressures that develop in the chalk formation between tunnels 3 & 4.
	I5.03S	112+35	5035	1396.34	1230.14		To record the hydrostatic pressures that develop in the chalk formation between tunnels 8 & 9.
	K5.03S	114+50	5035	1397.44	1229.34		To record the hydrostatic pressures that develop in the chalk formation between tunnels 11 & 12.
	L5.21	120+50	5210	1351.19	1323.79	Well Point	To check the efficiency of the pervious drain adjacent to the right spillway wall at the downstream toe of the embankment.
	C6.829	70+00	6829	1290.34	1218.84		To record the hydrostatic pressure in the valley alluvium at the toe of the chalk berm.
	H6.801	75+31	6801	1269.49	1218.19		
	H6.796	75+31	6796	1290.15	1231.95		To record the hydrostatic pressure at the downstream toe of the chalk berm.
3	PZ-1	61+30	5800	1302.13	1237.73	Well Point	To record the uplift pressures in the valley alluvium along the line of relief wells.
	PZ-2	63+30	5800	1301.24	1228.24		
	PZ-3	66+30	5800	1302.02	1225.02		
	PZ-4	69+30	5600	1301.24	1224.74		
	PZ-4A	69+30	5805	1300.96	1186.96		
	PZ-5	72+30	5800	1301.49	1221.19		
	PZ-6	75+15	5800	1303.18	1225.28		
	PZ-7	77+35	5850	1302.02	1223.52		
	PZ-8	79+35	5850	1301.92	1219.32		
	PZ-9	82+30	5875	1301.84	1224.64		
	PZ-10	84+70	5875	1301.58	1229.58		
	PZ-11	87+60	5875	1300.70	1229.70		
	PZ-12	90+60	5900	1279.95	1234.95		
	PZ-13	94+10	5900	1277.26	1235.26		
	PZ-14	96+03	5900	1274.78	1234.78		
5		Outlet Works Grid System					
	B	258+98	275	1272.85	1135.9	(Temporary Obs. Wells)	To record the artesian pressures in the Codell sandstone.
	C	263+04	215	1270.70	1135.0		
	G	257+79	1350	1324.02	1133.7	Obs. Well	
	B Wall	B Wall Outlet Works		1253.7	1135.7	Obs. Well (Perm. Relief Wells)	
	P-1	Spillway Pier 1		1353.5	1125		
	P-11	Spillway Pier 11		1353.5	1125		
	P-20	Spillway Pier 20		1353.5	1125		
	SP-1	Block 62 Rt. Spillway Wall		1258.88	1150	(Piezometer Wells)	
	SP-2	Block 62 Lt. Spillway Wall		1258.88	1150		

NOTE:

1. The elevation of top of pipe as given above is the most recent elevation available at the time of revision of this table.

2. Piezometer tubes in group 1 through 4 are located by the Embankment Grid System while group 5 observation wells are located by the Outlet Works Grid System.

(Additional Piezometers Since 1976)
Piezometer Tube and Observation Well Data

Group	Piezometer or Well Number	Location		Elevation		Type	Purpose of Piezometer Tube or Observation Well
		Station	Range	Top of Pipe	Bottom of Screen		
2	FR79-1	68+00	5425	1302.35	1206.35	Well Point	To record uplift pressure in the valley alluvium upstream but parallel to the line of embankment relief wells.
	FR79-3	68+00	5030	1396.80	1183.60		
	FR79-4	64+00	5425	1302.85	1182.05		
	FR79-5	74+00	5425	1304.50	1137.15		
	FR79-6	80+00	5425	1307.00	1173.00		
	FR79-7	86+00	5425	1306.50	1132.50		
	FR79-8	92+00	5445	1309.80	1174.10		
	FR79-9	96+00	5445	1302.10	1184.60		
3	PZ-8A	80+00	5850	1302.05	1121.85	Well Point	To record hydrostatic pressures and water gradient in the valley alluvial sand. PZ-8A and PZ-8B serve as checks on PZ-8C and PZ-8D.
	PZ-8B	79+90	5850	1302.25	1181.85		
	PZ-8C	79+80	5850	1302.15	1120.90*		
	PZ-8D	79+8	5850	1302.30	1181.75		
	PZ-10A	85+00	5875	1301.70	1135.35*		
	PZ-10B	85+00	5875	1302.05	1180.65		
4	PZ-15A	80+00	7400	1290.25	1114.70*	Well Point	To record hydrostatic pressures and water gradient in the downstream valley alluvial sand.
	PZ-15B	80+00	7400	1290.20	1164.25		
	PZ-16A	85+00	7400	1278.60	1131.03*		
	PZ-16B	85+00	7400	1278.75	1176.60		
5	PP-1	255+04	3+85.20	1260.15	1230.15	Well Point	To determine the water seepage level in the gravel backfill beneath the tunnel terminal
	PP-2	254+17	3+77.17	1260.50	1245.50		
	PF-3	254+86	3+57.20	1262.40	1247.40		

NOTE: *These piezometers are installed into same drill hole.

Also see notes on previous page.

EMBANKMENT CRITERIA AND PERFORMANCE REPORT   PLATE A10

LEGEND:
▲ CASAGRANDE TYPE PIEZOMETER TUBES
■ WELL POINT TYPE PIEZOMETER TUBES
● VALLEY ALLUVIUM RELIEF WELLS
✕ CODELL SANDSTONE RELIEF WELLS
○ CODELL SANDSTONE OBSERVATION WELLS
□ TILTMETERS

SCALE 1INCH = 400 FEET

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM-LAKE FRANCIS CASE

PLAN OF RELIEF WELLS AND
PIEZOMETER TUBES

	UPDATED FOR O&M MANUAL		MJO	FSB
DATE	DESCRIPTION		MADE	APPRD

THIS PLAN ACCOMPANIES CONTRACT NO.
DACA45
MODIFICATION NO.

RELIEF WELL LEGEND:

RW-00 — RELIEF WELL NUMBER
— TOP OF RISER

SCREEN AND
RISER PIPE
EXTENSION

— TOP OF WELL SCREEN
✕ — OBSTRUCTION

— DEPOSITION

— BOTTOM OF WELL

APPROXIMATE BEDROCK SURFACE

CHALK BERM

PRESENT GROUND SURFACE (1981)

TOP OF WELL SCREENS EL. 1270 (3'±)

75+00

80+00

PRESSURE RELIEF WELL LOCATIONS

RELIEF WELL NO.	STATION	RANGE	BOTTOM ELEVATION	TOP RISER ELEVATION
1	61+80	5800	1202.48	1298.48
2	62+80	5800	1177.70	1298.70
3	63+80	5800	1171.81	1298.81
4	64+80	5800	1169.27	1297.87
5	65+80	5800	1171.91	1298.11
6	66+80	5800	1177.88	1299.18
7	67+80	5800	1167.83	1298.03
8	68+80	5800	1164.26	1298.06
9	69+80	5800	1153.66	1298.60
10	70+80	5800	1161.86	1298.55
11	71+80	5800	1160.79	1298.70
12	72+80	5800	1160.08	1298.08
13	73+80	5800	1157.62	1298.42
14	74+80	5800	1145.86	1298.86
15	75+60	5800	1148.00	1299.80
16	76+20	5850	1151.33	1299.43
17	76+90	5850	1149.34	1299.94
18	77+60	5850	1146.98	1299.48
19	78+30	5830	1149.64	1299.94
20	79+00	5850	1148.44	1299.34
21	79+70	5850	1148.02	1299.32
22	80+40	5850	1148.83	1299.63

PIEZOMETER TUBE LOCATIONS

PIEZ TUBE NO.	STATION	RANGE	BOTTOM ELEVATION	TOP TUBE ELEVATION
1	61+30	5800	1237.64	1302.04
2	63+30	5800	1228.42	1301.12
3	66+30	5800	1225.10	1301.90
4	69+30	5800	1224.30	1301.10
4A	69+30	5805	1187.03	1300.83
5	72+30	5800	1220.84	1301.34
6	75+15	5800	1225.10	1303.00
7	77+35	5850	1223.35	1301.85
8	79+35	5850	1219.16	1301.76
8A	80+00	5850	1121.85	1302.05
8B	79+90	5850	1121.85	1302.25
8C	79+80	5850	1120.90	1302.15
8D	79+80	5850	1181.75	1302.30

DRILL HOLE LEGEND:

GW	Gravel or Sandy Gravel, Well-Graded
GP	Gravel or Sandy Gravel, Poorly-Graded
GM	Silty Gravel or Silty Sandy Gravel
GC	Clayey Gravel or Clayey Sandy Gravel
SW	Sand or Gravelly Sand, Well-Graded
SP	Sand or Gravelly Sand, Poorly-Graded
SM	Silty Sand or Silty Gravelly Sand
SC	Clayey Sand or Clayey Gravelly Sand
ML	Silts, Sandy Silts, Gravelly Silts or Diatomaceous Soils
CL	Lean Clays, Sandy Clays or Gravelly Clays
OL	Organic Silts or Lean Organic Clays
MH	Micaceous Clays or Diatomaceous Soils
CH	Fat Clays
OH	Fat Organic Clays
000	HOLE NUMBER
000	LT. OR RT. OFFSET FROM RANGE OF WELLS

D_{10} 000 000

NOTES:

1. Where laboratory classifications of samples have been made using the "Uniform Soils Classification" it is designated on the logs by such letters as ML, SW, GP, etc., otherwise field indentification of materials by inspectors is used and designated as such by a description of the material.
2. The values on the left side of the log are the D_{10} effective size.
3. "PZ" refers to Piezometer.
4. PZ-8A, 8B, 8C and 8D were installed after 1976.
5. Relief well and piezometer elevations shown are based on 1981 data.

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM - LAKE FRANCIS CASE

RELIEF WELLS &
PIEZOMETER TUBES
SOILS PROFILE SHEET 1

DESIGNED BY
DRAWN BY
TRACED BY
CHECKED BY
SUBMITTED BY
APPROVED
SCALE AS SHOWN
SPEC NO
DRAWING NUMBER
COL. C. E. DISTRICT ENGINEER

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

PLATE A12

PARKING AR-A

SWITCHYARD AREA

APPROXIMATE
ORIGINAL GROUND
SURFACE (1953)

NIOBRARA
CHALK
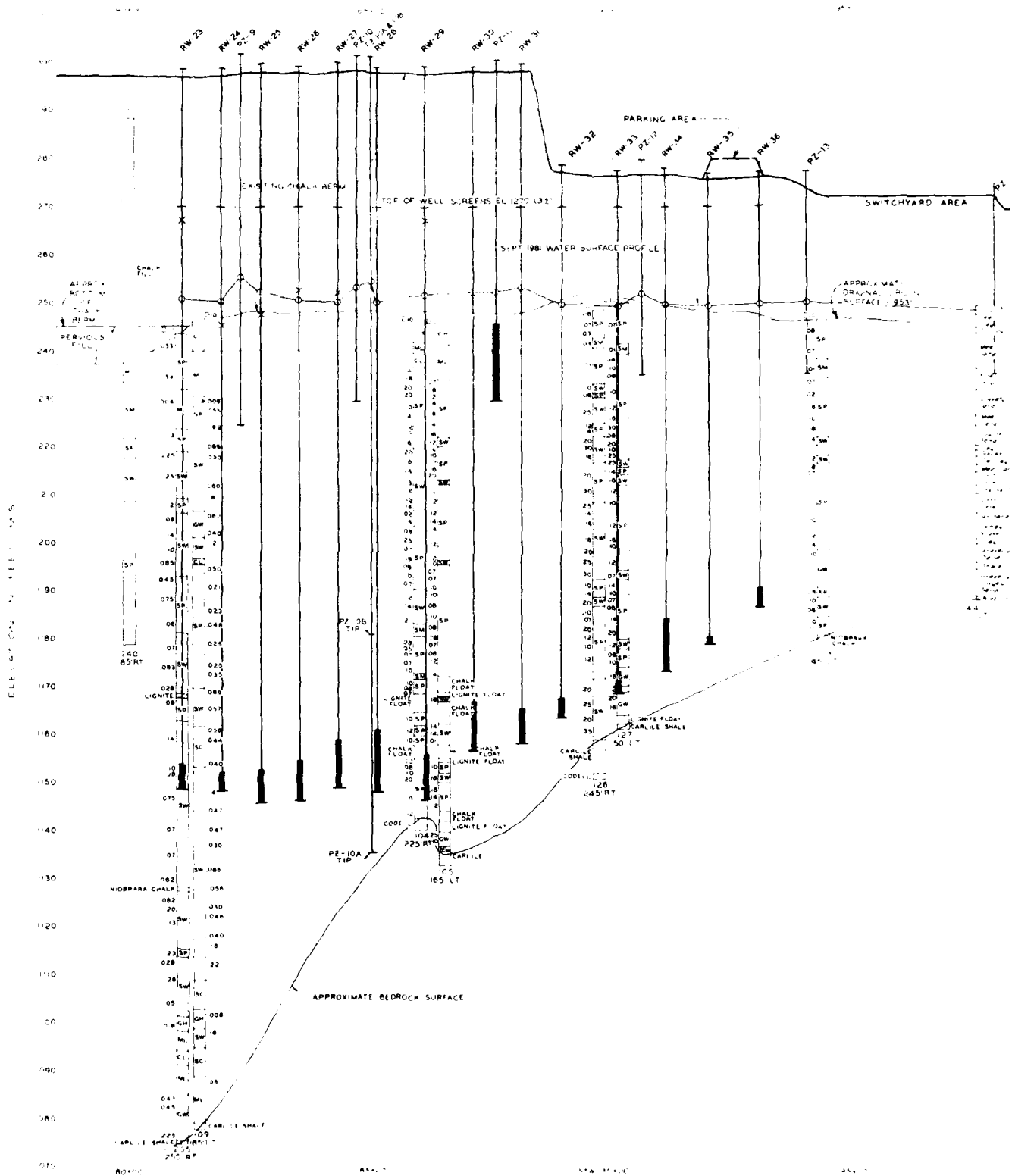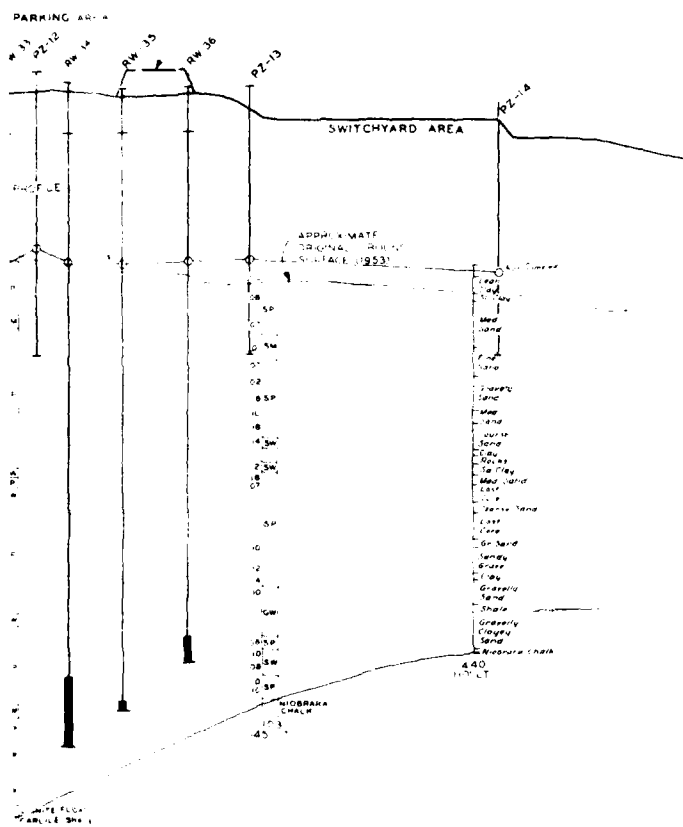
NOTES:

1. PZ-10A and 10B were installed after 1976.
2. PZ-11 has 15.90 feet of deposition as of 1981. Depositions in all other piezometers are negligible
3. For relief well legend refer to Plate A

THIS DRAWING HAS BEEN REDUCED TO
HALF ITS ORIGINAL SCALE

DATE	DESCRIPTION		
	REVISIONS		

CORPS OF ENGINEERS U S ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA NEBRASKA
MISSOURI RIVER
FORT RANDALL DAM - LAKE FRANCIS CASE
RELIEF WELLS &
PIEZOMETER TUBES
SOILS PROFILE SHEET 2

DESIGNED BY
DRAWN BY
TRACED BY
CHECKED BY
SUBMITTED BY

MAR 1972

APPROVED

TYPICAL PRESSURE RELIEF WELL

Labels (left diagram):
- ¼" Steel Plate Cover (See Detail Below)
- 2" Wood Cap
- Surface of Chalk Berm
- 36"
- 10' Section 36" Min. C.M.P. 12 gage
- Backfill around C.M.P. with Compacted Chalk
- Gravel Pack "B"
- 8" I.D. Wood stave pipe riser
- Top of screen - El 1270 (±3')
- 8" I.D. Slotted wood stave screen
- Discharge Flow
- Chalk Berm
- Top of Gravel Pack "A"
- Bottom of chalk berm El. Varies
- Foundation Sub-stratum
- Gravel Pack "A" Min. Thickness 6"
- Pervious Valley Alluvium Sands
- Underseepage Flow
- 2" Wood plug
- Bottom of screen - depth varies

TYPICAL PRESSURE RELIEF WELL
NO SCALE
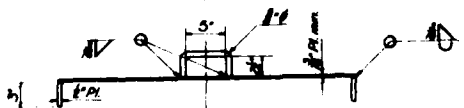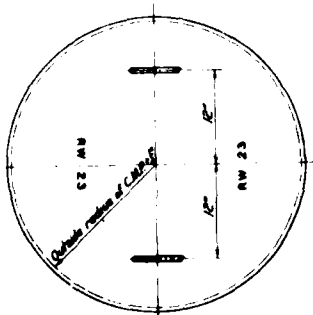
STEEL COVER DETAIL
SCALE 1½ INCHES = 1 FOOT
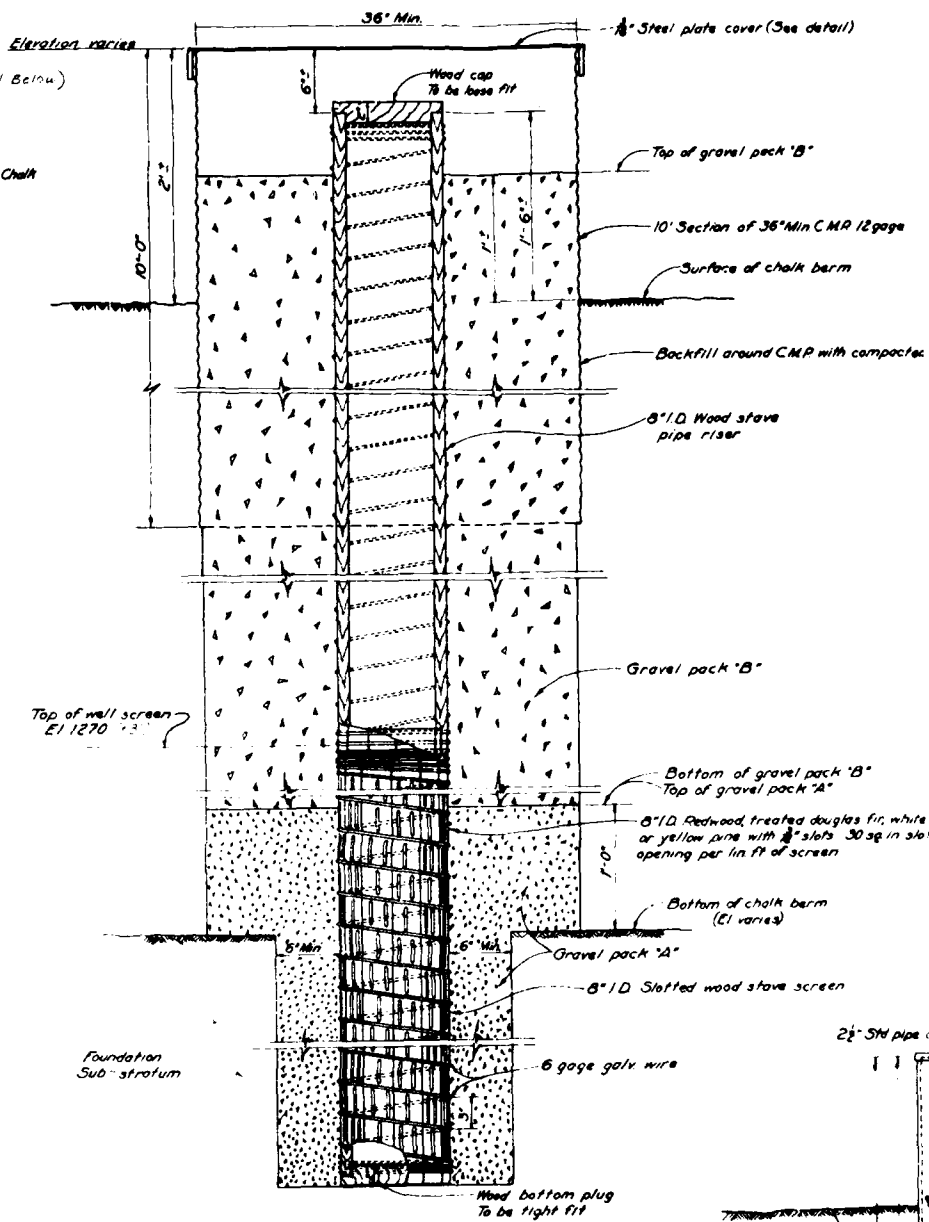
RELIEF WELL DETAILS
NO SCALE

Labels (center diagram):
- 36" Min.
- ¼" Steel plate cover (See detail)
- Wood cap To be base fit
- Top of gravel pack "B"
- 10' Section of 36" Min C.M.P. 12 gage
- Surface of chalk berm
- Backfill around C.M.P. with compacted
- 8" I.D. Wood stave pipe riser
- Gravel pack "B"
- Top of well screen El 1270 (±3')
- Bottom of gravel pack "B" Top of gravel pack "A"
- 8" I.D. Redwood, treated douglas fir, white or yellow pine with ¼" slots 30 sq. in slot opening per lin ft of screen
- Bottom of chalk berm (El varies)
- Gravel pack "A"
- 8" I.D. Slotted wood stave screen
- 6 gage galv. wire
- Foundation Sub-stratum
- Wood bottom plug To be tight fit

PIEZOMETER
NO SCALE

2½" Std pipe
Length varies

5" Min.

Wood cap
To be loose fit

⅜" Steel plate cover (See detail)

Top of gravel pack "B"

1'-6"

10' Section of 36" Min. C.M.P. 12 gage

Surface of chalk berm

Backfill around C.M.P. with compacted chalk

8" I.D. Wood stave
pipe riser

Gravel pack "B"

Bottom of gravel pack "B"
Top of gravel pack "A"

1'-0"

8" I.D Redwood, treated douglas fir, white pine,
or yellow pine with ⅛" slots 30 sq in slot
opening per lin ft of screen

Bottom of chalk berm
(El. varies)

Gravel pack "A"

16" Min.

8" I.D. Slotted wood stave screen

6 gage galv wire

Wood bottom plug
To be tight fit

LL DETAILS

CALE

TYPICAL SECTION
STA. 60+00 TO STA. 76+00

AXIS OF DAM

Chalk berm

EL 1303
R 5400

Relief well extended during Earthwork Stage II
76% Stage

impervious Fill

EL 1270, Top of screen
(3'±)
EL 1260 appres.
R 5777 appres.

Chalk berm added
Earthwork Stage V
Appro 1% slope
Chalk berm El. 1245

Pervious fill

River bed

Relief well initially installed

Chalk berm El. 1230

El varies see profile
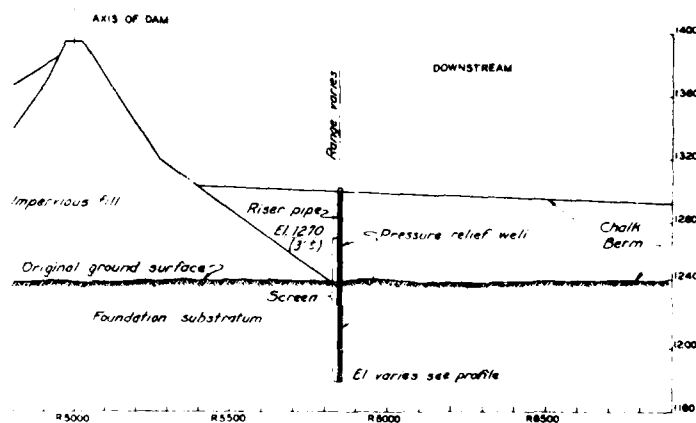
TYPICAL SECTION
STA. 76+00 TO STA. 93+70
SHOWING LOCATION OF PRESSURE RELIEF WELL

AXIS OF DAM

DOWNSTREAM

Range varies

Impervious fill

Riser pipe
El. 1270
(3'±)

Pressure relief well

Chalk
Berm

Original ground surface

Screen

Foundation substratum

El varies see profile
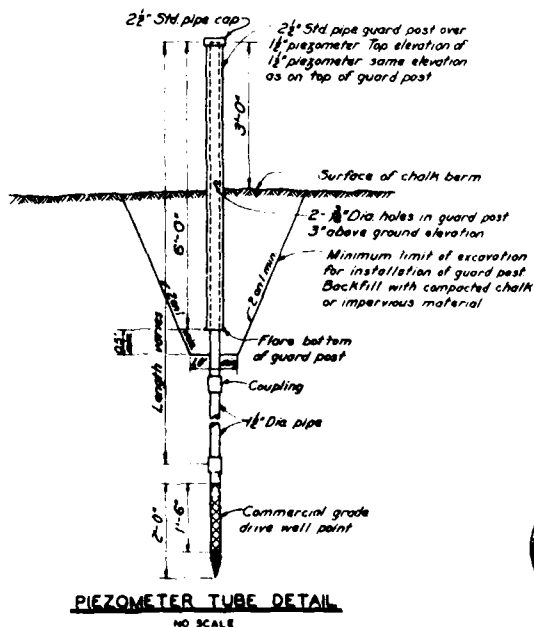
PIEZOMETER TUBE DETAIL
NO SCALE

2½" Std pipe cap

2½" Std pipe guard post over
1¼" piezometer Top elevation of
1¼" piezometer same elevation
as on top of guard post

3'-0"

Surface of chalk berm

2 - ⅝" Dia holes in guard post
3" above ground elevation

6'-0"

Minimum limit of excavation
for installation of guard post.
Backfill with compacted chalk
or impervious material

2' min

Flare bottom
of guard post

Coupling

1¼" Dia pipe

Length varies

2'-0"

1'-6"

Commercial grade
drive well point

NOTES:
1. All elevations shown refer to M.S.L. USC & GS 1929
general adjustments

DATE	DESCRIPTION		

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM - LAKE FRANCIS CASE
RELIEF WELLS &
PIEZOMETER TUBES
DETAILS

SCALE as shown   SPEC. NO.

SECTION 1-1

STA. 107+14.95

(ALONG ₵ OF TUNNEL NO 1)

SECTION 2-2

STA. 108+24.98

(ALONG ₵ OF TUNNEL NO. 4)

SECTION 3-3

STA. 108+84.98

ALONG ₵ OF TUNNEL NO.5

SECTION 4-4
STA. 113+44.98
(ALONG € OF TUNNEL NO. 10)

SECTION 5-5
(PROFILE THRU COULEE)

THIS DRAWING HAS BEEN REDUCED TO
THREE-EIGHTHS THE ORIGINAL SCALE.
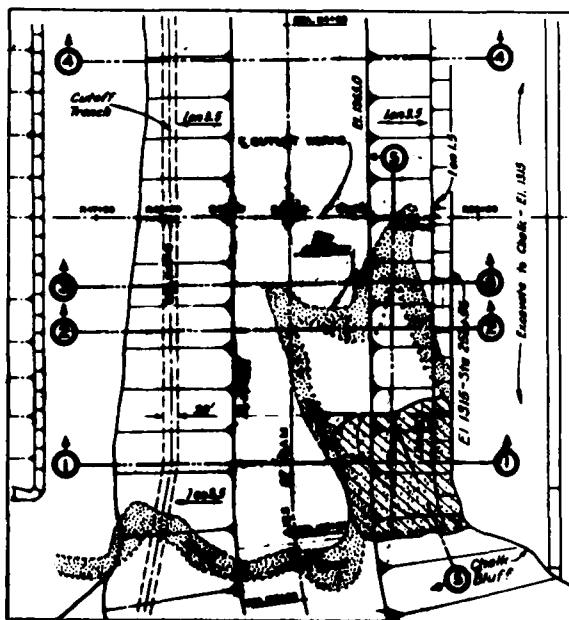
EMBANKMENT PLAN
€ STA. 1002 TO 842

SCALE: 1 INCH = 100 FEET

NOTES:

1. For location of plan see Dwg. No. R-156-4D/12.
2. Plan and Section 1-1, and Profile 5-5 show approximate limits of downstream pervious fill. Pervious fill elevations may be adjusted to meet field conditions, but this layer must have a minimum thickness of 10 feet and for it's full height must contact basal sand layer.

THIS PLAN ACCOMPANIES CONTRACT NO.
W-25-066-eng-1434   MODIFICATION NO. 7
THIS DRAWING ADDED TO ORIGINAL FOLIO
BY MODIFICATION NO.7

8-1-50	Revised to show "As-Built" conditions			
9-11-48	Revised D.S Pervious Drain			
DATE	DESCRIPTION		MADE	APPRD
	REVISIONS			

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
OFFICE OF THE DISTRICT ENGINEER
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL RESERVOIR
INITIAL EARTHWORK
DETAILED EMBANKMENT SECTIONS
LEFT ABUTMENT

SCALE AS SHOWN   SPEC NO

DRAWING NUMBER
R156-4/24

R 4770
S 118+68.3

R 4800

R 4900

R 48075

L on 3.5

R 4970
R 4970
S 118+68.3

EL 1395
STA 118+00   119+00   120+00   121+00

Δ=8°10'30"
D=1°01'32.28"
R=5588.84'
T=301.35'
L=602.07'

R 5030

R 5105

R 5140

R 5210

R 5225
R 5243
EL 1335

Transition   Transition

DETAIL - BACKFILL
RIVERWARD END SPILLWAY

SCALE 1 INCH=50 FEET
50   0   50'

RANGE VARIES SEE PLAN
EL 1385   See Detail A
RANGE VARIES SEE PLAN   Axis of dam   R 5030
EL 1370   EL 1385

See Detail B   Wall   1 on 2.5   Impervious fill   5' Pervious F

Niobrara chalk

R 4700   4800   4900   5000   5100

SECTION V-V

Riprap   R 4970
EL 1385   Axis of dam
R 5030
EL 1385

See Detail D'   R 48075   See Detail A'
EL 1370

1 on 3.5   Impervious fill

Shale   5' pervious fil

R 4700   4800   4900   5000   5100

SECTION W-W

1400   EL 1395   1400

Rolled embankment   Impervious fill

1350   1350

1300   Niobrara chalk   Wall   1300

1250
STA 116+00   117+00   118+00   119+00   120+00   121+00   1250

SECTION S-S

1400   1400

Rolled embankment   5' pervious fill   Wall drain
1350   Impervious fill   1350

1300   Shale and chalk   1 on 1   1300

STA 117+00   118+00   119+00   120+00   121+00   122+00

SECTION T-T

R 48075
EL 1370   R 4970
EL 1385   See "Detail A"
Axis of dam   R 5030
EL 1385

1 on 1
1400   1 on 2.5   Impervious fill   1400

1 on 3.5   1 on 3   1 on 1.5   1 on 2.5
1350   Rolled embankment   R 5255   1350
EL 1335   Pervious for drain

Niobrara chalk   Shale
1300   1300
R 4700   4800   4900   5000   5100   5200   5300

See "Detail C"   See "Detail A"

SECTION U-U

LOCATION PLAN
NO SCALE

APPROACH CHANNEL

AXIS OF DAM

OUTLET WORKS
SPILLWAY

DETAIL A
TOP OF RIPRAP
RIVERWARD OF WALL
NO SCALE

Top of dam
1 on 1
1 Spalls
2 Filter blanket
3 Riprap
1 on 2.5
Impervious fill
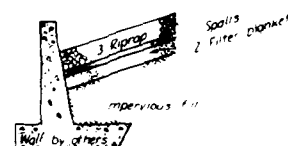
DETAIL B
TOE OF RIPRAP
BEHIND WALL
NO SCALE

3 Riprap
2 Filter blanket
Impervious fill
Wall by others
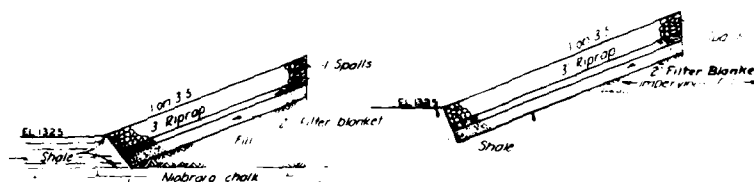
DETAIL C
RANGE 21+85 RIVERWARD
NO SCALE

1 Spalls
1 on 3.5
3 Riprap
Filter blanket
Fill
EL 1325
Shale
Niobrara chalk

DETAIL D
RANGE 21+00 TO 21+85
NO SCALE

1 on 3.5
3 Riprap
2 Filter Blanket
Impervious
EL 1325
Shale

DETAIL - BACKFILL
RIVERWARD END SPILLWAY
SCALE 1 INCH = 50 FEET

Spillway abutment
wall drain

SECTION V-V

RANGE VARIES SEE PLAN
EL 1395
See Detail A
Axis of dam
R 5030
EL 1395
RANGE VARIES SEE PLAN
EL 1370
1 on 1
R 5105
EL 1370
Detail B
1 on 2.5 warped
Impervious fill
1 on 3
R 5210
EL 1340
Wall
1 on 1
Warped slope
3 Pervious Fill
Niobrara chalk
See Detail

SECTION W-W

R 4970
EL 1395
Axis of dam
Piping
R 5030
EL 1395
See Detail D
R 48075
EL 1370
See Detail A
R 5103
EL 1370
Impervious fill
Shale
5 pervious fill
1 on 1
See Plate

NOTES

1. All ranges and stations are taken from embankment
   grids except as noted
2. All elevations shown refer to M.S.L. U.S.C.&G.S.
   929 general adjustment
3. (F) denotes finished surfaces

THIS DRAWING HAS BEEN REDUCED TO
THREE-FIFTHS THE ORIGINAL SCALE.

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA
MISSOURI RIVER
FORT RANDALL DAM - LAKE FRANCIS CASE
EMBANKMENT
EMBANKMENT DETAILS AT SPILLWAY
RIVERWARD ABUTMENT

LAYOUT

PROFILE-COLLECTOR PIPE

SECTION A-A

SECTION C-C

SECTION B-B

SECTION D-D

SECTION E-E

LAYOUT

& AXIS OF DAM
EL 1395
EL 1370
& OUTLET WORKS

MANHOLE NO 7
STA 252+25

EL 1395
EL 1370
PI
STA 253+00

EMB R 3140

LIMITS OF 5
PERVIOUS FILL
SEE PLATE A-16
STA 252+00

1 ON 2

LECTOR PIPE
R 1110
STA 252+55.8
& COLLECTOR PIPE
MANHOLE NO 6

MANHOLE
NO. 8
R 1780.0
STA 252+67

R 1260
STA 252+47.75

R 1595
STA 252+49
TRANSITION
TRANSITION

RANGE 1000
RANGE 1500

COLLECTOR PIPE

Invert El 1319.3
MANHOLE NO 7
Invert El 1321.7
TOP OF FILL OVER PIPE
LIMITS OF EXCAVATION
MANHOLE NO. 6
INVERT EL 1328

R 1110
STA 252+55.8
INVERT EL 1328

Invert El 1318.6
SLOPE = 0.00394
INVERT EL 1325.5
SLOPE = 0.018
INVERT EL 1328
SLOPE = 0.0025
SLOPE = 0.098

LIMITS OF EXCAVATION
BOTTOM OF PIPE TRENCH
IMPERVIOUS BLANKET

N I O B R A R A   C H A L K

LOCATION MAP
NO SCALE

MISSOURI RIVER
APPROACH CHANNEL
OUTLET WORKS
SPILLWAY
AXIS OF DAM

SECTION C - C

STA 252+26.5
0.5' TOP SOIL
12" PERFORATED CMP INVERT EL VARIES SEE PROFILE
PERVIOUS FILL
1 ON 1
FILTER, SEE DETAIL A
IMPERVIOUS BLANKET
1 ON 1
5' TOP OF BEDROCK
1' MIN
252+50   252+75

PROFILE - MH NO. 5 TO DI NO. I

& MANHOLE NO. 5
EXISTING GROUND LINE
DROP INLET NO. 1
INVERT EL 1315.32
S = 0.0215
140'-12" CMP
INVERT EL 1312.31

SECTION D - D

STA 252+31.5
0.5' TOP SOIL
LIMITS OF EXCAVATION
PERVIOUS FILL
12" PERFORATED CMP INVERT EL VARIES SEE PROFILE
2' MIN
APPROX TOP OF BEDROCK
252+50   252+75

SECTION E - E

0.5' TOP SOIL
PERVIOUS FILL
12" PERFORATED CMP INVERT EL VARIES SEE PROFILE
IMPERVIOUS FILL
APPROX TOP OF BEDROCK
FILTER, SEE DETAIL A
252+50   252+75

DETAIL A
NO SCALE

Pervious fill
12" perforated CMP
Filter (graded)
NOTE:
Perforation only in bottom 1/3 of pipe

SECTION F - F
NO SCALE

Excavation line
Impervious backfill
Existing ground line
12" CMP
2' 0"

NOTES:
1. All stations and ranges are taken from Outlet grids.
2. All CMP to be bituminous coated
3. (F) denotes finished surfaces.

THIS DRAWING HAS BEEN REDUCED TO
THREE-EIGHTHS THE ORIGINAL SCALE.

REVISIONS

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM - LAKE FRANCIS CASE

EMBANKMENT

TOE DRAIN - LEFT BANK

SCALE AS SHOWN   SPEC. NO.
DRAWING NUMBER

SAFETY PAYS

THIS PLAN ACCOMPANIES CONTRACT NO.
DA-25-066-eng- , MODIFICATION NO.

25000 N

70000 E

30000 N

1300

1320

1310

1320

1330

1340

1350

1360

1370

1380

1390

1 on 15

EL 1355

R 4710

25000 N

R 4920

R 4945

1 on 2.5

EL 1385

1 on 1.8

R 4930

AXIS OF DAM 2

20+00

R 50+00

EL 1365

R 5050

30+00

EL 1370

70000 E

1390

1400

1410

1 on 3

EL 1350

R 5175

STA 29+00

End collector ditch

Control line &
€ pipe

Pervious fill

1 on 10

STA 50+00

Chalk contact line
Transition

R 5282.3
STA 29+00

1 on 3

1 on 3

Manhole #4

1370

1380

1350

1340

Manhole #1

Manhole #2

R 5478.5
STA 37+00

Manhole #3

1 on 4.5
Slope to

1330

Toe of impervious fill
R 5837.5
STA 50+00

1320

1310

1300

Toe of chalk berm

SCALE 1 INCH = 200 FEET
200   0   200

1290

Notes
All elevat
GS 1929
Work an
facilities a
Be indica

EL 1338   R 4455

EL 1355   R 4710

EL 385   R 5000   R4870

R 5030   AXIS OF DAM   60+00   R 5030   R 5105   70+00   80+00

EL 370

R 5175

EL 1520   R 5280

Impervious fill   EL 1303   R 5300

Chalk contact line
Transition

Manhole #4   R 55575
STA 57+00

&. Pipe
Transition
chalk berm

See Plate A-9

Manhole #3

Toe of impervious fill   R 5830   END OF PIPE   STA 63+30   STA 66+30   STA 69+30   STA 72+30   STA 75+15
R 56375   STA 57+15   STA 76+20
STA 50+00   R5850

Toe of chalk berm   R 5900   STA 61+80   STA 72+60
STA 61+30

Chalk berm

LEGEND

●   RELIEF WELLS TO BE EXTENDED THIS
      CONTRACT - INSTALLED BY OTHERS

▲   RELIEF WELL PIEZOMETER

SCALE 1 INCH = 200 FEET
200   0   200

R 6880±   EL 1266±

R 6930±   EL 1266±

EL 1250±   EL 1245±

Notes
    All elevations shown refer to MSL, USC and
GS 1929 general adjustment.
2. Plan shows right bank embankment and drain
facilities at completion of embankment construction.
Toe roads are not shown.

LOCATION MAP
NO SCALE

APPROACH
CHANNEL

OUTLET
WORKS

SPILLWAY

AXIS OF DAM

THIS PLAN ACCOMPANIES CONTRACT NO.
DA-28-066-eng-   , MODIFICATION NO.

SAFETY PAYS

EXISTING ROAD SLOPE
(TOP EL 1405 APPROX)

P I V C
STA 13 + 60
EL 1398 83
V C 650

0.03588 %

STA 22+80

0.0131 %

STA 45+70

0.303 %

35'

IMPERVIOUS FILL

32'

EL 1305

STA 54+80

0.5'

0 %

STA 61+50

0.0808 %

STA 67+00

0 %

STA 72+00

0.0284 %

10+00   20+00   30+00   40+00   50+00   60+00   70+00   80+00   90+00   100+00

PROFILE
OVERBUILD ALONG CREST OF DAM

HOR SCALE 1 INCH = 400 FEET
400   0   400

VERT SCALE 1 INCH = 2 FEET
2   0   2

TABLE NO 1

MANHOLE NO	INVERT ELEVATIONS	
	INLET	OUTLET
1	—	1330 5
2	1310 0	1310 0
3	1293 0	1291 5
4	1280 0	1280 0

* * SEE MANHOLE SECTION AT

MANHOLE #1

1340   TOP 5' LAYER PERVIOUS FILL

MANHOLE #2

1320   0.0253   TOP OF ROLLED EMB

FLOW LINE OF 8" PERF CMP   0.0130   ORIGINAL GROUND LINE

MANHOLE #3

1300   APPROX 3'   IMPERVIOUS FILL

MANHOLE #4

CHALK BERM

1280   0.0184   FLOW LINE 8" CMP SEE DETAIL "B"
IMPERVIOUS FILL SEE DETAIL "A"   BOTTOM OF MANHOLE PIPE INV EL 1280

1260

0   400   800   1200   1600   2000   2400   2800   3200

EMB STA 29+00   EMB STA 37+00   EMB STA 50+00   EMB STA 57+00

PROFILE OF COLLECTOR DITCH
PERVIOUS FILL RIGHT BANK
ALONG CONTROL LINE

1340   1320   1300   1280   1260

R 56375 STA 50+00

CONTROL LINE & € OF PIPE

THIS KNOB REMOVED AS SHOWN

THIS AREA IMPERVIOUS FILL AS SHOWN

DETAIL "A"
EXCAVATION & FILL
LOWER PART PERVIOUS FILL RIGHT BANK
SCALE 1 INCH = 200 FEET
200   0   200

ALL STAS & RANGES ARE ON EMB GRID

1340   IMPERVIOUS FILL

1320   IMPERVIOUS FILL   TRANSITION SLOPE

1300   IMPERVIOUS FILL   PERVIOUS FILL   1 ON 10   PERVIOUS FILL
IMPERVIOUS FILL   1 ON 3

1280   PERVIOUS FILL   SEE DETAIL "E-1"

R 5200   R 5300   R 5500   R 5600

SAND DRAIN SECTION
STA 29+00 TO STA 55+40

1340   TRANS SLOPE   8" BITUMINOUS COATED CMP   DUMPED CHALK
1 ON 3   8" PERF BITUMINOUS COATED CMP (SEE DETAIL D)

1320   ROLLED EMB   30'   50'
€ MANHOLE   0.05% SLOPE   LEVEL
1280   ORIGINAL GROUND   END FILTER SAND

0   40   80

PROFILE I-I
OUTFALL RIGHT BANK
SAND DRAIN

SEE PLATE A-18
FOR CHALK CONTACT LINE

SLOPE VARIES SEE PLATE A-18
SLOPE VARIES
IMPERVIOUS FILL   DUMPED CHALK FILL
EXISTING PERVIOUS FILL   1 ON 3   PERVIOUS FILL
SEE DETAIL "A", "E-2" & PROFILE ABOVE

TYPICAL SECTION
DOWN STREAM EDGE SAND DRAIN
STA 55+40 TO STA 57+00
NO SCALE

CONTROL LINE (SEE PLATE A-18)

1 ON 10
IMPERVIOUS FILL   1 ON 3
PERVIOUS FILL   1 ON 10
ORIGINAL GROUND

1 ON 1   1 ON 1
8" PERFORATED BITUMINOUS COATED CMP   GRADED FILTER

DETAIL E-1
WHERE COLLECTOR PIPE
BELOW ORIGINAL GROUND SURFACE

IMPERVIOUS FILL
PERVIOUS FILL   PERVIOUS FILL
IMPERVIOUS FILL   1 ON 1
8" PERFORATED BITUMINOUS COATED CMP

AXIS OF DAM

OVERBUILD

CREST OF DAM

EL 1395
R 4870

EL 1395
R 4870

TYPICAL CROSS SECTION
OVERBUILD ALONG CREST OF DAM
NO SCALE

CREST OF DAM

FEET

FEET

STA 67+00
STA 72+00
STA 106+00

70+00   80+00   90+00   100+00   110+00

TABLE NO I

MANHOLE NO.	INVERT ELEVATIONS		DEPTH "d" @ @
	INLET	OUTLET	
1	—	1350.5	12'
2	1310.0	1310.0	12'
3	1293.0	1291.5	12'
4	1280.0	1280.0	20'

@ @ SEE MANHOLE SECTION AT RIGHT

1" Ø GALVANIZED STEEL BAR

LADDER RUNG DETAIL

SLOPE SEE PROFILE

8" PERF CMP

103°45'

DETAIL "B"
MANHOLE #4 & PIPE ALIGNMENT

SLOPE SEE PROFILE

8" PERF CMP

103°45'

DETAIL "C"
MANHOLE NO'S 1,2,3 & PIPE ALIGNMENT

MANHOLE DETAILS
NO SCALE

MANHOLE FRAME & TYPE "C" COVER
NEENAH R-1040-C OR EQUAL

DETAIL "B" & "C"

1" CEMENT PLASTER

8" BRICK WALL

4" DIA

INV EL VARIES
SEE TABLE #1

8" CMP

8" CONCRETE FLOOR

DEPTH "d" VARIES - SEE TABLE NO I

MANHOLE SECTION

1340
1320
1300
1280
1260

MANHOLE #4

CHALK BERM

FLOW LINE 8" CMP SEE DETAIL "B"

BOTTOM OF MANHOLE PIPE INV EL 1280

PERVIOUS FILL

DETAIL "A"

EMB STA 57+00

2800   3200

1290
1288
1286

R 54375
STA 50+00

R 54572
STA 57+00

4' DIA MANHOLE

R 5830
STA 57+15

END OF PIPE

CONTROL LINE &
£ OF PIPE

THIS KNOB REMOVED AS SHOWN

1275

103°45'

THIS AREA IMPERVIOUS FILL AS SHOWN

DETAIL "A"
EXCAVATION & FILL
LOWER PART PERVIOUS FILL   RIGHT BANK
SCALE 1 INCH = 200 FEET

ALL STAS & RANGES ARE ON EMB GRID

THIS DRAWING HAS BEEN REDUCED TO
THREE-EIGHTHS THE ORIGINAL SCALE

1340
1320
1300
1280

TRANS SLOPE

8" BITUMINOUS COATED CMP

DUMPED CHALK

8" PERF BITUMINOUS COATED CMP (SEE DETAIL D)

ROLLED EMB

£ MANHOLE

30'

0.05% SLOPE

30'

LEVEL

ORIGINAL GROUND

END FILTER SAND

1300
1280

0   40   80

PROFILE I-I
OUTFALL RIGHT BANK
SAND DRAIN

DUMPED CHALK

8"   D   8"

GRADED FILTER

NATURAL GROUND

8" PERFORATED BITUMINOUS COATED CMP

DETAIL D

NOTES:
1. All elevations shown refer to MSL USC&GS 1929 general adjustments.
2. All details and sections shown on this drawing are "As Built" at completion of embankment construction.

PERVIOUS FILL

1 ON 3

SEE DETAIL "E-1"

CONTROL LINE
(SEE PLATE A-18)

FILL

1 ON 3

VIOUS FILL

1 ON 18

AL GROUND

IMPERVIOUS FILL

CONTROL LINE

IMPERVIOUS FILL

1 ON 3

PERVIOUS FILL

PERVIOUS FILL

1 ON 18

IMPERVIOUS FILL

8.0'

1 ON 1

1 ON 1

ORIGINAL GROUND

GRADED FILTER

8" PERFORATED BITUMINOUS COATED CMP

GRADED FILTER

BITUMINOUS COATED CMP

GRADED FILTER

DETAIL E-1
WHERE COLLECTOR PIPE
BELOW ORIGINAL GROUND SURFACE

DETAIL E-2
WHERE COLLECTOR PIPE
IN IMPERVIOUS FILL

SAFETY PAYS

THIS PLAN ACCOMPANIES CONTRACT NO. DA-___-___ MODIFICATION NO.

SECTION A-A
STA 13+00

SECTION B-B
STA 8+00

SECTION C-C
STA 2+50

SECTION D-D
STA. 48+00

SECTION E-E
STA. 56+00

SECTION F-F
STA. 60+00

PLAN
SCALE: 1 INCH = 100 FEET

AXIS OF DAM   R 5000

LOCATION MAP
NO SCALE

CURVE DATA
Δ=55° 26' 46"
D= 4° 48' 08.4"
T = 627.15
L = 1154.5'
R = 1193.38'

IMPERVIOUS BLANKET

ROLLED EMBANKMENT

MISSOURI RIVER

LIMITS OF EXCAVATION
EXCAVATE & RECOMPACT TOP 5 FEET

AXIS OF DAM   R 5000

1 ON 20

FLOW

BANK LINE NOV. 1948
HYDROGRAPHIC SURVEY

NOTE:
1   All elevations shown refer to M.S.L.-U.S.C.& G.S. 1929 general
adjustment

PLAN
SCALE: 1 INCH = 100 FEET

TOP OF EXISTING GROUND
LIMITS OF EXCAVATION
LOESS
CLAY
SAND & GRAVEL
NIOBRARA CHALK

PROFILE R-R
LOESS REMOVAL

ORIGINAL GROUND LINE
LIMITS OF EXCAVATION
CLAY & CLAYEY SILT
SAND & GRAVEL
NIOBRARA CHALK
487 PROJ · 433 PROJ · 08 PROJ · 218 PROJ
50+00 · 55+00

PROFILE S-S
LOESS REMOVAL

LOESS
ORIGINAL GROUND LINE
LIMITS OF EXCAVATION
CLAY & CLAYEY SILT
SAND & GRAVEL
NIOBRARA CHALK
487 PROJ · 431 PROJ · 223 PROJ · 225 PROJ
45+00 · 50+00 · 55+00

PROFILE T-T
LOESS REMOVAL

LOESS
ORIGINAL GROUND LINE
LIMITS OF EXCAVATION
CLAY & CLAYEY SILT
SAND & GRAVEL
NIOBRARA CHALK
LOESS
487 PROJ · 429 PROJ · 427 PROJ
0 · 500

PROFILE U-U
& OF CUTOFF TRENCH

LOESS
CLAY & SILTY CLAY
ORIGINAL GROUND LINE
LIMITS OF EXCAVATION
SAND, GRAVEL & BOULDERS
BOTTOM OF CUTOFF TRENCH
NIOBRARA CHALK
172 PROJ · 12 PROJ

SECTION 1-1
R 4400

LIMITS OF EXCAVATION
ORIGINAL GROUND LINE
1 ON 2
1% SLOPE
EL 1300
50+00 · 55+00

SECTION 2-2
R 4700

LIMITS OF EXCAVATION
ORIGINAL GROUND LINE
1 ON 2
1% SLOPE
50+00 · 55+00

NOTES:
1. Elevations shown refer to M.S.L., U.S.C. & G.S. general adj.
2. Location of Profiles and Sections are shown on Plate 4.
3. Location of borings are shown on Plate 4.
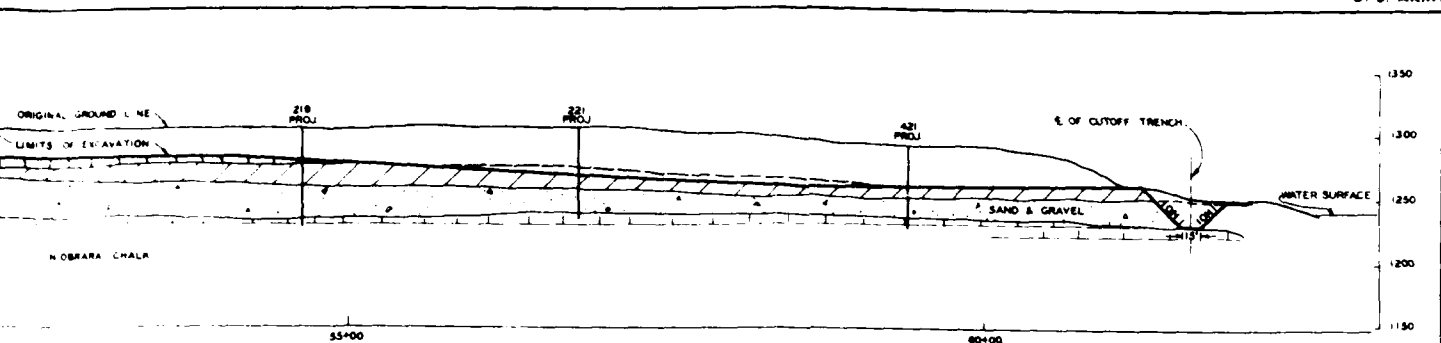
PROFILE R-R
LOESS REMOVAL
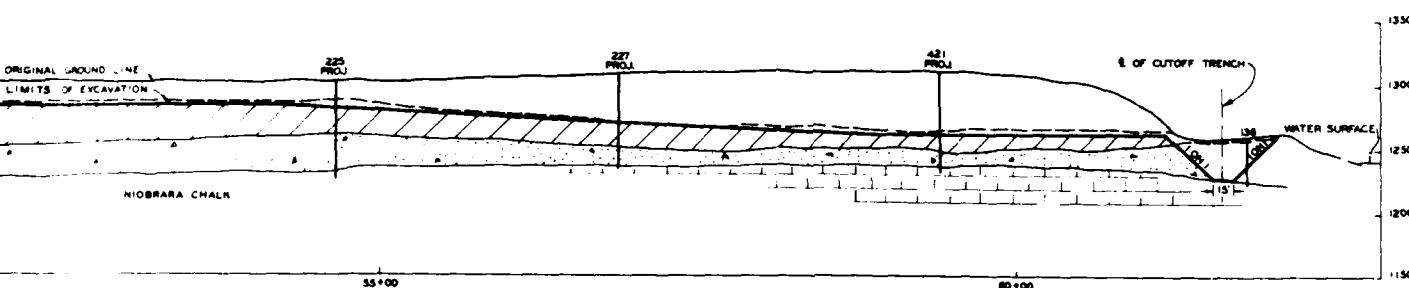
PROFILE S-S
LOESS REMOVAL
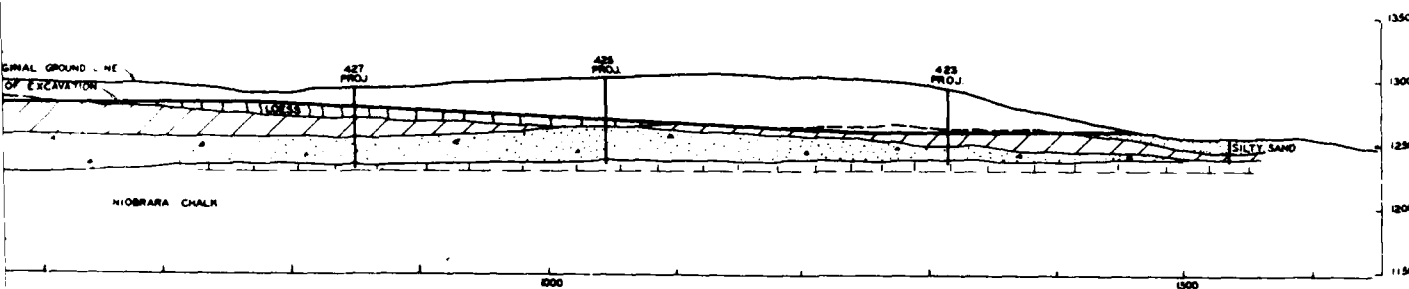
PROFILE T-T
LOESS REMOVAL
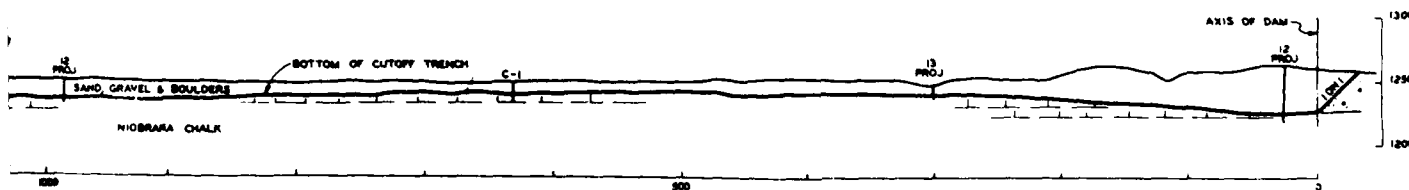
PROFILE U-U
¢ OF CUTOFF TRENCH

NOTES:
1 Elevations shown refer to M.S.L., U.S.C. & G.S. general adj.
2 Location of Profiles and Sections are shown on Plate A-20
3 Location of profiles are shown on Plate A-
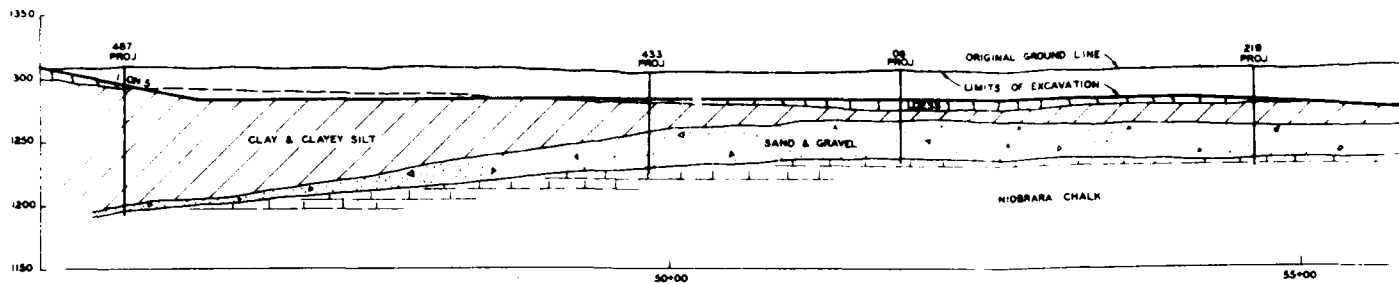
CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM - LAKE FRANCIS CASE

EMBANKMENT

RIGHT BANK
PROFILES

Bank protection placed under this
contract to be removed prior to
closure operations

Chalk bank protection placed
under prior contracts to be
removed under this contract

Bank line Nov. 1948

MISSOURI

FLOW

EL. 1255

Chalk & shale berm
EL. 1300 ±

Chalk and
Shale berm

EL. 1330.0

EL. 1255

EL. 1325.0

1 on 5
1 on 3
1 on 2
1 on 5
1 on 2
1 on 5
1 on 5
1 on 2
1 on 7
1 on 1½
1 on 2
1 on 3.5
1 on 3.5

℄ Cutoff trench

AXIS OF DAM R.3900

80+00
70+00

1 on 5

1000' at EL. 1245

EL. 1265

1240

1310

EL. 1300

1 on 7
1 on 7

Dumped chalk placed
under prior contract to be removed
under this contract.

RIVER

Bank line April 1949

Upstream limit
chalk blanket

Upstream limit
chalk base

Chalk berm

Chalk bank
protection

℄ DIVERSION DAM

R.8900

Downstream limit
Chalk base

RIGHT BANK DIVERSION STA. 0+00

LEFT BANK DIVERSION STA. 22+00

R.7900

Downstream limits
chalk blanket

A

PLAN
SCALE : 1 INCH = 800 FEET

1

SECTION B - B

1260
1250
1240
1230
1220

STA 22+90

Dumped

Dredged chalk diversion weir

Dredged chalk protection blanket

STA 22+00   20   18   16

SECTION A -

Axis of diversion

Dumped chalk

Weir crest
by dredging   EL. 1241
Diversion weir
FLOW   EL. 1236
Tail water EL. 1238.92

EL. 1230 ±

Dredged chalk protection
blanket. Placed 4/29 to 5/22
by dredging

1260
1250
1240
1230
1220

R.750

R.700   R.500   R.300

RIVER CHANNEL
RANGE 8900

RIGHT BANK

W.S.EL 1248.5

W.S.EL 1430

W.S.EL 1246

1250
1240
1230
1220
1210

SECTION B - B

Dumped chalk

Dredged chalk diversion weir

Dredged chalk protection blanket

Original river bed

STA 22+00 20 18 16 14 STA 12+00

Axis of diversion

Dumped chalk 100' El. 1265

Weir crest by dredging

El.1261

Diversion weir
FLOW

El.1256

Designed crown
El.1243

Head water El.1242.65

Tail water El.1238.92

El.1230'

Designed super base

Dredged chalk protection blanket. Placed 4/29 to 5/22 by dredging

Original river bed El 1225

R700 R500 R300 R100 0 R-(200)

Dredged chalk diversion dam

SECTION A - A

NOTE:
DESIGNED STAGES - DIVERSION WEIR
Stage ① Dredged chalk base
Stage ② Dredged chalk base
Stage ③ Dredged chalk base
Stage ④ Dredged chalk super base
Stage ⑤ Dredged chalk crown
Stage ⑥ Load & haul chalk

REFERENCE DRAWINGS:

NOTES
1 All elevations shown refer to MSL. USC&GS general adjustment
2 Side slopes of diversion dam shall be at natural angle of repose as determined in the field

RIGHT BANK LEFT BANK

WSEl 1248.5

WSEl 12430

WSEl 1243.4

WSEl 12446

WSEl 12410
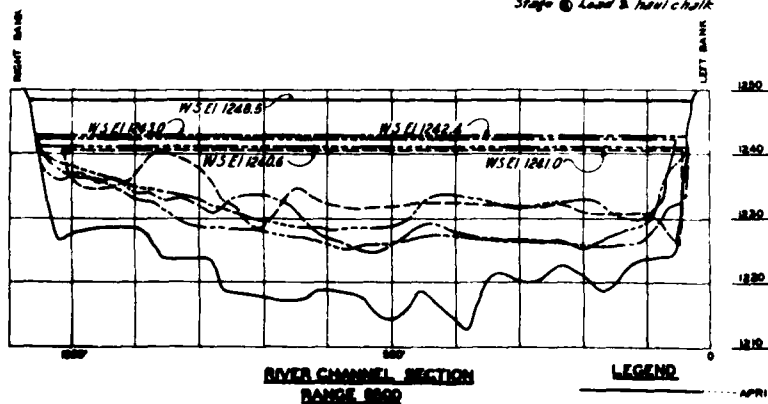
RIVER CHANNEL SECTION
RANGE 8800

LEGEND

.... APRIL, 1949
———— MAY, 1949
.... JUNE, 1949
———— JULY, 1949
———— AUG & SEPT, 1949

2-2-56 Revised to show "As Built" conditions

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL RESERVOIR
EARTHWORK
STAGE III
DIVERSION
PLAN & SECTIONS

MARCH 1956

SCALE AS SHOWN

MR14-31E15.1

1 ON 3
EL 1255
FLOW
IMPERVIOUS BLANKET
1 ON 150

1400 CHALK PROTECTION   BLANKET CLOSURE FILL THIRD STAGE   BLANKET CLOSURE FILL SECOND STAGE   EMB
BLANKET CLOSURE FILL FIRST STAGE   TO IMPERVIOUS BLANKET
1300   EL 1258   100'±   IMPERVIOUS BLANKET   EL 1275   50'   1 ON 3 EL 1288   EL 1301
1 ON 2   TOP OF EMBANKMENT
147'±
1200   TOP OF BLANKET FOUNDATION
BLANKET ZONE - MAX LIMITS OF DUMPED PERVIOUS FILL
1100   R 4000   R 4500

SEC
EMBANKMENT FOUNDATIO

ORDER OF CONSTRUCTION:

1. Blanket Closure Fill, First Stage. To permit lowering of the river level in the blanket and embankment areas to tailwater level.
2. Blanket Closure Fill, Second Stage. To provide flood protection for the rolled embankment construction and permit installation of unwatering equipment.
3. Embankment Closure Fill upstream of Range 5000.
4. Balance of Blanket Closure Fill, and Embankment Closure Fill downstream of Range 5000.

R 4400   EL 1288
1 ON 7
1 ON 8.5
R 4600   EL 1305
R 4700   ROLLED EMBANKMENT   R 4754
R 4800   EL 1345
60+00   70+00
AXIS OF DAM R 5000
APPROX   EL 1385
R 5300
R 5550
EMBANKMENT FOUNDATION
1 ON 3   R 5754
EL 1260±
SHALE BERM
DIVERSION DIKE EL 1295±

STA 9+50 R 0.00 } DIVERSION GRID
STA 99+79.00 STA 115+34.70 } DETOUR HIGHWAY

1400   EMBANKMENT CLOSURE FILL FIRST STAGE   EM
FLOW
BLANKET CLOSURE FILL SECOND STAGE
1300   BLANKET CLOSURE FILL FIRST STAGE   50' MIN   EL 1309
CHALK PROTECTION   TOP OF BLANKET FOUNDATION   ROLLED
EL 1258   100'±   EL 1275   TOP O
147'±   EL 1288   1 ON
1200   BLANKET ZONE - MAX LIMITS OF DUMPED PERVIOUS FILL   EMBANKMENT ZON
R 4000   R 4800

SECTI
EMBANKMENT FOU

ORDER OF CONSTRUCTION:

1. Blanket Closure Fill, First Stage. To permit lowering of the river level in the blanket and embankment areas to tailwater level.
2. Embankment Closure Fill, First Stage. To provide flood protection for the rolled embankment construction.
3. Balance of Embankment Closure Fill upstream of Range 5000.
4. Balance of Blanket Closure Fill, and Embankment Closure Fill downstream of Range 5000.

NOTE:
✦ SETTLEMENT GAGES

CLOSURE PLAN
AT END OF CLOSURE CONSTRUCTION YEAR
SCALE: 1 INCH = 200 FEET

1

SECTION THRU CLOSURE
EMBANKMENT FOUNDATION FILL PLACED IN UNWATERED RIVER CHANNEL
SCALE 1 INCH = 100 FEET

ORDER OF CONSTRUCTION:
1. Blanket Closure Fill, First Stage. To permit lowering of the river level in the blanket and embankment areas to tailwater level.
2. Blanket Closure Fill, Second Stage. To provide flood protection for the rolled embankment construction and permit installation of unwatering equipment.
3. Embankment Closure Fill upstream of Range 5000.
4. Balance of Blanket Closure Fill, and Embankment Closure Fill downstream of Range 5000.

NOTES:
1. Simultaneous construction of the various stages will be permitted to facilitate full use of construction equipment.
2. Pervious fill shall be placed to the minimum height above water which will permit satisfactory construction of the impervious blanket and rolled embankment.
3. All elevations shown refer to M.S.L., U.S.C.& G.S. general adj.

SECTION THRU CLOSURE
EMBANKMENT FOUNDATION FILL PLACED IN WATER
SCALE: 1 INCH = 100 FEET

ORDER OF CONSTRUCTION:
1. Blanket Closure Fill, First Stage. To permit lowering of the river level in the blanket and embankment areas to tailwater level.
2. Embankment Closure Fill, First Stage. To provide flood protection for the rolled embankment construction.
3. Balance of Embankment Closure Fill upstream of Range 5000.
4. Balance of Blanket Closure Fill, and Embankment Closure Fill downstream of Range 5000.

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

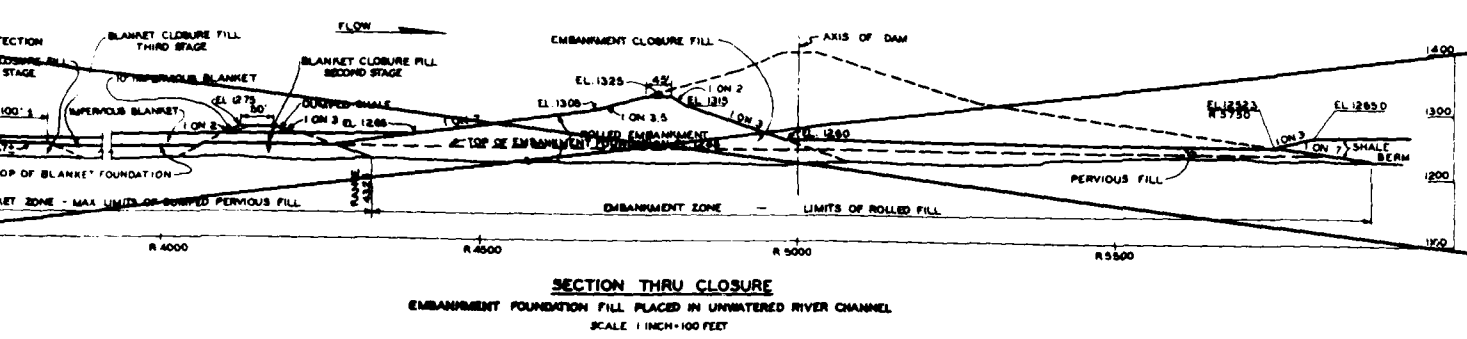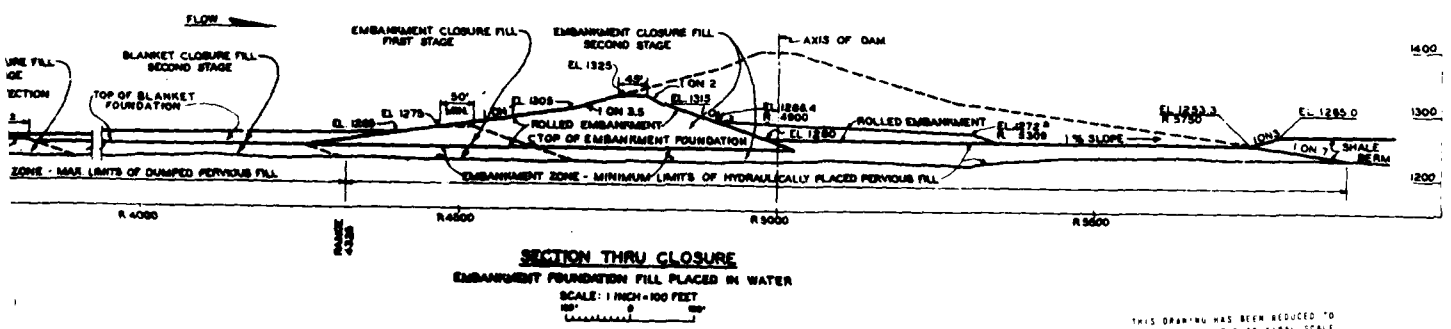|				
2-2-56	Revised to show As Built conditions			
	Revised encircled items			

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL RESERVOIR
EARTHWORK
STAGE III
CLOSURE
PLAN & SECTIONS

DATE: MARCH 1950

SCALE: AS SHOWN

MR14-31E16.2

THIS PLAN ACCOMPANIES CONTRACT NO. DA-25-050-ENG-360, MODIFICATION NO.

EMBANKMENT CRITERIA AND PERFORMANCE REPORT   PLATE A23

2

MECHANICAL ALYSIS CURVES

U.S. ENGINEER LABORATORY
U.S. ENGINEER OFFICE - MISSOURI RIVER DIVISION

Project: FORT RANDALL RESERVOIR MISSOURI RIVER BASIN

RANGE OF GRADATION
VALLEY SANDS
U.S. ENGINEER OFFICE OMAHA, NEBR.

EMBANKMENT CRITERIA AND PERFORMANCE REPORT   PLATE A24

MECHANICAL ANALYSIS CURVES

U.S. ENGINEER LABORATORY
U.S. ENGINEER OFFICE - MISSOURI RIVER DIVISION

Project ___ FORT RANDALL RESEBVOIR ___ MISSOURI RIVER BASIN ___
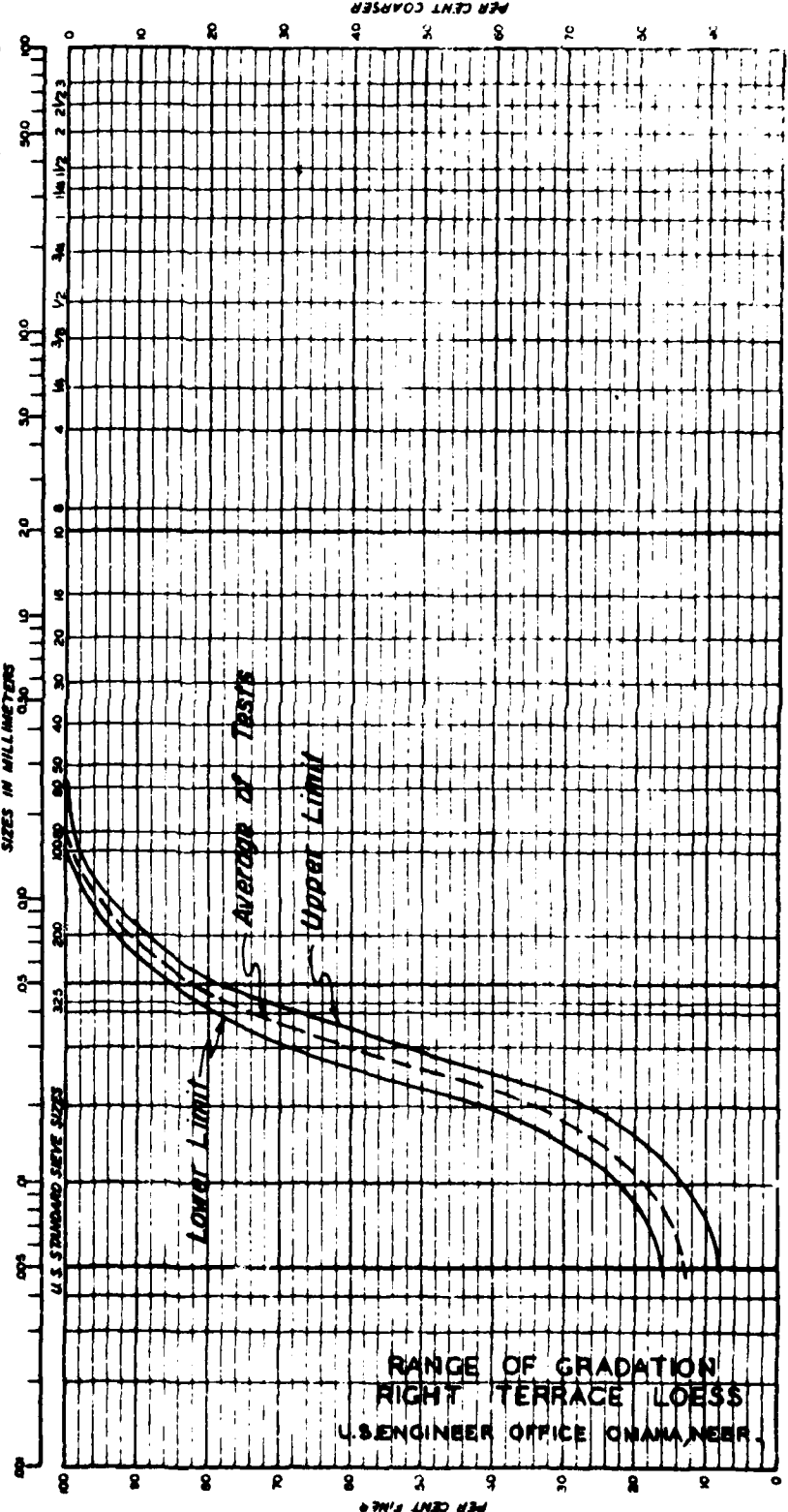
RANGE OF GRADATION
RIGHT TERRACE LOESS
U.S.ENGINEER OFFICE OMAHA.NEBR.

MECHANICAL ANALYSIS CURVES
U.S. ENGINEER LABORATORY
U.S. ENGINEER OFFICE - MISSOURI RIVER DIVISION

Project FORT RANDALL RESERVOIR   MISSOURI RIVER BASIN

RANGE OF GRADATION
RIGHT TERRACE CLAYS
U.S. ENGINEER OFFICE OMAHA, NEBR.
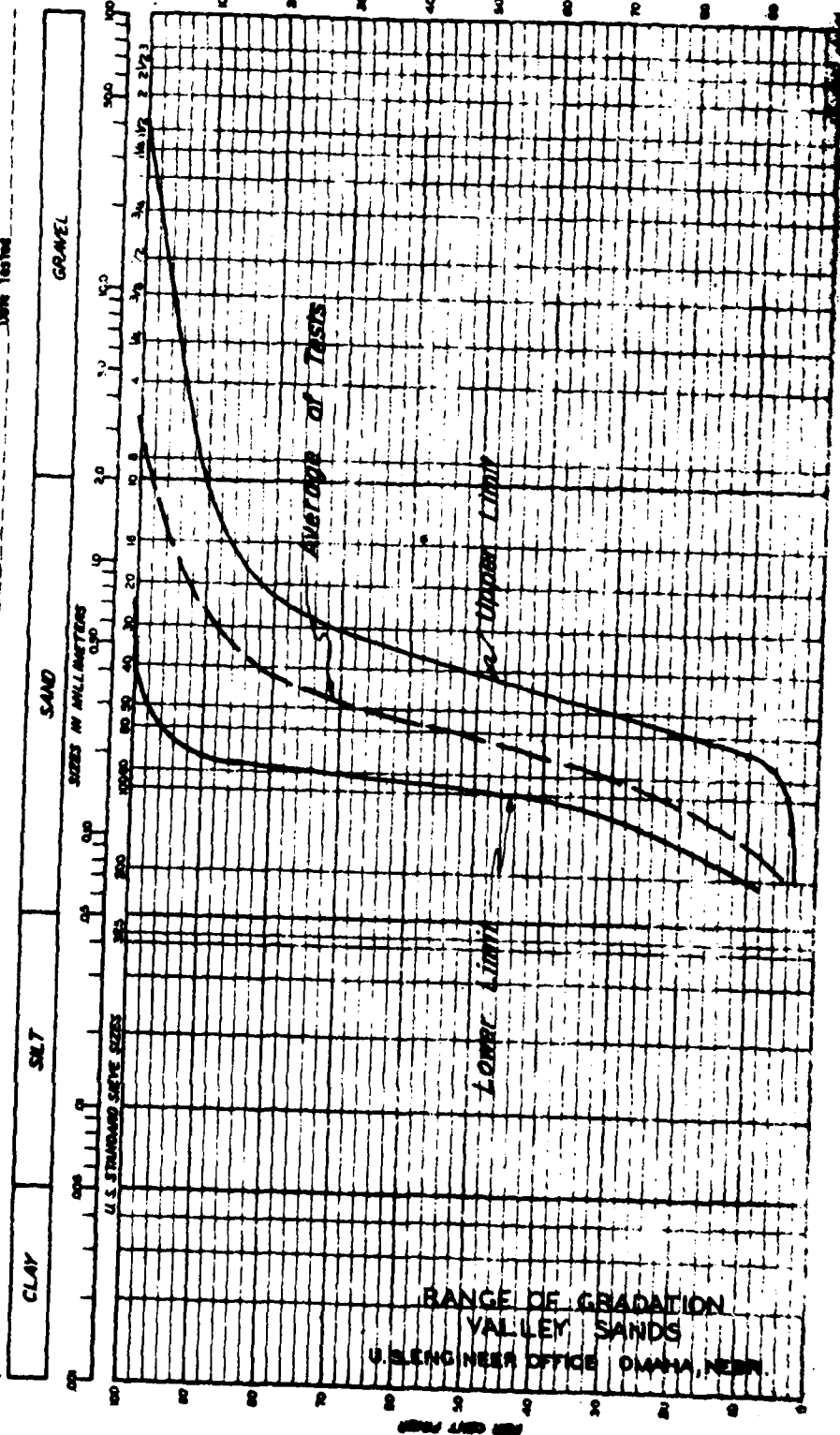
Average of Tests
Upper Limit
Lower Limit
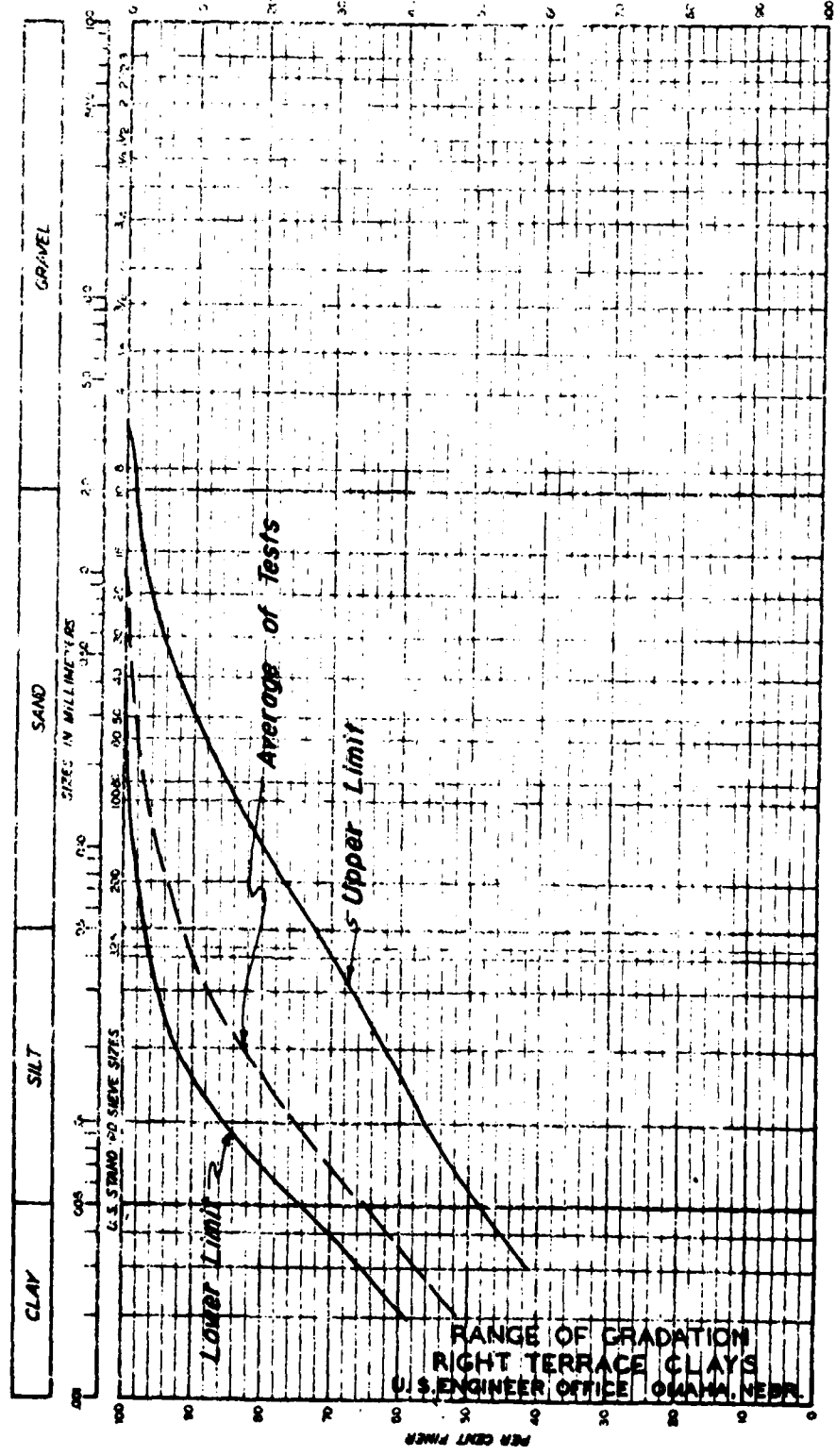
MECHANICAL ANALYSIS CURVES
U.S. ENGINEER LABORATORY
U.S. ENGINEER OFFICE - MISSOURI RIVER DIVISION

FORT RANDALL RESERVOIR   MISSOURI RIVER BASIN
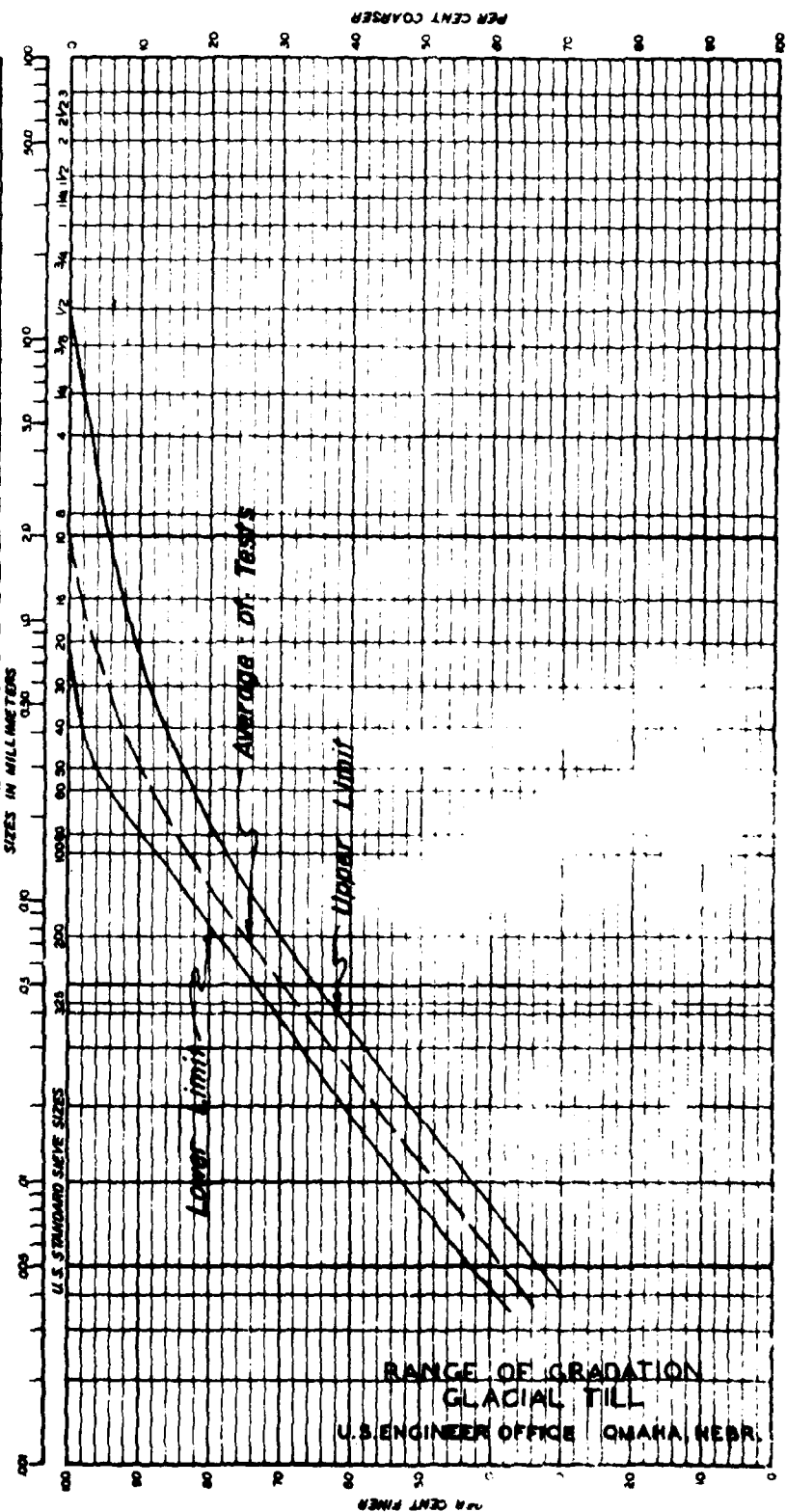
RANGE OF GRADATION
GLACIAL TILL
U.S. ENGINEER OFFICE   OMAHA, NEBR.

EMBANKMENT CRITERIA AND PERFORMANCE REPORT   PLATE   A27

STRESS STRAIN CURVES

HORIZONTAL LOAD-TONS / SQ. FT.

HORIZONTAL — MOVEMENT IN INCHES — VERTICAL

5T

3T

1T

SHEARING STRENGTH

HORIZONTAL LOAD-TONS / SQ. FT.

VERTICAL LOAD-TONS / SQ. FT.

Volumetric change from FME Moist.—83%
U. S. P. R. Classification————A-7

MECHANICAL ANALYSIS

CLAY	SILT	SAND

CUMULATIVE PER CENT FINER
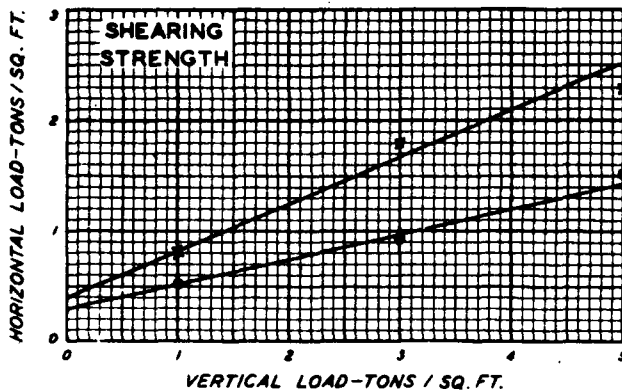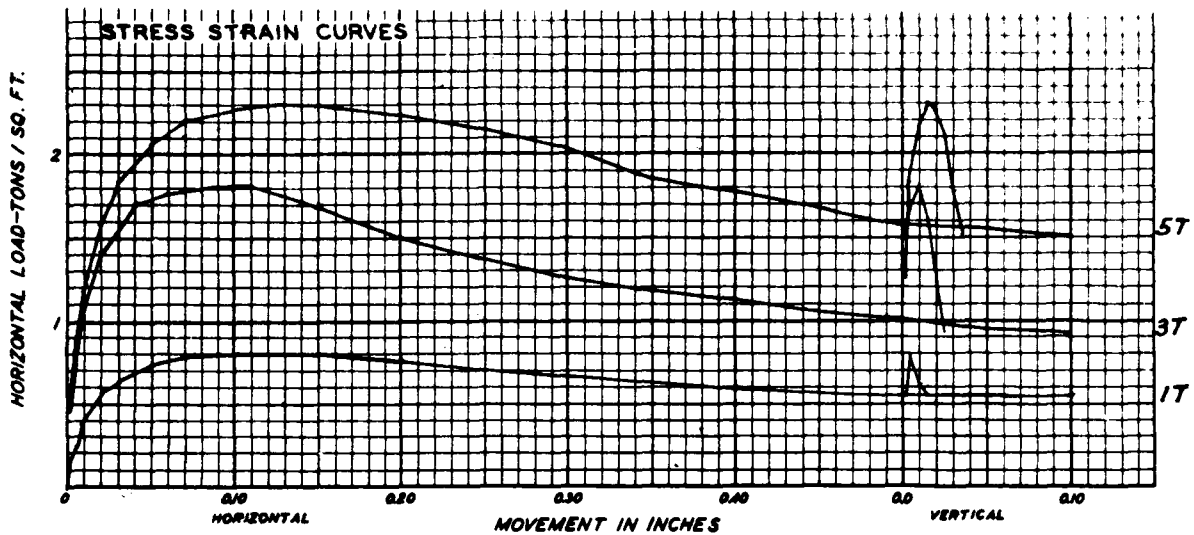
SHEARING STRENGTH
Maximum: c = .40   tan ∅ = .42
Ultimate: c = .30   tan ∅ = .22
Sample consolidated under vertical
load before application of strain.
Strain applied by motor drive.
  Moisture, before test = 29.5%
  Voids, before test   = 50.0%
  Specific Gravity   = 2.80
Liquid Limit———————— 90
Plastic Limit———————— 32
Plasticity Index———————— 58
Flow Index———————— 24
Shrinkage Limit———————— 17.7%
Shrinkage Ratio———————— 1.870
Lineal Shrinkage———————— 18%
Field Moisture Equiv.——— 62%
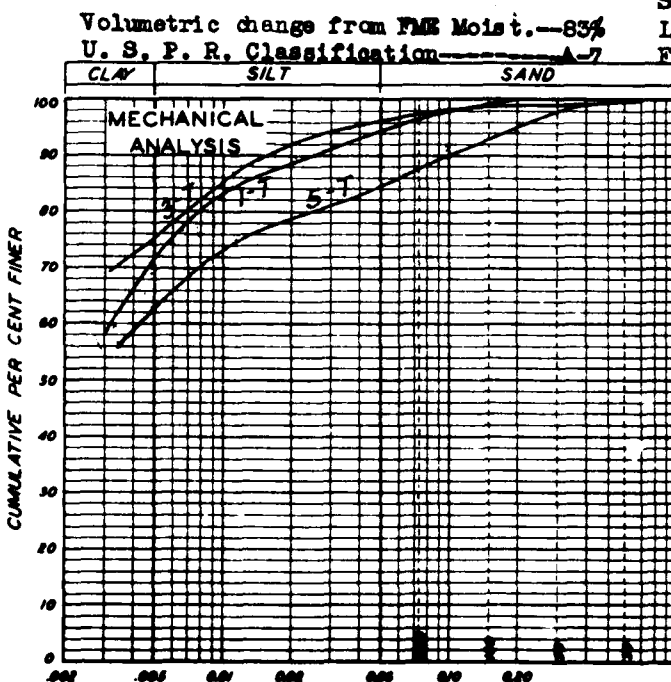
Project Ft. Randall
Stream Missouri River
Hole No. C-2
Location ▮▮▮▮▮▮▮▮▮
Ground Elev.
Sample No. U-6
Depth 46.0-46.7
Remarks

Insufficient material to run check
test.
Dry weight, before test= 87.5 lbs
                   /Cu.Ft.

RIGHT TERRACE CLAY
DIRECT SHEAR TEST

U.S. ENGINEER SOILS LABORATORY
U.S. ENGINEER OFFICE
MISSOURI RIVER DIVISION
KANSAS CITY, MO.

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

PLATE A28

STRENGTH DETERMINED FROM
ULTIMATE SHEAR VALUES

ADOPTED VALUES
C=.4 T/SQ FT
TAN ∅=0.22

VERTICAL LOAD IN TONS PER SQ. FT.

HORIZONTAL LOAD IN TONS PER SQ. FT.

FT. RANDALL RESERVOIR
MISSOURI RIVER BASIN
SOUTH DAKOTA
GRAPHIC SUMMARY
DIRECT SHEAR TESTS
RIGHT TERRACE CLAY
U. S. ENGINEER OFFICE OMAHA, NEBR. APRIL 1946

EMBANKMENT CRITERIA AND PERFORMANCE REPORT   PLATE   A29

STRENGTH DETERMINED FROM
MAXIMUM SHEAR VALUES

ADOPTED VALUES
C = 0.4 T/SQ FT.
TAN ϕ = 0.22

HORIZONTAL LOAD IN TONS PER SQ. FT.

VERTICAL LOAD IN TONS PER SQ. FT.

FT. RANDALL RESERVOIR
MISSOURI RIVER BASIN
SOUTH DAKOTA
GRAPHIC SUMMARY
DIRECT SHEAR TESTS
RIGHT TERRACE CLAY
U.S. ENGINEER OFFICE OMAHA, NEBR. APRIL 1948.

MICROCOPY RESOLUTION TEST CHART

NATIONAL BUREAU OF STANDARDS-1963-A

VOID RATIO

STRESS - TONS / SQ FT

Major Stress
at Failure

Minor Stress

#1 #2 #3

5   4   3   2   1

5   10   15   20
STRAIN - PER CENT

| C | SILT | SAND | GRAVEL |

CUMULATIVE PER CENT FINER

100
90
80
70
60
50
40
30
20
10
0

GRAIN SIZE IN MILLIMETERS AND SIEVE NUMBER

Liquid Limit --------------- 75
Plastic Limit -------------- 34
Plasticity Index ----------- 39
Flow Index ----------------- 18
Shrinkage Limit ------------ 16.9%
Shrinkage Ratio ------------ 1.865
Linear Shrinkage ----------- 20%
Field Moisture Equiv. ------ 46%
Volumetric Change from Air Moisture ---- 9%
USPR Classification -------- A-7

RIGHT TERRACE CLAY
TRIAXIAL
COMPRESSION TEST

U S ENGINEER SOILS LABORATORY

U S ENGINEER OFFICE
MISSOURI RIVER DIVISION
KANSAS CITY, MO

EMBANKMENT CRITERIA AND PERFORMANCE REPORT

PLATE A31
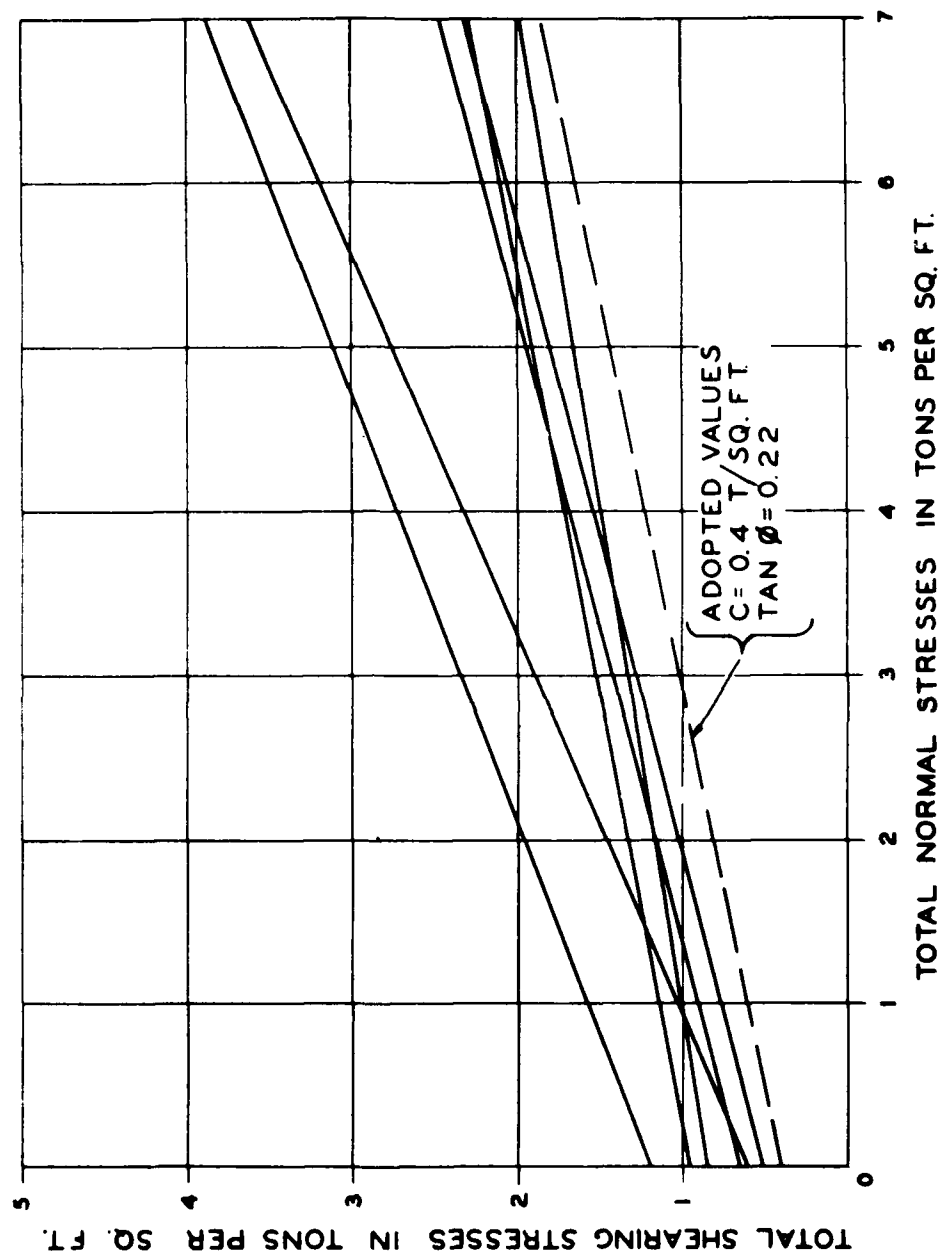
FT. RANDALL RESERVOIR
MISSOURI RIVER BASIN
SOUTH DAKOTA
GRAPHIC SUMMARY
TRIAXIAL COMPRESSION TESTS
RIGHT TERRACE CLAY

U. S. ENGINEER OFFICE OMAHA, NEBR. APRIL 1948.

Liquid Limit --------43
Plastic Limit ------26
Plasticity Index --17
Shrinkage Limit --19.4
Shrinkage Ratio -- 1.770
Linear Shrinkage --11 %
Volumetric Change
Liquid Limit --------42 %
U.S.R.R.Classification A-7

WEIGHT IN POUNDS PER CUBIC FOOT

PER CENT VOIDS

MOISTURE CONTENT - PER CENT OF DRY WEIGHT

TOTAL WEIGHT VOIDS
WET
DRY WEIGHT
AIR
VOIDS

CUMULATIVE PER CENT FINER

C	SILT	SAND	GRAVEL

A---Composite Sample
B---Optimum Sample

GRAIN SIZE IN MILLIMETERS AND SIEVE NUMBER

Test No ___238___   Specific Gravity___2.70___

Liquid Limit_____   Plastic Limit_____
STANDARD PROCTOR METHOD
Sample compacted in___1.5___inch layers by___25___blows
of a___5.5___lb hammer dropped___12___inches.
Diameter of Cylinder___4___inches
Source of Sample___Composite of samples 1 to 1C
Depth 0 to 25.' Hole C42. Elevation-
1434.9

Project___Fort Randall Damsite

Stream___Missouri River

GLACIAL TILL
OPTIMUM MOISTURE TEST

U. S. ENGINEER SOILS LABORATORY

U. S. ENGINEER OFFICE
MISSOURI RIVER DIVISION
KANSAS CITY. MO.

EMBANKMENT CRITERIA AND PERFORMANCE REPORT   PLATE A33

STRESS STRAIN CURVES

HORIZONTAL LOAD-TONS / SQ. FT

HORIZONTAL MOVEMENT IN INCHES VERTICAL

SHEARING STRENGTH

HORIZONTAL LOAD-TONS/SQ. FT.

VERTICAL LOAD-TONS / SQ FT.
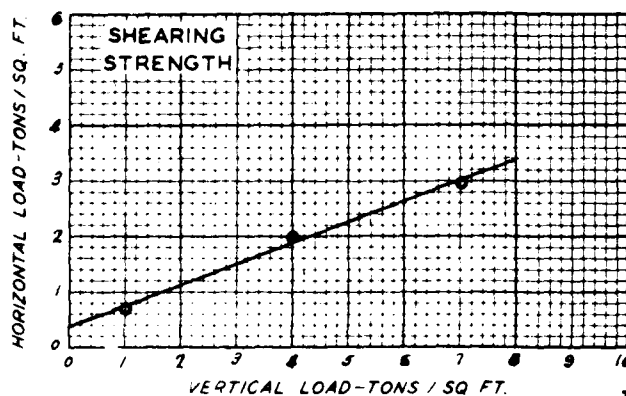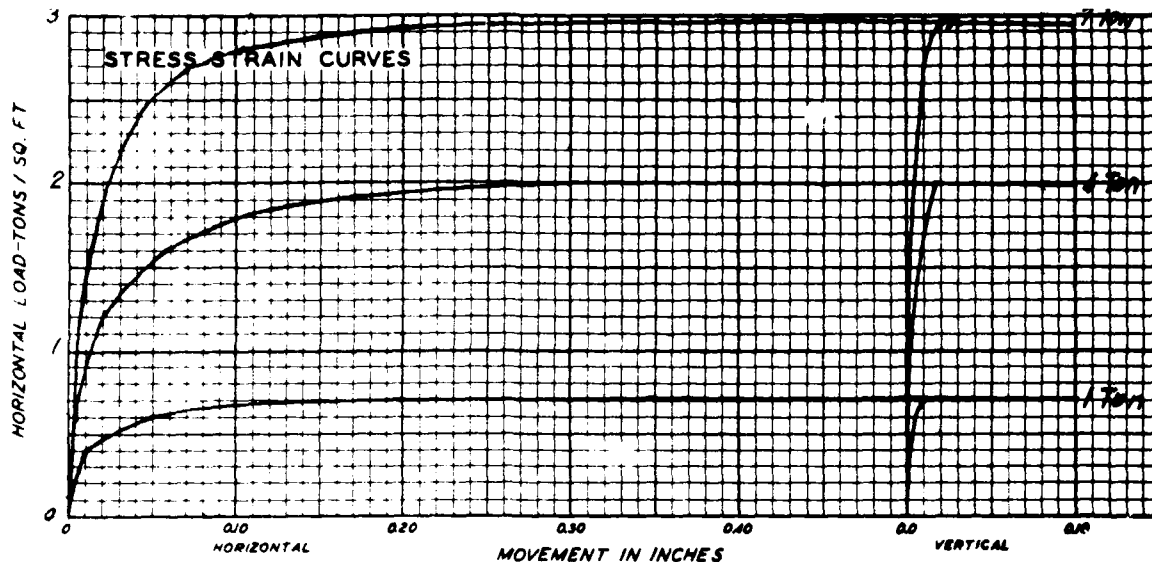
SHEARING STRENGTH
c = 0.40   tan ∅ = 0.37
Tests preconsolidated at vertical load.
Strain applied by motor drive:
    Moisture, before test --------20.9 %
    Voids, before test ---------41.9 %
    Dry Density -------98 lbs/cu. ft.
    Specific Gravity ------------ 2.70
    Liquid Limit ---------------43
    Plastic Limit ------------26
    Plasticity Index -----------17
    Shrinkage Limit ------------19.4 %
    Shrinkage Ratio ------------- 1.775
    Lineal Shrinkage ------------11 %
Volumetric Change from Liquid Limit-42 %
U. S. P. R.   Classification -----A-7

CLAY | SILT | SAND
MECHANICAL ANALYSIS

CUMULATIVE PER CENT FINER

1, 4, & 7 Ton

EMBANKMENT CRITERIA AND PERFORMANCE REPORT
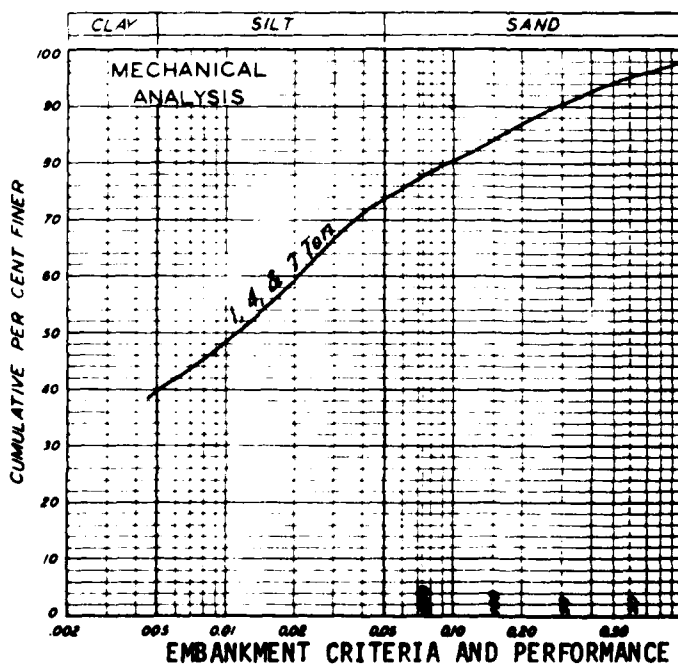
Project Fort Randall Damsite
Stream Missouri River
Hole No. C 42
Location
Ground Elev. 1434.9
Sample No. Composite of Samples
          1 to 10
Depth 0-25.'
Remarks Samples remolded.

RECOMPACTED
GLACIAL TILL
DIRECT SHEAR TEST

U.S. ENGINEER SOILS LABORATORY
U.S. ENGINEER OFFICE
MISSOURI RIVER DIVISION
KANSAS CITY, MO.

PLATE A34
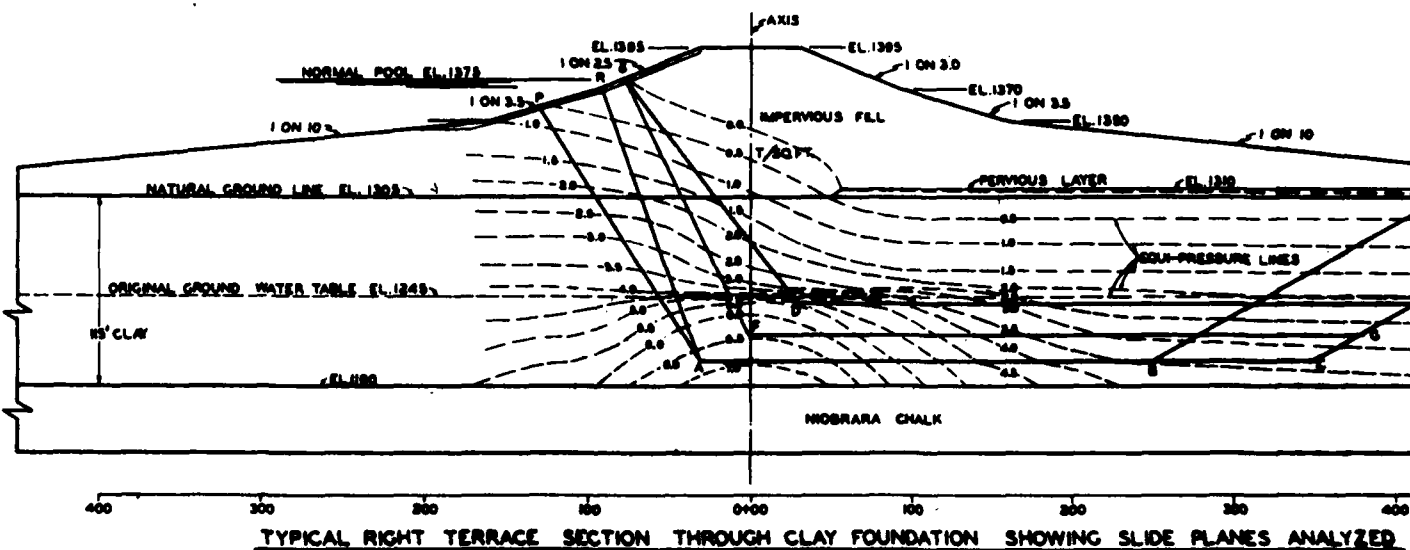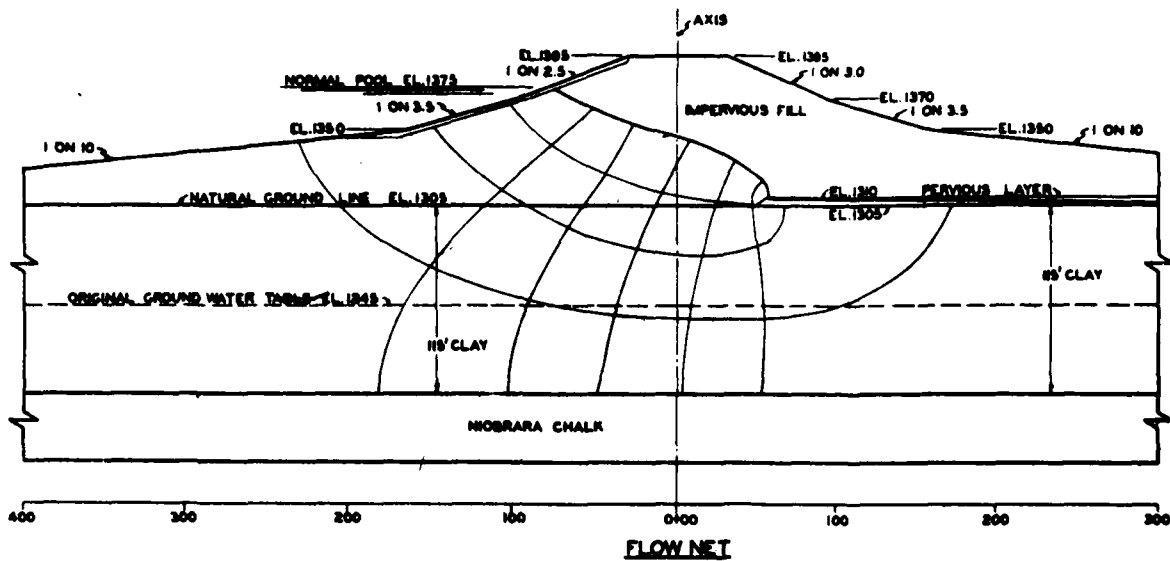
FLOW NET

TYPICAL RIGHT TERRACE SECTION THROUGH CLAY FOUNDATION SHOWING SLIDE PLANES ANALYZED

NOTES:
EQUI-PRESSURE LINES ARE OBTAINED FROM FLOW NETS.

DESIGN ASSUMPTIONS

MATERIAL	DENSITY — TONS/CU.FT		BEARING STRENGTH	
	AS PLACED	SATURATED	C - TONS/SQ.FT	TAN φ
ROLLED IMPERVIOUS FILL	.062	.064	0.35	0.35
CLAY FOUNDATION	IN PLACE .057	IN PLACE .058	0.40	0.22
DRAIN (PERVIOUS)	.058	.061	0	0.60

TRIAL	FACTOR OF SAFETY
PASK	
PACL	
RACL	
● SDEL	
SFGL	

● CRITICAL SECTION

EMBANKMENT

1

SAFETY FACTOR = $\frac{TAN \phi}{TAN \phi}$ = $\frac{c'}{c}$

C' & TAN ϕ = VALUES USED TO CLOSE POLYGON
C & TAN ϕ = DESIGN VALUES

0+30 UPSTREAM

EL.1395 — TOP OF DAM
SATURATION LINE — IMPERVIOUS ROLLED FILL
EFFECT. SOIL PRESSURE — NATURAL GROUND
ORIGINAL W.T. — IMPERVIOUS CLAY FOUNDATION
PORE WATER PRESSURE
//= NIOBRARA CHALK //=

0 2 4 6 8 10 12
PRESSURE IN TONS PER SQ. FT.
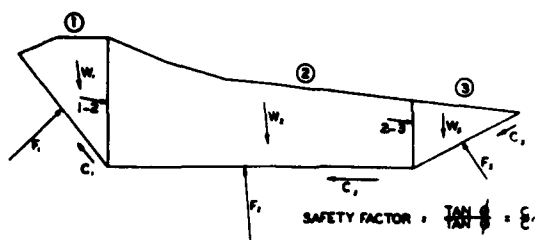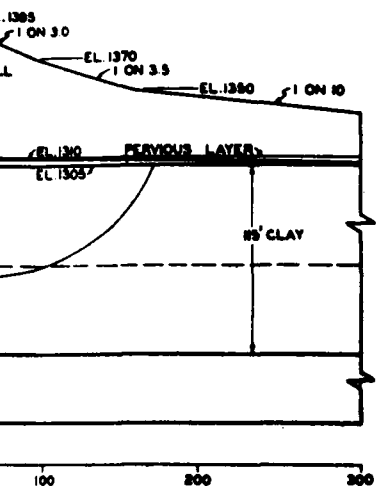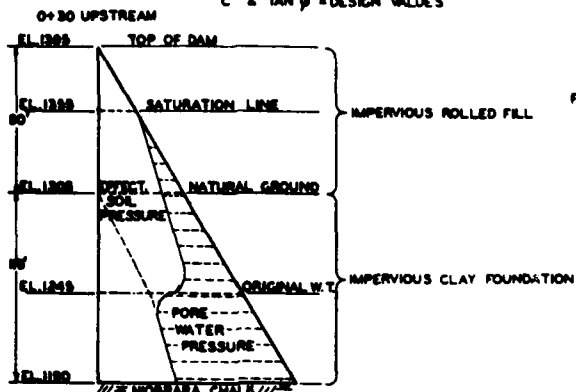
PORE PRESSURE DIAGRAM
SECTION THRU DAM AND FOUNDATION
THIRTY FEET UPSTREAM
FROM AXIS OF THE DAM

200   0   200
SCALE IN TONS

POLYGON OF FORCES
TRIAL SDEL
SAFETY FACTOR 2.2

EL.1385
1 ON 3.0
EL.1370
1 ON 3.5
EL.1380   1 ON 10
EL.1310
PERVIOUS LAYER
EL.1305
115' CLAY

100   200   300

1385
1 ON 3.0
EL.1370
1 ON 3.5
EL.1380   1 ON 10
PERVIOUS LAYER   EL.1310
K   L
SOIL-PRESSURE LINES
115' CLAY
A CHALK

100   200   300   400   500

FOUNDATION SHOWING SLIDE PLANES ANALYZED

TRIAL	FACTOR OF SAFETY
PASK	2.1
PACL	2.0
RACL	2.2
⊕ SDEL	2.2
SPSL	2.5

⊕ CRITICAL SECTION

	BEARING STRENGTH	
	C-TONS/SQ.FT	TAN ϕ
	0.35	0.36
	0.60	0.22
	0	0.60

FORT RANDALL RESERVOIR
MISSOURI RIVER BASIN
SOUTH DAKOTA
SLIDE ANALYSIS
RIGHT TERRACE SECTION
CLAY FOUNDATION LAYER
U.S. ENGINEER OFFICE   OMAHA, NEBR.   APRIL 1948.

FLOW NET

AXIS

NORMAL POOL EL. 1373

1 ON 2.5

EL. 1395
1 ON 2.0

EL. 1373

EL. 1397

IMPERVIOUS FILL

EL. 1370

1 ON 3.5

1 ON 3.5

EL. 1338

EL. 1331 SATURATION LINE

EL. 1309

1 ON 7

EL. 1320

EL. 1320

EL. 1305

ORIGINAL GROUND EL. 1345

75'

VERY FINE TO COARSE SAND

CARLILE SHALE

CODELL SANDSTONE

400 300 200 100 0+00 100 200 300

FLOW NET

TYPICAL VALLEY SECTION

AXIS

NORMAL POOL EL. 1373

1 ON 2.5   C

EL. 1395
1 ON 2.0

A

EL. 1370

1 ON 3.5

IMPERVIOUS FILL

1 ON 3.5

1 ON 7

EL. 1330

EL. 1320

1 ON 7

ORIGINAL GROUND EL. 1345

0 T/SQ. FT.
0.5 T/SQ. FT.
1.0 T/SQ. FT.
1.5 T/SQ. FT.
2.0 T/SQ. FT.
2.5 T/SQ. FT.
3.0 T/SQ. FT.
3.5 T/SQ. FT.

4.0 T/SQ. FT.

ORIGINAL GROUND EL. 1245

EQUI-PRESSURE LINES

75'

VERY FINE TO COARSE SAND

20'

CARLILE SHALE

CODELL SANDSTONE

400 300 200 100 0+00 100 200 300 400

TYPICAL VALLEY SECTION THROUGH CARLILE SHALE SHOWING SLIDE PLANES ANALYZED

NOTES:
EQUI-PRESSURE LINES ARE OBTAINED FROM FLOW NETS.

DESIGN ASSUMPTIONS

MATERIAL	DENSITY TONS/CU.FT.		SHEARING STRENGTH	
	AS PLACED	SATURATED	C- TONS/SQ.FT.	TAN φ
ROLLED IMPERVIOUS FILL	.062	.064	0.35	0.35
PERVIOUS FOUNDATION	.062	.061	0.00	0.60

SAND-SHALE CONTACT-SHEARING STRENGTH: C=0 T/SQ. FT. TAN φ
AS SHOWN IN TABLE "SUMMARY OF RESULTS"

SUMMARY OF

CURVE	REQUIRED SAND-SHALE ACT TO SAFETY
	1.5
A-X	.116
B-X	.144
C-X	.164
C-Y	.168
B-Y	.182
C-Z	.188
D-Y	.182

* CRITICAL SECTION
ASSUMED COHESION

1

EMBANKMENT

EL.1370

I ON 3.5

EL.1320

EL.1305

100   200   300
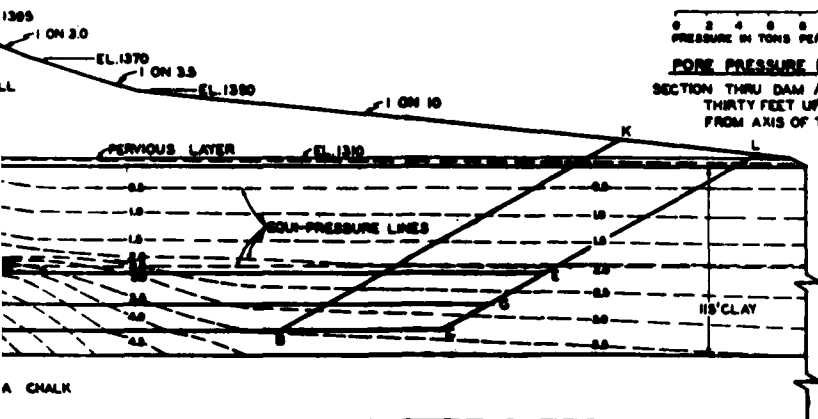
SAFETY FACTOR = $\frac{TAN \phi}{TAN \phi} = \frac{C}{C'}$

C & TAN ϕ = VALUES USED TO CLOSE POLYGON.
C & TAN ϕ = DESIGN VALUES.

EL.1305   AXIS OF DAM

150'

SOIL PRESSURE

EL.1300   SATURATION LINE

IMPERVIOUS ROLLED FILL

EL.1245   ORIG. GRD.

75'   PORE WATER PRESSURE

PERVIOUS (NATURAL)

EL.1170

0 2 4 6 8 10 12 14
PRESSURE IN TONS SQ. FT.

PORE PRESSURE DIAGRAM
SECTION THRU DAM AND FOUNDATION
ALONG AXIS OF DAM

POLYGON OF FORCES
CURVE C-Y
SAFETY FACTOR 2.0

400   0   400
SCALE IN TONS

EL.1370

I ON 3.5

EL.1320   I ON 7

.9 T/SQ.FT
.94 T/SQ.FT
1.8 T/SQ.FT
1.9 T/SQ.FT   ORIGINAL GROUND EL.1245   TAILWATER EL.1240
2.0 T/SQ.FT   EQUI-PRESSURE LINES   X   Y   Z
2.5 T/SQ.FT
3.0 T/SQ.FT   VERY FINE TO COARSE SAND
3.5 T/SQ.FT

CARLILE SHALE
CODELL SANDSTONE

100   200   300   400   500   600   700   800

SCALE SHOWING SLIDE PLANES ANALYZED

	SHEARING STRENGTH	
	C - TONS/SQ FT	TAN ϕ
	0.35	0.35
	0.00	0.00

NOTE: C=0 T/SQ FT. TAN ϕ
"SUMMARY OF RESULTS"

CURVE	REQUIRED TAN ϕ AT SAND-SHALE CONTACT TO PROVIDE A SAFETY FACTOR OF	
	1.5	2.0
A-X	.118	.232
B-X	.144	.300
C-X	.164	.340
C-Y	.199	.358 •
B-Y	.182	.323
C-Z	.188	.356 •
D-Y	.182	.356

SUMMARY OF RESULTS

• CRITICAL SECTION
ASSUMED COHESION CONTACT LAYER C=0

FORT RANDALL RESERVOIR
MISSOURI RIVER BASIN
SOUTH DAKOTA
SLIDE ANALYSIS
VALLEY SECTION
SAND-SHALE CONTACT
U.S. ENGINEER OFFICE   OMAHA, NEBR.   APRIL 1948

DISTANCE FROM CENTERLINE IN FEET

ELEVATION M.S.L.

VALLEY SECTION-STA 85+00

**STRESSES AND POINT
AT SAND-SHALE**

STRESSES — TONS/SQ.FT.

NORMAL STRESS

POINT SAFETY FACTOR
ASSUMING BASE OF INFINITE
DEPTH

POINT SAFETY FACTOR ASSUMING RIGI

HORIZONTAL SHEAR ASSUMING RIGI

HORIZONTAL SHEAR ASSUMING LAYER OF

DISTANCE FROM CEN

DESIGN ASSUMPTIONS

MATERIAL	DENSITY TONS/CU FT		SHEARING STRENGTH	
	AS PLACED	SATURATED	C TONS/SF	TAN Ø
ROLLED FILL	.062	.064	0.35	0.35
FOUNDATION SAND	IN PLACE .052	.061	0	0.6
SAND-SHALE CONTACT			0.1	0.3

EMBANKMENT CRIT

DISTANCE FROM CENTERLINE IN FEET

200 100 100 200 300 400 500 600 700 800 900 1000

VALLEY SECTION-STA 85+00

EL. 1370   EL. 1370   EL. 1320

IMPERVIOUS FILL

VALLEY SAND   POINTS FOR STRESS DETERMINATIONS

75'   +20   +21   +22   +23   +24   +25   +26   +27
20'   CARLILE SHALE
25'   CODELL SANDSTONE MEMBER
      CARLILE SHALE

ELEVATION M.S.L.
1400
1300
1200
1100

STRESSES AND POINT SAFETY FACTORS
AT SAND - SHALE CONTACT

STRESSES — TONS/SQ.FT.
14 12 10 8 6 4 2 0

NORMAL STRESS

POINT SAFETY FACTOR
ASSUMING BASE OF INFINITE
DEPTH

POINT SAFETY FACTOR ASSUMING RIGID BOUNDARY

HORIZONTAL SHEAR ASSUMING RIGID BOUNDARY AT SHALE CONTACT

HORIZONTAL SHEAR ASSUMING LAYER OF INFINITE DEPTH

0 100 200 300 400 500 600 700 800 900
DISTANCE FROM CENTERLINE IN FEET

POINT SAFETY FACTOR = $\dfrac{N_z \tan \phi + C}{S_{xz}}$

7 6 5 4 3 2 1
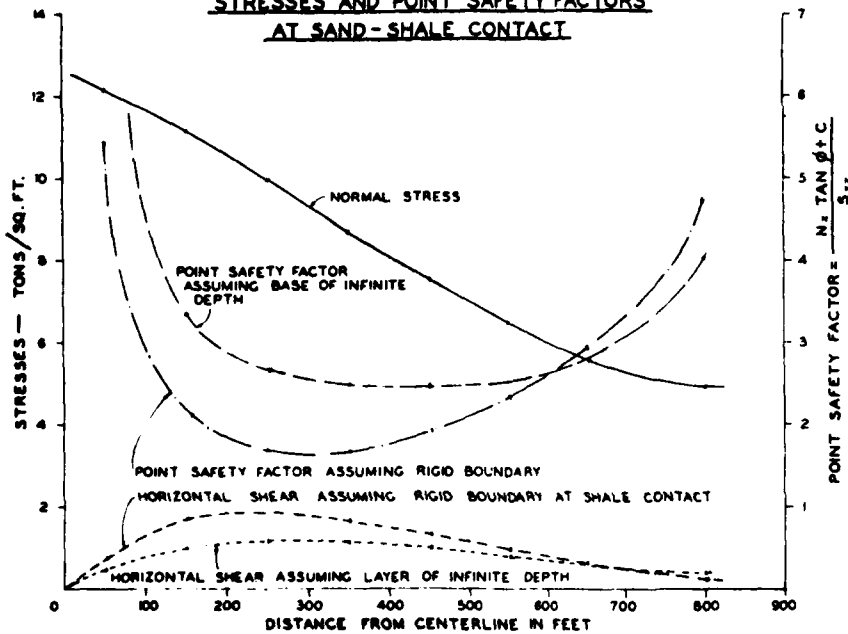
FORT RANDALL RESERVOIR
MISSOURI RIVER BASIN
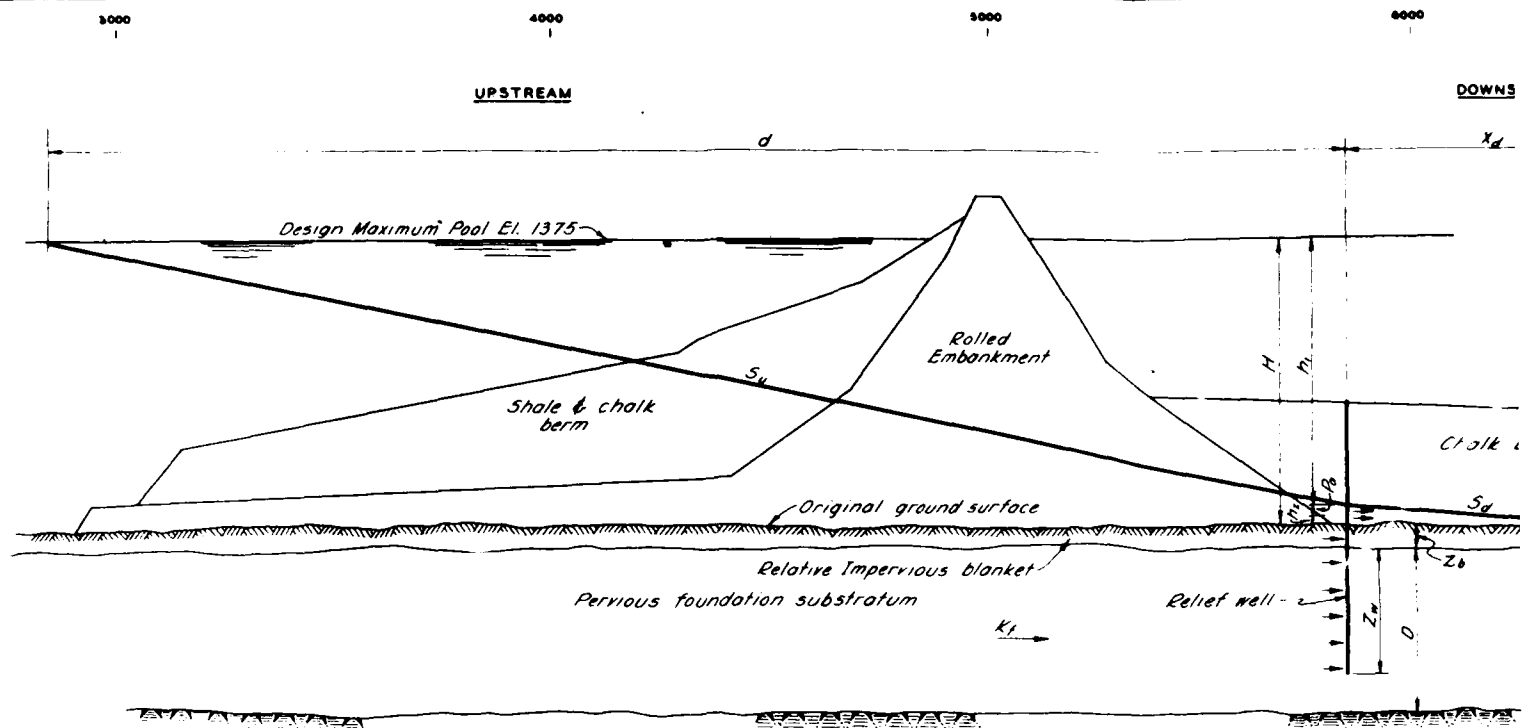SOUTH DAKOTA
STABILITY ANALYSIS
VALLEY SAND-SHALE CONTACT
ELASTIC THEORY METHOD
U.S. ENGINEER OFFICE OMAHA NEBR   APRIL 1946

EMBANKMENT CRITERIA AND PERFORMANCE REPORT   PLATE A37

2

SUMMARY OF RELIEF WELL SPACING & DISCHARGE COMPUTATIONS

	Stations				
	61+50 to 65+00	65+00 to 75+00	75+00 to 81+00	81+00 to 85+00	85+00 to 95+00
Orig. G.S.Elev.	1253	1235	1238	1247	1248
Ave. Bedrock Elev.	1170	1165	1080	1100	1160
Ave. Well Bottom Elev.	1170	1165	1150	1150	1160
Total Head, $H^{(1)}$	122	140	137	126	127
Thickness of Substratum, D	78	65	153	142	83
Depth of Well, Z_w	Full.Pene.	Full Pene.	83	92	Full Pene.
Well Spacing, a	120	100	70	80	100
Upstream Head Loss, h_1	118.0	136.3	131.6	123.2	123.4
Upstream Gradient, S_r	.0369	.0426	.0411	.0385	.0386
Downstream Gradient, S_1	.0100	.0093	.0135	.0120	.0090
Net Gradient, S	.0269	.0333	.0276	.0265	.0286
Well Discharge, Q_w(cfs)	.50	.43	.59	.60	.47
Head Loss out of Screen, h_x	2.06	1.85	2.30	2.32	1.97
Mean potential, P_a	1.87	1.83	3.14	2.46	1.57
Midpoint potential P_m	2.23	2.20	1.95	1.92	1.88
Total uplift, $P_m + h_x$	4.29	4.05	4.25	4.24	3.85

Notes:
1. See Plate A-39 for definition of symbols
2. Midpoint potential equals $P_a + h_3$ for full penetration well.
3. Upstream Resistance d = 3200
4. Downstream Resistance x_3 = 400'
5. Radius of Relief Well - 0.5'
6. Design pool elevation - 1375 msl.

UPSTREAM

d

X_d

Design Maximum Pool El. 1375

Rolled Embankment

H

h_1

Shale & chalk berm

S_u

Chalk

Original ground surface

S_d

Relative Impervious blanket

Z_b

Pervious foundation substratum

Relief well

Z_w

K_f

D

TYPICAL SECTION ILLUSTRATING NOMENCLATURE

TYPICAL COMPUTATION FOR FULLY PENETRATING WELL
STA. 65+00 TO STA. 75+00

DESIGN DATA

Average original ground surface elevation = 1235.
Average bedrock elevation = 1165.
H = 140 ft
D = 65 ft
Full penetrating well formula = $P_a = \left(\frac{a}{2\pi} \log_e \frac{a}{2\pi r_w}\right) S = F.S$

Assume well spacing "a" and value for $P_a + h_x$ and compute total uplift by trial and error. Total uplift $(P_a + h_x + h_3)$ ≦ Allowable uplift.

Assume $a = 100$

		1ST Trial	2ND Trial
$P_a + h_x$ (Assumed)	=	3.2	3.7
h_1	=	136.8	136.3
S_r	=	.0428	.0426
S_t	=	.0080	.0093
S	=	.0348	.0333
Q_w (in cfs)	=	.45	.43
h_x	=	1.95	1.89
$P_a = F.S = .555$	=	1.91	1.83
$P_a + h_x$	=	3.86	3.72
h_3	=	.38	.37
$h_x + P_a + h_3$	=	4.24	4.09

The computed value for $P_a + h_x$ corresponds to the assumed values in the second trial and the total computed potential, $h_x + P_a + h_3$ is slightly less than the allowable value, therefore the well spacing of 100 ft. is adequate.

TYPICAL COMPUTATION FOR PARTIALLY PENETRATING WELL
STA. 75+00 TO STA. 81+00

DESIGN DATA

Average ground surface elevation = 1238
Average bedrock elevation = 1080
Bottom of well elevation = 1150
H = 137 ft
D = 153 ft – Transformed D' = 612 ft
Z_w = 83 ft – Transformed Z'_w = 332 ft

Mean potential formula for partially penetrating wells

$$P_a = \left[\frac{aD'}{2\pi Z_w} \log_e \frac{a}{2^{(1-(X)-\frac{z_w}{D'})}\pi r_w}\right] S$$

Mid-point potential formula for partially penetrating wells

$$P_w = \left[\frac{aD'}{2\pi Z_w} \log_e \frac{a}{2^{(1-\frac{z_w}{D})}\pi r_w}\right] S$$

Assume well spacing "a" and value for $P_a + h_x$ and compute total uplift, using the transformed depths and permeability, by trial and error. Total uplift $P_w + h_x$ ≦ allowable uplift.

		1ST Trial	2ND Trial	3RD Trial
a (Assumed)	=	100	100	70
$P_a + h_x$ (Assumed)	=	6.0	6.4	5.4
h_1	=	131.0	130.6	131.6
S_r	=	.0409	.0408	.0411
S_t	=	.0150	.0160	.0135
S	=	.0259	.0248	.0276
Q_w	=	.79	.76	.59
h_x	=	2.79	2.71	2.30
P_a	=	3.83	3.70	3.14
$P_a + h_x$	=	6.62	6.41	5.44
P_w	=		2.80	1.95
$P_w + h_x$	=		5.51	4.25

The computed value for $P_a + h_x$ corresponds to the assumed value in the second trial however the total computed potential, $P_w + h_x$, is greater than the allowable value, therefore the spacing of 100 ft is inadequate. The computed value for $P_a + h_x$ corresponds to the assumed value in the third trial and the total computed potential $P_w + h_x$ is approximately equal to the allowable value, therefore the spacing of 70 ft is adequate.
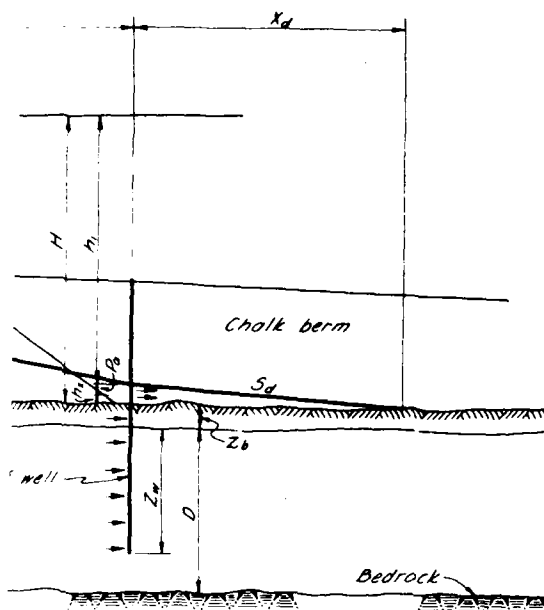
DOWNSTREAM

NOMENCLATURE

Z_b = Thickness of relatively impervious downstream blanket.
D = Thickness of pervious substratum.
Z_w = Depth of proposed relief well.
H = Total head.
P_a = Mean potential over plane of wells.
P_w = Surface potential at mid-point between partially penetrating wells.
h_x = Head loss due to flow out of well screen into chalk berm.
h_1 = $H-(P_a+h_x)$ = Total headloss from source to line of wells.
d = Effective resistance upstream of line of wells
X_d = Effective resistance downstream of line of wells.
S = $S_u - S_d$ = Net potential gradient producing discharge from wells.
a = Well spacing.
h_3 = $0.11 a S$ = Potential midway between fully penetrating wells in excess of the average potential over the plane
k_f = Horizontal permeability of pervious substratum.
Q_w = $k_f D a S$ = Discharge of well.
r_w = Radius of proposed well.

GENERAL DESIGN ASSUMPTIONS & DATA

1. $d = 3200$ ft. All gradients are approximately equal.
2. $X_d = 400$ ft as indicated by effective radius of well by pumping test.
3. $k_f = .002$ c f s = Horizontal permeability of previous substratum.
4. Ratio of horizontal permeability to vertical permeability of foundation material is in the order of 16 to 1.
5. In the case of partial penetrating wells the depth of the well and depth of the substratum is transformed in accordance with the item 4 above or $\sqrt{k_h/k_v} = \sqrt{16/1} = 4$
6. Weight of saturated blanket material = 115 lb/cu ft therefore the allowable uplift equals 0.84 Z_b for factor of safety of 1.0
7. Assume uniform average downstream blanket thickness of 5 feet across entire valley. Therefore allowable uplift = 0.84 x 5 = 4.2 ft at all locations.
8. Head loss for flow out of well screen = $h_x = (5.89 Q_w)\frac{1}{8}$
9. Design maximum pool water surface elevation = 1375.
10. Assume radius of well - $r_w = 0.5$ ft. Actual inside diameter of well screen is only 8 inches however it is assumed the effective radius is increased due to the gravel pack.
11. All wells with exception of those in deepest channel are fully penetrating. Those in the deep channel penetrate to a minimum elevation of 1150.

LLY PENETRATING WELL

81 + 00

= 1238
= 1080.
= 1150.

y penetrating wells.

] 5

rtially penetrating wells

for $P_a + h_x$ and compute
depths and perme-
uplift $P_w + h_x \leqq$ allow-

| uo Trial	3ᵃᵒ Trial
100	70
6.4	5.4
130.6	131.6
.0408	.0411
.0160	.0135
.0248	.0276
.76	.59
2.71	2.30
3.70	3.14
6.41	5.44
2.80	1.95
5.51	4.25

sponds to the assumed
the total computed potential,
able value, therefore
iate. The computed value
ned value in the third trial
$P_w + h_x$ is approximately
efore the spacing of 70 ft.

MISSOURI RIVER
FORT RANDALL DAM - LAKE FRANCIS CASE
EMBANKMENT RELIEF WELL STUDY
TYPICAL WELL SPACING
COMPUTATIONS
OFFICE OF THE DISTRICT ENGINEER
OMAHA NEBRASKA   MAY 1953

MISSOURI RIVER
FORT RANDALL DAM-LAKE FRANCIS CASE
EMBANKMENT RELIEF WELL STUDY
GRADATION CURVES
OFFICE OF THE DISTRICT ENGINEER
OMAHA, NEBRASKA   MAY 1963
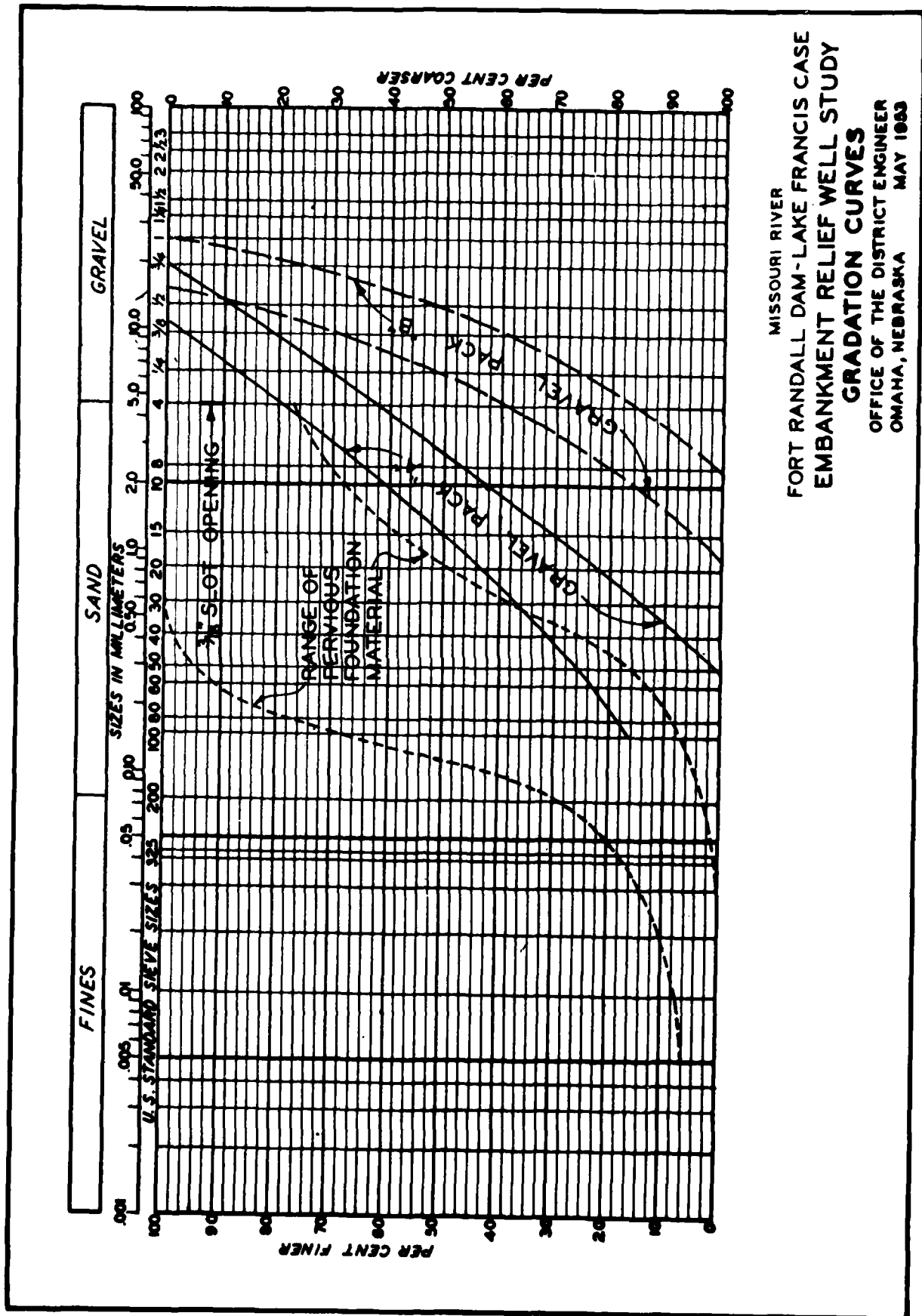
					FORT RANDALL DAM, PERMANENT RECORD SAMPLES (ROLLED EMBANKMENT)							
Box Sample No.	Station	Range	Elevation	Soil Classification	Mechanical Analysis			LL	PI	S. G.	M. C. %	Dry Density PCF
					GR. %	SA. %	FI. %					
A. Initial Earthwork:												
1	95+90	4337	1257.5	Sandy Clay	0	28	72			2.69	17.9	96.9
2	94+87	5218	1261.5	Sandy Clay	0	36	64			2.70	17.5	108.6
3	94+50	4955	1268.3	Sandy Clay	0	33	67			2.69	16.2	108.4
4	88+00	4890	1254.2	Sandy Clay	0	33	67			2.70	19.3	105.8
5	84+17	5250	1252.1	Sandy Clay	0	32	68			2.74	19.0	104.1
7	99+95	4595	1275.3	Sandy Clay	3	24	73			2.71	21.7	100.1
8	100+27	4978	1283.8	Sandy Clay	20	22	58			2.71	19.9	104.3
10	95+50	5200	1286.0	Clayey Sand	4	56	40			2.70	17.4	98.2
11	94+12	4798	1285.5	Clayey Sand	23	33	44			2.66	20.1	101.3
12	88+00	5142	1268.6	Clayey Sand	6	46	48			2.74	14.0	100.3
13	102+43	4925	1319.1	Sandy Clay	7	27	68			2.74	19.9	97.4
14	93+20	4936	1299.0	Sandy Clay	3	44	53			2.70	20.5	99.0
15	95+00	5075	1316.8	Sandy Clay	3	27	70			2.69	20.6	103.0
16	98+50	4900	1342.7	Sandy Clay	2	23	75			2.75	21.3	99.3
17	110+10	5025	1363.6	Sandy Clay	0	32	68			2.71	20.2	100.0
18	85+00	4675	1269.6	Sandy Clay	3	26	71			2.69	21.1	101.3
19	2+48	3+00	1336.5	Sandy Clay	3	22	75			2.70	20.8	100.3
20	110+05	4940	1365.7	Sandy Clay	0	22	78			2.68	21.8	100.6
B. Earthwork Stage II:												
1	90+00	4700	1281.2	Sandy Clay	7	18	75	47	26	2.67	19.8	93.0
2	87+45	4895	1279.8	Sandy Gr. Clay	23	20	57	44	22	2.72	19.2	99.0
3	84+00	4900	1282.5	Sandy Clay	3	39	58	40	19	2.66	19.8	100.0
5	86+00	5260	1280.7	Fat Clay	3	13	84			2.66	27.7	95.1
7	77+80	4750	1265.5	Sandy Clay	8	26	66	45	24	2.72	20.4	101.0
8	73+82	4620	1255.3	Sandy Clay	2	32	66	43	22	2.64	19.8	104.0
9	86+75	4687	1302.9	Sandy Clay	7	27	65	43	20	2.69	16.6	98.0
10	78+42	4909	1271.4	Sandy Clay	0	35	65	47	25	2.68	21.0	101.0
11	74+00	4746	1264.8	Sandy Clay	7	27	65	44	22	2.74	19.3	99.0
12	76+35	4947	1266.1	Sandy Clay	1	31	68	49	27	2.69	20.4	103.0
13	81+86	4893	1297.1	Sandy Clay	8	26	66	48	26	2.70	19.1	104.0
14	30+16	5330	1289.5	Fat Clay	0	15	85	64	38	2.65	25.3	96.0
15	75+51	4828	1299.7	Sandy Clay	6	31	63	46	25	2.66	19.3	99.0
16	78+00	4950	1308.2	Gr. Sandy Clay	17	28	55	48	28	2.67	19.5	108.0
17	82+30	5068	1314.0	Gr. Clay	33	16	51	70	41	2.72	26.6	93.0
C. Earthwork Stage III:												
1	90+00	4802	1315.4	Sandy Clay	13	24	63	38	18	2.67	21.0	101.0
2	95+00	4890	1322.9	Sandy Clay	2	24	74	42	20	2.70	18.7	96.0
3	92+82	5151	1317.7	Fat Clay	0	20	80	64	35	2.69	37.5	82.0
4	101+00	4479	1331.6	Sandy Clay	7	38	55	37	16	2.73	19.5	98.2
5	82+01	4775	1324.8	Sandy Clay	4	29	67	42	19	2.76	18.1	98.1
6	85+92	5012	1330.8	Sandy Clay	4	22	74	39	20	2.73	20.1	99.0
7	105+98	4962	1372.6	Sandy Clay	2	25	73	42	21	2.76	19.1	101.0
8	114+14	5000	1382.7	Sandy Clay	3	29	68	38	17	2.75	19.6	101.5
9	110+00	5025	1389.0	Sandy Clay	6	26	68	40	20	2.78	20.0	103.2
10	98+60	4891	1358.2	Sandy Clay	5	31	64	40	18	2.75	18.0	100.9
11	94+00	4905	1352.6	Sandy Clay	6	32	62	42	23	2.77	16.8	107.0
12	85+02	4867	1351.4	Sandy Clay	6	27	67	44	23	2.76	20.3	97.9
13	77+28	4899	1343.9	Sandy Clay	7	27	66	41	20	2.74	20.1	91.5
14	92+03	4909	1354.5	Sandy Clay	7	23	70	42	23	2.70	21.5	104.0
15	80+35	5043	1355.1	Sandy Clay	2	35	63	38	18	2.69	22.5	97.0
16	94+59	5109	1358.3	Sandy Clay	6	33	61	44	23	2.77	22.3	103.5
17	85+69	5061	1361.4	Sandy Clay	13	35	52	38	19	2.73	18.9	107.5
18	80+02	4896	1362.8	Sandy Clay	12	23	65	40	18	2.73	21.4	94.4
19	86+18	5030	1377.0	Sandy Clay	4	23	73	39	18	2.80	17.1	93.9
20	98+66	4979	1392.9	Sandy Clay	6	23	71	38	18	2.74	18.6	90.6
21	93+00	5000	1394.0	Sandy Clay	2	23	75	40	19	2.75	19.6	94.6
22	58+60	4425	1254.2	Silt	0	3	97	31	5	2.61	15.4	91.6
23	52+65	5260	1301.5	Silt	0	6	94	30	5	2.68	15.9	92.5
24	53+43	5177	1303.1	Fat Clay	0	11	89	75	48	2.79	24.8	89.8
D. Earthwork Stage IV:												
1	66+80	4985	1289.3	Fat Clay				60	34	2.71	23.1	87.0
2	69+12	4952	1300.3	Lean Clay				36	18	2.68	15.8	106.0
3	63+91	5024	1308.3	Lean Clay				31	13	2.66	17.0	109.0
4	64+95	5185	1290.0	Lean Clay				38	21	2.66	16.6	107.0
5	63+74	4968	1323.0	Lean Clay				37	19	2.73	18.7	109.0
6	61+00	4815	1338.0	Lean Clay				45	23	2.69	20.6	100.0
7	60+85	4897	1359.5	Lean Clay				43	23	2.72	19.6	104.0
E. Earthwork Stage V:												
2	150+35	20+16	1332.7	Sandy Clay	3	35	62	37	21	2.70	15.4	99.0
3	151+10	19+68	1360.0	Sandy Clay	5	33	62	32	16	2.71	16.8	106.0
4	40+00	5025	1355.0	Sandy Clay	5	30	65	45	27			
5	54+82	4984	1368.0	Fat Clay				68	38			
6	40+00	5125	1364.0	Lean Clay				44	23	2.64	20.9	99.0
7	15+40	5060	1385.0	Sandy Clay	0	27	73	45	26	2.68	18.4	80.0

FORT RANDALL DAM, PERMANENT RECORD SAMPLES (ROLLED EMBANKMENT)

Soil Classification	Mechanical Analysis			LL	PI	S. G.	M. C. %	Dry Density PCF	Void Ratio	Direct Shear		Unc. Comp. TSF			Perm. Coeff. CM/SEC
	GR. %	SA. %	FI %							COH TSF	Tan φ	1	2	3	
Sandy Clay	0	28	72			2.69	17.9	96.9							
Sandy Clay	0	36	64			2.70	17.5	108.6							
Sandy Clay	0	33	67			2.69	16.2	108.4							
Sandy Clay	0	33	67			2.70	19.3	105.8							
Sandy Clay	0	32	68			2.74	19.0	104.1							
Sandy Clay	3	24	73			2.71	21.7	100.1		0.70	0.51				
Sandy Clay	20	22	58			2.71	19.9	104.3		0.72	0.62				
Clayey Sand	4	56	40			2.70	17.4	98.2		0.20	0.73				
Clayey Sand	23	33	44			2.66	20.1	101.3		0.48	0.53				
Clayey Sand	6	46	48			2.74	14.0	100.3		0.20	0.65				
Sandy Clay	7	27	68			2.74	19.9	97.4		0.18	0.55				
Sandy Clay	3	44	53			2.70	20.5	99.0		0.35	0.51				
Sandy Clay	3	27	70			2.69	20.6	103.0		0.55	0.51				
Sandy Clay	2	23	75			2.75	21.3	99.3		0.45	0.57				
Sandy Clay	0	32	68			2.71	20.2	100.0		0.45	0.45				
Sandy Clay	3	26	71			2.69	21.1	101.3		0.45	0.45				
Sandy Clay	3	22	75			2.70	20.8	100.3		0.42	0.45				
Sandy Clay	0	22	78			2.68	21.8	100.6		0.40	0.57				
Sandy Clay	7	18	75	47	26	2.67	19.8	93.0	0.802	0.30	0.54				
Sandy Gr. Clay	23	20	57	44	22	2.72	19.2	99.0	0.710	0.62	0.49				
Sandy Clay	3	39	58	40	19	2.66	19.8	100.0	0.669	0.58	0.65				
Silt Clay	3	13	84			2.66	27.7	95.1							
Sandy Clay	8	26	66	45	24	2.72	20.4	101.0	0.676	0.42	0.57				
Sandy Clay	2	32	66	43	22	2.64	19.8	104.0	0.585	0.48	0.46				
Sandy Clay	7	27	65	43	20	2.69	16.6	98.0	0.721	0.25	0.56				
Sandy Clay	0	35	65	47	25	2.68	21.0	101.0	0.661	0.20	0.52				
Sandy Clay	7	27	65	44	22	2.74	19.3	99.0	0.738	0.40	0.52				
Sandy Clay	1	31	68	49	27	2.69	20.4	103.0	0.640	0.50	0.45	2.56			
Sandy Clay	8	26	66	48	26	2.70	19.1	104.0	0.630	0.60	0.55	2.30	3.42		
Silt Clay	0	15	85	64	38	2.65	25.3	96.0	0.732	0.65	0.42				
Sandy Clay	6	31	63	46	25	2.66	19.3	99.0	0.684	0.45	0.46				
Sandy Clay	17	28	55	48	28	2.67	19.5	108.0	0.548	0.63	0.45	1.54	1.48	1.74	
Clay	33	16	51	70	41	2.72	26.6	93.0	0.825	0.43	0.39				
Sandy Clay	3	24	63	38	18	2.67	21.0	101.0	0.658	0.23	0.50	0.83	0.87		8.5x10⁻⁹
Sandy Clay	2	24	74	42	20	2.70	18.7	96.0	0.764	0.22	0.50	1.58			2.3x10⁻⁶
Clay	0	20	80	64	35	2.69	37.5	82.0	1.049	0.20	0.36	0.59	0.58		1.7x10⁻⁸
Sandy Clay	7	38	55	37	16	2.73	19.5	98.2	0.732			2.03			
Sandy Clay	4	29	67	42	19	2.76	18.1	98.1				2.34	2.22	2.14	
Sandy Clay	4	22	74	39	20	2.73	20.1	99.0	0.724	0.02	0.60	1.53			
Sandy Clay	2	25	73	42	21	2.76	19.1	101.0	0.705			2.94			3.9x10⁻⁸
Sandy Clay	3	29	68	38	17	2.75	19.6	101.5				2.20			
Sandy Clay	6	26	68	40	20	2.78	20.0	103.2				2.12			
Sandy Clay	5	31	64	40	18	2.75	18.0	100.9				1.41			
Sandy Clay	6	32	62	42	23	2.77	16.8	107.0	0.621	0.60	0.50	1.77			
Sandy Clay	6	27	67	44	23	2.76	20.3	97.9	0.882			1.51			1.5x10⁻⁶
Sandy Clay	7	27	66	41	20	2.74	20.1	91.5				1.49			
Sandy Clay	7	23	70	42	23	2.70	21.5	104.0	0.597	0.55	0.40	2.06			
Sandy Clay	2	35	63	38	18	2.69	22.5	97.0	0.739	0.23	0.47	1.14			
Sandy Clay	6	33	61	44	23	2.77	22.3	103.5				1.55			
Sandy Clay	13	35	52	38	19	2.73	18.9	107.5		0.39	0.43	2.08			
Sandy Clay	12	23	65	40	18	2.73	21.4	94.4				1.13			8.6x10⁻⁸
Sandy Clay	4	23	73	39	18	2.80	17.1	93.9				1.58			
Sandy Clay	6	23	71	38	18	2.74	18.6	90.6				0.68			
Sandy Clay	2	23	75	40	19	2.75	19.6	94.6		0.10	0.52	1.51			
Clay	0	3	97	31	5	2.61	15.4	91.6				0.91			1.3x10⁻⁵
Clay	0	6	94	30	5	2.68	15.9	92.5	0.811	0	0.61	1.24	1.68		1.8x10⁻⁶
Clay	0	11	89	75	48	2.79	24.8	89.8	0.831			2.42	2.02		3.1x10⁻⁸
Clay				60	34	2.71	23.1	87.0	0.958	0.08	0.62				5.9x10⁻⁷
Clay				36	18	2.68	15.8	106.0				1.04			
Clay				31	13	2.66	17.0	109.0	0.547			1.24			1.1x10⁻⁶
Clay				38	21	2.66	16.6	107.0	0.554	0.35	0.58	2.55	1.97		
Clay				37	19	2.73	18.7	109.0				1.48			
Clay				45	23	2.69	20.6	100.0	0.689	0.18	0.58	2.38			
Clay				43	23	2.72	19.6	104.0	0.641	0.60	0.47	2.66			1.3x10⁻⁷
Clay	3	35	62	37	21	2.70	15.4	99.0	0.708	0.20	0.61	1.92			
Clay	5	33	62	32	16	2.71	16.8	106.0				1.22			1.18x10⁻⁷
Clay	5	30	65	45	27										
Clay				68	38										
Clay				44	23	2.64	20.9	99.0	1.400	0.15	0.58	2.30	2.18		
Clay	0	27	73	45	26	2.68	18.4	80.0	1.089	0.10	0.51	1.28			2.8x10⁻⁷

SUMMARY OF ATTERBERG LIMITS,
EMBANKMENT RECORD SAMPLES
FORT RANDALL DAM
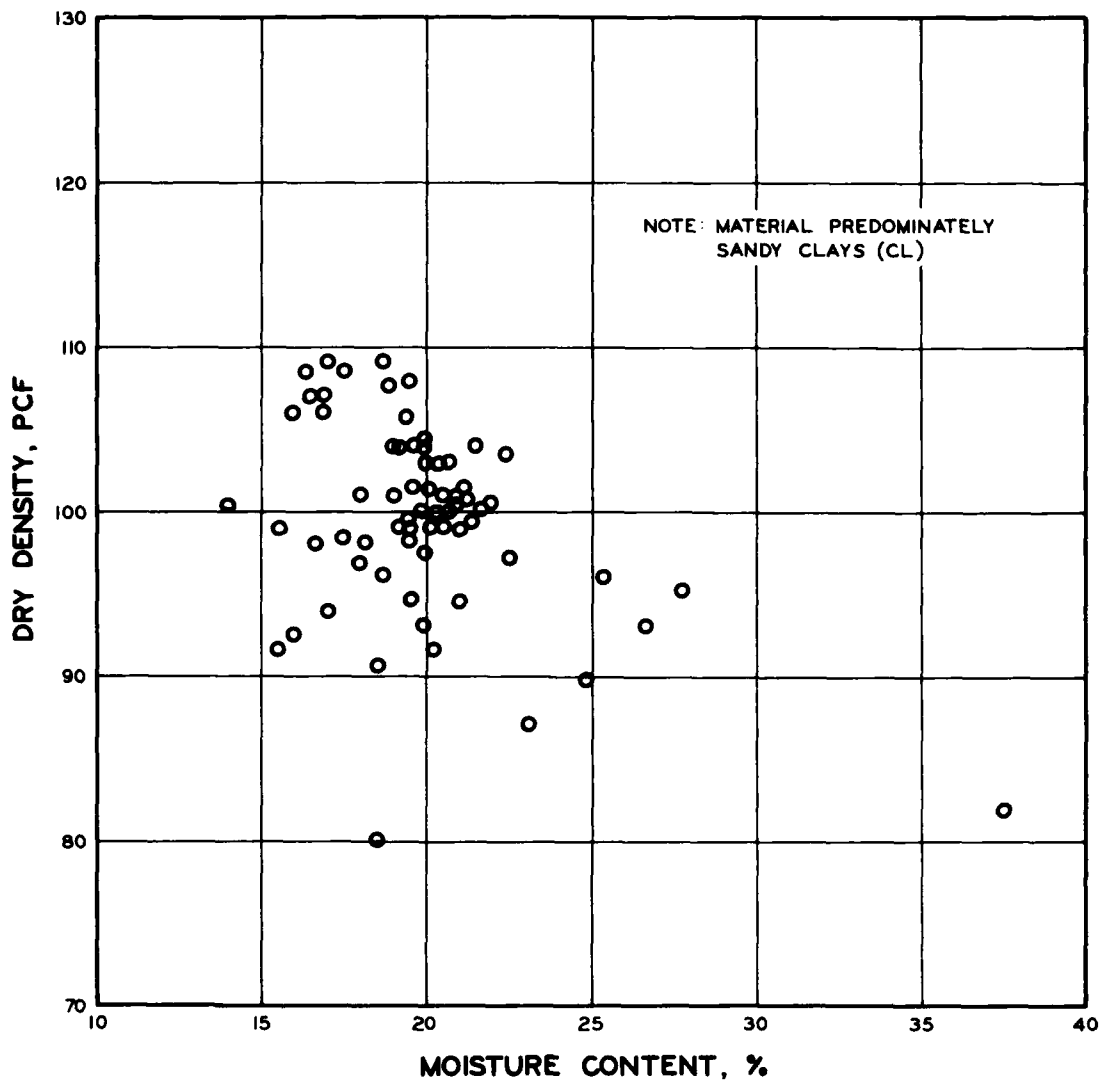
NOTE: MATERIAL PREDOMINATELY
SANDY CLAYS (CL)

SUMMARY OF MOISTURE CONTENTS AND
DRY DENSITIES, EMBANKMENT RECORD SAMPLES
FORT RANDALL DAM

NOTE: MATERIAL PREDOMINATELY
SANDY CLAYS (CL)

DRY DENSITY, PCF

MOISTURE CONTENT, %

SUMMARY OF MOISTURE CONTENTS AND
DRY DENSITIES, EMBANKMENT RECORD SAMPLES
FORT RANDALL DAM

UPPER AND LOWER STRENGTH ENVELOPES OF 44 DIRECT
SHEAR TESTS ON RECORD SAMPLES. (ALL TEST RESULTS
ARE NOT PLOTTED.)

UPPER LIMIT

LOWER LIMIT

DESIGN SHEAR STRENGTH
C = 0.35 TSF
TAN ϕ = 0.35

SHEAR STRENGTH, TSF

NORMAL STRESS, TSF
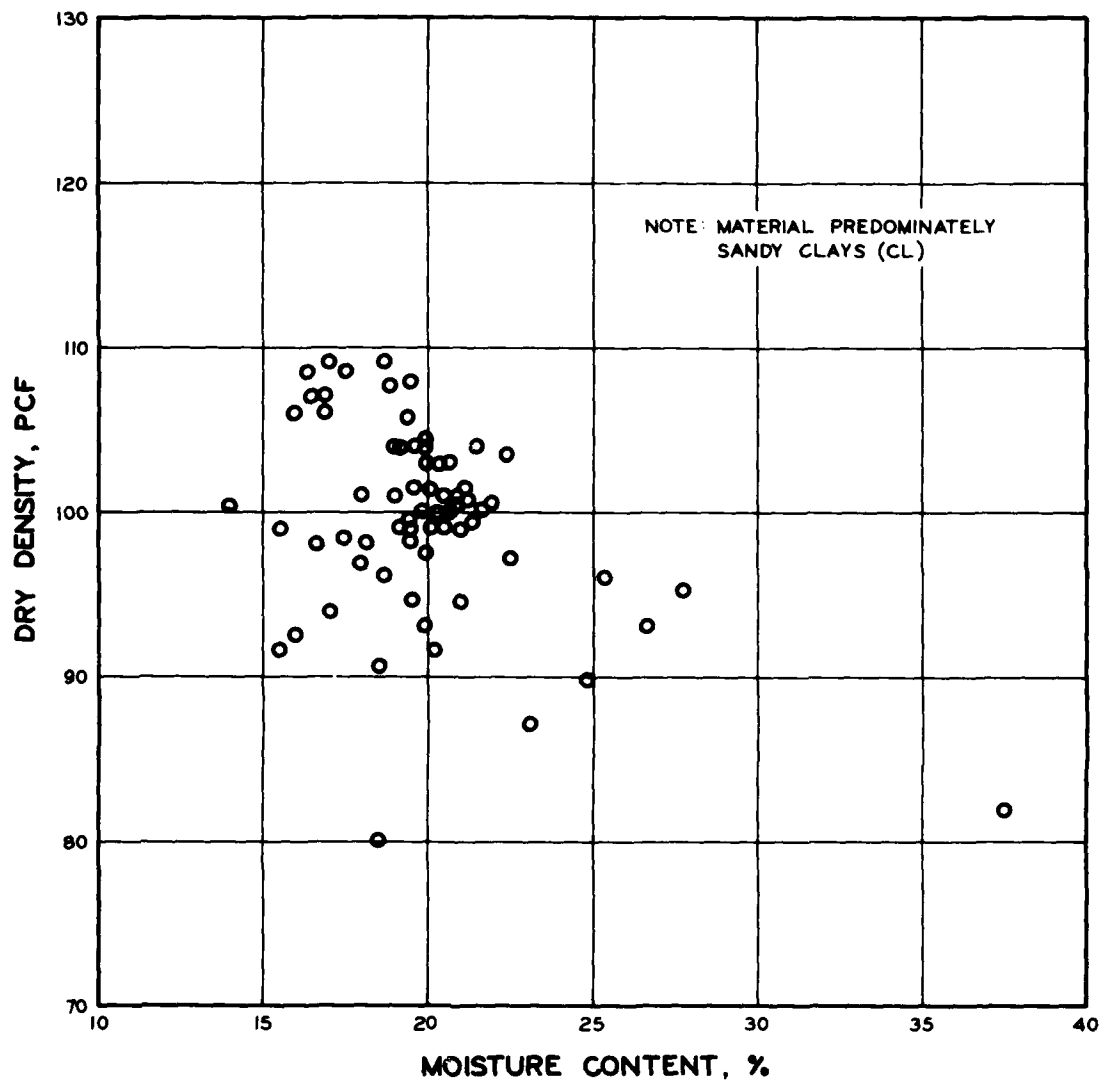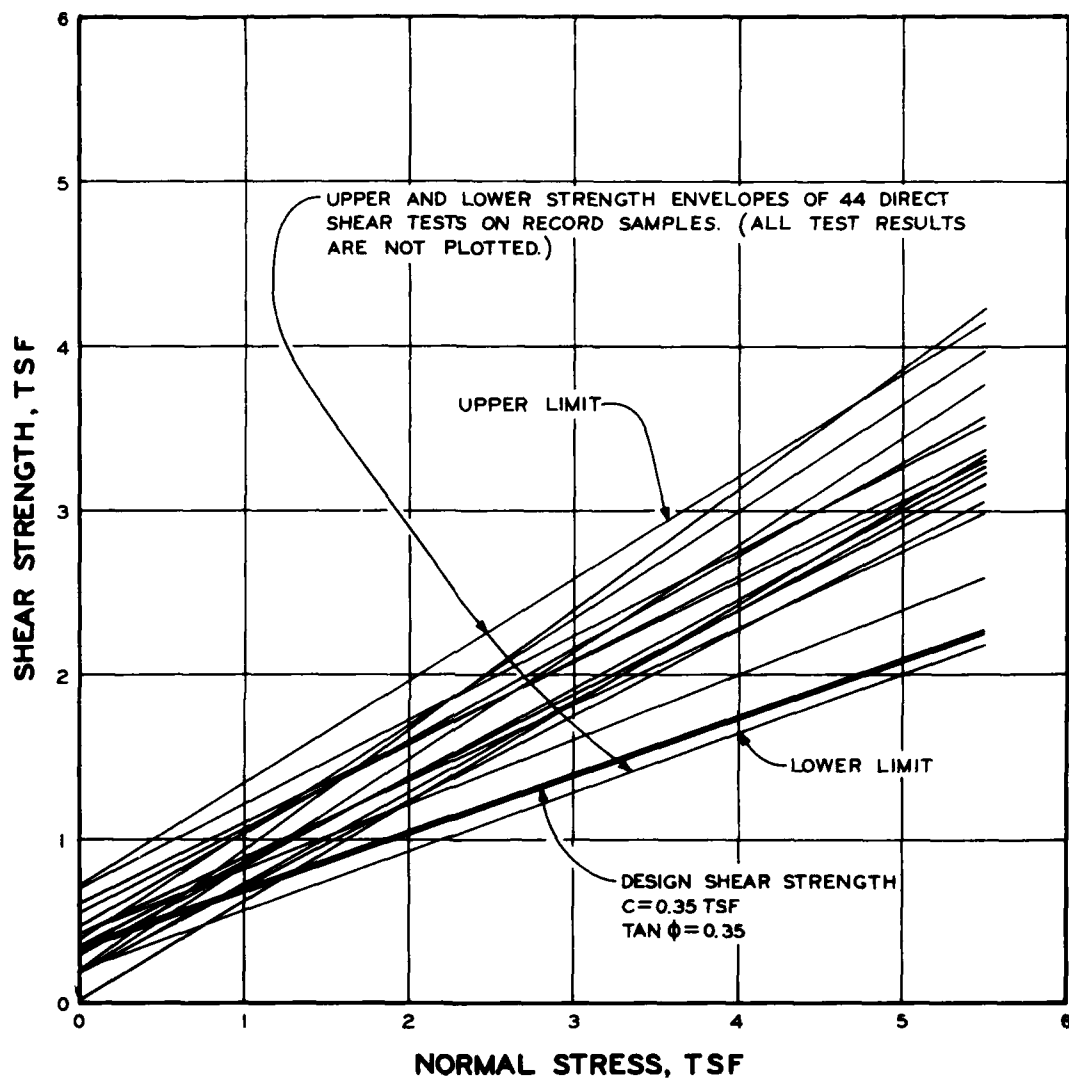
SUMMARY OF DIRECT SHEAR TESTS
ON EMBANKMENT RECORD SAMPLES
FORT RANDALL DAM

RW 3

RW-4

RW-5

PZ 3

'73 '74 '75 '76 '77 '78 '79 '80 '81 '82 '83 '84 '85 '86 '87 '88 '89 '90

↑ = INDICATES PRESSURE EVALUATION TESTS

DATE	DESCRIPTION	MADE	APP'D
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM
EMBANKMENT
RELIEF WELL AND RELIEF WELL PIEZ.
WATER LEVEL OBSERVATIONS

THIS PLAN ACCOMPANIES CONTRACT NO.
MODIFICATION NO.

EMBANKMENT CRITERIA AND PERFORMANCE REPORT   2   PLATE A-6

Reservoir Elevation

ELEVATION IN FEET - M.S.L

PZ-8
PZ-8A
PZ-8B
PZ-8C
PZ-8D
PZ-15A
PZ-15B
PZ-10
PZ-10A
PZ-10B
PZ-16A
PZ-16B

1360
1340
1260
1250
1260
1250
1260
1250
1260
1250
1260
1250
1240
1230
1240
1230
1260
1250
1260
1250
1260
1250
1240
1230
1240
1230

1977 '78 '79 '80 '81 '82 '83 '84 '85 '86 '87 '88 '89 '90 '91 '92 '93 '94

SECTION @ STA 81+50
N.T.S.

Blm Screen El 1229.3 PZ-10
1219.1 PZ-8

Blm Screen El 1181.8 PZ-8B PZ-8D
Blm Screen El 1180.7 PZ-10B

Blm Screen El 1176.6 PZ-16B
1164.2 PZ-15B

Blm Screen El 1120.3 PZ-8C
Blm Screen El 1135.4 PZ-10A
1121.9 PZ-8A

Blm Screen El 1131.1 PZ-16A
1147 PZ-15A

El 1330
AXIS OF DAM
El 1355
El 1370
El 1320

SEEPAGE AREAS

SWITCHYA

EI 1330

AXIS OF DAM

POWERHOUSE

EI 1355

EI 1370

EI 1920

EI 1300

SWITCHYARD

SEEPAGE AREAS

OUTLET WORKS

Stilling Basin

Terrace

FLOW

Discharge Channel

EI 1218.0

PZ-8A
PZ-8B
PZ-8C
PZ-8D

PZ-15A
Z-16A
PZ-16B

DATE	DESCRIPTION	MADE	APP'D
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM
EMBANKMENT

RELIEF WELL PIEZOMETERS PZ-8,
8A, 8B, 8C, 8D AND PZ-10, 10A, 10B
& DOWNSTREAM VALLEY PZ'S 15A, 15B, 16A & 16B

DESIGNED BY:			
DRAWN BY:			
CHECKED BY:			
SUBMITTED BY:			
CHIEF	SECTION		
RECOMMENDED			
CHIEF	BRANCH		
APPROVED		SCALE AS SHOWN	SPEC. NO.
COL. C. E., DISTRICT ENGINEER		SHEET	DRAWING NUMBER

RESERVOIR
ELEVATION

ELEVATION IN FEET - MSL

1310
1300
1290
1280
1270
1260
1250
1240
1230
1220

A 520
A 550
B 520
B 520

B 495: Dry since Pz rehab'd in 1977, Bot El 1269.3

B 550

TAILWATER ELEVATION

1974   '75   '76   '77   '78   '79   '80   '81   '82   '83   '84   '85   '86   '87   '88   '89   '90   '91

A 520

Chol'k berm

LINE "A" PIEZOMETERS

B 495
B52   B526   B55

LINE "B" PIEZOMETERS

'85   '86   '87   '88   '89   '90   '91   '92   '93   '94   '95   '96   '97   '98   '99   2000   '01   '02

A5.20

A5.50

LINE "A" PIEZOMETERS

B495

B52   B526

B55

B PIEZOMETERS

THIS DRAWING HAS BEEN REDUCED TO
THREE-FOURTHS THE ORIGINAL SCALE

DATE	DESCRIPTION	MADE	APPR'D
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM
EMBANKMENT
PIEZOMETER OBSERVATIONS
LINE A STA 48+00 - LINE B STA 56+00

SAFETY PAYS

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

EMBANKMENT CRITERIA AND PERFORMANCE REPORT   PLATE A42

2

'85　'86　'87　'88　'89　'90　'91　'92　'93　'94　'95　'96　'97　'98　'99　2000　'01　'02

UPSTREAM

DOWNSTREAM

MISSOURI RIVER
FORT RANDALL DAM-LAKE FRANCIS CASE
EMB. RELIEF WELLS AND PIEZOMETERS
UNDERSEEPAGE GRADIENT
PIEZ LINE "C" - STA 70+00
U.S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS OMAHA, NEBRASKA

IMPERVIOUS
EMBANKMENT

LINE OF RELIEF WELLS

CHALK BERM

IMPERVIOUS BLANKET

SEEPAGE AREA

PERVIOUS FILL

PIEZOMETER TUBES

RIVER BED

VALLEY ALLUVIAL SAND

BEDROCK AT EL 1160±

SCALE VERT 1" = 20'
HORIZ 1" = 400'

'THIS DRAWING HAS BEEN REDUCED TO
THREE-EIGHTHS THE ORIGINAL SCALE'

DATE	DESCRIPTION	MADE	APP'D
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM

EMBANKMENT

PIEZOMETER OBSERVATIONS
LINE C STA 70+00

THIS PLAN ACCOMPANIES CONTRACT NO.
MODIFICATION No.

'85 '86 '87 '88 '89 '90 '91 '92 '93 '94 '95 '96 '97 '98 '99 2000 '01 '02

DOWNSTREAM

MISSOURI RIVER
FORT RANDALL DAM-LAKE FRANCIS CASE
EMB. RELIEF WELLS AND PIEZOMETERS
UNDERSEEPAGE GRADIENT
PIEZ. LINE "D" - STA 81+50
U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS OMAHA, NEBRASKA

THIS DRAWING HAS BEEN REDUCED TO
THREE-EIGHTHS THE ORIGINAL SCALE.

IMPERVIOUS
EMBANKMENT

LINE OF RELIEF WELLS

CHALK BERM

APPROX EXTENT
OF BERM IN
ADJACENT AREA

APPROXIMATE
T W ELEV

SEEPAGE AREA

PIEZOMETER TUBES

SCALE VERT I INCH=20 FEET
HORIZ I INCH=400 FEET

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

DATE	DESCRIPTION		
	REVISIONS		

MISSOURI RIVER
FORT RANDALL DAM
EMBANKMENT
PIEZOMETER OBSERVATIONS
LINE D STA 81+50

SAFETY PAYS

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

EMBANKMENT CRITERIA AND PERFORMANCE REPORT   PLATE A69

2

Fort Randall Dam - Lake Francis Case, Embankment Relief Wells and Piezometers, Underseepage Gradient, Piez. Line "E" - Sta. 96+00

DOWNSTREAM

LEGEND

PIEZOMETER 385   5 NOV 1965
                 4 APR 1967
                 5 JUN 1971
                 22 DEC 1953

NOTE
TIME PLOT SHOWN ON PLATE A 13

IMPERVIOUS
EMBANKMENT

FS 63

FS 75

CHALK
BERM

PIEZOMETER TUBES

SCALE   VERT 1"=20'
        HORIZ 1"=400'

MISSOURI RIVER
FORT RANDALL DAM-LAKE FRANCIS CASE
EMB. RELIEF WELLS AND PIEZOMETERS
UNDERSEEPAGE GRADIENT
PIEZ LINE "E" - STA 95+00
U.S. ARMY ENGINEER DISTRICT. OMAHA
CORPS OF ENGINEERS OMAHA, NEBRASKA

DATE	DESCRIPTION	MADE	APP'D
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM
EMBANKMENT
PIEZOMETER OBSERVATIONS
LINE E STA 95+00

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

SAFETY
PAYS

SCALE AS SHOWN

EMBANKMENT CRITERIA AND PERFORMANCE REPORT   PLATE A50

2

1370

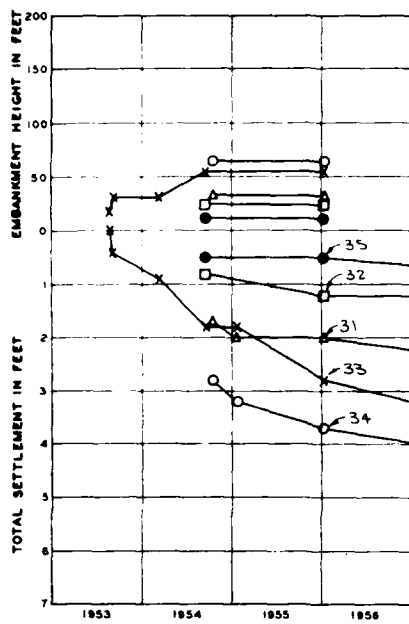RESERVOIR ELEVATION

1360

1350

1340

1330

1320

1310

ELEVATION IN FEET - MSL

1300

1290

1280

12 0

1260

1250

1240

1230

TAILWATER ELEVATION

1220

1974   '75   '76   '77   '78   '79   '80   '81   '82   '83   '84   '85   '86   '87   '88   '89   '90

F 5 12

F 4 96

L 5 21

F 4 9

F 5 04

PRESSURE EVALUATION TESTS 27 JULY 1977

LEGEND

		LOCATION
○	PIEZOMETER OBSERVATION	F 49 } LEFT ABUTMENT
◇		F 496 } PERVIOUS GLACIAL DEPOSIT
□	PIEZOMETER OBSERVATION	F 5 12 } DOWNSTREAM
△		F 5 04 } PERVIOUS
▼		L 5.21 } DRAIN

Note Piez L 5 21 DRY PRIOR TO 1974

EI 1400.0

Intake structure

flood pool EI 1375.0   EI 1380.0

Max normal pool EI 1365.0

Bulkhead slot

Emergency gate slot

Service gate slot

Air vents 7'-6" x 11'-0" (2 per tunnel)

EI 1320'

EI 1288'

Approach slab

FLOW

Tunnel and penstock slope 1 1025 % from Sta 246+00

Inv EI 1126.65

Inv EI 1123.13

STA 244+60   STA 245+70   STA 246+00

Closure section

50'-0"   110'-0"   50'-0"

400'-0"

Profile of dam

Impervious Fill

Riprap

F 49

F 496

F 5 04   EI 1393.0

Pervious Glacial strata

Rolled Emb

Upstream embedment

L 5 21   STA 250+50

F 5 12

Axis of dam STA 250+00

STA 248+15

EI 0

Penstock drain

28 Spcs @ 24'-0" - 672'-0"

4'-3'-0"

PROFILE OF PENSTOCK   4

'86 '87 '88 '89 '90 '91 '92 '93 '94 '95 '96 '97 '98 '99 2000 '01 '02

F5.04
EL.1395.0

THIS DRAWING HAS BEEN REDUCED TO
THREE-EIGHTHS THE ORIGINAL SCALE.

River Emb.

L5.21 (STA. 170+50)
F5.12

El 1320±
El 1315'
El 1300'   El Varies
El 1285'

Pervious Glacial Strata

Niobrara Chalk

STA. 254+73
STA. 255+07 (Limit of this contract)
STA. 255+48

28 Spcs. @ 24'-0" = 672'-0"

Tunnel inv. E. 1215.95

Powerhouse substructure

473'-0"   37'-0" 101'-0"

CK   4

DATE	DESCRIPTION	MADE	APPR'D
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM
EMBANKMENT
LEFT ABUTMENT
PIEZOMETER OBSERVATIONS
LINE "F" 4 "L"

DESIGNED BY:
DRAWN BY:
CHECKED BY:
SUBMITTED BY:
SCALE: AS SHOWN

EMBANKMENT CRITERIA AND PERFORMANCE REPORT   PLATE A51

2

NOTE:
1. * No readings were recorded between 3-68 to 9-76
2. THESE PIEZ'S WERE GIVEN PRESSURE EVALUATION TESTS 26 JUL & 18 AUG 1977

STATION 71+50

TYPICAL SECTION THRU EMBANKMENT CLOSURE SECTION

ELEVATION

'66 '67 '68 '76 '77 '78 '79 '80 '81 '82 '83 '84 '85 '86 '87 '88 '89

'66 '67 '68 '76 '77 '78 '79 '80 8 '82 '83 '84 '85 '86 '87 '88 '89

STATION 65+00

'66 '67 '68 '76 '77 '78 '79 '80 '81 '82 '83 '84 '85 '86 '87 '88 '89

STATION 71+50

FOR LOCATION PLAN SEE PLATE NO 45?

DATE	DESCRIPTION	MADE	APPR'D
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM
SETTLEMENT GAGE PIEZOMETERS
STATIONS 65+00 AND 71+50
WATER SURFACE ELEVATIONS

SAFETY PAYS

THIS PLAN ACCOMPANIES CONTRACT NO.
MODIFICATION NO.

SHEET 4

EMBANKMENT CRITERIA AND PERFORMANCE REPORT   PLATE A52

2

Normal Operating Pool Level

Impervious Blanket

El 1300
Upstream Chalk Berm

Cut off Trench

AXIS OF DAM

Fort Randall Cemetery

Limits of emer drain

Manholes

Collector pipe

Transition

Fort Randall Chapel Ruins

Site of old Fort Randall

Parking

POWER

SWITCH

Missouri River Bank (old)

Discharge Channel

TABLE
SETTLEMENT GAGE DATA

Gage Number	Station	Range	Initial Elevation Of Settlement Plate	Length Of Settlement Pipe
41	22+50	4950	1354.82	55.05
42	22+50	5100	1345.72	78.74
43	30+00	4950	1328.63	55.49
44	30+00	5035	1330.56	61.13
45	30+00	5250	1333.65	13.44
46	40+00	4950	1309.16	77.31
47	40+00	5035	1308.59	90.65
48	40+00	5250	1313.39	34.21
16	48+00	4950	1281.347	103.64
17	48+00	5035	1281.123	118.04
18	48+00	5250	1301.238	46.98
19	48+00	5510	1302.27	16.76
19	57+00	4950	1266.285	117.37
70	57+00	5180	1266.151	40.37
22	65+00	4920	1241.585	146.02
23	65+00	5035	1254.06	143.75
24	65+00	5300	1250.32	69.12
25	65+00	5510	1247.99	56.54
27	71+50	4920	1242.27	146.19
28	71+50	5035	1251.69	144.48
29	71+50	5300	1251.07	68.62
30	71+50	5510	1248.72	56.81
2	82+00	4920	1243.112	146.67
3	82+00	5035	1742.813	157.92
4	82+00	5300	1242.899	84.82
5	82+00	5510	1243.784	63.89
7	90+00	4920	1247.316	141.59
8	90+00	5035	1248.306	147.41
9	90+00	5300	1248.375	72.40
10	90+00	5510	1245.854	61.85
12	99+00	4920	1244.702	147.65
13	99+00	5035	1245.798	149.89
14	99+00	5100	1244.777	75.47
15	99+00	5510	1244.027	96.42

El 1227

1 on 3

El 1300
Upstream Chalk Berm

APPROACH CHANNEL

Limits of impervious blankets

El 1335

AXIS OF DAM

POWERHOUSE

El 1320

El 1300

INTAKE STRUCTURE

El 1270

SWITCHYARD

OUTLET WORKS

Pervious embankment

SPILLWAY

El 1250

Discharge Channel

El 1180

El 1180

Berm El 1370

To Pickstown

LEGEND

▲ Settlement gages

NOTE   For location of Sections A A B B C C D D and E E
see Plate A4

For location of Sections F F G G and H H see Plate A

THIS DRAWING HAS BEEN REDUCED TO
THREE-EIGHTHS THE ORIGINAL SCALE.

SCALE 1 INCH = 400 FEET

THIS DRAWING HAS BEEN REDUCED TO
THREE-EIGHTHS THE ORIGINAL SCALE

SETTLEMENT PLOTS, 1974-1981

SETTLEMENT PLOTS, 1953-1976

DATE	DESCRIPTION	MADE	APP'D
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM
FOUNDATION SETTLEMENT
STA 22+50, STA 30+00 & STA 40+00

SAFETY PAYS

THIS PLAN ACCOMPANIES CONTRACT NO.
MODIFICATION NO.

EMBANKMENT CRITERIA AND PERFORMANCE REPORT   PLATE A54
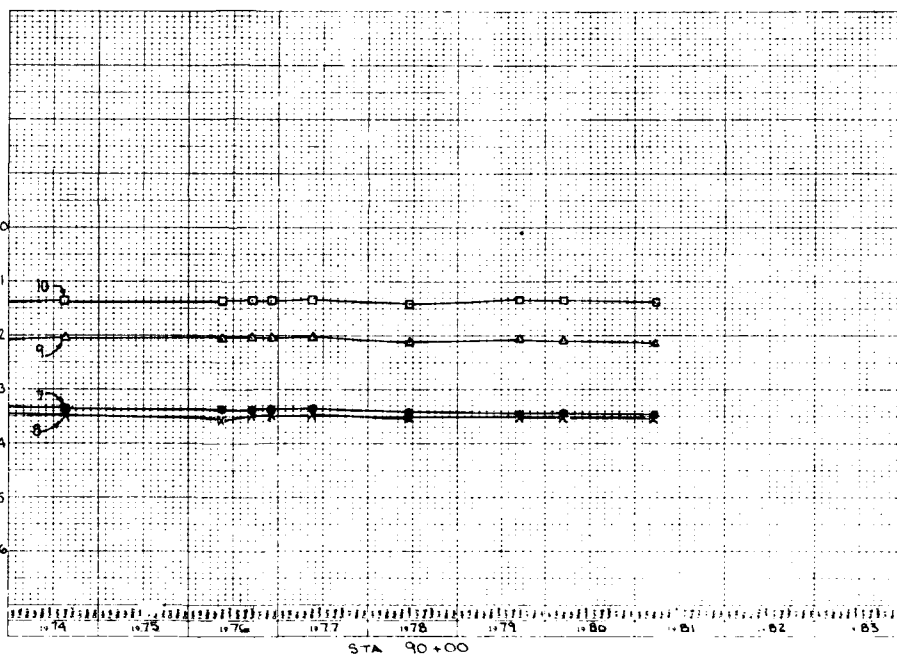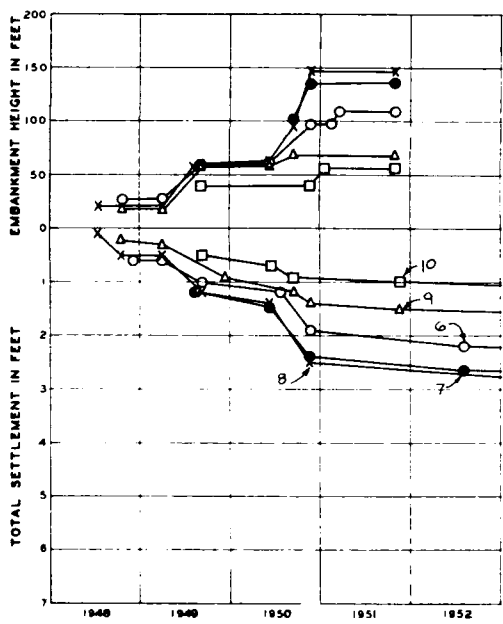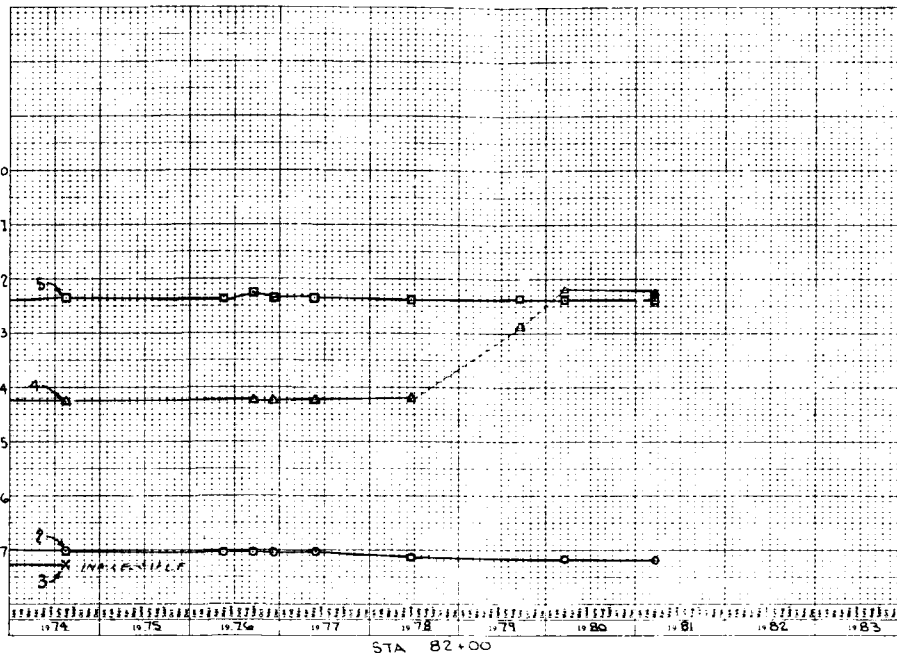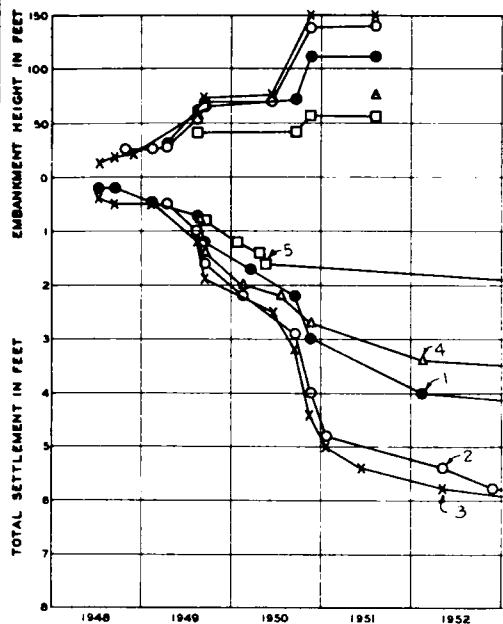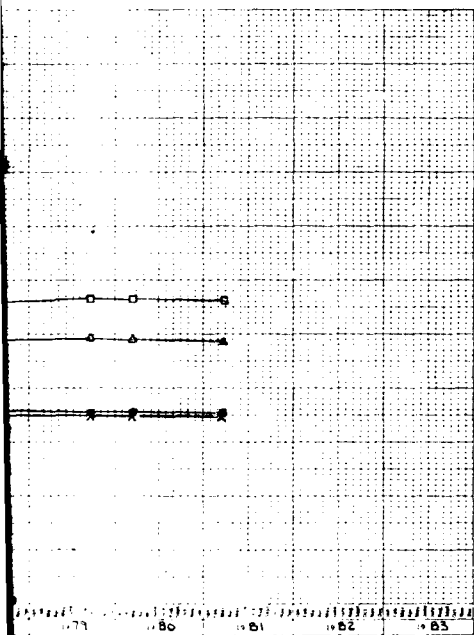
2

SETTLEMENT PLOTS, 1974-1981

SETTLEMENT PLOTS, 1948-1973

SAFETY PAYS

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

EMBANKMENT CRITERIA AND PERFORMANCE REPORT   PLATE A55

2

INITIAL ELEVATIONS OF MARKERS

MARKER NO	INITIAL ELEV	REMARKS
40A	1378.415	
86A	1378.411	
90A	1378.240	
40B	1398.240	
66B	1394.914	
90B	1394.197	
9C	1424.221	
17C	1397.854	
30C	1398.166	
40C	1398.776	
50C	1398.826	
60C	1394.500	
66C	1398.115	REPLACED WITH
72C	1398.035	FROST FREE
80C	1395.511	MOVEMENT MARKERS
90C	1394.790	WINTER OF 1980-1981
99C	1398.474	
40D	1342.726	
66D	1325.807	
90D	1334.816	
104B	1304.588	
66E	1301.228	
90E	1301.129	
72E	1308.040	
30C	1398.828	
36C	1396.834	
45C	1396.246	
81+30C	1394.710	

INITIAL DISTANCES BETWEEN MARKERS (C-LINE)

MARKER NO	DISTANCE IN FEET
9CX	
	820.666
17C	
	1323.738
30C	
	999.724
40C	
	999.792
50C	
	999.756
60C	
	999.876
66C	
	599.962
72C	
	700.912
80C	
	1000.100
90C	
	800.130
99C	

Impervious Blanket

Upstream Chalk Berm

AXIS OF DAM

Transition

Fort Randall Cemetery

Fort Randall Chapel Ruins

Site of old Fort Randall

Right River Bank (old)

1

LEGEND
● MOVEMENT MARKER
▲ CONTROL MONUMENT
▲ CP CONTROL POINT

NOTES:
1 MOVEMENT MARKERS ON LINES A TO E
INITIALLY INSTALLED ON EXACT ALIGNMENT
WITH ABUTMENT CONTROL MONUMENTS
2 MOVEMENT MARKERS AND CONTROL MONU-
MENTS INSTALLED AND INITIAL OBSERVATIONS
MADE DURING JULY AND AUGUST 1957

THIS DRAWING HAS BEEN REDUCED TO
THREE-EIGHTHS THE ORIGINAL SCALE

THIS DRAWING HAS BEEN REDUCED TO
THREE EIGHTHS THE ORIGINAL SCALE

SCALE: 1 INCH = 400 FEET

MISSOURI RIVER
FORT RANDALL DAM - LAKE FRANCIS CASE
RESERVOIR & POWERHOUSE
LOCATION PLAN
CREST & SLOPE MOVEMENT MARKERS

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
OMAHA DISTRICT
OMAHA, NEBRASKA

MAR 1972

THIS DRAWING HAS BEEN REDUCED TO
THREE-EIGHTHS THE ORIGINAL SCALE

FOR LOCATION PLAN SEE PLATE A56

DATE	DESCRIPTION	MADE	APP'D

REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER

FORT RANDALL DAM

CREST AND SLOPE MOVEMENT MARKERS

HORIZONTAL MOVEMENT

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

ENBANKMENT CRITERIA AND PERFORMANCE REPORT   PLATE A57

2

INITIAL AUG '57

VERTICAL MOVEMENT IN FEET

LINE "C"

LEGEND
JUNE '58
APRIL 1960
MAY 1961
MAY 1962
JUNE 1963
MAY 1964
JUNE 1965
JUNE 1966
NOV 1967
SEPT 1970
MAY 1972
MAY '76
MAR '81

NOTE:
DESTROYED MOVEMENT MARKER,
66C WAS REPLACED IN 9?3.

MARKER 90 HAS DESTROYED
PRIOR TO THE ?81 SURVEYS.

66C REESTABLISHED JULY '05

48C DESTROYED
RESTABLISHED JULY '73

66C DESTROYED

1958

1

KEY PLAN

DATE	DESCRIPTION	MADE	APPR'D
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM
SLOPE AND CREST MOVEMENT MARKERS
VERTICAL MOVEMENT
LINE "C"

SAFETY PAYS

THIS PLAN ACCOMPANIES CONTRACT No.
MODIFICATION No.

SLOTS 1-2
DEFLECTION IN UNITS

SLOTS 3-4
DEFLECTION IN UNITS

DEPTH VS. DEFLECTION

SLOTS 3-4
DEFLECTION IN UNITS

-5000 -4000 -3000 -2000 -1000 0 1000 2000 3000 4000 5000

ELEVATION IN FEET - MSL

-2.40   +1.20   0   1.20   2.40
DEFLECTION IN INCHES
RIGHT   LEFT

LEGEND
INITIAL   1-10-90

Fat Clay (Fill)
Gravelly Sandy Clay (Fill)

Fat Clay - Lean Clay (Fill)
   (border line)
Sandy Gravelly Fat Clay (Fill)

Silty Sandy Clay (Fill)   Base of Embankment
Silt
Fat Clay
Sandy Fat Clay

Gravelly Sandy Clay
Chalk
Sandstone
Shale
Sandstone

Interbedded Sandstone and Shale
Shale

Bottom of Hole 3199'

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

MISSOURI RIVER
FORT RANDALL DAM
TILTMETER OBSERVATIONS
T 40/503
EMB STA 40+00 RANGE 5030

SAFETY PAYS

THIS PLAN ACCOMPANIES CONTRACT NO.
DACA45
MODIFICATION NO.

APPENDIX B
PHOTOS

PHOTO NO. 1: Aerial view of Fort Randall Dam. June 1974

PHOTO NO. 2: Scarifying surface prior to placement
of fill. Earthwork Stage II. 20 May '49

PHOTO NO. 3: Blading operation in embankment
construction. Initial earthwork.

PLATE B-2
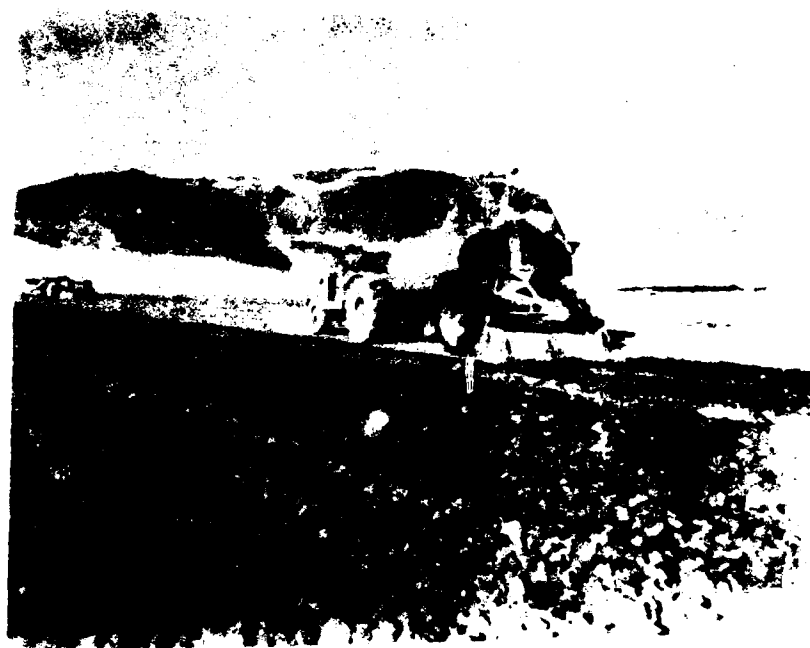
PHOTO NO. 4: Watering operation in embankment
            construction. Earthwork Stage II. 20 May '49

PHOTO NO. 5: Excavation of right bank cut-off trench,
            upstream end, looking towards river. 6 Nov '50

PHOTO NO. 6: Sheepsfoot tamping roller compacting
impervious fill. Earthwork Stage III. 10 Oct '50
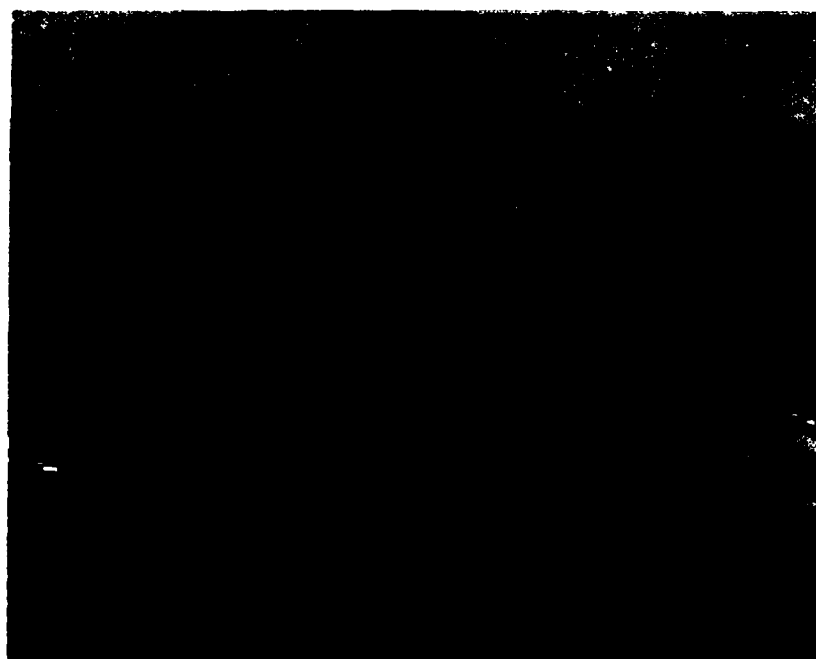
PHOTO NO. 7: Construction of intake structure and
embankment in foreground and excavation for spill-
way weir in background, looking toward the left
abutment.   11 May '51

PLATE B-4

PHOTO NO. 8: Dredging in chalk spoil area downstream of embankment for hydraulic filling to effect river closure. Earthwork Stage III. 17 Jul '52

PHOTO NO. 9: Final stage of river closure with dredged chalk fill. Water level at about El. 1242. 20 Jul '52

PHOTO NO. 10: Dumped chalk fill over the dredged
chalk weir crest. Earthwork Stage III. 23 Jul '52
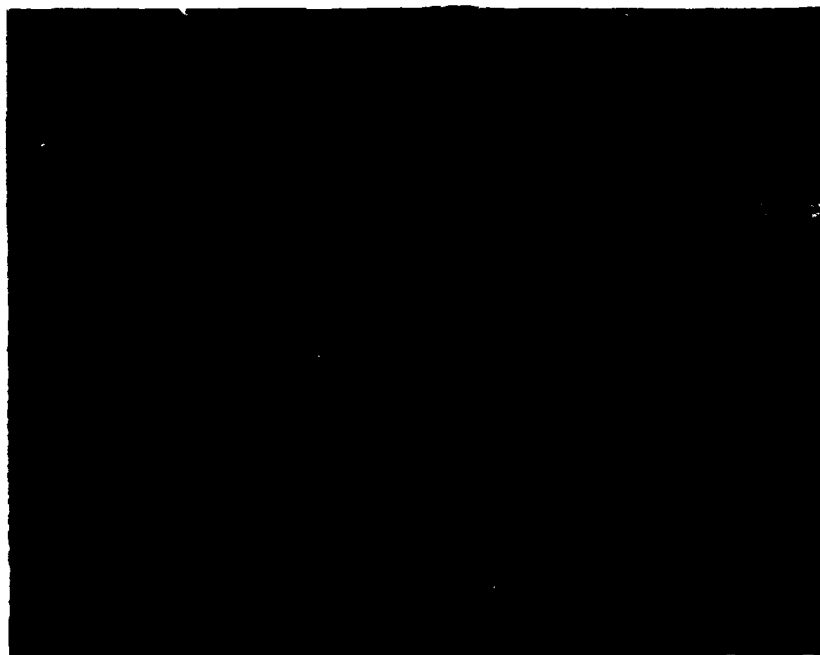
PHOTO NO. 11: Placement of pervious dike at location
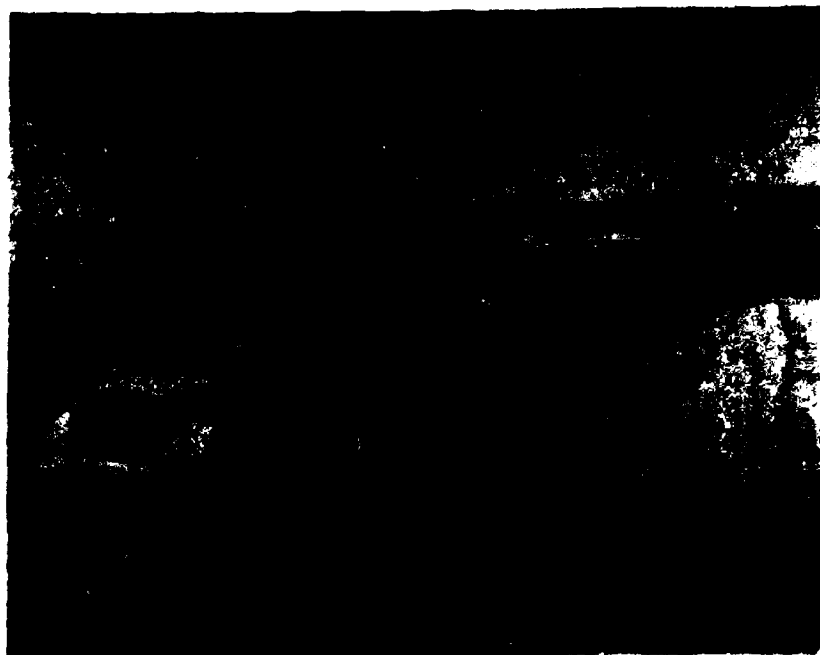of upstream toe of impervious blanket. Earth-
work Stage III. 1 Aug '52

PHOTO NO. 12: Embankment material being placed
on top of dredged hydraulic fill at location of up-
stream embankment toe. Earthwork Stage III. 1 Aug '52

PHOTO NO. 13: Same area as shown in Photo No. 12.
Pictured are double cat-dozer and towed double
drum sheepsfoot roller. 1 Aug '52

PHOTO NO. 14: Hydraulic filling of embankment
foundation in closure section. Earthwork
Stage III. 15 Sep '52.

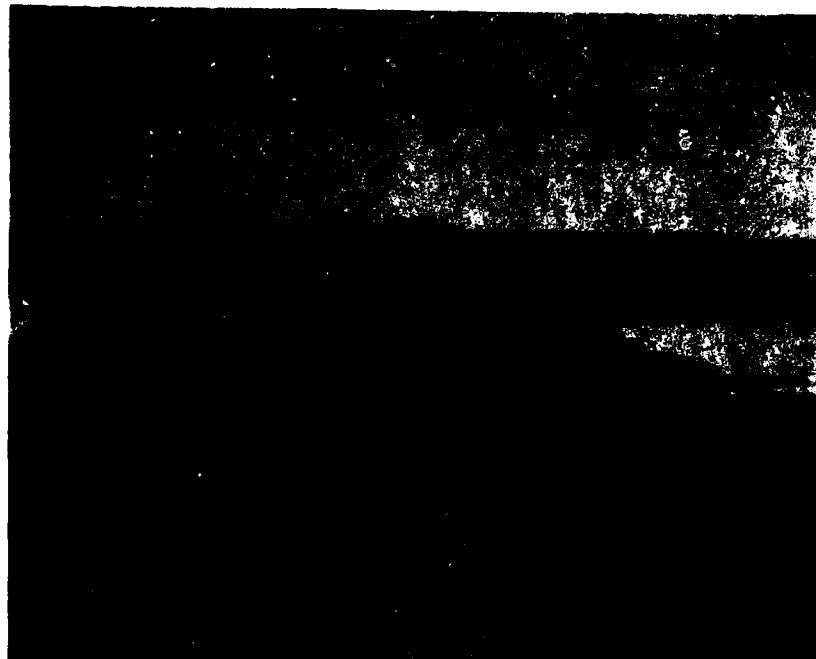PHOTO NO. 15: Aerial view of project looking down-
stream. 15 Sep '52

PLATE B-8

PHOTO NO. 16: Embankment construction in closure
area looking towards the left abutment. Fill at
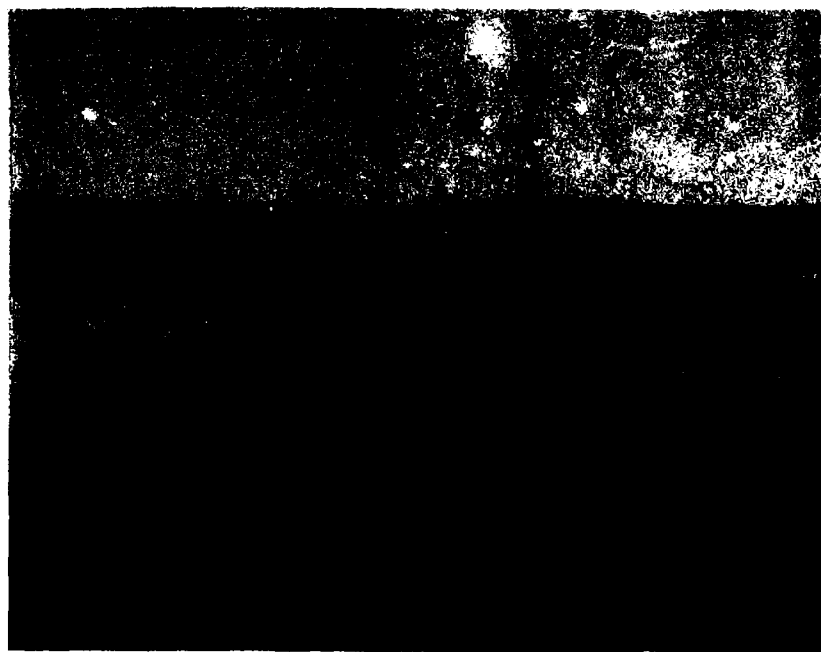approx. El. 1246. Earthwork Stage I I I. 14 Aug '52.

PHOTO NO. 17: Upstream portion of embankment,
looking towards the right abutment. Fill is at
approx. El. 1288. Earthwork Stage I I I. 29 Sep '52

PLATE B-9

PHOTO NO. 18: Embankment construction in closure
area to approx. El. 1325. Embankment in foreground
is at crest level, El. 1395. Earthwork Stage III.
(17 Oct '52)

PHOTO NO. 19: View of embankment construction
looking towards right abutment, showing placement
of upstream chalk berm. Earthwork Stage IV. 4 Aug '53

PLATE B-10

PHOTO NO. 20:   Construction of upstream chalk berm
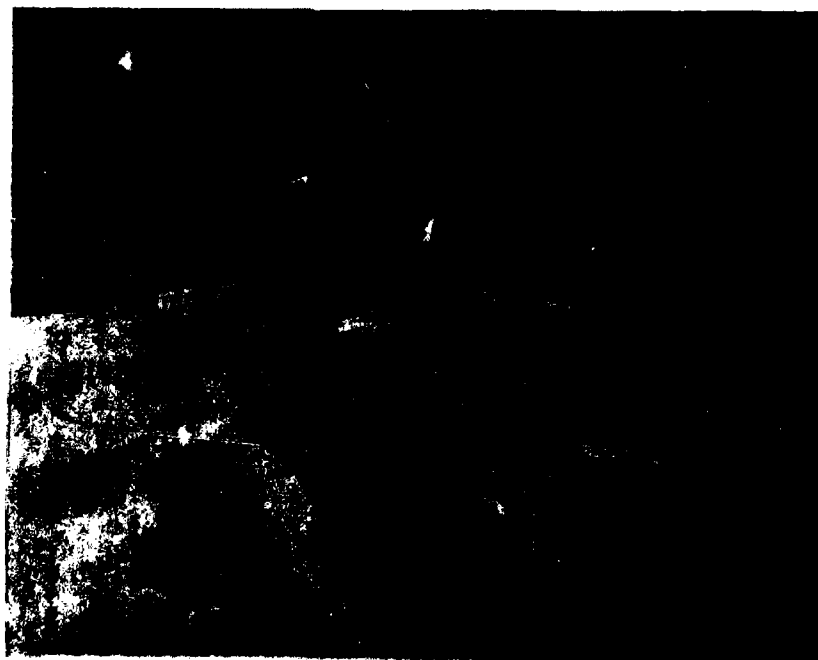looking SW towards right abutment.   4 Aug '53

PHOTO NO. 21:   Aerial view of construction during
Earthwork Stage V.   16 Nov '53

PHOTO NO. 22: View of embankment construction
looking along dam axis towards the right abutment.
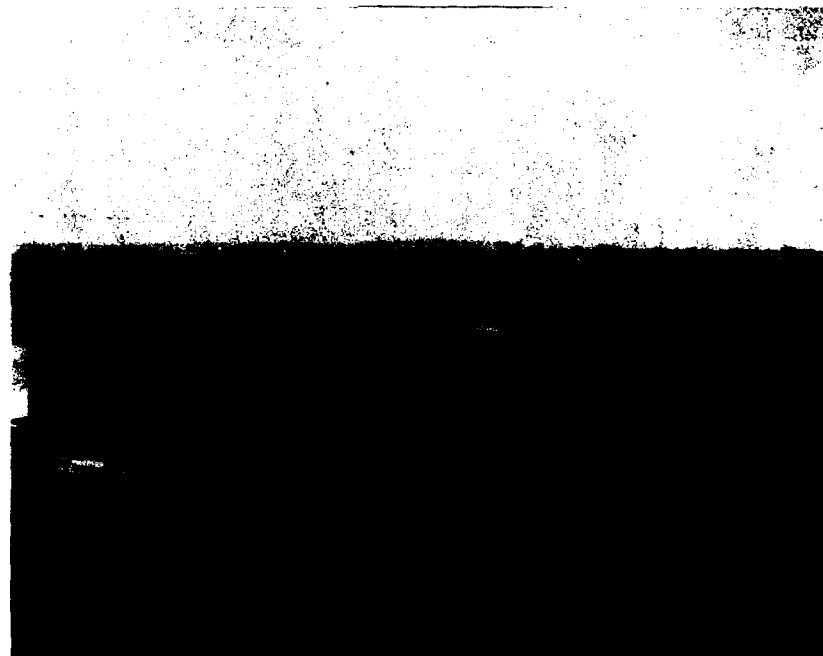Earthwork Stage V.   25 May '54

PHOTO NO. 23:   Fill placement between west spillway
abutment and previously completed embankment.
Top of fill is approx. El. 1360. 25 Jun '54.

PLATE B-12

PHOTO NO. 24: Aerial view of completed embankment,
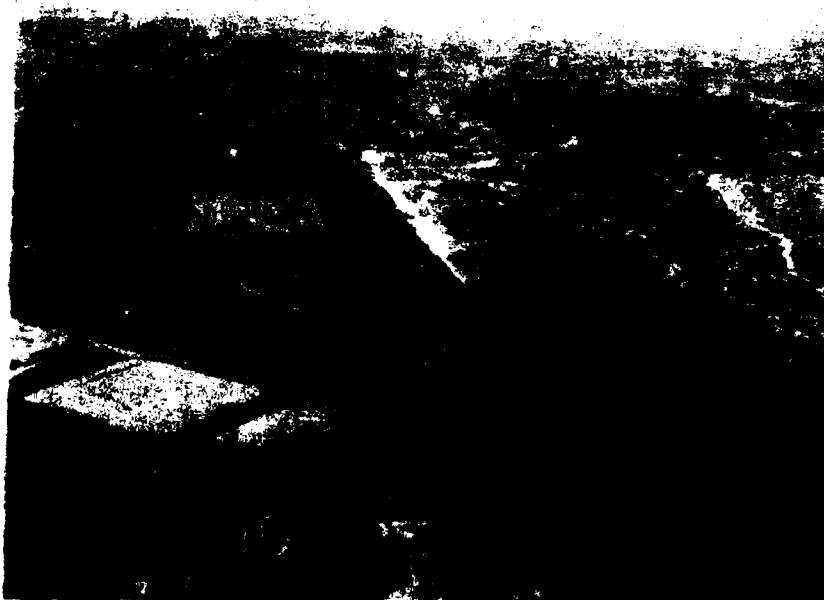looking from right abutment. 30 Jun '55

PHOTO NO. 25: Aerial view of project, looking SW.
29 Mar '55

PLATE B-13

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