Kansas Citys, Missouri and Kansas
Flood Damage Reduction Feasibility Study
(Section 216 – Review of Completed Civil Works Projects)
Engineering Appendix to the Interim Feasibility Report

Chapter A-7

GEOTECHNICAL ANALYSIS FAIRFAX-JERSEY CREEK (BPU FLOODWALL)

CHAPTER A-7
GEOTECHNICAL ANALYSIS
FAIRFAX-JERSEY CREEK (BPU FLOODWALL)

A-7.1 INTRODUCTION

This chapter presents the results of the geotechnical evaluation of an area in the Fairfax-Jersey Creek protection unit determined to have a certain probability of failure under the existing level of protection that warranted further study. The determination relies on historical borings and soil test information combined with recent subsurface borings and soil test information.

A-7.2 SOURCES OF EXISTING LEVEE DESIGN INFORMATION

The primary sources of information for this geotechnical analysis include the references listed in the References section of this chapter.

A-7.3 DESCRIPTION OF THE LEVEE UNIT

Refer to Section A-4.3.7 for a detailed description of the Fairfax-Jersey Creek Unit.

A-7.4 LEVEE DESIGN FEATURES

A-7.4.1 Existing Levee and Floodwall Sections

The Fairfax-Jersey Creek levee unit consists of levees, floodwalls, stoplop and sandbag gaps, riprap and levee toe protection, surfaced levee crown and ramps, drainage systems, the Jersey Creek sewer structure and shutter gates, and pumping plants. It was originally constructed as a local levee, but was removed and replaced using Federal standards in 1940. The final contract for construction of the project was completed in 1955.

A plan view of the Fairfax-Jersey Creek Unit and typical sections are provided in the Supplemental Exhibits section as Exhibits A-7.1 through A-7.11.

A-7.4.2 Future Flood Protection Concerns

This levee unit is being considered for a partial reestablishment of existing design level of protection near the Kansas River, or the beginning of the project. Some areas have been surveyed and appear to be lower than the original federal design elevation. The remainder of the system is not recommended for a raise based on the hydraulic analysis of the Missouri and Kansas Rivers.

As a result of an evaluation of the existing floodwall near the BPU facilities, it was concluded that Stations 287+85 to 302+32 required further evaluation. The 1993 flood did not reach the top of the floodwall. A full head to the top of the floodwall is analyzed in this chapter.

A-7.4.3 Area Site Characterization

The Corps of Engineers' boring and additional subsurface information shown in the Supplemental Exhibits section (Exhibits A-7.12 and A-7.13) was used to characterize

and model the foundation resistance for the existing precast concrete piles driven below the existing floodwall. The boring assignment for investigations of the Fairfax area was developed well in advance of the floodwall concerns. One Corps boring was proposed earlier, but was designated for investigation of an existing stoplog gap. The boring was eliminated when it was discovered that the stoplog gap had been permanently sealed. No additional borings were assigned for the study of the floodwall during the earlier stage of general investigations. Later, when concerns were raised regarding the integrity of the floodwall pile length and strength, additional investigations were conducted through a contractor. The Investigation Report is included as Exhibit A-7.14.

These borings were used for developing the resistance to vertical and horizontal loading. The characterization of the floodwall foundation relied on the Standard penetration resistance obtained by driving a split spoon through the foundation sands. An automatic trip hammer was used and appropriate corrections were made for energy input, overburden pressures, rod type, rod length, hole size, and other pertinent variables during the test. This information was used to develop the expected soil strength of the foundation sands (refer to Exhibits A-7.17 through A-7.20).

A-7.4.4 Pile Capacity

The soil strength and existing pile length and configuration below the pile cap was used to develop a spreadsheet model for determination of the available amount of side friction and end bearing for a circular driven precast concrete pile. The lateral coefficient of friction was chosen for the driven condition and a correction to the resistance was made, taking into consideration the expected foundation underseepage pressure during a high river condition. A flow net was used to model the underseepage pressures. This information was provided to the structural engineering team members for their use. Exhibit A-7.15 and Exhibits A-7.21 through A-7.23 contain the geotechnical calculations for pile capacity. A summary of the ultimate foundation resistance is provided in Exhibit A-7.24.

A-7.4.5 Underseepage Analyses

The model used to develop the expected pressure in the foundation sands along the length of the driven piles was a simple hand-drawn flow net. The cutoff wall was ignored and equipotential lines with perpendicular flow lines were developed below a mostly sand foundation. The equipotential lines were used to determine the pressure along the three piles located landside, the middle piles, and the piles riverside of the stem of the floodwall. These tabulated values were added to the model for the resistance to correct for lower effective overburden pressure.

A-7.5 REFERENCES

1. Fairfax Jersey Creek Levee Unit, Definite Project Report, December 1938.

2. Fairfax Jersey Creek Levee Unit, Definite Project Report, Raising the Existing Flood Protection,
September 1945.

3. Fairfax Jersey Creek Levee Unit, Analysis of Design Jersey Creek Sewer, March 1951.

4. Fairfax Jersey Creek Levee Unit, Supplement on Interior Drainage, September 1952.

5. Fairfax Jersey Creek Levee Unit, Analysis of Design Modifications to Existing Flood Protection, September 1952.

6. Fairfax Jersey Creek Levee Unit, Analysis of Design Pump Plant F2, F3, F6 and F9, December 1952.

7. Fairfax Jersey Creek Levee Unit, Analysis of Design Pump Plant F4, F5, F7 and F8, June 1953.

8. Fairfax Jersey Creek Levee Unit, Design Memorandum No. 1, August 1953.

9. Fairfax Jersey Creek Levee Unit, Design Memorandum No. 2 - Pump Plant F1, October 1953.

10. Fairfax Jersey Creek Levee Unit, Analysis of Design Station 1+54 to Station 29+77 Floodwall and I-wall, February 1954.

11. Fairfax Jersey Creek Levee Unit, Record Drawings – Operations and Maintenance Manual Volume One, May 1941 to October 1952.

12. Fairfax Jersey Creek Levee Unit, Record Drawings – Operations and Maintenance Manual Volume Two, November 1953 to July 1961.

13. Fairfax Jersey Creek Levee Unit, Record Drawings – Levee and Appurtenances February 1954.

14. Fairfax Jersey Creek Levee Unit, Record Drawings – Levee and Appurtenances, Appendix I, May 1959.

15. Fairfax Jersey Creek Levee Unit, Record Drawings – Pump Plants, Appendix II, September 1956.

16. Fairfax Drainage District – Flood Wall Pile Investigation Report, October 2003.

17. "Shear Strength Correlations for Geotechnical Engineering". Duncan, Horz, and Yang, August 1989.

18. APILE Plus 3.0 for Windows, Reese, Wang, and Arrellaga, 1998.

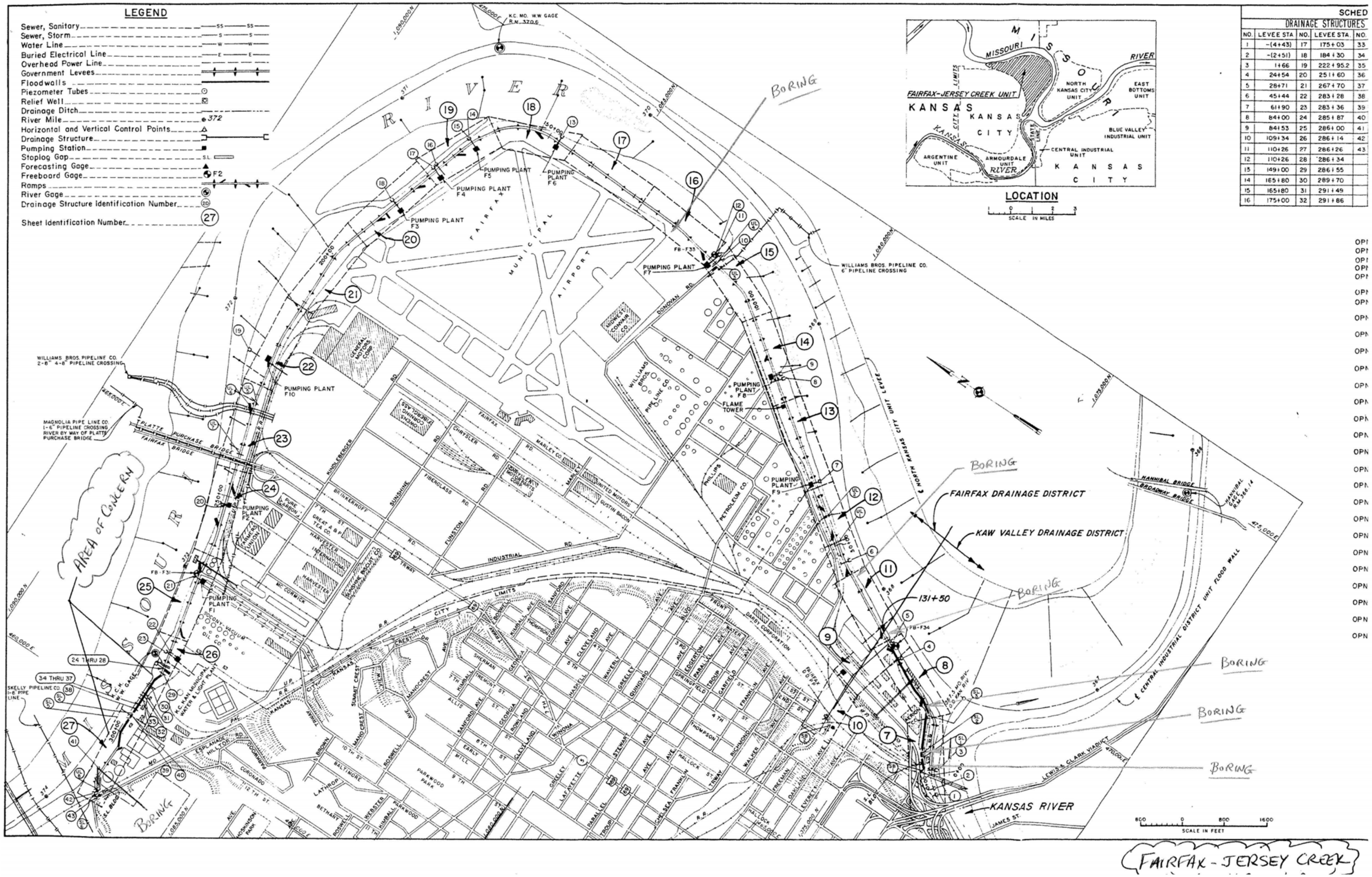
19. "Design of Pile Foundations", U.S. Army Corps of Engineers, Engineering Manual EM 1110-2-2906, 15 January 1991.

20. "Risk Analysis in Geotechnical Engineering for Support of Planning Studies", U.S. Army Corps of Engineering, Engineer Technical Letter, ETL 1110-2-556, 28 May 1999.

21. "Sheet Pile Wall and Levee Evaluation – Jersey Creek Outfall to Wharf Structure, Kansas City, Kansas", URS Corporation, October 7, 2004.

A-7.6 SUPPLEMENTAL EXHIBITS

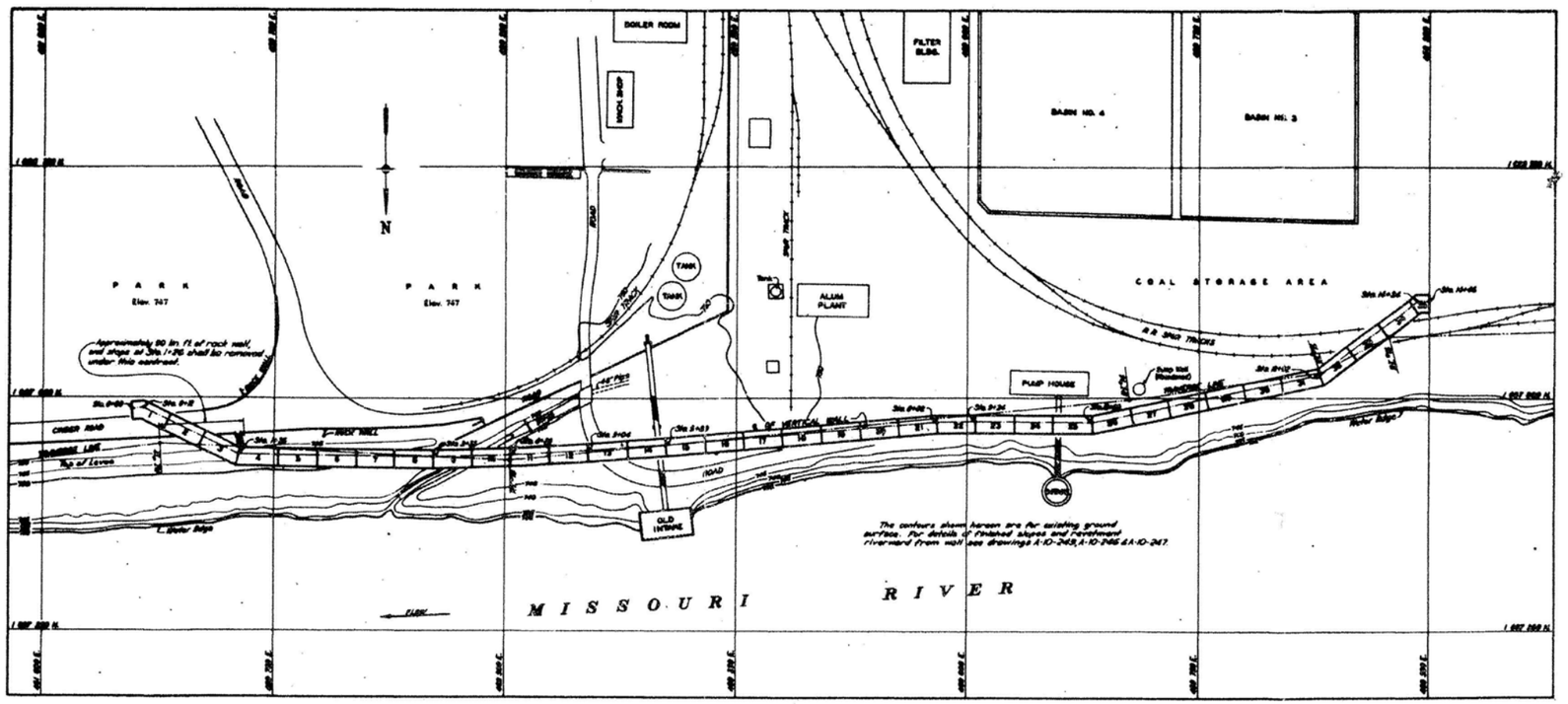
EXHIBIT A-7.1



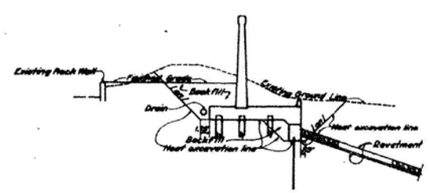
SCHED				
DRAINAGE STRUCTURES				
NO.	LEVEE STA.	NO.	LEVEE STA.	NO.
1	-(4+43)	17	175+03	33
2	-(2+51)	18	184+30	34
3	1+66	19	222+95.2	35
4	24+54	20	251+60	36
5	28+71	21	267+70	37
6	45+44	22	283+28	38
7	61+90	23	283+36	39
8	84+00	24	285+87	40
9	84+53	25	286+00	41
10	109+34	26	286+14	42
11	110+26	27	286+26	43
12	110+26	28	286+34	
13	149+00	29	286+55	
14	165+80	30	289+70	
15	165+80	31	291+49	
16	175+00	32	291+86	

EXHIBIT A-7.2

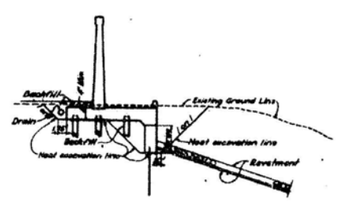
WAR DEPARTMENT — CORPS OF ENGINEERS, U.S. ARMY



GENERAL PLAN
Scale: 1" = 50'-0"



TYPICAL CROSS SECTION THRU WALL
(Showing Excavation and Backfill Lines Sta. 1+35 to Sta. 5+00)
Scale: 1" = 10'



TYPICAL CROSS SECTION THRU WALL
(Showing Excavation and Backfill Lines)
Scale: 1" = 10'

WALL ALIGNMENT DATA

STATION	DISTANCE	AZIMUTH	COORDINATE
0+00			1 027 012.16 / 490 588.06
	12.0'	32° 34' 16"	
0+12			1 027 010.06 / 490 599.92
	114.0'	90° 00' 00"	
1+26			1 027 010.00 / 490 707.92
	210.0'	90° 00' 00"	
3+36			1 027 010.00 / 490 917.92
	84.0'	90° 00' 00"	
4+20			1 027 010.13 / 491 001.88
	80.0'	90° 00' 00"	
5+00			1 027 010.25 / 491 081.88
	84.0'	90° 00' 00"	
5+84			1 027 010.37 / 491 165.88
	256.0'	90° 17' 20"	
8+40			1 027 015.94 / 491 420.18
	12.0'	90° 00' 00"	
8+52			1 027 015.88 / 491 431.18
	136.0'	90° 00' 00"	
10+28			1 027 015.75 / 491 567.17
	243.0'	90° 00' 00"	
12+68			1 027 015.94 / 491 810.16
	168.0'	90° 00' 00"	
14+36			1 027 015.88 / 491 977.16
	10.0'	90° 00' 00"	
14+46			1 027 015.88 / 491 987.16

Note: All Stationing measured on ℄ of Vertical Wall

The contours shown hereon are for existing ground surface. For details of finished slopes and revetment riverward from wall see drawings A-10-245, A-10-246 & A-10-247.

General Notes:
This drawing shows the general location of the flood wall to be constructed under this contract, and the principal existing structures in the vicinity of the wall.

AS BUILT
JANUARY 1945

NO.	DATE	BY	REVISIONS	APPD.

KANSAS CITY FLOOD CONTROL PROJECT

FAIRFAX – JERSEY CREEK

FLOOD WALL AT WATERWORKS
GENERAL PLAN AND SECTION

In 1 Sheet — Sheet No. 1 — Scale: as shown

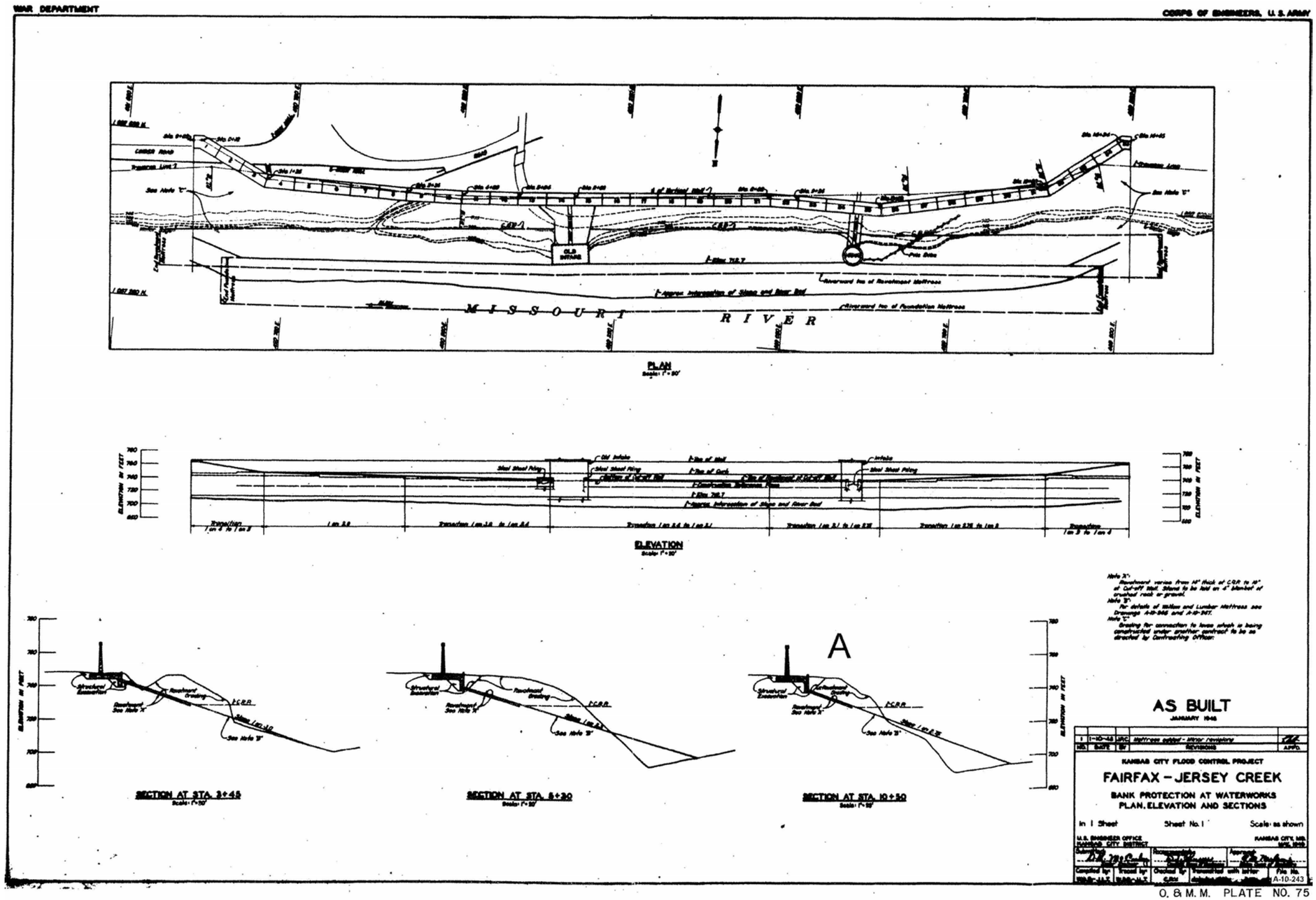
U. S. ENGINEER OFFICE, MISSOURI RIVER DIVISION — KANSAS CITY, MO. MAY, 1945

Submitted: [illegible] — Approved: [illegible]

Drawn by [illegible] — Transferred with letter dated [illegible] — File No. KC-2-2-1

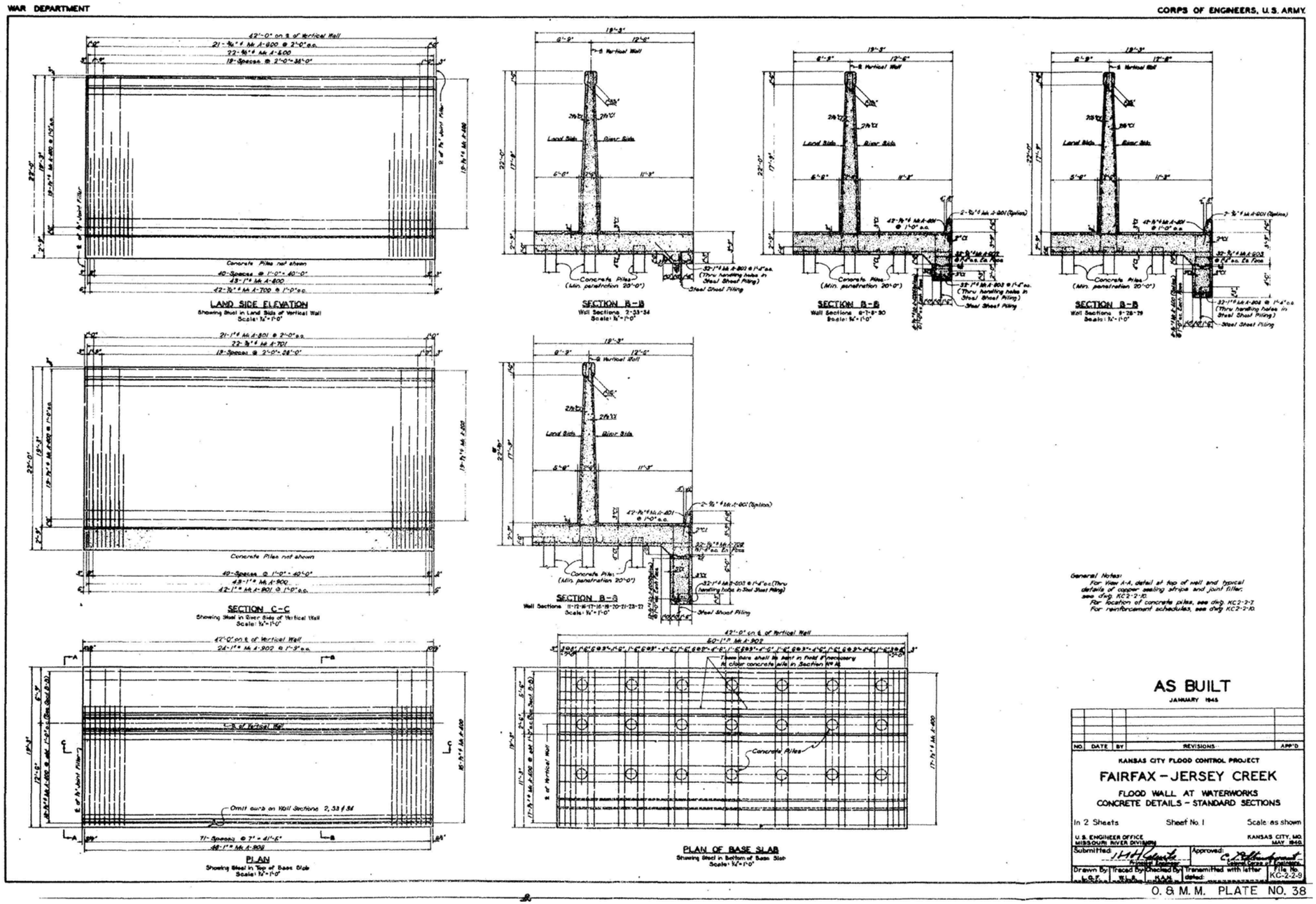
O. & M. M. PLATE NO. 30

EXHIBIT A-7.3



O. & M. M. PLATE NO. 75

EXHIBIT A-7.4


WAR DEPARTMENT
CORPS OF ENGINEERS, U.S. ARMY
LAND SIDE ELEVATION
Showing Steel in Land Side of Vertical Wall
Scale: 1/4"=1'-0"
SECTION B-B
Wall Sections 2-33-34
Scale: 1/4"=1'-0"
SECTION B-B
Wall Sections 9-17-18-30
Scale: 1/4"=1'-0"
SECTION B-B
Wall Sections 9-28-29
Scale: 1/4"=1'-0"
SECTION C-C
Showing Steel in River Side of Vertical Wall
Scale: 1/4"=1'-0"
SECTION B-B
Wall Sections 11-12-16-17-18-19-20-21-23-27
Scale: 1/4"=1'-0"
PLAN
Showing Steel in Top of Base Slab
Scale: 1/4"=1'-0"
PLAN OF BASE SLAB
Showing Steel in Bottom of Base Slab
Scale: 1/4"=1'-0"
General Notes:
For View A-A, detail at top of wall and typical details of copper sealing strips and joint filler; see dwg. KC2-2-12.
For location of concrete piles, see dwg. KC2-2-7.
For reinforcement schedules, see dwg. KC2-2-10.
AS BUILT
JANUARY 1945
NO. DATE BY REVISIONS APP'D
KANSAS CITY FLOOD CONTROL PROJECT
FAIRFAX - JERSEY CREEK
FLOOD WALL AT WATERWORKS
CONCRETE DETAILS - STANDARD SECTIONS
In 2 Sheets
Sheet No. 1
Scale: as shown
U.S. ENGINEER OFFICE
MISSOURI RIVER DIVISION
KANSAS CITY, MO.
MAY 1946
Submitted:
Approved:
Drawn By
Traced By
Checked By
Transmitted with letter dated:
File No. KC-2-2-9
O. & M.M. PLATE NO. 38

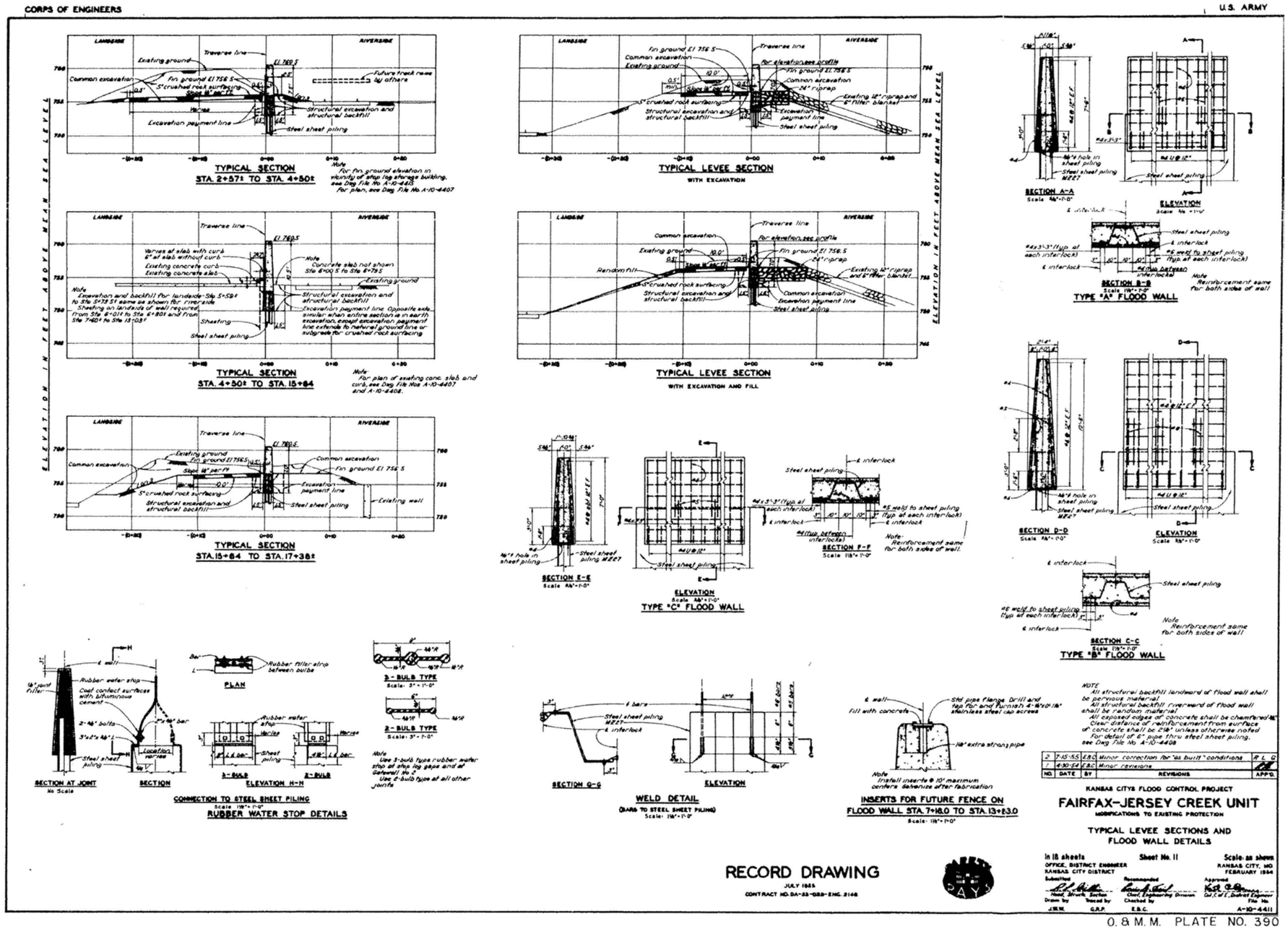

CORPS OF ENGINEERS
U.S. ARMY
TYPICAL SECTION
STA. 2+57± TO STA. 4+50±
TYPICAL SECTION
STA. 4+50± TO STA. 15+84
TYPICAL SECTION
STA. 15+84 TO STA. 17+38±
TYPICAL LEVEE SECTION
WITH EXCAVATION
TYPICAL LEVEE SECTION
WITH EXCAVATION AND FILL
SECTION A-A
ELEVATION
TYPE "A" FLOOD WALL
SECTION E-E
ELEVATION
TYPE "C" FLOOD WALL
SECTION F-F
SECTION D-D
ELEVATION
TYPE "B" FLOOD WALL
SECTION C-C
SECTION AT JOINT
SECTION
PLAN
ELEVATION H-H
CONNECTION TO STEEL SHEET PILING
RUBBER WATER STOP DETAILS
SECTION G-G
ELEVATION
WELD DETAIL
INSERTS FOR FUTURE FENCE ON
FLOOD WALL STA. 7+16.0 TO STA. 13+83.0
KANSAS CITY'S FLOOD CONTROL PROJECT
FAIRFAX-JERSEY CREEK UNIT
TYPICAL LEVEE SECTIONS AND
FLOOD WALL DETAILS
RECORD DRAWING
O. & M. M. PLATE NO. 390

EXHIBIT A-7.6

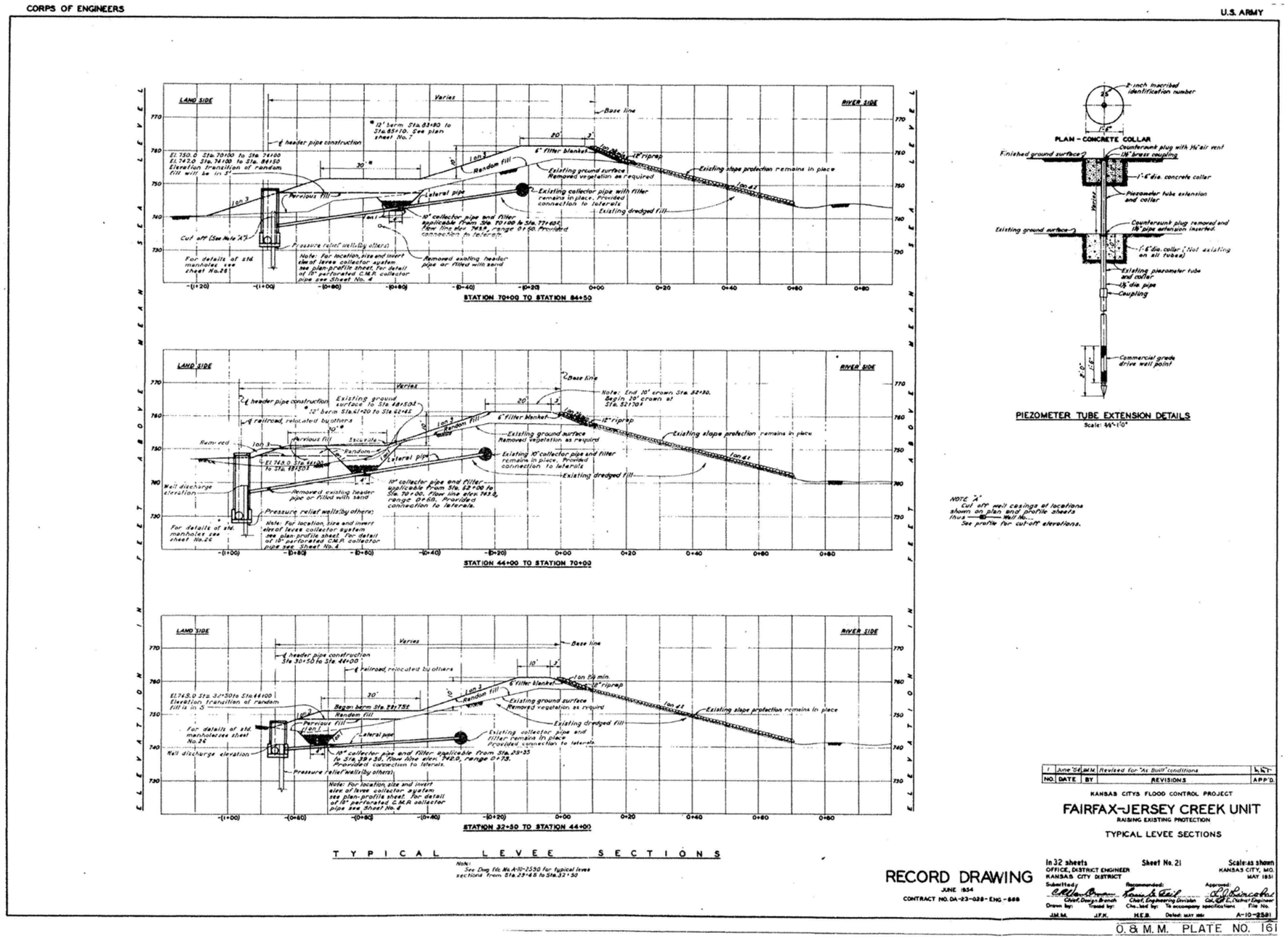

CORPS OF ENGINEERS
U.S. ARMY
LAND SIDE
RIVER SIDE
STATION 70+00 TO STATION 84+50
STATION 44+00 TO STATION 70+00
STATION 32+50 TO STATION 44+00
PLAN - CONCRETE COLLAR
PIEZOMETER TUBE EXTENSION DETAILS
TYPICAL LEVEE SECTIONS
RECORD DRAWING
KANSAS CITY'S FLOOD CONTROL PROJECT
FAIRFAX-JERSEY CREEK UNIT
RAISING EXISTING PROTECTION
TYPICAL LEVEE SECTIONS
O. & M. M. PLATE NO. 161

EXHIBIT A-7.7

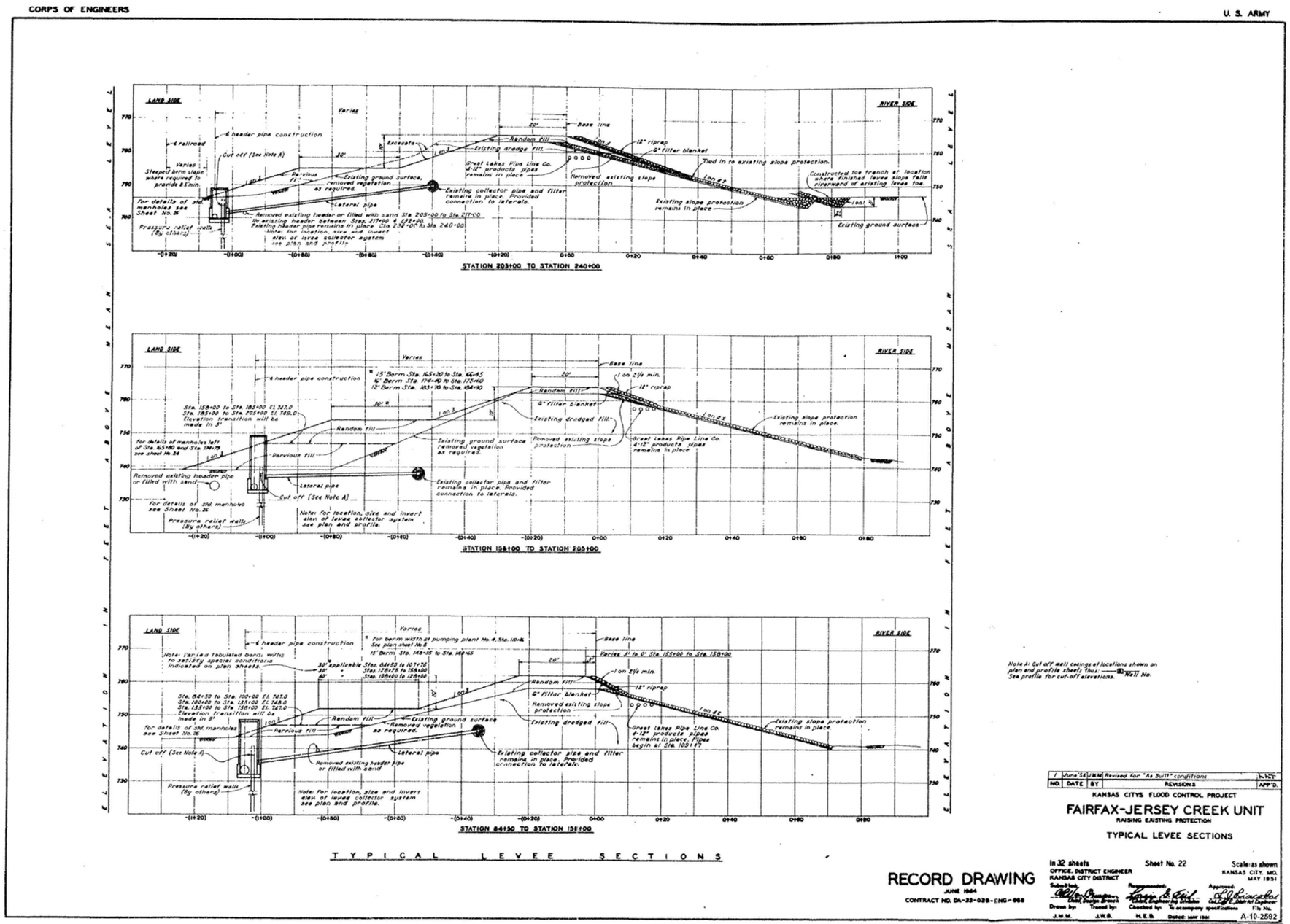
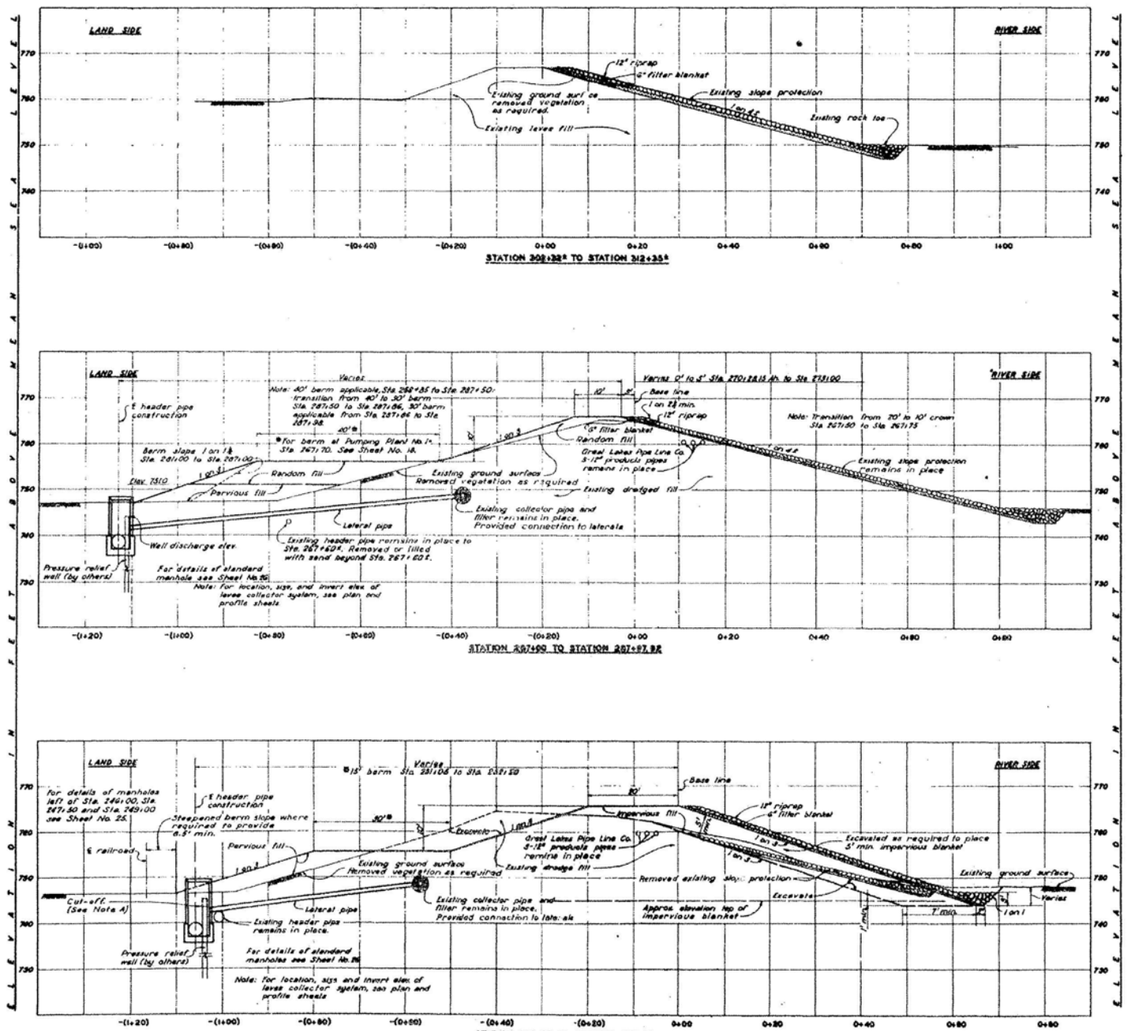

CORPS OF ENGINEERS
U. S. ARMY
LAND SIDE
RIVER SIDE
ELEVATION IN FEET ABOVE MEAN SEA LEVEL
STATION 203+00 TO STATION 240+00
STATION 158+00 TO STATION 205+00
STATION 84+50 TO STATION 158+00
Base line
Random fill
Existing dredge fill
12" riprap
6" filter blanket
Existing slope protection remains in place
Existing ground surface
Great Lakes Pipe Line Co. 4-12" products pipes remains in place
Removed existing slope protection
Existing collector pipe and filter remains in place. Provided connection to laterals.
Lateral pipe
Cut off (See Note A)
Pervious fill
Pressure relief walls (By others)
Note: For location, size and invert elev. of levee collector system see plan and profile.
TYPICAL LEVEE SECTIONS
RECORD DRAWING
JUNE 1964
CONTRACT NO. DA-23-028-ENG-868
KANSAS CITYS FLOOD CONTROL PROJECT
FAIRFAX-JERSEY CREEK UNIT
RAISING EXISTING PROTECTION
TYPICAL LEVEE SECTIONS
Sheet No. 22
Scale: as shown
KANSAS CITY, MO.
MAY 1951
A-10-2592
O. & M. M. PLATE NO. 162

EXHIBIT A-7.8

CORPS OF ENGINEERS

U.S. ARMY



Note A:
Cut off well casings at locations shown on plan and profile sheets thus ——— Well No.
See profile for cut-off elevations.

TYPICAL LEVEE SECTIONS

RECORD DRAWING
JUNE 1954
CONTRACT NO. DA-23-038-ENG-888

NO.	DATE	BY	REVISIONS	APP'D.
1	June 54	JMM	Revised for "As Built" conditions	

KANSAS CITY'S FLOOD CONTROL PROJECT

FAIRFAX - JERSEY CREEK UNIT

RAISING EXISTING PROTECTION

TYPICAL LEVEE SECTIONS

In 32 sheets — Sheet No. 23 — Scale: as shown

OFFICE, DISTRICT ENGINEER
KANSAS CITY DISTRICT
KANSAS CITY, MO.
MAY 1951

Submitted: Chief, Design Branch
Recommended: Chief, Engineering Division
Approved: Col. C.E., District Engineer

Drawn by: J.M.M. Traced by: H.E.S. Checked by: H.E.B. To accompany specification: Dated: MAY 1951 File No. A-10-2593

O. & M.M. PLATE NO. 163

EXHIBIT A-7.9

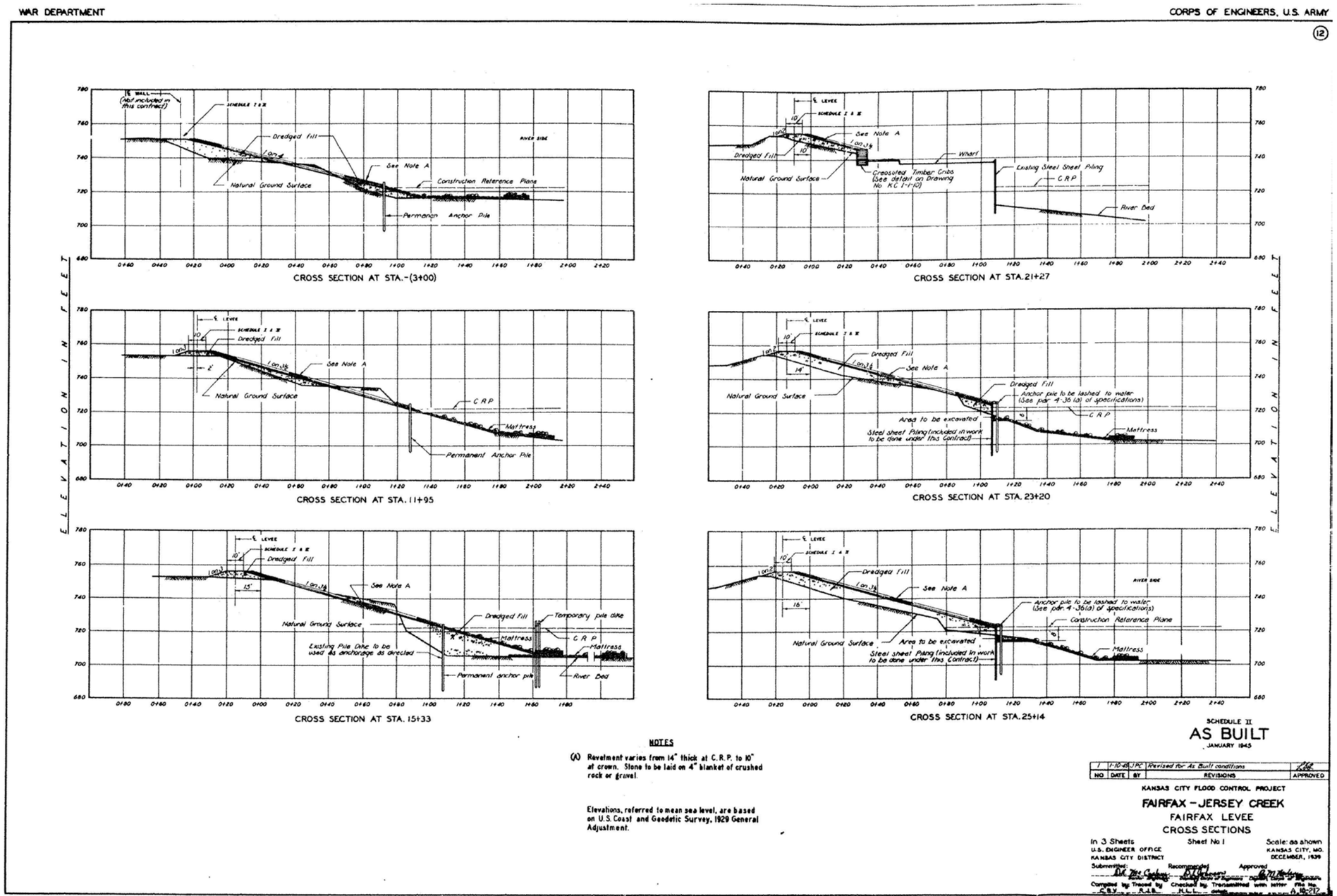
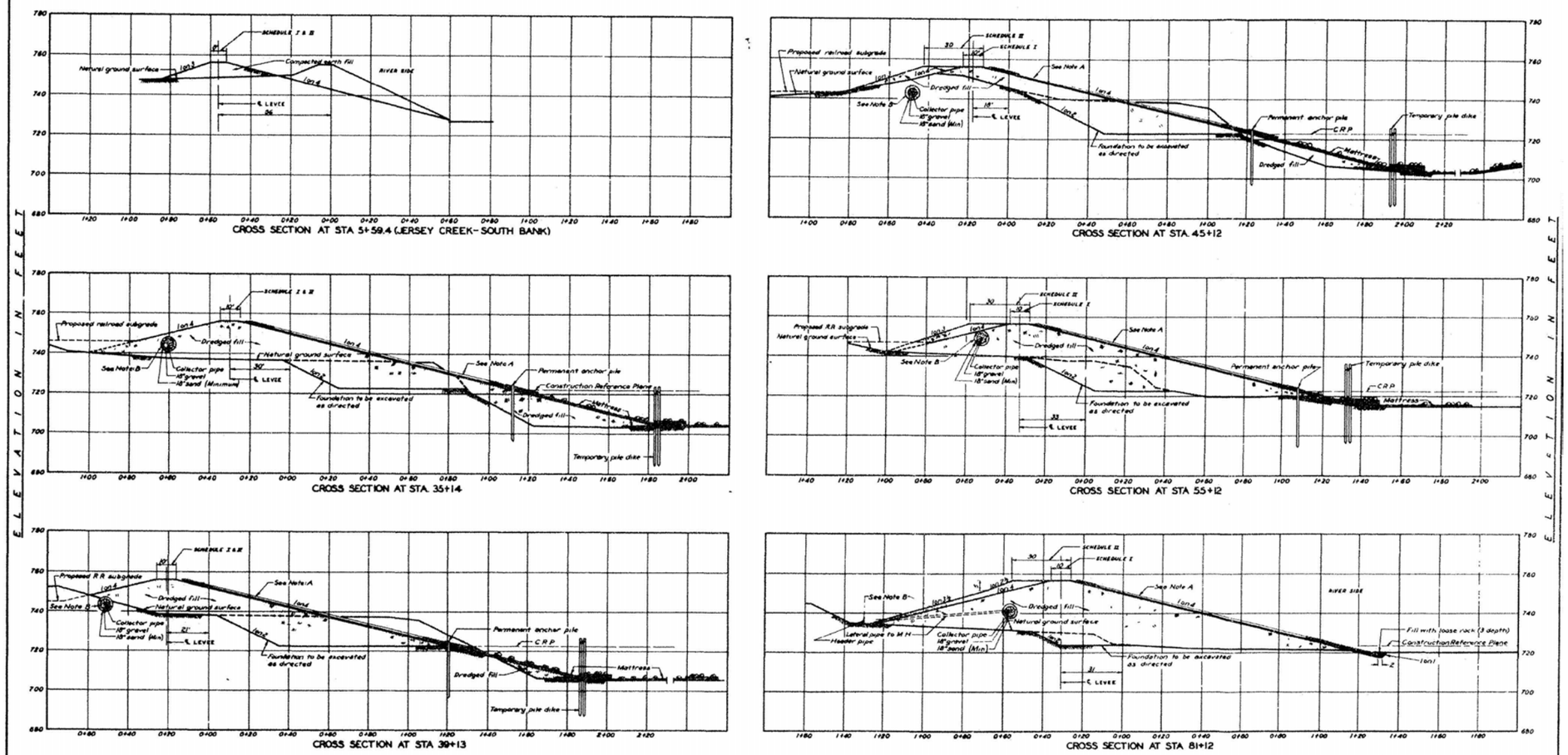

WAR DEPARTMENT
CORPS OF ENGINEERS, U.S. ARMY
12
ELEVATION IN FEET
CROSS SECTION AT STA.-(3+00)
CROSS SECTION AT STA. 21+27
CROSS SECTION AT STA. 11+95
CROSS SECTION AT STA. 23+20
CROSS SECTION AT STA. 15+33
CROSS SECTION AT STA. 25+14
Dredged Fill
Natural Ground Surface
See Note A
Construction Reference Plane
Permanent Anchor Pile
Wharf
Existing Steel Sheet Piling
C.R.P.
River Bed
Mattress
Temporary pile dike
Existing Pile Dike to be used as anchorage as directed
Area to be excavated
Steel sheet Piling (included in work to be done under this Contract)
Anchor pile to be lashed to water (See par. 4-36(a) of specifications)
NOTES
(A) Revetment varies from 14" thick at C.R.P. to 10" at crown. Stone to be laid on 4" blanket of crushed rock or gravel.
Elevations, referred to mean sea level, are based on U.S. Coast and Geodetic Survey, 1929 General Adjustment.
SCHEDULE II
AS BUILT
JANUARY 1945
1 1-10-45 JPC Revised for As Built conditions
NO DATE BY REVISIONS APPROVED
KANSAS CITY FLOOD CONTROL PROJECT
FAIRFAX - JERSEY CREEK
FAIRFAX LEVEE
CROSS SECTIONS
In 3 Sheets
Sheet No 1
Scale: as shown
U.S. ENGINEER OFFICE
KANSAS CITY DISTRICT
KANSAS CITY, MO.
DECEMBER, 1939
Submitted:
Recommended:
Approved:
Compiled by: Traced by: Checked by: Transmitted with letter
File No.
O. & M. M. PLATE NO. 2

EXHIBIT A-7.10

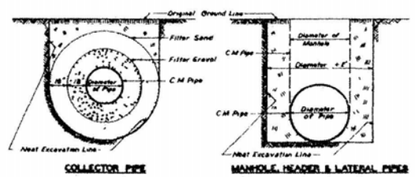
WAR DEPARTMENT

CORPS OF ENGINEERS U.S. ARMY

(13)



-NOTE-
The dimensions indicated for filter materials are the minimum allowable thicknesses. For convenience of placement, greater thicknesses will be permitted but payment will not be increased as a result of increased thickness.



COLLECTOR PIPE

MANHOLE, HEADER & LATERAL PIPES

TYPICAL NEAT EXCAVATION LINES FOR PLACEMENT OF LEVEE DRAINAGE PIPE BELOW ORIGINAL GROUND LINE

Not to Scale

NOTES

(A) Revetment varies from 14" thick at C.R.P. to 10" at crown. Stone to be laid on 4" blanket of crushed rock or gravel.

(B) Location of drainage system is based on Schedule II. For location of manholes, see Plan-profile.

Elevations, referred to mean sea level, are based on U.S. Coast and Geodetic Survey, 1929 General Adjustment.

NO.	DATE	BY	REVISIONS	APP'D.
1	1-10-55	J.P.C.	Revised for as built conditions	

SCHEDULE II

AS BUILT

JANUARY 1945

KANSAS CITY FLOOD CONTROL PROJECT

FAIRFAX - JERSEY CREEK

FAIRFAX LEVEE

CROSS SECTIONS

In 3 Sheets — Sheet No 2 — Scale as shown

U.S. ENGINEER OFFICE, KANSAS CITY DISTRICT — KANSAS CITY, MO. DECEMBER 1939

Submitted — Recommended — Approved

Compiled by C.B.V. — Traced by J.M.A. — Checked by H.L.L. — Transmitted with letter dated — File No. A-10-213

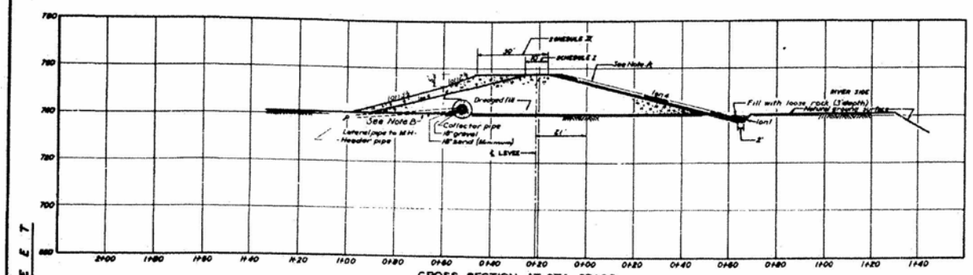
O. & M. M. PLATE NO. 3

EXHIBIT A-7.11

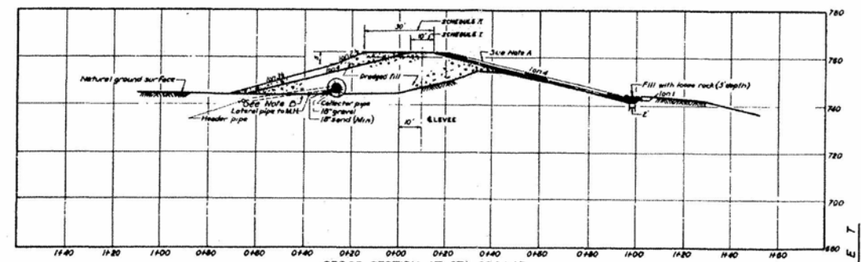
WAR DEPARTMENT

CORPS OF ENGINEERS, U.S. ARMY

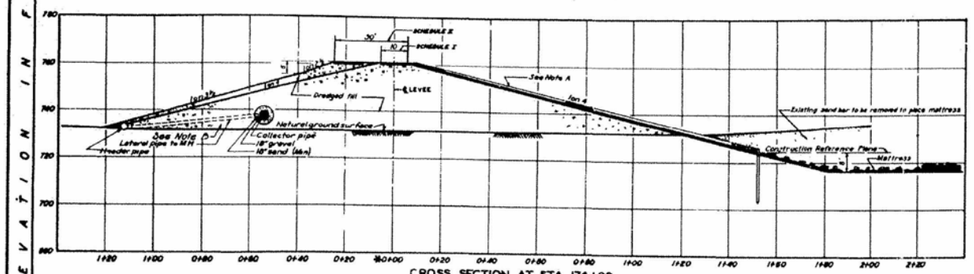
(14)



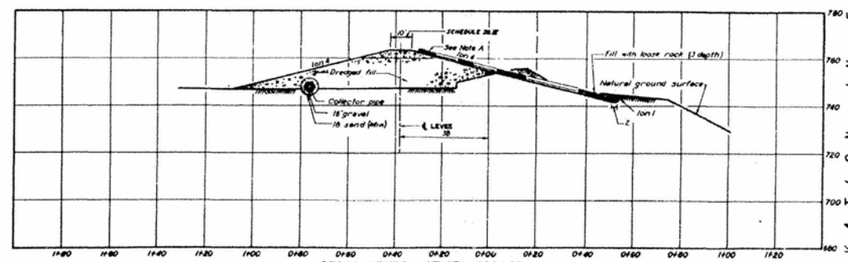
CROSS SECTION AT STA. 97+00



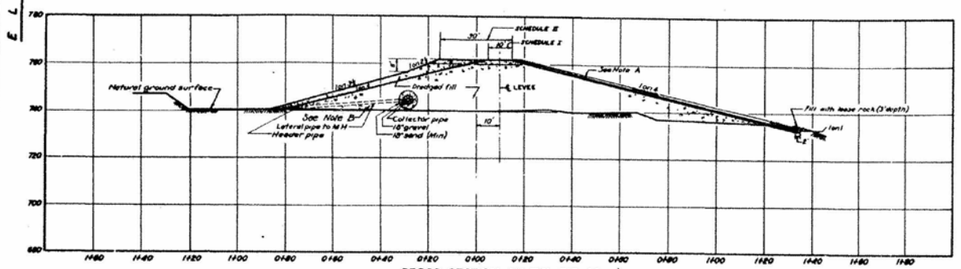
CROSS SECTION AT STA. 239+97



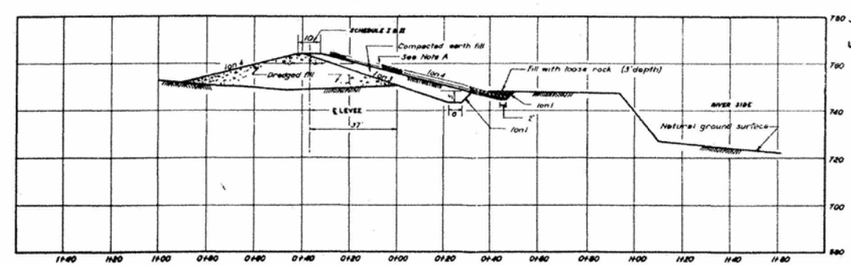
CROSS SECTION AT STA. 174+00



CROSS SECTION AT STA. 282+03



CROSS SECTION AT STA. 212+92



CROSS SECTION AT STA 308+53.23

* Note: Transverse stationing referred to ℄ of 10' crown levee

NOTES

(A) Revetment varies from 14" thick at C.R.P. to 10" at crown. Stone to be laid on 4" blanket of crushed rock or gravel.

(B) Location of drainage system is based on Schedule II. For location of manholes, see Plan-profile.

Elevations, referred to mean sea level, are based on U.S. Coast and Geodetic Survey, 1929 General Adjustment.

SCHEDULE II

AS BUILT

JANUARY 1945

KANSAS CITY FLOOD CONTROL PROJECT

FAIRFAX - JERSEY CREEK

FAIRFAX LEVEE

CROSS SECTIONS

In 3 Sheets — Sheet No. 3 — Scale as shown

U.S. ENGINEER OFFICE, KANSAS CITY DISTRICT — KANSAS CITY, MO. DECEMBER, 1939

Submitted: — Recommended: — Approved:

Compiled by: — Traced by: — Checked by: — Transmitted, print letter: — Filed Blueprint No. A-10-216

NO.	DATE	BY	REVISIONS	APP'D.
1	1-18-45	J.E.C.	Revised for as built conditions	

O. & M.M. PLATE NO. 4

Site Characterization Maps and Boring Information

1

(D-1142)
S-Gap7

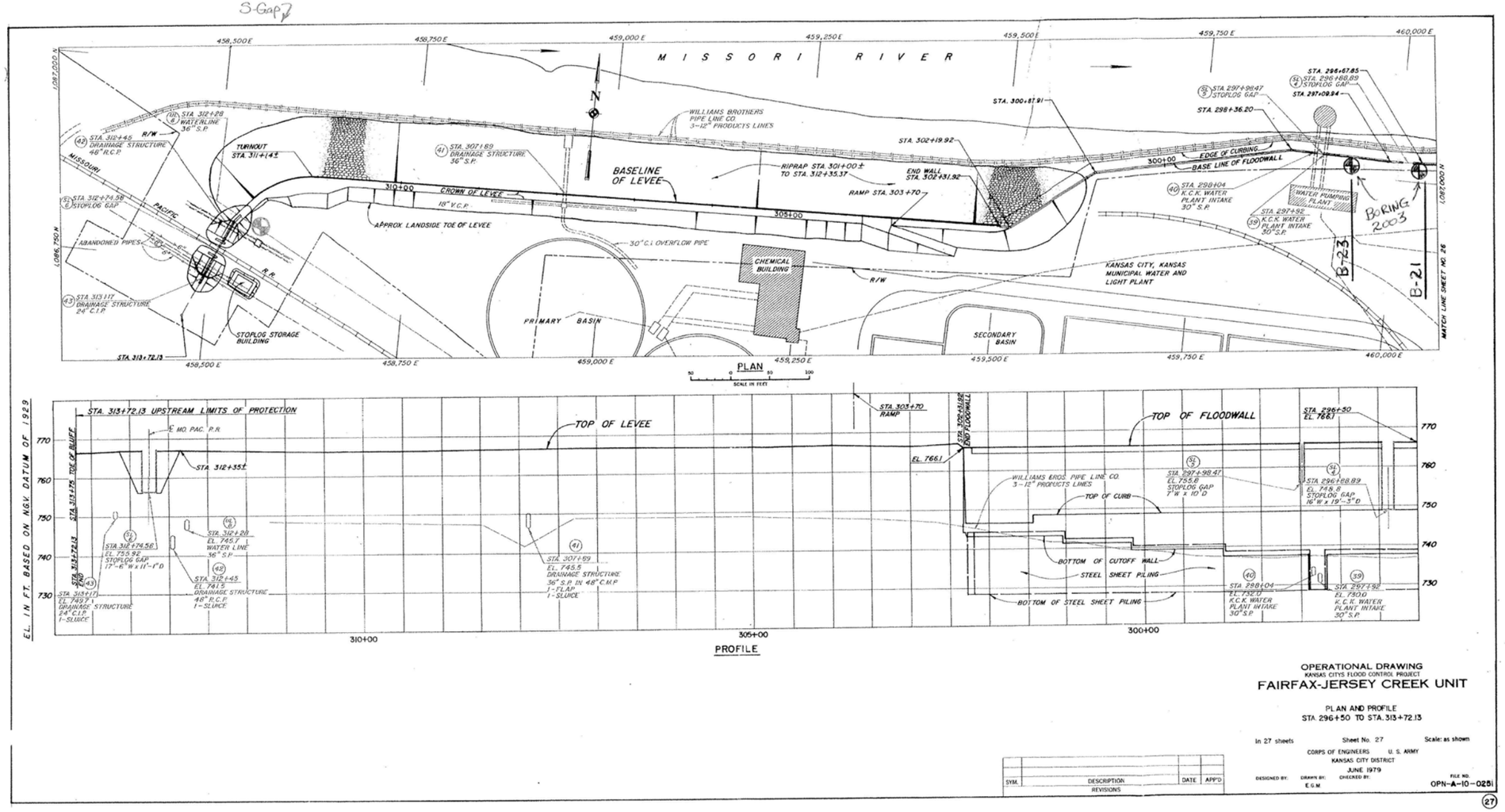
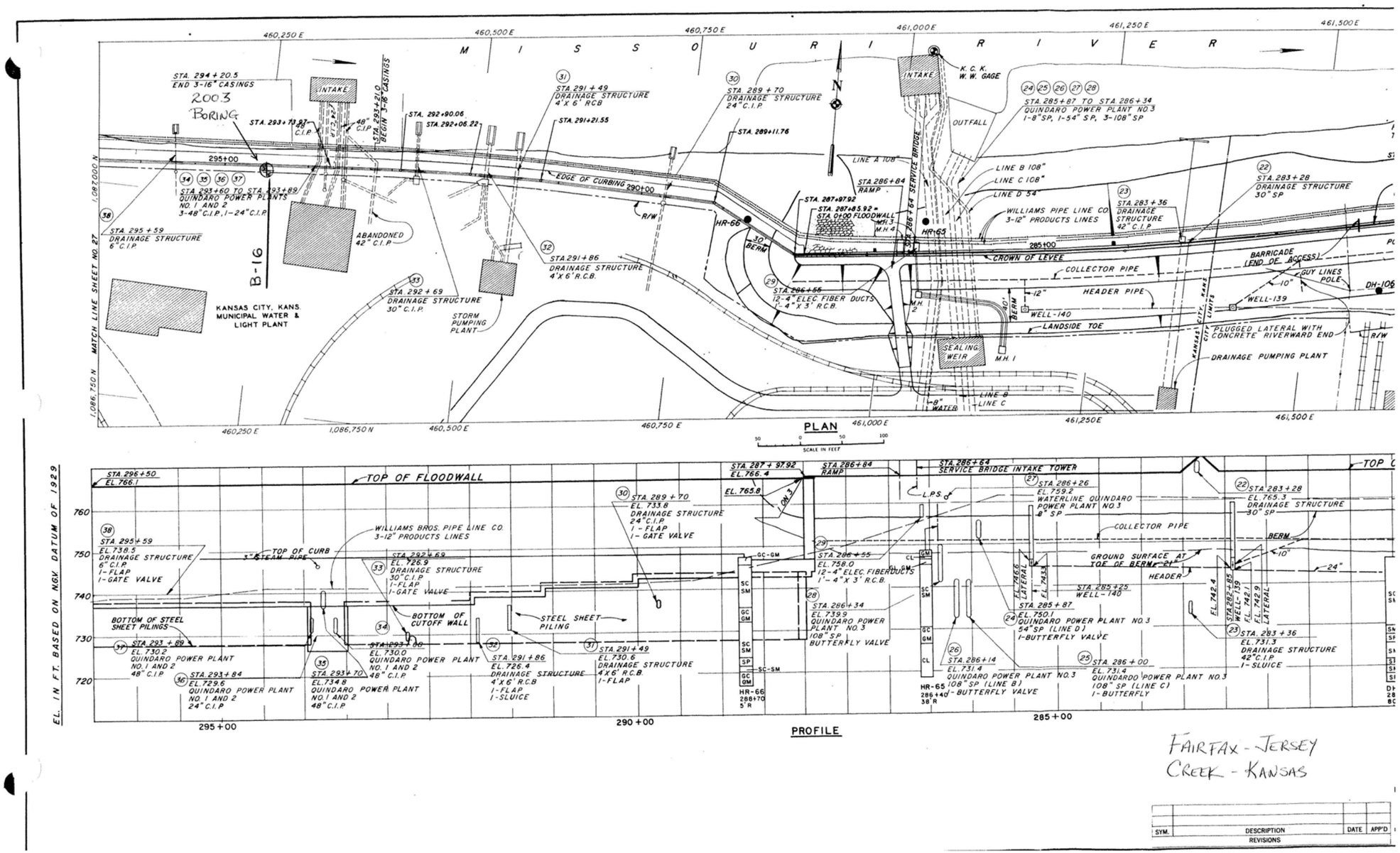


EXHIBIT A-7.13

1



FAIRFAX - JERSEY CREEK - KANSAS

SYM.	DESCRIPTION	DATE	APP'D
	REVISIONS		

EXHIBIT A-7.14
Flood Wall Pile Investigation Report

Fairfax Drainage District

Flood Wall Pile Investigation Report

October 20, 2003

U.S. Army Corps of Engineers, Kansas City District

Contract No. DACW 41-02-D-0006

By

Black & Veatch Special Projects

Overland Park, Kansas

Attachment A
Fairfax Flood Wall
Pile Investigation Report

SUBSURFACE EXPLORATION AND GEOPHYSICAL SURVEYS FAIRFAX FLOOD CONTROL SRUCTURE KANSAS CITY, KANSAS

Prepared for:

U.S. ARMY CORPS OF ENGINEERS,
KANSAS CITY DISTRICT
C/O BLACK & VEATCH
Overland Park, Kansas

Prepared by:

GEOTECHNOLOGY, INC.
Overland Park, Kansas

Geotechnology Job No. 0713201.3211

October 15, 2003

7132-fairfax-full.doc

SUBSURFACE EXPLORATION AND GEOPHYSICAL SURVEYS
FAIRFAX FLOOD CONTROL SRUCTURE
KANSAS CITY, KANSAS

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0713201.3211

SUBSURFACE EXPLORATION AND GEOPHYSICAL SURVEYS
FAIRFAX FLOOD CONTROL SRUCTURE
KANSAS CITY, KANSAS

SECTION I - EXECUTIVE SUMMARY

The executive summary is provided solely for the purposes of overview. The executive summary omits a number of details, any one of which could be crucial to the proper application of this report. Any party who relies on this report must read the full report.

- The project includes conducting an investigation to determine the length of existing piles and the subsurface soil conditions at the Fairfax flood control wall at Monoliths 16, 21 and 23.

- Below the surficial topsoil and silty clay fill, the soil stratigraphy general consists of silty sand underlain by fine to coarse sand.

- After subsurface exploration, the borings were enlarged to install 4-inch diameter PVC pipes for geophysical surveys. Three geophysical methods including parallel seismic, ground penetrating radar (GPR) and magnetic gradient were utilized: The distance from top of pile caps to the bottom of piles were between 21 to 25 feet and the reinforcing steel is between 10 to 13 feet below the top of the pile caps.

- The bottom of the pile cap and the upper 12 to 24 inches of a pile at Monolith 21 were exposed at the face of the excavation to determine the pile shape and cross-sectional dimensions. The exposed pile shows the existing piles are approximately 14-inch diameter, cast-in-place concrete piles (based on a measured pile circumference of 48 inches).

SECTION II - PROJECT DATA

AUTHORIZATION

The services documented in this report were provided in accordance with the terms, conditions and scope of services described in Geotechnology's August 7, 2003 proposal.

PURPOSE AND SCOPE OF SERVICES

The purpose of our services was to conduct subsurface exploration and geophysical surveys for the investigation of the existing piles as defined in our proposal. Briefly, services consisted of site reconnaissance, drilling three borings, laboratory testing, geophysical surveys and data interpretation, and preparation of this report. Important information prepared by The

Association of Engineering Firms Practicing in the Geosciences (ASFE) for studies of the type is included in Appendix A for your review.

PROJECT AND SITE DESCRIPTION

This project included an assessment of the soil conditions adjacent to the existing piling and a determination of the pile length for the Fairfax Drainage District Floodwall in Kansas City, Kansas. The shape and cross sectional dimensions of the existing piles were examined by exposing the pile cap and the upper 12 to 24 inches of pile at Monolith 21, and by exposing the pile caps only at Monoliths 16 and 23. The pile lengths were determined using a total of three types of Borehole Geophysical Surveys (parallel seismic, ground penetrating radar, and magnetic gradient) at Monoliths 16, 21 and 23. This investigation consisted of exposing the pile caps to permit a hammer strike at the top of the concrete to generate a signal source for performing the parallel seismic surveys and performing subsurface explorations. The subsurface explorations included drilling three 50-foot deep bore holes, installing 4-inch diameter PVC pipes, and associated down-hole geophysical surveys. The bore holes were located in the BPU power plant and were drilled adjacent to the existing pile caps.

SECTION III - FIELD EXPLORATION AND LABORATORY TESTING

FIELD EXPLORATION

The field subsurface exploration consisted of drilling and sampling three borings, designated as Borings B-16, B-21 and B-23, at the approximate locations at Monoliths 16, 21 and 23. All soil borings were drilled and sampled at the above referenced project site from September 22 through 25, 2003,. The borings were located in the field by the client.

All borings were drilled to a depth of 50 feet and were drilled using a CME 55 rotary drill rig. The boreholes were drilled using 4-inch flight augers to a depth of 15 feet with rotary wash drilling methods used from a depth of 15 feet to the boring termination depth of 50 feet below ground surface. Standard Penetration Tests (SPT's) were conducted after encountering natural soil deposits (approximately 10 to 13 feet below the ground surface) and using an automatic hammer. Split-spoon samples were obtained at intervals of 2.5 feet to the bottom of each boring. Detailed logs of borings are presented in Appendix B. A bulk sample was obtained during the excavation of the pile caps to determine compaction requirements for the backfilling operation.

An engineer of Geotechnology provided technical direction during field exploration, observed drilling and sampling, assisted in obtaining samples, and prepared descriptive logs of the material encountered. The boring logs represent conditions observed at the time of exploration but have been edited as a result of the laboratory test data as appropriate.

SET UP FOR GEOPHYSICAL SURVEYS

After finishing soil sampling at each boring, the bore holes were re-drilled and enlarged by using 6-1/4 inches inside diameter (I.D.) hollow stem augers to a depth of 15 feet from the ground surface. Eight inch diameter PVC pipe was installed to an approximate depth of 17 feet below the ground surface for use as temporary casing. After the temporary casing was installed, the bore hole was drilled to the termination depth using rotary wash drilling methods and a 7-7/8 inch diameter tri-cone roller bit. Access pipes for geophysical surveys consisting of 4-inch diameter PVC pipes were installed. The annular space between the bore hole and the 4-inch pipe was grouted. The access pipes were backfilled with grout after conducting three geophysical surveys.

LABORATORY TESTING

Laboratory testing was performed on the soil samples to determine index properties of the soil and recommended shear strength parameters for re-analysis of pile capacity. Moisture contents, Atterberg limits tests and sieve analyses were performed on selected samples. Results of these laboratory tests are presented on the boring logs and Appendix C. A proctor test was performed on the bulk sample. Summary results are presented in Appendix C.

SECTION IV - SUBSURFACE CONDITIONS

STRATIGRAPHY

Below the surficial 6 inches of top soil, generally 9.5 to 12.5 feet of silty clay fill overlies 10 to 18 feet of very loose to medium dense brown to gray silty sand with intermediate layers of soft, brown and gray clayey silt and silty clay. Below the silt sand, medium dense to dense gray to dark gray silty sand and fine to coarse poorly-graded and well-graded sand were encountered to boring termination at a depth of 50 feet.

GROUNDWATER

Groundwater levels were not observed in the borings due to rotary wash drilling methods being used. No groundwater was encountered while drilling with the flight augers to a depth of 15 feet below ground surface.

RECOMMENDED SHEAR STRENGTH PARAMETERS

Soil strength parameters for long-term conditions, including cohesion and angle of internal friction were estimated based on the in-situ SPT values. Buoyant unit weight should be used in any computation of allowable pile capacity based on the assumption that the design

APPENDIX B

DETAILED LOGS OF BORINGS B-16, B-21 AND B-23
BORING LOG: TERMS AND SYMBOLS

Surface Elevation _____ Completion Date: 9/24/03

Datum _____

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Topsoil - 6 inches			
	FILL : brown, silty SAND, with brick and concrete fragments			
5				
10				
	Very loose to loose, gray, silty SAND to sandy SILT - SM to ML		4-2-2	SS1
15			2-1-2	SS2
	Soft, gray, silty CLAY, trace fine sand - CL			
	Loose to very loose, gray, silty SAND - SM		5-5-3	SS3
20			4-4-4	SS4
	layers of brown silty CLAY and clayey SAND		2-0-2	SS5
25				
	Loose, gray, poorly-graded fine to medium, SAND, trace layers of gray silty sand - (SP)		5-3-2	SS6
			4-7-7	SS7

SHEAR STRENGTH, tsf

Δ - UU/2 O - QU/2 □ - SV

0.5 1.0 1.5 2.0 2.5

STANDARD PENETRATION RESISTANCE

(ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %

PL ⊢ ● ⊣ LL

10 20 30 40 50

NOTE: STRATIFICATION LINES REPRESENT AND THE TRANSITION MAY BE GRADUAL.

MATE BOUNDARIES BETWEEN SOIL TYPES LOG FOR ILLUSTRATION PURPOSES ONLY.

LOG OF BORING 2002 132-FAIRFAX.GPJ GTINC 0533301.GPJ 10/16/03

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

6.25 AUGER __ HOLLOW STEM

WASHBORING FROM 13.5 FEET

DWB DRILLER YAW LOGGER

CME 55HT DRILL RIG

HAMMER TYPE Auto

REMARKS:

Drawn by: yaw	Ck'd. by: yaw	App'vd. by: WLC
Date: 9/25/03	Date: 10/15/03	10/16/03

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Fairfax Flood Control Structure

LOG OF BORING: B-16

Project No. 0713201.3211

Surface Elevation _____ Completion Date: 9/24/03

Datum _____

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Medium dense to dense, gray, poorly-graded medium to coarse, SAND - SP *(continued)*			
			8-9-8	SS8
			7-10-22	SS9
35	Medium dense, gray, poorly-graded fine to medium, SAND - SP			
			8-9-8	SS10
	Meidum dense , gray, poorly-graded medium to coarse, SAND - SP			
			7-10-10	SS11
40				
			9-9-10	SS12
	Medium dense to dense, gray, poorly-graded fine, SAND with silt - (SP-SM)			
			15-22-15	SS13
45				
	Medium dense, gray, poorly-graded fine to medium, SAND - SP		7-12-11	SS14
			7-10-10	SS15
50	Boring terminated at 50 feet			
55				

SHEAR STRENGTH, tsf

Δ - UU/2 O - QU/2 □ - SV

0.5 1.0 1.5 2.0 2.5

STANDARD PENETRATION RESISTANCE

(ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %

PL 10 20 30 40 50 LL

NOTE: STRATIFICATION LINES REPRESENT ...IMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL ...IC LOG FOR ILLUSTRATION PURPOSES ONLY

LOG OF BORING 2002 v.4 7132-FAIRFAX.GPJ GTINC 063301.GPJ 10/16/03

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

6.25 AUGER ___ HOLLOW STEM

WASHBORING FROM 13.5 FEET

DWB DRILLER YAW LOGGER

CME 55HT DRILL RIG

HAMMER TYPE Auto

REMARKS:

Drawn by: yaw | Ck'd. by: yaw | App'vd. by: [illegible]

Date: 9/25/03 | Date: 10/15/03 | [illegible]

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Fairfax Flood Control Structure

CONTINUATION OF LOG OF BORING: B-16

Project No. 0713201.3211

Surface Elevation _____ Completion Date: 9/22/03

Datum _____

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Topsoil - 6 inches			
	FILL : brown, silty SAND with brick and concrete fragments and toe drain			
5				
10	Very loose to medium dense, brown to gray, silty SAND - SM		3-2-2-3	SS1
			20-7-5	SS2
15			8-2-3	SS3
	Stiff, brown to gray, sandy SILT - ML		2-6-4	SS4
20	Medium dense, brown to gray, silty SAND - SM		9-9-9	SS5
	Medium dense, brown to gray, poorly-graded fine, SAND with silt - SP-SM		6-8-7	SS6
25			7-10-10	SS7
	Medium dense, gray, poorly-graded fine, SAND, trace silt - SP		7-7-11	SS8

SHEAR STRENGTH, tsf

Δ - UU/2 O - QU/2 □ - SV

0.5 1.0 1.5 2.0 2.5

STANDARD PENETRATION RESISTANCE

(ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %

PL ⊢ ● ⊣ LL

10 20 30 40 50

NOTE: STRATIFICATION LINES REPRESENT ... ATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. G... LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

6.25 AUGER ___ HOLLOW STEM

WASHBORING FROM 10 FEET

DWB DRILLER TAW LOGGER

CME 55HT DRILL RIG

HAMMER TYPE Auto

REMARKS:

Drawn by: yaw	Ck'd. by: yaw	App'vd. by: [illegible]
Date: 9/25/03	Date: 10/15/03	10/16/03

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Fairfax Flood Control Structure

LOG OF BORING: B-21

Project No. 0713201.3211

LOG OF BORING 2002 ... 132-FAIRFAX.GPJ GT/NC 0633301.GPJ 10/16/03

Surface Elevation _____ Completion Date: 9/22/03

Datum _____

DEPTH IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Medium dense, gray, poorly-graded fine, SAND, trace silt - SP *(continued)*	6-10-10	SS9
		12-11-8	SS10
35	Medium dense, brown, silty SAND - SM	5-6-7	SS11
	Dense, brown, poorly-graded fine SAND - SP	19-19-23	SS12
40		5-16-15	SS13
		13-20-16	SS14
45		19-20-13	SS15
	Dense, brown, poorly-graded fine to medium SAND - SP	6-10-11	SS16
50	Boring terminated at 50 feet		
55			

GRAPHIC LOG

SHEAR STRENGTH, tsf

Δ - UU/2 O - QU/2 D - SV

0.5 1.0 1.5 2.0 2.5

STANDARD PENETRATION RESISTANCE

(ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %

PL 10 20 30 40 50 LL

NOTE: STRATIFICATION LINES REPRESENT THE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. ...G FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

6.25 AUGER ___ HOLLOW STEM

WASHBORING FROM 10 FEET

DWB DRILLER TAW LOGGER

CME 55HT DRILL RIG

HAMMER TYPE Auto

REMARKS:

Drawn by: yaw | Ck'd. by: Yaw | App'vd. by LW

Date: 9/25/03 | Date: 10/15/03 | 10/15/03

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Fairfax Flood Control Structure

CONTINUATION OF LOG OF BORING: B-21

Project No. 0713201.3211

LOG OF BORING 2002 - 32-FAIRFAX.GPJ GTINC 0538301.GPJ 10/16/03

Surface Elevation _____ Completion Date: 9/23/03

Datum _____

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Topsoil - 6 inches			
	FILL : brown, silty sand			
5				
10				
	Soft, brown, silty CLAY, trace fine sand - (CL)		1-2-1	SS1
15			0-0-4	SS2
	Soft, brown, clayey SILT - ML		1-3-4	SS3
20	Loose, brown, silty SAND - SM		4-5-5	SS4
	Loose, brown, poorly-graded medium, SAND, trace gray sandy silt - SP		3-2-2	SS5
25	Dense to medium dense, gray, poorly-graded fine to medium, SAND - SP		14-16-16	SS6
			7-9-8	SS7

SHEAR STRENGTH, tsf

Δ - UU/2 O - QU/2 □ - SV

0.5 1.0 1.5 2.0 2.5

STANDARD PENETRATION RESISTANCE

(ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %

PL ⊢ ● ⊣ LL

10 20 30 40 50

NOTE: STRATIFICATION LINES REPRESENT ... MATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL ... LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

6.25 AUGER ___ HOLLOW STEM

WASHBORING FROM 13.5 FEET

DWB DRILLER YAW LOGGER

CME 55HT DRILL RIG

HAMMER TYPE Auto

REMARKS:

Drawn by: yaw	Ck'd. by: yaw	App'vd. by: [illegible]
Date: 9/25/03	Date: 10/15/03	[illegible]

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Fairfax Flood Control Structure

LOG OF BORING: B-23

Project No. 0713201.3211

LOG OF BORING 2002 ... 132-FAIRFAX.GPJ GTINC 0638301.GPJ 10/16/03

Surface Elevation _____ Completion Date: 9/23/03

Datum _____

DEPTH IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
	Dense to medium dense, gray, poorly-graded fine to medium, SAND - SP *(continued)*		7-7-5	SS8
	Loose to medium dense, gray, poorly-graded fine, SAND, trace gray silty sand - SP		3-4-5	SS9
35			3-4-7	SS10
	Medium dense to dense, gray, poorly-graded medium, SAND with silt - SP-SM		6-7-5	SS11
40	trace fine gravel		6-16-12	SS12
			7-11-12	SS13
45			9-9-8	SS14
	Medium dense, gray, well-graded medium to coarse sand, trace fine gravel - SW		10-10-8	SS15
50	Boring terminated at 50 feet			
55				

SHEAR STRENGTH, tsf

Δ - UU/2 O - QU/2 □ - SV

0.5 1.0 1.5 2.0 2.5

STANDARD PENETRATION RESISTANCE

(ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %

PL 10 20 30 40 50 LL

NOTE: STRATIFICATION LINES REPRESENT DATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. LINES ARE FOR ILLUSTRATION PURPOSES ONLY.

LOG OF BORING 2002 32-FAIRFAX.GPJ GTINC 053801.GPJ 10/16/03

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

6.25 AUGER ___ HOLLOW STEM

WASHBORING FROM 13.5 FEET

DWB DRILLER YAW LOGGER

CME 55HT DRILL RIG

HAMMER TYPE Auto

REMARKS:

Drawn by: yaw	Ck'd. by: yaw	App'vd. by:
Date: 9/25/03	Date: 10/15/03	

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Fairfax Flood Control Structure

CONTINUATION OF LOG OF BORING: B-23

Project No. 0713201.3211

BORING LOG: TERMS AND SYMBOLS

GENERAL NOTES

1. *Information on each boring log is a compilation of subsurface conditions based on soil or rock classifications obtained from the field as well as from laboratory testing of samples. The strata lines on the logs may be approximate or the transition between the strata may be gradual rather than distinct. Water level measurements refer only to those observed at the times and places indicated, and may vary with time, geologic condition or construction activity.*
2. *Relative composition and Unified Soil Classification designations are based on visual estimates and are approximate only. If laboratory tests were performed to classify the soil, the unified designation is shown in parenthesis.*
3. *Value given in Unit Dry Weight/SPT Column is either a unit dry weight in pounds per cubic foot, if adjacent to a ST sample designation, or blows per 6-inch increment if adjacent to a SS sample designation.*

ABBREVIATIONS

UU/2	*Shear Strength from Unconsolidated - Undrained Triaxial Test (ASTM D2850)*
QU/2	*Shear Strength from Unconfined Compression Test (ASTM D2166)*
SV	*Shear Strength from Field Vane (ASTM D2573)*
PL	*Plastic Limit (ASTM D4318)*
LL	*Liquid Limit (ASTM D4318)*

LEGEND

CS	Continuous Sampler
GB	Grab Sample Taken From Auger Cuttings Or Wash Water Return
NX 100/42	NX Rock Core with Percent Recovery/R.Q.D. Given in Adjacent Column
PST	Three Inch Diameter Piston Tube Sample
SS	Split Spoon Sample (Standard Penetration Test)
ST	Three Inch Diameter Shelby Tube Sample
*	Sample Not Recovered
SV	Field Vane Test

SPLIT - BARREL SAMPLER DRIVING RECORD

Blow Per Foot (N-Value)	*Description*
25	*25 blows drove sampler 12 inches after initial 6 inches of seating.*
75/10"	*75 blows drove sampler 10 inches after initial 6 inches of seating.*
50/3"	*50 blows drove sampler 3 inches during initial 6 inch seating interval.*

NOTES: 1. To avoid damage to sampling tools, driving is limited to 50 blows during any six inch interval.
2. N-Value (Blow Count) is the standard penetration resistance based on the total number of blows, using a 140-lb. hammer with 30-inch free fall, required to drive a split spoon the last two of three, 6 inch drive increments. (Example: 4/7/9, N = 7 + 9 = 16). Values are shown as a summation on grid plot and may be shown as 4/7/9 in Unit Dry Weight -SPT column.

RELATIVE COMPOSITION

Trace	*0-10%*
With/Some	*11-35%*
Soil modifier such as silty, clayey, sandy, etc.	*> 35%*

DENSITY OF GRANULAR SOILS

Descriptive Term:	*N—Value*
Very Loose	*0-4*
Loose	*5-10*
Medium Dense	*11-30*
Dense	*31-50*
Very Dense	*> 50*

STRENGTH OF COHESIVE SOILS

Consistency	*Undrained Shear Strength Tons Per Sq. Ft.*	*Field Test*	*Approximate N-Value Range*
Very Soft	*less than 0.12*	*Thumb will penetrate soil more than 1".*	*0-1*
Soft	*0.13 to 0.25*	*Thumb will penetrate soil about 1".*	*2-4*
Medium Stiff	*0.26 to 0.50*	*Thumb will penetrate soil about ¼".*	*5-8*
Stiff	*0.51 to 1.00*	*Thumb hardly indents soil*	*9-15*
Very Stiff	*1.01 to 2.00*	*Thumb will not indent soil, but readily indented with thumbnail*	*16-30*
Hard	*greater than 2.00*	*Thumbnail will not indent soil*	*> 30*

SOIL GRAIN SIZE

U.S. STANDARD SIEVE

12"	3"	¾"	4	10	40	200

BOULDERS	COBBLES	GRAVEL		SAND			SILT	CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE		

300	76.2	19.1	4.76	2.00	0.42	0.074	.002

SOIL GRAIN SIZE IN MILLIMETERS

SOIL STRUCTURE

Calcareous — Having appreciable quantities of carbonate.

Fissured — Containing shrinkage or relief cracks, often filled with fine sand or silt; usually more or less vertical.

Slickensided — Having planes of weakness that appear slick and glossy. The degree of slickensidedness depends upon the spacing of slickensides and the ease of breaking along these planes.

Layer — Inclusion greater than 3 inches thick.

Seam — Inclusion 1/8 inch to 3 inches thick extending through the sample.

Parting — Inclusion less than 1/8 inch thick.

Pocket — Inclusion of material of different texture that is smaller than the diameter of the sample.

Interlayered — Soil samples composed of alternating layers of different soil types.

Intermixed — Soil samples composed of pockets of different soil types and a layered or laminated structure is not evident.

Laminated — Soil sample composed of alternating partings or seams of different soil type.

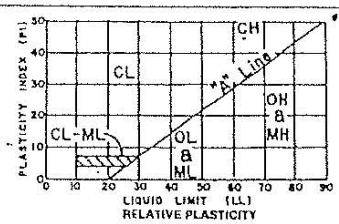
GEOTECHNOLOGY
ENGINEERING AND ENVIRONMENTAL SERVICES
SAINT LOUIS, MISSOURI

PLATE

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			SYMBOL	DESCRIPTION
Coarse-Grained Soils (more than 50% Larger than No. 200 Sieve Size)	Gravel and Gravelly Soils	Clean Gravels Little or no Fines	GW	Well-Graded Gravel, Gravel-Sand Mixture
			GP	Poorly-Graded Gravel, Gravel-Sand Mixture
		Gravels with Appreciable Fines	GM	Silty Gravel, Gravel-Sand-Silt Mixture
			GC	Clayey Gravel, Gravel-Sand-Clay Mixture
	Sand and Sandy Soils	Clean Sands Little or no Fines	SW	Well-Graded Sand, Gravelly Sand
			SP	Poorly-Graded Sand, Gravelly Sand
		Sands with Appreciable Fines	SM	Silty Sand, Sand-Silt Mixture
			SC	Clayey Sand, Sand-Clay Mixture
Fine-Grained Soils (more than 50% Smaller than No. 200 Sieve Size)	Silts and Clays	Liquid Limit Less than 50	ML	Silt, Clayey Silt, Silty or Clayey Very Fine Sand, Slight Plasticity
			CL	Clay, Sandy Clay, Silty Clay, Low to Medium Plasticity
			OL	Organic Silts or Silty Clays of Low Plasticity
	Silts and Clays	Liquid Limit More than 50	MH	Silt, Fine Sandy or Silty Soil with High Plasticity
			CH	Clay, High Plasticity
			OH	Organic Clay of Medium to High Plasticity
	Highly Organic Soils		PT	Peat, Humus, Swamp Soil

PLASTICITY CHART



VISUAL DESCRIPTION CRITERIA*

TABLE 1: CRITERIA FOR DESCRIBING ANGULARITY OF COARSE-GRAINED PARTICLES

Description	Criteria
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular	Particles are similar to angular description but have rounded edges
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges
Rounded	Particles have smoothly curved sides and no edges

TABLE 2: CRITERIA FOR DESCRIBING PARTICLE SHAPE

Description	Criteria
Flat	Particles with width/thickness X3
Elongated	Particles with length/width X3
Flat and Elongated	Particles meet criteria for both flat and elongated

TABLE 3: CRITERIA FOR DESCRIBING MOISTURE CONDITION

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

TABLE 4: CRITERIA FOR DESCRIBING REACTION WITH HCL

Description	Criteria
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

TABLE 6: CRITERIA FOR DESCRIBING CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure
Moderate	Crumbles or breaks with considerable finger pressure
Strong	Will not crumble or break with finger pressure

*NOTES: 1. Tables adapted from ASTM D 2488 "Description and Identification of Soils" (Visual-Manual Procedure)
2. Tables 5, 7 and 11 incorporated into other information on this plate.

TABLE 8: CRITERIA FOR DESCRIBING DRY STRENGTH

Description	Criteria
None	The dry specimen crumbles into powder with mere pressure of handling
Low	The dry specimen crumbles into powder with some finger pressure
Medium	The dry specimen breaks into pieces or crumbles with considerable finger pressure
High	The dry specimen cannot be broken with finger pressure. Specimen will break into pieces between thumb and a hard surface
Very High	The dry specimen cannot be broken between the thumb and a hard surface

TABLE 9: CRITERIA FOR DESCRIBING DILATANCY

Description	Criteria
None	No visible change in the specimen
Slow	Water appears slowly on the surface of the specimen during shaking and does not disappear or disappears slowly upon squeezing
Rapid	Water appears quickly on the surface of the specimen during shaking and disappears quickly upon squeezing

TABLE 10: CRITERIA FOR DESCRIBING TOUGHNESS

Description	Criteria
Low	Only slight pressure is required to roll the thread near the plastic limit. The thread and the lump are weak and soft.
Medium	Medium pressure is required to roll the thread to near the plastic limit. The thread and the lump have medium stiffness
High	Considerable pressure is required to roll the thread to near the plastic limit. The thread and the lump have very high stiffness

TABLE 12: IDENTIFICATION OF INORGANIC FINE-GRAINED SOILS FROM MANUAL TESTS

Soil Symbol	Dry Strength	Dilatancy	Toughness
ML	None to low	Slow to rapid	Low or thread cannot be formed
CL	Medium to high	None to slow	Medium
MH	Low to medium	None to slow	Low to medium
CH	High to very high	None	High

Geotechnical Calculations

EXHIBIT A-7.15
Fairfax-Jersey Creek Flood Wall
Pile Foundation Documentation

Appendix 4

Fairfax-Jersey Creek Flood Wall Documentation

Development of the Ultimate Resistance of the Piles Below the Floodwall : Station 287+85 to 302+32.

The geometry of the existing flood wall can be found on O & MM Plate 38 of reference 11. The 17'-9" high stem of the floodwall sits 12'-6" back from the riverside face of the keyed riverside pile cap. The total width of the pile cap is 19'-3". In a cross section through the wall, three driven 15.5-inch diameter concrete piles support the pile cap. Nondestructive subsurface investigations concluded that one area of the floodwall has piles with a length of 19 feet below the pile cap.

The structural engineers needed ultimate geotechnical resistance of the driven piles. The subsurface investigation boring results were used to determine a cohesionless strength of the foundation sands adjacent to and below the piles. Both side resistance and end bearing was considered to determine the ultimate geotechnical resistance of the piles.

Side Resistance. The side resistance, Q_s, that develops adjacent to the piles is dependent on the construction placement procedure, the correlated soil strength, the length and diameter of the piles, and the pore pressures developed during loading along the piles.

The loading conditions include end of construction and varying river stages.

The side resistance calculated for a given increment of depth Δl below the pile cap was computed using the equation $\Delta SR = fSR_z * \Delta AREA$ where fSR_z is the side resistance strength of the sand at depth z below the ground surface. The side resistance of the sand acts upon $\Delta AREA$ of the pile. $\Delta AREA$ is the perimeter of the piles for the length Δl or $\Delta AREA = \pi * d * \Delta l$ with d being the diameter of the pile. The side resistance of the sand is dependent on the correlated effective friction angle of the sand at depth z. The lateral earth forces acting upon the incremental Δl and the effective pore pressure acts upon the incremental area of the pile, $\Delta AREA$. The pore pressure is calculated based on the river stage, or head (H) acting on the flood wall and the sand foundation. The pore pressures were calculated using a simple two dimensional flow net based on the cross section of the flood wall and its foundation. The foundation is mostly sand with very little to no impervious blanket. The flow net provided equipotential drop lines from full head. These lines were used to determine the pore pressures along the piles for various river stages.

The magnitude of the effective pressure acting along the sides of the piles is sensitive to the placement techniques used to construct the piles. The piles were driven. The magnitude of the compressional coefficient of pressure used for analysis, K_c, was set to 1.0. EM 1110-2-2906 allows 1.0 to 2.0. The effective lateral pressure was

reduced by the pore pressure developed for each river stage used in the analysis. The effective lateral pressure was multiplied by the effective friction angle for sand acting on a concrete surface, δ'. The values suggested in EM 1110-2-2906 ranged from $0.9\phi'$ to $1.0\phi'$. The values used for the analysis are more conservative than the EM and are consistent with that suggested by Reese. The value δ' used was $\phi'-5^{o}$. DSR

The incremental side resistance, ΔSR, was computed as $f_s * \Delta AREA$ or $[(\sigma_{tz} - \mu_{wz}) * K_c] * [\tan(\delta') * \pi * d * (\Delta l_z)]$.

Where σ_{tz} is the total pressure at depth z;

and μ_{wz} is the pore pressure at depth z;

and Δl_z is the increment length of pile at depth z.

The spreadsheet calculates the resistance for each increment and adds the increments as the depth z increases. A total ultimate capacity is provided at the bottom for the total length of the pile.

End Bearing Resistance. The end bearing pressure acting on the area below the pile provides the end bearing capacity. The end bearing capacity is dependent on the effective friction angle of the sand, the effective overburden pressures below the pile and the cross sectional area of the bottom of the pile.

$Q_t = q_{br} * A_t$ below pile for depth z

Where $q_{br} = \sigma_v' * N_q$. N_q is the expected value of the bearing capacity factor dependent of the overburden

surcharge loading (depth of soil) and the area of the bottom of the pile, A_t, is $\pi * (d/2)^2$, with d being the diameter of the pile.

The bearing capacity factor used the mean values of Figure 4-4. The equation used in the spreadsheet was $N_q = 0.8158 * e^{(0.1165\phi')}$. The values used for σ_v' were based on the total overburden pressure less the pore pressures, $\sigma_{vz}t - \mu_{wz}$, developed for each river stage. The pore pressure was determined using the flow net.

Ultimate Pile Capacity. The ultimate capacity of the pile is determined by adding the side ressitance and the end bearing ressitance.

Results of the analysis. The results are provided for the landside, middle and riverside piles. A series of results are provided for the mean, mean - 1 standard deviation, mean + 1 standard deviation and for the one third ϕ' strength selection of the friction angle at the base of the piles. The friction angle along the sides of the piles varied with depth dependent on the blow counts taken. These values were also adjusted to provide mean, mean-1, mean+1 and one third strength values with depth. The results were provided for height of water on the flood wall of 8, 13 and 18 feet.

KCMOKS — FF-JC FLOODWALL

LOEHR EC-GD
Nov 15, 2004

PER EM 1110-2-2906 ; Dated 15 Jan 1991
p. 4-11

$Q_{ult} = Q_s + Q_t$

Q_s = shaft resistance
Q_t = tip resistance

$Q_s = f_s A_s$; f_s = average unit skin resistance
ΔSR_2

$f_s = K \sigma_v' \tan \delta$

Use f_s calculated to a maximum value at some critical depth $D_c = 10B$ — loose soils
$D_c = 15B$ — med dense soils
$D_c = 20B$ — dense soils

$\sigma_v' = \gamma' D$ for $D < D_c$

$\sigma_v' = \gamma' D_c$ for $D > D_c$

K = lateral earth pressure coefficient

from Table 4-3 p. 4-12
Value of δ → Concrete Pile δ varies from 0.9ϕ to 1.0ϕ
Per Reese use $\phi - 5° = \delta$ more conservative than 0.9ϕ

Values of K Table 4-4
for Sand $K_c \sim 1.0$ to 2.0 ; $K_t \sim 0.5$ to 0.7
Compressional Tensional

FF-JC (1)

KCMOKS FEAS STUDY

FF-JC FLOODWALL

Lochr EC-GD

Nov 15, 2004

END BEARING

DESIGN PURPOSES USE LINEAR INCREASE TO D_c THEN USE CONSTANT BELOW THAT

$$q_{br} = \sigma_v' N_q \qquad N_q \text{ from Fig 4-4}$$

Limit $\sigma_v' = \gamma' D$ for $D < D_c$

& $\sigma_v' = \gamma' D_c$ for $D > D_c$

Using $D_c = 15B$ where $B = 15.5''$

$$D_c = \left(\frac{15.5}{12}\right) * 15 = 19.4' \text{ Below Pile Cap}$$

Pile cap base thickness $\cong 3'$

Limiting depth below ground $D_c = 19+3 = 22'$ (DESIGN)

The limiting depth control was ignored for two reasons:

1) Depth of Piles $= 19' < 22'$

2) It is a requirement for DESIGN, we are studying the existing conditions.

FF-JC (2)

10/5/2004
Loehr EC-GD

Ref ETL 1110-2-556
dated 28 May 1999

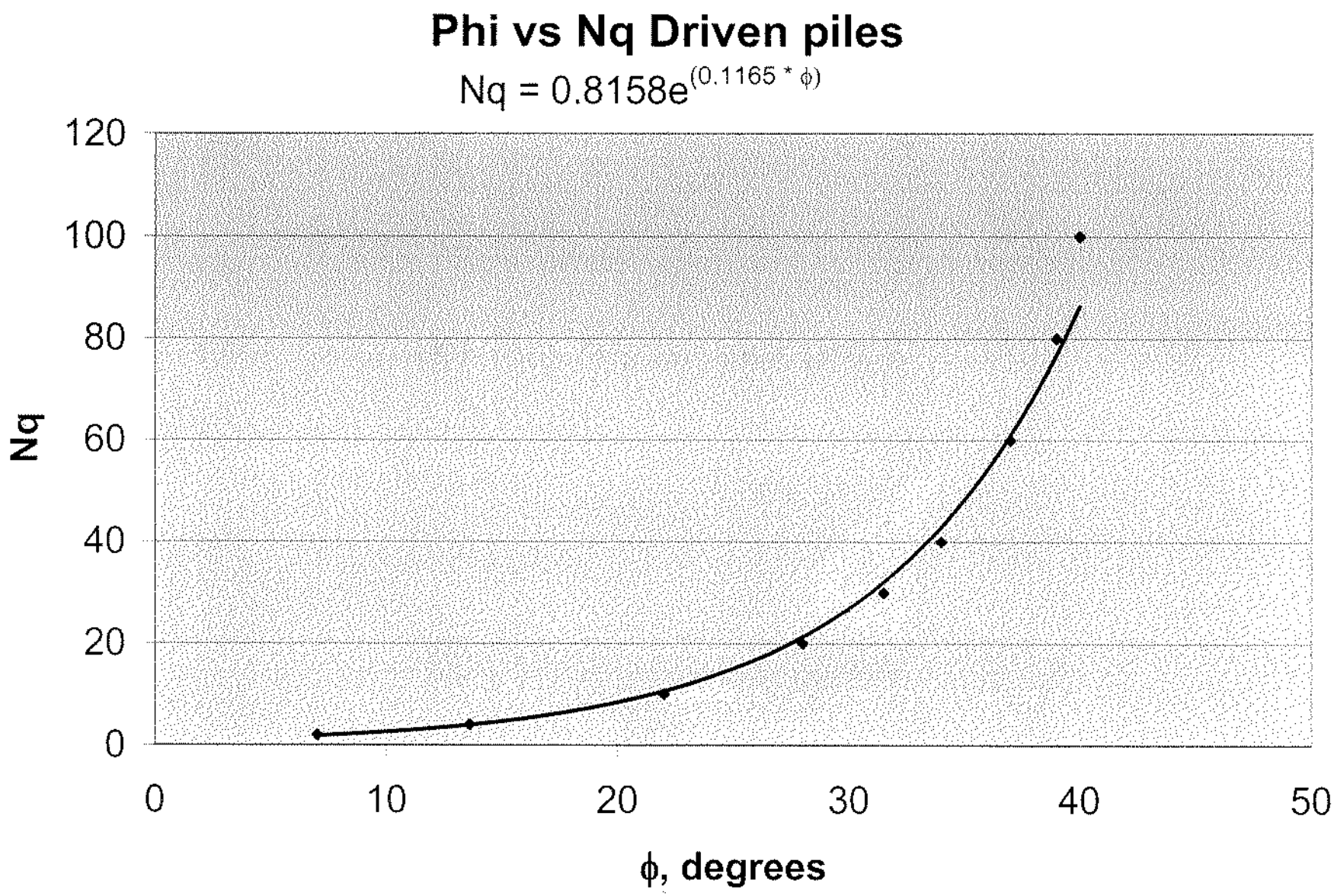
Use Variation of ϕ' parameter of 10%

Std Dev = σ_d = V * $E(\phi')$

Vertical Capacity of pile = Side Resist + End Bearing

$$= (\tau * \text{Perimeter} * h) + \sigma'_v N_q$$

$$= \left[(\sigma_h' * \tan\phi')(\pi d h)\right] + \left[\sigma'_v * N_q\right]$$


Phi vs Nq Driven piles
Nq = 0.8158e^(0.1165 * ϕ)
Nq
120
100
80
60
40
20
0
0
10
20
30
40
50
ϕ, degrees

"Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies"

Table 1 provides a summary of typical reported values for the coefficients of variation of commonly encountered geotechnical parameters. More detailed comment regarding the observed variability of relevant parameters is provided in the subsequent sections.

$V = \frac{\sigma}{E(c)}$

$\sigma = V * E(c)$

Table 1
Coefficients of Variation for Geotechnical Parameters

Parameter	Coefficient of Variation, percent	Reference
Unit weight	3 4 to 8	Hammitt (1966), cited by Harr (1987) assumed by Shannon and Wilson, Inc., and Wolff (1994)
Drained strength of sand ϕ'	3.7 to 9.3 12	Direct shear tests, Mississippi River Lock and Dam No. 2, Shannon and Wilson, Inc., and Wolff (1994) Schultze (1972), cited by Harr (1987)
Drained strength of clay ϕ'	7.5 to 10.1	S tests on compacted clay at Cannon Dam, Wolff (1985)
Undrained strength of clay s_u	40 30 to 40 11 to 45	Fredlund and Dahlman (1972) cited by Harr (1987) Assumed by Shannon and Wilson, Inc., and Wolff (1994) Q tests on compacted clay at Cannon Dam, Wolff (1985)
Strength-to-effective stress ratio s_u / σ'_v	31	Clay at Mississippi River Lock and Dam No. 2, Shannon and Wilson, Inc., and Wolff (1994)
Coefficient of permeability k	90	For saturated soils, Nielson, Biggar, and Erh (1973) cited by Harr (1987)
Permeability of top blanket clay k_b	20 to 30	Derived from assumed distribution, Shannon and Wilson, Inc., and Wolff (1994)
Permeability of foundation sands k_f	20 to 30	For average permeability over thickness of aquifer, Shannon and Wilson, Inc., and Wolff (1994)
Permeability ratio k_f / k_b	40	Derived using 30% for k_f and k_b; see Annex B
Permeability of embankment sand	30	Assumed by Shannon and Wilson, Inc., and Wolff (1994)

Use 10% = V

Unit Weight of Soil Materials

The coefficient of variation of the unit weight of soil material is usually on the order of 3 to 8 percent. In slope stability problems, uncertainty in unit weight usually contributes little to the overall uncertainty, which is dominated by soil strength. For stability problems, it can usually be taken as a deterministic variable in order to reduce the number of random variables and simplify calculations. It

where

c_a = adhesion between the clay and the pile

α = adhesion factor

c = undrained shear strength of the clay from a Q test

The values of α as a function of the undrained shear are given in Figure 4-5a.

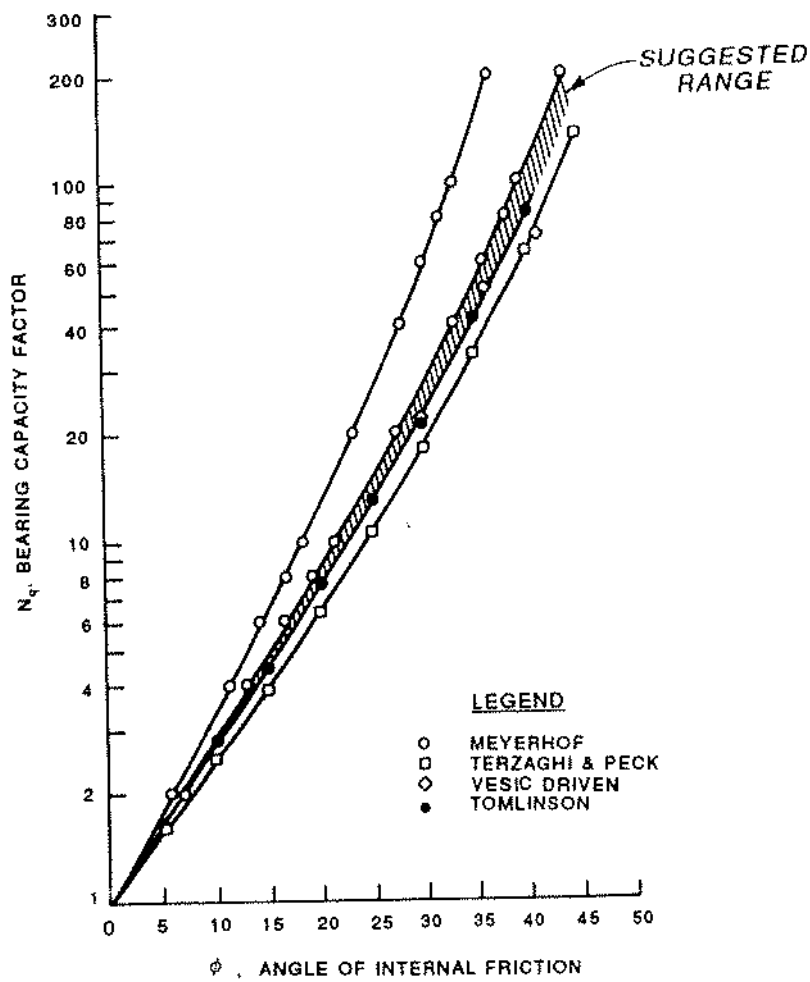


Figure 4-4. Bearing capacity factor

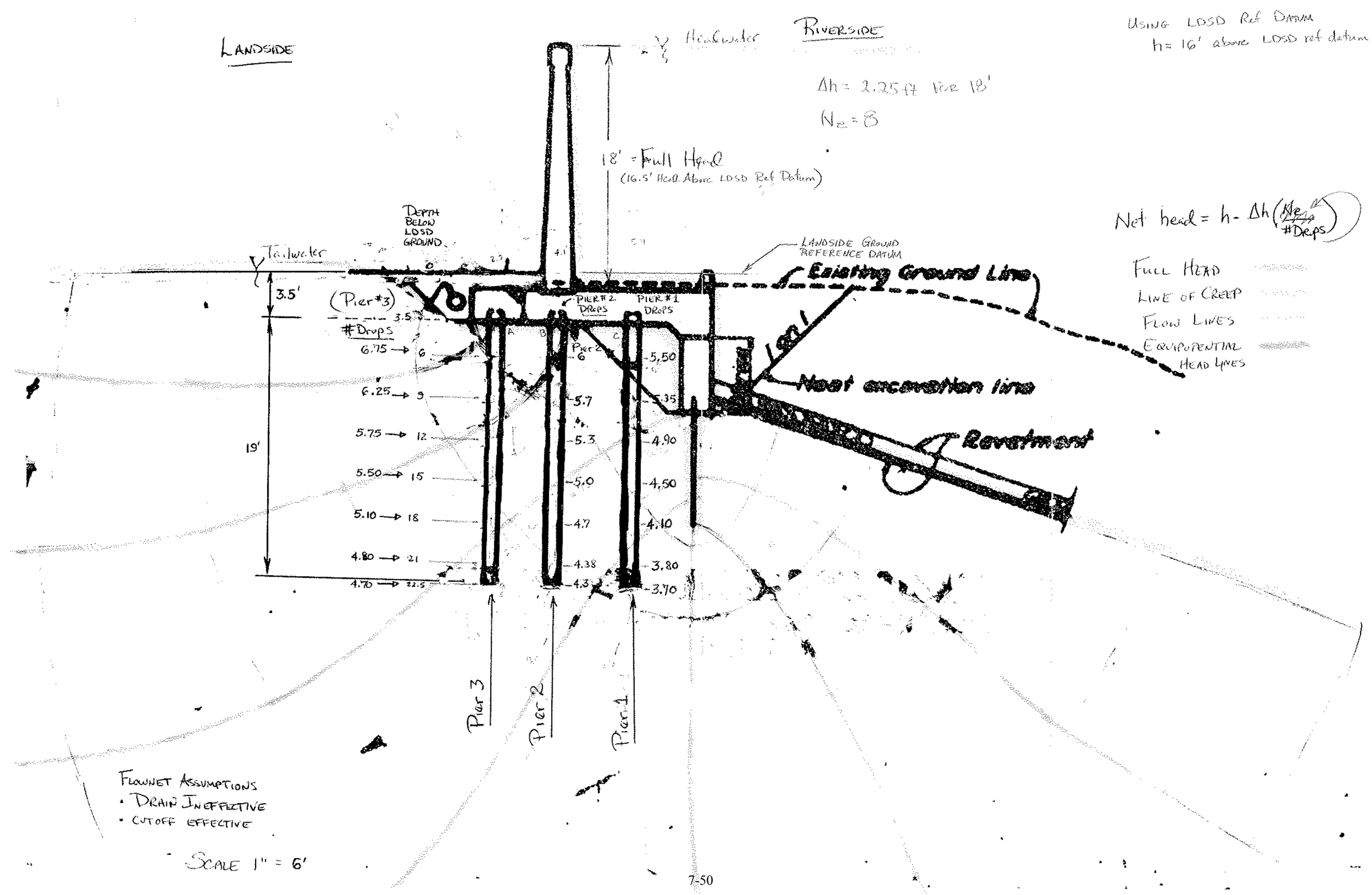
An alternate procedure developed by Semple and Rigden (Item 56) to obtain values of α which is especially applicable for very long piles is given in Figure 4-5b where:

$$\alpha = \alpha_1 \alpha_2$$

and

$$f_s = \alpha c$$

EXHIBIT A-7.16
Fairfax-Jersey Creek Flood Wall
Flownet


LANDSIDE
RIVERSIDE
Headwater
Using LDSD Ref Datum
h = 16' above LDSD ref datum
Δh = 2.25 ft for 18'
Nc = 8
18' = Full Head
(16.5' Head Above LDSD Ref Datum)
Net head = h - Δh (Nc / #Drops)
Depth Below LDSD Ground
Tailwater
3.5'
(Pier*3)
3.5
#Drops
6.75 → 6
6.25 → 9
5.75 → 12
5.50 → 15
5.10 → 18
4.80 → 21
4.70 → 22.5
19'
Pier #2 Drops
Pier #1 Drops
Pier 2
6
5.7
5.3
5.0
4.7
4.38
4.3
5.50
5.35
4.90
4.50
4.10
3.80
3.70
Landside Ground Reference Datum
Existing Ground Line
Heat excavation line
Revetment
Full Head
Line of Creep
Flow Lines
Equipotential Head Lines
Pier 3
Pier 2
Pier 1
Flownet Assumptions
• Drain Ineffective
• Cutoff Effective
Scale 1" = 6'

$\bar{p}$ = effective overburden pressure at the point in question, and

δ = the friction angle between the soil and the pile wall.

A value of K of 0.8 was recommended for open-ended pipe piles, that are driven unplugged, for loadings in both tension and compression. A value of K of 1.0 was recommended for full displacement piles. In the absence of data on δ, Table 3.1 was recommended as guidelines only for siliceous soil.

Equation 3.9 implies that the value of f increases without limit; however, Table 3.1 presents guidelines for limiting values.

$\delta = \bar{\Phi} - 5°$ (Assumed)

TABLE 3.1. Guideline for Side Friction in Siliceous Soil

Soil	δ, degrees	Limiting f, Kips/ft^2(kPa)
Very loose to medium, sand to silt	15	1.0 (47.8)
Loose to dense, sand to silt	20 ← for $\delta = 22°$, use →	1.4 (67.0)
Medium to dense, sand to sand-silt	25 ← for $\delta = 25°$ use →	1.7 (83.1)
Dense to very dense, sand to sand-silt	30 ← for $\delta = 28°$ use →	2.0 (95.5)
Dense to very dense, gravel to sand	35	2.4 (114.8)

3.3.5 End Bearing in Cohesionless Soil

For end bearing in cohesionless soils, API recommends the following.

$$q = \bar{p}_o N_q \tag{3.10}$$

where

$\bar{p}$ = effective overburden pressure at pile tip, and

N_q = bearing capacity factor.

Table 3.2 was recommended as a guideline only for siliceous soil.

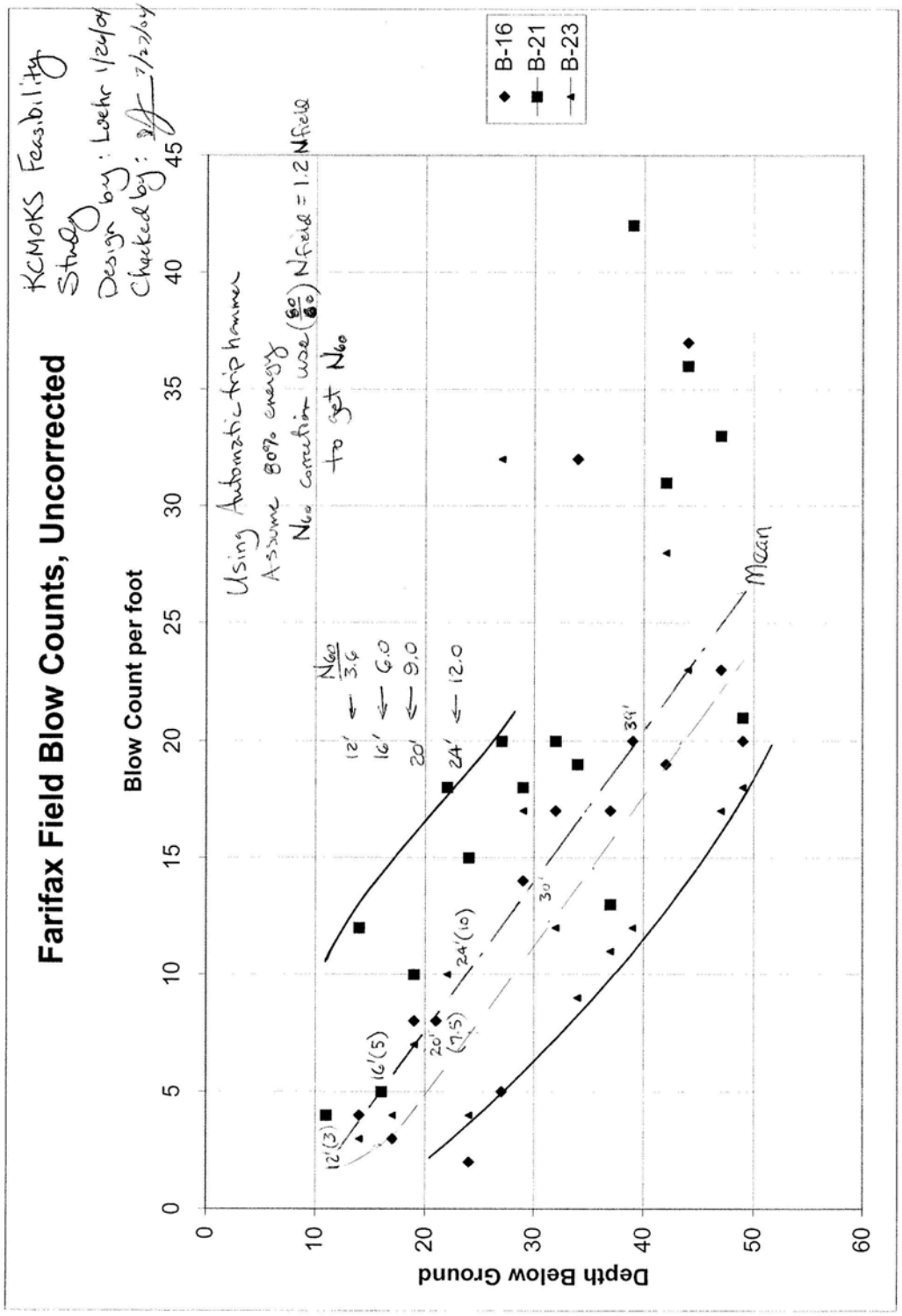
TABLE 3.2. Guideline for Tip Resistance in Siliceous Soil

Soil	N_q	Limiting q, kips/ft^2 MPa)
Very loose to medium, sand silt	8	40 (1.9)
Loose to dense, sand to silt	12 ← $\delta = 22°$ →	60 (2.9)
Medium to dense, sand to sand-silt	20 ← $\delta = 25°$ →	100 (4.8)
Dense to very dense sand to sand-silt	40 ← $\delta = 28°$ →	200 (9.6)
Dense to very dense, gravel to sand	50	250 (12.0)

The API publication points out that many soils do not fit the description of those in the tables and that the design parameters are not suitable for these soils. Examples are loose silts, soils containing large amounts of mica or volcanic grains, and calcareous sands. These latter soils are known to have substantially lower design parameters.

Drilled and grouted piles may have higher capacities than driven piles in calcareous soils.

EXHIBIT A-7.18



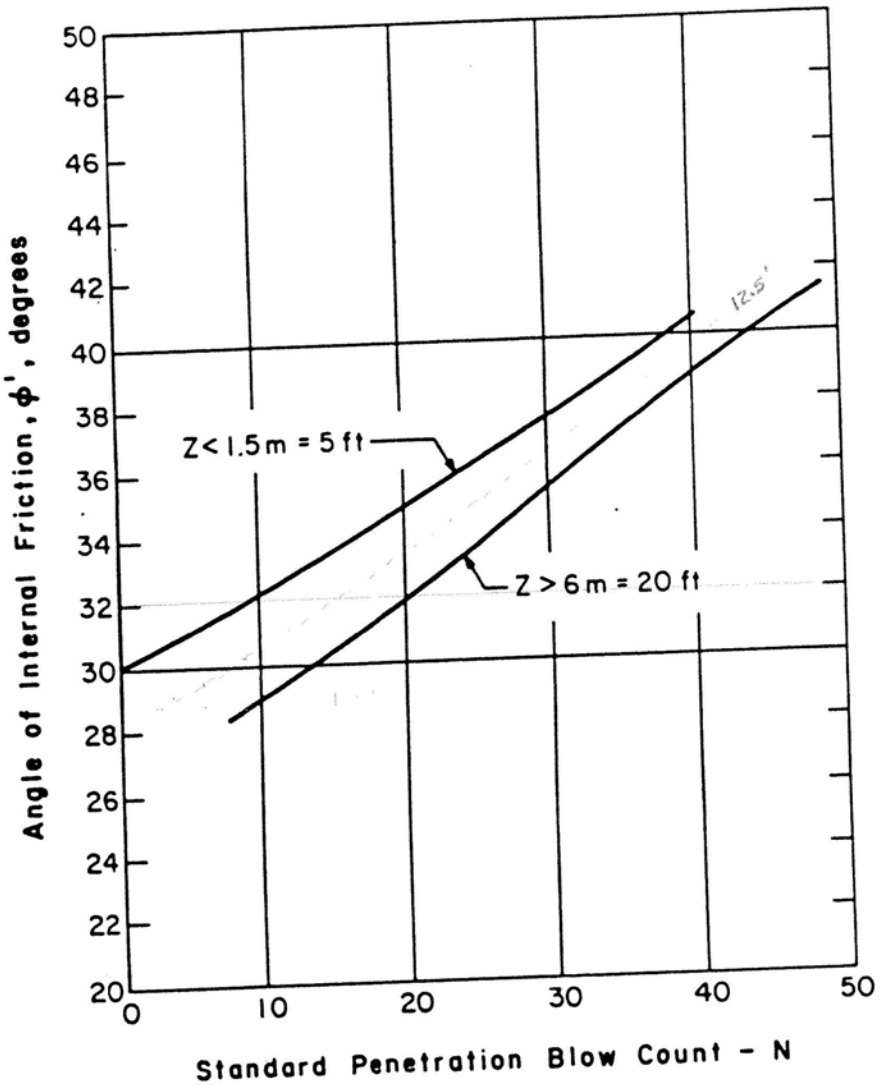
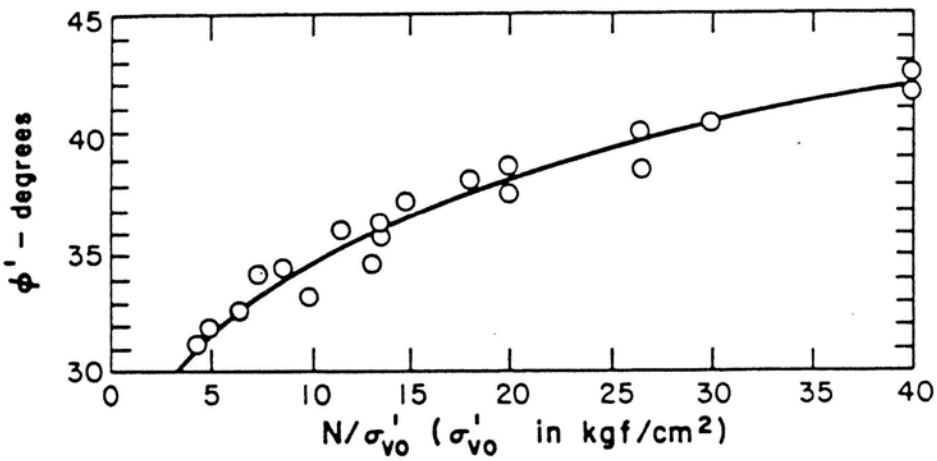
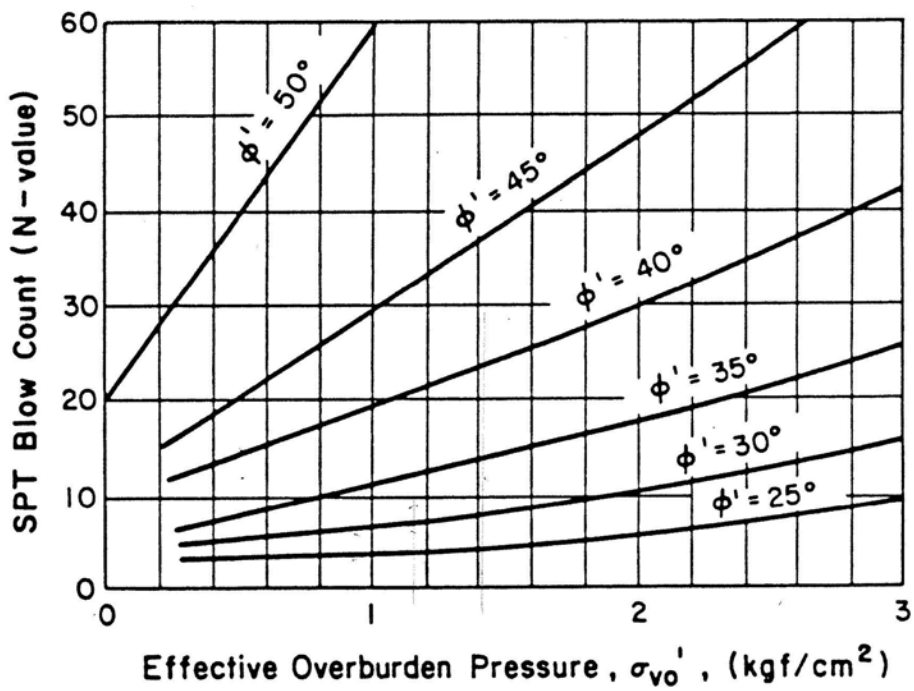


Fig.13 Relationship between Angle of Internal Friction of Cohesionless Soils and SPT Blow Count, (After Sowers, 1979).

Ref: "Shear Strength Correlations for Geotechnical Engineering". Duncan, Horz, Yang; August, 1989



(a) Relation between ϕ' and N/σ'_{vo}, (After Parry, 1977).



(b) Relationship among ϕ', SPT-N and σ'_{vo}, (After DeMello, 1971 and Schmertmann, 1975).

Fig.14 Relationship among Overburden Pressure, σ'_{vo}, Blow Count, N, and Peak Friction Angle, ϕ', for Sands.

EXHIBIT A-7.21

Fairfax Floodwall Study Feasibility

Riverside Pile

Geotechnology Results

Head = 8 | Automatic Trip Hammer | Use 80%

Using unit weight sand = 120 pcf moist

Water depth not encountered

Designer : Loehr 12/10/2004

Checker : Kuzniakowski 12/2004

Side resistance = f(phi and effective pressure)

$\tau = (\sigma_t - \mu_w) * \tan \delta '$

Using γsat = 125 pcf | Using Ko = 1 Driven piers

Using 15.5-inch diameter piers

Perimeter = 2 * pi * r = 4.058 feet

Area = pi * radius^2 1.310 sq ft

Depth	Uncorrected Blow count B-16	Material	Uncorrected Blow count B-21	Material	Uncorrected Blow count B-23	Material	Selected Mean Value	Conversion Automatic Trip Hammer Use 1.2	Sowers 1979 Phi Angle Fig 13 Degrees (1)	DeMello 1971 Schmertmann 1975 Phi Angle Fig 14 Degrees (2)	Effective Overburden Pressure (kgf / sq cm)	Average of (1) AND (2) degrees	$\delta = \phi$ - 5 degrees	Depth	Total Pressure Along Pile Psf	Pressure from Flow Net along Riverside Pile Psf (# drops)	Effective Overburden Pressure σ v ', Psf	Effective Overburden Pressure σ h ', Psf Driven Piers Use K =1	Ultimate Shear along Length τ , psf	Total Side Rst Along Pier Length pounds	Nqp End Bearing coefficient dimenionless	End Bearing Pressure qb, psf	End Bearing Resistance Qb, Pounds	Ultimate Side and End Bearing Pounds
0											Note : 1 Tsf = 0.976 kgf /	27	22	0	0	0	0	0	0	0	18.95	0	0	0
												27	22	3	375	309	66	66	0	0	18.95	1253	0	0
								Base of Floodwall : 3.5 feet								5.6								
5																			84	862				
												27	22	6	750	501	249	249		862	18.95	4716	6180	7042
																5.5								
																			136.94	1667				
												27	22	9	1125	696	429	429		2529	18.95	8131	10655	13184
10																5.35								
11			4	Silty Sand															206.67	2516				
12							3	**3.6**	29	25	0.703	27	22	12	1500	906	594	594		5045	18.95	11258	14752	19798
																4.9								
14	4	Silty Sand	12	Silty Sand	3	Silty Clay													280.75	3418				
												28	23	15	1875	1113	762	762		8463	21.29	16216	21249	29712
16			5	Silty Sand			5	**6**	29	29	0.937	29	24			4.5								
17	3	Silty Sand			4	Silty Clay						29.5	24.5						376.35	4582				
												30	25	18	2250	1321	929	929		13045	26.88	24974	32725	45770
19	8	Silty Sand	10	Silt	7											4.1								
20						Sand	7.5	**9**	28.5	32	1.171	30.25	25.25						476.71	5803				
21	8	Silty Sand										30.3	25.3	21	2625	1523	1102	1102		18848	27.84	30667	40184	59033
																3.8			544.19	3312				
22.5			18	Sand	10	Sand						30.5	25.5	22.5	2812.5	1622	1190	1190	567.83	22160	28.49	33921	44448	66609
														Pier Bottom		3.7								
24	2	Silty Sand	15	Sand	4	Sand	10	**12**	29.5	33	1.405	31.25	26.25	24	3000									
27	5	Sand	20	Sand	32	Sand								27	3375									
29	14	Sand	18	Sand	17	Sand								29										
30							14	**16.8**	31	35	1.757	33		30										
32	17	Sand	20	Sand	12	Sand								32										
34	32	Sand	19	Sand	9	Sand								34										
35														35										
37	17	Sand	13	Sand	11	Sand								37										
39	20	Sand	42	Sand	12	Sand	20	**24**	33	33	2.284	33		39										
42	19	Sand	31	Sand	28	Sand								42										
44	37	Sand	36	Sand	23	Sand								44										
46							25	**30**	35	37	2.694	36		46										
47	23	Sand	33	Sand	17	Sand								47										
49	20	Sand	21	Sand	18	Sand								49										

22160 Pounds Sum of Side Resistance 19' length pier

44448 Pounds End Resistance 19' length pier

66609 Pounds Sum of Total Resistance 19' length pier

For Ultimate Displacement expect 10% of pier diameter or 1.8 inches

Notes : Nqp from Figure 4-4 of EM 1110-2-2ᵹ with equation developed a $N_q = 0.8158e(0.1165*\phi')$

$q_b = \sigma v' * Nqp$

Side resistance Along Pier = Sum (τ_{di}*Area$_{di}$) , di = incremental pile length

Example : at l = 9 feet; Side resistance =[(66+249)/2 * (6-3.5)*4.058*tan 22)] + [(249+429)/2 * (9-6)*4.058*tan 22)] = 2529

Pressure Along Pier Due to Equipotential Drop Using Flow net

Net Head Above Landside Existing Ground Surface:

Net Head = h-Δh*(#Drops)

h = head above landside ground surface due to the river level

Δh = head drop for one equipotential drop = h / N_e

Ne = total equipotential drops

Drops : total increment of equipotential drop to the location in the foundation along the pier.

Example : Pier 3, 6 feet below the top of the landside ground surface for 8 feet of water on the floodwall h= 8' - 1.5' = 6.5'

N_e from the flow net = 8

$\Delta h = h/N_e$ = 6.5/8 = 0.813

Drops = 6

Net Head above landside ground surface = 6.5 - (0.813)5.5 = 2.02'

Total pressure at 6 feet ; P_6

P_6 = (Net head - elevation head) * Unit weight of water

P_6 = (2.02 - (-6)) * 62.4

P_6 = 501 psf

NOTES:

1. Blow count data was obtained in Appendix 3, Borings B-16, B-21 and B-23.
2. See pages A-2, A-3, A-4, A-5 for equations, correlations and relationships.
3. Ultimate Side Shear along piles include pore water pressure reduction in the effective overburden pressures.
4. The depth is considered negative below the landside ground surface to determine the total pressure head.

EXHIBIT A-7.21 (Continued)

Fairfax Floodwall Study Feasibility
Riverside Pile
Geotechnology Results
Water at TOW Head =13'
Automatic Trip Hammer
Use 80%

Using unit weight sand = 120 pcf moist
Water depth not encountered

Designer : Loehr 12/10/2004
Checker : Kuzniakowski 12/2004

Side resistance = f(phi and effective pressure)
$\tau = (\sigma t - \mu_w) * \tan \delta '$
Using γsat = 125 pcf
Using Ko = 1 - sin ϕ '

Using 15.5-inch diameter piers
Perimeter = 2 * pi * r = 4.058 feet
Area = pi * radius^2 1.310 sq ft

Depth	Uncorrected Blowcount B-16	Material	Uncorrected Blowcount B-21	Material	Uncorrected Blowcount B-23	Material	Selected Mean Value	Conversion Automatic Trip Hammer Use 1.2	Sowers 1979 Phi Angle Fig 13 Degrees (1)	DeMello 1971 Schmertmann 1975 Phi Angle Fig 14 Degrees (2)	Effective Overburden Pressure (kgf / sq cm)	Average of (1) AND (2) degrees	$\delta = \phi - 5$ degrees	Depth	Total Pressure Along Pile Psf	Pressure from Flow Net along Riverside Pile Psf	Effective Overburden Pressure σ v ', Psf	Effective Overburden Pressure σ h ', Psf	Ultimate Shear along Length τ , psf	Side Resistance Along Pier Length pounds	N_{qp} End Bearing coefficient dimenionless	End Bearing Pressure qb, psf	End Bearing Resistance Qb, Pounds	Ultimate Side and End Bearing Pounds	
											Note : 1 Tsf = 0.976 kgf /							Driven Piers Use K =1							
0												27	22	0	0	0	0	0	0	0					
												27	22	3	375	402	0	0		0					
									Top of Pile : 3.5'					3.5		5.6									
5																			31	310					
												27	22	6	750	599	151	151		310	18.95	2868	3759	4069	
																5.5									
																			96	1173					
												27	22	9	1125	799	326	326		1483	18.95	6173	8088	9572	
10																5.35									
11			4	Silty Sand															161	1964					
12							3	**3.6**	29	25	0.703	27	22	12	1500	1027	473	473		3448	18.95	8967	11750	15198	
																4.9									
14	4	Silty Sand	12	Silty Sand	3	Silty Clay													227	2769					
												28	23	15	1875	1250	625	625		6217	21.29	13310	17440	23657	
16			5	Silty Sand			5	**6**	29	29	0.937	29	24			4.5									
17	3	Silty Sand			4	Silty Clay						29.5	24.5						312	3799					
												30	25	18	2250	1473	777	777		10016	26.88	20886	27368	37384	
19	8	Silty Sand	10	Silt	7											4.1									
20						Sand	7.5	**9**	28.5	32	1.171	30.25	25.25						403	4901					
21	8	Silty Sand										30.3	25.3	21	2625	1687	938	938		14917	27.84	26107	34210	49126	
																3.8			465	2833					
22.5			18	Sand	10	Sand						30.5	25.5	22.5	2812.5	1790	1023	1023		17750	28.49	29143	38187	55937	
																3.7				**17750 Pounds Sum of Side Resistance 19' length pier**			**38187 Pounds Sum of Side Resistance 19' length pier**	**55937 Pounds Sum of Side Resistance 19' length pier**	**For Ultimate Displacement expect 10% of pier diameter or 1.8 inches**
24	2	Silty Sand	15	Sand	4	Sand		10	**12**	29.5	33	1.405	31.25	26.25	24										
27	5	Sand	20	Sand	32	Sand									27										

NOTES:
1. Blow count data was obtained in Appendix 3, Borings B-16, B-21 and B-23.
2. See pages A-2, A-3, A-4, A-5 for equations, correlations and relationships.
3. Ultimate Side Shear along piles include pore water pressure reduction in the effective overburden pressures.
4. The depth is considered negative below the landside ground surface to determine the total pressure head.

Notes : Nqp from Figure 4-4 of EM 1110-2-29 with equation developed a $Nq = 0.8158e(0.1165*\phi ')$
$q_b = \sigma v' * Nqp$

Side resistance Along Pier = Sum (τ_{dl}*$Area_{dl}$) , dl = incremental pile length

Example : at l = 9 feet; Side resistance =[(0+151)/2 * (6-3.5)*4.058*tan 22)] + [(151+326)/2 * (9-6)*4.058*tan 22)] = 1483

Pressure Along Pier Due to Equipotential Drop Using Flow net

Net Head Above Landside Existing Ground Surface:
Net Head = h-Δh*(#Drops)
h = head above landside ground surface due to the river level
Δh = head drop for one equipotential drop = h / N_e
Ne = total equipotential drops
# Drops : total increment of equipotential drop to the location in the foundation along the pier.

Example : Pier 3, 6 feet below the top of the landside ground surface for 13 feet of water on the floodwall h= 13' - 1.5' = 11.5
N_e from the flow net = 8
$\Delta h = h/N_e$ = 11.5/8 = 1.4375
Drops = 5.5
Net Head above landside ground surface = 11.5 - (1.4375)5.5 = 3.60'
Total pressure at 6 feet ; P_6
P_6 = (Net head - elevation head) * Unit weight of water
P_6 = (3.60 - (-6)) * 62.4
P_6 = 599 psf

EXHIBIT A-7.21 (Continued)

Fairfax Floodwall Study Feasibility
Riverside Pile
Geotechnology Results
Water at TOW Head =18'
Automatic Trip Hammer
Use 80%

Using unit weight sand = 120 pcf moist
Water depth not encountered

Designer : Loehr 12/10/2004
Checker : Kuzniakowski 12/2004

Side resistance = f(phi and effective pressure)
$\tau = (\sigma t - \mu_w) * \tan \delta '$
Using γsat = 125 pcf Using Ko = 1 - sin $\phi '$

Using 15.5-inch diameter piers
Perimeter = 2 * pi * r = 4.058 feet
Area = pi * radius^2 1.310 sq ft

Full Head Conditions =====> Head = 18 feet

Depth	Uncorrected Blowcount B-16	Material	Uncorrected Blowcount B-21	Material	Uncorrected Blowcount B-23	Material	Selected Mean Value	Conversion Automatic Trip Hammer Use 1.2	Sowers 1979 Phi Angle Fig 13 Degrees (1)	DeMello 1971 Schmertmann 1975 Phi Angle Fig 14 Degrees (2)	Effective Overburden Pressure (kgf / sq cm) Note : 1 Tsf = 0.976 kgf /	Average of (1) AND (2) degrees	$\delta = \phi - 5$ degrees	Depth	Total Pressure Along Pile Psf	Pressure from Flow Net along Riverside Pile Psf	Effective Overburden Pressure σ v ', Psf	Effective Overburden Pressure σ h ', Psf Driven Piers Use K =1	Ultimate Shear along Length τ , psf	Side Resistance Along Pier Length pounds	N_{qp} End Bearing coefficient dimenionless	End Bearing Pressure qb, psf	End Bearing Resistance Qb, Pounds	Ultimate Side and End Bearing Pounds	
0												27	22	0	0	0	0	0	0	0					
																			0	0					
												27	22	3	375	496	0	0	0	0					
														3.5		5.6									
5																			10.88	27					
												27	22	6	750	696	54	54		27	18.95	1021	1337	1365	
																5.5									
																			55.79	679					
												27	22	9	1125	903	222	222		706	18.95	4214	5522	6228	
10																5.35									
11			4	Silty Sand															116.07	1413					
12							3	**3.6**	29	25	0.703	27	22	12	1500	1148	352	352		2119	18.95	6675	8747	10867	
																4.9									
14	4	Silty Sand	12	Silty Sand	3	Silty Clay													174.13	2120					
												28	23	15	1875	1386	489	489		4239	21.29	10403	13632	17871	
16			5	Silty Sand			5	**6**	29	29	0.937	29	24			4.5									
17	3	Silty Sand			4	Silty Clay						29.5	24.5						247.86	3017					
												30	25	18	2250	1625	625	625		7257	26.88	16797	22010	29267	
19	8	Silty Sand	10	Silt	7											4.1									
20						Sand	7.5	**9**	28.5	32	1.171	30.25	25.25						328.40	3998					
21	8	Silty Sand										30.3	25.3	21	2625	1851	774	774		11254	27.84	21547	28235	39489	
22			18	Sand	10	Sand						30.5	25.5			3.8			386.79	2354					
												30.5	25.5	22.5	2812.5	1957	855	855		13609	28.49	24364	31926	45534	
24	2	Silty Sand	15	Sand	4	Sand	10	**12**	29.5	33	1.405	31.25	26.25	24	3000	3.7				**13609 Pounds Sum of Side Resistance 19' length pier**			**31926 Pounds Sum of Side Resistance 19' length pier**	**45534 Pounds Sum of Side Resistance 19' length pier**	**For Ultimate Displacement expect 10% of pier diameter or 1.8 inches**
27	5	Sand	20	Sand	32	Sand								27	3375										

NOTES:
1. Blow count data was obtained in Appendix 3, Borings B-16, B-21 and B-23.
2. See pages A-2, A-3, A-4, A-5 for equations, correlations and relationships.
3. Ultimate Side Shear along piles include pore water pressure reduction in the effective overburden pressures.
4. The depth is considered negative below the landside ground surface to determine the total pressure head. where pressure head = Net head above landside ground surface - depth below landside ground surface

Notes :

Nqp from Figure 4-4 of EM 1110-2-29 with equation developed a $Nq = 0.8158e(0.1165*\phi ')$
$q_b = \sigma v' * Nqp$

Side resistance Along Pier = Sum (τ_{dl}*$Area_{dl}$) , dl = incremental pile length

Example : at l = 9 feet; Side resistance =[(0+54)/2 * (6-3.5)*4.058*tan 22)] + [(54+222)/2 * (9-6)*4.058*tan 22)] = 706

Pressure Along Pier Due to Equipotential Drop Using Flow net

Net Head Above Landside Existing Ground Surface:
Net Head = h-Δh*(#Drops)
h = head above landside ground surface due to the river level
Δh = head drop for one equipotential drop = h / N_e
Ne = total equipotential drops
# Drops : total increment of equipotential drop to the location in the foundatio along the pier.

Example : Pier 3, 6 feet below the top of the landside ground surface for 18 feet of water on the floodwall h= 18' - 1.5' = 16.5
N_e from the flow net = 8
$\Delta h = h/N_e$ = 16.5/8 = 2.06
Drops = 6.75
Net Head above landside ground surface = 16.5 - (2.06)5.5 = 5.17
Total pressure at 6 feet ; P_6
P_6 = (Net head - elevation head) * Unit weight of water
P_6 = (5.17 - (-6)) * 62.4
P_6 = 696 psf

EXHIBIT A-7.22

Fairfax Floodwall Study Feasibility

Middle Pier

Geotechnology Results

Head = 8 | Automatic Trip Hammer | Use 80%

Using unit weight sand = 120 pcf moist

Water depth not encountered

Designer : Loehr 12/10/2004

Checker : Kuzniakowski 12/2004

Side resistance = f(phi and effective pressure)

$\tau = (\sigma t - \mu_w) * \tan \delta '$

Using γsat = 125 pcf | Using Ko = 1 Driven piers

Using 15.5-inch diameter piers

Perimeter = 2 * pi * r = 4.058 feet

Area = pi * radius^2 1.310 sq ft

. (1) . (2)

Depth	Uncorrected Blow count B-16	Material	Uncorrected Blow count B-21	Material	Uncorrected Blow count B-23	Material	Selected Mean Value	Conversion Automatic Trip Hammer Use 1.2	Sowers 1979 Phi Angle Fig 13 Degrees	DeMello 1971 Schmertmann 1975 Phi Angle Fig 14 Degrees	Effective Overburden Pressure (kgf / sq cm)	Average of (1) AND (2) degrees	$\delta = \phi$ - 5 degrees	Depth	Total Pressure Along Pile Psf	Pressure from Flow Net along Middle Pile Psf (# drops)	Effective Overburden Pressure σ v ', Psf	Effective Overburden Pressure σ h ', Psf Driven Piers Use K =1	Ultimate Shear along Length τ , psf	Total Side Rst Along Pier Length pounds	Nqp End Bearing coefficient dimenionless	End Bearing Pressure qb, psf	End Bearing Resistance Qb, Pounds	Ultimate Side and End Bearing Pounds
0											Note : 1 Tsf = 0.976 kgf /	27	22	0	0	0	0	0	0	0	18.95	0	0	0
												27	22	3	375	278	97	97	0	0	18.95	1830	0	0
									Base of Floodwall : 3.5 feet							6.2									
5																			84	862				
												27	22	6	750	476	274	274		862	18.95	5197	6809	7672
																6								
																			145.65	1773				
												27	22	9	1125	678	447	447		2635	18.95	8468	11096	13731
10																5.7								
11			4	Silty Sand															214.36	2609				
12							3	**3.6**	29	25	0.703	27	22	12	1500	886	614	614		5245	18.95	11642	15256	20501
																5.3								
14	4	Silty Sand	12	Silty Sand	3	Silty Clay													290.20	3533				
												28	23	15	1875	1088	787	787		8778	21.29	16756	21956	30734
16			5	Silty Sand			5	**6**	29	29	0.937	29	24			5								
17	3	Silty Sand			4	Silty Clay						29.5	24.5						388.77	4733				
												30	25	18	2250	1291	959	959		13510	26.88	25792	33797	47307
19	8	Silty Sand	10	Silt	7											4.7								
20						Sand	7.5	**9**	28.5	32	1.171	30.25	25.25						490.99	5977				
21	8	Silty Sand										30.3	25.3	21	2625	1493	1132	1132		19488	27.84	31514	41294	60782
																4.4			558.64	3400				
22.5			18	Sand	10	Sand						30.5	25.5	22.5	2812.5	1592	1221	1221	582.34	22888	28.49	34788	45584	68472
														Pier Bottom		4.3								
24	2	Silty Sand	15	Sand	4	Sand	10	**12**	29.5	33	1.405	31.25	26.25	24	3000									
27	5	Sand	20	Sand	32	Sand								27	3375									
29	14	Sand	18	Sand	17	Sand								29										
30							14	**16.8**	31	35	1.757	33		30										
32	17	Sand	20	Sand	12	Sand								32										
34	32	Sand	19	Sand	9	Sand								34										
35														35										
37	17	Sand	13	Sand	11	Sand								37										
39	20	Sand	42	Sand	12	Sand	20	**24**	33	33	2.284	33		39										
42	19	Sand	31	Sand	28	Sand								42										
44	37	Sand	36	Sand	23	Sand								44										
46							25	**30**	35	37	2.694	36		46										
47	23	Sand	33	Sand	17	Sand								47										
49	20	Sand	21	Sand	18	Sand								49										

22888 Pounds Sum of Side Resistance 19' length pier

45584 Pounds End Resistance 19' length pier

68472 Pounds Sum of Total Resistance 19' length pier

For Ultimate Displacement expect 10% of pier diameter or 1.8 inches

Notes : Nqp from Figure 4-4 of EM 1110-2-29 with equation developed as $Nq = 0.8158e(0.1165*\phi')$

$q_b = \sigma v' * Nqp$

Side resistance Along Pier = Sum (τ_{di} $Area_{di}$) , di = incremental pile length

Example : at l = 9 feet; Side resistance =[(97+274)/2 * (6-3.5)*4.058*tan 22)] + [(274+447)/2 * (9-6)*4.058*tan 22)] = 2635

Pressure Along Pier Due to Equipotential Drop Using Flow net

Net Head Above Landside Existing Ground Surface:

Net Head = h-Δh*(#Drops)

h = head above landside ground surface due to the river level

Δh = head drop for one equipotential drop = h / N_e

Ne = total equipotential drops

Drops : total increment of equipotential drop to the location in the foundation along the pier.

Example : Pier 3, 6 feet below the top of the landside ground surface

for 8 feet of water on the floodwall h= 8' - 1.5' = 6.5'

N_e from the flow net = 8

$\Delta h = h/N_e$ = 6.5/8 = 0.813

Drops = 6

Net Head above landside ground surface = 6.5 - (0.813)6 = 1.622'

Total pressure at 6 feet ; P_6

P_6 = (Net head - elevation head) * Unit weight of water

P_6 = (1.622 - (-6)) * 62.4

P_6 = 476 psf

NOTES:

1. Blow count data was obtained in Appendix 3, Borings B-16, B-21 and B-23.
2. See pages A-2, A-3, A-4, A-5 for equations, correlations and relationships.
3. Ultimate Side Shear along piles include pore water pressure reduction in the effective overburden pressures.
4. The depth is considered negative below the landside ground surface to determine the total pressure head. where pressure head = Net head above landside ground surface - depth below landside ground surface

EXHIBIT A-7.22 (Continued)

Fairfax Floodwall Study Feasibility
Middle Pile
Geotechnology Results
Water at TOW Head =13' | Automatic Trip Hammer | Use 80%

Using unit weight sand = 120 pcf moist
Water depth not encountered

Designer : Loehr 12/10/2004
Checker : Kuzniakowski 12/2004

Side resistance = f(phi and effective pressure)
$\tau = (\sigma t - \mu_w) * \tan \delta '$
Using γsat = 125 pcf | Using Ko = 1 - sin $\phi '$

Using 15.5-inch diameter piers
Perimeter = 2 * pi * r = 4.058 feet
Area = pi * radius^2 1.310 sq ft

Depth	Uncorrected Blowcount B-16	Material	Uncorrected Blowcount B-21	Material	Uncorrected Blowcount B-23	Material		Selected Mean Value	Conversion Automatic Trip Hammer Use 1.2	Sowers 1979 Phi Angle Fig 13 Degrees (1)	DeMello 1971 Schmertmann 1975 Phi Angle Fig 14 Degrees (2)	Effective Overburden Pressure (kgf / sq cm) Note : 1 Tsf = 0.976 kgf /	Average of (1) AND (2) degrees	$\delta = \phi$ - 5 degrees	Depth	Total Pressure Along Pier Psf	Pressure from Flow Net along Middle Pile Psf	Effective Overburden Pressure σ v ', Psf	Effective Overburden Pressure σ h ', Psf Driven Piers Use K =1	Ultimate Shear along Length τ , psf	Side Resistance Along Pier Length pounds	N_{qp} End Bearing coefficient dimenionless	End Bearing Pressure qb, psf	End Bearing Resistance Qb, Pounds	Ultimate Side and End Bearing Pounds	
0													27	22	0	0	0	0	0	0	0					
													27	22	3	375	349	26	26		0					
										Top of Pile : 3.5'					3.5		6.2									
5																				45	456					
													27	22	6	750	554	196	196		456	18.95	3718	4872	5328	
																	6									
																				112	1361					
													27	22	9	1125	768	357	357		1817	18.95	6768	8868	10685	
10																	5.7									
11			4	Silty Sand																175	2130					
12								3	**3.6**	29	25	0.703	27	22	12	1500	991	509	509		3947	18.95	9647	12641	16587	
																	5.3									
14	4	Silty Sand	12	Silty Sand	3	Silty Clay														244	2972					
													28	23	15	1875	1205	670	670		6919	21.29	14265	18692	25611	
16			5	Silty Sand				5	**6**	29	29	0.937	29	24			5									
17	3	Silty Sand			4	Silty Clay							29.5	24.5						334	4067					
													30	25	18	2250	1419	831	831		10986	26.88	22332	29263	40249	
19	8	Silty Sand	10	Silt	7												4.7									
20						Sand		7.5	**9**	28.5	32	1.171	30.25	25.25						428	5208					
21	8	Silty Sand											30.3	25.3	21	2625	1633	992	992		16194	27.84	27605	36173	52367	
																	4.4			491	2989					
22.5			18	Sand	10	Sand							30.5	25.5	22.5	2812.5	1736	1077	1077		19183	28.49	30676	40196	59379	
																	4.3									
24	2	Silty Sand	15	Sand	4	Sand		10	**12**	29.5	33	1.405	31.25	26.25	24						**19183 Pounds Sum of Side Resistance 19' length pier**			**40196 Pounds Sum of Side Resistance 19' length pier**	**59379 Pounds Sum of Side Resistance 19' length pier**	**For Ultimate Displacement expect 10% of pier diameter or 1.8 inches**
27	5	Sand	20	Sand	32	Sand									27											

NOTES:
1. Blow count data was obtained in Appendix 3, Borings B-16, B-21 and B-23.
2. See pages A-2, A-3, A-4, A-5 for equations, correlations and relationships.
3. Ultimate Side Shear along piles include pore water pressure reduction in the effective overburden pressures.
4. The depth is considered negative below the landside ground surface to determine the total pressure head.

Notes : Nqp from Figure 4-4 of EM 1110-2-29l with equation developed as $Nq = 0.8158e(0.1165*\phi ')$
$q_b = \sigma v' * Nqp$

Side resistance Along Pier = Sum (τ_{dl}*$Area_{dl}$) , dl = incremental pile length

Example : at l = 9 feet; Side resistance =[(26+196)/2 * (6-3.5)*4.058*tan 22)] + [(196+357)/2 * (9-6)*4.058*tan 22)] = 1817

Pressure Along Pier Due to Equipotential Drop Using Flow net

Net Head Above Landside Existing Ground Surface:
Net Head = h-Δh*(#Drops)
h = head above landside ground surface due to the river level
Δh = head drop for one equipotential drop = h / N_e
Ne = total equipotential drops
# Drops : total increment of equipotential drop to the location in the foundation along the pier.

Example : Pier 3, 6 feet below the top of the landside ground surface for 13 feet of water on the floodwall h= 13' - 1.5' = 11.5'
N_e from the flow net = 8
$\Delta h = h/N_e$ = 11.5/8 = 1.4375
Drops = 6.75
Net Head above landside ground surface = 11.5 - (1.4375)6 = 2.88'
Total pressure at 6 feet ; P_6
P_6 = (Net head - elevation head) * Unit weight of water
P_6 = (2.88 - (-6)) * 62.4
P_6 = 554 psf

EXHIBIT A-7.22 (Continued)

Fairfax Floodwall Study Feasibility
Middle Pile
Geotechnology Results
Water at TOW Head =18' | Automatic Trip Hammer | Use 80%

Using unit weight sand = 120 pcf moist
Water depth not encountered

Designer : Loehr 12/10/2004
Checker : Kuzniakowski 12/2004

Side resistance = f(phi and effective pressure)
$\tau = (\sigma t - \mu_w) * \tan \delta '$
Using γsat = 125 pcf | Using Ko = 1 - sin $\phi '$

Using 15.5-inch diameter piers
Perimeter = 2 * pi * r = 4.058 feet
Area = pi * radius^2 1.310 sq ft

Full Head Conditions =====> Head = 18 feet

Depth	Uncorrected Blowcount B-16	Material	Uncorrected Blowcount B-21	Material	Uncorrected Blowcount B-23	Material	Selected Mean Value	Conversion Automatic Trip Hammer Use 1.2	Sowers 1979 Phi Angle Fig 13 Degrees (1)	DeMello 1971 Schmertmann 1975 Phi Angle Fig 14 Degrees (2)	Effective Overburden Pressure (kgf / sq cm)	Average of (1) AND (2) degrees	$\delta = \phi - 5$ degrees	Depth	Total Pressure Along Pier Psf	Pressure from Flow Net along Middle Pile Psf	Effective Overburden Pressure $\sigma v '$, Psf	Effective Overburden Pressure $\sigma h '$, Psf (Driven Piers Use K =1)	Ultimate Shear along Length τ , psf	Side Resistance Along Pier Length pounds	N_{qp} End Bearing coefficient dimenionless	End Bearing Pressure qb, psf	End Bearing Resistance Qb, Pounds	Ultimate Side and End Bearing Pounds	
0											Note : 1 Tsf = 0.976 kgf /	27	22	0	0	0	0	0	0	0					
																			0	0					
												27	22	3	375	419	0	0	0	0					
														3.5		6.2									
5																			23.88	60					
												27	22	6	750	632	118	118		60	18.95	2240	2935	2995	
																6									
																			77.89	948					
												27	22	9	1125	858	267	267		1008	18.95	5068	6640	7648	
10																5.7									
11			4	Silty Sand															135.57	1650					
12							3	**3.6**	29	25	0.703	27	22	12	1500	1096	404	404		2658	18.95	7651	10026	12684	
																5.3									
14	4	Silty Sand	12	Silty Sand	3	Silty Clay													198.12	2412					
												28	23	15	1875	1322	553	553		5070	21.29	11773	15427	20497	
16			5	Silty Sand			5	**6**	29	29	0.937	29	24			5									
17	3	Silty Sand			4	Silty Clay						29.5	24.5						279.38	3401					
												30	25	18	2250	1548	702	702		8471	26.88	18873	24730	33201	
19	8	Silty Sand	10	Silt	7											4.7									
20						Sand	7.5	**9**	28.5	32	1.171	30.25	25.25						364.65	4439					
21	8	Silty Sand										30.3	25.3	21	2625	1774	851	851		12910	27.84	23697	31052	43962	
22			18	Sand	10	Sand						30.5	25.5			4.4			423.45	2577					
												30.5	25.5	22.5	2812.5	1880	932	932		15488	28.49	26564	34809	50297	
24	2	Silty Sand	15	Sand	4	Sand	10	**12**	29.5	33	1.405	31.25	26.25	24	3000	4.3				**15488 Pounds Sum of Side Resistance 19' length pier**			**34809 Pounds Sum of Side Resistance 19' length pier**	**50297 Pounds Sum of Side Resistance 19' length pier**	**For Ultimate Displacement expect 10% of pier diameter or 1.8 inches**
27	5	Sand	20	Sand	32	Sand								27	3375										

NOTES:
1. Blow count data was obtained in Appendix 3, Borings B-16, B-21 and B-23.
2. See pages A-2, A-3, A-4, A-5 for equations, correlations and relationships.
3. Ultimate Side Shear along piles include pore water pressure reduction in the effective overburden pressures.
4. The depth is considered negative below the landside ground surface to determine the total pressure head.

Notes : Nqp from Figure 4-4 of EM 1110-2-29l with equation developed as $Nq = 0.8158e(0.1165*\phi ')$
$q_b = \sigma v' * Nqp$

Side resistance Along Pier = Sum (τ_{dl}*$Area_{dl}$) , dl = incremental pile length

Example : at l = 9 feet; Side resistance =[(0+118)/2 * (6-3.5)*4.058*tan 22)] + [(118+267)/2 * (9-6)*4.058*tan 22)] = 1008

Pressure Along Pier Due to Equipotential Drop Using Flow net

Net Head Above Landside Existing Ground Surface:
Net Head = h-Δh*(#Drops)
h = head above landside ground surface due to the river level
Δh = head drop for one equipotential drop = h / N_e
Ne = total equipotential drops
# Drops : total increment of equipotential drop to the location in the foundation along the pier.

Example : Pier 3, 6 feet below the top of the landside ground surface for 13 feet of water on the floodwall h= 18' - 1.5' = 16.5'
N_e from the flow net = 8
$\Delta h = h/N_e$ = 16.5/8 = 2.06
Drops = 6
Net Head above landside ground surface = 16.5 - (2.06)6 = 4.14'
Total pressure at 6 feet ; P_6
P_6 = (Net head - elevation head) * Unit weight of water
P_6 = (4.14 - (-6)) * 62.4
P_6 = 632 psf

EXHIBIT A-7.23

Fairfax Floodwall Study Feasibility

Landside Pile

Geotechnology Results

Head = 8 | Automatic Trip Hammer | Use 80%

Using unit weight sand = 120 pcf moist

Water depth not encountered

Designer : Loehr 12/10/2004

Checker : Kuzniakowski 12/2004

Side resistance = f(phi and effective pressure)

$\tau = (\sigma t - \mu_{so}) * \tan \delta '$

Using γsat = 125 pcf | Using Ko = 1 Driven piers

Using 15.5-inch diameter piers

Perimeter = 2 * pi * r = 4.058 feet

Area = pi * radius^2 1.310 sq ft

. (1) . (2)

Depth	Uncorrected Blow count B-16	Material	Uncorrected Blow count B-21	Material	Uncorrected Blow count B-23	Material		Selected Mean Value	Conversion Automatic Trip Hammer Use 1.2	Sowers 1979 Phi Angle Fig 13 Degrees	DeMello 1971 Schmertmann 1975 Phi Angle Fig 14 Degrees	Effective Overburden Pressure (kgf / sq cm)	Average of (1) AND (2) degrees	$\delta = \phi$ - 5 degrees	Depth	Total Pressure Along Pile Psf	Pressure from Flow Net along Landside Pile Psf (# drops)	Effective Overburden Pressure σ v ', Psf	Effective Overburden Pressure σ h ', Psf Driven Piers Use K =1	Ultimate Shear along Length τ , psf	Total Side Rst Along Pier Length pounds	Nqp End Bearing coefficient dimenionless	End Bearing Pressure qb, psf	End Bearing Resistance Qb, Pounds	Ultimate Side and End Bearing Pounds
0												Note : 1 Tsf = 0.976 kgf /	27	22	0	0	0	0	0	0	0	18.95	0	0	0
													27	22	3	375	238	137	137	0	0	18.95	2598	0	0
									Base of Floodwall : 3.5 feet								7								
5																				84	862				
													27	22	6	750	438	312	312		862	18.95	5917	7754	8616
																	6.75								
																				158.96	1935				
													27	22	9	1125	650	475	475		2798	18.95	8996	11788	14586
10																	6.25								
11			4	Silty Sand																224.60	2734				
12								3	**3.6**	29	25	0.703	27	22	12	1500	863	637	637		5532	18.95	12075	15822	21354
																	5.75								
14	4	Silty Sand	12	Silty Sand	3	Silty Clay														300.17	3654				
													28	23	15	1875	1063	812	812		9186	21.29	17296	22664	31850
16			5	Silty Sand				5	**6**	29	29	0.937	29	24			5.5								
17	3	Silty Sand			4	Silty Clay							29.5	24.5						398.93	4856				
													30	25	18	2250	1270	980	980		14042	26.88	26337	34511	48553
19	8	Silty Sand	10	Silt	7												5.1								
20						Sand		7.5	**9**	28.5	32	1.171	30.25	25.25						500.51	6093				
21	8	Silty Sand											30.3	25.3	21	2625	1473	1152	1152		20135	27.84	32078	42034	62169
																	4.8			568.27	3459				
22.5			18	Sand	10	Sand							30.5	25.5	22.5	2812.5	1571	1241	1241	592.01	23594	28.49	35365	46341	69936
															Pier Bottom		4.7								
24	2	Silty Sand	15	Sand	4	Sand		10	**12**	29.5	33	1.405	31.25	26.25	24	3000									
27	5	Sand	20	Sand	32	Sand									27	3375									
29	14	Sand	18	Sand	17	Sand									29										
30								14	**16.8**	31	35	1.757	33		30										
32	17	Sand	20	Sand	12	Sand									32										
34	32	Sand	19	Sand	9	Sand									34										
35															35										
37	17	Sand	13	Sand	11	Sand									37										
39	20	Sand	42	Sand	12	Sand		20	**24**	33	33	2.284	33		39										
42	19	Sand	31	Sand	28	Sand									42										
44	37	Sand	36	Sand	23	Sand									44										
46								25	**30**	35	37	2.694	36		46										
47	23	Sand	33	Sand	17	Sand									47										
49	20	Sand	21	Sand	18	Sand									49										

23594 Pounds Sum of Side Resistance 19' length pier

46341 Pounds End Resistance 19' length pier

69936 Pounds Sum of Total Resistance 19' length pier

For Ultimate Displacement expect 10% of pier diameter or 1.8 inches

Notes : Nqp from Figure 4-4 of EM 1110-2-29(with equation developed as $Nq = 0.8158e(0.1165*\phi')$

$q_b = \sigma v' * Nqp$

Side resistance Along Pier = Sum (τ_{dl}*$Area_{dl}$) , dl = incremental pile length

Example : at l = 9 feet; Side resistance =[(137+312)/2 * (6-3.5)*4.058*tan 22)] + [(312+475)/2 * (9-6)*4.058*tan 22)] = 2798

Pressure Along Pier Due to Equipotential Drop Using Flow net

Net Head Above Landside Existing Ground Surface:

Net Head = h-Δh*(#Drops)

h = head above landside ground surface due to the river level

Δh = head drop for one equipotential drop = h / N_e

Ne = total equipotential drops

Drops : total increment of equipotential drop to the location in the foundation along the pier.

Example : Pier 3, 6 feet below the top of the landside ground surface for 8 feet of water on the floodwall h= 8' - 1.5' = 6.5'

N_e from the flow net = 8

$\Delta h = h/N_e$ = 6.5/8 = 0.813

Drops = 6.75

Net Head above landside ground surface = 6.5 - (0.813)6.75 = 1.01'

Total pressure at 6 feet ; P_6

P_6 = (Net head - elevation head) * Unit weight of water

P_6 = (1.10 - (-6)) * 62.4

P_6 = 438 psf

NOTES:

1. Blow count data was obtained in Appendix 3, Borings B-16, B-21 and B-23.
2. See pages A-2, A-3, A-4, A-5 for equations, correlations and relationships.
3. Ultimate Side Shear along piles include pore water pressure reduction in the effective overburden pressures.
4. The depth is considered negative below the landside ground surface to determine the total pressure head. where pressure head = Net head above landside ground surface - depth below landside ground surface.

EXHIBIT A-7.23 (Continued)

	Fairfax Floodwall Study Feasibility						Using unit weight sand = 120 pcf moist								Side resistance = f(phi and effective pressure)						Using 15.5-inch diameter piers				
	Landside Pile						Water depth not encountered			Designer : Loehr 12/10/2004					$\tau = (\sigma t - \mu_w) * \tan \delta'$						Perimeter = 2 * pi * r =			4.058	feet
			Geotechnology Results							Checker : Kuzniakowski 12/2004						Using γsat = 125 pcf		Using Ko = 1 - sin ϕ'			Area = pi * radius^2			1.310	sq ft
	Water at TOW Head =13'		Automatic Trip Hammer		Use 80%																				
Depth	Uncorrected Blowcount B-16	Material	Uncorrected Blowcount B-21	Material	Uncorrected Blowcount B-23	Material		Selected Mean Value	Conversion Automatic Trip Hammer Use 1.2	. (1) Sowers 1979 Phi Angle Fig 13 Degrees	. (2) DeMello 1971 Schmertmann 1975 Phi Angle Fig 14 Degrees	Effective Overburden Pressure (kgf / sq cm)	Average of (1) AND (2) degrees	$\delta = \phi - 5$ degrees	Depth	Total Pressure Along Pile Psf	Pressure from Flow Net along Landside Pile Psf	Effective Overburden Pressure σ v ', Psf	Effective Overburden Pressure σ h ', Psf	Ultimate Shear along Length τ , psf	Side Resistance Along Pier Length pounds	N_{qp} End Bearing coefficient dimenionless	End Bearing Pressure qb, psf	End Bearing Resistance Qb, Pounds	Ultimate Side and End Bearing Pounds	
																			Driven Piers Use K =1						
0												Note : 1 Tsf = 0.976 kgf /	27	22	0	0	0	0	0	0	0				
													27	22	3	375	277	98	98		0				
										Top of Pile : 3.5'					3.5		7								
5																				73	741				
													27	22	6	750	487	263	263		741	18.95	4993	6543	7284
																	6.75								
																				135	1647				
													27	22	9	1125	719	406	406		2388	18.95	7703	10093	12482
10																	6.25								
11			4	Silty Sand																193	2351				
12								3	**3.6**	29	25	0.703	27	22	12	1500	951	549	549		4739	18.95	10412	13643	18382
																	5.75								
14	4	Silty Sand	12	Silty Sand	3	Silty Clay														262	3187				
													28	23	15	1875	1160	715	715		7926	21.29	15220	19943	27869
16			5	Silty Sand				5	**6**	29	29	0.937	29	24			5.5								
17	3	Silty Sand			4	Silty Clay							29.5	24.5						352	4286				
													30	25	18	2250	1383	867	867		12212	26.88	23297	30527	42739
19	8	Silty Sand	10	Silt	7												5.1								
20						Sand		7.5	**9**	28.5	32	1.171	30.25	25.25						445	5413				
21	8	Silty Sand											30.3	25.3	21	2625	1597	1028	1028		17625	27.84	28604	37482	55107
																	4.8			508	3093				
22.5			18	Sand	10	Sand							30.5	25.5	22.5	2812.5	1700	1112	1112		20718	28.49	31698	41536	62254
																	4.7				**20718 Pounds Sum of Side Resistance 19' length pier**			**41536 Pounds Sum of Side Resistance 19' length pier**	**62254 Pounds Sum of Side Resistance 19' length pier**
24	2	Silty Sand	15	Sand	4	Sand		10	**12**	29.5	33	1.405	31.25	26.25	24										**For Ultimate Displacement expect 10% of pier diameter or 1.8 inches**
27	5	Sand	20	Sand	32	Sand									27										

NOTES:
1. Blow count data was obtained in Appendix 3, Borings B-16, B-21 and B-23.
2. See pages A-2, A-3, A-4, A-5 for equations, correlations and relationships.
3. Ultimate Side Shear along piles include pore water pressure reduction in the effective overburden pressures.
4. The depth is considered negative below the landside ground surface to determine the total pressure head.
where pressure head = Net head above landside ground surface - depth below landside ground surface
and the depth to the point in the foundation is considered negative below the landside ground surface.

Notes :

Nqp from Figure 4-4 of EM 1110-2-29(with equation developed as Nq = 0.8158e(0.1165*ϕ')
$q_b = \sigma v' * Nqp$

Side resistance Along Pier = Sum (τ_{dl}*$Area_{dl}$) , dl = incremental pile length

Example : at l = 9 feet; Side resistance =[(98+263)/2 * (6-3.5)*4.058*tan 22)] + [(263+406)/2 * (9-6)*4.058*tan 22)] = 2388

Pressure Along Pier Due to Equipotential Drop Using Flow net

Net Head Above Landside Existing Ground Surface:
Net Head = h-Δh*(#Drops)
h = head above landside ground surface due to the river level
Δh = head drop for one equipotential drop = h / N_e
Ne = total equipotential drops
# Drop : total increment of equipotential drop to the location in the foun along the pier.

Example :
Pier 3, 6 feet below the top of the landside ground surface
for 13 feet of water on the floodwall h= 13' - 1.5' = 11.5'
N_e from the flow net = 8
Δh = h/N_e = 11.5/8 = 1.4375
Drops = 6.75
Net Head above landside ground surface = 11.5 - (1.4375)6.75 = 1.80'
Total pressure at 6 feet ; P_6
P_6 = (Net head - elevation head) * Unit weight of water
P_6 = (1.80 - (-6)) * 62.4
P_6 = 487 psf

EXHIBIT A-7.23 (Continued)

Fairfax Floodwall Study Feasibility
Landside Pile
Geotechnology Results
Water at TOW Head =18' | Automatic Trip Hammer | Use 80%

Using unit weight sand = 120 pcf moist
Water depth not encountered

Designer : Loehr 12/10/2004
Checker : Kuzniakowski 12/2004

Side resistance = f(phi and effective pressure)
$\tau = (\sigma t - \mu_{sp}) * \tan \delta '$
Using γsat = 125 pcf | Using Ko = 1 - sin ϕ '

Using 15.5-inch diameter piers
Perimeter = 2 * pi * r = 4.058 feet
Area = pi * radius^2 1.310 sq ft

Full Head Conditions =====> Head = 18 feet

Depth	Uncorrected Blowcount B-16	Material	Uncorrected Blowcount B-21	Material	Uncorrected Blowcount B-23	Material	Selected Mean Value	Conversion Automatic Trip Hammer Use 1.2	. (1) Sowers 1979 Phi Angle Fig 13 Degrees	. (2) DeMello 1971 Schmertmann 1975 Phi Angle Fig 14 Degrees	Effective Overburden Pressure (kgf / sq cm)	Average of (1) AND (2) degrees	$\delta = \phi$ - 5 degrees	Depth	Total Pressure Along Pile Psf	Pressure from Flow Net along Landside Pile Psf	Effective Overburden Pressure σ v ', Psf	Effective Overburden Pressure σ h ', Psf	Ultimate Shear along Length τ , psf	Side Resistance Along Pier Length pounds	N_{qp} End Bearing coefficient dimenionless	End Bearing Pressure qb, psf	End Bearing Resistance Qb, Pounds	Ultimate Side and End Bearing Pounds	
											Note : 1 Tsf = 0.976 kgf /							Driven Piers Use K =1							
0												27	22	0	0	0	0	0	0	0					
																			0	0					
												27	22	3	375	316	59	59	0	0					
														3.5		7									
5																			55.32	138					
												27	22	6	750	535	215	215		138	18.95	4069	5332	5471	
																6.75									
																			111.69	1360					
												27	22	9	1125	787	338	338		1498	18.95	6409	8398	9896	
10																6.25									
11			4	Silty Sand															161.57	1967					
12							3	**3.6**	29	25	0.703	27	22	12	1500	1038	462	462		3465	18.95	8749	11464	14929	
																5.75									
14	4	Silty Sand	12	Silty Sand	3	Silty Clay													223.44	2720					
												28	23	15	1875	1258	617	617		6185	21.29	13144	17223	23408	
16			5	Silty Sand			5	**6**	29	29	0.937	29	24			5.5									
17	3	Silty Sand			4	Silty Clay						29.5	24.5						305.16	3715					
												30	25	18	2250	1496	754	754		9900	26.88	20257	26543	36443	
19	8	Silty Sand	10	Silt	7											5.1									
20						Sand	7.5	**9**	28.5	32	1.171	30.25	25.25						388.82	4733					
21	8	Silty Sand										30.3	25.3	21	2625	1722	903	903		14633	27.84	25130	32929	47563	
22			18	Sand	10	Sand						30.5	25.5			4.8			447.90	2726					
												30.5	25.5	22.5	2812.5	1829	984	984		17360	28.49	28031	36731	54090	
24	2	Silty Sand	15	Sand	4	Sand	10	**12**	29.5	33	1.405	31.25	26.25	24	3000	4.7				**17360 Pounds Sum of Side Resistance 19' length pier**			**36731 Pounds Sum of Side Resistance 19' length pier**	**54090 Pounds Sum of Side Resistance 19' length pier**	**For Ultimate Displacement expect 10% of pier diameter or 1.8 inches**
27	5	Sand	20	Sand	32	Sand								27	3375										

NOTES:
1. Blow count data was obtained in Appendix 3, Borings B-16, B-21 and B-23.
2. See pages A-2, A-3, A-4, A-5 for equations, correlations and relationships.
3. Ultimate Side Shear along piles include pore water pressure reduction in the effective overburden pressures.
4. The depth is considered negative below the landside ground surface to determine the total pressure head.
where pressure head = Net head above landside ground surface - depth below landside ground surface.

Notes : Nqp from Figure 4-4 of EM 1110-2-29(with equation developed as Nq = 0.8158e(0.1165*ϕ ')
$q_p = \sigma v' * Nqp$

Side resistance Along Pier = Sum (τ_{dl}*$Area_{dl}$) , dl = incremental pile length

Example : at l = 9 feet; Side resistance =[(59+215)/2 * (6-3.5)*4.058*tan 22)] + [(215+338)/2 * (9-6)*4.058*tan 22)] = 1498

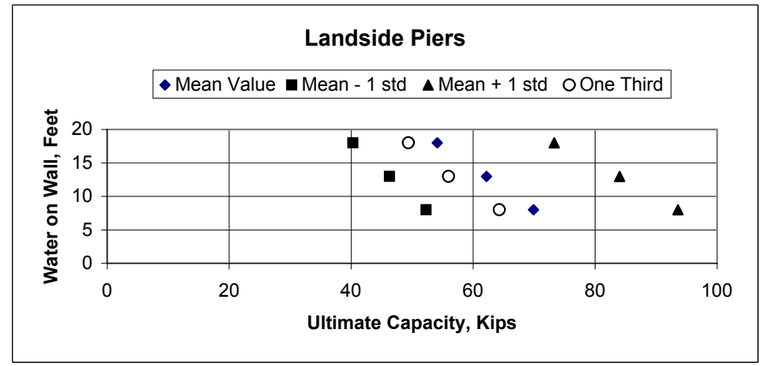
Pressure Along Pier Due to Equipotential Drop Using Flow net

Net Head Above Landside Existing Ground Surface:
Net Head = h-Δh*(#Drops)
h = head above landside ground surface due to the river level
Δh = head drop for one equipotential drop = h / N_e
Ne = total equipotential drops
# Drop : total increment of equipotential drop to the location in the foun along the pier.

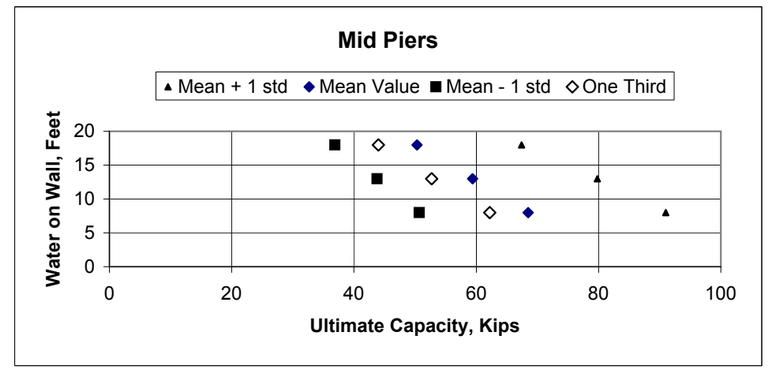
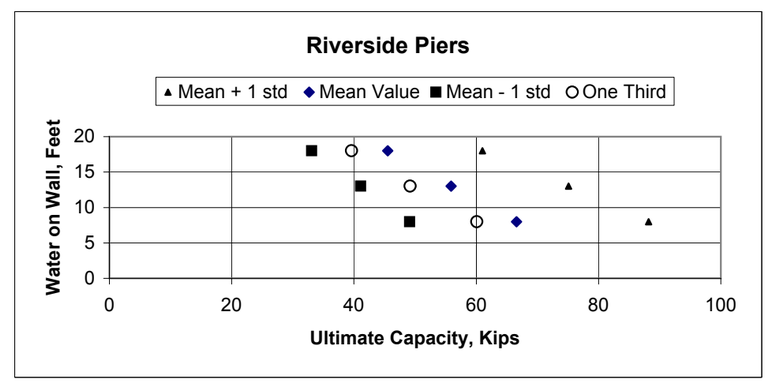
Example : Pier 3, 6 feet below the top of the landside ground surface for 13 feet of water on the floodwall h= 18' - 1.5' = 16.5'
N_e from the flow net = 8
$\Delta h = h/N_e$ = 16.5/8 = 2.06
Drops = 6.75
Net Head above landside ground surface = 16.5 - (2.06)6.75 = 2.58'
Total pressure at 6 feet ; P_6
P_6 = (Net head - elevation head) * Unit weight of water
P_6 = (2.58 - (-6)) * 62.4
P_6 = 535 psf

EXHIBIT A-7.24

Phi angle	15.5 dia Pier Location	Height on Wall Feet	Ultimate Capacity, Kips
Mean Value	Landside	8	69.9
		13	62.2
		18	54.1
Mean - 1 std	Landside	8	52.3
		13	46.3
		18	40.3
Mean + 1 std	Landside	8	93.6
		13	84
		18	73.3
One Third	Landside	8	64.3
		13	56
		18	49.4



Phi angle	15.5 dia Pier Location	Height on Wall Feet	Ultimate Capacity, Kips
Mean Value	Mid	8	68.5
		13	59.4
		18	50.3
Mean - 1 std	Mid	8	50.7
		13	43.8
		18	36.9
Mean + 1 std	Mid	8	91
		13	79.8
		18	67.4
One Third	Mid	8	62.2
		13	52.7
		18	44



Phi angle	15.5 dia Pier Location	Height on Wall Feet	Ultimate Capacity, Kips
Mean Value	Riverside	8	66.6
		13	55.9
		18	45.5
Mean - 1 std	Riverside	8	49.1
		13	41.1
		18	33.1
Mean + 1 std	Riverside	8	88.2
		13	75.1
		18	61
One Third	Riverside	8	60.1
		13	49.2
		18	39.6