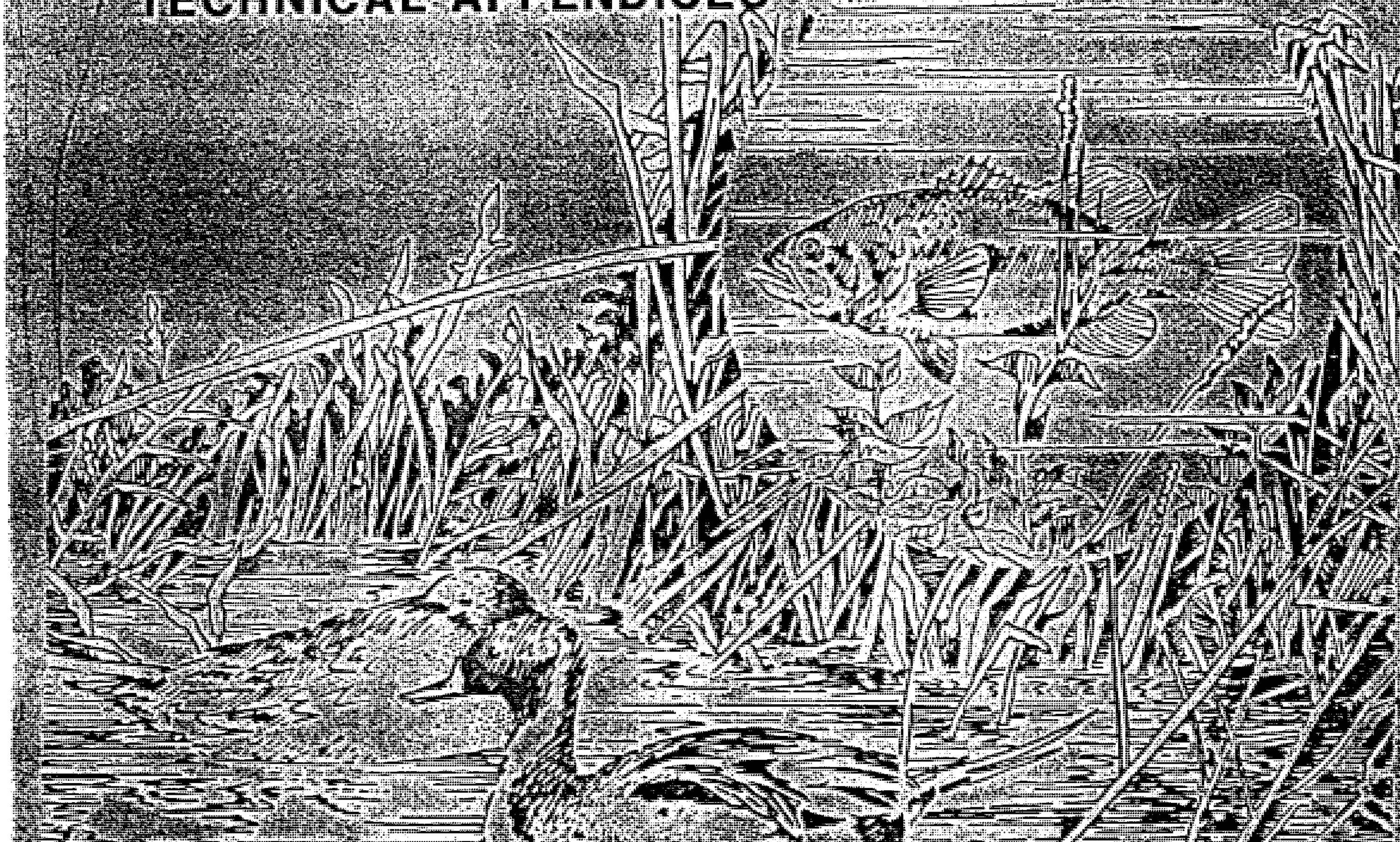
UPPER MISSISSIPPI RIVER SYSTEM
ENVIRONMENTAL MANAGEMENT PROGRAM
DEFINITE PROJECT REPORT (R-4)
WITH INTEGRATED ENVIRONMENTAL ASSESSMENT

ANDALUSIA REFUGE
REHABILITATION AND ENHANCEMENT

TECHNICAL APPENDICES



NOVEMBER 1988

US Army Corps
of Engineers
Rock Island District

Pool 16
UPPER MISSISSIPPI RIVER
ROCK ISLAND COUNTY, ILLINOIS

UPPER MISSISSIPPI RIVER SYSTEM
ENVIRONMENTAL MANAGEMENT PROGRAM
DEFINITE PROJECT REPORT

ANDALUSIA REFUGE
REHABILITATION AND ENHANCEMENT

POOL 16, MISSISSIPPI RIVER MILES 462 THROUGH 463

ROCK ISLAND COUNTY, ILLINOIS

TECHNICAL APPENDICES

HYDROLOGY AND HYDRAULICS

APPENDIX A

UPPER MISSISSIPPI RIVER SYSTEM

ENVIRONMENTAL MANAGEMENT PROGRAM

DEFINITE PROJECT REPORT (R-4)

ANDALUSIA REFUGE

REHABILITATION AND ENHANCEMENT

POOL 16, RIVER MILES 462 THROUGH 463

ROCK ISLAND COUNTY, ILLINOIS

APPENDIX A

HYDROLOGY AND HYDRAULICS

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ROCK ISLAND COUNTY, ILLINOIS

APPENDIX A

HYDROLOGY AND HYDRAULICS

GENERAL

The Andalusia Refuge area, shown on plate 1 of the main report, is located within the Upper Mississippi Wildlife and Fish Refuge between river miles 462 and 463 in Pool 16. This area, located 1 mile north of Illinois City, is currently managed as a waterfowl refuge by the Illinois Department of Conservation.

The purpose of this appendix is to present the development and evaluation of proposed improvements which will provide a water control structure system. This system will provide a moist soil management unit with controlled water levels, reduce sedimentation into the refuge area, and divert upland sedimentation from the refuge area. Approximately 1.55 square miles of overland area will drain into the moist soil management unit. The elevation area and capacity curves for the project are shown on plate A-1.

CLIMATE

The climate in east-central Iowa is characterized by extreme temperatures and moderate precipitation. The National Weather Service operates a weather station in Moline, Illinois, located about 25 miles north of Andalusia, which has over 50 years of record. Temperatures range from a maximum of 107 degrees Fahrenheit in the summer to a minimum of -26 degrees Fahrenheit in the winter. The normal temperature is 49.5 degrees Fahrenheit.

Most of the precipitation occurs in summer and fall months, with May, June, and July normally the wettest months, having a monthly average of over 4 inches. Winters are normally the driest parts of the year. The average annual precipitation is 37.2 inches and the average annual snowfall is 28 inches. Table A-1, shown below, lists the appropriate monthly precipitation amounts at the Moline gage for the 36 years of record during the periods 1951 to 1987.

TABLE A-1

Average Monthly Precipitation

Month	Inches	Month	Inches
January	1.64	July	4.88
February	1.30	August	3.76
March	2.77	September	3.74
April	3.97	October	2.70
May	4.21	November	2.16
June	4.32	December	1.92

HYDROLOGY

Mississippi River discharge frequency relationships and corresponding water surface profiles were promulgated by the Upper Mississippi River Basin Commission (UMRBC) in a November 1979 study entitled Upper Mississippi River Water Surface Profiles, River Mile 0.0 to River Mile 847.5. Plate A-2 presents pertinent data from this study. Actual water elevations are recorded daily at Fairport, Iowa (RM 462.0). Plates 5 and 6 of the main report show daily stage hydrographs for the period of record 1967 through 1986 (gage zero equals 535.16 feet above mean sea level (MSL). These data were used to compute monthly and year-round elevation duration relationships for the project site as presented on plates A-3 through A-6. The 50 percent duration elevation can be interpreted as the average elevation. The months of July, August, and September have the lowest normal elevations, referenced to feet above MSL, of 545.4, 545.3, and 545.3, respectively. The year round normal elevation is about 545.5 feet. Typical floods appear to last for at least 25 days and raise the water surface about 5 feet.

SEDIMENT CONDITIONS

Historical records of past sedimentation rates are essentially nonexistent. A paper by J. Roger McHenry dated March 1981 entitled "Recent Sedimentation Rates in Two Backwater Channel Lakes, Pool 14, Mississippi River" indicates widely varying deposition rates, with an average of about 0.1 foot per year. Diversion of the upland drainage from the refuge area and the proposed levee with 2-year flood protection will decrease the sedimentation rate. A detailed discussion of sedimentation is presented in Section 2 of the main report.

HYDRAULICS OF PROPOSED PROJECT CONDITION

The proposed project includes a levee constructed to provide protection from the 2-year flood event. The levee height will be 552.8 feet MSL at the most upstream end and slope to 550.8 feet MSL at the most downstream end. The levee will be tied into the natural ground elevation of 551.8 feet MSL at both ends as shown on plates 8 and 12 of the main report.

Located at the downstream end of the levee is a 600-foot armored section designed for overflow purposes. The overflow section was designed to be the area where overtopping will first occur during flood events greater than the 2-year frequency. Once overtopping of the overflow section occurs, the interior of the levee will fill before overtopping of the main levee section occurs. This will equalize the hydrostatic pressure and reduce damage during flood events greater than the 2-year frequency. The riprap for this armored section is discussed in a following section.

The area of conveyance for the 100-year flood event was computed for existing conditions and compared to that of the proposed conditions. There was approximately a 7 percent reduction in the cross-sectional area at the project site. The reduction occurs in the over bank area which does not normally convey much of the flood flow. The estimated difference in flood elevations for all floods is substantially less than 0.1 foot. A channel cross section for existing and proposed conditions is shown on plate A-7. Table A-2 lists the number of times per month the 2-year flood elevation was exceeded during the years 1965 through 1987 at the Fairport gage.

TABLE A-2

Number Of Times The 2-Year Elevation

Was Exceeded (1965-1987)

Month	Number	Month	Number
January	0	July	0
February	0	August	0
March	3	September	0
April	9	October	1
May	7	November	0
June	1	December	0

WATER CONTROL STRUCTURE

A significant aspect of the project is the upstream gated water control structure between Dead Slough and the Refuge as shown on plate 10 of the main report. The purpose of this structure is to allow river water entry into the moist soil unit during non-managed periods of the year. The unit consists of one gate well with a slide gate as shown on plate 21 of the main report. The structure was designed to fill or drain the interior with 200 acre-feet of volume in a 14-day period. It was concluded that an average head of about .5 foot will be available and an area of 1.8 square feet will be required. This will be provided with a 36-inch pipe as shown on plate 21 of the main report, resulting in an average discharge velocity of approximately 4.0 feet per second.

RIPRAP DESIGN

An 18-inch layer of riprap was designed to armor the overflow portion of the levee. The overflow levee will be used in flood events to back water into the refuge area. During the initial stages of overtopping, the overflow section will have water overtopping in free flow, which is considered the most critical time for stability. For floods greater than the 2-year event, the project levee will be in a submerged state and less likely to be damaged.

Routing a typical Mississippi River flood through the overflow portion of the levee resulted in a maximum head of approximately 0.55 foot above the levee crest prior to submergence. Technical

Report NO. 2-650 "Stability of Riprap and Discharge Characteristics, Overflow Embankments, Arkansas River, Arkansas" was referenced in order to determine the stability of the overflow section. The data in this report did not cover small head elevations above the crest. However, interpolation of the data would result in the overflow section being borderline unstable if unprotected.

The overflow portion of the levee is located in a north-south direction and will be exposed to expected wind velocities as high as 70 miles per hour. In accordance with CETN-I-6 "Revised Methods for Wave Forecasting in Shallow Water," it was determined that 2.5-foot waves could be caused as a result of wind velocities, fetch, and depth of water at the project area. Due to the possible unstable condition of the overflow levee and the possible wave attack, a minimum 15-inch layer of riprap was determined to be required to armor the levee. This was based on a density 165 pcf which results in a D50 of .58 feet. However, as discussed in Appendix C - Geotechnical Considerations, based on experience 18-inch thick riprap will be provided for adequate protection during all overflow scenarios. The minimum required riprap design gradation was determined in accordance with procedures in EM 1110-1601 and ETL 1110-2-120. The following is the required minimum and the recommended gradations for the riprap:

Percent Lighter by Weight	Limits of Stone wt., lbs.	
	Minimum	Recommended
100	170-70	150-400
50	50-35	60-170
15	25-10	15-50

The riprap blanket should extend beyond the toe of the bank. A bedding layer 6 inches thick should be provided under the riprap.

Riprap protection is also recommended at the upstream gated water control structure. A horizontal blanket of riprap will be provided to prevent scour during the worst case scenario for which the velocities could be as high as 12 feet per second. This is based on assumed conditions including incorrect operation of the gate, involving opening it with low pond conditions during a 2-year flood event on the river. The required riprap blanket was determined in accordance with procedures in Research report H-70-2, "Erosion and Riprap Requirements at Culvert and Storm-Drain Outlets." The riprap blanket, as shown on plate 21 of the main report, will be 18 inches thick with a D50 of .58 foot. The minimum and recommended riprap gradation are the same as shown above for the overflow portion of the levee.

PUMP SIZE

Another significant aspect of the project is the pump station located at the downstream end of the levee as shown on plate 12 of the main report. The station will be a two pump system with the capability to pump into the river from the moist soil unit during desirable periods and will also have the capability to pump from the river into the moist soil unit during low river events to ensure adequate water depth during critical periods. The effects of normal rainfall, seepage, evaporation, and upland drainage were all considered in the pump design.

One pump was designed with the capability to pump from the refuge area in order to drawdown the refuge from flat pool elevation 545 (MSL) to approximately 543.5 (MSL) within 14 days. This will be accomplished by a 5,000 gallons per minute (gpm) pump. Alternative pump sizes studied were 10,000 and 2,000 gpm. The 10,000 gpm pump would enable the refuge operator to draw down the refuge within 14 days during periods of extended high flow without utilizing gravity flow. The 2,000 and 5,000 gpm pump would utilize gravity flow to draw the refuge area down to elevation 545 feet (MSL) and then complete the drawdown to elevation 543.5 feet (MSL) within 14 days. The time increment for gravity flow from overflow elevation of 550.8 (MSL) to flat pool elevation of 545 (MSL) would depend upon river flood recession. A typical Mississippi River flood will recede approximately .5 foot per day. The Refuge area will drop at about the same rate as the river. Therefore pumping should not be initiated until the flood event passes and normal levels of approximately 545 to 546 (MSL) occur. Operating in this manner is economically consistent with a low operating budget, and meets Refuge objectives. The 5,000 gpm pump was the selected alternative based on being most economical within the desired 14-day drawdown period. The 2,000 gpm pump would require longer running time and would limit the flexibility of the Refuge operator.

The second pump was designed with the capability to raise and maintain the water elevation within the levee from elevation 545 feet (MSL) to 547.0 feet (MSL) within 10 days. This will be provided by a 3,500 gpm pump. A smaller pump would require a longer time to fill the interior and larger size would be unnecessary for typical operating procedures. Table A-3 lists the number of pumping days required to raise the water level within the MSMU to selected elevations from flat pool elevation of 545 feet (MSL). The pumping days shown are conservative estimates because average rainfall was not considered. Studies indicate that evaporation during typical pumping periods is negligible.

TABLE A-3

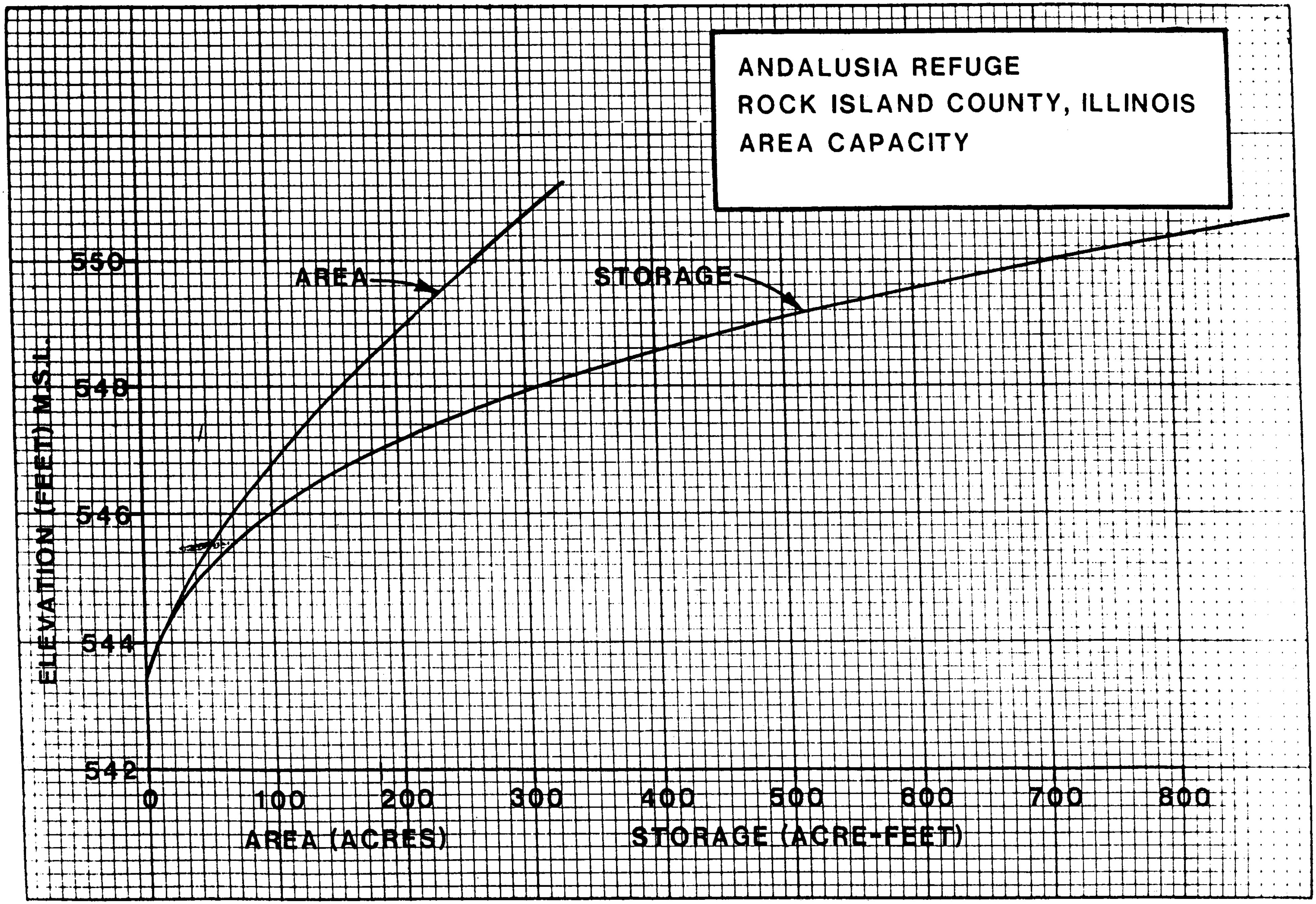
PUMPING DAYS REQUIRED

FOR 3500 GPM PUMP

ELEVATION ft. (MSL)	PUMPING DAYS
546	3.5
547	9.5
548	17
549	28
550	43
550.8	55

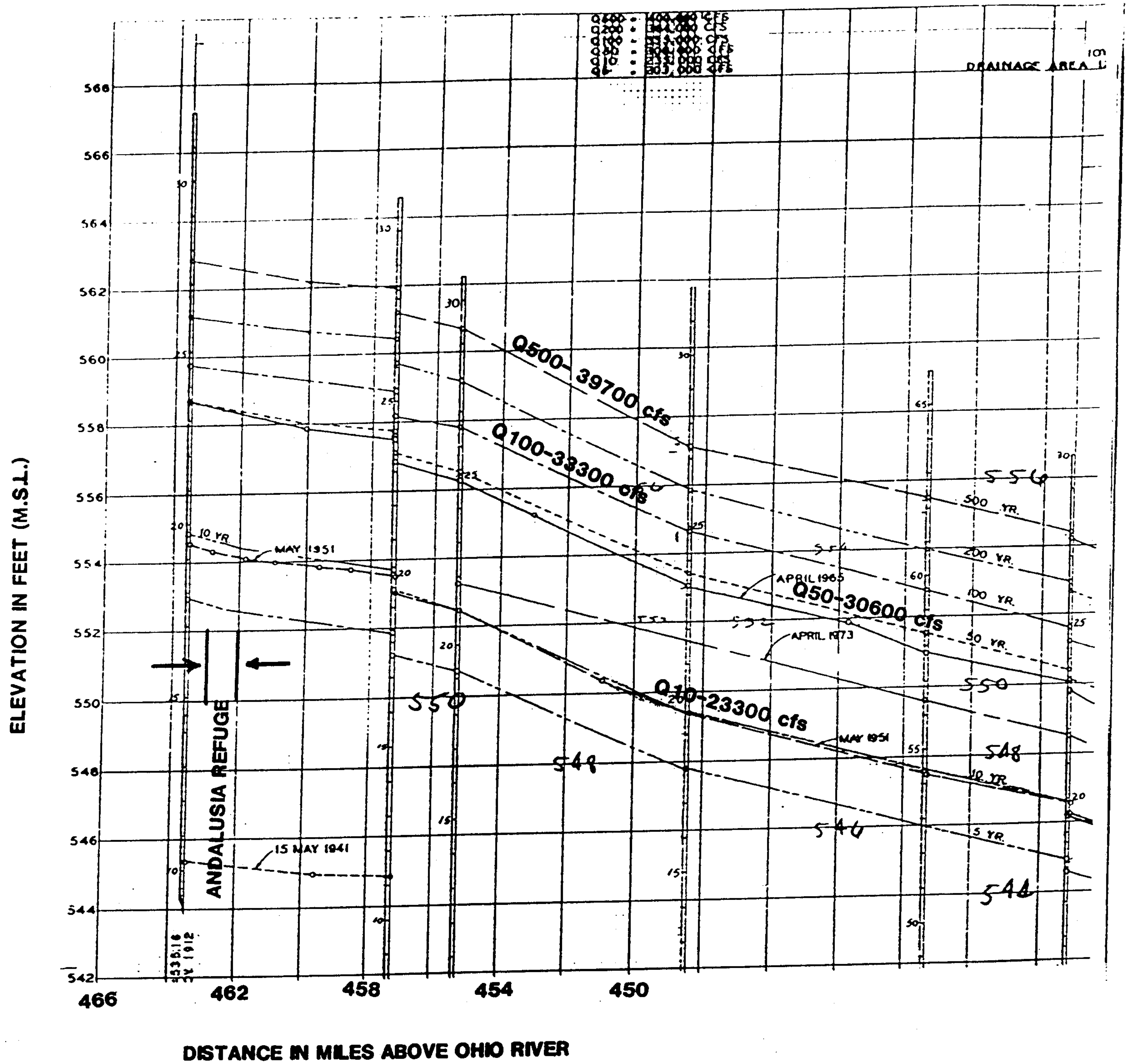
DIVERSION DITCH DESIGN

The drainage ditch shown on plate 8 of the main report was diverted because of the objective of reducing upland erosion sedimentation from entering the project site. The ditch has a 1.8-square-mile drainage area and was designed for a discharge of 340 cubic feet per second, which is approximately the 2-year frequency flood and is consistent with the existing ditch capacity. The ditch cross section should have a 30-foot bottom width and 3:1 side slopes. A profile slope of .0025 ft./ft. was designed to match existing profile conditions as close as possible, as shown on plate 16 of the main report. A design flood will result in an average velocity of approximately 3 feet per second.



ANDALUSIA REFUGE
ROCK ISLAND COUNTY, ILLINOIS
AREA CAPACITY
AREA
STORAGE
550
548
546
544
542
ELEVATION (FEET) M.S.L.
0
100
200
300
400
500
600
700
800
AREA (ACRES)
STORAGE (ACRE-FEET)

PLATE A-1



REVISION	DATE	DESCRIPTION	BY

CORPS OF ENGINEERS, U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
ROCK ISLAND, ILLINOIS

DRAWN BY: GEH
TRACED BY:
CHECKED BY: JMS
SUBMITTED:

CHIEF HYDRAULICS BRANCH
APPROVED:

CHIEF, ENGINEERING DIVISION

DATE:

UPPER MISSISSIPPI RIVER

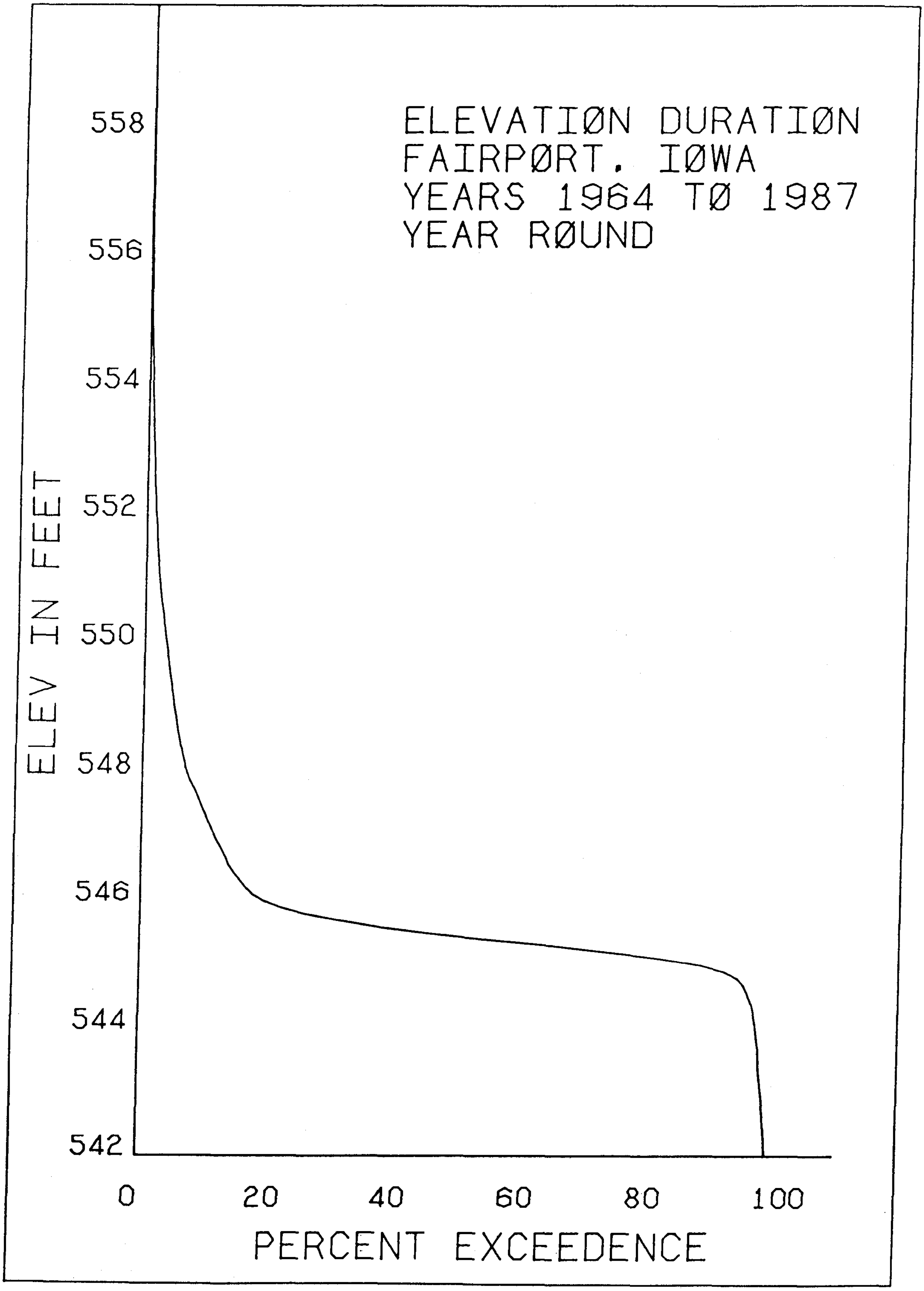
STANDARD FLOOD PROFILES

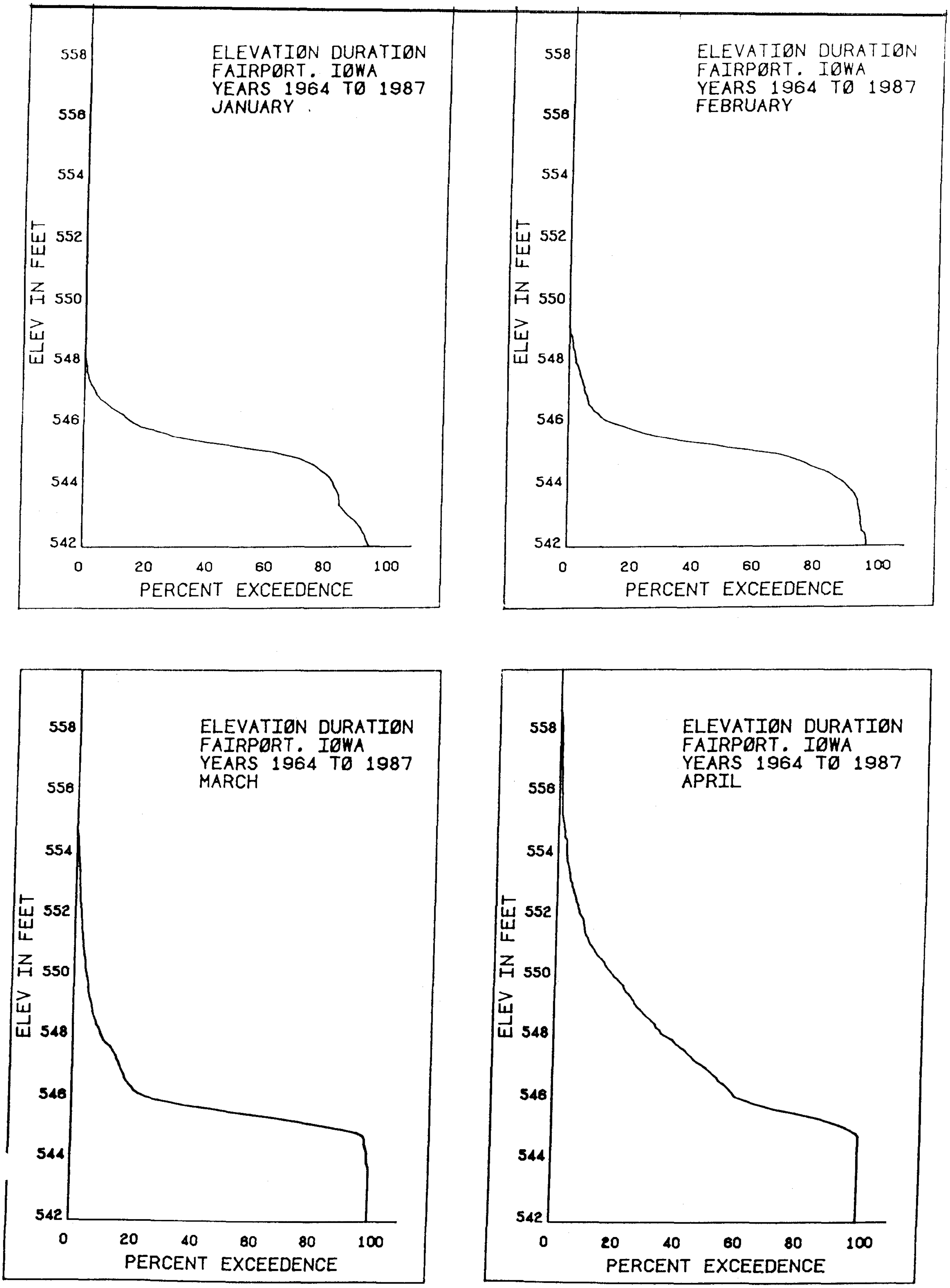
ROCK ISLAND DISTRICT

5, 10, 50, 100, 200, & 500 YEAR FLOOD

RIVER MILES 518.52 TO 463.5

PLATE A-2


ELEVATIØN DURATIØN
FAIRPØRT, IØWA
YEARS 1964 TØ 1987
YEAR RØUND
558
556
554
552
550
548
546
544
542
ELEV IN FEET
0
20
40
60
80
100
PERCENT EXCEEDENCE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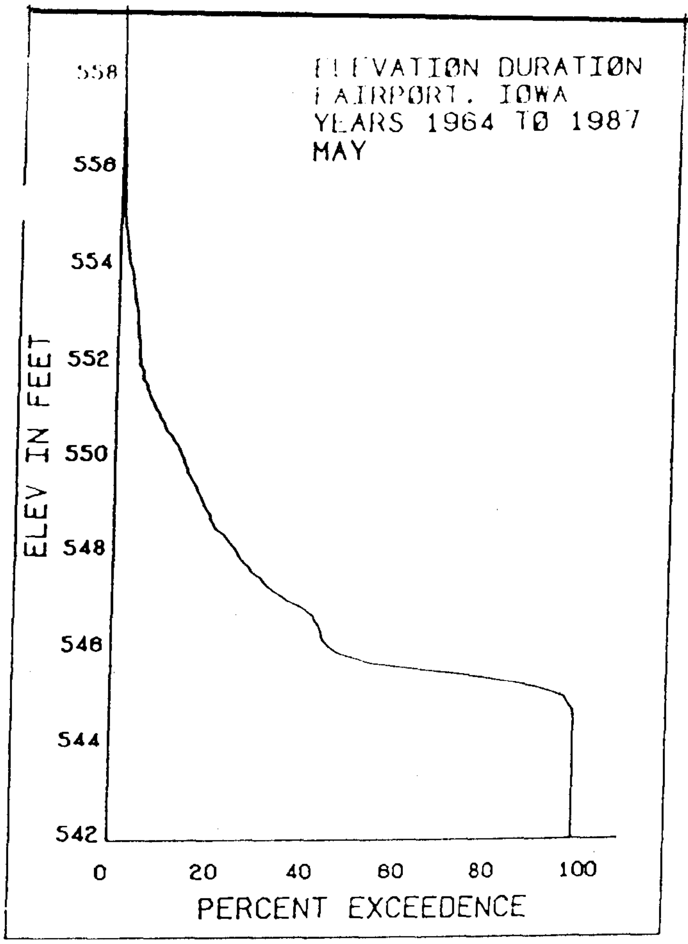
FAIRPØRT. IØWA
YEARS 1964 TØ 1987
JANUARY
ELEV IN FEET
558
558
554
552
550
548
546
544
542
0
20
40
60
80
100
PERCENT EXCEEDENCE
ELEVATIØN DURATIØN
FAIRPØRT. IØWA
YEARS 1964 TØ 1987
FEBRUARY
ELEV IN FEET
558
558
554
552
550
548
546
544
542
0
20
40
60
80
100
PERCENT EXCEEDENCE
ELEVATIØN DURATIØN
FAIRPØRT. IØWA
YEARS 1964 TØ 1987
MARCH
ELEV IN FEET
558
556
554
552
550
548
546
544
542
0
20
40
60
80
100
PERCENT EXCEEDENCE
ELEVATIØN DURATIØN
FAIRPØRT. IØWA
YEARS 1964 TØ 1987
APRIL
ELEV IN FEET
558
556
554
552
550
548
546
544
542
0
20
40
60
80
100
PERCENT EXCEEDENCE

PLATE A-4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FAIRPØRT. IØWA
YEARS 1964 TØ 1987
MAY
ELEV IN FEET
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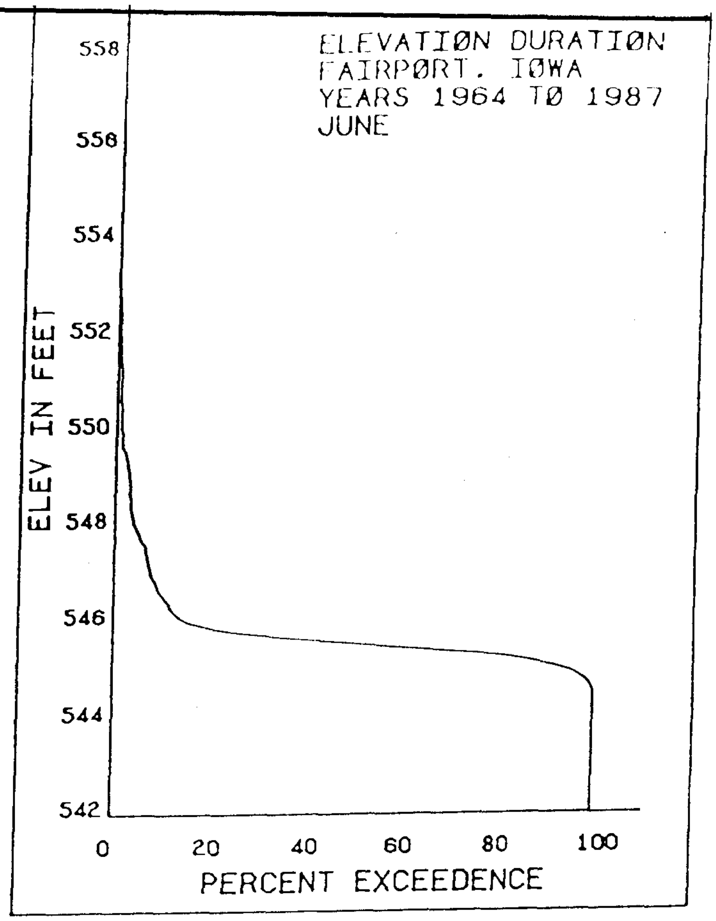
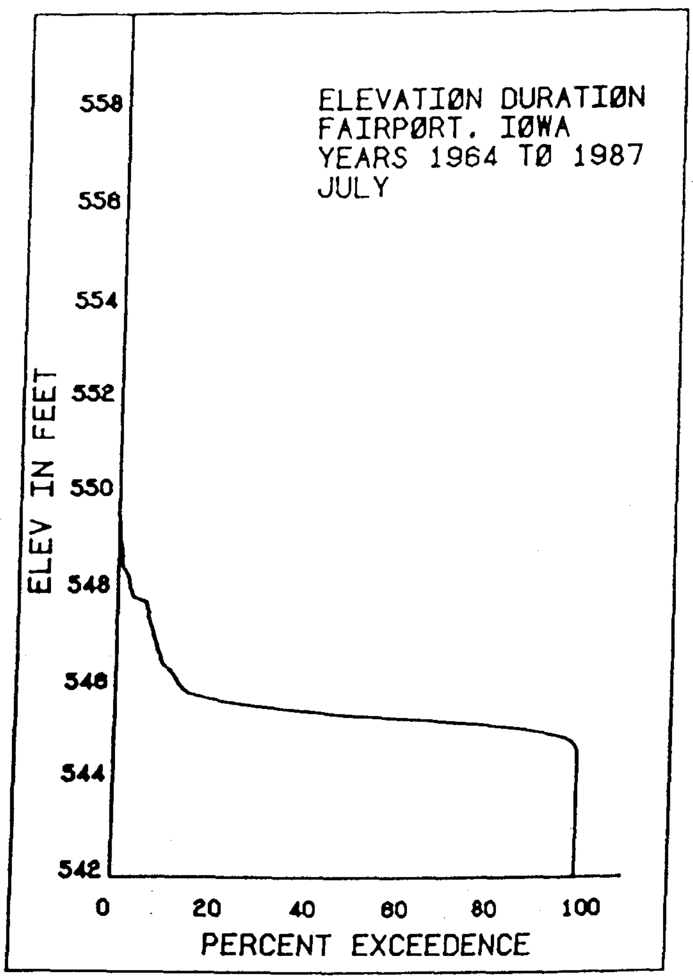
556
554
552
550
548
546
544
542
0
20
40
60
80
100
PERCENT EXCEEDENCE


ELEVATIØN DURATIØN
FAIRPØRT. IØWA
YEARS 1964 TØ 1987
JUNE
ELEV IN FEET
558
556
554
552
550
548
546
544
542
0
20
40
60
80
100
PERCENT EXCEEDENCE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FAIRPØRT. IØWA
YEARS 1964 TØ 1987
JULY
ELEV IN FEET
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546
544
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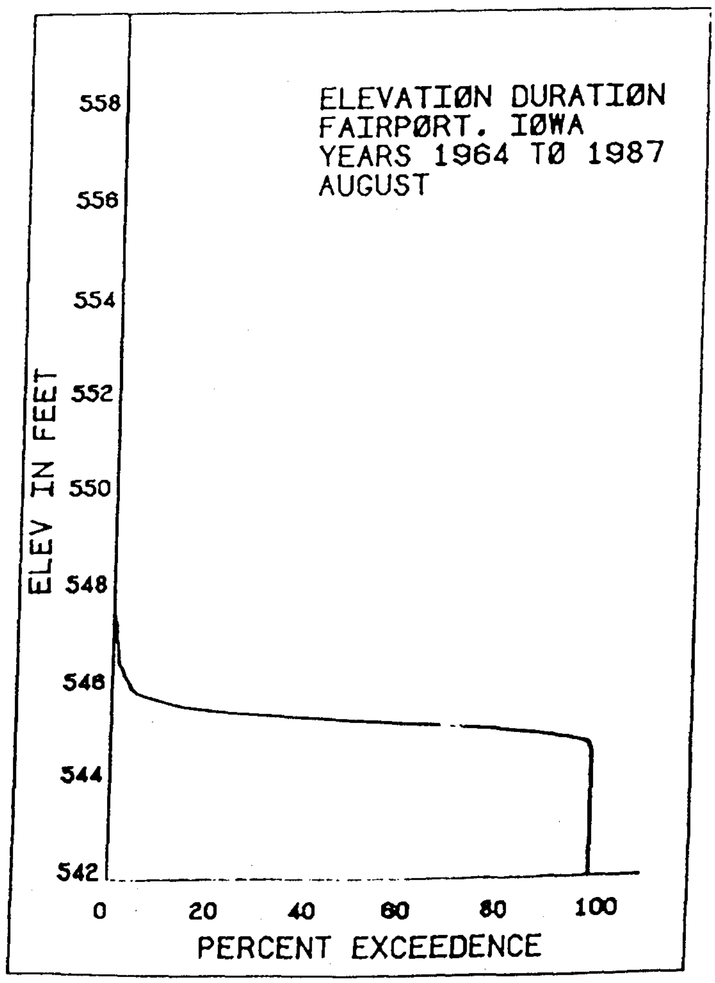
40
60
80
100
PERCENT EXCEEDENCE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FAIRPØRT. IØWA
YEARS 1964 TØ 1987
AUGUST
ELEV IN FEET
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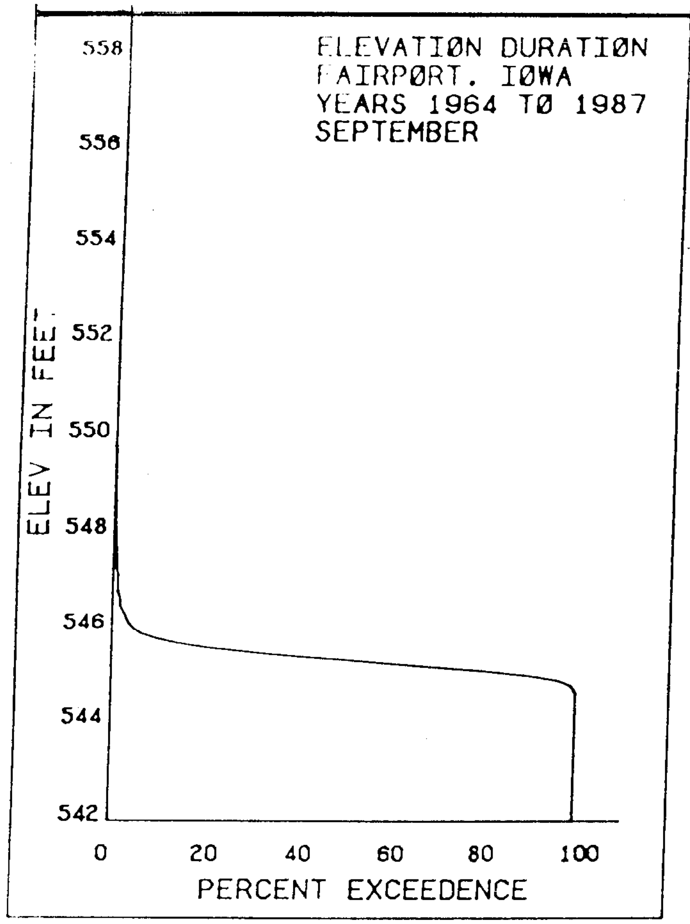
556
554
552
550
548
546
544
542
0
20
40
60
80
100
PERCENT EXCEEDENCE

PLATE A-5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FAIRPØRT. IØWA
YEARS 1964 TØ 1987
SEPTEMBER
ELEV IN FEET
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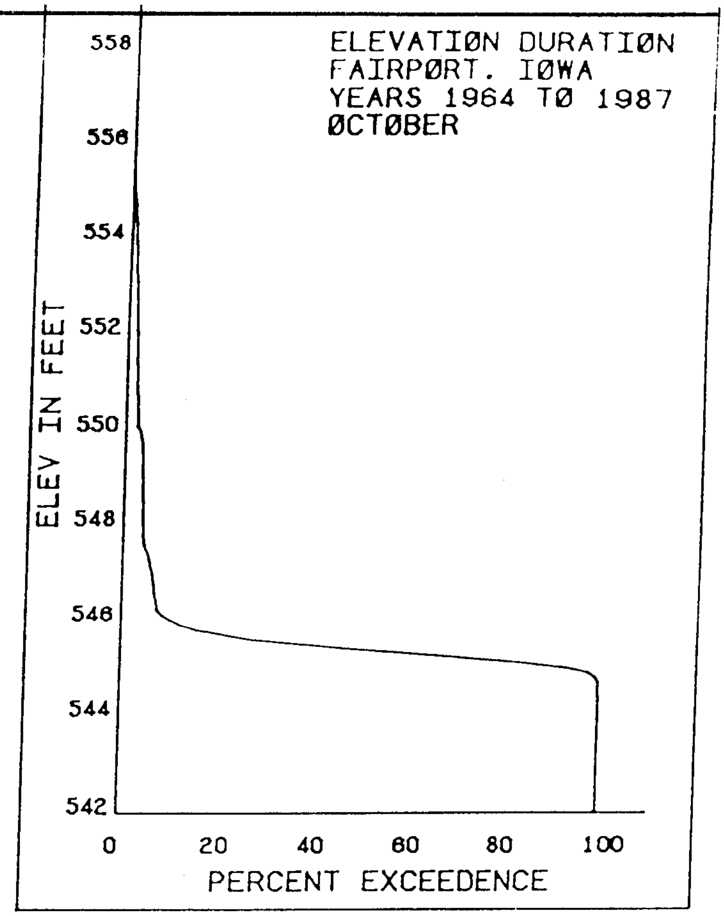
556
554
552
550
548
546
544
542
0
20
40
60
80
100
PERCENT EXCEEDENCE


ELEVATIØN DURATIØN
FAIRPØRT. IØWA
YEARS 1964 TØ 1987
ØCTØBER
ELEV IN FEET
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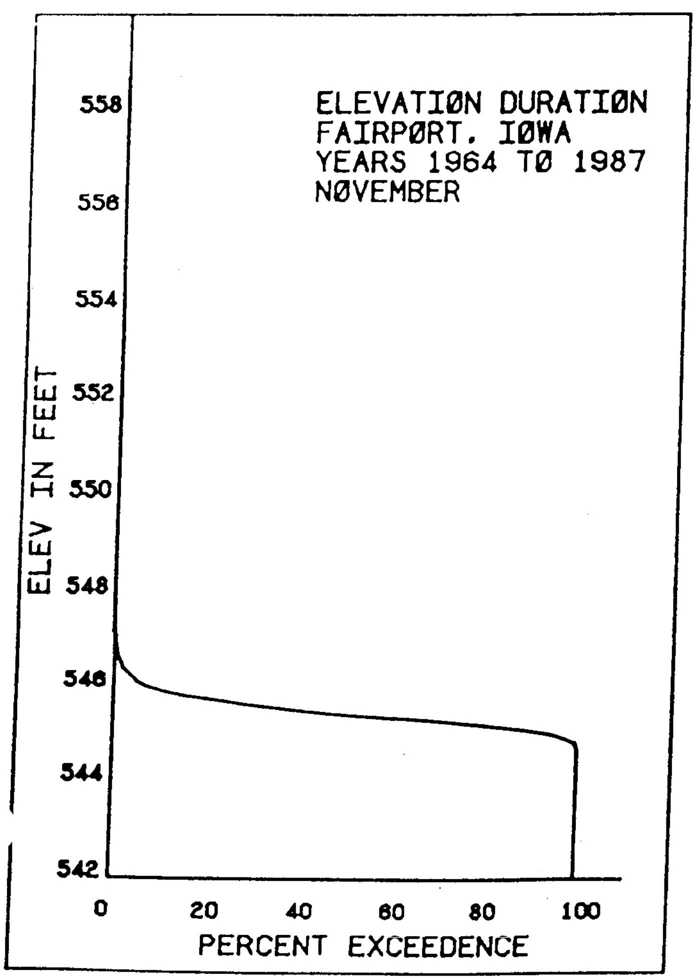
556
554
552
550
548
546
544
542
0
20
40
60
80
100
PERCENT EXCEEDENCE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FAIRPØRT. IØWA
YEARS 1964 TØ 1987
NØVEMBER
ELEV IN FEET
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548
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544
542
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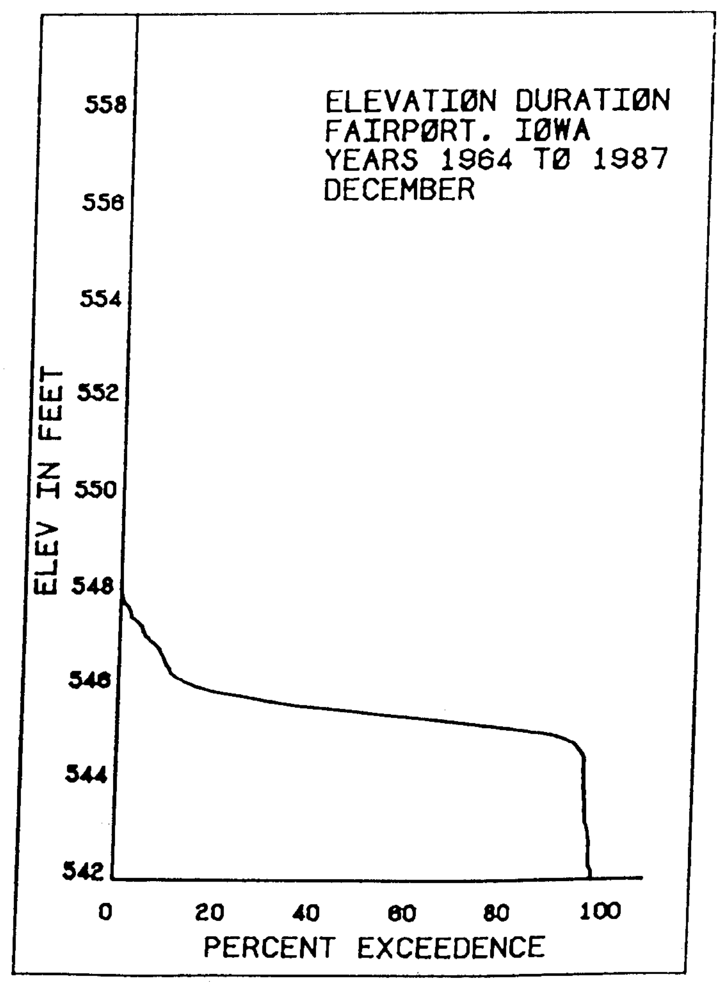
20
40
60
80
100
PERCENT EXCEEDENCE


ELEVATIØN DURATIØN
FAIRPØRT. IØWA
YEARS 1964 TØ 1987
DECEMBER
ELEV IN FEET
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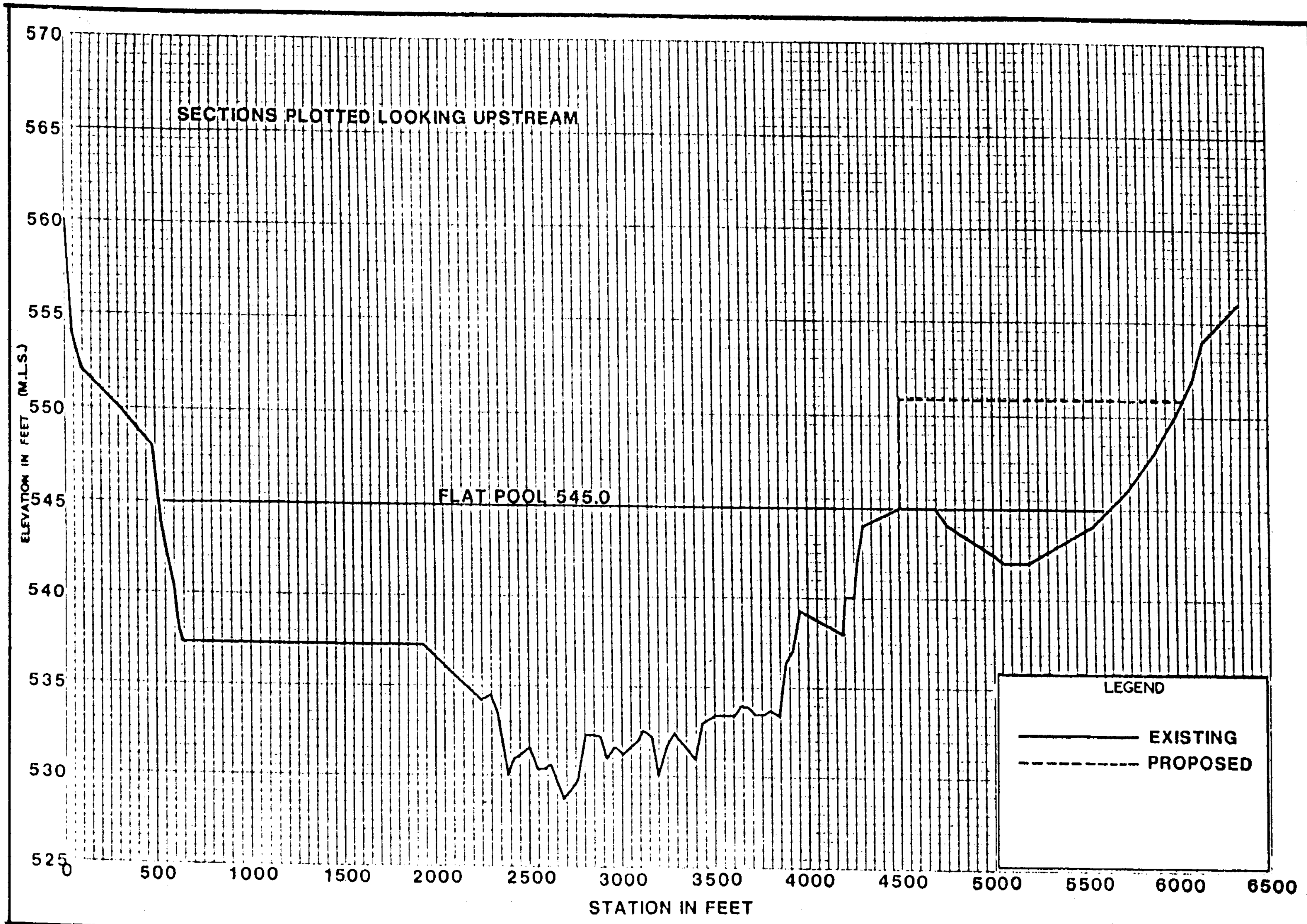
556
554
552
550
548
546
544
542
0
20
40
60
80
100
PERCENT EXCEEDENCE

PLATE A-6



CHANNEL SECTION AT RIVER MILE 462.0

MISSISSIPPI RIVER

ANDALUSIA REFUGE

PLATE A-7

WATER QUALITY

A
P
P
E
N
D
I
X

B

UPPER MISSISSIPPI RIVER SYSTEM
ENVIRONMENTAL MANAGEMENT PROGRAM
DEFINITE PROJECT REPORT (R-4)

ANDALUSIA REFUGE REHABILITATION AND ENHANCEMENT

POOL 16, MISSISSIPPI RIVER MILES 462 THROUGH 463
ROCK ISLAND COUNTY, ILLINOIS

APPENDIX B
WATER QUALITY

OVERVIEW

Several water quality related factors bear on the suitabliity of the backwater complex to support valuable aquatic and semi-aquatic species. These include the chemical composition of the water, the chemical and physical composition of the sediment, and the depth and availability of water throughout the year. Representatives of the Iowa Department of Natural Resources (DNR) indicated that water quality within Andalusia Slough is currently adequate to support the native fisheries during the summer months. However, under ice cover, conditions develop which result in periods where dissolved oxygen (D.O.) becomes depleted to the point where fish kills can occur. In fact, fish kills have been observed on several occasions during the early spring immediately after ice out. Although it is not known with certainty that the cause of these fish kills is low D.O., it is reasonable to assume that it is at least a contributing factor. As the entire backwater area is heavily laden with aquatic vascular plants during the summer months, it is easy to envision the decomposition of this organic material during the winter leading to low D.O. levels.

In order to assess the existing water quality situation and predict the impacts of any enhancement efforts, a monitoring program was initiated in 1987. Water samples were taken every 2 weeks during the summer and less frequently during the remainder of the year. In addition, sediment and elutriate samples were collected. These data provide the basis for the assessment of water quality within the study area.

METHODS

Ambient water samples were collected on eight occasions between January and September 1987. An attempt to collect samples was made on two other occasions, however, insufficient water depth existed to permit taking representative samples. On January 28, 1987, samples were taken from the ice at locations R2 and R3 shown on plate 23 of the main report. On seven occasions, samples were taken from boat at the single location A1 shown on plate 23. Due to the shallow water and abundant aquatic plant growth, it was not possible to collect samples during the summer from the immediate project area. The location selected was as close to the project site as water conditions would allow. Due to the lack of significant flow through the backwater area and the relative proximity of the winter and summer sampling locations, it is quite likely that little, if any, difference in water quality exists between the sampling sites. In all cases, grab samples were taken from immediately below the surface using a Kemmerer sampler. Field analyses (temperature, pH, D.O., specific conductance and secchi disk depth) were performed immediately, while the samples requiring laboratory analysis were appropriately preserved, placed on ice, and shipped the same day they were collected.

Sediment and elutriate samples were taken at six locations on August 12, 1988. The locations are shown on plate 23 and coincide quite closely with the locations of the surface samples collected on January 28, 1987. Locations R1 - R3 were taken using a 48-inch coring device. The resulting cores were between 24 and 36 inches in length. At locations L1 - L3, no water was present and the soil was quite dry and compacted. Samples at these locations were taken using a shovel and were from the upper 1 to 2 feet of the soil. All water samples taken for the purpose of preparing the elutriate samples were collected and handled in the manner described above. All sediment samples were placed on ice and shipped to the laboratory the same day they were collected.

Grain size analyses were performed in accordance with U.S. Army Corps of Engineers, Engineer Manual 1110-2-1906, Appendix V, November 1970. Chemical analyses were performed according to "Standard Methods for the Examination of Water and Wastewater," 16th Edition, American Public Health Association, Washington, D.C., 1985. Elutriate samples were prepared by mixing 1 part sediment with 4 parts ambient water, shaking for 30 minutes, and allowing 4 hours to settle.

RESULTS

Results of all field and laboratory analyses are presented in tables B-1 through B-4. Table B-1 lists the results of grain size analyses of samples collected on June 21, 1988. It is apparent from the results that the sediment is very fine throughout the backwater area. For a complete hydrometer analysis, see appendix C. Table B-2 lists the results of all laboratory and field tests performed on ambient water samples. From the results it can be seen that D.O. concentrations are low during several weeks of the observation period. While levels do not fall below 4.0 mg/l, they approach this level and probably do fall below this level during the night. The chlorophyll concentrations indicate that phytoplankton are quite abundant at certain times and probably contribute to fluctuations in D.O. concentrations.

Even though samples were taken some distance from the actual project location due to access problems, water depth was still quite shallow and was barely adequate to permit reaching this point by boat. This was true despite the fact that the river was at or above flat pool each day that samples were collected.

Table B-3 lists the results of bulk sediment analyses performed on samples collected on August 12, 1988. As can be seen from the data, all inorganic contaminants except total volatile solids fell in the range considered to be nonpolluted based on EPA's draft criteria for Great Lakes sediment. Total volatile solids undoubtedly exceeded the criteria due to the large amount of detritus present in the sediments. There is no evidence of large concentrations of soluble organic contaminants as seen from the fact that all pesticide concentrations were below the detection limits.

Table B-4 lists the results of elutriate analyses performed on samples collected on August 12, 1988. As can be seen from the data, concentrations of most parameters were quite low. The only exception to this is ammonia nitrogen. Concentrations were observed which exceeded the general water quality standards at two locations. All pesticide concentrations were below the detection limits.

CONCLUSIONS

Based on field observations and analytical results, water quality within the project area appears adequate to support aquatic life during the majority of the time. During the summer there may be periods when D.O. approaches levels considered to be detrimental to certain fish species. This was observed during the study period, although no fish kills were observed. During the winter there may be ice and snow conditions, which, in combination with decayed organic matter, could develop into a "winter kill." Although this was not observed during the study period, it has been reported by DNR personnel. Results from the analyses of sediment and elutriate samples show no excessive concentrations of contaminants as compared with interim EPA criteria for Great Lakes Sediment and the State water quality standards, with the exception of ammonia nitrogen. Concentrations of this parameter should be viewed in light of the proposed mixing zone, and, if necessary, toxic effects can be minimized by coordinating construction with those periods when water temperature and pH are low.

TABLE B-1

Grain Size Analyses

Location	Percentage Passing a #230 Sieve (<0.062um)
1L	98.2
1R	99.7
2L	93.6
2R	98.2
3R	83.6
3L	82.5

TABLE B-2

Ambient Water Quality Results, 1987

Parameter	1/28	1/28	6/8	6/22	7/6	7/20	8/10	8/24	9/8	9/21
Location	R3	R2	A1	A1	A1	A1	A1	A1	A1	A1
Time	1030	1045	1030	1020	1025	0925	1055	1055	1025	1205
Ice Thickness (cm)	12.5	12.5	-	-	-	-	-	-	-	-
Water Temp. (C)	-	-	25.0	26.7	26.7	27.8	24.4	20.0	21.7	15.6
Depth (M)	.3	.5	.9	.8	.7	.6	.6	.7	.8	.7
D.O. (mg/l)	15.4	23.2	6.9	4.3	5.7	6.6	4.4	4.7	4.5	4.9
pH (units)	-	-	7.6	7.2	7.2	7.6	7.3	7.4	7.0	7.2
Sp. Cond. (umhos/cm)	-	-	436	472	408	381	410	381	475	369
Secchi Depth (M)	-	-	1.5	2.0	2.0	2.0	2.0	2.0	2.5	2.0
Sus. Solids (mg/l)	-	-	24	15	10	9	4	3	4	3
Chl. a (mg/cu m)	-	-	46	12	14	6	7	10	8	5
Chl. b (mg/cu m)	-	-	2	2	2	2	2	2	1	1
Chl. c (mg/cu m)	-	-	4	2	2	2	2	2	1	1
Pheo. a (mg/cu m)	-	-	18	13	6	8	4	4	4	2

TABLE B-3

Bulk Sediment Analyses, August 12, 1988 (mg/kg)

Parameter	Location					
	R1	R2	R3	L1	L2	L3
Arsenic (Total)	2.0	2.7	1.8	2.7	4.0	2.9
Barium (Total)	74	67	62	98	88	94
Cadmium (Total)	<0.87	<0.69	<0.82	<0.79	<0.79	<0.68
Chromium (Total)	12	13	12	17	19	13
Copper (Total)	11	11	11	16	17	15
Lead (Total)	6.3	6.3	5.2	17	17	15
Mercury (Total)	<0.019	<0.027	<0.025	<0.031	<0.026	<0.026
Nickel (Total)	15	12	11	20	25	18
Selenium (Total)	<0.87	<0.69	<0.83	<0.79	<0.79	<0.68
Zinc (Total)	67	64	63	91	94	78
Ammonia-N	120	82	89	41	41	37
Total Organic Carbon	7.5%	6.9%	7.7%	8.2%	8.3%	7.9%
Oil and Grease	50	190	190	110	250	360
Total Volatile Solids	9300	19500	13200	12600	4200	13300
Aldrin	<8.0 *	<8.0 *	<8.0 *	<8.0 *	<8.0 *	<8.0 *
Chlordane	<80 *	<80 *	<80 *	<80 *	<80 *	<80 *
DDD	<16 *	<16 *	<16 *	<16 *	<16 *	<16 *
DDE	<16 *	<16 *	<16 *	<16 *	<16 *	<16 *
DDT	<16 *	<16 *	<16 *	<16 *	<16 *	<16 *
Dieldrin	<16 *	<16 *	<16 *	<16 *	<16 *	<16 *
Endrin	<16 *	<16 *	<16 *	<16 *	<16 *	<16 *
Heptachlor	<8.0 *	<8.0 *	<8.0 *	<8.0 *	<8.0 *	<8.0 *
Hepachlor Epoxide	<8.0 *	<8.0 *	<8.0 *	<8.0 *	<8.0 *	<8.0 *
Lindane	<8.0 *	<8.0 *	<8.0 *	<8.0 *	<8.0 *	<8.0 *
Methoxychlor	<80 *	<80 *	<80 *	<80 *	<80 *	<80 *
Toxaphene	<160 *	<160 *	<160 *	<160 *	<160 *	<160 *
2,4-D	— *	— *	— *	— *	— *	— *
2,4,5-TP	— *	— *	— *	— *	— *	— *
Total PCB's	<160 *	<160 *	<160 *	<160 *	<160 *	<160 *

* ug/kg

TABLE B-4

Elutriate Results from August 2, 1988, Sampling (mg/l)

Parameter	Location						Ambient Water
	L1	L2	L3	R1	R2	R3	R2
Arsenic	<0.003	<0.003	<0.003	<0.003	<0.003	<0.003	0.012
Barium	0.07	0.08	0.08	0.01	0.09	0.10	0.07
Cadmium	<0.005	<0.005	<0.005	<0.005	<0.005	<0.005	<0.005
Chromium	<0.009	<0.009	<0.009	0.01	<0.009	<0.009	0.03
Copper	<0.009	<0.009	<0.009	<0.009	<0.009	<0.009	<0.009
Lead	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001	<0.001
Mercury	<0.0001	<0.0001	<0.0001	<0.0001	<0.0002	<0.0001	<0.0001
Nickel	<0.025	<0.025	0.03	0.12	0.06	0.03	0.03
Selenium	<0.005	<0.005	<0.005	<0.005	<0.005	<0.005	<0.005
Zinc	0.03	0.09	0.11	0.11	0.09	0.11	0.04
Ammonia-N	4.2	2.4	2.9	15	18	0.13	0.13
Total Vol Solids	530	450	420	580	640	420	300
Oil and Grease	4.0	16	6.0	<2.0	32	<2.0	-
TOC	48	47	16	1100	50	42	13
Aldrin*	<0.10	<0.10	<0.15	<0.10	<0.10	<0.05	<0.05
Chlordane*	<1.0	<1.0	<1.5	<1.0	<1.0	<0.50	<0.50
DDD*	<0.20	<0.20	<0.30	<0.20	<0.20	<0.10	<0.10
DDE*	<0.20	<0.20	<0.30	<0.20	<0.20	<0.10	<0.10
DDT*	<0.20	<0.20	<0.30	<0.20	<0.20	<0.10	<0.10
Dieldrin*	<0.20	<0.20	<0.30	<0.20	<0.20	<0.10	<0.10
Endrin*	<0.20	<0.20	<0.30	<0.20	<0.20	<0.10	<0.10
Heptachlor*	<0.10	<0.10	<0.15	<0.10	<0.10	<0.05	<0.05
Heptachlor Epoxide*	<0.10	<0.10	<0.15	<0.10	<0.10	<0.05	<0.05
Lindane*	<0.10	<0.10	<0.15	<0.10	<0.10	<0.05	<0.05
Methoxychlor*	<1.0	<1.0	<1.5	<1.0	<1.0	<0.50	<0.50
Toxaphene*	<2.0	<2.0	<3.0	<2.0	<2.0	<1.0	<1.0
2,4-D*	-	-	-	-	-	-	-
2,4,5-TP*	-	-	-	-	-	-	-
Total PCB's*	<2.0	<2.0	<3.0	<2.0	<2.0	<1.0	<1.0

* Concentrations of all organics are expressed as ug/l.

GEOTECHNICAL CONSIDERATIONS

A
P
P
E
N
D
I
X

C

UPPER MISSISSIPPI RIVER
ENVIRONMENTAL MANAGEMENT PROGRAM
DEFINITE PROJECT REPORT

ANDALUSIA REFUGE
REHABILITATION AND ENHANCEMENT
POOL 16 MISSISSIPPI RIVER MILES 462 THROUGH 463
ROCK ISLAND COUNTY, ILLINOIS

APPENDIX C
GEOTECHNICAL CONSIDERATIONS

TABLE OF CONTENTS

List of Plates

LOCATION

The Andalusia EMP Project is situated within the Upper Mississippi Wildlife and Fish Refuge, between river mile 462 and 463. The site is bordered by Dead Slough directly to the north, forested loess-covered bluffs to the south, which are part of 393 acres managed by the State of Illinois, Department of Conservation. The major part of Andalusia Slough continues to the east; and Drury Slough to the west. The project area lies within the Galesberg Plain section of the Central Lowlands Province.

PHYSIOGRAPHY

The topography within the project site consists of a series of sloughs and shallow backwater lakes. Site elevation varies from 546-548 feet MSL (Mean Sea Level). Land surface configuration was originally controlled by the shape of the underlying bedrock surface. The Mississippi River valley is constricted from Andalusia to Muscatine with little or no flood plain. At the point of the project site, the valley is only about 1-1/4 miles wide. The area is known as the lower part of the "upper narrows" (Savage 1921). This narrowing is caused by the unusual thickness of Pennsylvanian age sandstone which the river runs across. On the valley walls, the erosional slopes are concealed by a mantle of unconsolidated material derived from slumping and landslides over underlying Pennsylvanian age shales.

The presence of more resistant bedrock is indicated by the steep lower slopes of the valley bedrock. Maximum relief of valley sides adjacent to the project site is 100-150 feet (Horberg 1956b).

BEDROCK

The bedrock of the project area consists of Pennsylvanian age sandstones, shales, and coals, setting unconformably on top of Devonian age shale and limestones (Fitzgerald 1985). Middle Devonian age limestones outcrop 10 miles upstream in Buffalo, Iowa and in the near by town of Andalusia, Illinois. These rocks are almost horizontal and in general slope toward the southwest at an average rate of 10 feet per mile. Overall the Devonian age rocks are about 140 feet thick. There is a Middle Devonian age shale lying directly beneath a thick section of Pennsylvanian age rocks at Wyoming Hill, and outcrop about a mile downstream from the project site on the Iowa side of the river. Overall the Pennsylvanian age rocks are up to 150 feet thick; but at Wyoming

Hill, along highway 22, NE 1/4, Sec. 34, T77N, RIW, (Horberg 1956b), the outcrop is about 100 feet thick, and consists of sandstones shales, and coals.

PLEISTOCENE AND RECENT DEPOSITS

Above, and to some extent within the Mississippi River, and tributary valleys are deposits of Pleistocene drift or till, loess, terrace deposits, recent alluvium, and dune deposits. Maximum thickness of these deposits do not exceed 100 feet around project area (Savage 1921)(Horberg 1956b). Pleistocene deposits resulted from the numerous advances and retreats of glaciers which blanketed this entire area. A road cut 4 miles upstream from the project site at Loud Thunder Forest Preserve, SW 1/4, SE 1/4, Sec. 27, TT7N, R4W, exposes 43 feet of Pleistocene sediments. The outcrop includes 23 feet of Pre-Illinoian age tills, 8 feet of Illinoian age tills, and 12 feet of Wisconsinan age, Peorian loess (Horberg 1956b).

In the Mississippi River valley at the mouth of the adjacent watersheds, the sediments consists of alluvial deposits. These alluvial deposits are unconsolidated glacial outwash sands and gravels on bedrock, with deposits of alluvial silts and clays on top. The outwash sands and gravels are of the Henry formation of Wisconsinan age, overlain by Cahokia alluvial silts and clays, Wisconsinan or Holocene in age (Willman 1970). Thicknesses of alluvial deposits along the Mississippi River from Le Claire, Iowa to Muscatine, Iowa usually varies from 15 to 45 feet (Savage 1921). At the project site, borings were drilled no futher than 30.5 feet and bedrock was not encountered.

SUBSURFACE EXPLORATIONS

Access to the project site was limited by surface water. During February 1987, eight primary borings A-87-1 through A-87-8 were taken. Borings A-87-1, 2, 5, 6, and 7 were obtained by hand with a 4-inch Iwan Auger. Borings A-87-3, 4, and 8 were obtained with a CME-45 drill rig using a 5-inch hallow stem auger.

During January and February 1988, fourteen additional exploratory borings were taken. These numbered A-88-1 through A-88-14. Holes A-88-6 and A-88-8 were obtained by using 4-inch Iwan hand auger. The other holes were obtained using CME-45 drill rig. Locations of the borings and boring logs are show on plate 2 of the main report. The deepest boring taken with the drilling rig extended to a depth of 30.5 feet, approximate elevation 517.8 feet MSL.

PERIMETER LEVEE EMBANKMENT

The proposed perimeter levee as shown on plate 2 of the main report, is approximately 6 feet high, and approximately 8,600 feet long. The purpose of the levee is to create a moist soil management unit with controlled water levels for wildlife habitat on the landside of the levee. The crown of the levee will be at least 12 feet wide for ease of construction and normal maintenance and operation. The side slopes of the levee will be 1 vertical (V) on 4 horizontal (H). From station 12+00 to 8+00 CE, the proposed levee will be wider (60-foot wide) compared to other reaches of the levee. This portion of the levee will be built with clay borrowed from the Dead Slough excavation. The majority of the excavated material from Dead Slough will be placed in the adjacent levee as a thicker, instead of transporting it to a disposal site. The typical cross sections of the proposed levee are shown on plate 18 of the main report.

From station 12+21 C to station 25+18±CE, the side slopes of the levee will be grass seeded since a heavy timber growth is evident on both sides of the proposed levee. Therefore, it is anticipated that grass protection will be adequate against wave wash. From stations 25+18±CE to 29+18±CE, the profile of the levee will be placed on a steeper gradient than the natural river flood profile to ensure overtopping occurs from the downstream end. Therefore, both side slopes of the levee will be protected against the wave wash and current action by an 18-inch thickness of riprap with the following gradation:

Percent Lighter by Weight	Weight of Stone in Pounds
100	150-400
50	60-170
15	15- 50

A similar gradation used on various similar installations has served satisfactorily for several years. A bedding layer of 6-inch thickness will be of the following gradation:

U.S. Standard Sieve Size	Percentage Passing (By Weight)
1 - 1/2	85 - 100
3/4	40 - 85
3/8	15 - 45
No. 4	0 - 20
No. 8	0 - 5

The entire levee will be built with uncompacted impervious material lie, (ie, fill place by casting).

FOUNDATION FOR EMBANKMENTS

The entire foundation beneath the proposed levee embankment will be stripped of vegetation and other deleterious materials to a depth of 6 inches. All top roots, lateral roots, and trees within the embankment foundation areas will be removed to a depth of 3 feet below natural ground surface.

An extensive field investigation was made to ascertain the proposed levee foundation conditions. According to borings which were pertinent to approximately 6 feet high perimeter levee foundation analyses, the foundation material consists of alluvial deposits. Boring logs are shown on plate 7 of the main report. The top stratum varies in thickness from 16 to 20 feet, and consists of normally consolidated impervious alluvial deposits (SC, CL, CL-CH, and CH). The moisture content ranges from 24 to 37 percent for CL soils, 27 to 35 for CL-CH soils, and 29 to 48 for CH soils. Borings A-88-8, A-88-9, and A-88-14 show that the top 1-foot consists of slightly organic clay with moisture content of 85 percent.

The Atterberg limits testing was performed on the selected soil samples after throughly evaluating each soil sample. Atterberg limits testing reveals a range from 44/16 (liquid limit/plastic limit) to 44/18 for CL soils, 55/21 to 58/22 for CL-CH soils, and 71/28 to 82/29 for CH soils. The standard penetration test "N" values recorded during the drilling operations for top stratum ranged from 2 to 9 blow counts, with average "N" values of 6. The shear strength of the top stratum based on standard penetration tests varies from 250 psf to 1,125 psf with an average of 750 psf. The pocket penetrometer tests were also run on the selected clay samples. The pocket penetrometer tests indicate a range in cohension from 250 to 1250 psf.

The soils below the impervious substratum are found to be medium to fine sand (SP). The "N" values obtained for the sand ranged from 4 to 23, with average "N" values of 11. Detailed descriptions of the encountered materials are shown on boring logs, on plate 7 of the main report. None of these borings were extended to bedrock.

FOUNDATION FOR OTHER STRUCTURES

A water control structure near station 21+00 will be built as a part of the proposed project. The location of the proposed structure is shown on plate 2 of the main report. Boring A-88-3, 32 feet deep was taken to evaluate physical characteristic of subsurface conditions. Detailed descriptions of soils encountered are shown on boring logs, see plate 7 of the main report. The boring does not show undesirable or soft material.

The unsuitable material which might not have been encountered by this boring will be replaced with appropriate fill. The replacement material will be placed and compacted to obtain a density equal to the adjacent undisturbed foundation. A dewatering system will be required to maintain the excavation area in dry condition. Foundation design details of the proposed structures are given in Appendix D.

Borings have not been completed due to site change location of pump plant. Therefore, two additional borings will be taken prior to final design to determine engineering properties of the soils underlying the pump station foundation.

GROUNDWATER

Water level observations were monitored during the boring operations and are noted on the boring log as shown on plate 7 of the main report. Based on these observations the ground water levels encountered in the vicinity of the proposed embankment area approximately from station 4+00 to 21+50 were found to be fairly consistant from hole to hole. The depth at which water was located ranged from 2 to 3 feet; from elevations 544.5 to 545.5 feet MSL. From approximately stations 21+50CE to 31+00CE (end of the levee) the ground water was found to be .2 to 1 foot above the ground surface; from elevations 545 to 546 feet MSL. The water levels should be expected to fluctuate with changes in climatic conditions and river levels.

SUBSURFACE CONDITIONS AT EXCAVATION SITES

REFUGE DREDGING/DITCHING. Refuge excavation/ditching is proposed as shown on plate 18 of the main report. The site indicates removal of 2 to 4 feet of soils, which will have a width of 50 feet. Borings A-87-4, A-88-2, and A-88-13 were taken to identify the subsurface conditions and the engineering characteristics of the encountered material at the proposed site.

Borings revealed the presence of about 4 feet of very soft clay (CH-OH, CL, CH). The moisture content varied from 43% to 71%. This is underlain by medium clay (CL-CH). The moisture content varied from 32% to 37%. A detailed description of the encountered material is shown in the logs of soil borings on plate 7 of the main report.

DEAD SLOUGH EXCAVATION. Dead slough site will be located approximately 40 feet from riverside toe of the proposed levee as shown on plate 18 of the main report. The site indicates the need for removal of 6 to 9 feet of soils, which will have a width of 60 feet. Borings A-88-10 and A-88-11 were considered pertinent taken to determine the various soil profile components and the engineering characteristics of the material for dredging the dead slough. A typical soil profile of this site consists of very soft clay. The average water content was 98 percent, with a range of 98 to 107 percent. A detailed description of the encountered materials is shown on the logs of soil borings on plate 7 of the main report.

SLOPE STABILITY

The proposed levee near station 28+50 CE is found to be most critical for slope stability analysis for end of construction condition. The stability of slopes was analyzed by the Modified Swedish Method for a circular Arc Slope Stability Analysis in accordance with EM 1110-2-1902, "Engineering Design Stability of Earth and Rockfill Dams," dated 1 April 1970.

A sudden drawdown and steady seepage conditions were not evaluated since high water levels will be of such short duration that saturation of uncompacted impervious embankment cannot occur and the Mississippi River low water level will not impose any seepage pressure on the levee.

A range of extremely conservative shear strengths (Q) was assumed for the most severe configuration of embankment and foundation, to estimate the stability of the embankment. These valves are shown on plate C-1 and are based on tests and samples from other projects with generally similar soils and construction. Successive trails of various sliding surfaces were analyzed and determination of the critical failure arc having the lowest safety factor was made. The summary of the slope stability analysis and the solution of the most critical arc appears on plate C-1. The computed minimum, safety factor of 2.3 for end of construction condition exceed the 1.3 required by EM 1110-2-1913, "Design and Construction of Levees," dated March 31, 1978. Therefore, no slope stability problems are expected.

UNDERSEEPAGE

The underseepage analyses for the proposed is based on a thorough study of thickness and permeability, engineering characteristics of the impervious stratum and the pervious substratum, in addition to the extent of the riverward and landward top strata.

Case 2 (EM 1110-2-1913)-Impervious Top Stratum Both Riverside and Landside was considered appropriate since 15- to 20-foot thick top stratum appears to exist on the both sides of the approximately 6 feet high levee, and continuing infinitely on the landward side. For such a condition seepage will not occur through the landside top stratum; therefore, underseepage and berm analyses were not made.

SETTLEMENT

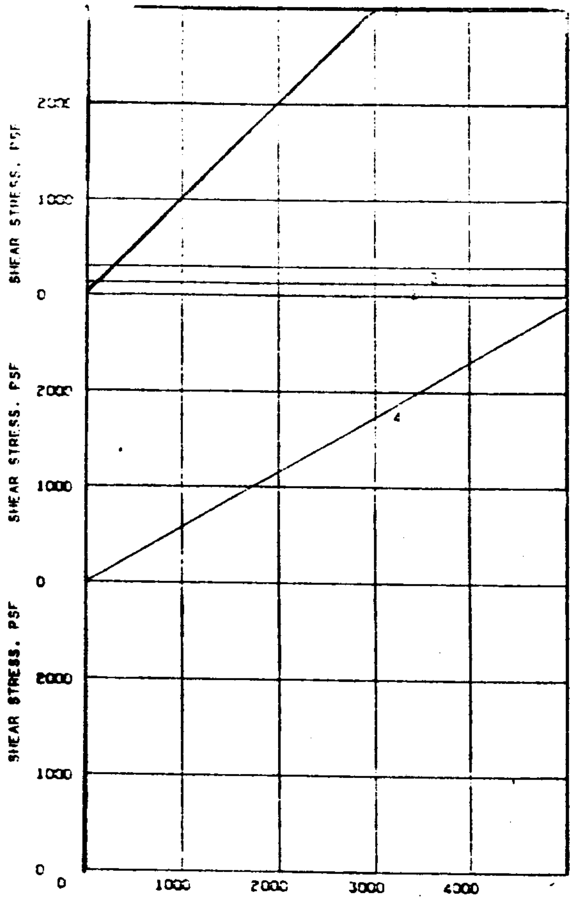
The proposed levee, from approximately station 23+00CE to 30+18CE is found to be most critical with respect to settlement. A study at station 28+50 was selected for analysis, where the levee is approximately 7 feet high.

The foundation, in this reach of the levee consists of a 1-to 2-foot thick layer of very soft organic clay. The soil below the soft clay consists of clay of a higher shear strength and a low compressibility. It is anticipated that the very soft clay will be displaced during the construction of the levee by stronger fill material. Therefore, a 9-foot high levee is considered appropriate for the settlement analysis. The 9-foot high levee will impose a maximum load of 810 pounds per square foot on the 15-foot-thick alluvial clay top stratum foundation. A settlement analysis conforming to Joseph E. Bowles "Foundation Analysis and Design," 3rd edition, 1982, indicates total settlement to be on the order of 14 inches, as shown on plate C-3. In order to anticipate the unexpected settlement, a shrinkage allowance of 24 percent of the levee height will be provided in the specifications to allow for any consolidation of the embankment and settlement in the foundation.

BORROW MATERIAL

The borrow material will be removed from areas as shown on plate 18 of the main report. The source of the borrow site location was determined to be as close as adjacent to the levee toe. A 40-foot width berm will be left in place between the toe of the levee and near the edge of borrow site to ensure levee stability and to facilitate construction. According to borings which are pertinent to borrow areas, the borrow material consist of very wet soft clay, exceeding the liquid limit. The moisture content varies from 43 to 107 percent. Atterberg limits testing reveals a range from 71/28(liquid limit/plastic limit) to 82/29. These borrow areas are economically feasible source of material to construct the uncompacted levee. Because it involves a short or no haul distance and is conducive to dragline operation. Due to the soft nature of the borrow soils, care will be required in excavation and placement to insure the soils will stay in place on the slopes.

A dragline will be used to excavate and place the material. Excessive displacement of the excavated material should be expected due to very soft material of low strength and standing water. The material placement will require gentle laying of the excavated material from bottom to top to minimize the disturbance. The excavated material will be left inplace for a period until it regains strength. The excavated material will not be stockpiled higher than the height of the proposed embankment or the embankment will be constructed in multiple stages.

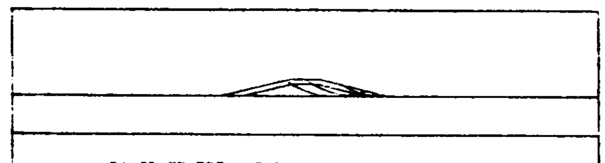


NORMAL STRESS, PSF
DESIGN ENVELOPES

MATERIAL		SOIL WT. LBS/CUFT		SHEAR STRENGTH Q		R		S		(R+S)/2	
		MOIST	SAT	PHI DEGREES	COHESION PSF	PHI DEGREES	COHESION PSF	PHI DEGREES	COHESION PSF	PHI DEGREES	COHESION PSF
RIPRAP AND BEDDING ELEV 544 TO 551	1	100.00	100.00	45.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
CLAY EMBANKMENT ELEV 544 TO 548	2	90.00	90.00	0.00	130.00	0.00	0.00	0.00	0.00	0.00	0.00
CLAY FOUNDATION ELEV 528 TO 544	3	115.00	115.00	0.00	300.00	0.00	0.00	0.00	0.00	0.00	0.00
SAND FOUNDATION ELEV TO 528	4	120.00	120.00	30.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

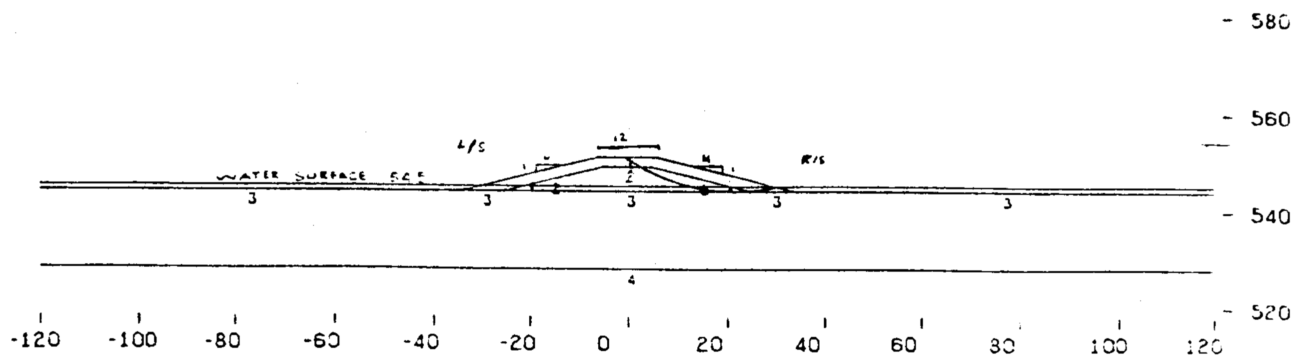
CIRCLE FAILURE SURFACE
RESULTS BY HARRIS-500
COMPUTER PROGRAM 741-H5-F424A
TANGENT TO ELEV 544.00
TRIAL ARCS

RADIUS OF CIRCLE	CENTER OF CIRCLE DISTANCE FROM ℄	ELEV	F.S. Q-SHEAR STR. ψ= 0.00	
39.00	22.00	583.00	2.30	
34.00	12.00	578.00	2.87	
34.00	32.00	578.00	3.84	
24.00	22.00	568.00	2.44	
44.00	22.00	588.00	2.31	
29.00	22.00	573.00	2.34	



RANGE OF TRIAL FAILURE SURFACES

NOTES
1- ANALYSES WERE RUN ACCORDING TO EM 1110-2-1902 DATED APRIL 1970
2- THE SIDE EARTH FORCE DIRECTION WAS TAKEN AS THE AVERAGE OF THE EMBANKMENT SLOPES IMMEDIATELY ADJACENT TO THE SLICE INTERFACE
3- PSI-SEISMIC COEFFICIENT USED IN ANALYSES



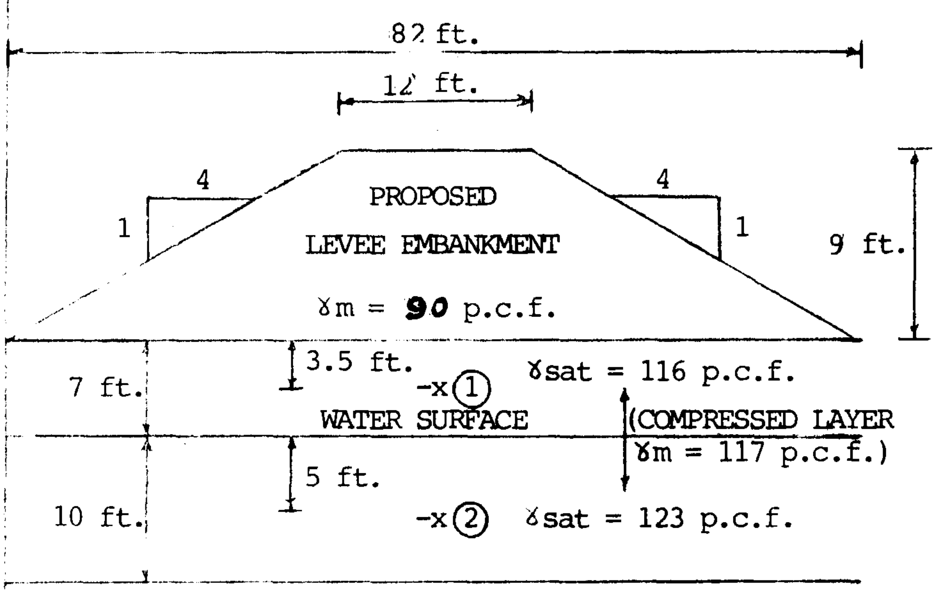
ANDALUSIA EMP
SLOPE STABILITY ANALYSIS
END OF CONSTRUCTION
STA. 28+00
JULY 1980 (SAZ)

PLATE C-1

Subject		Date
ANDALUSIA EMP ANDALUSIA, ILLINOIS		JUNE 88
Computed by ZS	Checked by GC	Sheet 1 of 2

ANDALUSIA EMP
ANDALUSIA, ILLINOIS

SETTLEMENT ANALYSIS



Layer 1

Moisture Content = 36%

Ave. LL/(WL) = 71%

Specific Gravity (Gs) = 2.7

e_o = WGs = .36(2.7) = .972

Layer 2

Ave. Moisture Content (Wn) = 28%

Ave. LL/(WL) = 44%

Specific Gravity (Gs) = 2.7

e_o = WGs = (.28)(2.7) = 0.76

P = Stress exerted by the 7 ft. high embankment

P_o = Initial stress at the center of the compressible layer

ΔP_o= Change in stress in compressible layer due to applied loading

C_c = Compression Index

e_o = Initial void Ratio

H = Thickness of compressible layer over which settlement is being calculated

C_c = .30 (e_o-.27): Ref. Physical & Geotechnicial Properties of Soils by Joseph E. Bowles.

C_c = .210 (layer 1), C_c = .147 (layer 2)

C_c = .009 (LL-10): Ref. Soil Mechanics in Engineering Practice by Terzaghid & Peck

C_c = .549 (layer 1), C_c = .306 (layer 2)

Use C_c = .37(e_o + .003WL + .0004Wn - .34), C_c = .318 (layer 1), C_c = .208 (layer 2) Ref. fnd. Analysis & design by Joseph E. Bowles. It is recommended by Joseph E. Bowles which has a reported 86% reliability.

P_o:

@ mid depth of layer No. 1 = 3.5(116-62.4) = 188 p.s.f.

@ mid depth of layer No. 2 = 7(116-62.4) + 5(123 - 62.4) = 678 p.s.f.

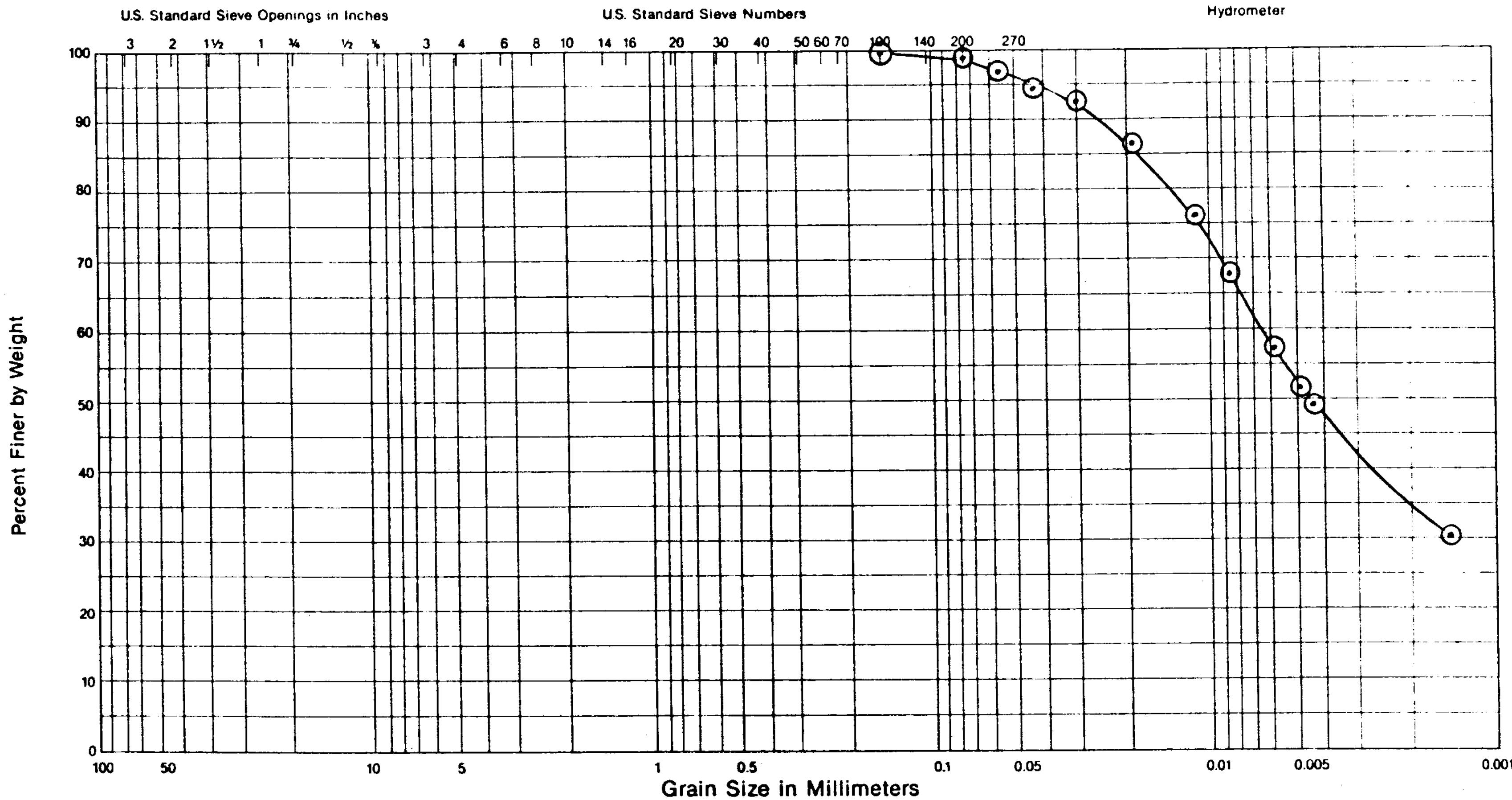
ΔP_o @ layer 1 Boussinesq coefficient * h * m.
(.982)*(9)*(90) = 795 p.s.f.

@ layer 2 Boussinesq coefficient * h * m.
(.861)*(9)*(90) = 697 p.s.f.

$$S = \frac{C_c}{1 + e_o} H \log_{10} \frac{P_o + P_o}{P_o}$$

Depth (ft.)		C_c	P_o (p.s.f.)	ΔP_o (p.s.f.)	H (ft.)	S (ft.)
0						
	3.5					
7		.318	188	795	7	.81
	10					
17		.208	678	697	10	.36

TOTAL SETTLEMENT = 1.2 ft. 14 inches



GRAVEL		SAND			SILT or CLAY
Coarse	Fine	Coarse	Medium	Fine	

GRAIN SIZE DISTRIBUTION CURVE

Boring No.	Sample No.	Depth or Elev.	Description	Unified Symbol	Natural WC	LL	PL	PI
A-4	1	2.5' to 3.5'						

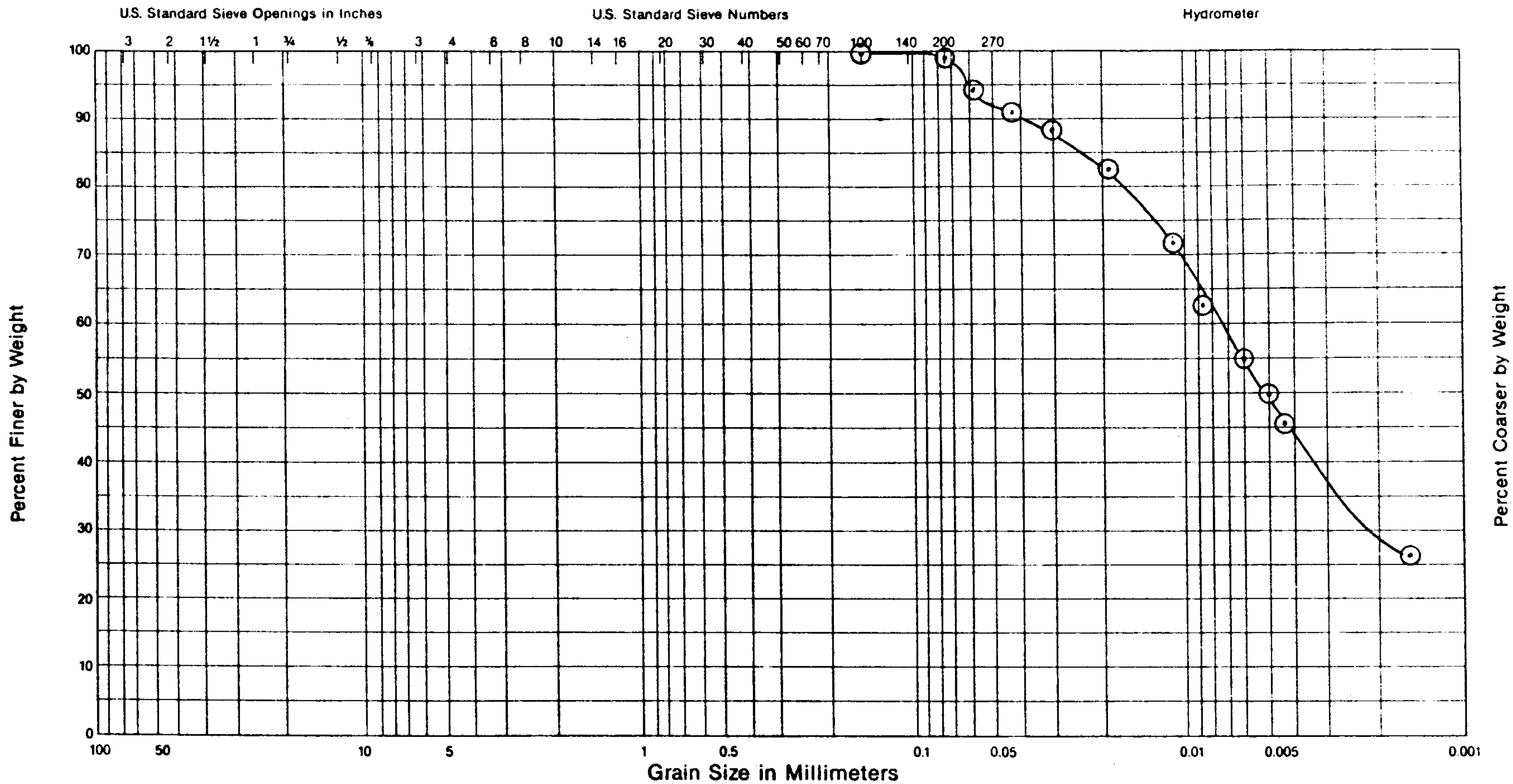
Project Andalusia Refuge Emp

Call #7002 (18Mar87) Job No. 07861088D Date 4-10-87

Terracon

PLATE C-4

Form 124



GRAVEL		SAND			SILT or CLAY
Coarse	Fine	Coarse	Medium	Fine	

GRAIN SIZE DISTRIBUTION CURVE

Boring No.	Sample No.	Depth or Elev.	Description	Unified Symbol	Natural WC	LL	PL	PI
A-5	1	3'		CH		61	27	34

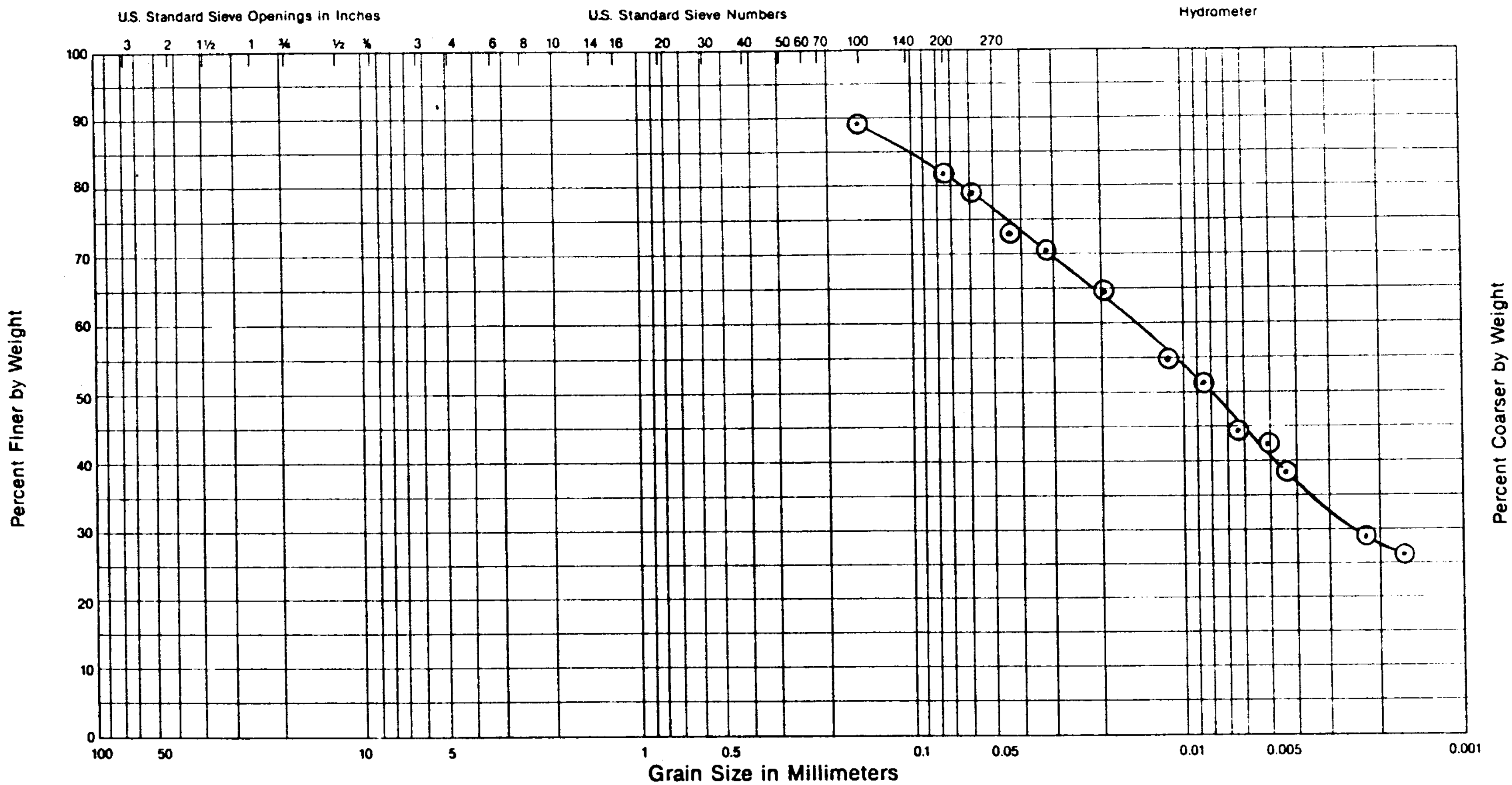
Project Andalusia Refuge Emp

Call #7002 (18Mar87) Job No. 07861088D Date 4-10-87

Terracon

Form 124

PLATE C-5



GRAVEL		SAND			SILT or CLAY
Coarse	Fine	Coarse	Medium	Fine	

GRAIN SIZE DISTRIBUTION CURVE

Boring No.	Sample No.	Depth or Elev.	Description	Unified Symbol	Natural WC	LL	PL	PI
A-5	3	7'	Specific Gravity = 2.654	CL		46	16	30

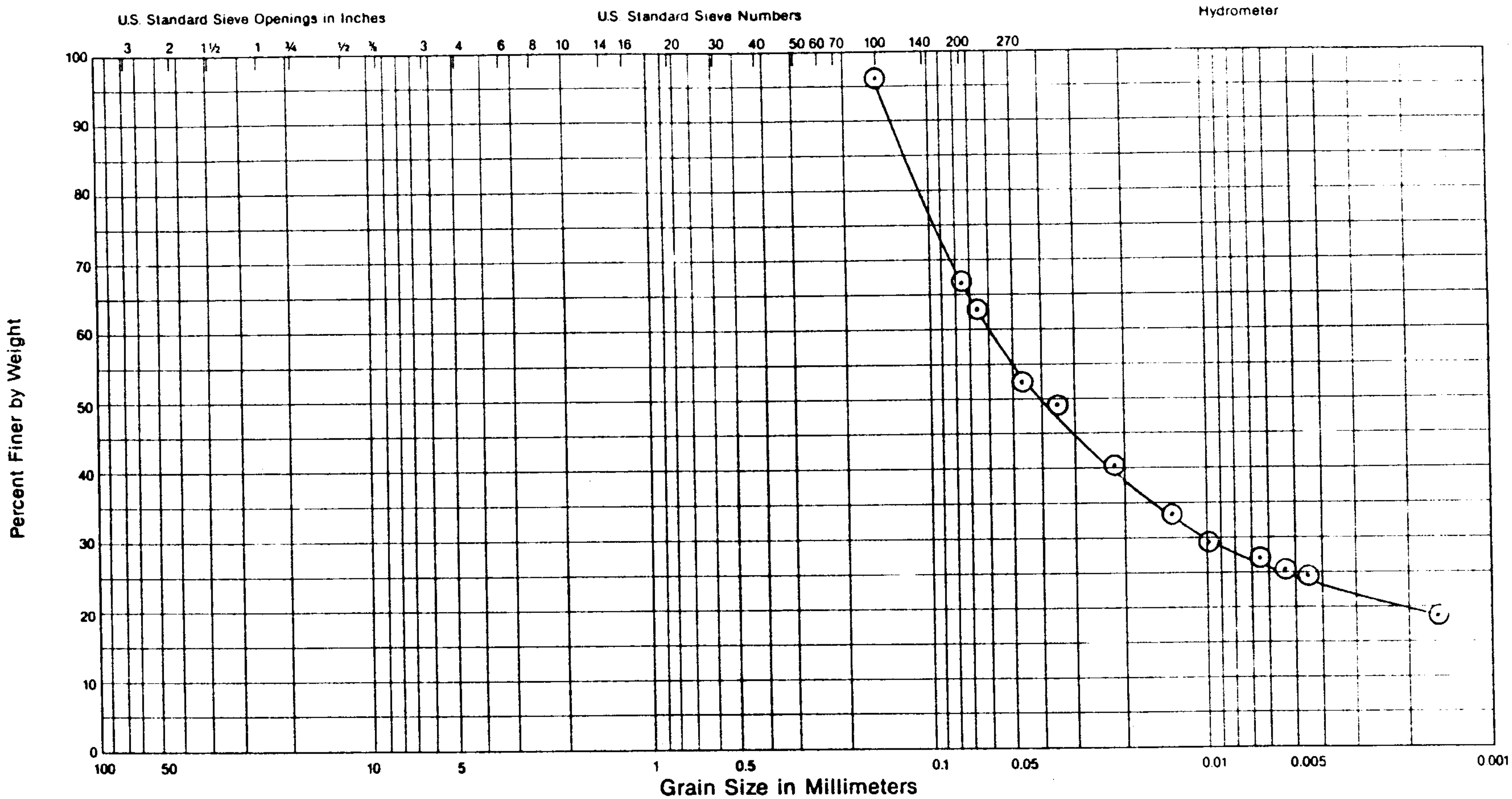
Project Andalusia Refuge Emp

Call #7002 (18Mar87) Job No. 07861088D Date 4-13-87

Terracon

PLATE C-6

Form 124



GRAVEL		SAND			SILT or CLAY
Coarse	Fine	Coarse	Medium	Fine	

GRAIN SIZE DISTRIBUTION CURVE

Boring No.	Sample No.	Depth or Elev.	Description	Unified Symbol	Natural WC	LL	PL	PI
A-2	2	4'		CL		34	17	17

Project Andalusia Refuge Emp

Call #7002 (18Mar87) Job No. 07861088D Date 4-10-87

Terracon

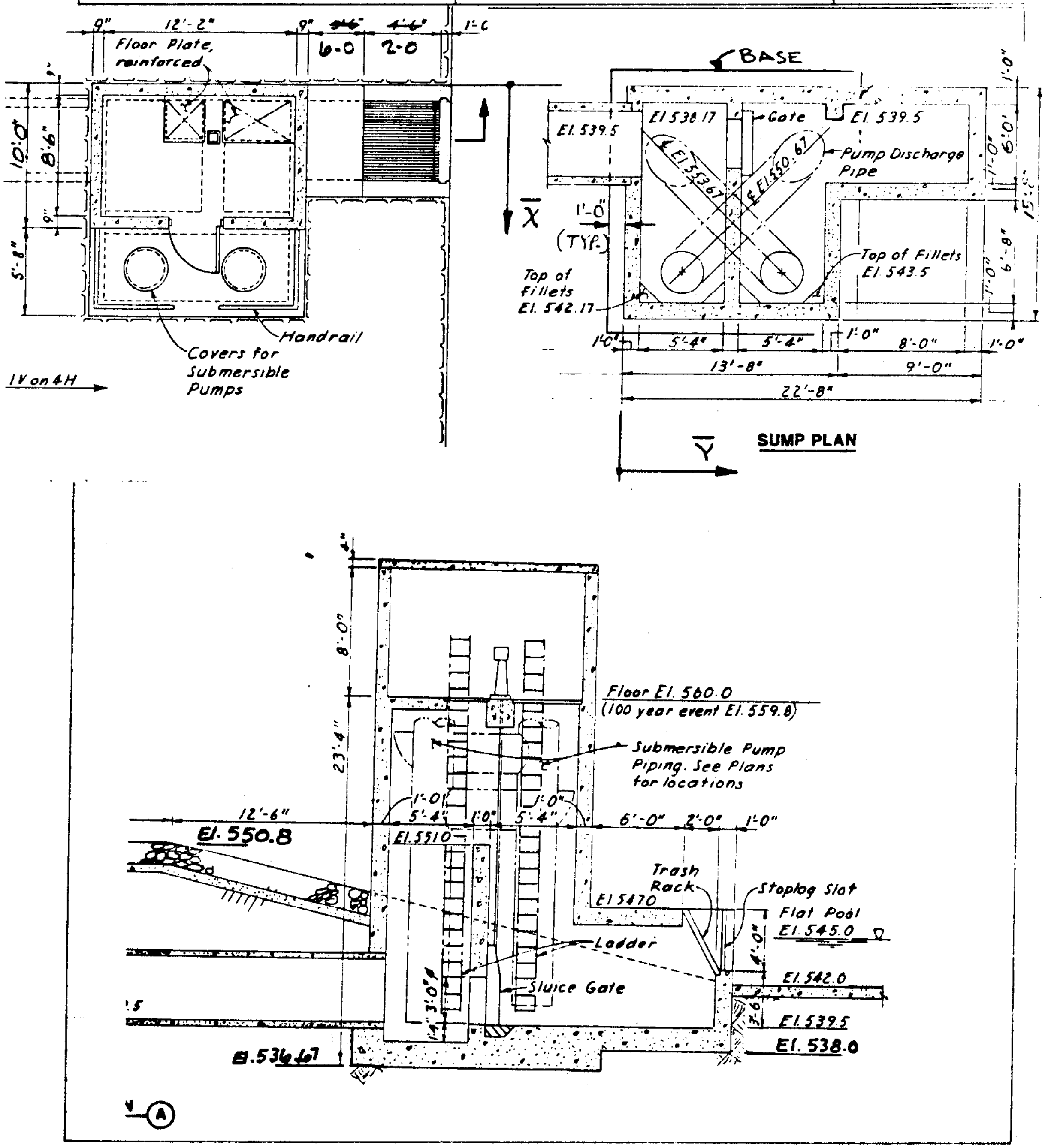
PLATE C-7

Form 124

STRUCTURAL DESIGN

APPENDIX D

Subject ANDALUSIA SLOUGH - PUMP STATION		Date 18 MAY 8?
Computed by KEW	Checked by chJ	Sheet PS-A



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1 Aug 80

Subject ANDALUSIA SLOUGH - PUMP STATION	Date 15 Feb 88
Computed by KEW / Checked by chJ	Sheet PS 1 of

FORCES ON PUMP STATION

REFERENCES

1. TM 5-809-1/AFM 88-3, CHAP-1, "LOAD ASSUMPTIONS FOR BUILDINGS", 28 MAR. 8?
2. "STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES" AASHTO, 1983
3. EM 1110-2-2607, "NAVIGATION DAM MASONRY" 1 JULY '58

WIND FORCES REF. (1), CHAPTER 5

$V = 80$ mph

ASSUME EXPOSURE C

$q_z = 0.00256\, K_z (IV)^2$ WHERE K_z = VELOCITY PRESSURE COEF. (Table 5-?)

HEIGTH OF PUMP STATION IS
$\simeq 20^{FT}$ ABOVE THE LEVEE
$\simeq 9^{FT}$ ABOVE 100 YR. FLOOD.

I = IMPORTANCE FACTOR (Table 5-?)
$= 1.00$

z	K_z	q_z
0-15	0.80	13.11 psf
20	0.87	14.25 psf

$p = q\, G_h\, C_p$ WHERE G_h = GUST RESPONSE FACTOR (Table 5-?)

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Subject ANDALUSIA SLOUGH - PUMP STATION		Date 15 [illegible]
Computed by KEW	Checked by chJ	Sheet of PS2

FORCES ON PUMP STATION

Z	G_h
0-15	1.32
20	1.20

C_p = EXTERNAL PRESSURE COEFF. (Figure [illegible])

Z	WINDWARD WALL $C_p = 0.8$	LEEWARD WALL $C_p = -0.5$
0-15	$p_{WALL} = 13.85$ psf	$p_{WALL} = -8.[illegible]$
20	$p_{WALL} = 14.71$ psf	$p_{WALL} = -[illegible]$

p_{WALLS} =

Z	TOTAL
0-15	22.51 psf
20	2[illegible].9[illegible] psf

$p_{ROOF} = 14.25\ (1.20)\ (-0.7) = 12.87$ psf

RIVER FLOW FORCES REF. (2), SEC. 3.18.1

$p = KV^2$

$= 1.375\ (7.0)^2$

$= 68$ psf

V = FLOW OF RIVER
= 7 ft/sec. (FOLE [illegible] ED-HW)

K = CONSTANT = 1.375 FOR FLAT SURFACE

Subject ANDALUSIA SLOUGH - PUMP STATION	Date 27 MAY 8
Computed by KEW	Checked by chJ	Sheet PS 3 of

WEIGHT OF PUMP TUBE

ASSUME 1/4" PLATE FOR TUBES, WT. = 10.2 #/FT²

TUBE I.D. = 32" ; PERIMETER = π(D) = π($\frac{32.2}{12}$)

= 8.443 FT²

VERT. TUBE LENGTH

INTAKE PUMP = 562.0 - 539.5 - 1.33 = 21.17 FT

OUTLET PUMP = 562.0 - 539.5 = 22.50 FT

VERT. TUBE WEIGHT

INTAKE PUMP = 10.2(8.443)(21.17) = 1,823 #

OUTLET PUMP = 10.2(8.443)(22.50) = 1,938 #

HORIZ. TUBE LENGTH

INTAKE PUMP = 10.2(8.443)(11.25) = 969 #

OUTLET PUMP = " " " = 969 #

WEIGHT OF PUMPS = 2,870 # EACH (MANUF. DATA.)

WEIGHT OF H_2O IN VERT. TUBES

INTAKE PUMP = $\pi\frac{(16)^2}{144}(21.17-1.04)(62.4)$ = 7,016 #

OUTLET PUMP = $\pi\frac{(16)^2}{144}(22.50-1.04)(62.4)$ = 7,479 #

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Subject ANDALUSIA SLOUGH – PUMP STATION		Date 27 MAY 88
Computed by KEW	Checked by chJ	Sheet PS4 of

WEIGHT OF H_2O IN HORIZ. TUBES

INTAKE PUMP $= \pi \frac{(16)^2}{144} (11.25)(62.4) =$ 3,921 #

OUTLET PUMP = 3,921 #

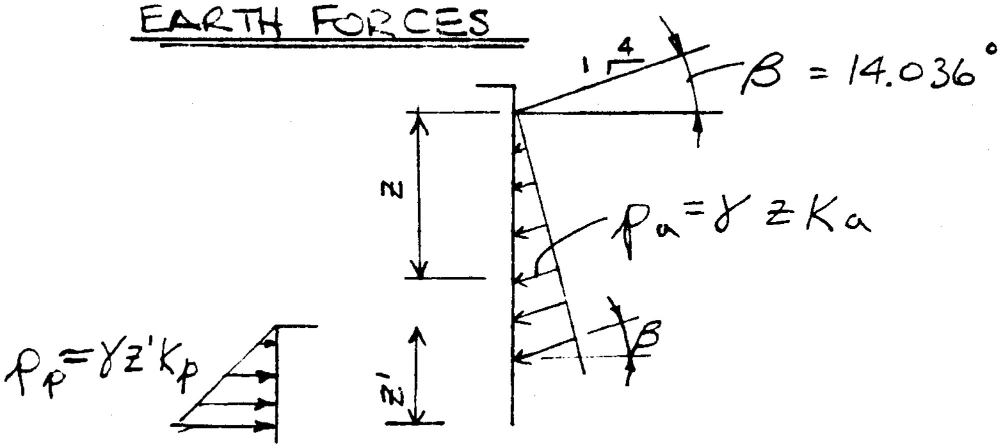
Subject ANDALUSIA SLOUGH - PUMP STATION		Date 24 JUNE 88
Computed by KEVI	Checked by chJ	Sheet PS5 of

FORCES ON PUMP STATION

BACKFILL MAT'L

$\gamma_{DAMP} = 115$ pcf
$c = 0$
$\phi = 32°$

EARTH FORCES



$$K_a = \cos\beta \left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}} \right]$$

$$K_a = 0.97014 \left[\frac{0.97014 - \sqrt{(0.97014)^2 - (0.84805)^2}}{0.97014 + \sqrt{(0.97014)^2 - (0.84805)^2}} \right]$$

$$= 0.97014 \frac{(0.97014 - 0.47115)}{(0.97014 + 0.47115)} = 0.33587$$

$$K_{a_H} = 0.97014\,(0.33587) = 0.32584$$

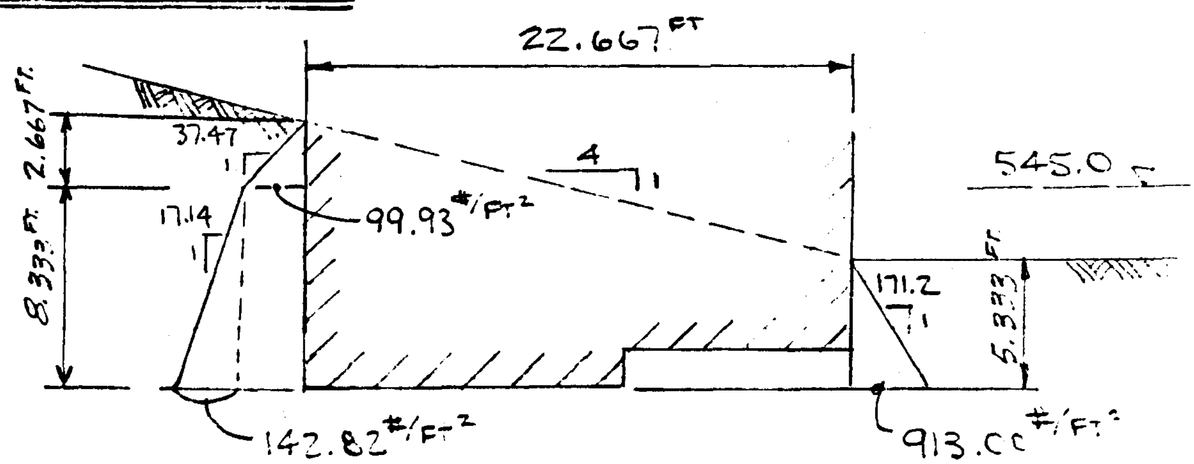
$$K_p = \tan^2\left(45 + \frac{\phi}{2}\right)$$

$$= \tan^2\left(45 + \frac{32}{2}\right) = 3.25459$$

Subject		Date
ANDALUSIA SLOUGH - PUMP STATION		?? JUNE 88
Computed by KEW	Checked by ckJ	Sheet PS6 of

FORCES ON PUMP STATION

EARTH FORCES



ICE FORCES

BECAUSE THE REFUGE IS ABOVE THE PUMP STATION AND FLAT POOL AND HIGH WATER WITHIN THE REFUGE IS BELOW THE TOP OF THE LEVEE, ICE FORCES WILL NOT ACT ON THE PUMP STATION.

FLOATING OBJECTS

THE PUMP STATION IS IN A PROTECTED AREA, OUT OF THE MAIN FLOW OF THE RIVER. IT WILL NOT BE SUBJECT TO LARGE FLOATING OBJECTS.

Subject ANDALUSIA SLOUGH - PUMP STATION		Date 18 MAY 88
Computed by KEW	Checked by chJ	Sheet PS7 of

REVISED 15 OCT. 88

STABILITY ($\bar{Y}$)

UNIT	FORCE	ARM	MOMENT
ROOF 0.33(150)(10.00)(13.67)	6,835	6.833	46,704
WALLS 0.75(150)(8.00)(13.67)(2)	24,606	6.833	168,133
− 0.75(150)(7.17)(3.33)	− 2,686	6.833	− 18,353
0.75(150)(8.00)(8.50)	7,650	0.375	2,869
0.75(150)(8.00)(8.50)	7,650	13.292	101,684
FLOOR 0.50(150)(13.67)(11.67)	11,965	6.833	81,755
− 0.50(150)(2.50)(2.50)	− 469	5.917	− 2,775
− 0.50(150)π(1.333)^2	− 419	3.667	− 1,536
− 0.50(150)π(1.333)^2	− 419	10.000	− 4,190
− 0.50(150)(2.5)(4.17)	− 782	10.583	− 8,276
BMS 0.50(150)(8.00)(0.83)	498	7.833	3,901
(BELOW SLAB) 1.33(150)(11.67)(1.25)	2,910	6.833	19,884
WALLS 1.00(150)(20.50)(15.67)	48,185	13.167	634,451
− 1.00(150)(7.00)(5.00)	− 5,250	13.167	− 69,127
1.00(150)(20.50)(15.67)	48,185	0.500	24,093
− 1.00(150)π(2.00)^2	− 1,885	0.500	− 942
1.00(150)(20.50)(11.67)(2)	71,770	6.833	490,404
1.00(150)(11.50)(13.67)	23,581	6.833	161,129
− 1.00(150)(3.00)(3.00)	− 1,350	6.833	− 9,225
BASE 2.83(150)(17.67)(15.67)	117,539	6.833	803,144
− 1.33(150)(13.67)(5.33)	− 14,536	3.667	− 53,304
	343,578#		2,370,423 FT-#

$\bar{Y}$ = 6.899 FT.

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Subject ANDALUSIA SLOUGH - PUMP STATION		Date 19 MAY 88
Computed by KEW	Checked by chJ	Sheet PS 8 of

REVISED 12 OCT. 8?

STABILITY ($\bar{Y}$)

UNIT	FORCE	ARM	MOMENT
WIND 24(10.00)(8.33)	2,000	27.495	54,990
24(15.67)(9.20)	3,460	18.733	64,816
RIVER FLOW 68(15.67)(9.20)	9,803	18.733	183,640
EARTH LOADS (NORMAL)(SHT. PS6)			
ACTIVE			
$37.47(2.667)\left(\frac{2.667}{2}\right)(15.67)$	2,088	9.222	19,256
$37.47(2.667)(8.333)(15.67)$	13,049	4.167	54,375
$17.14(8.333)\left(\frac{8.333}{2}\right)(15.67)$	9,325	2.777	25,896
PASSIVE			
$171.2(5.333)\left(\frac{5.333}{2}\right)(15.67)$	-38,172*	—	
*PASSIVE FORCE IS GREATER THAN ACTIVE FORCES INCLUDING WIND AND EARTH, ∴ USE Σ ACTIVE	-29,922	1.777	-53,171
EARTH LOADS (100YR.)			
ACTIVE			
$17.14(11.00)\left(\frac{11.00}{2}\right)(15.67)$	16,249	3.667	59,585
PASSIVE (SEE ABOVE)	-38,172**		
**PASSIVE FORCE IS GREATER THAN ACTIVE FORCES INCLUDING WIND, RIVER FLOW & EARTH, ∴ USE Σ ACTIVE	-28,052	1.777	-49,848

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Subject: ANDALUSIA SLOUGH - PUMP STATION		Date: 19 MAY 88
Computed by: KEW	Checked by: chJ	Sheet PS9 of

REVISED 18 OCT. 8

STABILITY ($\bar{Y}$)

UNIT	FORCE	ARM	MOMENT
PUMPS & PIPING			
2,870 + 1,938 (1.10)	5,002	3.667	18,342
2,870 + 1,823 (1.10)	4,875	10.000	48,750
969 (1.10)	1,066	5.500	5,863
969 (1.10)	1,066	8.167	8,706
LIVE LOAD 100(8.5)(12.167)	10,342	6.833	70,667
SNOW LOAD 20 (15.667)(13.667)	4,282	6.833	29,259
INLET 0.50 (150)(6.00)(6.00)	2,700	16.667	45,001
1.00 (150)(7.50)(9.00)2	20,250	18.167	367,882
1.00 (150)(6.00)(3.50)	3,150	22.167	69,826
1.50 (150)(8.00)(8.00)	14,400	18.667	268,805
	40,500#	18.556	751,514 FT-#

UNIT	FORCE	ARM	MOMENT
WATER (NORMAL)			
(542.00 - 538.17)(62.4)(5.33)(13.67)	17,413	3.667	63,853
(545.00 - 539.50)(62.4)(5.33)(13.67)	25,006	10.000	250,060
(545.00 - 539.50)(62.4)(6.00)(9.00)	18,533	17.167	318,156
(545.00 - 543.00)(62.4)(6.00)(1.00)	749	22.167	16,603
(545.00 - 539.50)(62.4)(15.67)(1.00)2	10,756	6.833	73,496
(545.00 - 539.50)(62.4)(7.67)(1.00)	2,632	14.167	37,288
(545.00 - 539.5)(62.4)(15.67)(1.00)	5,378	-0.500	- 2,689
$-\pi(2.333)^2$ (62.4)(1.00)	- 1,067	-0.500	534
	79,400#		757,301

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Subject: ANDALUSIA SLOUGH - PUMP STATION		Date: 19 MAY 8?
Computed by: KEW	Checked by: chJ	Sheet: PS10 of

REVISED DEC ...

STABILITY ($\bar{Y}$)

UNIT	FORCE	ARM	MOMENT
EARTH ON BASE PROJ. (NORMAL)			
$(115-62.4)(5.5)(15.67)(1.00)2$	9,067	6.833	61,955
$-(115-62.4)(1.0)(\frac{4.000}{2})(1.00)2$	- 210	13.333	- 2,800
$115\,(547.917-545.0)(\frac{11.67}{2})(1.00)2$	3,915	2.889	11,310
$(115-62.4)(4.625)(7.67)(1.00)$	1,866	14.167	26,436
$(115-62.4)(5.5)(15.67)(1.00)$	4,533	-0.500	- 2,267
$115\,(2.792)(15.67)(1.00)$	5,031	-0.500	- 2,516
$-(115-62.4)\pi(2.333)^2(1.00)$	- 899	-0.500	450
	23,303#	3.972	92,569
UPLIFT (NORMAL)			
$-(545.0-536.67)(62.4)(17.67)(15.67)$	-143,925	6.833	- 983,440
$-(545.0-538.00)(62.4)(8.00)(8.00)$	- 27,955	18.667	- 521,840
	- 171,880#	8.758	- 1,505,280
WATER (100 YR.)			
$(559.30-538.17)(62.4)(5.33)(13.67)$	96,068	3.667	352,281
$(559.30-539.50)(62.4)(5.33)(13.67)$	90,021	10.000	900,210
$(559.30-551.00)(62.4)(1.00)(13.67)$	7,080	6.833	48,378
$(559.80-539.50)(62.4)(6.00)(8.00)$	60,803	17.667	1,074,207
$(546.50-539.50)(62.4)(1.00)(5.00)$	2,184	13.167	28,757
$-0.50(62.4)(6.00)(6.00)$	- 1,123	16.667	- 18,717
$(559.80-543.00)(62.4)(1.00)(6.00)$	6,290	22.167	139,430
$(559.80-547.00)(62.4)(1.00)(9.00)2$	14,377	18.167	261,187
$(559.80-539.50)(62.4)(15.67)(1.00)2$	39,699	6.833	271,263

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Subject: LNCLNSIA SLOVGH - PUMP STATION	Date: 18 OCT 88
Computed by: KEW / Checked by: chJ	Sheet: PS10a

STABILITY (Y)

UNIT	FORCE	ARM	MOMENT
WATER (100 YR) (CONT.)			
(559.80 - 539.5)(62.4)(7.67)(1.00)	9,716	14.167	137,647
(559.80 - 539.5)(62.4)(15.67)(1.00)	19,850	-0.500	- 9,925
$-\pi(2.333)^2(62.4)(1.00)$	- 1,067	-0.500	534
	343,898 #		3,185,252 Ft-#
EARTH ON BASE PROJ. (100 YR)			
52.6 (5.5)(15.67)(1.00) 2	9,067	6.833	61,955
$-52.6\ (1.0)\left(\frac{4.00}{2}\right)(1.00)\ 2$	- 210	13.333	- 2,800
$52.6\ (2.917)\left(\frac{11.67}{2}\right)(1.00)\ 2$	1,791	2.889	5,174
52.6 (4.625)(7.67)(1.00)	1,866	14.167	26,436
52.6 (8.292)(15.67)(1.00)	6,835	-0.500	- 3,417
$-52.6\ \pi\ (2.333)^2(1.00)$	- 899	-0.500	450
	18,450 #		87,798 Ft-#
UP LIFT (100 YR.)			
-(559.8 - 536.67)(62.4)(17.67)(15.67)	-399,637	6.833	-2,730,720
-(559.8 - 538.00)(62.4)(8.00)(8.00)	- 87,060	18.667	- 1,625,158
	-486,697 #		-4,355,878 Ft-#

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Subject ANDALUSIA SLOUGH – PUMP STATION		Date 18 MAY 88
Computed by KEW	Checked by cKJ	Sheet PS 11 of

REVISED 18 Oct. 88

STABILITY ($\bar{x}$)

UNIT	FORCE	ARM	MOMENT
ROOF 0.33 (150)(10.00)(13.67)	6,835	5.000	34,175
WALLS 0.75 (150)(8.00)(13.67)	12,303	0.375	4,614
0.75 (150)(8.00)(13.67)	12,303	9.625	118,416
−0.75 (150)(7.17)(3.33)	−2,686	9.625	−25,853
0.75 (150)(8.00)(8.50)(2)	15,300	5.000	76,500
FLOOR 0.50 (150)(13.67)(11.67)	11,965	7.833	93,722
− 0.50 (150)(2.50)(2.50)	− 469	2.250	− 1,055
− 0.50 (150) π $(1.333)^2$ (2)	− 838	12.667	− 10,615
− 0.50 (150)(2.5)(4.17)	− 782	2.250	− 1,760
BMS. 0.50 (150)(8.00)(0.83)	498	5.000	2,490
(BELOW SLAB) 1.33 (150)(11.67)(1.25)	2,910	9.625	28,009
WALLS 1.00 (150)(20.50)(15.67)(2)	96,370	7.833	754,866
− 1.00 (150)(7.00)(5.00)	− 5,250	4.500	− 23,625
− 1.00 (150) π $(2.00)^2$	− 1,885	4.500	− 8,483
1.00 (150)(20.50)(11.67)	35,885	0.500	17,943
1.00 (150)(20.50)(11.67)	35,885	15.167	544,268
1.00 (150)(11.50)(13.67)	23,581	7.833	184,710
− 1.00 (150)(3.00)(3.00)	− 1,350	4.500	− 6,075
BASE 2.83 (150)(17.67)(15.67)	117,539	7.833	920,683
− 1.33 (150)(13.67)(5.33)	− 14,536	7.833	− 113,860
	343,578#		2,589,070

$\bar{X}$ = 7.536 FT.

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Subject: ANDALUSIA SLOUGH - PUMP STATION	Date: 19 MAY 88
Computed by: KEW / Checked by: chJ	Sheet PS12 of

REVISED 18 OCT. 88

STABILITY ($\bar{x}$)

UNIT	FORCE	ARM	MOMENT
WIND 24 (13.67)(20.66)	6,778	21.337	144,622
24 (13.67)(3.417) ÷ 2	561	9.867	5,535
24 (9.00)(2.75)	594	8.958	5,321
24 (9.00)(2.25) ÷ 2	243	6.833	1,660
	8,176#		157,138 FT-#
RIVER FLOW - NONE	—	—	—
EARTH - EQUAL ON OPP. SIDES	—	—	—
PUMPS & PIPING			
2,870 + 1938 (1.10)	5,002	12.667	63,360
2,870 + 1823 (1.10)	4,875	12.667	61,752
969 (1.10)(2)	2,132	8.667	18,478
LINE LOAD 100 (8.5)(12.167)	10,342	5.000	51,710
SNOW LOAD 20 (15.667)(13.667)	4,282	7.833	33,541
INLET 0.50 (150)(6.0)(6.0)	2,700	4.000	10,800
1.00 (150)(7.5)(9.00)	10,125	0.500	5,063
1.00 (150)(7.5)(9.00)	10,125	7.500	75,937
1.00 (150)(6.00)(3.50)	3,150	4.000	12,600
1.50 (150)(8.00)(8.00)	14,400	4.000	57,600
	40,500#	4.000	162,000 FT-#

* AVG. EXPOSED HT.

NCR Form 381b 1 Aug 80

Subject ANDALUSIA SLOUGH - PUMP STATION	Date 19 AUG 88
Computed by KEW / Checked by ch	Sheet

REVISED 18 OCT 88

STABILITY ($\bar{X}$)

UNIT	FORCE	ARM	MOMENT
WATER (NORMAL)			
(542.00 - 538.17)(62.4)(5.33)(13.67)	17,413	7.833	136,396
(545.00 - 539.50)(62.4)(5.33)(13.67)	25,006	7.833	195,872
(545.00 - 539.50)(62.4)(6.00)(9.00)	18,533	4.000	74,132
(545.00 - 543.00)(62.4)(6.00)(1.00)	749	4.000	2,996
(545.00 - 539.50)(62.4)(15.67)(1.00)	5,378	- 0.500	- 2,689
(545.00 - 539.50)(62.4)(15.67)(1.00)	5,378	16.167	86,946
(545.00 - 539.50)(62.4)(7.67)(1.00)	2,632	11.833	31,144
(545.00 - 539.50)(62.4)(15.67)(1.00)	5,378	7.833	42,126
- π (2.333)²(62.4)(1.00)	- 1,067	4.500	- 4,802
	79,400#		562,121 Ft-#
EARTH ON BASE PROJ. (NORMAL)			
(115 - 62.4)(5.5)(15.67)(1.00)2	9,067	7.833	71,022
- (115 - 62.4)(1.0)(4.00/2)(1.00)2	- 210	7.833	- 1,645
115 (547.917 - 545.0)(11.67/2)(1.00)2	3,915	7.833	30,666
(115 - 62.4)(4.625)(7.67)(1.00)	1,866	11.833	22,080
(115 - 62.4)(5.5)(15.67)(1.00)	4,533	7.833	35,507
115 (2.792)(15.67)(1.00)	5,031	7.833	39,408
- (115 - 62.4) π (2.333)² (1.00)	- 899	4.500	- 4,046
	23,303#		192,992 Ft-#
UPLIFT (NORMAL)			
- (545.0 - 536.67)(62.4)(17.67)(15.67)	- 143,925	7.833	- 1,127,365
- (545.0 - 539.00)(62.4)(8.00)(8.00)	- 27,955	4.00	- 111,820
	- 171,880#		- 1,239,185 Ft-#

NCR Form 381b
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Subject: ANDALUSIA SLOUGH - PUMP STATION		Date: 18 OCT. 88
Computed by: KEW	Checked by: chJ	Sheet: PS.3a of

STABILITY ($\bar{X}$)

UNIT	FORCE	ARM	MOMENT
WATER (100 YR.)			
(559.30 − 538.17)(62.4)(5.33)(13.67)	96,068	7.833	752,501
(559.30 − 539.50)(62.4)(5.33)(13.67)	90,021	7.833	705,134
(559.30 − 551.00)(62.4)(1.00)(13.67)	7,080	7.833	55,458
(559.80 − 539.50)(62.4)(6.00)(8.00)	60,803	4.000	243,212
(546.50 − 539.50)(62.4)(1.00)(5.00)	2,184	4.500	9,828
− 0.50 (62.4)(6.00)(6.00)	− 1,123	4.000	− 4,492
(559.80 − 543.00)(62.4)(1.00)(6.00)	6,290	4.000	25,160
(559.80 − 547.00)(62.4)(1.00)(9.00) 2	14,377	4.000	57,508
(559.80 − 539.50)(62.4)(15.67)(1.00) 2	39,699	7.833	310,962
(559.80 − 539.50)(62.4)(7.67)(1.00)	9,716	11.833	114,969
(559.80 − 539.50)(62.4)(15.67)(1.00)	19,850	7.833	1[illegible],4[illegible]
− π (2.333)² (62.4)(1.00)	− 1,067	4.500	− 4,802
	343,898#		2,420,923 FT-#

UNIT	FORCE	ARM	MOMENT
EARTH ON ~~BASE~~ PROJ. (100 YR.)			
52.6 (5.5)(15.67)(1.00) 2	9,067	7.833	71,022
− 52.6 (1.0) $\left(\frac{4.00}{2}\right)$ (1.00) 2	− 210	7.833	− 1,645
52.6 (2.917) $\left(\frac{11.67}{2}\right)$ (1.00) 2	1,791	7.833	14,029
52.6 (4.625)(7.67)(1.00)	1,866	11.833	22,080
52.6 (8.292)(15.67)(1.00)	6,835	7.833	53,539
− 52.6 π (2.333)² (1.00)	− 899	4.500	− 4,046
	18,450#		154,9[illegible] FT-#

UNIT	FORCE	ARM	MOMENT
UPLIFT (100 YR.)			
−(559.8 − 536.67)(62.4)(17.67)(15.67)	− 399,637	7.833	− 3,130,357
−(559.8 − 538.00)(62.4)(8.00)(8.00)	− 87,060	4.000	− 348,240
	− 486,697#		− 3,478,5[illegible] FT-#

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Subject ANDALUSIA SLOUGH - PUMP STATION		Date 20 JUNE 88
Computed by KEW	Checked by chJ	Sheet PS14 of

REVISED 18 OCT. 88

STABILITY ($\bar{Y}$)

LOAD CASE I NORMAL CONDITIONS

UNIT	FORCE	ARM	MOMENT
	#		FT-#
DEAD LOAD OF STRUCTURE ↓	343,578	6.899	2,370,423
WIND →	2,000	27.495	54,990
	3,460	18.733	64,816
EARTH - ACTIVE →	2,088	9.222	19,256
	13,049	4.167	54,375
	9,325	2.777	25,896
- PASSIVE ←	- 29,922	1.777	- 53,171
PUMPS & PIPING ↓	12,009	6.800	81,661
LIVE LOAD ↓	10,342	6.833	70,667
SNOW LOAD ↓	4,282	6.833	29,259
INLET ↓	40,500	18.556	751,514
WATER ↓	79,400	9.538	757,301
EARTH ON BASE PROJ. ↓	23,303	3.972	92,568
UPLIFT ↑	-171,880	8.758	-1,505,280
	341,534 #	8.240	2,814,275 FT-#

$$e_Y = 8.240 - \frac{13.667}{2} = 1.407^{FT} < \frac{13.667}{6} = 2.278^{FT}$$

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Subject ANDALUSIA SLOUGH - PUMP STATION		Date 20 JUNE 88
Computed by KEW	Checked by chJ	Sheet PS15 of

REVISED 19 OCT. 88

STABILITY ($\bar{X}$)

LOAD CASE I NORMAL CONDITIONS

UNIT		FORCE	ARM	MOMENT
DEAD LOAD OF STRUCTURE	↓	# 343,578	7.536	FT-# 2,589,070
WIND (NOT IN CONJ. W/ $\bar{Y}$ WIND)	→	—	—	—
PUMPS & PIPING	↓	12,009	11.957	143,590
LIVE LOAD	↓	10,342	5.000	51,710
SNOW LOAD	↓	4,282	7.833	33,541
INLET	↓	40,500	4.000	162,000
WATER	↓	79,400	7.080	562,121
EARTH ON BASE PROJ.	↓	23,303	8.282	192,992
UPLIFT	↑	-171,880	7.210	-1,239,185
		# 341,534	7.308	FT-# 2,495,839

$$e_x = 7.308 - \frac{15.667}{2} = -0.526^{FT} < \frac{15.667}{6} = 2.611^{FT}$$

Subject: ANDALUSIA SLOUGH – PUMP STATION		Date: 23 JUNE 88
Computed by: KEW	Checked by: chJ	Sheet PS16 of

REVISED 19 OCT. 88

STABILITY

LOAD CASE I NORMAL CONDITIONS

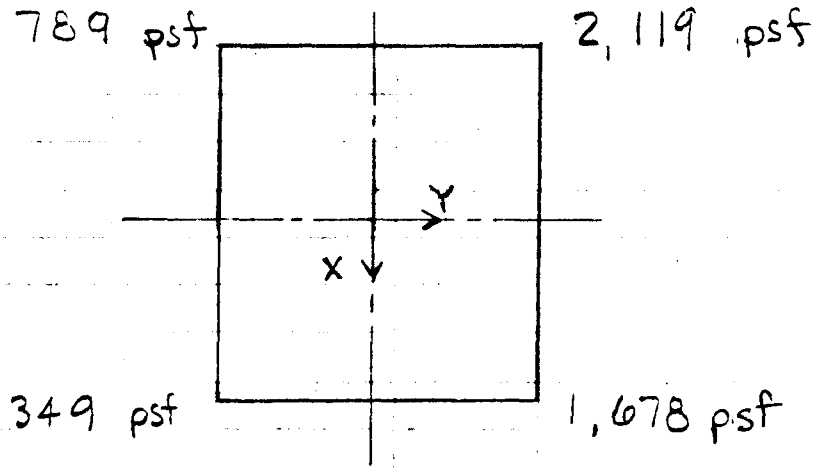
BEARING PRESSURES

$$\frac{P}{A} \pm \frac{Pe_Y}{S_Y} \pm \frac{Pe_X}{S_Y}$$

NOTE: INLET FOOTPRINT WAS NOT USED IN COMPUTING BRG. PRESSURES.

$$\frac{341,534}{15.667(17.667)} \pm \frac{341,534(1.407)6}{17.667(15.667)^2} \pm \frac{341,534(0.526)(6)}{15.667(17.667)^2}$$

$$= 1,233.92 \text{ psf} \pm 664.83 \text{ psf} \mp 220.42 \text{ psf}$$



ALSO SEE SHT. PS 20

NCR Form 381b
1 Aug 80

Subject: ANDALUSIA SLOUGH - PUMP STATION		Date: 20 JUNE 88
Computed by: KEW	Checked by: chd	Sheet PS 17 of

REVISED 19 OCT. 88

STABILITY ($\bar{Y}$)

LOAD CASE II 100YR. FLOOD CONDITIONS

UNIT		FORCE	ARM	MOMENT
DEAD LOAD OF STRUCTURE	↓	343,578	6.899	2,370,423
WIND	→	2,000	27.495	54,990
RIVER FLOW	→	9,803	18.733	183,640
EARTH - ACTIVE	→	16,249	3.667	59,585
- PASSIVE	←	- 28,052	1.777	- 49,848
PUMPS & PIPING	↓	12,009	6.800	8[illegible], 66[illegible]
LIVE LOAD	↓	10,342	6.833	70,667
INLET	↓	40,500	18.556	751,514
WATER	↓	343,898		3,185,252
EARTH ON BASE PROJ	↓	18,450		87,798
UPLIFT	↑	- 486,697		- 4,355,878
		282,080 #	8.649	2,439,804 FT-#

$$e_Y = 8.649 - \frac{13.667}{2} = 1.816^{FT} < \frac{13.667}{6} = 2.278^{FT}$$

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Subject ANDALUSIA SLOUGH - PUMP STATION	Date 20 JUNE 88
Computed by KEW / Checked by cfJ	Sheet PS18 of

REVISED 19 OCT. 88

STABILITY ($\overline{X}$)

LOAD CASE II 100 YR. FLOOD CONDITIONS

UNIT		FORCE	ARM	MOMENT
DEAD LOAD OF STRUCTURE	↓	343,578 #	7.536	2,589,070 FT-#
WIND (NOT IN CONJ. W/ $\overline{Y}$ WIND)	→	—	—	—
PUMPS & PIPING	↓	12,009	11.957	143,590
LIVE LOAD	↓	10,342		51,710
INLET	↓	40,500		162,000
WATER	↓	343,898		2,420,923
EARTH ON BASE PROJ.	↓	18,450		154,974
UPLIFT	↑	-486,697		-3,478,597
		282,080 #	7.245	2,043,670 FT-#

$$e_x = 7.245 - \frac{15.667}{2} = -0.589^{FT} < \frac{15.667}{6} = 2.611^{FT}$$

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Subject ANDALUSIA SLOUGH - PUMP STATION		Date 23 JUNE 88
Computed by KEW	Checked by chJ	Sheet PS 19 of

REVISED 19 OCT. 88

STABILITY

LOAD CASE II 100 YR. FLOOD CONDITIONS

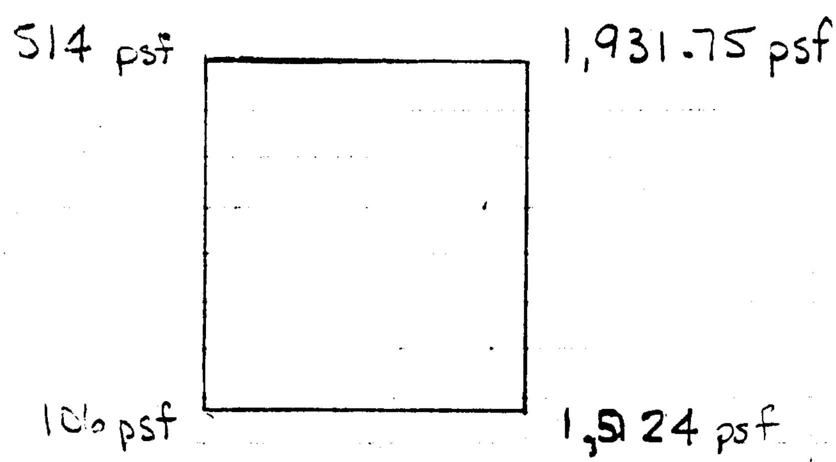
BEARING PRESSURES

$$\frac{P}{A} \pm \frac{Pe_y}{S_y} \pm \frac{Pe_x}{S_y}$$

NOTE: INLET FOOTPRINT WAS NOT USED IN COMPUTING BRG. PRESSURES.

$$= \frac{282,080}{(15.667)(17.667)} \pm \frac{282,080\,(1.816)(6)}{17.667\,(15.667)^2} \pm \frac{282,080(-0.589)(6)}{15.667\,(17.667)^2}$$

$$= 1,019.12 \text{ psf} \pm 708.77 \text{ psf} \mp 203.86 \text{ psf}$$



ALSO SEE SHT. PS 20

NCR Form 381b
1 Aug 80

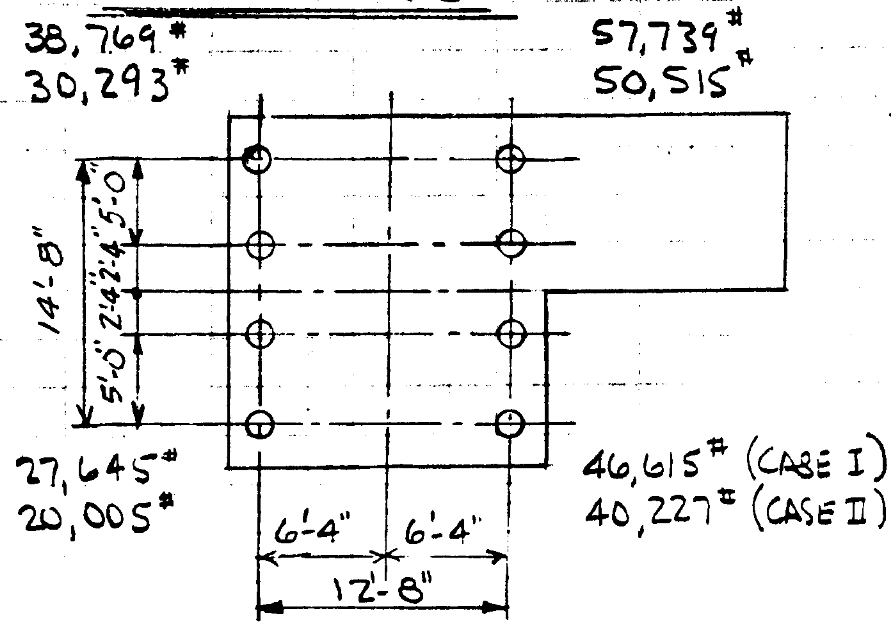
Subject: ANDALUSIA SLOUGH - PUMP STATION | Date: 23 JUNE 88

Computed by: KEW | Checked by: chJ | Sheet: PS 20 of

REVISED 19 OCT. 88

STABILITY

PILE LOADS



$$A = 8$$

$$I_Y = 8(6.333)^2 = 320.85$$

$$I_X = 4(7.333)^2 + 4(2.333)^2 = 236.86$$

LOAD CASE I NORMAL CONDITIONS

$$\frac{P}{A} \pm \frac{P e_Y C_Y}{I_Y} \pm \frac{P e_X C_X}{I_X}$$

$$= \frac{341{,}534}{8} \pm \frac{341{,}534(1.407)(6.333)}{320.85} \pm \frac{341{,}534(-0.526)(7.333)}{236.86}$$

$$= 42{,}692 \pm 9{,}485 \mp 5{,}562 = 46{,}615\#;\ 57{,}739\#;\ 27{,}645\#;\ 38{,}769\#$$

LOAD CASE II 100 YR. FLOOD CONDITIONS

$$= \frac{282{,}080}{8} \pm \frac{282{,}080(1.816)(6.333)}{320.85} \pm \frac{282{,}080(-0.589)(7.333)}{236.86}$$

$$= 35{,}260 \pm 10{,}111 \mp 5{,}144 = 40{,}227\#;\ 50{,}515\#;\ 20{,}005\#;\ 30{,}293\#$$

Subject ANDALUSIA SLOUGH - PUMP STATION	Date 24 OCT. 83
Computed by KEW / Checked by chj	Sheet PS20a

STABILITY

CHECK SLIDING

CASE I (NORMAL)

WIND	5,460#
EARTH - ACTIVE	24,462
	29,922#
EARTH - PASSIVE	38,172#
PILES 8(1,500)	12,000
	50,172#

$$F.S. = \frac{50,172}{29,922} = 1.68$$

CASE II - (100 YR.)

WIND	2,000#
RIVER FLOW	9,803
EARTH - ACTIVE	16,249
	28,052#
EARTH - PASSIVE	38,172#
PILES 8(1,500)	12,000
	50,172#

$$F.S. = \frac{50,172}{28,052} = 1.79$$

BATTER RIVERWARD PILES

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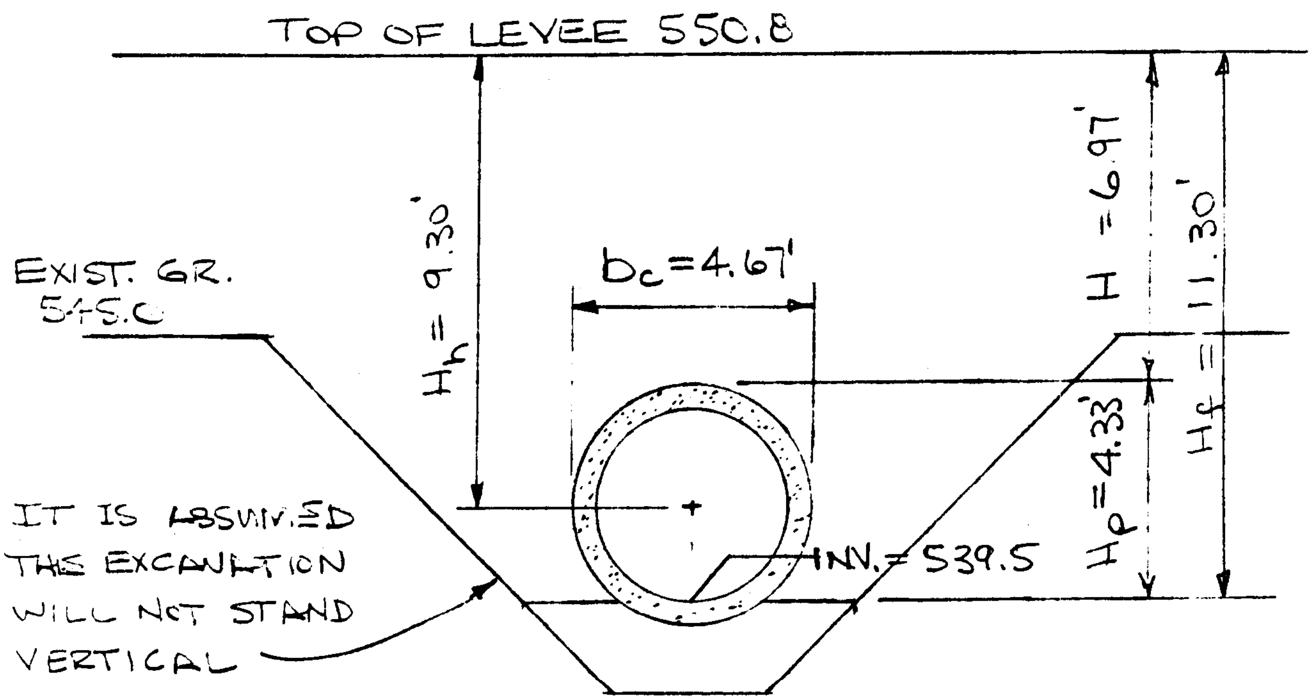
Subject: H.W.LUSIK SLOUGH - PUMP STATION | Date: 22 SEPT. 8[illegible]

Computed by: KEK | Checked by: CAN | Sheet [illegible] of 21

PIPE DESIGN AND SUPPORT

REF. (A) CONDUITS, CULVERTS AND PIPES, EM 1110-2-2902, 3 MAR. 1969

(B) CRETEX CONCRETE PRODUCT CAT. 1972



SECTION THRU PIPE AT PUMP STATION

LOAD CONDITION III - REF. (A)

CASE I.	CASE II.
$W_e = 1.5\,\gamma\, b_c H_h$	$W_e = \gamma\, b_c H_h$
$p_e = 0.5\,\gamma H$	$p_e = \gamma H$

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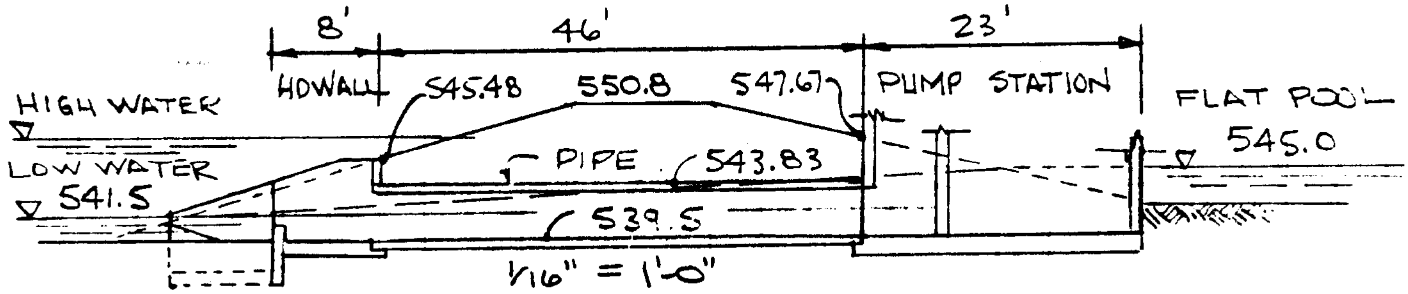
Subject ANDALUSIA SLOUGH - PUMP STATION		Date 22 SEPT. 8?
Computed by KEW	Checked by chJ	Sheet PS 22 of

PIPE DESIGN AND SUPPORT



1/16" = 1'-0"

EARTH LOAD

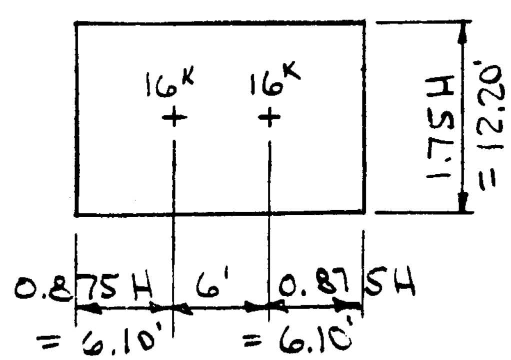
$$W_{e,MAX.} = 1.5(0.115)(4.67)(9.30) = 7.492 \text{ KLF}$$

OR

$$W_{e,MIN.} = 0.115(4.67)(3.98) = 2.137 \text{ KLF}$$

LIVE LOAD

H20 TRUCK = 16.0^K POINT LOAD AT 6' x 14' SPC'G.



$$w_{LL} = \frac{2(16.0)}{12.20(18.20)} = 0.144 \text{ KSF}$$

$$W_{LL} = w_{LL}(b_c) = 0.144(4.67) = 0.672 \text{ KLF}$$

Subject ANDALUSIA SLOUGH - PUMP STATION		Date 23 SEPT. 88
Computed by KEV	Checked by ch	Sheet PS 23 of

PIPE DESIGN AND SUPPORT

WATER LOAD

LOW WATER CONDITION (MAX. LOAD)

A. PIPE HALF FULL OF WATER.

B. WATER OUTSIDE OF PIPE IS APPROX. AT TOP OF PIPE AT PUMP STATION AND 1'-0 BELOW TOP OF PIPE AT HEAD WALL

$$W_{H_2O} = \pi \frac{(2.0)^2}{2} (0.0624) = 0.392 \text{ KLF}$$

$$W_{BOUY} = 0.0624\, b_c H_w = 0.0624(4.67)(4.67 - 0.50)$$

$$= 1.215 \text{ KLF}$$

DEWATERED CONDITION (MIN. LOAD)

A. PIPE EMPTY

B. WATER OUTSIDE OF PIPE AT 545.0 FOR LENGTH OF PIPE

$$W_{BOUY} = 0.0624\, b_c (545.0 - 539.17) = 0.0624(4.67)(5.83)$$

$$= 1.699 \text{ KLF}$$

Subject ANDLLVISIA SLOUGH-PUMP STATION	Date 26 SEPT. 88
Computed by KEN / Checked by ch	Sheet PS 24 of

PIPE DESIGN AND SUPPORT

TOTAL LOADS

LOAD	MAXIMUM	MINIMUM
EARTH	7.492 KLF	2.137 KLF
LIVE	0.672	—
	8.164 KLF	
PIPE WT. $0.145\pi\left[(2.33)^2-(2.00)^2\right]$	0.651	0.651
WATER	0.392	—
	9.207 KLF	2.788 KLF
BOUYANCY	−1.215	−1.699
	7.992 KLF	1.089 KLF

CHECK F.S AGAINST UPLIFT

$$F.S. = \frac{2.788}{1.699} = 1.64 > 1.50$$ OKAY FOR TEMPORARY

SELECT CLASS OF ASTM PIPE (REF. B)

$$D_{.01} = \left(\frac{W_E}{L_{fE}} + \frac{W_L}{L_{fL}}\right)\frac{S_{f01}}{D}$$

GIVEN:

PIPE SIZE = 48" ($b_c = 4.67'$)
BEDDING CLASS = A OR C
PROJECTION RATIO = 0.90 CLASS A
= 0.90 OR 0.70 CLASS C
SETTLEMENT RATIO = 1.00 CLASS A
0.30 CLASS C

Subject ANDALUSIA SLOUGH - PUMP STATION | Date 26 SEPT. 88
Computed by KEN | Checked by clw | Sheet PS25 of

PIPE DESIGN AND SUPPORT

GIVEN: (CONT.)

SAFTY FACTOR = 1.00 (0.01" CRACK)

FIND LOAD FACTOR FROM PAGE Q-25

$$\frac{H}{b_c} = \frac{6.97}{4.67} = 1.493 \quad \text{SAY } 1.50$$

		P=0.9	P = 0.70	USE
CLASS "A"	L_f =	4.49	—	4.00
"C"	L_f =	2.16	1.99	2.00

FIND PIPE CLASS FOR "A" OR "C" BEDDING

CLASS "A" BEDDING

$$D_{.01} = \frac{8.164}{4.00}\frac{(1.0)}{(4.0)} = 0.510^K = 510^{\#}$$

CLASS 1 PIPE D-LOAD = 800#

CLASS "C" BEDDING

$$D_{.01} = \frac{8.164}{2.00}\frac{(1.0)}{(4.0)} = 1.020^K = 1,020^{\#}$$

CLASS 3 PIPE D-LOAD = 1350#

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Subject ANDALUSIA SLOUGH - PUMP STATION		Date 26 SEPT 8?
Computed by KEW	Checked by ckJ	Sheet PS 26 of

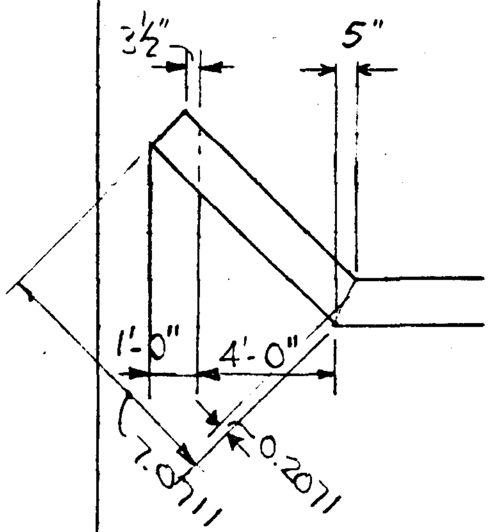
PIPE DESIGN AND SUPPORT

NOTE: CLASS "A" BEDDING WAS USED FOR PIPE SUPPORTED ON PILES.

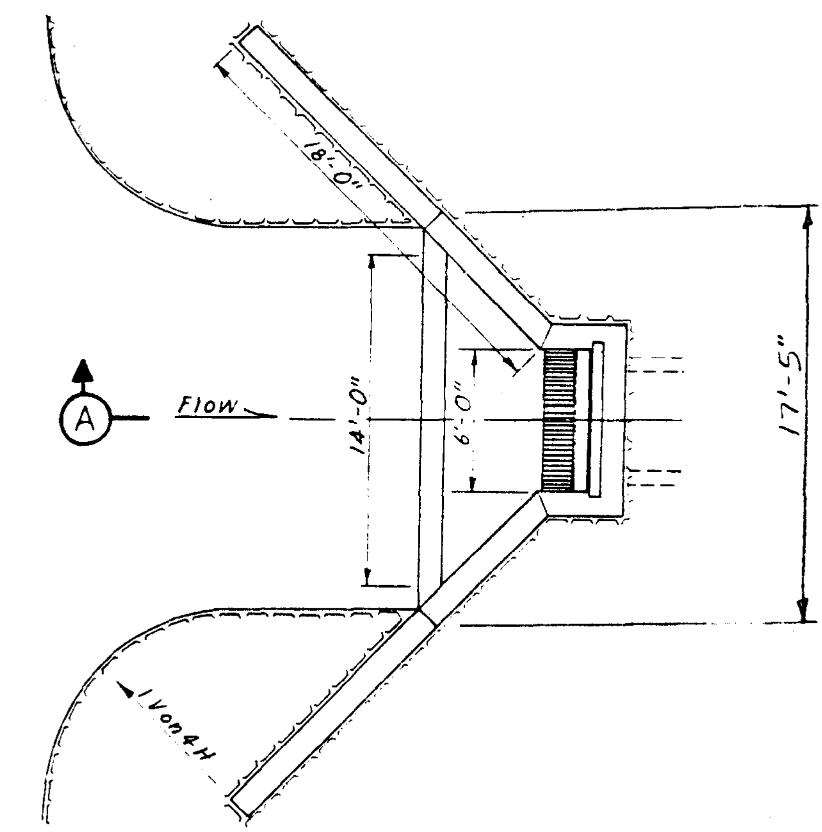
THE SOILS ARE SUCH THAT THE PUMP STATION SHOULD BE SUPPORTED ON PILES. IN ORDER TO CONTROL DIFFERENTIAL SETTLEMENT BETWEEN THE PIPE AND PUMP STATION THE PIPE IS ALSO SUPPORTED ON PILES. SEE SHEET PF-5.

Subject ANDALUSIA SLOUGH - PUMP STA. HEADWALL		Date 24 JUNE 8?
Computed by KEW	Checked by chJ	Sheet HW 1 of

STABILITY



HEADWALL CNTR

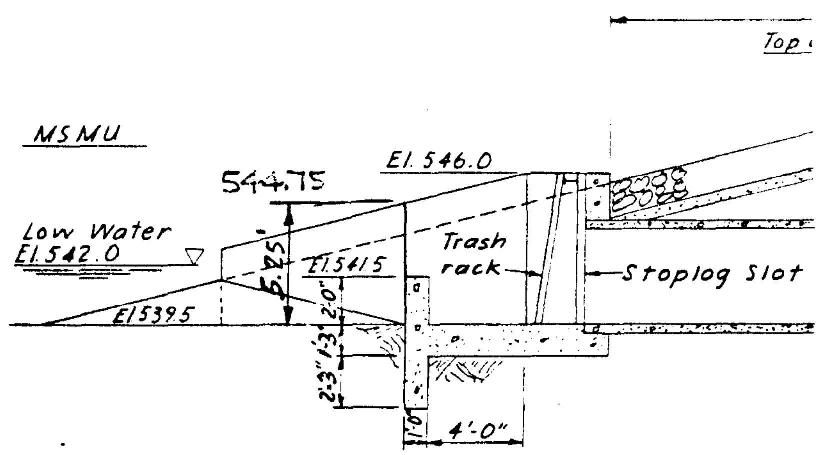


BACKFILL MAT'L.

$\gamma_{DAMP} = 115$ pcf

$c = 0$

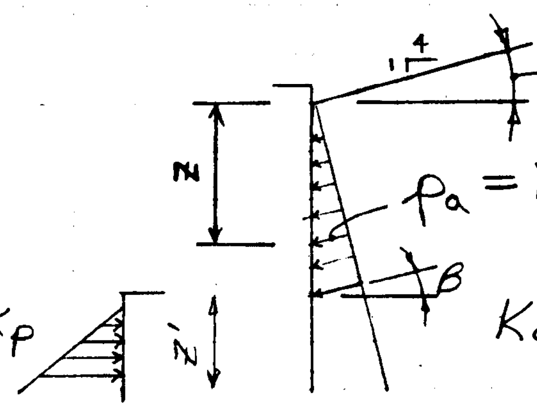
$\phi = 32°$



Subject: ANDALUSIA SLOUGH - PUMP STA. HEADWALL | Date: 24 JUNE 8?

Computed by: KEW | Checked by: chJ | Sheet HW. ? of

STABILITY HEADWALL CONT'D



$\beta = 14.036^\circ$

$p_a = \gamma z K_a$

$p_p = \gamma z' K_p$

$$K_a = \cos\beta \left[\frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}\right]$$

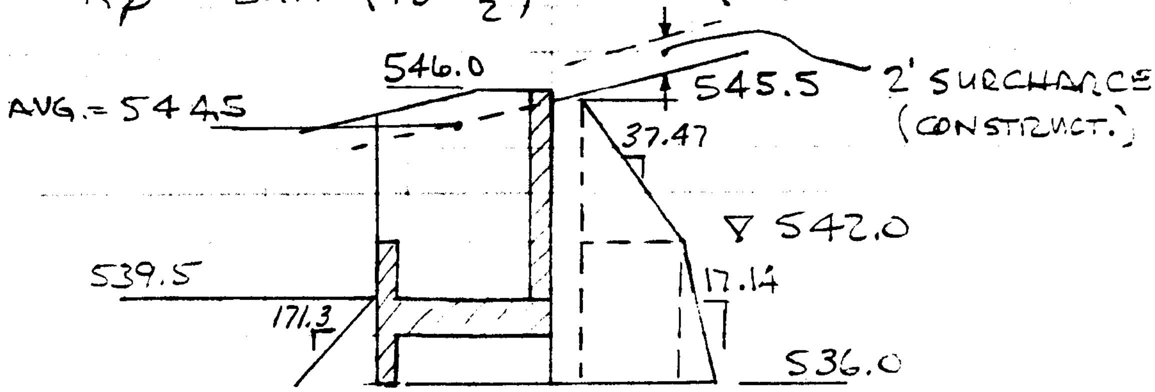
$$K_a = 0.97014 \left[\frac{0.97014 - \sqrt{(0.97014)^2 - (0.84805)^2}}{0.97014 + \sqrt{(0.97014)^2 - (0.84805)^2}}\right]$$

$$= 0.97014 \frac{(0.97014 - 0.47115)}{(0.97014 + 0.47115)} = 0.97014 \frac{(0.49899)}{1.44129}$$

$$= 0.33587$$

$$K_{a_H} = 0.97014(0.33587) = 0.32584$$

$$K_p = \tan^2\left(45 + \frac{\phi}{2}\right) = \tan^2(61) = 3.255$$



Subject ANDALUSIA SLOUGH - PUMP STA. HEADWALL	Date 24 JUNE 8?
Computed by KEW / Checked by chV	Sheet HW3 of

STABILITY HEADWALL CONT.

UNIT	FORCE	ARM	MOMENT
WALLS 1.00(150)(8.0)(6.5)	7,800	0.50	3,900
-1.00(150)(π)(4.0)^2÷4	- 1,884	0.50	- 942
1.00 (150)(2.29)(6.5) 2	4,466	2.145	9,580
1.00 (150)(15.00)(5.5)	12,375	8.00	99,000
1.00 (150)(6.86)(5.25) 2	10,805	6.00	64,830
1.00 (150)(6.86)($\frac{1.25}{2}$) 2	1,286	5.167	6,645
BASE 1.25 (150)(8.0)(7.5)	11,250	3.75	42,188
1.25(150)(4.4167)($\frac{4.4167}{2}$) 2	3,658	6.028	22,049
WATER 2.50(62.4)(6.00)(7.5)	7,020	3.750	26,325
2.50 (62.4)(4.00)($\frac{4.00}{2}$) 2	2,496	6.1667	15,392
0.50(62.4)(1.00)(15.0)	468	8.00	3,744
UPLIFT -3.75(62.4)(8.00)(7.5)	-14,040	3.750	- 52,650
-3.75 (62.4)(4.4167)($\frac{4.4167}{2}$) 2	- 4,565	6.028	- 27,518
-6.00 (62.4)(1.00)(16.5)	- 6,178	8.00	- 49,424
	# 34,957	4.6663	FT-# 163,118
EARTH LOADS (NORMAL)(SHT. HW2)			
ACTIVE			
37.47 (2.0)(9.50)(8.00)	5,695	4.75	27,051
37.47 (2.0)(8.50)(17.4142-8.0)	5,997	4.25	25,487
37.47 (3.5)($\frac{3.5}{2}$)(8.00)	1,836	7.167	13,159
37.47 (2.5)($\frac{2.5}{2}$)(17.4142-8.0)	1,102	6.833	7,530
37.47 (3.5)(6.0)(8.00)	6,295	3.00	18,885

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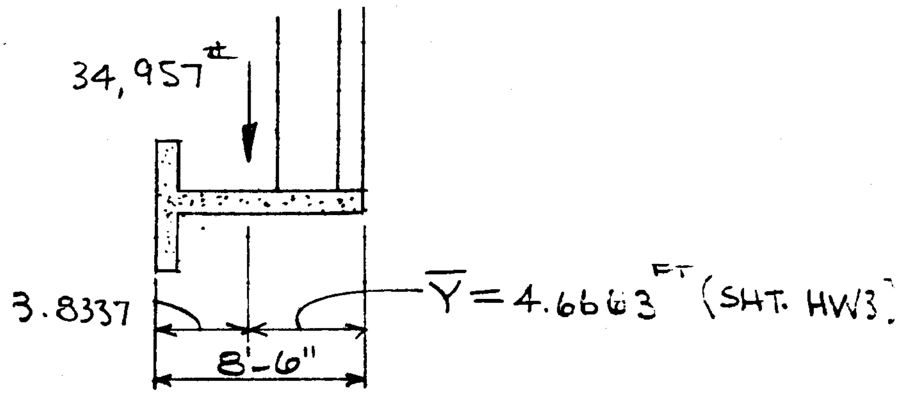
Subject: ANDALUSIA SLOUGH – PUMP STA. HEADWALL		Date: 24 JUNE 88
Computed by: KEW	Checked by: chJ	Sheet: HW 4 of

STABILITY HEADWALL CNTR.

UNIT	FORCE	ARM	MOMENT
ACTIVE (CONT.)			
$37.47(2.50)(6.00)(17.4142-8.0)$	5,291	3.00	15,873
$17.14(6.00)\left(\frac{6.00}{2}\right)(17.4142)$	5,373	2.00	10,746
$-37.47(5.50)(\pi)(2.5)^2$	−4,046	6.00	−24,276
	27,543 #		94,455 FT-#
PASSIVE			
$171.3(3.50)\left(\frac{3.50}{2}\right)(17.4142)$	−18,271 #	1.1667	−21,317 FT-#

CHECK OVERTURNING F.S.

30 JUNE 88



$$M_R = 34{,}957(3.8337) + M_{PASSIVE}$$

$$= 134{,}016 + 21{,}317 = 155{,}333 \text{ FT-\#}$$

$$M_o = M_{ACTIVE} = 94{,}455 \text{ FT-\#}$$

$$F.S. = \frac{M_R}{M_o} = \frac{155{,}333}{94{,}455} = 1.64 > 1.50 \text{ OKAY}$$

Subject ANDALUSIA SLOUGH - PUMP STA. HEADWALL		Date 30 JUNE 8?
Computed by KEW	Checked by chd	Sheet HW.5 of

STABILITY - HEADWALL CNTR.

CHECK SLIDING

$P_{ACTIVE} = 27{,}543^{\#}$

$P_{PASSIVE} + \mu P_{VERT} = 18{,}271 + 0.30(34{,}957)$

$= 28{,}758^{\#}$

$F.S. = \dfrac{28{,}758}{27{,}543} = 1.05 < 1.5$ NO GOOD

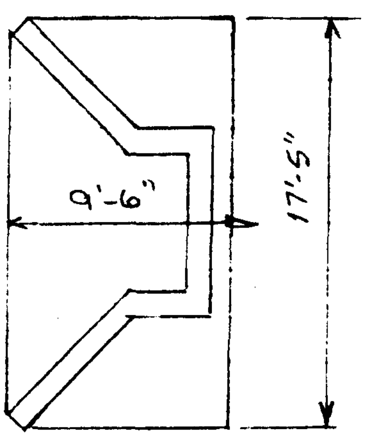
WITHOUT CONSTRUCTION SURCHARGE

$P_{ACTIVE} = 27{,}543 - (5{,}695 + 5{,}997)$

$= 15{,}851^{\#}$

$F.S = \dfrac{28{,}758}{15{,}851} = 1.82 < 2.0$ NO GOOD

INCREASE BASE TO INCREASE VERT. LOAD



Subject ANDALUSIA SLOUGH - PUMP STA. HEADWALL		Date 30 JUNE 89
Computed by	Checked by	Sheet HW 6 of

STABILITY HEADWALL CONTD.

RECOMPUTED LOADS

UNIT	FORCE	ARM	MOMENT
ADDED BASE $1.25(150)(1.0)(17.4142)$	3,265	-0.50	- 1,633
$1.25(150)(2.4167)(17.4142-8.0)$	4,266	1.2084	5,155
$1.25(150)(4.7071)(\frac{4.7071}{2})2$	4,154	3.9857	16,557
ADDED EARTH $6.0(115)(1.0)(17.4142)$	12,016	-0.50	- 6,008
$-115(\pi)(2.5)^2(1.0)$	- 2,258	-0.50	1,129
$5.6(115)(2.4167)(17.4142-8.0)$	14,652	1.2084	17,705
$4.8(115)(4.7071)(\frac{4.7071}{2})2$	12,231	3.9857	48,749
ADDED UPLIFT $-3.75(62.4)(1.0)(17.4142)$	-4,075	-0.50	2,038
$-3.75(62.4)(2.4167)(17.4142-8.0)$	-5,324	1.2084	- 6,434
$-3.75(62.4)(4.7071)(\frac{4.7071}{2})2$	-5,185	3.9857	-20,666
PREVIOUS VERT. LOADS	34,957	4.6663	163,118
	68,699 #	3.1982	219,710 Ft-#
EARTH LOADS			
ACTIVE			
$37.47(2.0)(9.75)(17.4142)$	12,724	4.875	62,030
$37.47(3.75)(\frac{3.75}{2})(17.4142)$	4,588	7.250	33,263
$37.47(3.75)(6.00)(17.4142)$	14,681	3.00	44,043
$17.14(6.00)(\frac{6.00}{2})(17.4142)$	5,373	2.00	10,746
$-37.47(5.75)(\pi)(2.5)^2$	- 4,230	6.00	- 25,380
	33,136 #		124,702 Ft-#
PASSIVE (SEE SHT. HW 4)	-18,271 #	1.1667	- 21,317 Ft-#

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Subject ANDALUSIA SLOUGH - PUMP STA. HEADWALL	Date 30 JUNE 88
Computed by KEW / Checked by CHJ	Sheet HW7 of

STABILITY - HEADWALL CNTR

RE-CHECK SLIDING

$P_{ACTIVE} = 33{,}136^{\#}$

$P_{PASSIVE} + \mu P_{VERT} = 18{,}271 + 0.30(68{,}699)$

$= 38{,}881^{\#}$

$F.S. = \frac{38{,}881}{33{,}136} = 1.17 < 1.50$ NO GOOD

WITHOUT CONSTRUCTION SURCHARGE

$P_{ACTIVE} = 33{,}136 - 12{,}724 = 20{,}412^{\#}$

$F.S. = \frac{38{,}881}{20{,}412} = 1.90 < 2.00$ ALMOST GOOD ENOUGH

CHECK BEARING PRESSURES

$P_V = 68{,}699^{\#}$ $\quad \Sigma M = 323{,}095^{FT-\#}$

$\overline{Y} = 4.703^{FT}$

$e = 4.703 + 1.00 - \frac{9.50}{2} = 0.953^{FT} < \frac{9.5}{6}$

< 1.583

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Subject ANDALUSIA SLOUGH - PUMP STA. HEADWALL		Date 30 JUNE 84
Computed by KEW	Checked by chJ	Sheet HW8 of

STABILITY - HEADWALL CNTR.

$$\frac{P}{A} \pm \frac{Me_Y}{S_Y} = \frac{68,699}{9.5(17.4142)} \pm \frac{68,699(0.953)(6)}{(9.5)^2(17.4142)}$$

$$= 415.3 \pm 249.9 = 665.2 \text{ psf}$$
OR
$$165.4 \text{ psf}$$

NOTE: ALTHOUGH BEARING APPEARS TO BE NO PROBLEM, SLIDING IS. PROVIDING TIE RODS BETWEEN THE PUMP STATION AND THE HEADWALL CNTR. WILL PREVENT SLIDING.

$$F = \frac{33,136}{2} = 16,568\ \#/\text{ROD}$$

USED BATTERED PILES

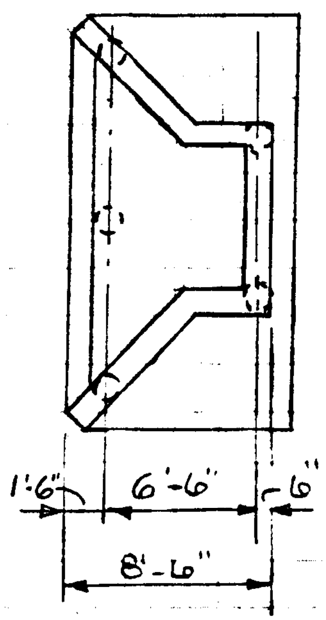
PROVIDE 2 - 1"ϕ A 522, GR.50 RODS W/ TURNBUCKLES

CONSIDER PILE FOUNDATION WITH BATTERED (4:1) FRONT PILES)

Subject ANDALUSIA SLOUGH - PUMP STR. HEADWALL		Date 30 JUNE 88
Computed by KEW	Checked by cHJ	Sheet HW9 of

STABILITY - HEADWALL CNTR

PILE LOADS - 5 PILES



$A = 5$

$C = \frac{2(0) + 3(6.5)}{5}$

$= 3.90^{FT}$

$I = 2(3.90)^2 + 3(2.6)^2$

$= 30.42 + 20.28$

$= 50.70 \ FT^2$

$e = \bar{Y} - C = 4.703 - 3.90$

$= 0.803^{FT}$

$\frac{P}{A} \pm \frac{PeC}{I} = \frac{68,699}{5} \pm 68,699 \frac{(0.803)}{50.70}(2.6 \text{ OR } 3.9)$

$= 13,740 \ \begin{matrix} + 2,829 \\ - 4,244 \end{matrix} = 16,569^{\#}$ OR $9,496^{\#}$

PILE LOADS - 3 PILES

$A = 3 \qquad C = \frac{1(0) + 2(6.5)}{3} = 4.333^{FT}$

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Subject ANDALUSIA SLOUGH - PUMP STA. HEADWALL		Date 30 JUNE 88
Computed by KEN	Checked by	Sheet HWC

STABILITY - HEADWALL CNTR.

$$I = 1(4.333)^2 + 2(2.167)^2$$

$$= 18.77 + 9.39 = 28.16 \text{ FT}^2$$

$$e = \bar{Y} - C = 4.703 - 4.333 = 0.370 \text{ FT}$$

$$\frac{P}{A} \pm \frac{Pec}{I} = \frac{68,699}{3} \pm 68,699 \frac{(0.370)}{28.16}(2.167 \text{ or } 4.333)$$

$$= 22,900 \begin{array}{l} + 1,956 \\ - 3,911 \end{array} = \begin{array}{l} 24,856^{\#} \\ \text{or} \\ 18,989^{\#} \end{array}$$

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Subject ANDALUSIA SLOUGH - PUMP STATION		Date 20 SEPT. 88
Computed by KEW	Checked by chJ	Sheet PF-1 of

PILE FOUNDATION DESIGN

REF: ① FOUNDATION DESIGN, WAYNE C. TENG, 1962

② FOUNDATIONS & EARTH STRUCTURES, NAVFAC DM-7.2, MAY 1982

PUMP STATION (SEE SHT. PS-20)

PILE LOAD = 57,739# = 28.87 TON (RIVERSIDE R...

FOR BATTERED PILE (4:1), F = 1.031 (28.87)

= 29.76 TON

FROM BORING A-88-9

a.) PILES DRIVEN INTO MEDIUM TO FINE SAND WITH BLOW COUNT OF 8 BLOWS / FT

b.) DEPTH OF BORING IS ONLY 16 FT BELOW BOTTOM OF PUMP STATION

$\Phi = 35°$; $\gamma = 115$ pcf REF.①, page 12

$N_q = 40$; $N_\gamma = 45$ REF.①, page 58

$$Q_{ULT} = \pi R_T^2 (\gamma D N_q + 0.6 \gamma R_T N_\gamma)$$

$$+ 2\pi R_A L (\gamma Z + q_L) K \tan\phi$$ REF.① Eq (8-1) + Eq (8-2) pg. 212 & 213

Subject ANDALUSIA SLOUGH - PUMP STATION		Date 20 SEPT. 88
Computed by KEW	Checked by chl	Sheet PF-2 of

PILE FOUNDATION DESIGN

WHERE : R_T = RADIUS OF PILE TIP
R_A = AVG. RADIUS OF PILE
γ = UNIT WEIGHT OF SOIL (BOUYANT WT.)
D = TOTAL PENETRATION OF PILE
L = LENGTH OF PENETRATION INTO GRANULAR SOIL.
Z = DEPTH OF CENTER OF GRAVITY OF EMBEDDED PORTION OF PILE.
q = PERMANENT SURCHARGE LOAD.
K = COEFFICIENT OF LATERAL EARTH PRESSURE (ASSUME 1.25) REF. 2, pg 7.2-194

ASSUME ∴
40 FT PENETRATION INTO GRANULAR SOIL
47.5 FT TOTAL PENETRATION (50 FT PILE)
44" BUTT CIRCUM. 7.00 RADIUS
22 TIP CIRCUM. 3.50 RADIUS
33 AVG. CIRCUM. 5.25 RADIUS

$$Q_{ULT_S} = \pi \left(\frac{3.50}{144}\right)^2 \left[(\overset{52.5}{115-62.5})(40) + 0.6 (\overset{52.5}{115-62.5})\left(\frac{3.50}{12}\right)(45) \right]$$

$$+ 2\pi \left(\frac{5.25}{12}\right)(40) \left[(\overset{52.5}{115-62.5})\left(\frac{40}{2} + 7.5\right)\right](1.25)(\tan 35)$$

$$= 672 + 138,945 = 139,617^{\#}$$

CALCULATE REDUCTION DUE TO GROUP ACTION

NOTE: SEE PS-20, PILES ARE FAR ENOUGH APART IN ONE DIRECTION (7 DIAM) TO BE CONSIDERED A SINGLE ROW GROUP. REF. 2 pg 7.2-204

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Subject ANDALUSIA SLOUGH - PUMP STATION		Date 20 SEPT. 88
Computed by KEV.	Checked by ch.J	Sheet PF-3 of

PILE FOUNDATION DESIGN

REF. (3) DESIGN OF PILE FOUNDATIONS AND STRUCTURES, EM 1110-2-2906

$$F = 1 - \left(2 - \frac{1}{n} - \frac{1}{m}\right)\frac{\theta}{90}$$ REF. (3), fig 11

WHERE F = EFFICIENCY FACTOR OF A PILE IN A GROUP

n = NUMBER OF PILE IN A ROW

m = NUMBER OF ROWS

$\theta = \tan^{-1} \frac{d}{S}$

d = PILE DIAM

S = PILE SPACING

$$F = 1 - \left(2 - \frac{1}{4} - \frac{1}{1}\right)\frac{\tan^{-1}\left(\frac{10.50}{12}/4.667\right)}{90}$$

$$= 0.9115$$

$$\therefore Q_{ULT_G} = 139,617(0.9115) = 127,262^{\#} = 63.63^{T}$$

ASSUME 5 TON NEG FRICTION DUE TO DRIVING OF ADJACENT PILES

ALSO ASSUME F.S. = 2.0

Subject ANDALUSIA SLOUGH - PUMP STATION		Date 20 SEPT. 89
Computed by KEW	Checked by c/d	Sheet PF-4 of

PILE FOUNDATION DESIGN

PILE CAPACITY

$$Q = \frac{Q_{ULT_G}}{F.S.} - 5.00 = \frac{63.63}{2.0} - 5.00$$

$$= 26.82^{TON} < 29.76^{TON} \text{ REQ'D}$$

11.0% UNDER DESIGNED

CONSIDERED OKAY BECAUSE THE PILE BEING CONSIDERED IS THE LAST ONE IN THE ROW AND THE ONE WITH THE MAX LOAD. ALSO OVERTURNING IN THE TRANSVERSE DIRECTION WILL BE RESISTED BY EARTH.

PILE LOAD = $38{,}769^{\#} = 19.38^{TON}$ (LANDSIDE ROW)

ASSUME 35^{FT} PENETRATION INTO GRANULAR SOIL (45^{FT} PILE / 44" BUTT & 24" TIP)

$$Q_{ULT_S} = \pi \frac{(3.82)^2}{144} \left[(115-62.5)(40) + 0.6(115-62.5)\left(\frac{3.82}{12}\right)(40)\right]$$

$$+ 2\pi\left(\frac{5.41}{12}\right)(35)\left[(115-62.5)\left(\frac{35}{2}+7.5\right)\right](1.25)(\tan 35)$$

$$= 812 + 113{,}894 = 114{,}706^{\#}$$

$$Q_{ULT_G} = 114{,}706(0.9115) = 104{,}554^{\#} = 52.28^{TON}$$

Subject ANDALUSIA SLOUGH - PUMP STATION		Date 20 SEPT. 88
Computed by KEW	Checked by c.l.J	Sheet PF-5 of

PILE FOUNDATION DESIGN

PILE CAPACITY

$$q = \frac{Q_{ULT_G}}{F.S.} - 5.00 = \frac{52.28}{2.0} - 5.00$$

$$= 21.14^{TON} > 19.38^{TON}$$

PIPE SUPPORT (SEE SHT. PS 24)

PILE LOAD $= 7,992(3.5) = 27,972^{\#}$ OR 13.99^{TON}

NOTE: 30^{FT} PENETRATION (40 PILE) SHOULD PROVIDE ENOUGH SUPPORT

HEADWALL (SEE SHT. HW 10)

PILE LOAD $= 24,856^{\#} = 12.43^{TON}$

FOR BATTERED PILE (4:1), $P = 1.031(12.43)$

$$= 12.82^{TON}$$

ASSUME 25^{FT} PENETRATION INTO GRANULAR SOIL (30^{FT} PILE/44" BUTT & 27" TIP)

$$Q_{ULT_S} \simeq 963 + 2\pi\left(\frac{5.65}{12}\right)(25)\left[(115-62.5)\left(\frac{25}{2}+7.5\right)\right](1.25)(\tan 35)$$

$$= 963 + 67,969 = 68,932^{\#} \text{ OR } 34.46^{TON}$$

Subject ANDALUSIA SLOUGH - PUMP STATION		Date 20 SEPT. 8?
Computed by KEW	Checked by c/w	Sheet PF-6 of

PILE FOUNDATION DESIGN

PILE CAPACITY

$$Q = \frac{Q_{ULTS}}{F.S.} - 5.00 = \frac{34.46}{2} - 5.00$$

$$= 12.23^{TON} \simeq 12.82^{TON}$$

4.8% UNDER DESIGN

Subject ANDALUSIA SLOUGH – PUMP STATION		Date 4 MAY 8?
Computed by KEW	Checked by chJ	Sheet MISC. 1 of

TRASH RACK DESIGN

EM 1110-2-3104

DESIGN FOR DIFFERENTIAL HEAD OF 5 FT. DUE TO TRASH BUILD-UP.

EM 1110-2-3102

BAR SPACING 1 3/4" INCHES CLEAR (3" MAX. IF JUSTIFIED)

FLOW THRU GROSS RACK AREA $\leq$ 2.5 FT/SEC.

PUMPING RATE = 10,000 GPM

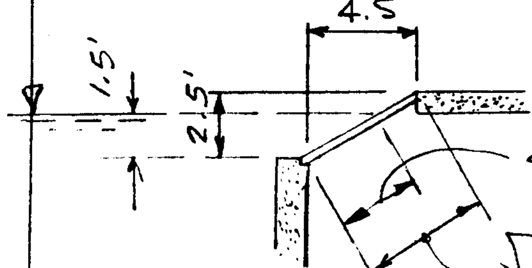
$= \frac{10,000}{7.48} = 1,337$ CU. FT/MIN

$= \frac{1,337}{60} = 23$ CU. FT/SEC

RIVER SIDE TRASH RACK

$A = \overset{\text{WIDTH}}{5.00}(3.083) = 15.44\ FT^2$

$V = \frac{23}{15.44} = 1.49$ FT/SEC



$5.148\left(\frac{1.5}{2.5}\right) = 3.089$ FT

$\sqrt{4.5^2+2.5^2} = 5.148$ FT

Subject ANDALUSIA SLOUGH - PUMP STATION		Date 4 MAY 88
Computed by KEW	Checked by chJ	Sheet MISC 2 of

TRASH RACK DESIGN

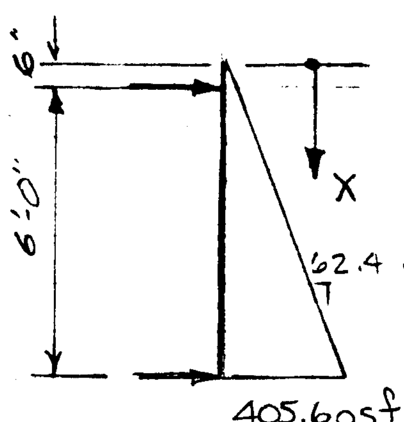
LANDSIDE TRASH RACK

$$A = \underset{\text{WIDTH}}{6.00}\ \underset{H_2O \text{ DEPTH}}{(2.00)} = 12.00\ FT^2$$

$$V = \frac{23}{12} = 1.92\ FT/SEC.$$

6'-6" HEAD

$$R_{TOP} = 405.6\frac{(6.5)}{2}\frac{(6.5)}{3}\frac{(1)}{6.0} = 476.0^{\#}$$



$$R_{BOTT.} = 405.6\frac{(6.5)}{2}\left[\frac{2}{3}(6.5) - 0.5\right]\frac{(1)}{6.0} = 842.2^{\#}$$

$$V = 0 = 476.0 - 62.4(X)\frac{X}{2}$$

$$X = 3.906\ FT.$$

$$M = 476.0\,(3.906 - 0.50) - 62.4\,\frac{(3.906)^3}{6}$$

$$= 1,621.2 - 619.8 = 1,001.5\ FT\text{-}\#/FT$$

$$= 12,017.8\ IN\text{-}\#/FT$$

ASSUME 2" BAR SPACING 3/8" BAR + 1 5/8" CLR.

$$M = 2,003.0\ IN\text{-}\#/BAR$$

Subject: ANDALUSIA SLOUGH - PUMP STATION		Date: 4 MAY 88
Computed by: KEW	Checked by: chJ	Sheet MISC 3 of

TRASH RACK DESIGN

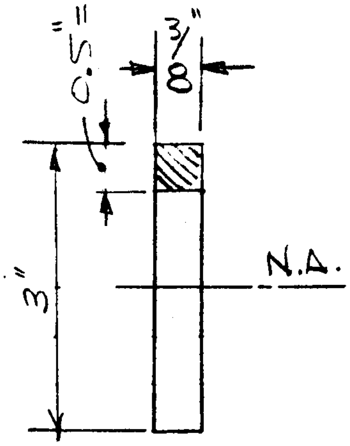
TRY 3" x 3/8" BAR

$$S = 3/8 \frac{(3.0)^2}{6} = 0.5625 \text{ IN}^3$$

$$f_b = \frac{2{,}003.0}{0.5625} = 3{,}561 \text{ psi}$$

ASSUME BAR ACTS AS STEM OF TEE

$$\frac{b}{t} = \frac{2.5}{0.375} = 6.67 < \frac{127}{\sqrt{F_y}} = 21$$



$$r_T = \sqrt{\frac{I}{A}} = \sqrt{\frac{\frac{0.5(0.375)^3}{12}}{0.5\,(0.375)}}$$

$$= 0.10825 \text{ IN}$$

$$\frac{\lambda}{r_T} = \frac{8.5\,(12)}{0.10825} = 942.2$$

AISC 1.5.1.4.5

$$\sqrt{\frac{510 \times 10^3}{F_y}} = 119.0 < \frac{\lambda}{r_T}\ ; \quad \therefore F_b = \text{LARGER OF BELOW}$$

$$F_b = \frac{170 \times 10^3}{\left(\frac{\lambda}{r_T}\right)^2} = \frac{170 \times 10^3}{(942.2)^2} = 0.192 \text{ KSI}$$

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OR

Subject ANDALUSIA SLOUGH - PUMP STATION		Date 4 MAY 88
Computed by KEW	Checked by chV	Sheet MISC. 4 of

TRASH RACK DESIGN

$$F_b = \frac{12{,}000}{\frac{ld}{A_f}} = \frac{12{,}000\,(0.5)(0.375)}{8.5(12)(3.0)}$$

$$= 7.353 \text{ KSI} > f_b = 3.561 \text{ KSI}$$

BEARING BEAM

ASSUME 1,000 #/FT LOAD (NO GUIDE)

$$M = 1{,}000 \frac{(6.00)^2}{8} = 4{,}500^{\,FT/\#}$$

$$S = \frac{4{,}500(12)}{24{,}000} = 2.25 \text{ IN}^2$$

USE W4×13

$S = 5.46 \text{ IN}^3$

Subject: ANDALUSIA SLOUGH – PUMP STATION		Date: 4 MAY 88
Computed by: KEV	Checked by: chJ	Sheet MISC.5 of

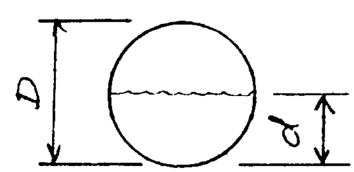
SIZE CONCRETE PIPE

1. ASSUME PIPE RUNS HALF-FULL
2. SIZE SO THAT VELOCITY IN THE PIPE IS LESS THEN 5.0 FT/SEC.

PUMPING RATE = 10,000 GPM

$$= \frac{10,000}{448.8} = 22.28 \text{ CU.FT./SEC.}$$

PIPE DIAM	VELOCITY		
36"Ø	$\frac{22.28}{\frac{\pi}{2}(1.5)^2}$	= 6.30 FT/SEC.	
42"Ø	$\frac{22.28}{\frac{\pi}{2}(1.75)^2}$	= 4.63 FT/SEC.	OKAY
48"Ø	$\frac{22.28}{\frac{\pi}{2}(2.0)^2}$	= 3.55 FT/SEC.	⇐ USE



$\frac{d}{D} = 0.50$

L = 48'-0" ±

SLOPE OF PIPE

$$S = \left[\frac{Qn}{0.232\,D^{8/3}}\right]^2$$

$$= \left[\frac{22.28(0.015)}{0.232\,(4.0)^{2.667}}\right]^2 = (0.0357)^2 = 0.00128 \text{ FT/FT}$$

$= 0.737$ IN TOTAL

NIL – DO NOT SLOPE PIPE

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Subject ANDALUSIA SLOUGH – PUMP STATION		Date 19 MAY 88
Computed by KEW	Checked by chJ	Sheet RF 1 of

ROOF SLAB (SPAN = 8'-6")

LOADS

SLAB 50 psf

MISC 10

60 psf

$$d = 4 - \underset{CLR}{0.75} - \underset{BAR/2}{0.25} = 3 \text{ IN}$$

SNOW 30 psf

ACI 318-83

$$w_u = 1.7(30) + 1.4(60) = 135 \text{ psf}$$

$$V_{ud} = 135\left(\frac{8.5}{2} - \frac{3}{12}\right) = 540^{\#}$$

$$M_u = 135\frac{(8.5)^2}{8} = 1{,}220^{FT-\#}$$

BALANCED RATIO OF REINFORCEMENT

LET $f_y = 48{,}000$ psi ; $\beta_1 = 0.85$ ACI 10.2.7.3

$f'_c = 3{,}000$ psi

$$\frac{X_b}{d} = \frac{87{,}000}{87{,}000 + f_y} = \frac{87{,}000}{135{,}000} = 0.6444$$

$$\rho_b = 0.85\frac{f'_c}{f_y}\beta_1\frac{X_b}{d} = 0.85\left(\frac{3.0}{48.0}\right)(0.85)(0.6444)$$

$$= 0.0291$$

$$0.75\rho_b = 0.75(0.0291) = 0.02183$$

Subject ANDALUSIA SLOUGH - PUMP STATION		Date 20 MAY 88
Computed by KEW	Checked by chU	Sheet RF 2 of

ROOF SLAB

DETERMINE DEPTH OF SECTION

$$M_u = \phi \rho f_y b d_b^2 \left(1 - 0.59 \rho \frac{f_y}{f'_c}\right)$$

$$1.220(12) = 0.90(0.02183)(48)(12)(d_b)^2\left[1 - 0.59(0.02183)\left(\frac{48}{3}\right)\right]$$

$$1.220(12) = 8.984 d_b^2$$

$$\therefore d_b = \sqrt{\frac{1.220(12)}{8.984}} = 1.28\text{ IN} \qquad \text{SAY } d = 3\text{ IN}$$

CHECK SHEAR

$$V_c = 2\sqrt{f'_c}\, b d$$

$$= 2\sqrt{3{,}000}(12)(3) = 3{,}943^{\#}$$

$$\phi V_c = 0.85(3{,}943) = 3{,}352^{\#} > V_{u_d} = 540^{\#}$$

COMPUTE REINFORCING

REF: TECH. REPORT SL-80-4 "STRENGTH DESIGN OF REINFORCED CONCRETE HYDRAULIC STRUCTURES" - REPORT #2

$$\frac{h}{a} = \frac{4}{3} = 1.33 \Leftarrow$$

$f'_c = 3.00$ KSI AND $f_y = 48.00$ KSI

NCR Form 381b
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Subject ANDALUSIA SLOUGH - PUMP STATION		Date 20 MAY 85
Computed by KEW	Checked by chU	Sheet RF3 of

ROOF SLAB

$$\frac{M_N}{bd^2} = \frac{1.220\,(12)}{0.90\,(12)(3.0)^2} = 0.151$$

FIG. 10 $\rho = 0.0033$

$$\rho_{min} = \frac{200}{48{,}000} = 0.00417 > \rho = 0.0033$$

$$> \frac{4}{3}\rho = 0.0044$$

$$\therefore A_S = 0.0044(12)(3)$$

$$= 0.158 \text{ IN}^2/\text{FT} \qquad \#3 @ 8 = 0.165 \text{ IN}^2/\text{FT}$$

$$A_{S_{TEMP}} = 0.0025\,(12)(4)$$

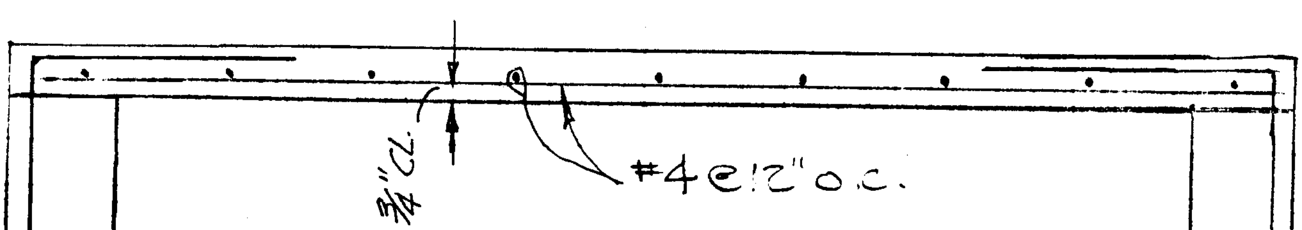
$$= 0.120 \text{ IN}^2/\text{FT} < A_S$$

$$A_{S_{TEMP\ EDGE}} = 0.004\,(12)(4)$$

$$= 0.192 \text{ IN}^2/\text{FT} > A_S$$

USE #4 @12" O.C. EA. WAY

NOTE: BECAUSE THE SLAB IS ONLY 4" THICK USE ONE LAYER OF REINF.



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HYDRAULIC DREDGING - WATER COLUMN DATA

APPENDIX E

The University of Iowa

Iowa City, Iowa 52242

Civil/Environmental Engineering
Environmental Engineering Laboratories
105 Water Plant
(319) [redacted] 335-5177

December 24, 1987

Corps of Engineers
Rock Island District
ATTN: CENCR-ED-DG (Holmes)
Clock Tower Building
P.O. Box 2004
Rock Island, Illinois 61204-2004

Dear Mr. Holmes:

Enclosed are the results of the third set of settling column analyses completed in December. Table E-1 is the data obtained using the bulk sample from location 1. This sample was loaded at a concentration of ****** , which is equivalent to 145 grams/liter dry weight. Table F-1 was obtained from the bulk sample from location 2, with a loading concentration of *********, which is equivalent to 145 grams/liter dry weight.

If you have any questions please let me know.

Sincerely,

J. Kent Johnson, Ph. D
Laboratory Director

Table E-1

SEDIMENT STUDY
DECEMBER, 1987

Reference
Boring A-87-2
Typ OH soil
Wc 83% in-situ

SAMPLE 1

TIME (HRS)	A	B	C	D	E	F	G	G Moisture Content Wc, %
0	128.3	129.1	123.2	126.0	123.0	121.4	119.9	796
.5	123.0	118.2	119.6	123.2	114.0	127.7	156.4	–
1	124.5	123.4	118.2	128.6	119.8	132.3	181.7	–
2	6.3	116.3	115.0	117.6	120.6	174.0	216.0	–
4	1.2	7.4	96.2	114.6	126.6	233.6	232.5	–
6	1.8	2.9	3.2	145.9	248.0	287.0	252.7	–
12	0.4	0.4	0.5	0.4	273.0	295.0	263.6	–
24	0.4	0.6	0.5	0.6	283.0	321.0	300.0	296
DAY								
2	0.2	0.2	0.2	0.2	0.3	334.1	318.7	–
3	0.1	0.1	0.1	0.1	0.2	364.0	374.1	–
4	0.1	0.0	0.1	0.1	0.1	355.5	422.0	–
5	0.0	0.1	0.1	0.0	0.1	225.6	~~**~~ 477	172
10	0.0	0.0	0.0	0.0	0.0	0.3	** 484	169
15	0.0	0.0	0.0	0.0	0.0	0.0	** –	–

** SAMPLES WERE TOO CONCENTRATED TO RUN SUSPENDED SOLIDS.
THE PERCENT DRY WEIGHTS ARE AS FOLLOWS:

TIME (DAY)	PORT	PERCENT DRY WEIGHT
5	G	36.8
10	G	37.2
15	G	32.5

Table E-2

SEDIMENT STUDY

DECEMBER, 1987

SAMPLE 2

Reference
Boring A-87-2
Typ CL soil
wc = 31-38 % in-situ

TIME (HRS)	A	B	C	D	E	F	G	G Moisture Content, w %
0	136.4	132.0	135.2	129.4	133.9	138.4	132.5	117
.5	133.8	132.9	127.1	124.6	126.0	139.8	141.8	–
1	116.5	128.0	124.0	136.4	130.5	130.1	140.8	–
2	122.1	124.7	121.9	129.6	126.9	133.6	135.1	–
4	104.3	112.8	117.0	126.1	126.8	129.9	201.8	–
6	0.8	117.4	122.8	124.5	127.1	131.6	247.1	–
12	0.3	0.4	116.7	117.7	120.7	203.1	264.9	–
24	0.2	0.3	5.0	15.9	216.5	241.2	297.0	299
DAY								
2	0.1	0.2	0.2	0.2	241.4	229.2	292.3	–
3	0.1	0.1	0.1	0.1	233.8	255.2	300.8	–
4	0.1	0.1	0.1	0.1	218.8	226.8	291.1	–
5	0.1	0.1	0.1	0.1	221.5	221.8	316.7	278
10	0.0	0.0	0.0	0.0	0.4	260.9	333.2	262
15	*0.0	0.0	0.0	0.0	0.0	0.0	~~**~~ 568	139

* THIS SAMPLE WAS TAKEN FROM THE SURFACE OF THE WATER COLUMN.

** THE SAMPLE WAS TOO CONCENTRATED TO RUN SUSPENDED SOLIDS. THE PERCENT DRY WEIGHT OF THE SAMPLE IS 42.0.

MECHANICAL AND ELECTRICAL CONSIDERATIONS

A P P E N D I X F

UPPER MISSISSIPPI RIVER SYSTEM
ENVIRONMENTAL MANAGEMENT PROGRAM
DEFINITE PROJECT REPORT (R-4)

ANDALUSIA REFUGE REHABILITATION AND ENHANCEMENT

POOL 16
MISSISSIPPI RIVER MILE 462 to 463
Rock Island County, Illinois

APPENDIX F
MECHANICAL AND ELECTRICAL CONSIDERATIONS

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List of Plates

UPPER MISSISSIPPI RIVER SYSTEM
ENVIRONMENTAL MANAGEMENT PROGRAM
DEFINITE PROJECT REPORT (R-4)

ANDALUSIA REFUGE REHABILITATION AND ENHANCEMENT

POOL 16
MISSISSIPPI RIVER MILE 462 to 463
Rock Island County, Illinois

APPENDIX F
MECHANICAL AND ELECTRICAL CONSIDERATIONS

Purpose and Scope. The purpose of this appendix is to present preliminary design for the pumping station development at the Andalusia Refuge. Pump manufacturers' engineering information for standard catalog units were used to develop the design presented in this appendix. Pump sizing and layout are based on the efficient operation of the station and ease of normal maintenance.

General. One pumping station containing two submersible propeller type pumps is proposed for the Andalusia Refuge. The pumping station will serve a dual function; discharging interior drainage from the protected area to maintain constant water surface elevation during the drawdown cycle; to discharge river water into the protected refuge during the waterfowl migration seasons for the purpose of creating as large a surface area as possible.

The pumping station will be located on the downstream end of the moist soil unit protected from the main channel of the river and associated debris. The pumping station will be constructed integral with the levee river toe section. The levee fill will be placed, allowed to naturally consolidate for approximately three months and then excavated for the pumping station.

Pump units are sized to complete the drawdown period within a two week period. Pump operation will utilize automatic controls for setting and maintaining water elevation within the moist soil unit. The power and control panels will be housed within the pumping station super-structure and will be protected from condensation damage with unit heaters.

Pump and motor removal can be accomplished through secured sealed manhole accesses exterior of the pump station super-structure. Hand-cleaned trash racks are provided at both intake and discharge ends for maximum protection of the pump impellers against debris. The superstructure will have gravity ventilators and louvers for air circulation. Design of the station is based on the Hydraulic Institute Standards, 13th Edition, 1975, and on applicable sections of EM 1110-2-3102, 03, and 05.

Station Features. This station is fed by a new 48-inch reinforced concrete pipe from the moist soil unit passing through the levee section and by a pump forebay section from the Mississippi River. A sump divider wall separates the two pumps up to elevation 551.0. A slide gate in the divider wall permits gravity flow between the moist soil unit and the Mississippi River. Stoplog slots will be provided at each end to facilitate sump dewatering for maintenance purposes. Gate closure of the gravity outlet occurs for water management operation, at which time the required pump is energized manually, with further control being automatic through the float system. One 24-inch, 5,000 gpm submersible pump of axial or mixed flow type will be utilized for pumping from the moist soil unit and one 24 inch 3500 gpm submersible pump of axial or mixed flow type will be utilized for pumping from the Mississippi River. Discharge of both pumps will be piped over the sump divider wall into a stilling basin that directs flow by gravity out to the river or moist soil unit respectively. Access to the sump area will be by ladder through removable floor hatches at the operating floor level. System head computations and curves and example pump selections are shown on plates F-1 through F-11. The estimated operating energy cost of $1200 per year is computed on Plate F-18.

Control Sequence. The sluice gate of the pump station should be operated in an open position except during periods of moist soil unit management by Illinois Department of Conservation personnel. During desired drawdown periods, the sluice gate should be closed and the pump station activated for drawdown purposes. The pump station must be manually activated but will automatically turn off at low water level of 542.0. The float control system will automatically turn the pump on at elevation 542.5 to maintain the 542.0 drawdown elevation.

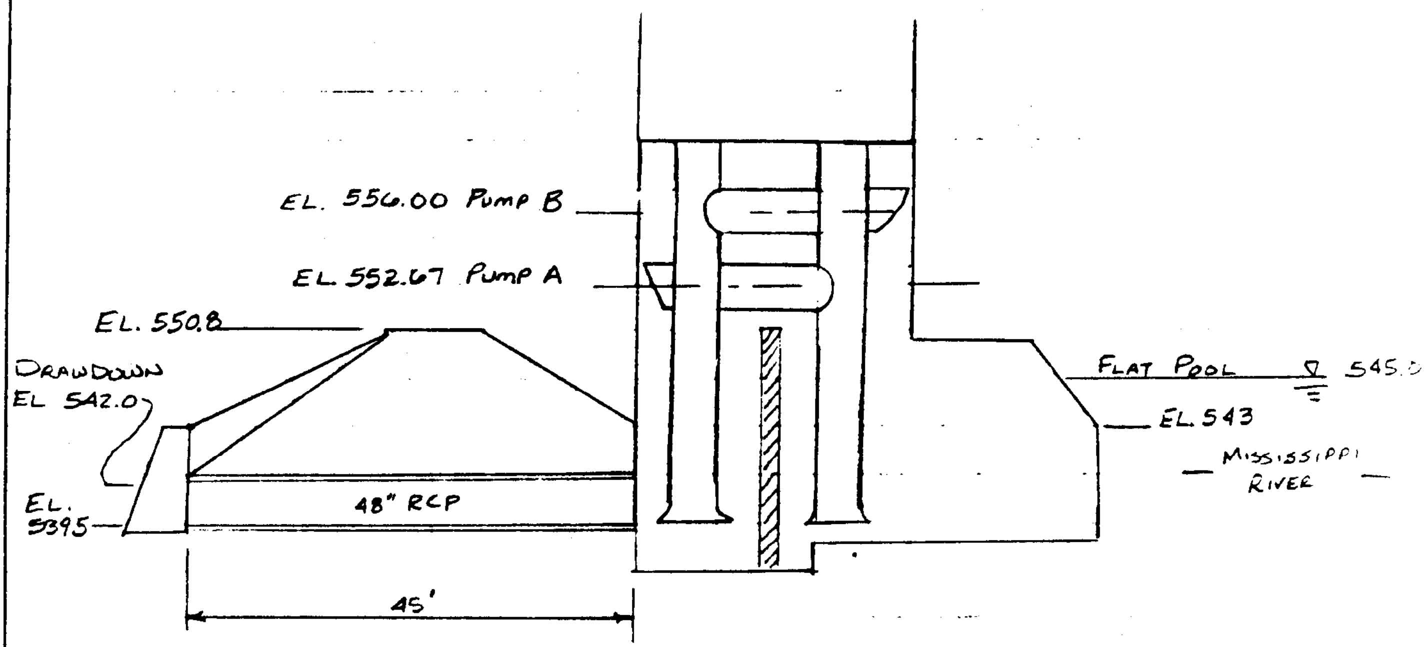
When it is desired to pump from the river into the moist soil unit, the station must be manually activated and will continue pumping automatically until elevation 547 (which can be adjustable to elevation 550.8, the elevation of the levee overflow). Pumping will be at a slower rate of approximately 3500 gpm to permit a slow filling of the moist soil unit.

Electrical. The two submersible pumps at the station will be operated with electric motors. Power will be provided by the Iowa-Illinois Gas and Electric Company of Davenport, Iowa. Iowa-Illinois Gas and Electric Company has fossil and nuclear generating plants, is interconnected with many other utilities, and is considered to be a very reliable source of power. Two high voltage power systems are available within the area, 13 KV, 3 phase and 7.6 KV single phase. The nearest 13.2 KV, 3 phase connection point is at the switchgear in Illinois City, Illinois, thereby requiring construction of 3-4 miles of new power line. The 7.6 KV, single phase line can be tapped within one-half mile of the site bringing to conclusion that power to the pump station be tapped from the 7.6 KV line, transformed down with a 37.5 KVA transformer to 480 volt and converted to 3-phase, 480 volts using a power phase converter located at the pump station location. The high voltage line will span the ± 150 feet of levee from high ground to the east wall of the pump station. The transformer and power phase converter will be mounted on the pumping station roof. Local ownership of the power service will be on the low voltage side of the transformer near the pump station. The Government, through its contractor, will pay for connection charges including powerline, transformers and power converters, and the Iowa-Illinois Gas and Electric Company will own and maintain the high voltage service.

The pumping station will have pump motor loads of approximately 8 KW and 18 KW and motors of 10 hp and 25 hp, one motor of about 3 hp to operate the sluice gate, and a circuit for one motor of about 3/4 hp for the sump pump. A power control panel will be located within the pump superstructure, will house a 480/240/120 volt transformer for lighting, receptacle and the control circuitry.

Short circuit analysis for the station is shown on plates F-12 through F-17. Electrical schematics are shown on plate 22.

Subject		Date
ANDALUSIA REFUGE PUMP STATION		JULY 88
Computed by WGH	Checked by DJH	Sheet 1 of



GENERAL CONDITIONS

1. PUMP STATION IS INTEGRAL PART OF RIVERSIDE LEVEE TOE

2. PUMP A PUMPS FROM RIVER TO REFUGE WITH FREE PIPE END DISCHARGE

3. PUMP B PUMPS FROM REFUGE TO RIVER WITH FREE PIPE END DISCHARGE

4. REQUIRED PUMPING RATES
   PUMP A = 3500 gpm max
   PUMP B = 5000 gpm min.

5. FLAT POOL EL. 545.0
   REFUGE MAX POND EL = 550.8
   REFUGE MIN POND EL. = 542.0
   SUMP FLOOR EL. = 539.5

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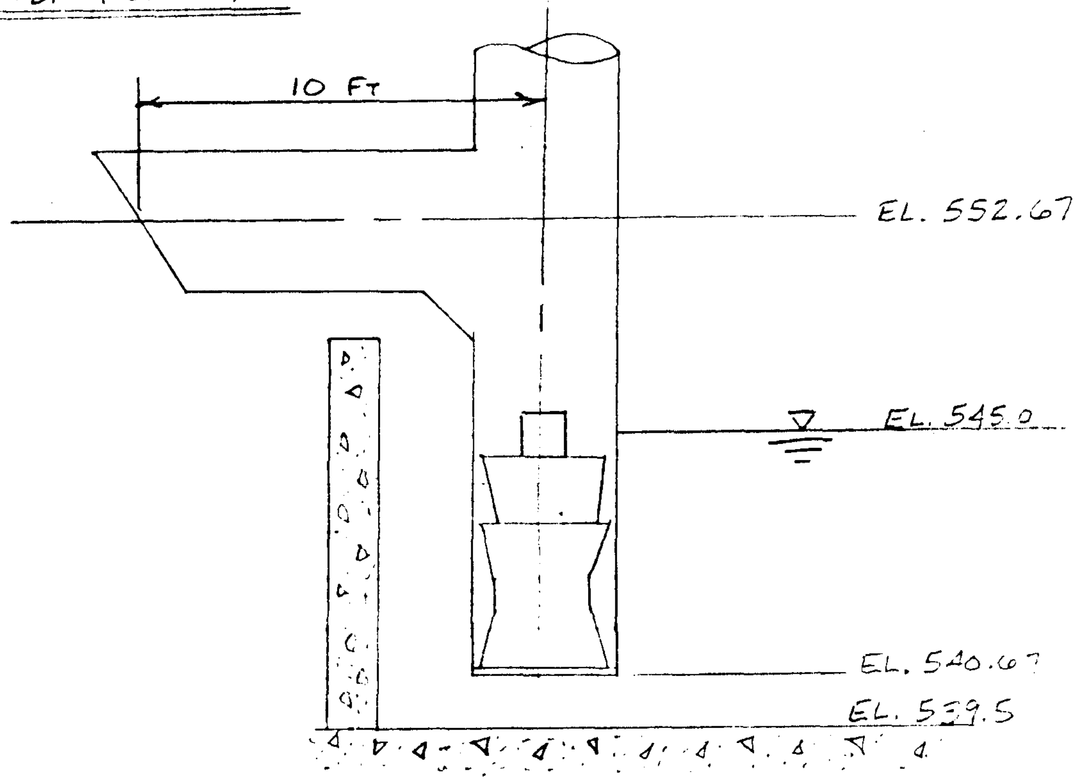
Subject ANDALUSIA REFUGE PUMP STATION		Date JULY 83
Computed by WGH	Checked by DJH	Sheet 2 of

DESIGN FOR PUMP A



PARAMETERS:

$Q_{MAX} = 3500$ gpm

$H_{STATIC} = 552.67 - 545.0 = 7.67$ FT

SUBMERSIBLE PUMP WITH 26" Φ STEEL PIPING

COMPUTE SYSTEM HEAD LOSSES:

PIPE HEAD LOSSES ARE INCLUDED INTO PUMP CURVES UP TO 20 inches ABOVE UNIT. ASSUME UNIT HEIGHT EQUAL 50 INCHES.

Subject: ANDALUSIA REFUGE PUMP STATION		Date: JULY 83
Computed by: WGH	Checked by: DJH	Sheet: 3 of

$$\text{PIPE FLOW VELOCITY} = \left(3500 \frac{\text{GAL}}{\text{MIN}}\right)\left(\frac{\text{FT}^3}{7.48\ \text{GAL}}\right)\left(\frac{4}{\pi(2.167\ \text{FT})^2}\right)\left(\frac{\text{MIN}}{60\ \text{SEC}}\right)$$

$$= 2.115\ \text{FT/SEC}$$

$$\text{HEAD}_{\text{VEL.}} = \frac{V^2}{2g} = \frac{(2.115\ \text{FT/S})^2}{2(32.2\ \text{FT/S}^2)} = 0.069\ \text{FT.}$$

SYSTEM COMPONENTS:

- PIPE LOSS ~ 0.3 H_V/100 FT
- PIPE LENGTH = $(552.67 - 540.67) - \frac{70\ \text{IN}}{12\ \text{IN/FT}} + 10\ \text{FT} = 16.17\ \text{F}$
- ONE ELBOW ~ 0.33 H_V
- EXIT LOSS ~ 1.0 H_V
- TRASHRACK LOSS ~ 0.1 H_V
- INLET SUMP LOSS ~ 0.1 H_V

$$\text{TOTAL LOSS} = 0.069\left(0.3\left(\frac{16.17}{100}\right) + 0.33 + 1.0 + 0.1 + 0.1\right)$$
$$= 0.109\ \text{FT.}$$

TOTAL DYNAMIC HEAD = 7.67 FT + 0.109 FT = 7.78 FT.

PUMP SELECTION:

FLYGT - 7050, 20 KW, 700 RPM, 4 BLADE,
5° BLADE ANGLE, CURVE 63-700E4

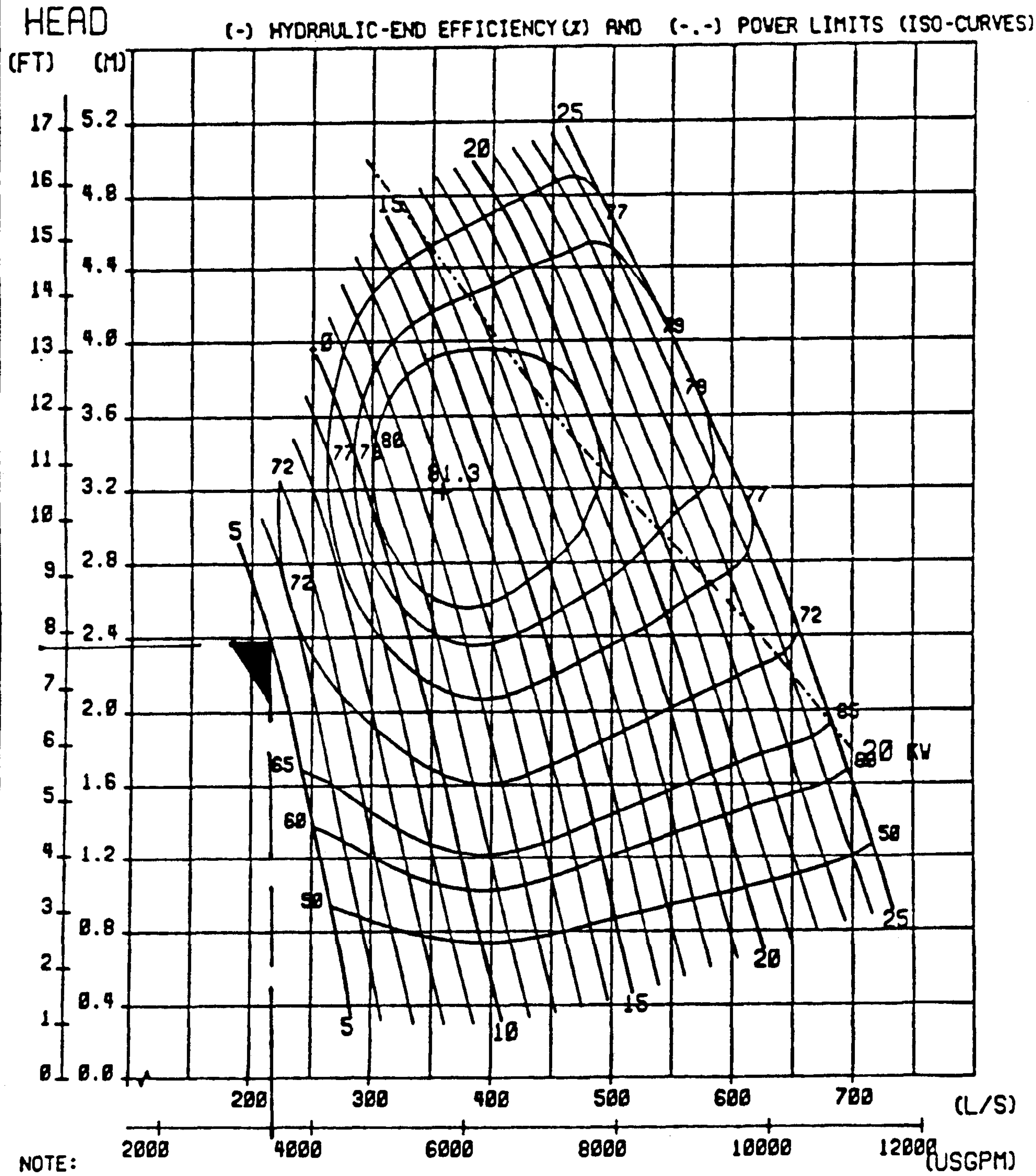
Q ≐ 3500 GPM @ 7.8 FT WITH e_{ff} ≐ 70%

CHECK MAXIMUM RECOMMENDED PUMP SPEED IN ACCORDANCE WITH HYDRAULIC INSTITUTE STANDARDS & FIG 60

$$N = \frac{N_S H^{3/4}}{\sqrt{Q}} = \frac{20{,}000\,(8)^{0.75}}{\sqrt{3500}}$$

N = 1608 RPM > 700 RPM O.K.

PERFORMANCE CURVE	PROD. 7050
FREQ 60 HZ / NOMINAL HYDRAULIC-END SPEED 700 RPM	CURVE NO 63-700B4

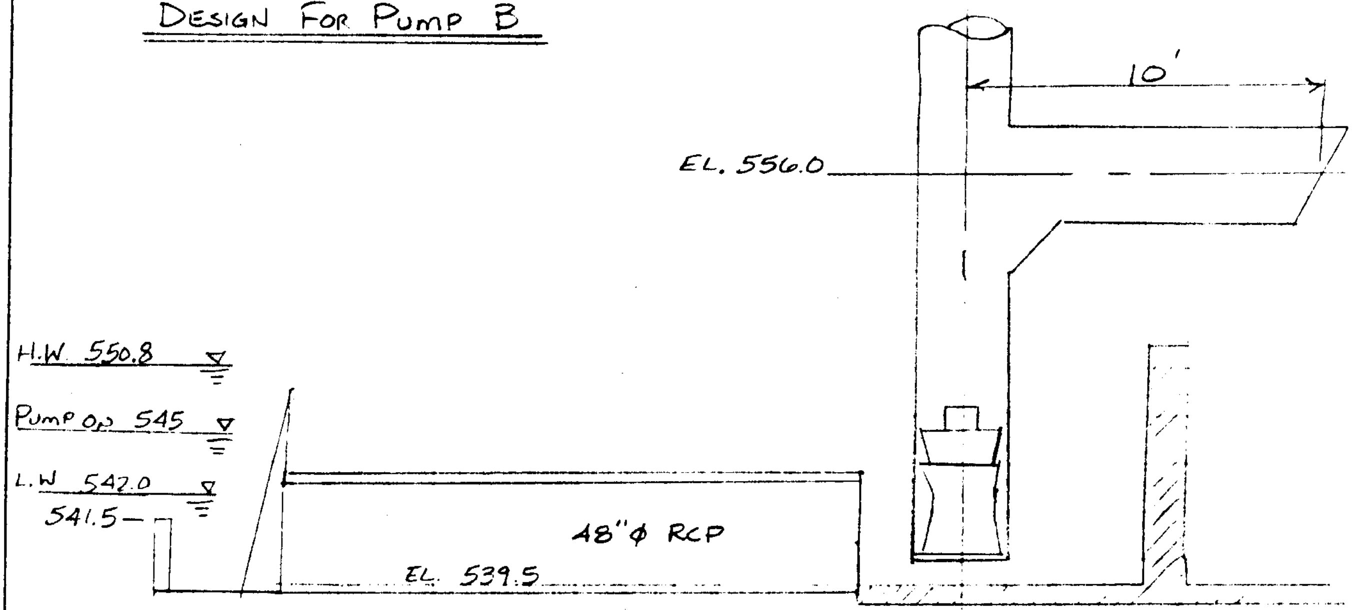


NOTE:
CURVES ARE BASED ON NOMINAL CONSTANT HYDRAULIC-END SPEED AND SHOW PERFORMANCE WITH CLEAR WATER.
ALL HYDRAULIC LOSSES UP TO 500 MM. ABOVE THE PUMP/MOTOR TOP ARE INCLUDED

PLATE F-4

Subject ANDALUSIA REFUGE PUMP STATION		Date JULY 83
Computed by WGH	Checked by DJH	Sheet 5 of

DESIGN FOR PUMP B



PARAMETERS:

Q_{MIN} = 5,000 GPM = 11.14 cfs

$H_{STATIC\ MIN}$ = 556.0 - 545.0 = 11.0 FT.

$H_{STATIC\ MAX}$ = 556.0 - 542.0 = 14.0 FT.

SUBMERSIBLE PUMP WITH 26" ϕ STEEL PIPING

COMPUTE SYSTEM HEAD LOSSES:

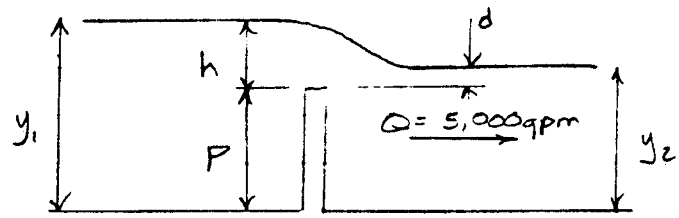
LOSSES TO BE CONSIDERED

a) WEIR LOSSES
b) TRASHRACK LOSSES
c) RCP FRICTION LOSSES
d) INLET SUMP LOSSES
e) PUMP PIPE LOSSES
f) PUMP ELBOW LOSSES
g) EXIT LOSS

Subject: ANDALUSIA REFUGE PUMP STATION | Date: JULY 88

Computed by: WGH | Checked by: DJH | Sheet 6 of

a) WEIR LOSSES



CASE 1: WATER ELEVATION = 545 y_1 = 5.5 FT.

WEIR HEIGHT P = 1.75 FT.

CREST HEIGHT = h_1 = 3.75 FT

USING EM 1110-2-5027, 30 SEPT 87 pg. 4-21

$$h = \left[0.3\ \frac{Q}{L}\right]^{2/3}$$

solving For Q_1

$$Q_1 = (3.75)^{1.5}\left(\frac{14\ FT}{0.3}\right) = 338.9\ cfs = 152{,}092\ gpm$$

USING HANDBOOK OF APPLIED HYDRAULICS, 3rd ed, DAVIS & SORENSEN

$$\frac{Q}{Q_1} = \left[1 - \left(\frac{d}{h}\right)^{1.5}\right]^{0.385}$$

SOLVING FOR d

$$d = \left[1 - \left(\frac{5{,}000}{152{,}092}\right)^{1/.385}\right]^{2/3} 3.75 = 3.749$$

$$h_L = h - d = 3.75 - 3.749 = 0.001\ FT.$$

Subject: ANDALUSIA REFUGE PUMP STATION		Date: JULY 83
Computed by: WGH	Checked by: DJH	Sheet 7 of

CASE 2: WATER ELEVATION = 542

WEIR HEIGHT = $P = 1.75$ FT.

CREST HEIGHT = $h = 0.75$

$$Q = (0.75)^{1.5}\left(\frac{14\text{ FT}}{0.3}\right) = 30.31 \text{ cfs} = 13,603 \text{ gpm}$$

$$d = \left[1 - \left(\frac{5,000}{13,603}\right)^{1/.385}\right]^{2/3} 0.75 = 0.712 \text{ FT}$$

$$h_L = h - d = 0.75 - 0.712 = \underline{0.038 \text{ FT.}}$$

b. TRASHRACK LOSSES

CONTROLING CASE IS WHEN REFUGE = EL. 542.0

ELEVATION ENTERING TRASHRACK = 542 - 0.25 = EL. 541.75

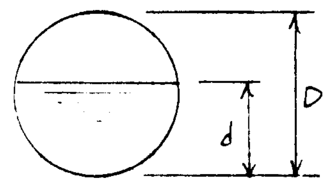
$$V_{T.R} = \frac{11.14 \text{ FT}^3/\text{s}}{(6 \text{ FT})(541.75 - 539.5 \text{ FT})} = 0.825 \text{ FT/s}$$

$$\frac{V^2}{2g} = \frac{(0.825 \text{ FT/s})^2}{64.4 \text{ FT/s}^2} = 0.011 \text{ FT}$$

$$H_{L_{T.R}} \doteq 0.1 \frac{V^2}{2g} = 0.001 \text{ FT.}$$

Subject: ANDALUSIA REFUGE PUMP STATION		Date: JULY 88
Computed by: WGH	Checked by: DJH	Sheet 8 of

C. REINFORCED FRICTION LOSSES



D = 4 FT
d ≐ 541.75 - 539.5 = 2.25 F
Q = 5,000 gpm
L = 45 FT.

Check for critical depth   REF. EM1110-2-1602, PLATE II

FOR $Q = 11.14$ cfs , $D = 4$ Ft

$y_c/D \doteq 0.345$

$y_c = 0.345(4') = 1.38$ FT.

SINCE $y_c < d$, SUBCRITICAL FLOW   O.K.

Check AREA   REF. - U.S. DEPT OF INTERIOR BUREAU OF RECLAMATION - HYDRAULIC & EXCAVA- TABLES

AT y_c

FOR $\frac{d}{D} = \frac{2.25}{4} = 0.563$

$Q = 1.7505(4)^{2.5} = 56.018$ cfs

$h_v = 0.2296(4) = 0.9184$ FT $\therefore$ $V = 7.6906$ fps

AND $A = \frac{Q}{V} = \frac{56.018}{7.6906} = 7.28$ FT^2

COMPUTE VELOCITY ACTUAL

$V = \frac{Q}{A} = \frac{11.14}{7.28} = 1.53$ fps

WETTED PERIMETER = $R\left[\pi + \frac{2}{180}\left(\sin^{-1}\frac{.25}{2}\right)\pi\right] = 1.0798\,\pi R$

Subject: ANDALUSIA REFUGE PUMP STATION		Date: JULY 88
Computed by: WGH	Checked by: DJH	Sheet 9 of

COMPUTE HYDRAULIC RADIUS

$$R = \frac{\text{AREA}}{\text{W. PERIMETER}}$$

$$R = \frac{7.28\ \text{FT}^2}{1.0798\pi(2\ \text{FT})} = 1.073\ \text{FT}$$

USING MANNING EQUATION

$n = 0.013$ FOR CONCRETE PIPE

$$H_L = S = \left(\frac{nV}{1.49 R^{2/3}}\right)^2 (45\ \text{FT})$$

$$H_{L pipe} = \left[\frac{0.013(1.53\ \text{fps})}{1.49(1.073\ \text{FT})^{2/3}}\right]^2 (45\ \text{FT})$$

$$H_{L pipe} = 0.007\ \text{FT.}$$

d, e, f, g) COMPUTE PUMP ASSOCIATED LOSSES

$$\text{PUMP PIPE VELOCITY} = 11.14\ \frac{\text{FT}}{\text{S}}\left(\frac{4}{\pi(2.167\ \text{FT})^2}\right) = 3.02\ \text{FT/S}$$

$$H_{VEL} = \frac{V^2}{2g} = \frac{(3.02)^2}{2(32.2)} = 0.142\ \text{FT.}$$

PIPE LOSS ~ $0.3\ H_V/100$ FT
ELBOW ~ $0.33\ H_V$
EXIT ~ $1.0\ H_V$
INLET SUMP ~ $0.2\ H_V$

Subject: ANDALUSIA REFUGE PUMP STATION		Date: JULY 8?
Computed by: WGH	Checked by: DJH	Sheet 10 of

$$\text{PIPE LENGTH} = 556 - 540.67 - \frac{70}{12} + 10\text{ FT} = 19.50\text{ FT}$$

$$H_{L.pump} = 0.142\left(0.3\left(\frac{19.5}{100}\right) + 0.33 + 1.0 + 0.2\right)$$

$$H_{L\,pump} = 0.225\text{ FT}$$

COMPUTE SYSTEM LOSS

$$H_{L\,SYST.} = H_{L\,weir} + H_{L\,T.R.} + H_{L\,pipe} + H_{L\,pump}$$

$$H_{L\,SYST.} = 0.038\text{ FT} + 0.001\text{ FT} + 0.007\text{ FT} + 0.225\text{ FT}$$

$$H_{L\,SYSTEM} = 0.272\text{ FT.}$$

$$\text{PUMP TDH} = 14 + 0.272 = 14.272\text{ FT.}$$

PUMP SELECTION:

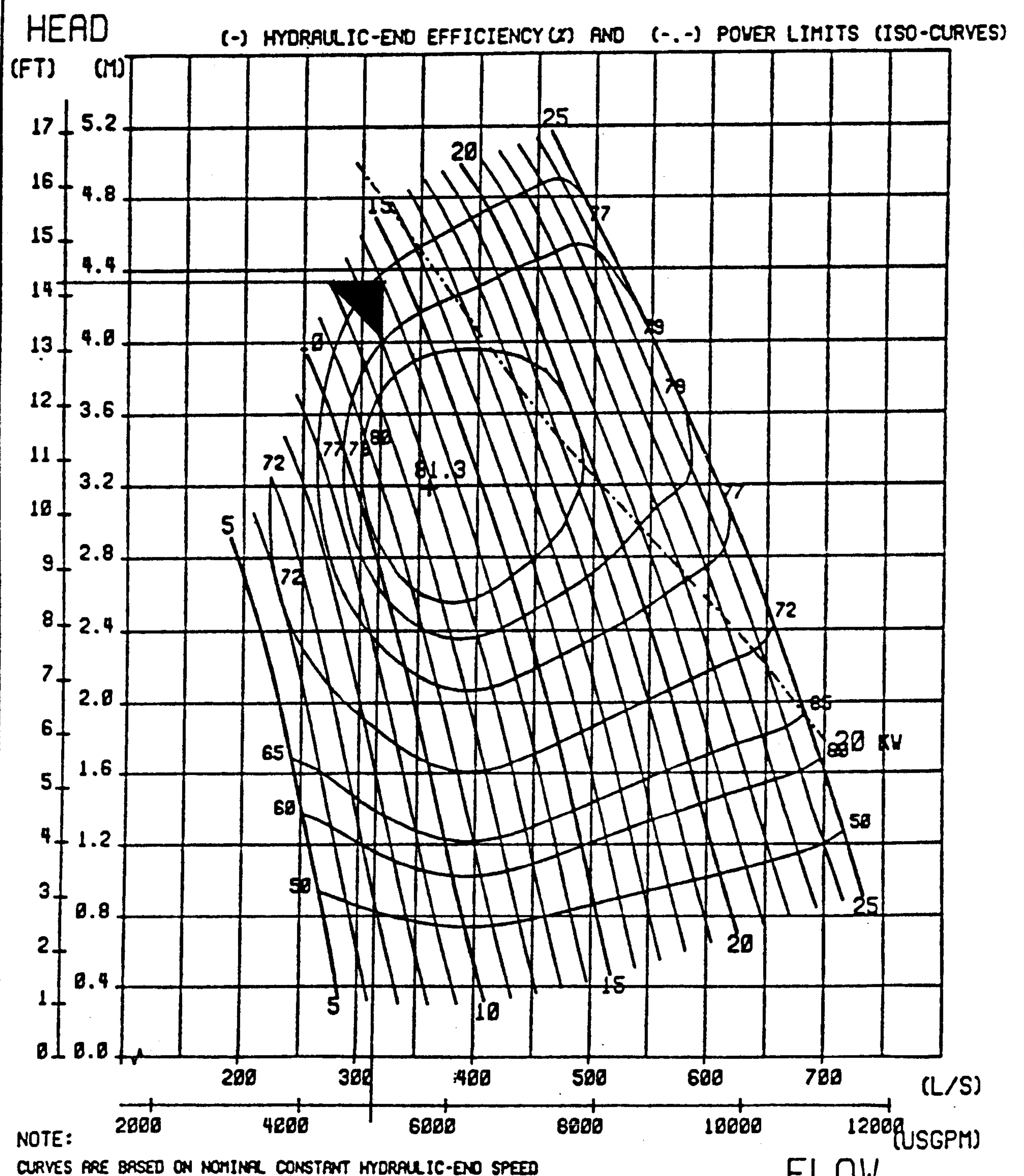
FLYGT - 7050, 20 KW, 700 RPM, 4 BLADE,
13° BLADE ANGLE, CURVE 63-70084
Q = 5,000 gpm @ 14.2 FT. @ 77% eff.

CHECK MAXIMUM RECOMMENDED PUMP SPEED

$$N = \frac{N_S H^{3/4}}{\sqrt{Q}} = \frac{15{,}000\,(14.3\text{FT})^{0.75}}{\sqrt{5{,}000}}$$

N = 1,560 RPM > 700 RPM O.K.

PERFORMANCE CURVE	PROD. 7050
FREQ 60 HZ / NOMINAL HYDRAULIC-END SPEED 700 RPM	CURVE NO 63-700B4



NOTE:
CURVES ARE BASED ON NOMINAL CONSTANT HYDRAULIC-END SPEED
AND SHOW PERFORMANCE WITH CLEAR WATER.
ALL HYDRAULIC LOSSES UP TO 500 MM. ABOVE THE PUMP/MOTOR TOP ARE INCLUDED

PLATE F-11

Subject		Date
ANDALUSIA REFUGE PUMP STATION		JULY 88
Computed by WGH	Checked by DJH	Sheet 12 of

LOAD STUDY

EXISTING POWER SYSTEM - HIGH VOLTAGE 7620 VOLT, 1ϕ
POWER STATION SYSTEM - SECONDARY VOLTAGE 480/277/ 3ϕ

SUBMERSIBLE PUMP LOAD REQUIREMENTS

PUMP A = 7.4 KW = 10 H.P.
PUMP B = 17.4 KW = 25 H.P.

ASSUME ONLY ONE WILL BE PUMPING AT ANY GIVEN TIME

$$I_L = \frac{17.4(1000)}{480\sqrt{3}(.95)} = 22.0 \text{ AMP}$$

GIVEN A 25 HP MOTOR ON THE PUMP

I_L = 34 AMP NEC TABLE 430-150

SIZE BRANCH CIRCUIT CONDUCTOR FOR PUMP B NEC TABLE 310-16

I_{FL} = (34)(1.25) = 42.5 AMP SELECT # 8 I = 50

SIZE BRANCH CIRCUIT CONDUCTOR FOR PUMP A (10 H.P.)

I_{FL} = (14 A)(1.25) = 17.5 amp SELECT # 12 I = 25

POWER REQUIREMENT FOR PUMP B

(.746)(25 HP)(1.25) = 23.3 KVA

NCR Form 381b
1 Aug 80

Subject: ANDALUSIA REFUGE PUMP STATION		Date: JULY 83
Computed by: WGH	Checked by: DJH	Sheet 13 of

SUMP PUMP LOAD REQUIREMENTS

3/4 H.P. CLASS G, 240 V, 1ϕ, 60 Hz, 1800 RPM

$$I_L = \frac{(0.75\ HP)(1000)}{(240\ V)(0.65)} = 4.8\ A$$

USING NEC TABLE 430-148 $I_L = 6.9\ A$

SIZE BRANCH CIRCUIT CONDUCTOR FOR SUMP PUMP

$I_{FL} = (6.9\ A)(1.25) = 8.65\ A$ SELECT # 12 AWG

POWER REQUIREMENT FOR SUMP PUMP

$(746)(0.75\ HP)(1.25) = 0.70$ KVA

SLUICE GATE LOAD REQUIREMENTS

ASSUME

3 H.P. CLASS 5, 3ϕ, 480 V, 60 Hz, 1200 RPM

$$I_L = \frac{(3\ HP)(1000)}{(480)\sqrt{3}\,(.75)} = 4.8\ A$$

USING NEC TABLE 430-150 $I_L = 4.8\ A$

SIZE BRANCH CIRCUIT CONDUCTOR FOR SLUICE GATE

$I_{FL} = (4.8\ A)(1.25) = 6.0\ A$ SELECT # 12 AWG

POWER REQUIREMENT

$(746)(3\ HP)(1.25) = 2.8$ KVA

Subject		Date
ANDALUSIA REFUGE PUMP STATION		JULY 93
Computed by WGH	Checked by DJH	Sheet 14 of

DETERMINE TRANSFORMER SIZE

$LOAD_{MAX} = 23.3\ KVA + 0.7\ KVA + 2.8\ KVA = 26.8\ KVA$

CHOOSE A 37.5 KVA, 1ϕ

DETERMINE SIZE OF MAIN BREAKER OF FUSE

$I = 2.5(34\ AMP) + \frac{4.8\ A}{2\sqrt{3}} + 4.8\ A$

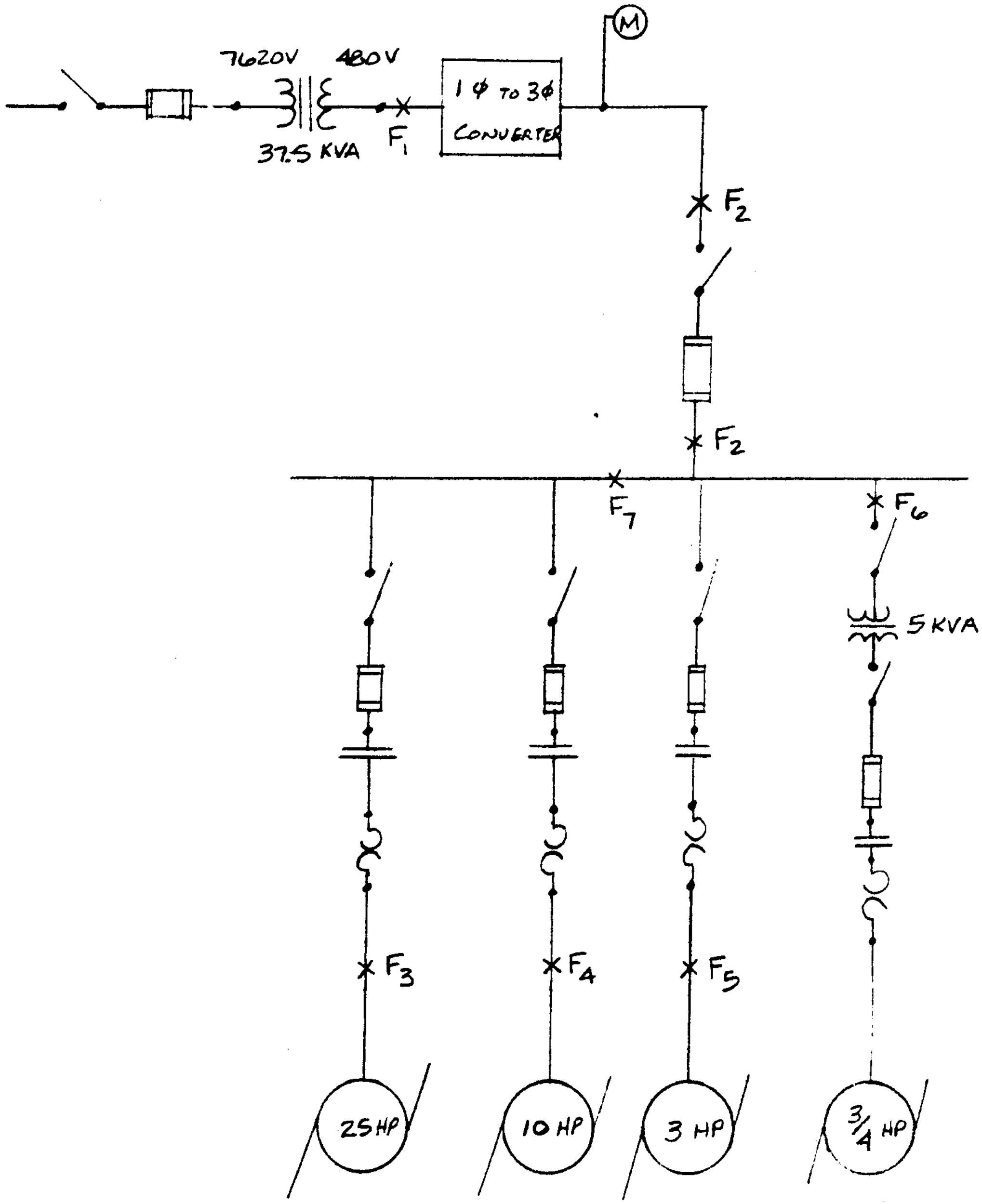
$I = 92\ AMP$

CHOOSE 150 A BUS
100 A BREAKER
3 - #3 AWG MAIN FEEDER WITH #6 AWG GRD.

NCR Form 381b
1 Aug 80

Subject ANDALUSIA REFUGE PUMP STATION		Date JULY 85
Computed by WGH	Checked by DJH	Sheet 15 of

FAULT STUDY



Subject		Date
ANDALUSIA REFUGE PUMP STATION		JULY 89
Computed by WGH	Checked by DJH	Sheet 16 of

SYSTEM BASE - 500 MVA

PUMP B FEEDER - 3-#8 AWG

$C = 1230$

$R = 0.778$ ohm / FT $\times 10^3$

PUMP A FEEDER - 3 # 12 AWG

$C = 617$

$R = 1.98$ ohm / FT $\times 10^3$

SUMP & SLUICE FEEDER - 3 # 12 AWG

$C = 617$

$R = 1.98$ ohm / FT $\times 10^3$

BUS FEEDER - 3 # 3 AWG

$C = 3830$

$R = 0.245$ ohm / FT $\times 10^3$

Z TRANSFORMER = 2 %

AT F_1

$$I_{sc} = \frac{(37.5\ KVA)(1000)}{480\ (0.02)} = 3906 \text{ AMP}$$ AT TRANSFORMER SECONDARY

AT F_2

$$f = \frac{1.73\ L\ I}{C\ V} = \frac{1.73(50)(3906\ A)}{3830(480)} = 0.1838$$

$$I_{F_2} = 3906 \left(\frac{1}{1 + 0.1838}\right) = 3300 \text{ AMP}$$

AT F_3

$$f = \frac{1.73(20)(3300\ A)}{(480)(1230)} = 0.1934$$

$$I_{F_3} = 3300 \left(\frac{1}{1 + 0.1934}\right) = 2765 \text{ AMP}$$

NCR Form 381b
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Subject		Date
ANDALUSIA REFUGE PUMP STATION		JULY 88
Computed by WGH	Checked by DJH	Sheet 17 of

AT F_4 $f = \frac{1.73(20)(3300A)}{(480V)(617)} = 0.3855$

$I_{F_4} = 3300\left(\frac{1}{1+0.3855}\right) = 2382$ AMP

AT F_5 $f = \frac{1.73(20)(3300A)}{(480V)(617)} = 0.3855$

$I_{F_5} = 3300\left(\frac{1}{1+0.3855}\right) = 2382$ AMP

AT F_6 $f = \frac{2(10FT)(3300)}{480(617)} = 0.2229$

$I_{F_6} = 3300\left(\frac{1}{1+0.2229}\right) = 2700$ AMP

CONCLUSIONS: ALL EQUIPMENT SHALL BE RATED FOR 10,000 AMP RMS

NCR Form 381b
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Subject		Date
ANDALUSIA REFUGE PUMP STATION		JULY 88
Computed by WGH	Checked by DJH	Sheet 18 of

PUMP STATION OPERATING ENERGY COST

DRAINAGE AREA = 925 ACRE

STATION CAPACITY PUMP A = 3500 GPM ≑ 15.75 ACRE FT/DAY

STATION CAPACITY PUMP B = 5,000 GPM ≑ 22.5 ACRE FT/DAY

TIME PERIOD	OPERATION	PUMP	INITIAL VOL. ACRE-FT	RAINFALL (IN)	RUNOFF C=.5 (ACRE FT)	EVAPOR. (ACRE-FT)	EVENT VOL. (ACRE-FT)	TOTAL (A-FT)
JUNE	DRAWDOWN	B	42	4.32	166.5	- 16.8	2.1 *	193.8
JULY	DRAWDOWN	B	0	4.88	188.1	- 17.5	2.1 *	172.7
AUG	DRAWDOWN	B	0	3.76	144.9	- 14.9	2.1 *	132.1
							SUBTOTAL	498.6
SEPT	FILL	A	180	3.74	- 288.3	38.5		0
OCT	FILL	A	0	2.70	- 208.0	38.5		0
NOV	FILL	A	0	2.16	- 166.5	38.5		0
							SUBTOTAL	0

* VOLUME IS BASED ON DURATION OF EVENT CAUSING OVERFLOWING EL. 550.0 REQUIRING A SECOND DRAWDOWN − (5%)(42) = 2.1 ACRE FT.

$$\text{PUMP B RUN TIME} = \left(498.6\ \tfrac{\text{ACRE-FT}}{\text{YR}}\right)\left(\tfrac{\text{DAY}}{22.5\ \text{ACRE-FT}}\right) = 22.16\ \tfrac{\text{DAY}}{\text{YR}} = 532\ \tfrac{\text{HR}}{\text{YR}}$$

$$\text{PUMP A RUN TIME} = \left(0\ \tfrac{\text{ACRE-FT}}{\text{YR}}\right)\left(\tfrac{\text{DAY}}{15.75\ \text{ACRE-FT}}\right) = 0\ \tfrac{\text{DAY}}{\text{YR}} = 0\ \tfrac{\text{HR}}{\text{YR}}$$

POWER REQUIREMENT − ASSUME 5 HR RUN TIME FOR MAINTENANCE & TESTING / YR

$$P_{Pump} = (17.4\ \text{KW})(532 + 5\ \tfrac{\text{HR}}{\text{YR}}) + (7.4\ \text{KW})(0 + 5\ \tfrac{\text{HR}}{\text{YR}}) = 9{,}378\ \text{KWH/YR}$$

$$P_{\text{heaters \& CONTROLS}} = (0.3\ \text{KW})(24\ \tfrac{\text{HR}}{\text{DAY}})(365\ \tfrac{\text{DAY}}{\text{YR}}) = 2{,}628\ \text{KWH/YR}$$

$$\text{AVERAGE OPERATING COST} = \left(9{,}378\ \tfrac{\text{KWH}}{\text{YR}} + 2{,}628\ \tfrac{\text{KWH}}{\text{YR}}\right)(\$0.093/\text{KWH})$$

OPERATING COST = \$1,117.00/YR. SAY \$1200/YR

NCR Form 381b
1 Aug 80

SEDIMENTATION STUDY

A P P E N D I X G

UPPER MISSISSIPPI RIVER SYSTEM
ENVIRONMENTAL MANAGEMENT PROGRAM
DEFINITE PROJECT REPORT (R-4)

ANDALUSIA REFUGE REHABILITATION AND ENHANCEMENT
POOL 16 MISSISSIPPI RIVER MILES 462 TO 463
ROCK ISLAND COUNTY, ILLINOIS

APPENDIX G
SEDIMENTATION STUDY

A sedimentation study was conducted to evaluate sedimentation in Dead Slough and in the Refuge area during the period 1936 through 1987. The scope of this study, as presented in this appendix, consisted of determining net erosion from 1936 (pre-lock and dam) through 1987, estimating annual adjacent watershed erosion/deposition, evaluating estimated river source sedimentation, and evaluating proposed project impacts on sedimentation.

Baseline elevations were established from 1936 plane table topographic maps. Additional sections were taken by survey crews during 1987. Eleven ranges were used to construct cross sections of this area. Elevations in 1936 were compared with present elevations in 1987 to show net changes in elevation. Table G-1 provides a summary of net sedimentation.

The two predominant sedimentation sources are the Mississippi River and upland erosion. Adjacent watersheds were studied to estimate approximate soil loss from these areas. Estimates were derived from the Universal Soil Loss Equation, reference: Predicting Rainfall Erosion Losses, USDA, Handbook Number 537, December 1978. Estimated adjacent watershed erosion is presented in table G-2.

TABLE G-1

Andalusia Refuge and Dead Slough Total Sedimentation

		Total Sediment Deposition in Andalusia Refuge & Dead Slough 1936-1987		Dead Slough Sediment Deposition, 1936-1987 Below Elevation 545.0 (Flat Pool)	
Range	Station	Average Depth, Ft	Average Annual Depth, In/Yr	Average Depth, Ft	Annual Average In/Yr
A	15+15	2.1	0.49	2.9	0.68
B	23+75	2.3	0.54	3.8	0.90
C	30+75	2.5	0.58	2.7	0.63
D	38+25	1.7	0.40	3.9	0.91
E	45+75	1.7	0.40	4.2	1.00
F	2+75.15CE	2.2	0.53	-	-
G	4+55.62CE	2.3	0.54	-	-
H	8+88.12CE	2.4	0.56	-	-
I	12+70CE	2.5	0.58	-	-
J	16+16.26CE	2.2	0.52	-	-
K	20+75CE	2.2	0.52	-	-
OVERALL		2.2	0.52		0.82 in/yr

TABLE G-2

Estimated Sediment from Adjacent Watersheds

		Watershed Gross Erosion			Delivery Ratio 1/	Sediment Yield 2/	
Watershed	Area, Ac	T/Ac/Yr	T/Yr	Ac-Ft/Yr		T/Yr	Ac-Ft/Yr
A	519	13	6,800	3.9	.55	3,700	2.2
B	774	16	12,000	6.9	.55	6,600	3.8
C	208	10	2,100	1.2	.65	1,400	0.8
D	1,152	13	15,000	8.6	.48	7,200	4.2
Total	2,653	-	35,900	20.6	-	18,900	11.0

1 Reference: "Sediment Delivery Ratios vs. Drainage Area," USDA, Soil Conservation Service, drawing number 5, N-30,509, dated October 1970.

2 Sediment Yield = Gross Erosion x Delivery Ratio. Sediment yield is the portion of the gross watershed erosion that actually reaches the watershed mouth.

Net river sedimentation was estimated by subtracting the adjacent watershed sedimentation (table G-2) from the net sedimentation (table G-1). Results of this comparison and potential reductions in sedimentation due to proposed project features are presented in table G-3.

TABLE G-3

Comparison of River Versus Upland Erosion Sedimentation

Sedimentation Source	Existing Conditions Ac-Ft/Yr	Existing Conditions %	Sediment Reduction into Andalusia Refuge Due to Proposed Project Ac-Ft/Yr	Sediment Reduction into Andalusia Refuge Due to Proposed Project %
Adjacent Watershed	11.0	64.7	4.2	24.7
River	6.0	35.3	0.0	0.0
Net	17.0	100.0	4.2	24.7

ILLINOIS DEPARTMENT OF CONSERVATION
FISHERIES INVESTIGATION OF DEAD SLOUGH

A P P E N D I X H

MISSISSIPPI RIVER INVESTIGATIONS

County Rock Island
Location from nearest town 7 mi. West of Andalusia
Date May 16, 1988

Station Sampled Dead Slough/Andalusia Refuge Pool No. or Name 16

Stations Location in River Miles 462.4-463.0

UMRCC Habitat Classification: Tailwater ____ Lake X Pond ____
Main channel ____ Main channel border ____ Side channel ____ Slough X

Time Temp. Rec. 1:40 AM (PM) Temperature: Air ____ °F, Water Surface 13 c °F

Water Color Clear Sky(weather) Cloudy Wind W-15mph

Turbidity: Secchi To bottom = 1.5' : Cause: ____

Chemistry: pH 8.0 Alkalinity 137 Other Cond:580 DO: 11.2

Water Level: Low X Normal ____ High ____ Flood ____

Velocity(ft./sec.) 0 Max. Depth 18" Avg. Depth 12"
Max. Width 100 yds. Min. Width 100 yds. Avg. Width 100 yds.

Bottom types(%): Silt(muck) X Sand ____ Gravel ____ Rubble ____ Boulders ____ Bedrock ____

Type of Shoreline(%): Gravel bar ____ Sandbar ____ Mud flat ____ Rocky ____
Steep Mud bank 100 Other ____

Aquatic Vegetation(% & type): 95% coverage coontail, curlyleaf potamogeton, duckweed,

Fish Habitat available: Brush X Logs X Stumps X Rock dikes ____
Pile dikes ____ Gravel ____ Rip rap ____ Aquatic vegetation ____
Other ____

Recent Angler Success No Access

Fish Population Analysis: Bag Seine Hauls (Size ____, No. ____);
Hoop Net (No. ____ Size ____ Hrs. Set ____); Trap Net (No. ____ Size ____ Hrs. Set ____);
Gill Net (No. ____ Size ____ Hrs. Set ____); Trammel Net(No. ____ Size ____ Hrs. Set ____);
Electro-Fishing (Time 1:30 AM (PM) 45 minutes, effeciency Poor
Toxicant (area treated ____ acres); Other ____

Fish condition: Some fin rot and lerneae, generally poor condition

Fish Diseases: ____

Number of species collected 10 Observed but not collected ____

Pollution ____

Fisherman usage No Access

Time on Job: 4.0 hrs. Personnel Involved Dan Sallee, Ed Walsh
Reported by: Dan Sallee Date of Report 5/16/88

Additional Remarks and Map on Reverse Side:

ILLINOIS DEPARTMENT OF CONSERVATION
DIVISION OF FISHERIES

FISH POPULATION ANALYSIS

(Condition factor & Length-Frequency Summary

COUNTY Rock Island

WATER (NAME) Miss.,Dead Sl/Andalusia Ref.

DATE OF COLLECTION 5/16/88

Species	½" Group	Number	Percent of Total	Weight	Condition Factor	Rating
Shortnose gar	553	1	100%	540		
Bowfin	663	1	100%	2300		
Gizzard shad	274	1	12.5	230		
	281	1	12.5	250		
	282	1	12.5	250		
	286	1	12.5	220		
	312	1	12.5	370		
	322	1	12.5	300		
	336	1	12.5	480		
	377	1	12.5	520		
		8	100%			
Central mudminnow		1	(Preserved)			
Carp	114	1	12.5	30		
	157	1	12.5	90		
	171	1	12.5	90		
	173	1	12.5	90		
	177	1	12.5	90		
	233	1	12.5	250		
	477	1	12.5	1180		
	616	1	12.5	2850		
		8	100%			

Sampling Time Involved: 45 min. Method of Collection: Cartop electofishing

Biologist: Dan Sallee Date of Report: 5/16/88

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All Others — District Office Only.

F.M. 12.0 (8-70)

ILLINOIS DEPARTMENT OF CONSERVATION
DIVISION OF FISHERIES

FISH POPULATION ANALYSIS

(Condition factor & Length-Frequency Summary

COUNTY Rock Island

WATER (NAME) Miss., Dead Sl/Andalusia Ref

DATE OF COLLECTION 5/16/88

Species	½" Group	Number	Percent of Total	Weight	Condition Factor	Rating
Golden shiner	67	1	16.6	- -		
	73	1	16.6	- -		
	88	1	16.6	- -		
	89	1	16.6	- -		
	92	1	16.6	- -		
	95	1	16.6	- -		
		6	100%			
Smallmouth buffalo	132	1	16.6	30		
	138	1	16.6	40		
	145	1	16.6	50		
	147	2	33.3	100		
	165	1	16.6	70		
		6	100%			
Bluegill	65	1	2.5	- -		
	73	1	2.5	- -		
	77	1	2.5	- -		
	78	1	2.5	- -		
	82	1	2.5	- -		
	84	1	2.5	- -		
	93	1	2.5	- -		
	96	1	2.5	20		
	97	1	2.5	20		
	101	1	2.5	20		
	103	1	2.5	20		
	105	1	2.5	20		
	106	1	2.5	25		

(continued on next page)

S‑‑ling Time Involved: 45 min. Method of Collection: Cartop electrofishing

Biologist: Dan Sallee Date of Report: 5/16/88

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ILLINOIS DEPARTMENT OF CONSERVATION
DIVISION OF FISHERIES

FISH POPULATION ANALYSIS

(Condition factor & Length-Frequency Summary

COUNTY Rock Island

WATER (NAME) Miss., Dead Sl/Andalusia ?f.

DATE OF COLLECTION 5/16/88

Species	½" Group	Number	Percent of Total	Weight	Condition Factor	Rating
Bluegill (continued)	107	1	2.5	25		
	108	2	5.0	60		
	109	1	2.5	30		
	115	1	2.5	40		
	118	1	2.5	40		
	126	1	2.5	40		
	134	2	5.0	120		
	137	1	2.5	80		
	148	1	2.5	90		
	152	1	2.5	100		
	153	3	7.5	285		
	155	1	2.5	100		
	156	1	2.5	100		
	158	1	2.5	100		
	160	1	2.5	100		
	166	2	5.0	230		
	167	1	2.5	130		
	172	2	5.0	310		
	178	2	5.0	310		
	212	1	2.5	260		
		40	100%			
Largemouth bass	170	1	50.0	60		
	251	1	50.0	280		
		2	100%			

Sampling Time Involved: 45 min. Method of Collection: Cartop electrofishing

Biologist: Dan Sallee Date of Report: 5/16/88

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ILLINOIS DEPARTMENT OF CONSERVATION
DIVISION OF FISHERIES

FISH POPULATION ANALYSIS

(Condition factor & Length-Frequency Summary

COUNTY Rock Island

WATER (NAME) Miss., Dead Sl/Andalusia Ref.

DATE OF COLLECTION 5/16/88

Species	½" Group	Number	Percent of Total	Weight	Condition Factor	Rating
Black crappie	144	1	20.0	40		
	233	1	20.0	200		
	243	1	20.0	240		
	255	1	20.0	250		
	267	1	20.0	300		
		5	100%			

S ing Time Involved: 45 min. Method of Collection: Cartop electrofishing

Biologist: Dan Sallee Date of Report: 5/16/88

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WATERFOWL OBSERVATION DATA
FOR ANDALUSIA REFUGE

A
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The following waterfowl observation data originates from annual U.S. Fish and Wildlife Service aerial census counts made along the Upper Mississippi River. The records for Andalusia refuge are incomplete since during some counts the refuge area was not separated from other locations in Pool 16. Previous to 1987, Dead Slough was not included in the count since it was not part of the refuge until 1987.

DATE	MALL-ARD	BLK	PINT	BWT	GWT	WIG	GAD	SHOV	SCAUP	RN	CANS	RH	RUDDY	GLD EYE	BH	COM MER	RB MER	H MER	CG	Sm CG	GT CG	BSG	COOT
10-27-87	400	20	80		50	200	80	75	50										40				650
11-2-87	525	15	20		50	80	30	100	200	75	30			5					50				600
11-9-87	400	25				50	30		200	40				30		10			50				200
11-17-87	475	25	60			100			125	40			40	20	35								75
11-23-87	350	15							250	40	30			125	30	40			100				
12-02-87	225	25							325	75	25		75	175	60	50		20	80				
12-16-87	200	15												350		125		15	125				

AERIAL CENSUS DATA — ANDALUSIA REFUGE (INCLUDES DEAD SLOUGH)

DATE	MALLARD	BLK	PINT	BWT	GWT	WIG	GAD	SHOV	SCAUP	RN	CANS	RH	RUDDY	GLD. EYE	BH	COM MER	RB MER	H MER	CG	SM CG	GT. CG	BSG	COOT	BALD EAGLE
1-5-84	0																							
2-20-84	30													50		15								2
3-5-84	175	15												75		20			25					
3-21-84	500	40							225	75				80		40			50					3
4-11-84		NO	CENSUS																					
9-4-84	40			60																				
9-11-84	125			200	50	50																		
10-29-84	175	10			125	80																	350	
11-5-84	200	10				60			30														150	1
11-12-84	200	20				60																	75	1
11-20-84	80	5							60	30				40		15								2
11-26-84	125	10												15										2
12-3-84	125	15												30		10								2
12-10-84	40																							2

AERIAL CENSUS DATA - ANDALUSIA REFUGE

DATE	MALLARD	BLK	PINT	BWT	GWT	WIG	GAD	SHOV	SCAUP	RN	CANS	RH	RUDDY	GLD EYE	BH	COM MER	RB MER	H MER	CG	Sm CG	Gt CG	BSG	COOT	
10-17-83	35																							
10-25-83	150	10							40														60	
11-1-83	NO CENSUS - HEAVY RAIN																							
11-8-83	75	10			30				30														25	
11-14-83	200	10				75			50														25	1
11-21-83	100	5																						1
11-29-83	80	10																						1
12-8-83	25													10										1

AERIAL CENSUS DATA - ANDALUSIA REFUGE

DATE	MALLARD	BLK	PINT	BWT	GWT	WIG	GAD	SHOV	SCAUP	RN	CANS	RH	RUDDY	GLD. EYE	BH	COM MER	RB MER	H MER	CG	Sm CG	GT. CG	BSG	COOT	BALD EAGLE
1-3-83	60													25		5								1
3-1-83	225	10							60	30				75		20								2
3-7-83	275	20							350	175	60		40	5					50					4
3-14-83	600	30				50			650	325	80	40		100		30			50				75	
3-22-83	325	20							275	150	50	25		75		30								2
3-28-83	800	30				175			650	275	175	75		40	35	25							100	2
4-4-83	400	30				30			650	275	80	150		25	20	10							500	2
4-11-83	250	10							900	375	150	50	30	125		40							750	1
4-25-83	60					25			125	50			15	30		10							250	
9-6-83	25			60																				
9-13-83	50			80	30																			
9-19-83	80			50																			25	
9-26-83	60			30																			20	
10-3-83	80		25																				30	
10-11-83	35																						30	

AERIAL CENSUS DATA — ANDALUSIA REFUGE

DATE	MALL-ARD	BLK	PINT	BWT	GWT	WIG	GAD	SHOV	SCAUP	RN	CANS	RH	RUDDY	GLD. EYE	BH	COM MER	RB MER	H MER	CG	Sm CG	Cgt CG	BSG	COOT	BALD EAGLE
9-14-82	20			65																			50	
9-21-82	60					25																	40	
9-27-82	50			30		25																	60	
10-4-82	40					15																	40	
10-11-82	75		25			50																	100	
10-18-82	175					80																	75	1
10-26-82	175	5				80		35															75	
11-4-82	NO	CENSUS		DUE	TO	FOG																		
11-8-82	225	5	40			80																	100	
11-15-82	80	25																						1
12-6-82	80								30					10		10								1
12-13-82	200	20		MISS. RIVER ABOVE FLOOD STAGE										75		25								2

AERIAL CENSUS DATE — ANDALUSIA REFUGE