FEASIBILITY REPORT
TOPEKA, KANSAS, LOCAL PROTECTION PROJECT

APPENDIX A

ENGINEERING

APRIL 2008

DEPARTMENT OF THE ARMY
Kansas City District, U.S. Army Corps of Engineers
Kansas City, Missouri

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FEASIBILITY REPORT
Topeka, Kansas, Local Protection Project

APPENDIX A
ENGINEERING

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A-1 GENERAL

A-1.1 INTRODUCTION

The purpose of the Engineering Appendix is to document engineering efforts completed during the Topeka, Kansas, Local Protection Project Feasibility Study development.

The focus of the engineering effort during the feasibility study is on understanding existing conditions, associated data collection and inventories, framing the nature of problems, developing potential solutions to those problems, refining solutions in light of evaluation criteria, and offering the final engineering necessary to support a plan (or plans) within the planning process.

The engineering for this study was developed to the level of detail sufficient to prepare a feasibility baseline cost estimate(s), general project schedule, and support the recommended plan. The results of engineering investigations, studies, and feasibility level designs (hereinafter normally termed "design") are presented in this engineering appendix to the feasibility report. The location and vicinity map of the project is shown on Plate A-1.1.

This engineering appendix supports the Feasibility Report which is aimed at examining potential improvements to increase the existing project performance consistent with the original authorization. This engineering appendix (similar to the main report) focuses on four of the six levee units that compose the Topeka system: Waterworks, South Topeka, Oakland, and North Topeka. The Auburndale and Soldier Creek Units were determined to meet the authorized level of protection assuming continued adequate operations and maintenance efforts.

A-1.2 PROJECT LOCATION AND LIMITS

The existing project extends along approximately 10 miles of the Kansas River as it passes through Topeka, and includes levees on two tributaries, Soldier Creek and Shunganunga Creek. The six units of the flood protection system were designed and constructed in conjunction with each other, but are independently operated to some extent. The total protected area covers about 32 square miles and is characterized by industrial, commercial, and residential development.

A-1.3 ENGINEERING EFFORTS

A Corps of Engineers (COE) reconnaissance level report was completed in September, 1997. The Reconnaissance Report identified a Federal interest in further investigations. That recommendation led to the current Feasibility Study. An early effort under feasibility was development of the Review of Existing Local Flood Protection Project Report prepared and submitted to the COE by HDR Engineering, Inc. in January, 2000 (the HDR Report). The general purpose was to review and document available historic and current design information and condition of the structural features of each unit.

The HDR Report was incorporated into work on existing conditions analysis of each unit in the system. Additionally, information was gathered (where available) from the original design documents, Operation and Maintenance (O&M) manuals, and associated studies. The Corps utilized current hydrology/hydraulics models, and geotechnical/structural risk and uncertainty (R&U) study methods to develop the engineering portions of the existing conditions (baseline) analysis of the existing project. Much of this analysis was based on data and observations from recent high water events (since the original project design), especially those in 1993. This new engineering analysis, along with the economic (HEC-FDA) analysis, established a complete R&U approach to estimating existing conditions flood damages. The engineering and economic evaluations taken together with a summary baseline environmental review and an HTRW review of the study area formed the full picture of existing conditions. A review of existing conditions results by the study team provided guidance during the scoping and development of future conditions (with and without project). This Engineering Appendix to the Feasibility Report identifies those areas.

The engineering risk and uncertainty analysis is summarized below. Details and calculations supporting the results appear within the various chapters of the engineering appendix.

Geotechnical and Structural engineers determined the most likely expected modes and sites of failure prior to overtopping in each Unit. A full range of conditional probabilities of failure versus river stage elevation encompassing the Probable Failure Point (PFP) and Probably Non-Failure Point (PNP) were determined by geotechnical and structural engineer PDT members for each site/mode of failure in each Unit. The geotechnical probabilities of failure were developed based on procedures identified in ETL 1110-2-556, "Risk-Based analyses for Geotechnical Engineering for Support of Planning Studies", except that the acceptable factor of safety identified in the ETL was modified to a more realistic factor of safety based on Kansas City District 1993 flood observations and historical experience. To produce the structural probability of failure versus river stage curve, critical sections of each structure were analyzed (stability and strength factors of safety determined) using material strengths and soil properties. Next, the soil and material parameters were varied to plus and minus one standard deviation from the mean, one at a time, and the factor of safety was recomputed. A Taylor series expansion was used to compute a probability of failure.

The areas of interest are as follows:

Waterworks Floodwall. Findings for structural risk have led the PDT to undertake evaluations which are aimed at increasing the unit's overall level of performance. This portion of the study examined methods for reduction of structural stability risk.

South Topeka Levee. Findings for geotechnical risk have led the PDT to undertake evaluation of measures to better control underseepage in a reach of the South Topeka levee. The recommended solution is construction of a landside underseepage berm.

South Topeka Floodwall. Findings for structural risk have led the PDT to undertake evaluation of strengthening and/or replacement measures for this floodwall. The South Topeka floodwall is a pile-founded wall with steel sheet pile to provide protection from underseepage. The wall is approximately 1900 ft. long. The wall was constructed in 1938 and original design and construction parameters are not available. The timber piles may be inadequate to support the floodwall under some conditions. The recommended solution is removal and replacement of the existing wall.

South Topeka – Kansas Avenue Pump Station. Findings for structural risk have led the PDT to undertake evaluation of strengthening the foundation of the station to increase its strength bearing capacity. The recommended solution is interior reinforcement of the foundation wall through the installation of a wall stiffener.

Oakland Levee. Findings for geotechnical risk have led the PDT to undertake evaluation of measures to better control underseepage in a reach of the Oakland levee adjacent to the Oakland Wastewater Treatment Plant. The recommended solution is construction of a landside underseepage berm.

Oakland Unit – Shunganunga Floodwall. Findings for structural risk have led the PDT to undertake evaluations which are aimed at increasing structural reliability of the floodwall reach of the Oakland Unit along Shunganunga Creek.

Oakland Unit – East Oakland Pump Station. Findings for structural risk have led the PDT to undertake evaluation of measures to better control uplift at the Station. The recommended solution is construction of a heel extension.

North Topeka Levee. Findings for geotechnical risk indicate the need for measures to improve underseepage control in two areas lying along the left (north) bank of the Kansas River. The recommended solution for the first area is the construction of a landside underseepage berm. The recommended solution of the second area is construction of a series of pressure relief wells with a header discharging to a manhole and provision for temporary pumping to effectively draw down the pressures in this area.

North Topeka Unit – Fairchild Pump Station. Findings for structural risk led the PDT to evaluation measures to better control uplift at the station. The recommended plan is removal of the station.

A-1.4 SELECTED PLAN

The selected plan is the National Economic Development Plan (NED) that maximizes the net benefits while providing a favorable benefit to cost ratio. The NED plan was developed for each of the four units containing the areas of interest and the combination of these individual NED plans is considered the overall system NED plan.

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

Chapter A-2

HYDROLOGY and HYDRAULICS ANALYSIS

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Topeka, Kansas
Flood Risk Management Feasibility Study
Appendix A – Engineering
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A-2 HYDROLOGIC AND HYDRAULIC ANALYSES

A-2.1 KANSAS RIVER

A-2.1.1 INTRODUCTION

As part of the feasibility study, a hydraulic investigation was conducted on the Kansas River using the HEC-RAS computer software developed by the Hydrologic Engineering Center, U.S. Army Corps of Engineers. The program was used to calculate water surface profiles on the reach of the Kansas River that runs through Topeka, Kansas. The study covers approximately river miles 73 through 96.5 of the Kansas River. A backwater model of this reach was developed using 1997 field surveys and 1995 aerial contour maps, and was calibrated using high water marks from the 1993 Flood. The levee units that protect Topeka along the Kansas River are: North Topeka Unit, Water Works Unit, Auburndale Unit, South Topeka Unit, and Oakland Unit. A general location map can be found with the plates at the end of the main report.

A-2.1.2 PURPOSE

The purpose of this investigation is to develop Kansas River water surface profiles through the City of Topeka reflecting the base (or existing) conditions. The resulting hydraulic model will be used to evaluate a series of alternatives for improving the integrity of the existing flood control system.

A-2.1.3 HYDROLOGY

In March 2002, the Corps of Engineers completed the Kansas River Hydrology[1] study with special attention to the Kansas River near Topeka. This study used a similar procedure as the Upper Mississippi River System Flow Frequency Study[2] (UMRFFS), which is a complex evaluation of the regulated and unregulated flows on the Mississippi, lower Illinois, and Missouri Rivers. UMRFFS has already been published, and the Kansas River Hydrology study has been subject to a full independent technical review. The Kansas River Hydrology study utilized the regulated and unregulated flow data developed in UMRFFS for the Kansas River basin to determine the discharge-frequency relationships at the Kansas River gages. By combining these results, regionalization equations were developed relating drainage area and discharge for different frequency events. These equations were used to determine the discharges on the Kansas River. The results from the Kansas River Hydrology study near Topeka are shown in Table 1-1.

[1] "Kansas River Hydrology with Special Attention To: Kansas River Hydrology near Topeka, KS." U.S. Army Engineer District, Kansas City, March 2002.
[2] "Upper Mississippi River System Flow Frequency Study, Appendix E, Hydrology." U.S. Army Engineer District, Kansas City, pending publication.

Table 1-1 Flow Frequency Data as developed in Kansas River Hydrology 2002

Percent Chance of Exceedance	At Topeka Gage (cfs)	Downstream of Soldier Creek (cfs)	Downstream of Shunganunga Creek (cfs)
0.2	348,000	348,000	348,000
0.5	268,000	270,000	271,000
1	217,000	220,000	221,000
2	173,000	176,000	177,000
5	123,000	126,000	126,000
10	93,600	96,600	97,200
20	67,200	69,600	70,200
50	36,600	38,100	38,500
Drainage Area (sq mi)	56,720 sq. mi.	57,024 sq. mi.	57,094 sq. mi.

Since flood events above the 0.2% chance exceedance (500 year) event need to be considered in this study, the discharge-frequency curves were extended up to the 0.04% chance exceedance (2500 year) event. To accomplish this, a straight-line extrapolation was used on a log-probability plot of the discharge frequency events at the Topeka gage. As in the 0.2% event, the extreme floods do not vary downstream of the Soldier and Shunganunga confluence. Plate A2-1-1 at the end of this chapter shows the discharge-frequency curve for the Kansas River at the Topeka gage. Table 1-2 summarizes all of the discharges used on the Kansas River for the existing conditions model.

Table 1-2 Summary of Flood Discharges Used in this Study

Percent Chance of Exceedance	Return Interval (yr)	At Topeka Gage (cfs)	Downstream of Soldier Creek (cfs)	Downstream of Shunganunga Creek (cfs)
0.04	2500	500,000	500,000	500,000
0.1	1000	410,000	410,000	410,000
0.133	750	387,000	387,000	387,000
0.2	500	348,000	348,000	348,000
0.5	200	268,000	270,000	271,000
1	100	217,000	220,000	221,000
2	50	173,000	176,000	177,000
10	10	93,600	96,600	97,200

A-2.1.4 Hydrologic Uncertainty

In the past, the Corps of Engineers used freeboard as a factor of safety in designing levees to account for uncertainties in discharge, stage, and other engineering parameters (such as geotechnical and structural). Now, the Corps of Engineers has adopted a new methodology called Risk Based Analysis (RBA) for formulating flood risk management projects. This method considers all of the same engineering parameters, but accounts for the uncertainties directly in the analysis in lieu of using freeboard. Using RBA, the project performance will be expressed as the average return period in years of the largest flood that can be accommodated by the plan under study, with a conditional non-exceedance probability of 90%. The concept of freeboard is no longer used.

To use RBA, the hydrologic uncertainty must be characterized. This information is entered into the computer program HEC-FDA (Flood Damage Analysis), which uses Monte Carlo algorithms to quantify the uncertainties. The uncertainty bands used in this program are based on the effective record lengths used to develop the flow frequency estimates. According to Table 4-5 in EM 1110-2-1619, for a regional study the effective record length is taken as the average length of records used. According to the Kansas River Hydrology Study, the length of record used at Wamego, Lecompton, and Desoto was 77 years; at Topeka the record length was 95 years. This averages to 82 years, which was considered the effective record length.

HEC-FDA calculates the uncertainty either analytically or graphically. For an analytical computation the log Pearson Type III statistics are inputted directly. A graphical approach is used on regulated streams, when the stream gage records are small or incomplete, or when partial duration data is used. For the Kansas River, the discharge-probability curve was defined graphically. HEC-FDA uses the procedures outlined in ETL 1110-2-537 “Uncertainty Estimates for Nonanalytic Frequency Curves” to calculate the error limit curves using order statistics. This is related as standard deviation of the discharge estimate. To produce realistic estimates of the uncertainty curves, high probability flood events needed to be estimated. Using the graphical plot features in HEC-FDA, the values were adjusted to obtain a reasonably shaped curve. The full range of discharges was then entered into HEC-FDA under the graphical curve option. Table 1-3 shows hydrologic uncertainty results on the Kansas River near the Topeka gage. For the HEC-FDA analysis, an arbitrary index point was selected for each levee unit to calculate the damage-probability curve. Since the index point on each levee is located upstream of the Soldier Creek confluence, only the discharge uncertainty in the reach near the gage was calculated.

Table 1-3 Hydrologic Uncertainty on Kansas River near Topeka Gage

Exceedance Probability	Discharge (cfs)	Confidence Limit Curves (standard error) Discharge (cfs)			
		-2 SD	-1 SD	+1 SD	+2 SD
0.999	6000	4130	4980	7230	8710
0.99	8880	6490	7590	10,390	12,160
0.95	12,980	9990	11,380	14,790	16,860
0.9	16,070	12,810	14,350	18,000	20,160
0.8	21,060	17,310	19,090	23,230	25,620
0.7	25,800	21,380	23,490	28,340	31,140
0.5	36,600	30,280	33,290	40,240	44,230
0.3	53,450	43,420	48,170	59,290	65,780
0.2	67,200	53,360	59,880	75,420	84,640
0.1	93,600	70,260	81,100	108,030	124,690
0.04	134,350	92,550	111,510	161,870	195,030
0.02	173,000	113,700	140,250	213,400	263,240
0.01	217,000	136,420	172,060	273,680	345,170
0.004	286,250	170,120	220,670	371,310	481,640
0.002	348,000	198,590	262,880	460,680	609,830
0.001	417,980	229,470	309,700	564,120	761,360

A-2.1.5 HYDRAULICS

The hydraulic analysis for this report centered on the development of the HEC-RAS computer model for the study reach of the Kansas River at Topeka, Kansas. For this analysis, version 3.0.1 of the HEC-RAS (River Analysis System) developed by the Hydrologic Engineering Center was used. The computer model was calibrated to the 1993 flood using known water surface elevations (high-water marks) and discharge. Once the model was calibrated, a series of steady flow water surface profiles were created based on flood discharges in Table 1-2 above.

Original Design Water Surface Elevations

The elevation of the crown of the existing levee was determined by selecting a design water surface elevation and then adding freeboard to account for uncertainties. Freeboard for all levee units in the Topeka system on the Kansas River was three feet except at the Waterworks Levee Unit, which ranged from 2.2 to 2.8 feet. The design water surface elevations were determined by using a backwater computer model with the design discharges. The original design discharges for the Topeka levee system assumed the discharge from Soldier Creek was 50,000 cfs while the discharge above Soldier Creek on the Kansas River was 314,000 cfs. The combined flow downstream of the confluence was 364,000 cfs. The resulting top of protection was approximately equal to the 50% non-exceedance probability for the 0.2%-chance (500-yr) flood.

Geometric Data

The computer model required cross section geometry along the length of the study reach. The information used to create the cross-section geometry was obtained from two sources. The U.S. Army Corps of Engineers provided 1997 cross-section surveys of the channel that covered the entire length of the study (RM 73 – 96.5). The City of Topeka provided a surveyed levee-top profile of the North Topeka Unit, and two and four foot contours, from 1995 aerial mapping, within the Topeka city limits. Top of Levee elevations were also obtained from a 2004 COE survey for the Waterworks, Auburndale, Oakland, and North Topeka Units. Outside of the city limits, the overbanks were modeled using United States Geological Survey (U.S.G.S.) 7.5 minute quadrangle maps. In order for the model to more accurately compute friction losses, additional cross sections were interpolated between surveyed cross sections and then modified based on aerial photographs and on-site inspection.

Based on field investigations and review of aerial photography, appropriate Manning's “n” coefficients were selected for each cross section. Values from 0.020 to 0.035 were selected for the channel throughout the entire study reach. Overbank “n” values ranged from 0.040 for well maintained grassy areas to 0.15 for heavily treed areas with dense undergrowth.

Bridge data was obtained from engineering drawings provided by: Kansas Department of Transportation, City of Topeka, Shawnee County, the U.S. Army Corps of Engineers, and the Burlington Northern Santa Fe Railroad. The operational drawings of the Chicago Rock Island Railroad Bridge, located at RM 84.64, detail emergency procedures to raise the bridge eleven feet when a flood event of a certain magnitude is forecast. Since the procedures are in place and the mechanisms tested regularly, this bridge was modeled in the "up" position.

There is a weir in the channel near the waterworks that was not surveyed. Since it is inconsequential during the larger events, it was not included in the model.

Starting Water Surface Elevation

The starting water surface elevations for all discharges are from a rating curve developed from water surface elevation/discharge relationships at the starting point of the study reach (near the confluence of Whetstone Creek). These relationships were taken from the Shawnee County Kansas Flood Insurance Study (Revised May 17, 1993).

Calibration

The model was calibrated using high-water marks that were set during the 1993 flood. The discharge used for these high-water marks was 170,000 cubic feet per second (cfs) and was obtained from U.S.G.S. Peak Flow Data (Water Year 1993) for the gage on the right bank at the downstream side of Sardou Bridge (RM 83.1, U.S.G.S. Station Number 6889000). The 170,000 cfs was used from the beginning cross section at river mile 72.84 to the upper end of the study at river mile 96.55. Soldier Creek enters the Kansas River at approximately river mile 80.6. The discharge from Soldier Creek on the day of the peak Kansas River discharge in 1993 was only 2200 cfs and was not considered in the calibration.

The calibration of the backwater program to the high-water marks was accomplished by adjusting the Manning's "n" values for the channel until the profile matched the high-water marks. In this case, the 170,000 cfs required "n" values of 0.03 to 0.035 in the channel downstream of Sardou Avenue Bridge. Starting just upstream of Sardou Bridge, the "n" values changed to 0.02 and did not vary until about RM 86. Above this point, the "n" values again ranged from 0.03 or 0.035. The overbank "n" values ranged from 0.04 to 0.15 based on overbank conditions. Higher values of "n" were also used to reduce flow in overbanks that were either very wide or contained obstructions. For the side slopes of the levees, "n" values of either 0.040 or 0.045 were used. Table 1-4 compares the observed high water marks to the computed water surface elevation for the 1993 flood event. Plate A2-1-2 shows a graph of this same information.

Table 1-4 Comparison of 1993 High Water Mark Elevations and Computed Water Surface Elevations (WSEL) - Kansas River

HEC-RAS River Station	Location	Observed 1993 High-Water Mark Elevation (ft)	Computed Water Surface Elevation* (ft)	Difference: Computed WSEL vs. High Water Mark (ft)
76.25	Oakland (285+00)	870.5	871.2	0.7
77	Oakland Drainage Structure	871.4	872.2	0.8
77.4	FB-3 Oakland	874.4	872.5	-1.9
78.5	Belmont Rd Ramp on Oakland Levee	874.9	874.3	-0.6
80.4	FB-4 Oakland	877.2	876.9	-0.3
82.7	FB-8 Oakland	880.9	880.6	-0.3
83.1	Sardou Gage	881.6	881.0	-0.6
83.78	DS face A.T. & S.F. RR Bridge	881.6	882.0	0.4
83.79	US face A.T. & S.F. RR Bridge	881.9	882.2	0.3
84.21	DS face of Kansas Ave. Bridge	882.5	883.2	0.7
84.22	US face of Kansas Ave. Bridge	882.7	883.3	0.6
84.57	US face of Topeka Ave. Bridge	883.1	883.5	0.4
85.64	FB-18 South Topeka	884.3	884.6	0.3
87.1	Waterworks Drainage Structure	886.5	886.7	0.2
87.92	Highway 75	888.1	888.1	0.0

*Note: Computed Water Surface Elevation was interpolated the HEC-RAS River Station

Most of the computed water surface elevations matched the 1993 high-water marks within a few tenths of a foot. However, not all the high-water marks were matched precisely. The reason for this may be due, in part, to errors in the establishment of those marks. Some of the high-water marks were taken immediately after the 1993 flood crest receded, by examining the location of debris along the banks or levees. Another set of high-water marks, obtained from the City of Topeka, were taken from the tops of flood walls, freeboard gages, or the tops of gated structures. One example of a problem in meeting these marks occurred downstream of the Oakland Expressway bridge where there is a large jump in the marks between STA. 77 and STA. 77.4. Assuming these elevations were correct, the model could not be made to match this inconsistency without adjusting the “n” values to unreasonable extremes. In general, there are a number of different scenarios that can cause errors or inconsistencies with high water marks. These may include swellhead from debris blockage, relative proximity to the channel, and misinterpretation of field conditions. Because the validity of these particular high water marks is unknown, no additional effort was made to reproduce them in the model.

The flooding limits of the model were compared to a Flooded Area Map from the U.S. Army Corps of Engineers Post Flood Report, 1993 Kansas River Basin Flood, shown on Plates A2-1-3 and A2-1-4. During this process, it became apparent that the aerial photographs used to make the maps were not taken at the flood peak (170,000 cfs). The maps are dated JUL & AUG 1993. The actual river stage at Sardou bridge was within five feet of the peak flow stage (7-25-93) for only five days (five feet of elevation significantly changes the flooding extents). When the HEC-RAS model is run with a discharge of 110,000 cfs, it closely resembles the shape of the flood extents depicted on the map.

Model Verification

A gaging station is operated by the U.S.G.S. at the Sardou Bridge, Kansas River mile 83.1. The 1996 rating curve (rating no. 46 shown on Plate A2-1-5) developed for this gage was used to check the computed stage vs. discharge at this location. During the process of examining various discharges, a check was made of how well the model predicted the water surface elevation at the gage at Sardou Bridge.

To test the calibration of the model over a wide range of discharges, water surface profiles were computed for a series of discharges: 50, 20, 10, 4, 2, 1, 0.5, and 0.2-percent chance (2, 5, 10, 25, 50, 100, 200, and 500-year flood events) as determined by the Corps of Engineers in the 2002 Kansas River Hydrology Report (see Table 1-1). Table 1-5 shows these eight event discharge elevations versus the expected water surface elevation at the gage at Sardou Bridge. The gage elevations were determined from the 1996 rating curve which shows the stage versus the discharge. The stage was converted to an elevation by simply adding the elevation of the gage datum (846.66 feet, N.G.V.D.) to the stage reading. The largest discharge of the 1996 rating curve was 172,000 cfs, so that stages larger than 172,000 cfs were obtained by extrapolation. Rating curve no. 46 was used from January 1996 to September 2000.

Table 1-5 Computed Water Surface Elevation versus Expected Gage Height

Percent Chance of Exceedance (%)	Annual Event Discharge (cfs)	Computed Water Surface Elevation (ft)	Sardou Rating Curve Elevation (ft)	Difference: Computed vs Expected Gage Elevation (ft)
50	36,600	865.74	864.40	1.34
20	67,200	871.59	870.86	0.73
10	93,600	875.30	875.59	-0.29
5	123,000	877.70	878.42	-0.72
2	173,000*	881.24	881.86	-0.62
1	217,000*	883.66	884.46	-0.80
0.5	268,000*	886.04	886.86	-0.82
0.2	348,000*	889.28	890.16	-0.88
Note: All model elevations are from STA 83.1003				

*Discharge values greater than 172,000 cfs were determined by extrapolation of the Rating Curve #46

Kansas River Existing Condition (Base) Profiles

Once the model was calibrated, the existing conditions water surface profiles were generated using the discharges of Table 1-2 above. Plate A2-1-6 shows the profiles for the 10, 2, 1, 0.5, 0.2, 0.133, 0.1, and 0.04-percent chance (10, 50, 100, 200, 500, 750, 1000, and 2500-year) flood events. The tabular data is presented in Table 1-6, found at the end of this section.

The HEC-RAS model indicates that none of the Kansas River Levee Units in this study physically overtop until the water surface elevation reaches the 50% non-exceedance probability stage for the 0.2% chance exceedance (500-yr) flow. Discretion should be used when applying profiles higher than the top of the levee. The model used a confined cross sectional area from levee to levee. Essentially, overbank flow beyond the levee height was not taken into consideration. This assumption was made to avoid trying to predict where a levee would fail. Within the Topeka levee systems, there are many different combinations of failure scenarios that could physically occur. Potentially, each could produce a different overbank flow path. HEC-RAS is a one-dimensional steady state model. It is beyond the limitations for HEC-RAS to predict the overbank flow scenarios or to model multi-dimensional flow. Profiles for the rare frequency events that exceed the top of levee are highly speculative and would not necessarily match what would physically happen. These events were produced to formulate frequency-stage curves for economic analyses in the HEC-FDA computer program.

Hydraulic Uncertainty

Uncertainties in computed stage result from two main sources: natural variations in the river and modeling errors. Natural variations include uncertainties in physical factors such as bed forms, debris and other obstructions, channel scour or deposition, sediment transport, and waves. Modeling uncertainty includes factors such as inexact geometry and loss coefficients, variation in hydraulic roughness with season, and error in setting high water marks (EM 1110-2-1619).

In Risk Based Analysis, the stage uncertainty is express as standard deviation (in feet). The total standard deviation depends on the standard deviation based on natural variations and the standard deviation based on model errors according to the formula below:

$$\text{Total Standard Deviation} = \sqrt{S_{natural}{}^{2} + S_{model}{}^{2}}$$

where $S_{natural}$ = standard deviation based on natural variations
S_{model} = standard deviation based on modeling uncertaities

For a gaged reached, $S_{natural}$ is calculated by comparing observed data with the latest rating curve at the gage in the study reach. To avoid potential problems due to shifts in the rating curve over time, only observed data going back to 1990 was used. Only data values for bank full discharges and greater were analyzed. The following formula is used to calculate $S_{natural}$.

$$S_{natural} = \sqrt{\frac{(X-M)^2}{(N-1)}}$$

where: X=Stage corresponding to measured Q
M=best fit curve estimate of stage corresponding to Q
N=number of stage-discharge observations in the range being analyzed

The standard deviation based on historical data and gage readings, $S_{natural}$, was computed as 0.48 feet.

Table 5-2 in EM 1110-2-1619 quantifies S_{model} based on the quality of topographic data and the reliability of the Manning's n-value. A standard deviation of 0.7 feet was chosen since the cross-sections were based on current aerial mapping and the Manning's n-values were assumed to be "fairly" reliable.

Once $S_{natural}$ and S_{model} are known, a total standard deviation can be computed. For this study a total standard deviation of 0.85 feet was computed for the entire discharge set.

A-2.1.6 SUMMARY

A hydraulic investigation was conducted on the Kansas River using the HEC-RAS computer software developed by the Hydrologic Engineering Center, U.S. Army Corps of Engineers. The program was used to calculate water surface profiles on the reach of the Kansas River that runs through Topeka, Kansas. The model was calibrated using high water marks from the 1993 flood. Water surface profiles were then generated for eight different discharge events. These include the 10, 2, 1, 0.5, 0.2, 0.133, 0.1, and 0.04-percent chance (10, 50, 100, 200, 500, 750, 1000, and 2500-year) flood events. The model shows that the existing levees are overtopped by the 0.2% chance exceedance (500-year) flood event with a 50% non-exceedance probability. Finally, the uncertainty in both stage and discharge were calculated. The standard deviation of stage is 0.85 feet. The discharge uncertainty results are shown above in Table 1-3 for a range of frequencies.

Table 1-6 Kansas River Existing Conditions Water Surface Profiles

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
72.843	0.04% (2500-yr)	500000	879.17	879.38	0.000101	5.79	220745.4	12666.12	0.16
72.843	0.1% (1000-yr)	410000	876.13	876.36	0.000114	5.82	182829.5	12267.71	0.17
72.843	0.133% (750-yr)	387000	875.35	875.59	0.000117	5.82	173333.6	12165.89	0.17
72.843	0.2% (500-yr)	348000	874.03	874.28	0.000122	5.79	157414.3	11993.25	0.17
72.843	0.5% (200-yr)	271000	871.43	871.7	0.000127	5.63	113783.7	11652.27	0.17
72.843	1% (100-yr)	221000	869.74	869.98	0.000117	5.21	100747.4	10508.8	0.17
72.843	2% (50-yr)	177000	867.46	867.71	0.000122	5.04	83149.5	9153.81	0.17
72.843	10% (10-yr)	97200	860.67	861.06	0.000195	5.3	31090.44	7354.23	0.2
73.355	0.04% (2500-yr)	500000	879.48	879.71	0.000125	6.25	223394.7	13601.15	0.18
73.355	0.1% (1000-yr)	410000	876.47	876.73	0.000142	6.33	182671.1	13464.22	0.19
73.355	0.133% (750-yr)	387000	875.7	875.97	0.000147	6.34	172318.5	13429.19	0.19
73.355	0.2% (500-yr)	348000	874.39	874.67	0.000154	6.33	154806.2	13369.73	0.19
73.355	0.5% (200-yr)	271000	871.8	872.09	0.000158	6.07	111270	13235.94	0.19
73.355	1% (100-yr)	221000	870.08	870.35	0.000149	5.68	97258.56	13134.64	0.19
73.355	2% (50-yr)	177000	867.82	868.07	0.000143	5.27	81009.14	7130.71	0.18
73.355	10% (10-yr)	97200	861.23	861.63	0.000237	5.62	34497.89	6980.34	0.22
74.307	0.04% (2500-yr)	500000	880.05	880.33	0.000111	5.8	195333.3	14305.34	0.17
74.307	0.1% (1000-yr)	410000	877.11	877.43	0.000127	5.86	154632.8	13404.42	0.18
74.307	0.133% (750-yr)	387000	876.36	876.71	0.000134	5.93	138366.3	13166.64	0.18
74.307	0.2% (500-yr)	348000	875.08	875.44	0.000139	5.88	124175.6	12711.61	0.18
74.307	0.5% (200-yr)	271000	872.5	872.87	0.000144	5.67	95608.34	11814.83	0.18
74.307	1% (100-yr)	221000	870.74	871.11	0.000142	5.4	76125.95	11762.57	0.18
74.307	2% (50-yr)	177000	868.46	868.76	0.000128	4.85	58643.15	5769.48	0.17
74.307	10% (10-yr)	97200	862.25	862.5	0.00013	4.06	29402.84	3882.23	0.16
75.21	0.04% (2500-yr)	500000	880.48	880.81	0.000152	6.62	189776.1	14836.82	0.19
75.21	0.1% (1000-yr)	410000	877.62	878.05	0.000189	6.99	137550.8	14613.31	0.21
75.21	0.133% (750-yr)	387000	876.91	877.36	0.000196	7.03	128996.5	14545.27	0.22
75.21	0.2% (500-yr)	348000	875.66	876.15	0.00021	7.1	114044.6	14425.83	0.22
75.21	0.5% (200-yr)	271000	873.13	873.71	0.000239	7.16	83947.42	12601.82	0.23
75.21	1% (100-yr)	221000	871.37	872.01	0.000254	7.08	63255.57	12263.26	0.24
75.21	2% (50-yr)	177000	869.04	869.62	0.000246	6.56	41029.38	6269.33	0.23
75.21	10% (10-yr)	97200	862.93	863.36	0.000234	5.32	20681.82	2005.55	0.22
75.309	0.04% (2500-yr)	500000	880.6	880.89	0.000156	6.56	199643.8	15531.12	0.19
75.309	0.1% (1000-yr)	410000	877.77	878.15	0.000199	7.02	144352	15317.31	0.22
75.309	0.133% (750-yr)	387000	877.06	877.46	0.000208	7.08	135595.3	15248.63	0.22
75.309	0.2% (500-yr)	348000	875.82	876.26	0.000228	7.21	120211.1	15127.93	0.23
75.309	0.5% (200-yr)	271000	873.27	873.84	0.000275	7.48	88727.73	13143.05	0.25
75.309	1% (100-yr)	221000	871.48	872.17	0.000315	7.66	66574.78	13064.69	0.26
75.309	2% (50-yr)	177000	869	869.86	0.000387	7.96	39847.99	8419.97	0.29
75.309	10% (10-yr)	97200	862.91	863.59	0.000398	6.65	15060.17	975.51	0.28
75.484	0.04% (2500-yr)	500000	880.72	881.05	0.000192	7.5	187650.4	16183.75	0.22
75.484	0.1% (1000-yr)	410000	877.93	878.35	0.000242	7.98	140278.3	14465.62	0.24

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
75.484	0.133% (750-yr)	387000	877.22	877.67	0.000256	8.1	131509.6	14396.68	0.25
75.484	0.2% (500-yr)	348000	875.98	876.5	0.000286	8.35	116088.9	14275.39	0.26
75.484	0.5% (200-yr)	271000	873.43	874.16	0.000367	8.96	84341.23	12444.85	0.29
75.484	1% (100-yr)	221000	871.6	872.59	0.000451	9.52	61518.19	12434.25	0.32
75.484	2% (50-yr)	177000	869	870.49	0.000624	10.5	33710.06	8945.02	0.37
75.484	10% (10-yr)	97200	863.02	864.18	0.0006	8.66	11329.21	623.85	0.34
76.29	0.04% (2500-yr)	500000	881.28	881.67	0.000194	7.51	169714.7	14888.49	0.22
76.29	0.1% (1000-yr)	410000	878.66	879.1	0.000222	7.64	136154.8	12581.6	0.23
76.29	0.133% (750-yr)	387000	878	878.46	0.000228	7.65	128527	12520.28	0.23
76.29	0.2% (500-yr)	348000	876.89	877.37	0.000239	7.66	115553.9	12414.21	0.24
76.29	0.5% (200-yr)	270000	874.71	875.24	0.00025	7.49	90610.23	12197.03	0.24
76.29	1% (100-yr)	220000	873.3	873.84	0.000246	7.19	74685.91	12059.46	0.23
76.29	2% (50-yr)	176000	871.5	872.13	0.000266	7.17	54719.37	11884.64	0.24
76.29	10% (10-yr)	96600	865.34	865.95	0.000289	6.31	18339.51	2854.59	0.24
77.045	0.04% (2500-yr)	500000	881.84	882.29	0.000187	7.49	170751.1	12572.11	0.22
77.045	0.1% (1000-yr)	410000	879.31	879.81	0.000205	7.47	131600.8	12352.35	0.22
77.045	0.133% (750-yr)	387000	878.68	879.19	0.000207	7.42	125122.6	12284.54	0.22
77.045	0.2% (500-yr)	348000	877.61	878.13	0.000209	7.29	114048.2	12165.88	0.22
77.045	0.5% (200-yr)	270000	875.52	876.02	0.000201	6.83	92369.34	11936.17	0.22
77.045	1% (100-yr)	220000	874.14	874.6	0.000183	6.33	78243.71	10077.32	0.21
77.045	2% (50-yr)	176000	872.49	872.93	0.000172	5.9	62056.17	9595.31	0.2
77.045	10% (10-yr)	96600	866.46	866.8	0.000159	4.81	28111.46	2852.69	0.18
77.73	0.04% (2500-yr)	500000	882	883.43	0.000645	11.44	133787.5	12048.43	0.34
77.73	0.1% (1000-yr)	410000	879.62	881.02	0.000641	10.88	106230.7	10041.62	0.34
77.73	0.133% (750-yr)	387000	879.04	880.4	0.000631	10.66	100344.8	9963.58	0.33
77.73	0.2% (500-yr)	348000	878.03	879.33	0.000608	10.24	90396.52	9830.28	0.32
77.73	0.5% (200-yr)	270000	876.04	877.14	0.000525	9.11	71078.86	9566.11	0.3
77.73	1% (100-yr)	220000	874.68	875.61	0.000449	8.16	58242.48	9385.96	0.27
77.73	2% (50-yr)	176000	873.08	873.86	0.000388	7.29	43500.34	8997.63	0.25
77.73	10% (10-yr)	96600	867.13	867.6	0.000306	5.5	17966.27	1125.15	0.21
78.577	0.04% (2500-yr)	500000	884.26	885.58	0.000609	11.78	137476.1	9622.91	0.34
78.577	0.1% (1000-yr)	410000	881.97	883.2	0.000575	10.99	115477.7	9595.44	0.32
78.577	0.133% (750-yr)	387000	881.37	882.56	0.000563	10.75	109705.9	9588.22	0.32
78.577	0.2% (500-yr)	348000	880.32	881.44	0.000536	10.28	99276.34	9332.05	0.31
78.577	0.5% (200-yr)	270000	878.09	879.03	0.00046	9.11	82742.23	7039.63	0.28
78.577	1% (100-yr)	220000	876.49	877.29	0.0004	8.21	71905.92	6481.55	0.26
78.577	2% (50-yr)	176000	874.69	875.38	0.000353	7.41	60253.82	6457.09	0.24
78.577	10% (10-yr)	96600	868.49	869.1	0.000355	6.32	20325.08	6431.25	0.23
78.853	0.04% (2500-yr)	500000	885.18	886.25	0.000493	10.54	133877.8	7007.12	0.3
78.853	0.1% (1000-yr)	410000	882.88	883.83	0.000453	9.69	117836.3	6982	0.29
78.853	0.133% (750-yr)	387000	882.27	883.19	0.000441	9.44	113564.9	6974.2	0.28
78.853	0.2% (500-yr)	348000	881.19	882.04	0.000418	9	106027.8	6960.4	0.27
78.853	0.5% (200-yr)	270000	878.85	879.56	0.000362	7.98	90138.63	6625.38	0.25
78.853	1% (100-yr)	220000	877.16	877.76	0.000318	7.22	78929.08	6591.17	0.23

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
78.853	2% (50-yr)	176000	875.28	875.81	0.000284	6.53	66623.87	6552.88	0.22
78.853	10% (10-yr)	96600	869.11	869.58	0.000294	5.61	26625.19	6401.51	0.21
79.654	0.04% (2500-yr)	500000	886.97	887.69	0.000386	9.79	122596.4	5950.71	0.27
79.654	0.1% (1000-yr)	410000	884.55	885.2	0.000364	9.13	108200.5	5947.69	0.26
79.654	0.133% (750-yr)	387000	883.9	884.53	0.000357	8.94	104322.7	5946.9	0.26
79.654	0.2% (500-yr)	348000	882.74	883.34	0.000345	8.61	97455.21	5945.74	0.25
79.654	0.5% (200-yr)	270000	880.22	880.75	0.000316	7.86	82450.42	5939.83	0.24
79.654	1% (100-yr)	220000	878.38	878.86	0.000293	7.3	71526.41	5917.92	0.23
79.654	2% (50-yr)	176000	876.39	876.85	0.000278	6.81	59811.82	5891.31	0.22
79.654	10% (10-yr)	96600	870.35	871.19	0.000451	7.49	18049.65	4119.18	0.27
79.858	0.04% (2500-yr)	500000	887.48	887.94	0.000223	7.23	123074.5	5263.07	0.2
79.858	0.1% (1000-yr)	410000	885.04	885.44	0.000206	6.65	110242.9	5256.98	0.19
79.858	0.133% (750-yr)	387000	884.38	884.76	0.000201	6.49	106783.3	5255.33	0.19
79.858	0.2% (500-yr)	348000	883.22	883.57	0.000192	6.2	100657.6	5252.98	0.19
79.858	0.5% (200-yr)	270000	880.67	880.96	0.000171	5.56	87271.55	5240.02	0.17
79.858	1% (100-yr)	220000	878.8	879.06	0.000155	5.09	77530.07	5216.72	0.16
79.858	2% (50-yr)	176000	876.81	877.04	0.000143	4.67	67164.35	5189.33	0.16
79.858	10% (10-yr)	96600	871.2	871.54	0.000201	4.77	25291.26	4864.94	0.18
79.862		Bridge							
79.867	0.04% (2500-yr)	500000	887.57	887.98	0.000202	6.88	123482.5	5263.27	0.19
79.867	0.1% (1000-yr)	410000	885.1	885.46	0.000188	6.36	110487.5	5257.09	0.19
79.867	0.133% (750-yr)	387000	884.44	884.78	0.000184	6.21	107012.4	5255.44	0.18
79.867	0.2% (500-yr)	348000	883.27	883.59	0.000176	5.95	100872.7	5253.06	0.18
79.867	0.5% (200-yr)	270000	880.71	880.98	0.000159	5.36	87446.2	5240.23	0.17
79.867	1% (100-yr)	220000	878.84	879.08	0.000146	4.93	77688.97	5217.14	0.16
79.867	2% (50-yr)	176000	876.84	877.06	0.000136	4.55	67288.21	5189.66	0.15
79.867	10% (10-yr)	96600	871.26	871.57	0.000191	4.66	25385.12	4865.82	0.17
80.037	0.04% (2500-yr)	500000	887.6	888.21	0.000275	8.09	114528.3	4919.53	0.23
80.037	0.1% (1000-yr)	410000	885.13	885.67	0.000254	7.44	102416.3	4915.84	0.22
80.037	0.133% (750-yr)	387000	884.47	884.99	0.000247	7.25	99175.27	4914.85	0.21
80.037	0.2% (500-yr)	348000	883.31	883.78	0.000235	6.93	93447.89	4913.1	0.21
80.037	0.5% (200-yr)	270000	880.76	881.15	0.000208	6.19	80925.71	4892.7	0.19
80.037	1% (100-yr)	220000	878.89	879.24	0.000187	5.64	71838.7	4862.47	0.18
80.037	2% (50-yr)	176000	876.9	877.2	0.00017	5.14	62194.46	4647.27	0.17
80.037	10% (10-yr)	96600	871.41	871.75	0.000192	4.73	25499.77	4568.1	0.17
80.593	0.04% (2500-yr)	500000	888.35	889.34	0.000495	10.69	95307.88	4581.22	0.3
80.593	0.1% (1000-yr)	410000	885.83	886.74	0.000477	10.03	83766.68	4572.4	0.29
80.593	0.133% (750-yr)	387000	885.15	886.04	0.000471	9.85	80668.34	4570.03	0.29
80.593	0.2% (500-yr)	348000	883.95	884.81	0.00046	9.51	75188.16	4565.83	0.29
80.593	0.5% (200-yr)	270000	881.32	882.1	0.000433	8.75	63204.71	4531.03	0.27
80.593	1% (100-yr)	220000	879.41	880.09	0.000393	8	54781.08	4167.8	0.26
80.593	2% (50-yr)	176000	877.37	877.99	0.000373	7.44	46322.62	4123.24	0.25
80.593	10% (10-yr)	96600	871.96	872.65	0.000418	6.81	19551.62	3851.21	0.25

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
80.945	0.04% (2500-yr)	500000	888.95	890.34	0.000463	10.86	69519.51	2857	0.31
80.945	0.1% (1000-yr)	410000	886.47	887.64	0.000417	9.85	62430.02	2857	0.29
80.945	0.133% (750-yr)	387000	885.8	886.91	0.000404	9.57	60515.35	2857	0.29
80.945	0.2% (500-yr)	348000	884.61	885.62	0.00038	9.07	57121.83	2853.93	0.28
80.945	0.5% (200-yr)	268000	881.99	882.79	0.000321	7.9	49664.05	2840.2	0.25
80.945	1% (100-yr)	217000	880.04	880.7	0.00028	7.06	44153.85	2825.97	0.23
80.945	2% (50-yr)	173000	878.02	878.56	0.000245	6.29	38448.99	2812.15	0.21
80.945	10% (10-yr)	93600	872.85	873.18	0.00018	4.68	24016.14	2766.89	0.18
81.633	0.04% (2500-yr)	500000	890.28	892.64	0.000767	14.43	62794.39	2744	0.4
81.633	0.1% (1000-yr)	410000	887.7	889.72	0.000693	13.13	55710.8	2744	0.38
81.633	0.133% (750-yr)	387000	887	888.92	0.000672	12.76	53787.62	2744	0.37
81.633	0.2% (500-yr)	348000	885.75	887.52	0.000634	12.12	50371.33	2741.24	0.36
81.633	0.5% (200-yr)	268000	882.99	884.4	0.00054	10.6	42833.62	2716.41	0.33
81.633	1% (100-yr)	217000	880.94	882.11	0.000474	9.51	37270.23	2706.82	0.3
81.633	2% (50-yr)	173000	878.82	879.8	0.000415	8.49	31617.52	2623.2	0.28
81.633	10% (10-yr)	93600	873.49	874.08	0.000295	6.25	19106	2097.84	0.23
82.333	0.04% (2500-yr)	500000	893.12	894.67	0.000451	11.13	62690.79	2210	0.31
82.333	0.1% (1000-yr)	410000	890.27	891.57	0.000408	10.08	56388.97	2210	0.29
82.333	0.133% (750-yr)	387000	889.49	890.72	0.000396	9.79	54669.69	2210	0.29
82.333	0.2% (500-yr)	348000	888.11	889.23	0.000375	9.28	51613.27	2210	0.28
82.333	0.5% (200-yr)	268000	885	885.88	0.000324	8.11	44777.52	2189.5	0.25
82.333	1% (100-yr)	217000	882.71	883.43	0.000288	7.27	39773.99	2177.9	0.24
82.333	2% (50-yr)	173000	880.38	880.97	0.000256	6.49	34714.91	2165.08	0.22
82.333	10% (10-yr)	93600	874.61	874.96	0.000194	4.83	22354.68	2110.59	0.18
83.032	0.04% (2500-yr)	500000	894.37	896.63	0.000529	13.86	58958.72	2169.13	0.37
83.032	0.1% (1000-yr)	410000	891.42	893.35	0.000482	12.62	52555.92	2169.13	0.35
83.032	0.133% (750-yr)	387000	890.61	892.45	0.000469	12.28	50805.81	2169.13	0.35
83.032	0.2% (500-yr)	348000	889.18	890.87	0.000446	11.68	47694.04	2169.13	0.34
83.032	0.5% (200-yr)	268000	885.95	887.3	0.000389	10.27	40760.62	2129.52	0.31
83.032	1% (100-yr)	217000	883.56	884.7	0.000348	9.27	35703.82	2115.69	0.29
83.032	2% (50-yr)	173000	881.15	882.1	0.000311	8.31	30615.44	2101.82	0.27
83.032	10% (10-yr)	93600	875.23	875.79	0.000227	6.12	18552.4	1610.55	0.22
83.1	0.04% (2500-yr)	500000	894.45	896.88	0.000565	14.35	56502.53	2052.6	0.38
83.1	0.1% (1000-yr)	410000	891.5	893.57	0.000513	13.05	50455.98	2052.6	0.36
83.1	0.133% (750-yr)	387000	890.7	892.66	0.000499	12.69	48802.15	2052.6	0.36
83.1	0.2% (500-yr)	348000	889.26	891.07	0.000473	12.06	45860.31	2052.6	0.34
83.1	0.5% (200-yr)	268000	886.03	887.47	0.00041	10.58	39304.98	2012.78	0.32
83.1	1% (100-yr)	217000	883.65	884.85	0.000366	9.53	34522.04	1999.02	0.3
83.1	2% (50-yr)	173000	881.23	882.23	0.000325	8.53	29709.37	1984.95	0.27
83.1	10% (10-yr)	93600	875.3	875.88	0.000235	6.26	18289.41	1550.8	0.23
83.105		Bridge							
83.109	0.04% (2500-yr)	500000	894.64	897.04	0.000555	14.27	56866.77	2052.6	0.38

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
83.109	0.1% (1000-yr)	410000	891.58	893.63	0.00051	13.02	50591.57	2052.6	0.36
83.109	0.133% (750-yr)	387000	890.77	892.72	0.000496	12.66	48922.08	2052.6	0.36
83.109	0.2% (500-yr)	348000	889.33	891.12	0.00047	12.03	45974.36	2052.6	0.34
83.109	0.5% (200-yr)	268000	886.09	887.52	0.000408	10.56	39397.76	2013.05	0.32
83.109	1% (100-yr)	217000	883.7	884.9	0.000364	9.51	34599.95	1999.17	0.29
83.109	2% (50-yr)	173000	881.27	882.27	0.000324	8.52	29773.18	1985.22	0.27
83.109	10% (10-yr)	93600	875.34	875.92	0.000234	6.25	18329.31	1558.91	0.22
83.429	0.04% (2500-yr)	500000	894.31	898.21	0.000375	16.9	38472.84	1282	0.47
83.429	0.1% (1000-yr)	410000	891.45	894.62	0.000332	15.16	34798.59	1282	0.43
83.429	0.133% (750-yr)	387000	890.68	893.67	0.00032	14.68	33812.84	1282	0.42
83.429	0.2% (500-yr)	348000	889.31	891.98	0.000298	13.81	32071.35	1269.2	0.41
83.429	0.5% (200-yr)	268000	886.2	888.2	0.000249	11.89	28167.43	1239.12	0.37
83.429	1% (100-yr)	217000	883.87	885.47	0.000216	10.53	25303.21	1220.45	0.34
83.429	2% (50-yr)	173000	881.49	882.75	0.000187	9.27	22422.63	1201.38	0.31
83.429	10% (10-yr)	93600	875.58	876.22	0.000127	6.52	16136.55	983.83	0.24
83.699	0.04% (2500-yr)	500000	894.51	898.88	0.000411	17.66	35229.29	1144.85	0.49
83.699	0.1% (1000-yr)	410000	891.67	895.2	0.000362	15.78	31983.06	1137.22	0.45
83.699	0.133% (750-yr)	387000	890.91	894.22	0.000347	15.26	31117.97	1131.08	0.44
83.699	0.2% (500-yr)	348000	889.55	892.49	0.000322	14.34	29588.8	1120.15	0.42
83.699	0.5% (200-yr)	268000	886.42	888.62	0.000268	12.3	26119.78	1099.58	0.38
83.699	1% (100-yr)	217000	884.08	885.82	0.000231	10.87	23563.79	1084.49	0.35
83.699	2% (50-yr)	173000	881.69	883.04	0.000199	9.54	20987.64	1069.07	0.32
83.699	10% (10-yr)	93600	875.73	876.41	0.000135	6.7	15295.66	897.62	0.25
83.783	0.04% (2500-yr)	500000	894.26	899.27	0.000484	19	33305.11	1104.22	0.53
83.783	0.1% (1000-yr)	410000	891.43	895.56	0.000431	17.09	29250.57	1099.89	0.49
83.783	0.133% (750-yr)	387000	890.69	894.55	0.000413	16.51	28508.26	1097.09	0.48
83.783	0.2% (500-yr)	348000	889.37	892.79	0.000382	15.5	27184.48	1090.36	0.46
83.783	0.5% (200-yr)	268000	886.31	888.85	0.000316	13.26	24145.74	1069.56	0.41
83.783	1% (100-yr)	217000	884.01	886.01	0.000271	11.71	21880.56	1053.37	0.38
83.783	2% (50-yr)	173000	881.64	883.2	0.000233	10.27	19573.45	1039.1	0.34
83.783	10% (10-yr)	93600	875.71	876.51	0.000161	7.27	14137.98	877.36	0.27
83.786		Bridge							
83.789	0.04% (2500-yr)	500000	897.84	901.88	0.000353	17.17	37242.78	1104.22	0.46
83.789	0.1% (1000-yr)	410000	894.23	897.62	0.000326	15.6	33264.85	1104.22	0.43
83.789	0.133% (750-yr)	387000	893.26	896.47	0.000319	15.17	32185.83	1104.22	0.43
83.789	0.2% (500-yr)	348000	891.41	894.4	0.000311	14.52	29221.01	1099.78	0.42
83.789	0.5% (200-yr)	268000	886.54	889.04	0.000308	13.16	24366.42	1071.17	0.41
83.789	1% (100-yr)	217000	884.3	886.25	0.000262	11.58	22152.98	1055.37	0.37
83.789	2% (50-yr)	173000	881.85	883.38	0.000227	10.19	19768.8	1040.35	0.34
83.789	10% (10-yr)	93600	875.81	876.6	0.000159	7.24	14212.22	878.38	0.27
84.047	0.04% (2500-yr)	500000	899.64	902.37	0.00021	13.59	40856.88	1004	0.36
84.047	0.1% (1000-yr)	410000	895.84	898.06	0.000192	12.23	37044.76	1004	0.34
84.047	0.133% (750-yr)	387000	894.81	896.9	0.000186	11.86	36009.63	1004	0.33

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
84.047	0.2% (500-yr)	348000	892.93	894.81	0.000178	11.22	34128.72	1004	0.32
84.047	0.5% (200-yr)	268000	887.94	889.44	0.00017	10.01	29133.73	990.6	0.31
84.047	1% (100-yr)	217000	885.42	886.59	0.000146	8.81	26653	981.04	0.28
84.047	2% (50-yr)	173000	882.77	883.67	0.000127	7.72	24068.26	970.99	0.26
84.047	10% (10-yr)	93600	876.33	876.79	0.00009	5.48	17896.44	946.54	0.21
84.209	0.04% (2500-yr)	500000	900.02	902.56	0.000198	13.2	42942.83	1085.01	0.35
84.209	0.1% (1000-yr)	410000	896.14	898.23	0.000183	11.93	38739.81	1085.01	0.33
84.209	0.133% (750-yr)	387000	895.09	897.07	0.000178	11.58	37601.03	1085.01	0.32
84.209	0.2% (500-yr)	348000	893.19	894.97	0.000171	10.98	35537.16	1085.01	0.31
84.209	0.5% (200-yr)	268000	888.14	889.59	0.000166	9.85	30108.18	1067.18	0.3
84.209	1% (100-yr)	217000	885.58	886.72	0.000144	8.69	27385	1058.08	0.28
84.209	2% (50-yr)	173000	882.9	883.78	0.000126	7.65	24558.93	1047.98	0.25
84.209	10% (10-yr)	93600	876.41	876.87	0.000092	5.48	17956.86	987.54	0.21
84.214		Bridge							
84.218	0.04% (2500-yr)	500000	900.36	902.9	0.000195	13.16	42989.02	1085.01	0.34
84.218	0.1% (1000-yr)	410000	896.35	898.45	0.000181	11.92	38640.84	1085.01	0.33
84.218	0.133% (750-yr)	387000	895.29	897.27	0.000177	11.57	37493.84	1085.01	0.32
84.218	0.2% (500-yr)	348000	893.37	895.15	0.00017	10.97	35407.8	1084.11	0.31
84.218	0.5% (200-yr)	268000	888.28	889.73	0.000165	9.84	29970.51	1052.5	0.3
84.218	1% (100-yr)	217000	885.69	886.82	0.000143	8.68	27263.98	1036.08	0.28
84.218	2% (50-yr)	173000	882.98	883.86	0.000125	7.64	24482.56	1018.93	0.25
84.218	10% (10-yr)	93600	876.46	876.92	0.000091	5.47	17970	977.59	0.21
84.309	0.04% (2500-yr)	500000	900.62	903	0.000182	12.73	44291.75	1109.68	0.33
84.309	0.1% (1000-yr)	410000	896.57	898.54	0.000169	11.54	39802.56	1109.68	0.32
84.309	0.133% (750-yr)	387000	895.51	897.36	0.000165	11.2	38617.84	1109.68	0.31
84.309	0.2% (500-yr)	348000	893.57	895.24	0.000158	10.62	36465.5	1108.92	0.3
84.309	0.5% (200-yr)	268000	888.45	889.81	0.000154	9.52	30873.44	1077.3	0.29
84.309	1% (100-yr)	217000	885.83	886.9	0.000134	8.41	28069.37	1060.68	0.27
84.309	2% (50-yr)	173000	883.1	883.93	0.000117	7.4	25193.48	1043.35	0.25
84.309	10% (10-yr)	93600	876.53	876.96	0.000086	5.31	18477.37	1001.72	0.2
84.556	0.04% (2500-yr)	500000	900.31	903.52	0.000236	14.78	38755.65	986.6	0.38
84.556	0.1% (1000-yr)	410000	896.36	898.99	0.000217	13.35	34850.93	986.6	0.36
84.556	0.133% (750-yr)	387000	895.31	897.79	0.000211	12.95	33819.29	986.6	0.35
84.556	0.2% (500-yr)	348000	893.43	895.63	0.000199	12.2	31976.77	944.59	0.34
84.556	0.5% (200-yr)	268000	888.4	890.15	0.000188	10.84	27292.46	916.89	0.32
84.556	1% (100-yr)	217000	885.81	887.18	0.000161	9.53	24942.32	902.59	0.29
84.556	2% (50-yr)	173000	883.11	884.16	0.000138	8.34	22522.94	887.16	0.27
84.556	10% (10-yr)	93600	876.58	877.11	0.000095	5.86	16856.88	848.34	0.21
84.563		Bridge							
84.569	0.04% (2500-yr)	500000	900.61	903.77	0.000231	14.67	39048.13	986.6	0.38
84.569	0.1% (1000-yr)	410000	896.59	899.19	0.000213	13.27	35077.22	986.6	0.36
84.569	0.133% (750-yr)	387000	895.52	897.98	0.000207	12.88	34028.54	986.6	0.35

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
84.569	0.2% (500-yr)	348000	893.61	895.79	0.000196	12.14	32144.4	945.56	0.34
84.569	0.5% (200-yr)	268000	888.54	890.28	0.000185	10.8	27424.02	917.68	0.32
84.569	1% (100-yr)	217000	885.92	887.27	0.000159	9.5	25038.55	903.2	0.29
84.569	2% (50-yr)	173000	883.19	884.24	0.000137	8.31	22599.25	887.65	0.27
84.569	10% (10-yr)	93600	876.62	877.15	0.000095	5.85	16892.72	848.6	0.21
84.621	0.04% (2500-yr)	500000	900.95	903.85	0.000211	14.09	40356.91	979.31	0.36
84.621	0.1% (1000-yr)	410000	896.88	899.27	0.000194	12.73	36373.62	979.31	0.34
84.621	0.133% (750-yr)	387000	895.8	898.05	0.000189	12.34	35320.49	979.31	0.33
84.621	0.2% (500-yr)	348000	893.83	895.86	0.000181	11.69	33390.07	978.79	0.32
84.621	0.5% (200-yr)	268000	888.73	890.34	0.000172	10.41	28463.78	949.94	0.31
84.621	1% (100-yr)	217000	886.06	887.32	0.000148	9.16	25956.43	934.74	0.28
84.621	2% (50-yr)	173000	883.31	884.28	0.000127	8.03	23402.17	918.46	0.26
84.621	10% (10-yr)	93600	876.69	877.18	0.000089	5.66	17452.75	877.74	0.21
84.641	0.04% (2500-yr)	500000	900.83	903.94	0.000227	14.56	39037.95	951	0.37
84.641	0.1% (1000-yr)	410000	896.79	899.34	0.000209	13.15	35192.67	951	0.35
84.641	0.133% (750-yr)	387000	895.72	898.12	0.000203	12.75	34175.86	951	0.35
84.641	0.2% (500-yr)	348000	893.76	895.92	0.000194	12.08	32310.9	950.02	0.34
84.641	0.5% (200-yr)	268000	888.67	890.39	0.000184	10.75	27547.61	921.91	0.32
84.641	1% (100-yr)	217000	886.02	887.37	0.000158	9.46	25127.59	907.22	0.29
84.641	2% (50-yr)	173000	883.28	884.31	0.000136	8.29	22658.81	891.49	0.27
84.641	10% (10-yr)	93600	876.67	877.2	0.000095	5.85	16899.8	852.03	0.21
84.644		Bridge							
84.647	0.04% (2500-yr)	500000	903.23	906.01	0.000191	13.81	41308.02	951	0.35
84.647	0.1% (1000-yr)	410000	898.48	900.82	0.000182	12.62	36797.09	951	0.33
84.647	0.133% (750-yr)	387000	897.13	899.36	0.000181	12.31	35506.64	951	0.33
84.647	0.2% (500-yr)	348000	897.04	898.85	0.000147	11.09	35421.37	951	0.3
84.647	0.5% (200-yr)	268000	888.76	890.48	0.000182	10.72	27625.74	922.38	0.32
84.647	1% (100-yr)	217000	886.15	887.49	0.000156	9.42	25238.61	907.93	0.29
84.647	2% (50-yr)	173000	883.39	884.41	0.000135	8.26	22747.27	892.06	0.27
84.647	10% (10-yr)	93600	876.74	877.26	0.000094	5.83	16945.37	852.35	0.21
84.812	0.04% (2500-yr)	500000	903.84	906.28	0.000416	13.58	45822	1160	0.34
84.812	0.1% (1000-yr)	410000	898.92	901.07	0.00041	12.66	40106.95	1160	0.33
84.812	0.133% (750-yr)	387000	897.51	899.6	0.000411	12.44	38479.39	1160	0.33
84.812	0.2% (500-yr)	348000	897.34	899.05	0.000337	11.24	38279.73	1160	0.3
84.812	0.5% (200-yr)	268000	888.95	890.74	0.000442	11.3	28668.06	1117.62	0.34
84.812	1% (100-yr)	217000	886.28	887.73	0.000387	10.09	25708.39	1099.79	0.31
84.812	2% (50-yr)	173000	883.46	884.64	0.000343	9	22640.56	1080.7	0.29
84.812	10% (10-yr)	93600	876.77	877.42	0.000246	6.58	15617.84	961.85	0.24
84.974	0.04% (2500-yr)	500000	904.57	906.53	0.000143	11.8	56970.96	1408	0.3
84.974	0.1% (1000-yr)	410000	899.62	901.31	0.000141	10.89	50010.2	1408	0.29
84.974	0.133% (750-yr)	387000	898.22	899.84	0.000141	10.66	48034.15	1408	0.29
84.974	0.2% (500-yr)	348000	897.91	899.24	0.000117	9.67	47599.05	1408	0.26
84.974	0.5% (200-yr)	268000	889.68	890.99	0.00015	9.47	36159.36	1363.57	0.29

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
84.974	1% (100-yr)	217000	886.9	887.95	0.000134	8.43	32390.3	1346.53	0.27
84.974	2% (50-yr)	173000	884	884.83	0.00012	7.49	28503.63	1329.52	0.25
84.974	10% (10-yr)	93600	877.1	877.56	0.000095	5.52	19487.6	1259.1	0.21
85.337	0.04% (2500-yr)	500000	905.39	906.82	0.000112	10.31	74115.1	1939	0.26
85.337	0.1% (1000-yr)	410000	900.33	901.59	0.000112	9.6	64312.88	1939	0.26
85.337	0.133% (750-yr)	387000	898.9	900.12	0.000113	9.42	61540.6	1939	0.26
85.337	0.2% (500-yr)	348000	898.47	899.48	0.000095	8.58	60692.99	1939	0.24
85.337	0.5% (200-yr)	268000	890.25	891.28	0.000127	8.54	44922.93	1891.16	0.26
85.337	1% (100-yr)	217000	887.36	888.2	0.000117	7.69	39476.09	1874.01	0.25
85.337	2% (50-yr)	173000	884.37	885.06	0.000109	6.93	33893.61	1856.51	0.24
85.337	10% (10-yr)	93600	877.29	877.75	0.000101	5.44	17320.85	1672.17	0.22
85.642	0.04% (2500-yr)	500000	905.8	907.01	0.000105	10.29	84216.66	2520	0.26
85.642	0.1% (1000-yr)	410000	900.63	901.78	0.000111	9.83	71188.91	2520	0.26
85.642	0.133% (750-yr)	387000	899.17	900.31	0.000114	9.73	67508.28	2520	0.26
85.642	0.2% (500-yr)	348000	898.68	899.63	0.000097	8.89	66268.88	2520	0.24
85.642	0.5% (200-yr)	268000	890.38	891.53	0.000142	9.35	45481.76	2489.37	0.28
85.642	1% (100-yr)	217000	887.45	888.45	0.000135	8.6	38308.87	2401.3	0.27
85.642	2% (50-yr)	173000	884.42	885.31	0.000131	7.92	31100.96	2365.65	0.26
85.642	10% (10-yr)	93600	877.37	877.97	0.000117	6.21	15286.77	1260.52	0.23
85.931	0.04% (2500-yr)	500000	906.57	907.24	0.000136	7.68	97839.95	3095	0.19
85.931	0.1% (1000-yr)	410000	901.35	902.02	0.000151	7.49	81687.36	3095	0.2
85.931	0.133% (750-yr)	387000	899.88	900.55	0.000157	7.46	77137.04	3095	0.2
85.931	0.2% (500-yr)	348000	899.27	899.84	0.000136	6.87	75241.77	3095	0.19
85.931	0.5% (200-yr)	268000	891	891.83	0.000234	7.78	45609.87	2989.75	0.24
85.931	1% (100-yr)	217000	887.99	888.74	0.00023	7.23	38178.81	2965.11	0.23
85.931	2% (50-yr)	173000	884.92	885.59	0.000229	6.72	30614.58	2939.93	0.23
85.931	10% (10-yr)	93600	877.81	878.22	0.000199	5.11	18373.13	942.77	0.2
86.127	0.04% (2500-yr)	500000	906.73	907.39	0.000174	7.44	98245.02	2962	0.19
86.127	0.1% (1000-yr)	410000	901.54	902.19	0.000188	7.19	82886.7	2962	0.19
86.127	0.133% (750-yr)	387000	900.09	900.73	0.000194	7.13	78569.52	2962	0.19
86.127	0.2% (500-yr)	348000	899.45	899.99	0.000168	6.56	76674.9	2962	0.18
86.127	0.5% (200-yr)	268000	891.38	892.1	0.000263	7.13	49529.69	2847.06	0.22
86.127	1% (100-yr)	217000	888.38	889	0.000251	6.53	42426.75	2823.32	0.21
86.127	2% (50-yr)	173000	885.31	885.85	0.000241	5.97	35194.48	2799.11	0.2
86.127	10% (10-yr)	93600	878.12	878.43	0.000209	4.53	21708.27	1549.19	0.18
86.339	0.04% (2500-yr)	500000	906.5	907.85	0.000312	10.05	70111.62	2344	0.25
86.339	0.1% (1000-yr)	410000	901.38	902.65	0.000327	9.57	58096.62	2344	0.25
86.339	0.133% (750-yr)	387000	899.94	901.19	0.000333	9.45	54722.97	2344	0.25
86.339	0.2% (500-yr)	348000	899.33	900.39	0.000285	8.67	53294.17	2344	0.23
86.339	0.5% (200-yr)	268000	891.44	892.57	0.00038	8.71	35429.77	1925.92	0.26
86.339	1% (100-yr)	217000	888.49	889.42	0.000346	7.83	30485.48	1426.54	0.25
86.339	2% (50-yr)	173000	885.47	886.22	0.000316	7	26220.88	1300.66	0.23
86.339	10% (10-yr)	93600	878.31	878.72	0.000249	5.13	18379.5	965.38	0.2

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
86.608	0.04% (2500-yr)	500000	906.87	908.33	0.000316	10.14	66069.94	2026	0.25
86.608	0.1% (1000-yr)	410000	901.81	903.11	0.000318	9.48	55941.48	1980.15	0.25
86.608	0.133% (750-yr)	387000	900.39	901.65	0.00032	9.31	53137.25	1973.77	0.25
86.608	0.2% (500-yr)	348000	899.72	900.78	0.000275	8.53	51811.75	1970.95	0.23
86.608	0.5% (200-yr)	268000	892.05	893.09	0.000337	8.28	37549.83	1749.37	0.25
86.608	1% (100-yr)	217000	889.05	889.89	0.000305	7.42	32320.97	1730.55	0.23
86.608	2% (50-yr)	173000	885.98	886.64	0.000275	6.59	28059.54	1180.82	0.22
86.608	10% (10-yr)	93600	878.69	879.05	0.000217	4.82	19652.95	1089.9	0.18
86.809	0.04% (2500-yr)	500000	907.37	908.67	0.000301	9.82	71519.61	2149	0.25
86.809	0.1% (1000-yr)	410000	902.26	903.46	0.000311	9.28	60550.16	2133.38	0.25
86.809	0.133% (750-yr)	387000	900.83	902	0.000315	9.14	57513.57	2116.71	0.25
86.809	0.2% (500-yr)	348000	899.96	901.12	0.000307	8.89	47311.64	2106.72	0.24
86.809	0.5% (200-yr)	268000	892.38	893.49	0.000372	8.58	35944.58	2042.91	0.26
86.809	1% (100-yr)	217000	889.35	890.25	0.000339	7.69	31449.16	2023.74	0.24
86.809	2% (50-yr)	173000	886.25	886.97	0.00031	6.86	27118.43	1756.4	0.23
86.809	10% (10-yr)	93600	878.91	879.31	0.00025	5.06	18514.37	890.76	0.2
86.992	0.04% (2500-yr)	500000	907.68	908.96	0.000304	9.76	71776.44	2505.4	0.25
86.992	0.1% (1000-yr)	410000	902.55	903.77	0.000321	9.32	59052.97	2455.37	0.25
86.992	0.133% (750-yr)	387000	901.12	902.32	0.000328	9.19	55559.79	2435.02	0.25
86.992	0.2% (500-yr)	348000	900.41	901.42	0.000284	8.46	53819.08	2424.81	0.23
86.992	0.5% (200-yr)	268000	892.82	893.85	0.000362	8.34	37971.92	1624.89	0.25
86.992	1% (100-yr)	217000	889.73	890.58	0.000337	7.54	32991.37	1595.76	0.24
86.992	2% (50-yr)	173000	886.58	887.28	0.000316	6.79	28266.28	1344.89	0.23
86.992	10% (10-yr)	93600	879.15	879.57	0.000277	5.18	18791.44	1242.26	0.2
87.681	0.04% (2500-yr)	500000	908.95	910.02	0.000266	8.71	75171.95	2565.67	0.23
87.681	0.1% (1000-yr)	410000	903.92	904.87	0.000274	8.16	64956.38	2540.01	0.23
87.681	0.133% (750-yr)	387000	902.52	903.44	0.000277	8.01	62109.86	2532.86	0.23
87.681	0.2% (500-yr)	348000	901.62	902.4	0.000244	7.4	60271.37	2528.25	0.21
87.681	0.5% (200-yr)	268000	894.33	895.1	0.000312	7.25	45534.09	2422.1	0.23
87.681	1% (100-yr)	217000	891.11	891.76	0.000305	6.65	39095.53	2370.16	0.22
87.681	2% (50-yr)	173000	887.86	888.42	0.000303	6.1	32873.54	1715.14	0.22
87.681	10% (10-yr)	93600	880.29	880.67	0.00033	4.94	20384.42	1557.93	0.22
87.907	0.04% (2500-yr)	500000	908.88	910.55	0.000318	10.95	60985.01	1717.82	0.29
87.907	0.1% (1000-yr)	410000	903.9	905.39	0.000329	10.27	52511.45	1689.56	0.29
87.907	0.133% (750-yr)	387000	902.52	903.96	0.000334	10.09	50174.28	1683.32	0.29
87.907	0.2% (500-yr)	348000	901.62	902.86	0.000294	9.33	48668.72	1679.48	0.27
87.907	0.5% (200-yr)	268000	894.42	895.64	0.000382	9.18	36682.91	1648.49	0.3
87.907	1% (100-yr)	217000	891.24	892.28	0.000377	8.45	31542.16	1524.37	0.29
87.907	2% (50-yr)	173000	888.02	888.93	0.000382	7.79	26665.87	1511.09	0.29
87.907	10% (10-yr)	93600	880.56	881.2	0.000446	6.46	15806.45	1337.95	0.29
87.911		Bridge							
87.916	0.04% (2500-yr)	500000	909.07	910.72	0.000314	10.9	61302.86	1718.87	0.29
87.916	0.1% (1000-yr)	410000	904.05	905.52	0.000325	10.23	52750.25	1690.25	0.29

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
87.916	0.133% (750-yr)	387000	902.65	904.08	0.00033	10.05	50393.98	1683.93	0.29
87.916	0.2% (500-yr)	348000	901.71	902.94	0.000292	9.3	48817.25	1679.86	0.27
87.916	0.5% (200-yr)	268000	894.49	895.7	0.00038	9.16	36782.65	1648.8	0.3
87.916	1% (100-yr)	217000	891.3	892.35	0.000374	8.42	31634.88	1524.69	0.29
87.916	2% (50-yr)	173000	888.09	888.98	0.000379	7.77	26754.39	1511.33	0.29
87.916	10% (10-yr)	93600	880.63	881.26	0.00044	6.44	15884.48	1341.61	0.29
87.933	0.04% (2500-yr)	500000	909.22	910.76	0.000296	10.6	61555.89	1719.73	0.28
87.933	0.1% (1000-yr)	410000	904.18	905.56	0.000308	9.99	52952.36	1690.93	0.28
87.933	0.133% (750-yr)	387000	902.77	904.12	0.000314	9.83	50583.02	1684.48	0.28
87.933	0.2% (500-yr)	348000	901.82	902.97	0.000279	9.1	48970.82	1680.26	0.27
87.933	0.5% (200-yr)	268000	894.57	895.74	0.000368	9.03	36904.29	1649.2	0.29
87.933	1% (100-yr)	217000	891.37	892.38	0.000366	8.33	31718.19	1525	0.29
87.933	2% (50-yr)	173000	888.14	889.02	0.000372	7.7	26820.03	1511.53	0.28
87.933	10% (10-yr)	93600	880.67	881.3	0.000437	6.42	15927.16	1344.12	0.29
87.938		Bridge							
87.943	0.04% (2500-yr)	500000	909.29	910.89	0.000304	10.76	61662.72	1720.1	0.29
87.943	0.1% (1000-yr)	410000	904.24	905.68	0.000316	10.12	53048.38	1691.26	0.29
87.943	0.133% (750-yr)	387000	902.83	904.23	0.000322	9.96	50676.89	1684.75	0.29
87.943	0.2% (500-yr)	348000	901.86	903.07	0.000286	9.22	49045.46	1680.46	0.27
87.943	0.5% (200-yr)	268000	894.63	895.83	0.000374	9.11	36993.11	1649.48	0.3
87.943	1% (100-yr)	217000	891.43	892.46	0.000369	8.39	31798.38	1525.28	0.29
87.943	2% (50-yr)	173000	888.2	889.09	0.000374	7.73	26901.56	1511.75	0.28
87.943	10% (10-yr)	93600	880.75	881.37	0.000431	6.39	16020.64	1348.41	0.29
88.254	0.04% (2500-yr)	500000	910.95	911.26	0.000084	5.78	184385.9	10759.21	0.15
88.254	0.1% (1000-yr)	410000	905.27	906.14	0.000196	8.07	71053.93	10754.95	0.22
88.254	0.133% (750-yr)	387000	903.82	904.7	0.000206	8.06	66291.4	10753.91	0.23
88.254	0.2% (500-yr)	348000	902.69	903.49	0.000189	7.58	62600.62	10753.35	0.22
88.254	0.5% (200-yr)	268000	895.43	896.38	0.000279	7.98	38731.09	6182.27	0.26
88.254	1% (100-yr)	217000	892.18	893	0.000275	7.35	31400.32	1447.46	0.25
88.254	2% (50-yr)	173000	888.94	889.63	0.000273	6.71	26743.19	1426.85	0.24
88.254	10% (10-yr)	93600	881.52	881.96	0.000281	5.31	17649.8	1098.59	0.23
89.065	0.04% (2500-yr)	500000	911.38	911.61	0.000076	5.4	211507	11529.74	0.14
89.065	0.1% (1000-yr)	410000	906.45	906.83	0.00012	6.28	137837.5	11506.59	0.18
89.065	0.133% (750-yr)	387000	905.03	905.45	0.000133	6.46	126199	11501.6	0.18
89.065	0.2% (500-yr)	348000	903.78	904.19	0.000132	6.28	116027.9	11497.19	0.18
89.065	0.5% (200-yr)	268000	896.72	897.6	0.000288	8.07	58392.13	8411.05	0.26
89.065	1% (100-yr)	217000	893.37	894.37	0.000343	8.15	32351.39	2081.11	0.28
89.065	2% (50-yr)	173000	890.13	891	0.000348	7.54	25691.1	2011.23	0.27
89.065	10% (10-yr)	93600	882.8	883.35	0.00036	5.98	15757.27	1018.43	0.26
89.85	0.04% (2500-yr)	500000	911.73	911.94	0.000075	5.4	229743.4	12763.6	0.14
89.85	0.1% (1000-yr)	410000	907.03	907.33	0.000111	6.1	169718.9	12739.13	0.17
89.85	0.133% (750-yr)	387000	905.65	906.01	0.000128	6.39	152199.6	12732.25	0.18
89.85	0.2% (500-yr)	348000	904.38	904.76	0.000132	6.37	136061.4	12725.91	0.18

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
89.85	0.5% (200-yr)	268000	897.97	898.79	0.000275	8.12	59974.77	10023.69	0.26
89.85	1% (100-yr)	217000	894.92	895.67	0.000277	7.61	45093.14	2472.44	0.25
89.85	2% (50-yr)	173000	891.65	892.33	0.000286	7.12	37059.01	2437.47	0.25
89.85	10% (10-yr)	93600	884.25	884.76	0.000319	5.94	20636.03	1982.72	0.25
90.204	0.04% (2500-yr)	500000	911.86	912.07	0.000066	5.11	227673.5	13917.41	0.13
90.204	0.1% (1000-yr)	410000	907.21	907.53	0.000097	5.74	165988.1	13881.54	0.16
90.204	0.133% (750-yr)	387000	905.87	906.23	0.000111	6.01	148135.8	13867.06	0.17
90.204	0.2% (500-yr)	348000	904.61	904.99	0.000114	5.98	131440.7	13848.94	0.17
90.204	0.5% (200-yr)	268000	898.45	899.25	0.000224	7.46	54894.03	10747.9	0.23
90.204	1% (100-yr)	217000	895.41	896.14	0.000224	6.96	36993.15	2574.47	0.23
90.204	2% (50-yr)	173000	892.18	892.8	0.000218	6.36	30531.17	2511.53	0.22
90.204	10% (10-yr)	93600	884.87	885.24	0.000198	4.86	19301.62	1119.65	0.2
90.551	0.04% (2500-yr)	500000	912	912.19	0.000075	4.56	234145.1	15877.01	0.13
90.551	0.1% (1000-yr)	410000	907.41	907.71	0.000118	5.3	161352.9	15804.41	0.16
90.551	0.133% (750-yr)	387000	906.08	906.44	0.000137	5.6	140450.5	15794.15	0.17
90.551	0.2% (500-yr)	348000	904.83	905.21	0.000145	5.61	120603	15784.41	0.17
90.551	0.5% (200-yr)	268000	899.05	899.7	0.000253	6.56	50770.23	9196.71	0.22
90.551	1% (100-yr)	217000	896.01	896.59	0.000252	6.09	36698.95	2246.29	0.22
90.551	2% (50-yr)	173000	892.76	893.24	0.000255	5.61	31393	1577.93	0.22
90.551	10% (10-yr)	93600	885.34	885.68	0.000297	4.68	19999.72	1408.31	0.22
91.207	0.04% (2500-yr)	500000	912.22	912.53	0.000118	6.4	205061.6	15571.7	0.18
91.207	0.1% (1000-yr)	410000	907.72	908.3	0.000206	7.81	135060.4	15518.3	0.23
91.207	0.133% (750-yr)	387000	906.43	907.16	0.000248	8.38	115092.1	15487.71	0.25
91.207	0.2% (500-yr)	348000	905.17	906	0.000273	8.58	95562.35	15457.73	0.26
91.207	0.5% (200-yr)	268000	899.55	901.23	0.00053	10.6	34259.94	5921.11	0.35
91.207	1% (100-yr)	217000	896.59	898.09	0.000531	9.86	23422.39	1206.14	0.34
91.207	2% (50-yr)	173000	893.45	894.7	0.000526	9	19769.12	1131.83	0.33
91.207	10% (10-yr)	93600	886.39	887.18	0.000543	7.11	13218.2	870.57	0.32
91.503	0.04% (2500-yr)	500000	912.4	912.73	0.000125	6.49	200125.4	15554.19	0.18
91.503	0.1% (1000-yr)	410000	908.02	908.65	0.000217	7.93	132247.8	15478.43	0.23
91.503	0.133% (750-yr)	387000	906.8	907.57	0.000261	8.49	113281.5	15470.79	0.25
91.503	0.2% (500-yr)	348000	905.57	906.46	0.000288	8.71	94224.27	15463.4	0.26
91.503	0.5% (200-yr)	268000	900.52	902.03	0.000481	10.1	37292.94	8209.73	0.33
91.503	1% (100-yr)	217000	897.52	898.9	0.00049	9.46	24252.93	1162.65	0.33
91.503	2% (50-yr)	173000	894.35	895.5	0.000488	8.64	20675.69	1101.79	0.32
91.503	10% (10-yr)	93600	887.27	888	0.000512	6.86	13686.93	912.94	0.31
91.984	0.04% (2500-yr)	500000	912.72	913.06	0.000141	6.83	197526.1	16159.24	0.19
91.984	0.1% (1000-yr)	410000	908.57	909.24	0.000249	8.46	130730.8	16029.02	0.25
91.984	0.133% (750-yr)	387000	907.45	908.28	0.000299	9.07	112823.2	15983.14	0.27
91.984	0.2% (500-yr)	348000	906.27	907.26	0.000335	9.38	94080.72	15925.88	0.28
91.984	0.5% (200-yr)	268000	901.64	903.44	0.000568	11.08	37061.8	8835.82	0.36
91.984	1% (100-yr)	217000	898.65	900.37	0.000595	10.54	21265.72	978.86	0.36
91.984	2% (50-yr)	173000	895.51	896.93	0.000582	9.57	18307.48	876.74	0.35
91.984	10% (10-yr)	93600	888.55	889.41	0.000566	7.44	12588.13	791.16	0.33

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
92.648	0.04% (2500-yr)	500000	913.21	913.55	0.000146	6.89	198381.6	16750.94	0.19
92.648	0.1% (1000-yr)	410000	909.46	910.09	0.000241	8.27	135827.3	16635.89	0.24
92.648	0.133% (750-yr)	387000	908.55	909.28	0.000274	8.68	120610.3	16604.08	0.26
92.648	0.2% (500-yr)	348000	907.53	908.36	0.000295	8.82	103765.1	16553.72	0.27
92.648	0.5% (200-yr)	268000	903.86	905.19	0.000429	9.87	50773.52	9917.19	0.32
92.648	1% (100-yr)	217000	900.71	902.31	0.000528	10.16	22164.52	996.5	0.34
92.648	2% (50-yr)	173000	897.51	898.83	0.000514	9.22	19089.03	892.79	0.33
92.648	10% (10-yr)	93600	890.44	891.22	0.000486	7.11	13188.23	798.04	0.31
93.523	0.04% (2500-yr)	500000	913.82	914.47	0.000248	8.53	157886.7	16447.8	0.25
93.523	0.1% (1000-yr)	410000	910.46	911.56	0.000377	9.87	103679.2	16164.02	0.3
93.523	0.133% (750-yr)	387000	909.7	910.92	0.000409	10.13	91476.16	16085.93	0.31
93.523	0.2% (500-yr)	348000	908.79	910.08	0.000418	10.05	76976.01	15992.65	0.31
93.523	0.5% (200-yr)	268000	905.92	907.11	0.000405	9.28	49116.2	7434.02	0.3
93.523	1% (100-yr)	217000	903.33	904.54	0.000429	8.97	30549.05	7170.86	0.31
93.523	2% (50-yr)	173000	900	901.06	0.000441	8.31	23757.86	1585.31	0.31
93.523	10% (10-yr)	93600	892.75	893.44	0.000468	6.68	14139.64	1093.61	0.3
94.323*	0.04% (2500-yr)	500000	914.75	915.56	0.000302	9.19	144924.1	16632.14	0.27
94.323*	0.1% (1000-yr)	410000	911.97	913.09	0.000391	9.9	99752.65	16318.71	0.31
94.323*	0.133% (750-yr)	387000	911.39	912.55	0.000402	9.91	90333.29	16249.77	0.31
94.323*	0.2% (500-yr)	348000	910.58	911.74	0.000396	9.66	77330.42	16169.7	0.31
94.323*	0.5% (200-yr)	268000	907.61	908.83	0.000419	9.3	45352.68	7724.41	0.31
94.323*	1% (100-yr)	217000	905.23	906.3	0.0004	8.56	32189.68	1865.08	0.3
94.323*	2% (50-yr)	173000	901.93	902.87	0.000416	7.96	26706.13	1821.55	0.3
94.323*	10% (10-yr)	93600	894.77	895.44	0.000479	6.61	15196.7	1472.81	0.3
95.122	0.04% (2500-yr)	500000	916.17	916.8	0.000335	8.29	150789.3	16829.69	0.25
95.122	0.1% (1000-yr)	410000	913.9	914.69	0.000397	8.63	112711.8	16680.62	0.27
95.122	0.133% (750-yr)	387000	913.38	914.2	0.000403	8.61	104145.7	16648.23	0.27
95.122	0.2% (500-yr)	348000	912.56	913.39	0.000401	8.44	90471.64	16596.41	0.26
95.122	0.5% (200-yr)	268000	909.73	910.64	0.000432	8.23	54779.72	7297.95	0.27
95.122	1% (100-yr)	217000	907.24	908.05	0.000418	7.62	40700.43	2093.94	0.26
95.122	2% (50-yr)	173000	903.97	904.68	0.000432	7.09	33918.22	2055.73	0.26
95.122	10% (10-yr)	93600	896.96	897.47	0.000476	5.83	19831.92	1950.24	0.26
95.837*	0.04% (2500-yr)	500000	917.13	917.66	0.000284	7.47	165489.9	15794.15	0.23
95.837*	0.1% (1000-yr)	410000	915.17	915.7	0.000285	7.19	134730.4	15596.63	0.22
95.837*	0.133% (750-yr)	387000	914.71	915.23	0.00028	7.05	127530.4	15452.45	0.22
95.837*	0.2% (500-yr)	348000	913.92	914.43	0.000267	6.78	115543.5	15169.51	0.22
95.837*	0.5% (200-yr)	268000	911.3	911.82	0.000277	6.49	80020.52	11775.16	0.22
95.837*	1% (100-yr)	217000	908.76	909.28	0.000291	6.25	53808.43	3272.71	0.22
95.837*	2% (50-yr)	173000	905.51	906	0.000316	5.94	43241.64	3244.08	0.22
95.837*	10% (10-yr)	93600	898.67	899.06	0.000384	5.12	21845.7	3026.51	0.23
96.553	0.04% (2500-yr)	500000	917.98	918.33	0.000206	6.17	170407.9	14580.41	0.19
96.553	0.1% (1000-yr)	410000	916.07	916.4	0.000199	5.82	143418.8	13615.95	0.19
96.553	0.133% (750-yr)	387000	915.6	915.92	0.000195	5.7	137090.8	13379.76	0.18

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
96.553	0.2% (500-yr)	348000	914.79	915.1	0.000185	5.46	126480.3	12974.08	0.18
96.553	0.5% (200-yr)	268000	912.25	912.55	0.000188	5.18	95120.86	11693.12	0.18
96.553	1% (100-yr)	217000	909.79	910.12	0.000211	5.14	67865.09	10557.95	0.18
96.553	2% (50-yr)	173000	906.64	906.99	0.000252	5.11	49109.74	4409.57	0.2
96.553	10% (10-yr)	93600	900.1	900.45	0.000353	4.72	20711.3	4158.98	0.22

A-2.2 SOLDIER CREEK

A-2.2.1 INTRODUCTION

As part of the feasibility study, hydrologic and hydraulic analyses were conducted on Soldier Creek, located in Topeka, Kansas, and Shawnee and Jefferson Counties. The hydrologic analysis was completed to determine the expected discharges at the flood reduction works based upon statistical analyses of four stream flow gages in the watershed. The hydraulic investigation was completed to calculate water surface profiles on the first ten miles of Soldier Creek. To accomplish this, the HEC-RAS (River Analysis System) computer software developed by the Hydrologic Engineering Center, U.S. Army Corps of Engineers was used. The hydraulic model was developed using 1997 field surveys and 1995 aerial contour maps used in the reconnaissance report, supplemented by additional four-foot contours, supplied by the City of Topeka. Plates A2-2-1 and A2-2-2 show maps of the study area.

A-2.2.2 PURPOSE

The purpose of this investigation is to develop Soldier Creek water surface profiles from the Kansas River to the upstream limit of the flood reduction works reflecting the base (or existing) conditions. The resulting hydraulic model will be used to evaluate a series of alternatives for improving the integrity of the existing flood control system.

A-2.2.3 BACKGROUND

The Soldier Creek Diversion Unit, which was included in the Topeka, Kansas Flood Reduction Project, was authorized by the Flood Control Act approved in September 1954, House Document 642, 81st Congress, 2nd Session. Construction was initiated in March 1957 and was completed in November 1961.

The Soldier Creek study area is located near the north side of the Kansas River valley. The flood reduction project, developed by the Kansas City District, consists of approximately 10 miles of new and modified Soldier Creek channel and about 18 miles of levees along one or both sides of the modified channel. Tieback levees were also provided for several left bank tributaries.

The combination of the Soldier Creek Diversion Unit and the North Topeka Unit, which is located on the north bank of the Kansas River, provides flood reduction for 5,130 acres of agricultural, commercial and residential land.

A-2.2.4 HYDROLOGY

The following steps were used to complete the hydrologic investigation. First, a statistical frequency analysis was conducted on four USGS gages within Soldier Creek watershed. Next, relationships were developed between drainage area and discharge based for each frequency event. These relationships were then applied to the drainage areas within the flood reduction works to determine discharges for the first ten miles of Soldier Creek. Lastly, the hydrologic uncertainty was quantified.

Frequency Analysis

The frequency analysis was completed using the HEC-FFA (Flow Frequency Analysis) computer program developed by the Hydrologic Engineering Center, U.S. Army Corps of Engineers. There are four different USGS gages (Soldier, Circleville, Delia, and Topeka) within the Soldier Creek watershed. Plate A2-2-3 shows the location of the gages. The Topeka gage had the longest record (78 years) and is located within the study reach.

A frequency analysis of Soldier Creek was originally completed for the feasibility study in 2003, but in October of 2005, Soldier Creek experienced the largest flood of record at the Topeka and Delia gages. The magnitude of this flood relative to the rest of the gage record warranted a restudy of Soldier Creek's frequency discharges. Therefore, a new frequency analysis was conducted for the Topeka and Delia gages with a period of record through water year 2006. The full details of that analysis are recorded in a Memorandum for NWK-PM-PF prepared by Gordon Lance that was dated January 25, 2006.

The frequency curve results from the HEC-FFA analyses are illustrated in Figures 1 and 2 and summarized in Table 2-1. The confidence limits for these plots are set at +/- one standard deviation. It is noted that there were very large discharges for 1999 in both records, and an extremely high value for 2006. For the analysis at Delia, the frequency curve has an extremely high positive skew, even with the 2006 discharge (59,600 cfs) being treated as a high outlier. It is noted, however, that almost all of the data points on Figure 1 fall within one standard deviation of the computed value. The obvious exception is the very great value for the 2006 event, which is clearly an isolated high outlier. An estimated frequency for the 2006 event would be in the 0.5 % to 0.2% chance flood range.

Figure 2-1 Plot of Soldier Creek near Delia Gage Record

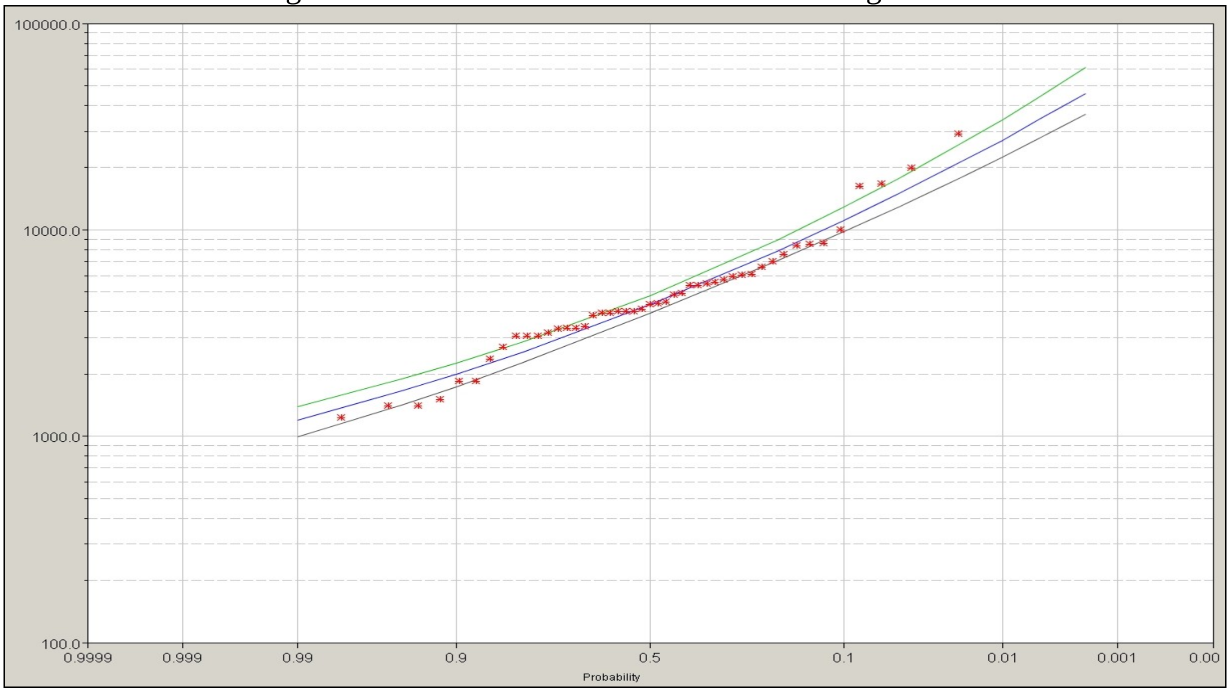


Figure 2-2 Plot of Soldier Creek near Topeka Gage

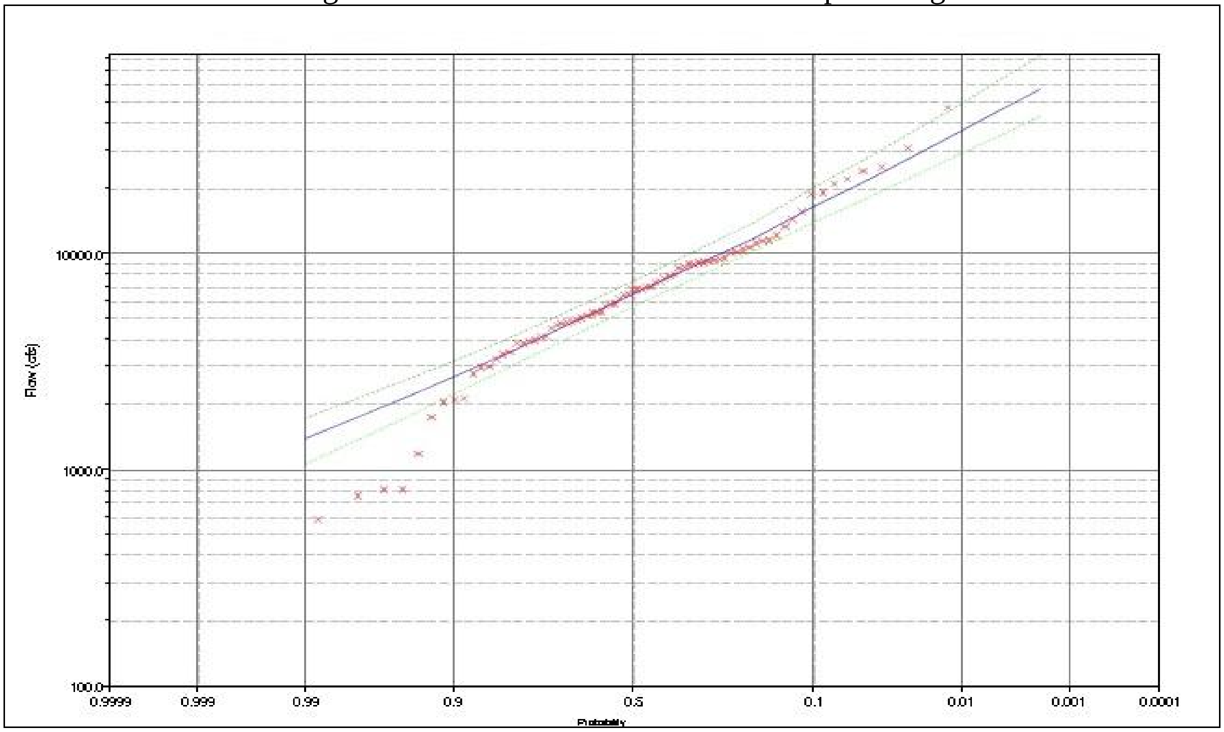


Table 2-1 Frequency Analysis Results

% Chance Exceedance	Discharge (cfs)	
	Delia	Topeka
0.2	103,000	56,400
0.5	66,100	44,300
1	46,800	36,400
2	32,800	29,400
5	20,000	21,500
10	13,400	16,300
20	8,650	11,800
50	4,290	6,480
Mean	3.6831	3.7759
Std Deviation	0.3316	0.3536
Regional Skew	0.9314	0.1531
Drainage Area	157 sq. mi.	290 sq. mi.
Period of Record	1958 to 2006	1929 to 2006
Yrs of Record	48	78

The Topeka record contains six very low peak annual discharge records. These records reflect the drought conditions experienced in the 1930's and 1950's, periods before the Delia gage was established. Since this is a flood study, it is important to secure a better definition of the right side of the curve. Therefore, the low outlier screen was set at 1000 cfs to screen out the effects of the four lowest discharges. Once this was done, the skew turned mildly positive to a value of +0.15, and the fit to the data points at the high end of the curve was improved. The use of a positive skew in lieu of the negative skew used in the previous study did not have the dramatic effect one might expect. This is due to the reduction in the standard deviation resulting from abandoning the four low outliers. One may note from the results in Table 2-1 that the peak discharges for large flood events actually decrease downstream from Delia to Topeka. The floodplain widens out considerably downstream of Delia, and the available storage causes attenuation of the peak flows, as occurred during October 2005 and other historic flood events.

Feasibility Discharges

The discharges were calculated for the first ten miles of Soldier Creek for the 0.2, 0.5, 1, 2, 5, 10, 20, 50- percent frequency events based on an analysis of the flows through the October 2005 flood event. The following recommended discharges, which are based on the Topeka gage record as described above, are proposed for the entire studied reach of Soldier Creek upstream of Halfday Creek. Proposed discharges downstream from the mouth of that creek have been increased using the coefficients proposed by HNTB in the previous hydrology report. Table 2-2 summarizes the feasibility discharges used on the Kansas River for the existing conditions model. Since flood events above the 0.2% chance exceedance (500 year) event need to be considered in this study, the discharge-frequency curves were extended up to the 0.04% chance exceedance (2500 year) event. This was accomplished through a straight-line extrapolation on a log-probability plot of the discharge frequency events at the Topeka gage.

Table 2-2 Feasibility Flood Discharges

Percent Chance of Exceedance	Approximate Return Interval	Study Limits to Halfday Creek	From Halfday Creek to Indian Creek	From Indian Creek to the Mouth
	(yrs)	(cfs)	(cfs)	(cfs)
River Miles		9.870 – 4.396	4.396 – 1.681	1.681 - 0
0.2	500	56400	61500	64300
0.5	200	44300	48300	50500
1	100	36400	39700	41500
2	50	29400	32000	33500
5	20	21500	23400	24500
10	10	16300	17800	18600
20	5	11800	12900	13500
50	2	6480	7060	7390

Hydrologic Uncertainty

In the past, the Corps of Engineers used freeboard as a factor of safety in designing levees to account for uncertainties in discharge, stage, and other engineering parameters (such as geotechnical and structural). Now, the Corps of Engineers has adopted a new methodology called Risk Based Analysis (RBA) for formulating flood risk management projects. This method considers all of the same engineering parameters, but accounts for the uncertainties directly in the analysis in lieu of using freeboard. Using RBA, the project's performance will be expressed as the average return period in years of the largest flood that can be accommodated by the plan under study, with a conditional non-exceedance probability of 90%. The concept of freeboard is no longer used.

To use RBA, the hydrologic uncertainty must be characterized. This information is entered into the computer program HEC-FDA (Flood Damage Analysis), which uses Monte Carlo algorithms to quantify the uncertainties. The uncertainty bands used in this program are based on the effective record lengths used to develop the flow frequency estimates. On Soldier Creek, the hydrology was computed using gage statistics from 1929 through 2006. This gives an equivalent record length of 78 years.

HEC-FDA calculates the uncertainty either analytically or graphically. For an analytical computation the log Pearson Type III statistics are inputted directly. A graphical approach is used on regulated streams, when the stream gage records are small or incomplete, or when partial duration data is used. On Soldier Creek, it was possible to use the analytical approach due to the type of stream and the available gage records. For the HEC-FDA analysis, an arbitrary index point was selected at River Mile 4.2, just downstream of the Halfday Creek confluence. To calculate the hydrologic uncertainty at this point, the "compute synthetic statistics" option was used in HEC-FDA. With this option, the program fits a log Pearson Type III curve to the 50, 10, and 1 percent chance exceedance frequency events. The discharge uncertainty was calculated for the reach containing the index point at river mile 4.2.

A-2.2.5 HYDRAULICS

The hydraulic analysis for this report centered on the development of the HEC-RAS computer model for the study reach of Soldier Creek near Topeka, Kansas. For this analysis, version 3.1.3 of the HEC-RAS (River Analysis System) developed by the Hydrologic Engineering Center was used. The computer model was used to generate a series of steady flow water surface profiles based on flood discharges in Table 2-2 above.

<u>Original Design Water Surface Elevations</u>

The elevation of the crown of the existing levee was determined by selecting a design water surface elevation and then adding freeboard of 3 feet to account for uncertainties. The design water surface elevations were determined by using a backwater computer model with the design discharges. The original design discharge for Soldier Creek was 50,000 cfs.

<u>Geometric Data</u>

The computer model required cross section geometry along the length of the study reach. The cross section locations are shown in Plates A2-2-1 and A2-2-2. Field surveys were primarily made at bridges and selected channel locations. Where available, City of Topeka aerial contour maps (2' interval), dated 1995, were used to supplement the field survey data. Beyond the limits of the City mapping, in areas north of Soldier Creek and without a constructed levee, U.S.G.S. mapping and field investigation were used to extend cross sections to completely describe the overbank flow area.

When available, existing bridge plans were obtained and utilized in the model. Bridge plans were collected for U.S. Hwy. 24, U.S. Hwy 75, Topeka Avenue, Union Pacific Railroad, and the Santa Fe Railroad bridges. The levee heights were determined in three ways. First, where available, the top of levee elevation was taken from the cross section surveys (January 1997).

Second, where survey data was not available, the top of levee elevation was interpolated from spot elevation on the City of Topeka aerial contour maps. Third, when necessary, levee elevations were taken from the "Operation and Maintenance Manual" for the Topeka Flood Protection Project[3].

Manning's n-values were estimated through field investigations and limited calibration of the 1993 and 2005 floods. Downstream of the gage, the Manning's n-value for the channel was 0.031. Some portions of the upstream channel were assigned an n-value of 0.040, because of thicker vegetation on the channel banks. Overbank n-values ranged from 0.040 in the well-maintained areas between the channel and the levee, to 0.080 and 0.100 in wooded areas north of the channel in reaches with no north levee.

During a field investigation trip, accumulations of significant quantities of debris were observed at the Santa Fe and Atchison Railroad, Rochester Road, abandoned railroad, Brickyard Road, Menoken Road, and Landon Road bridges. The effects of this debris were not incorporated into the hydraulic model. Other observations made during the field investigation included exposed footings at the U.S. Hwy. 24 and Atchison and Santa Fe Railroad bridges and a scour hole at the bridge at Button Road.

<u>Starting Water Surface Elevations</u>

The starting water surface elevation was determined using the Topeka USGS gage records on Soldier Creek and Kansas River. Plate A2-2-5 shows a plot of the annual instantaneous peak Soldier Creek discharge (between 1960 and 1997) versus the daily discharge on the Kansas River. A curve was drawn through the upper portion of the data points which represents a conservative estimate of the highest discharge on the Kansas River that could reasonably be expected based on the Soldier Creek discharge. Using a rating curve developed from the calculated water surface profiles of the HEC –RAS computer model, the corresponding Kansas River elevations were determined. Table 2-3 lists the corresponding discharges and Soldier Creek starting water surface elevations.

Table 2-3 Soldier Creek Starting Water Surface Elevations

Percent Chance of Exceedance	Soldier Creek Discharge at Topeka Gage (cfs)	Kansas River Discharge at Topeka Gage (cfs)	Soldier Creek Starting Water Surface Elevation (ft)
0.2	56400	209,600	879.13
0.5	44300	179,700	877.73
1	36400	157,400	876.48
2	29400	136,300	875.10
5	21500	108,600	873.10
10	16300	88,100	871.29
20	11800	67,800	868.33
50	6480	39,900	863.18

[3] "Operation and Maintenance Manual for Flood Protection Project, Topeka, Kansas, Volume Eight, Master Flood Emergency Operation and Maintenance Manual." U.S. Army Engineer District, Kansas City, August 1978.

Model Calibration

The model was calibrated to the July 10, 1993 flood event, and the calibration was later checked against data from the October 2, 2005 flood event. The Soldier Creek near Topeka gaging station (06889500) is operated by the U.S.G.S. on the downstream side of Brickyard Road. The gage reading at this site was the only available information to calibrate the model from the 1993 event. The Corps of Engineers provided high water mark data on Soldier Creek from the 1993 flood. However, the high water marks were influenced by backwater from the July 25, 1993 flood event on the Kansas River and could not be used. Previously recorded high water marks under the U.S. 75 Bridge were eliminated when the bridge was replaced in 1995. According to City personnel, during the 1993 flood event, no readings were made using freeboard gages.

The Topeka gage reading on July 10, 1993 was 23.42 feet, M.S.L. and the discharge was 18,900 cfs. With the gage datum of 862.95 feet, M.S.L., the target elevation at the gage was 886.37 feet. Table 2-4 shows the discharges used in the calibration run. These discharges were determined by multiplying the ratio of drainage areas to the discharge at the gage.

Table 2-4 Calibration Discharges on Soldier Creek

Upstream of Messhoss Creek (cfs)	Messhoss Creek to Silver Lake Ditch (cfs)	Silver Lake Ditch to Halfday Creek (cfs)	Halfday Creek to Indian Creek (cfs)	Indian Creek to the Mouth (cfs)
17,100	18,100	18,900	20,500	21,600

The model was started at 865.0 feet, which is the estimated Kansas River elevation based on the daily discharge of 47,300 cfs recorded on July 10, 1993. Only the channel "n" was varied in the calibration runs, because there was no overbank flow at most cross sections. A change in starting river stage of 2 feet at the Kansas River resulted in less than 0.10 feet difference at the gage. Plate A2-2-6 shows the resulting water surface profile. The computed water surface elevation at the gage was 886.38 ft, only 0.01 foot higher than the observed reading. The model is calibrated as well as possible with the limited data available.

During the 2005 flood event, a discharge measurement was made at the gage by the USGS as the event was nearing its peak. The recorded peak discharge at the Topeka gage on Soldier Creek was 47,800 cfs with a stage of 34.78 ft (at elevation 897.73 ft NGVD 1929). Several locations upstream of US Hwy 75 also experienced levee overtopping during the 2005 event, and the simulated overtopping locations from the HEC-RAS model were checked against the actual observed overtopping locations. The profile and overtopping locations of the model were found to be consistent with the observed data.

To test the calibration of the model over a wider range of discharges, water surface profiles were computed for a series of discharges: 50, 20, 10, 4, 2, 1, 0.5 and 0.2-percent chance (2, 5, 10, 25, 50, 100, 200, and 500-year flood events). The starting water surface elevations were taken from Table 2-3 above. The computed water surface elevation at the gage was compared to the expected gage elevation. Table 2-5 lists the discharges and expected water surface elevations. The expected gage elevations were determined from rating curve number 43, in use between 1993 and 1997, which shows the stage versus discharge. The stage was converted to an elevation by simply adding the elevation of the gage datum (862.95 feet, N.G.V.D.) to the stage reading. The largest discharge of the rating curve was 19,000 cfs. Stages larger than that were obtained by extrapolation.

The results show that the computed water surface profiles match the expected gage heights fairly well, except for the largest discharge. At this discharge, water downstream is higher than the levee. Therefore, the computed water surface profile would not necessarily match what would physically happen. This phenomenon is discussed in more detail in the following section.

Table 2-5 Computed Water Surface Elevation versus Expected Gage Height

Percent Chance of Exceedance (%)	Annual Event Discharge (cfs)	Computed Water Surface Elevation (ft)	Soldier Creek near Topeka Rating Curve Elevation (ft)	Difference: Computed vs Expected Gage Elevation (ft)
50	6080	875.57	875.94	-0.37
20	11,800	881.14	881.8	-0.66
10	16,400	884.78	884.96	-0.18
5	21,300	887.83	887.65*	0.18
2	28,300	891.08	890.85*	0.23
1	33,900	893.20	892.95*	0.25
0.5	40,000	895.31	894.95*	0.36
0.2	48,500	899.96	897.95*	2.01
Note: All model elevations are from STA 6.0				

<u>Soldier Creek Existing Condition (Base) Profiles</u>

Once the model was calibrated, the existing conditions water surface profiles were generated using the discharges of Table 2-5 above. Plate A2-2-7 shows the profiles for the 50% non-exceedance probability profiles for the 50, 20, 10, 5, 2, 1, 0.5, 0.2-percent chance (2, 5, 10, 20, 50, 100, 200, and 500-year) flood events. The tabular data is presented in Table 2-6, located at the end of this section.

The HEC-RAS model indicates that none of the Soldier Creek Levee Units in this study begin to physically overtop until the water surface elevation reaches approximately the 50% non-exceedance probability stage for the 0.5% chance exceedance (200-year) event. Discretion should be used when applying profiles higher than the top of the levee. The model used a confined cross sectional area from levee to levee. Essentially, overbank flow beyond the levee height was not taken into consideration. This assumption was made to avoid trying to predict where a levee would fail. Within the Topeka levee systems, there are many different combinations of failure scenarios that could physically occur. Potentially, each could produce a different overbank flow path. HEC-RAS is a one-dimensional steady state model. It is beyond the limitations for HEC-RAS to predict the overbank flow scenarios or to model multi-dimensional flow. Profiles for the rare frequency events that exceed the top of levee are highly speculative and would not necessarily match what would physically happen. These events were produced to formulate frequency-stage curves for economic analyses in the HEC-FDA computer program.

Hydraulic Uncertainty

Uncertainties in computed stage result from two main sources: natural variations in the river and modeling errors. Natural variations include uncertainties in physical factors such as bed forms, debris and other obstructions, channel scour or deposition, sediment transport, and waves. Modeling uncertainty includes factors such as inexact geometry and loss coefficients, variation in hydraulic roughness with season, and error in setting high water marks (EM 1110-2-1619).

In Risk Based Analysis, the stage uncertainty is express as standard deviation (in feet). The total standard deviation depends on the standard deviation based on natural variations and the standard deviation based on model errors according to the formula below:

$$\text{Total Standard Deviation} = \sqrt{S_{natural}{}^{2} + S_{model}{}^{2}}$$

where $S_{natural}$ = standard deviation based on natural variations
S_{model} = standard deviation based on modeling uncertainties

For a gaged reached, $S_{natural}$ is calculated by comparing observed data with the latest rating curve at the gage in the study reach. To avoid potential problems due to shifts in the rating curve over time, only observed data going back to 1990 was used. Only data values for bank full discharges and greater were analyzed. The following formula is used to calculate $S_{natural}$.

$$S_{natural} = \sqrt{\frac{\sum(X-M)^2}{(N-1)}}$$

where: X=Stage corresponding to measured Q
M=best fit curve estimate of stage corresponding to Q
N=number of stage-discharge observations in the range being analyzed

The best fit curve through data from the rating curve is defined by the equation,
Stage = -4.638E-8*Q^2 + 0.001957*Q + 865.55 where Q is the measured discharge. The standard deviation based on historical data and gage readings, $S_{natural}$, was computed as 0.75 feet.

Table 5-2 in EM 1110-2-1619 quantifies S_{model} based on the quality of topographic data and the reliability of the Manning's n-value. A standard deviation of 1.5 feet was chosen since some of the cross-sections were based on mapping and the Manning's n-values were assumed to have "poor" reliability (due to the limited amount of calibration data available).

Once $S_{natural}$ and S_{model} are known, a total standard deviation can be computed. For this study a total standard deviation of 1.68 was computed for the entire discharge set.

A-2.2.6 SUMMARY

First, a hydrologic analysis was completed to determine the expected discharges at the flood reduction works based upon statistical analyses of two stream flow gages in the watershed. Next, a hydraulic investigation was conducted on Soldier Creek using the HEC-RAS computer software developed by the Hydrologic Engineering Center, U.S. Army Corps of Engineers. The program was used to calculate water surface profiles on the first ten miles of Soldier Creek in Topeka, Kansas. The model was calibrated using the Topeka gage height during the 1993 flood and then checked against observed stages from the 2005 flood event. Water surface profiles were then generated for eight different discharge events. These include the 50, 20, 10, 5, 2, 1, 0.5, and 0.2-percent chance flood events. The model shows that the existing levees are not overtopped until the 0.5% chance exceedance (200-year) flood event. Last, the uncertainty in both stage and discharge were calculated. The standard deviation of stage is 1.68 feet.

Table 2-6 Soldier Creek Existing Conditions Water Surface Profiles

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
0.334	10% (10-yr)	17800	871.29	871.73	0.000371	5.31	3355.23	237.39	0.25
0.334	2% (50-yr)	31800	875.1	875.94	0.000536	7.39	4435.05	311.3	0.31
0.334	1% (100-yr)	38700	876.48	877.53	0.000611	8.3	4871.4	321.09	0.33
0.334	0.5% (200-yr)	46000	877.73	879.02	0.00069	9.2	5278.3	329.95	0.36
0.334	0.2% (500-yr)	56400	879.13	880.79	0.000819	10.47	5747.19	339.88	0.39
0.334	0.133% (750-yr)	61100	879.79	881.6	0.000864	10.97	5973.04	344.56	0.41
0.334	0.1% (1000-yr)	63900	880.03	881.96	0.000909	11.34	6055.96	346.26	0.42
0.334	0.04% (2500-yr)	76500	881.35	883.77	0.001063	12.73	6519.18	355.62	0.46
0.391	10% (10-yr)	17800	871.38	871.84	0.000325	5.4	3298.1	202.27	0.24
0.391	2% (50-yr)	31800	875.2	876.13	0.000564	7.74	4107.01	221.29	0.32
0.391	1% (100-yr)	38700	876.58	877.77	0.000684	8.76	4415.77	228.33	0.35
0.391	0.5% (200-yr)	46000	877.82	879.3	0.000813	9.78	4703.15	237.88	0.38
0.391	0.2% (500-yr)	56400	879.19	881.15	0.000988	11.22	5028.95	247.13	0.43
0.391	0.133% (750-yr)	61100	879.83	881.99	0.001051	11.8	5180.52	251.09	0.44
0.391	0.1% (1000-yr)	63900	880.06	882.38	0.00111	12.21	5235.66	252.53	0.46
0.391	0.04% (2500-yr)	76500	881.32	884.29	0.001324	13.83	5533.37	260.31	0.5
0.3945		Bridge							
0.398	10% (10-yr)	17800	871.4	871.85	0.000324	5.39	3301.86	202.37	0.24
0.398	2% (50-yr)	31800	875.25	876.18	0.00056	7.72	4118.38	221.54	0.32
0.398	1% (100-yr)	38700	876.65	877.83	0.000677	8.73	4432.75	228.91	0.35
0.398	0.5% (200-yr)	46000	877.92	879.39	0.000802	9.73	4727.63	238.68	0.38
0.398	0.2% (500-yr)	56400	879.35	881.28	0.000964	11.14	5066.35	248.11	0.42
0.398	0.133% (750-yr)	61100	880.02	882.15	0.001021	11.7	5225.36	252.27	0.44
0.398	0.1% (1000-yr)	63900	880.27	882.54	0.001076	12.1	5284.46	253.81	0.45
0.398	0.04% (2500-yr)	76500	881.6	884.5	0.001272	13.67	5599.63	262.05	0.49
0.424	10% (10-yr)	17800	871.45	871.9	0.000313	5.38	3325.6	223.99	0.23
0.424	2% (50-yr)	31800	875.36	876.26	0.000483	7.66	4376.59	298.46	0.3
0.424	1% (100-yr)	38700	876.8	877.93	0.000559	8.62	4813.19	307.44	0.32
0.424	0.5% (200-yr)	46000	878.13	879.5	0.000636	9.56	5226.45	315.71	0.35
0.424	0.2% (500-yr)	56400	879.65	881.41	0.000755	10.87	5716.13	325.23	0.39
0.424	0.133% (750-yr)	61100	880.38	882.29	0.000796	11.38	5952.62	329.73	0.4
0.424	0.1% (1000-yr)	63900	880.66	882.7	0.000835	11.74	6047.12	331.51	0.41
0.424	0.04% (2500-yr)	76500	882.18	884.7	0.000966	13.12	6556.61	340.95	0.44
0.461	10% (10-yr)	17800	871.42	872.01	0.000422	6.18	2878.21	172.51	0.27
0.461	2% (50-yr)	31800	875.24	876.47	0.000746	8.91	3569.35	189.21	0.36
0.461	1% (100-yr)	38700	876.62	878.2	0.000909	10.09	3834.18	195.19	0.4
0.461	0.5% (200-yr)	46000	877.87	879.84	0.001083	11.27	4081.35	200.62	0.44
0.461	0.2% (500-yr)	56400	879.26	881.85	0.001355	12.92	4366.03	206.7	0.5
0.461	0.133% (750-yr)	61100	879.92	882.78	0.001462	13.57	4503.12	209.56	0.52
0.461	0.1% (1000-yr)	63900	880.16	883.22	0.00155	14.03	4554.04	210.61	0.53
0.461	0.04% (2500-yr)	76500	881.47	885.36	0.00189	15.83	4833.58	216.3	0.59
0.47		Bridge							

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
0.479	10% (10-yr)	17800	871.49	872.07	0.000417	6.16	2889.8	172.81	0.27
0.479	2% (50-yr)	31800	875.42	876.63	0.000727	8.82	3604.05	190	0.36
0.479	1% (100-yr)	38700	876.88	878.42	0.000876	9.96	3885.62	196.34	0.39
0.479	0.5% (200-yr)	46000	878.23	880.13	0.001031	11.07	4154.81	202.21	0.43
0.479	0.2% (500-yr)	56400	879.82	882.28	0.001262	12.59	4481.4	209.11	0.48
0.479	0.133% (750-yr)	61100	880.57	883.26	0.001348	13.17	4639.34	212.37	0.5
0.479	0.1% (1000-yr)	63900	880.88	883.75	0.001417	13.58	4706.7	213.74	0.51
0.479	0.04% (2500-yr)	76500	882.53	886.08	0.001665	15.1	5064.96	220.9	0.56
0.507	10% (10-yr)	17800	871.55	872.14	0.00039	6.13	2964.42	255.32	0.26
0.507	2% (50-yr)	31800	875.66	876.74	0.000569	8.52	4159.16	307.18	0.32
0.507	1% (100-yr)	38700	877.24	878.55	0.00064	9.48	4653.62	318.5	0.35
0.507	0.5% (200-yr)	46000	878.74	880.3	0.000706	10.38	5140.42	329.27	0.37
0.507	0.2% (500-yr)	56400	880.58	882.48	0.000796	11.57	5759.94	342.48	0.4
0.507	0.133% (750-yr)	61100	881.46	883.48	0.000821	12.01	6061.43	348.73	0.4
0.507	0.1% (1000-yr)	63900	881.86	883.98	0.000847	12.32	6201.83	351.6	0.41
0.507	0.04% (2500-yr)	76500	883.89	886.37	0.000914	13.41	6932.13	366.18	0.43
0.602	10% (10-yr)	17800	871.74	872.35	0.00042	6.28	2870.08	249.27	0.27
0.602	2% (50-yr)	31800	875.91	877.05	0.000605	8.71	4034.16	294.02	0.33
0.602	1% (100-yr)	38700	877.52	878.9	0.000678	9.67	4516.17	305.08	0.35
0.602	0.5% (200-yr)	46000	879.05	880.68	0.000746	10.59	4990.6	315.59	0.38
0.602	0.2% (500-yr)	56400	880.93	882.92	0.00084	11.8	5596.08	328.52	0.4
0.602	0.133% (750-yr)	61100	881.81	883.93	0.000866	12.25	5887.8	334.56	0.41
0.602	0.1% (1000-yr)	63900	882.22	884.45	0.000893	12.56	6026.09	337.39	0.42
0.602	0.04% (2500-yr)	76500	884.28	886.87	0.000962	13.68	6734.14	351.53	0.44
0.719	10% (10-yr)	17800	872.08	872.59	0.000341	5.74	3125.33	211.02	0.24
0.719	2% (50-yr)	31800	876.43	877.4	0.000493	8.01	4331.21	365.62	0.3
0.719	1% (100-yr)	38700	878.14	879.3	0.000542	8.84	4960.59	370.61	0.32
0.719	0.5% (200-yr)	46000	879.77	881.12	0.000584	9.59	5571.81	375.39	0.34
0.719	0.2% (500-yr)	56400	881.82	883.41	0.000637	10.57	6345.5	381.35	0.36
0.719	0.133% (750-yr)	61100	882.76	884.44	0.000651	10.92	6705.36	384.1	0.36
0.719	0.1% (1000-yr)	63900	883.22	884.97	0.000666	11.17	6882.29	385.44	0.37
0.719	0.04% (2500-yr)	76500	885.43	887.44	0.000704	12.07	7743.73	393.89	0.38
0.837	10% (10-yr)	17800	872.28	872.82	0.000363	5.85	3058.25	200.15	0.25
0.837	2% (50-yr)	31800	876.72	877.72	0.000515	8.13	4244.2	365.06	0.31
0.837	1% (100-yr)	38700	878.46	879.65	0.000562	8.95	4883.49	370.22	0.33
0.837	0.5% (200-yr)	46000	880.12	881.49	0.000601	9.69	5503.8	375.16	0.34
0.837	0.2% (500-yr)	56400	882.2	883.82	0.000652	10.64	6290.38	381.32	0.36
0.837	0.133% (750-yr)	61100	883.15	884.86	0.000664	11	6653.58	384.14	0.37
0.837	0.1% (1000-yr)	63900	883.62	885.39	0.000678	11.24	6834.42	385.53	0.37
0.837	0.04% (2500-yr)	76500	885.86	887.88	0.000712	12.11	7706.28	392	0.39
0.883	10% (10-yr)	17800	872.36	872.92	0.000421	6	2968.71	183.69	0.26
0.883	2% (50-yr)	31800	876.8	877.89	0.000621	8.37	3817.1	198.57	0.33
0.883	1% (100-yr)	38700	878.49	879.86	0.000702	9.38	4167.39	215.41	0.35

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
0.883	0.5% (200-yr)	46000	880.09	881.75	0.00078	10.36	4517.31	223.34	0.38
0.883	0.2% (500-yr)	56400	882.05	884.15	0.000889	11.68	4964.95	233.1	0.41
0.883	0.133% (750-yr)	61100	882.94	885.22	0.000926	12.2	5173.77	237.52	0.42
0.883	0.1% (1000-yr)	63900	883.37	885.78	0.000957	12.53	5277.34	239.68	0.43
0.883	0.04% (2500-yr)	76500	885.44	888.36	0.001055	13.83	5781.2	247.63	0.46
0.884		Bridge							
0.885	10% (10-yr)	17800	872.46	873.01	0.000414	5.96	2986.22	184.01	0.26
0.885	2% (50-yr)	31800	877.02	878.09	0.0006	8.28	3861.87	199.33	0.32
0.885	1% (100-yr)	38700	878.81	880.13	0.000669	9.24	4235.19	216.97	0.35
0.885	0.5% (200-yr)	46000	880.5	882.1	0.000735	10.18	4609.87	225.4	0.37
0.885	0.2% (500-yr)	56400	882.62	884.62	0.000823	11.4	5099.52	235.96	0.4
0.885	0.133% (750-yr)	61100	883.58	885.75	0.000851	11.88	5327.93	240.73	0.4
0.885	0.1% (1000-yr)	63900	884.07	886.34	0.000874	12.19	5445.46	242.84	0.41
0.885	0.04% (2500-yr)	76500	885.91	888.73	0.000995	13.59	5897.95	248.94	0.44
0.914	10% (10-yr)	17800	872.42	873.13	0.000543	6.77	2653.28	191.66	0.3
0.914	2% (50-yr)	31800	876.97	878.25	0.000745	9.16	3651	244.18	0.37
0.914	1% (100-yr)	38700	878.76	880.3	0.000804	10.09	4103.72	261.11	0.39
0.914	0.5% (200-yr)	46000	880.48	882.28	0.000857	10.97	4564.84	275.39	0.4
0.914	0.2% (500-yr)	56400	882.64	884.8	0.000924	12.09	5173.61	287.17	0.42
0.914	0.133% (750-yr)	61100	883.62	885.92	0.00094	12.51	5458	292.51	0.43
0.914	0.1% (1000-yr)	63900	884.12	886.52	0.000958	12.79	5604.44	295.22	0.44
0.914	0.04% (2500-yr)	76500	886.03	888.9	0.001057	14.07	6176.49	301.03	0.46
1.057	10% (10-yr)	17800	872.82	873.56	0.00058	6.92	2590.69	187.67	0.31
1.057	2% (50-yr)	31800	877.52	878.83	0.00077	9.25	3607.39	242.49	0.37
1.057	1% (100-yr)	38700	879.36	880.92	0.000823	10.17	4068.89	259.85	0.39
1.057	0.5% (200-yr)	46000	881.12	882.93	0.00087	11.02	4539.33	274.88	0.41
1.057	0.2% (500-yr)	56400	883.33	885.5	0.000929	12.11	5162.57	286.96	0.43
1.057	0.133% (750-yr)	61100	884.33	886.63	0.000943	12.53	5450.99	292.38	0.43
1.057	0.1% (1000-yr)	63900	884.84	887.24	0.000959	12.8	5601.86	295.18	0.44
1.057	0.04% (2500-yr)	76500	886.86	889.69	0.001042	14	6205.03	301.13	0.46
1.199	10% (10-yr)	17800	873.29	873.99	0.000559	6.71	2653.82	175.11	0.3
1.199	2% (50-yr)	31800	878.22	879.41	0.000736	8.81	3743.22	257.78	0.36
1.199	1% (100-yr)	38700	880.13	881.54	0.000769	9.62	4251.29	273.68	0.37
1.199	0.5% (200-yr)	46000	881.96	883.58	0.000798	10.38	4766.18	288.91	0.39
1.199	0.2% (500-yr)	56400	884.28	886.19	0.000836	11.34	5458.42	308.21	0.4
1.199	0.133% (750-yr)	61100	885.31	887.33	0.000842	11.7	5780.61	314.53	0.41
1.199	0.1% (1000-yr)	63900	885.86	887.95	0.000851	11.92	5952.96	316.77	0.41
1.199	0.04% (2500-yr)	76500	888.01	890.46	0.000906	12.98	6645.35	323.76	0.43
1.342	10% (10-yr)	17800	873.71	874.43	0.000591	6.85	2598.47	173.32	0.31
1.342	2% (50-yr)	31800	878.76	879.98	0.000762	8.91	3695.73	256.24	0.37
1.342	1% (100-yr)	38700	880.7	882.13	0.000791	9.71	4207.56	272.35	0.38
1.342	0.5% (200-yr)	46000	882.55	884.2	0.000817	10.45	4726.24	287.76	0.39
1.342	0.2% (500-yr)	56400	884.9	886.83	0.00085	11.4	5424.72	307.3	0.41

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
1.342	0.133% (750-yr)	61100	885.93	887.98	0.000855	11.76	5747.43	314.1	0.41
1.342	0.1% (1000-yr)	63900	886.49	888.6	0.000863	11.98	5921.79	316.37	0.41
1.342	0.04% (2500-yr)	76500	888.69	891.15	0.000912	13	6628.75	323.7	0.43
1.372	10% (10-yr)	17800	873.81	874.53	0.000581	6.8	2616.37	173.9	0.31
1.372	2% (50-yr)	31800	878.87	880.11	0.000758	8.92	3600.25	213.06	0.37
1.372	1% (100-yr)	38700	880.79	882.27	0.000804	9.81	4020.42	226.03	0.38
1.372	0.5% (200-yr)	46000	882.61	884.36	0.000848	10.67	4442.86	238.35	0.4
1.372	0.2% (500-yr)	56400	884.9	887.03	0.00091	11.8	5006.36	253.86	0.42
1.372	0.133% (750-yr)	61100	885.9	888.19	0.00093	12.25	5264.43	260.65	0.43
1.372	0.1% (1000-yr)	63900	886.43	888.83	0.000946	12.53	5404.07	264.26	0.43
1.372	0.04% (2500-yr)	76500	888.54	891.43	0.001034	13.8	5980.02	283.32	0.46
1.375		Bridge							
1.378	10% (10-yr)	17800	873.86	874.58	0.000575	6.78	2626.05	174.22	0.31
1.378	2% (50-yr)	31800	878.99	880.21	0.000742	8.87	3625.07	213.85	0.36
1.378	1% (100-yr)	38700	880.93	882.4	0.000784	9.74	4053.82	227.02	0.38
1.378	0.5% (200-yr)	46000	882.79	884.51	0.000825	10.58	4485.97	239.57	0.39
1.378	0.2% (500-yr)	56400	885.12	887.21	0.000881	11.68	5064.47	255.41	0.41
1.378	0.133% (750-yr)	61100	886.15	888.39	0.000899	12.12	5329.13	262.33	0.42
1.378	0.1% (1000-yr)	63900	886.69	889.04	0.000914	12.4	5473.22	266.03	0.43
1.378	0.04% (2500-yr)	76500	888.71	891.56	0.001013	13.7	6027.04	284.67	0.45
1.389	10% (10-yr)	17800	873.9	874.61	0.000571	6.76	2632.79	174.44	0.31
1.389	2% (50-yr)	31800	879.08	880.25	0.000717	8.74	3778	258.9	0.36
1.389	1% (100-yr)	38700	881.08	882.45	0.000741	9.51	4312.09	275.53	0.37
1.389	0.5% (200-yr)	46000	883	884.57	0.000761	10.23	4856.31	291.5	0.38
1.389	0.2% (500-yr)	56400	885.44	887.29	0.000788	11.14	5594.3	311.85	0.39
1.389	0.133% (750-yr)	61100	886.53	888.47	0.00079	11.48	5934.23	316.53	0.4
1.389	0.1% (1000-yr)	63900	887.11	889.12	0.000797	11.69	6118.96	318.92	0.4
1.389	0.04% (2500-yr)	76500	889.29	891.66	0.000855	12.76	6822.94	324.41	0.42
1.535	10% (10-yr)	17800	874.37	875.04	0.000521	6.53	2724.41	177.37	0.29
1.535	2% (50-yr)	31800	879.7	880.78	0.000634	8.41	3952.87	264.46	0.34
1.535	1% (100-yr)	38700	881.73	883	0.000657	9.16	4507.3	281.36	0.35
1.535	0.5% (200-yr)	46000	883.68	885.14	0.000678	9.86	5071.41	297.58	0.36
1.535	0.2% (500-yr)	56400	886.17	887.88	0.000704	10.75	5836.65	315.26	0.37
1.535	0.133% (750-yr)	61100	887.26	889.06	0.000709	11.08	6182.09	319.73	0.38
1.535	0.1% (1000-yr)	63900	887.85	889.72	0.000715	11.3	6371.4	322.16	0.38
1.535	0.04% (2500-yr)	76500	890.09	892.3	0.000768	12.34	7100.35	325.42	0.4
1.681	10% (10-yr)	17800	874.76	875.47	0.000569	6.78	2624.91	169.64	0.3
1.681	2% (50-yr)	31800	880.18	881.31	0.000702	8.6	4002.61	341.45	0.35
1.681	1% (100-yr)	38700	882.26	883.53	0.000701	9.22	4724.76	354.03	0.36
1.681	0.5% (200-yr)	46000	884.28	885.67	0.000695	9.76	5452.9	366.28	0.36
1.681	0.2% (500-yr)	56400	886.87	888.43	0.000689	10.44	6423.12	382	0.36
1.681	0.133% (750-yr)	61100	888	889.62	0.000683	10.7	6857.53	388.83	0.37
1.681	0.1% (1000-yr)	63900	888.61	890.28	0.000685	10.87	7097.66	394.71	0.37

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
1.681	0.04% (2500-yr)	76500	891.04	892.9	0.0007	11.63	8067.45	401.2	0.38
1.867	10% (10-yr)	17200	875.44	875.99	0.000451	5.92	2904.86	196.21	0.27
1.867	2% (50-yr)	30400	881.08	881.91	0.000494	7.35	4343.15	328.52	0.3
1.867	1% (100-yr)	36800	883.19	884.14	0.000495	7.89	5061.74	347.38	0.3
1.867	0.5% (200-yr)	43500	885.24	886.28	0.000492	8.36	5779.96	355.07	0.31
1.867	0.2% (500-yr)	53200	887.85	889.04	0.000493	8.99	6721.21	364.9	0.31
1.867	0.133% (750-yr)	58100	888.96	890.23	0.000503	9.33	7127.63	369.07	0.32
1.867	0.1% (1000-yr)	60800	889.58	890.89	0.000505	9.5	7357.38	371.4	0.32
1.867	0.04% (2500-yr)	72800	892.03	893.53	0.000524	10.24	8274.39	374	0.33
2.053	10% (10-yr)	17200	875.87	876.42	0.000432	5.92	2904.4	189.64	0.27
2.053	2% (50-yr)	30400	881.56	882.42	0.000512	7.44	4143.16	289.43	0.3
2.053	1% (100-yr)	36800	883.65	884.65	0.00052	8.04	4808.73	336.71	0.31
2.053	0.5% (200-yr)	43500	885.69	886.8	0.00052	8.55	5511.4	351.07	0.31
2.053	0.2% (500-yr)	53200	888.3	889.56	0.000522	9.21	6444.16	364.12	0.32
2.053	0.133% (750-yr)	58100	889.41	890.76	0.000531	9.55	6852.76	369.7	0.33
2.053	0.1% (1000-yr)	60800	890.03	891.42	0.000534	9.72	7083.57	372.81	0.33
2.053	0.04% (2500-yr)	72800	892.5	894.09	0.000552	10.47	8006.83	374	0.34
2.239	10% (10-yr)	17200	876.28	876.86	0.000442	6.08	2830.68	180.26	0.27
2.239	2% (50-yr)	30400	882.04	882.96	0.000551	7.71	3942.05	206.16	0.31
2.239	1% (100-yr)	36800	884.13	885.23	0.0006	8.39	4384.83	222	0.33
2.239	0.5% (200-yr)	43500	886.14	887.39	0.00061	9.01	4934.41	321.09	0.34
2.239	0.2% (500-yr)	53200	888.72	890.17	0.000617	9.75	5790.88	341.76	0.34
2.239	0.133% (750-yr)	58100	889.83	891.39	0.000627	10.11	6176.08	350.67	0.35
2.239	0.1% (1000-yr)	60800	890.45	892.06	0.000629	10.29	6394.86	355.62	0.35
2.239	0.04% (2500-yr)	72800	892.92	894.74	0.000647	11.07	7309.51	374	0.36
2.277	10% (10-yr)	17200	876.38	876.94	0.000435	6.04	2848.13	180.85	0.27
2.277	2% (50-yr)	30400	882.16	883.07	0.000541	7.66	3969.27	207.1	0.31
2.277	1% (100-yr)	36800	884.27	885.35	0.000591	8.33	4416.53	216.69	0.33
2.277	0.5% (200-yr)	43500	886.27	887.52	0.000637	8.95	4860.64	226.44	0.34
2.277	0.2% (500-yr)	53200	888.84	890.31	0.000699	9.75	5457.52	239.77	0.36
2.277	0.133% (750-yr)	58100	889.94	891.54	0.000734	10.15	5724.82	245.5	0.37
2.277	0.1% (1000-yr)	60800	890.55	892.21	0.00075	10.35	5875.99	248.69	0.38
2.277	0.04% (2500-yr)	72800	892.98	894.93	0.000817	11.21	6495.55	267.64	0.4
2.2805		Bridge							
2.284	10% (10-yr)	17200	876.44	877	0.000431	6.02	2859.2	181.13	0.27
2.284	2% (50-yr)	30400	882.26	883.16	0.000534	7.62	3990.68	207.57	0.31
2.284	1% (100-yr)	36800	884.39	885.46	0.000581	8.28	4443.68	217.26	0.32
2.284	0.5% (200-yr)	43500	886.42	887.65	0.000626	8.89	4894.02	227.2	0.34
2.284	0.2% (500-yr)	53200	889.01	890.47	0.000685	9.67	5500.53	240.7	0.36
2.284	0.133% (750-yr)	58100	890.13	891.71	0.000718	10.06	5773.32	246.53	0.37
2.284	0.1% (1000-yr)	60800	890.75	892.39	0.000733	10.26	5927.38	249.76	0.37
2.284	0.04% (2500-yr)	72800	894.85	896.52	0.000642	10.43	6989	343.37	0.35

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
2.299	10% (10-yr)	17200	876.48	877.04	0.000427	6	2865.69	181.13	0.27
2.299	2% (50-yr)	30400	882.31	883.21	0.00053	7.6	3998.54	207.39	0.31
2.299	1% (100-yr)	36800	884.44	885.51	0.00057	8.27	4456.53	238.36	0.32
2.299	0.5% (200-yr)	43500	886.48	887.7	0.000578	8.86	5046.74	323.88	0.33
2.299	0.2% (500-yr)	53200	889.13	890.52	0.000581	9.56	5930.78	345.02	0.33
2.299	0.133% (750-yr)	58100	890.28	891.77	0.000589	9.91	6333.14	354.23	0.34
2.299	0.1% (1000-yr)	60800	890.92	892.45	0.00059	10.08	6561.09	359.34	0.34
2.299	0.04% (2500-yr)	72800	895.1	896.6	0.00049	10.11	8124.26	374	0.32
2.479	10% (10-yr)	17200	876.88	877.44	0.000428	6.03	2851.79	179.38	0.27
2.479	2% (50-yr)	30400	882.81	883.71	0.000525	7.61	3993.58	205.79	0.3
2.479	1% (100-yr)	36800	884.99	886.05	0.000571	8.26	4452.53	215.49	0.32
2.479	0.5% (200-yr)	43500	887.04	888.26	0.000596	8.87	4919.71	256.26	0.33
2.479	0.2% (500-yr)	53200	889.66	891.1	0.000607	9.65	5713.5	348.18	0.34
2.479	0.133% (750-yr)	58100	890.82	892.35	0.000615	10.01	6126.83	360	0.35
2.479	0.1% (1000-yr)	60800	891.46	893.04	0.000616	10.18	6356.86	360	0.35
2.479	0.04% (2500-yr)	72800	895.53	897.1	0.000519	10.27	7822.84	360	0.33
2.527	10% (10-yr)	17200	876.99	877.55	0.000419	5.99	2871.47	179.44	0.26
2.527	2% (50-yr)	30400	882.95	883.84	0.000514	7.57	4016.82	205.19	0.3
2.527	1% (100-yr)	36800	885.14	886.19	0.000558	8.22	4476.93	214.67	0.32
2.527	0.5% (200-yr)	43500	887.2	888.41	0.000599	8.83	4927.76	223.56	0.33
2.527	0.2% (500-yr)	53200	889.82	891.26	0.000654	9.62	5528.59	234.9	0.35
2.527	0.133% (750-yr)	58100	890.96	892.52	0.00067	10.02	5797.99	239.83	0.36
2.527	0.1% (1000-yr)	60800	891.59	893.21	0.000674	10.23	5946.08	243.75	0.36
2.527	0.04% (2500-yr)	72800	895.57	897.28	0.000597	10.51	7118.09	382.7	0.35
2.53		Bridge							
2.533	10% (10-yr)	17200	877.06	877.61	0.000414	5.96	2883.76	179.74	0.26
2.533	2% (50-yr)	30400	883.07	883.95	0.000506	7.52	4041.14	205.7	0.3
2.533	1% (100-yr)	36800	885.29	886.32	0.000548	8.16	4507.99	215.29	0.31
2.533	0.5% (200-yr)	43500	887.37	888.56	0.000587	8.76	4966.21	224.31	0.33
2.533	0.2% (500-yr)	53200	890.03	891.44	0.000638	9.54	5577.31	235.79	0.35
2.533	0.133% (750-yr)	58100	891.12	892.66	0.000655	9.96	5835.98	240.78	0.35
2.533	0.1% (1000-yr)	60800	891.78	893.38	0.000658	10.15	5990.13	244.94	0.36
2.533	0.04% (2500-yr)	72800	897.28	898.76	0.000485	9.85	7770.18	382.7	0.31
2.546	10% (10-yr)	17200	877.09	877.64	0.000412	5.95	2889.86	180.1	0.26
2.546	2% (50-yr)	30400	883.11	883.98	0.000503	7.5	4052.58	206.51	0.3
2.546	1% (100-yr)	36800	885.33	886.36	0.000545	8.14	4522.44	216.27	0.31
2.546	0.5% (200-yr)	43500	887.42	888.6	0.000573	8.73	4999.26	247.19	0.33
2.546	0.2% (500-yr)	53200	890.09	891.48	0.000602	9.49	5730.17	307.16	0.34
2.546	0.133% (750-yr)	58100	891.21	892.71	0.000613	9.87	6135.8	418.9	0.34
2.546	0.1% (1000-yr)	60800	891.89	893.43	0.00061	10.02	6442.07	486.55	0.34
2.546	0.04% (2500-yr)	72800	897.59	898.82	0.000399	9.23	9281.02	498	0.29
2.66	10% (10-yr)	17200	877.35	877.89	0.000394	5.86	2936.92	181.25	0.26
2.66	2% (50-yr)	30400	883.43	884.28	0.000481	7.38	4120.09	207.94	0.29

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
2.66	1% (100-yr)	36800	885.69	886.68	0.00052	8	4599.79	217.84	0.31
2.66	0.5% (200-yr)	43500	887.8	888.94	0.000544	8.58	5094.28	254.65	0.32
2.66	0.2% (500-yr)	53200	890.5	891.84	0.000568	9.32	5863.09	347.76	0.33
2.66	0.133% (750-yr)	58100	891.63	893.07	0.000578	9.69	6321.11	461.02	0.33
2.66	0.1% (1000-yr)	60800	892.31	893.79	0.000575	9.83	6653.45	498	0.34
2.66	0.04% (2500-yr)	72800	897.86	899.06	0.000386	9.13	9415.22	498	0.28
2.691	10% (10-yr)	17200	877.44	877.95	0.00038	5.77	2981.51	183.42	0.25
2.691	2% (50-yr)	30400	883.54	884.36	0.000466	7.26	4188.51	211.88	0.29
2.691	1% (100-yr)	36800	885.81	886.77	0.000503	7.86	4681.03	222.45	0.3
2.691	0.5% (200-yr)	43500	887.95	889.04	0.000505	8.39	5433.22	503.21	0.31
2.691	0.2% (500-yr)	53200	890.79	891.95	0.000477	8.81	6963.37	556.94	0.3
2.691	0.133% (750-yr)	58100	892	893.18	0.00047	9.01	7639.63	562.98	0.3
2.691	0.1% (1000-yr)	60800	892.71	893.9	0.00046	9.07	8042.3	563	0.3
2.691	0.04% (2500-yr)	72800	898.23	899.14	0.000299	8.25	11148.97	563	0.25
2.837	10% (10-yr)	17200	877.73	878.29	0.000458	6.01	2863.98	190.29	0.27
2.837	2% (50-yr)	30400	883.9	884.73	0.000487	7.32	4259.36	325.14	0.29
2.837	1% (100-yr)	36800	886.22	887.15	0.000474	7.79	5037.45	342.92	0.3
2.837	0.5% (200-yr)	43500	888.4	889.41	0.000464	8.21	5800.82	357.58	0.3
2.837	0.2% (500-yr)	53200	891.17	892.31	0.000458	8.78	6818.19	376.98	0.3
2.837	0.133% (750-yr)	58100	892.34	893.55	0.000463	9.09	7266.48	385.8	0.3
2.837	0.1% (1000-yr)	60800	893.04	894.27	0.000461	9.21	7534.14	385.8	0.3
2.837	0.04% (2500-yr)	72800	898.34	899.44	0.00034	8.87	9581.75	385.8	0.27
2.954	10% (10-yr)	17200	878.01	878.55	0.000385	5.89	2919.34	175.17	0.25
2.954	2% (50-yr)	30400	884.18	885.02	0.000437	7.39	4289.43	292.82	0.28
2.954	1% (100-yr)	36800	886.48	887.44	0.000441	7.95	4983.13	309.1	0.29
2.954	0.5% (200-yr)	43500	888.64	889.71	0.000444	8.46	5667.41	324.36	0.29
2.954	0.2% (500-yr)	53200	891.39	892.62	0.000452	9.13	6586.23	342	0.3
2.954	0.133% (750-yr)	58100	892.56	893.86	0.000461	9.48	6984.86	342	0.31
2.954	0.1% (1000-yr)	60800	893.25	894.59	0.000462	9.63	7219.57	342	0.31
2.954	0.04% (2500-yr)	72800	898.46	899.71	0.00036	9.44	9001.85	342	0.28
2.992	10% (10-yr)	17200	878.09	878.62	0.000381	5.86	2936.9	176.37	0.25
2.992	2% (50-yr)	30400	884.27	885.11	0.000487	7.36	4129.89	211.73	0.29
2.992	1% (100-yr)	36800	886.57	887.55	0.000533	7.94	4633.45	227.03	0.31
2.992	0.5% (200-yr)	43500	888.71	889.83	0.000572	8.47	5136.53	241.35	0.32
2.992	0.2% (500-yr)	53200	891.45	892.74	0.000618	9.14	5821.06	259.13	0.34
2.992	0.133% (750-yr)	58100	892.6	894	0.000624	9.49	6122.17	263.34	0.34
2.992	0.1% (1000-yr)	60800	893.28	894.73	0.000622	9.66	6301.43	265.81	0.35
2.992	0.04% (2500-yr)	72800	898.45	899.83	0.000461	9.46	8001.43	379.2	0.31
2.9955		Bridge							
2.999	10% (10-yr)	17200	878.14	878.67	0.000378	5.84	2945.43	176.62	0.25
2.999	2% (50-yr)	30400	884.35	885.19	0.000482	7.33	4147.05	212.27	0.29
2.999	1% (100-yr)	36800	886.66	887.63	0.000527	7.91	4655.28	227.67	0.31
2.999	0.5% (200-yr)	43500	888.82	889.93	0.000565	8.42	5163.22	242.09	0.32

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
2.999	0.2% (500-yr)	53200	891.54	892.83	0.000609	9.1	5845.32	259.48	0.34
2.999	0.133% (750-yr)	58100	894.3	895.52	0.000494	8.85	6574.65	269.55	0.31
2.999	0.1% (1000-yr)	60800	894.58	895.88	0.000523	9.17	6646.79	270.54	0.32
2.999	0.04% (2500-yr)	72800	899.8	901.05	0.000395	9.02	8513.97	379.2	0.29
3.018	10% (10-yr)	17200	878.18	878.71	0.000375	5.83	2949.51	176.05	0.25
3.018	2% (50-yr)	30400	884.41	885.23	0.000421	7.31	4357.6	294.46	0.28
3.018	1% (100-yr)	36800	886.74	887.68	0.000427	7.88	5061.94	310.89	0.28
3.018	0.5% (200-yr)	43500	888.91	889.98	0.000434	8.42	5755.45	326.27	0.29
3.018	0.2% (500-yr)	53200	891.65	892.88	0.000449	9.15	6671.75	342	0.3
3.018	0.133% (750-yr)	58100	894.41	895.57	0.00038	8.95	7615.94	342	0.28
3.018	0.1% (1000-yr)	60800	894.69	895.93	0.000402	9.26	7711.77	342	0.29
3.018	0.04% (2500-yr)	72800	899.87	901.09	0.000328	9.26	9486.19	342	0.27
3.173	10% (10-yr)	17200	878.48	879.03	0.000405	5.96	2886.95	177.14	0.26
3.173	2% (50-yr)	30400	884.8	885.58	0.000414	7.21	5230.98	544.81	0.27
3.173	1% (100-yr)	36800	887.19	888.03	0.000397	7.58	6558.79	563.44	0.27
3.173	0.5% (200-yr)	43500	889.44	890.33	0.000384	7.92	7847.21	580.96	0.27
3.173	0.2% (500-yr)	53200	892.29	893.24	0.000375	8.39	9524.75	595	0.28
3.173	0.133% (750-yr)	58100	895.02	895.88	0.000308	8.08	11149.52	595	0.25
3.173	0.1% (1000-yr)	60800	895.34	896.26	0.000324	8.33	11344.65	595	0.26
3.173	0.04% (2500-yr)	72800	900.53	901.36	0.000252	8.13	14430.88	595	0.24
3.327	10% (10-yr)	17200	878.83	879.37	0.000425	5.91	2908.77	186.31	0.26
3.327	2% (50-yr)	30400	885.26	885.91	0.000369	6.7	7350.8	875.97	0.26
3.327	1% (100-yr)	36800	887.69	888.34	0.00034	6.91	9523.41	915.52	0.25
3.327	0.5% (200-yr)	43500	889.97	890.64	0.000318	7.12	11652.02	952.67	0.25
3.327	0.2% (500-yr)	53200	892.86	893.54	0.000298	7.4	14420.07	959.2	0.24
3.327	0.133% (750-yr)	58100	895.53	896.12	0.000241	7.07	16977.9	959.2	0.22
3.327	0.1% (1000-yr)	60800	895.89	896.52	0.000251	7.27	17323.55	959.2	0.23
3.327	0.04% (2500-yr)	72800	901.02	901.57	0.00019	6.99	22243.72	959.2	0.2
3.482	10% (10-yr)	17200	879.17	879.74	0.000449	6.05	2842.03	183.22	0.27
3.482	2% (50-yr)	30400	885.73	886.2	0.000304	6.04	9330.2	1212.57	0.23
3.482	1% (100-yr)	36800	888.19	888.6	0.000251	5.92	12341.77	1232.2	0.22
3.482	0.5% (200-yr)	43500	890.5	890.88	0.000218	5.87	15203.68	1250.57	0.21
3.482	0.2% (500-yr)	53200	893.41	893.77	0.000191	5.9	18876.29	1267.24	0.2
3.482	0.133% (750-yr)	58100	896.01	896.31	0.000149	5.53	22180.1	1275.9	0.18
3.482	0.1% (1000-yr)	60800	896.4	896.71	0.000154	5.66	22674.39	1277.19	0.18
3.482	0.04% (2500-yr)	72800	901.46	901.71	0.00011	5.29	29180.6	1289.2	0.15
3.623	10% (10-yr)	17200	879.49	880.09	0.000476	6.2	2773.86	180.01	0.28
3.623	2% (50-yr)	30400	885.87	886.52	0.000401	6.8	6815.44	876.84	0.27
3.623	1% (100-yr)	36800	888.27	888.89	0.000349	6.84	8968.25	911.42	0.25
3.623	0.5% (200-yr)	43500	890.55	891.14	0.000312	6.9	11075.75	939.76	0.24
3.623	0.2% (500-yr)	53200	893.43	894.01	0.00028	7.03	13810	952.63	0.24
3.623	0.133% (750-yr)	58100	896.01	896.5	0.000221	6.64	16271.19	955.21	0.21
3.623	0.1% (1000-yr)	60800	896.4	896.91	0.000229	6.81	16639.21	955.6	0.22
3.623	0.04% (2500-yr)	72800	901.44	901.87	0.000166	6.42	21470.46	959.2	0.19

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
3.65	10% (10-yr)	17200	879.73	880.16	0.000319	5.3	3244.49	201.68	0.23
3.65	2% (50-yr)	30400	885.94	886.57	0.000312	6.49	5357.32	515.58	0.24
3.65	1% (100-yr)	36800	888.27	888.96	0.000304	6.86	6643.97	582.55	0.24
3.65	0.5% (200-yr)	43500	890.5	891.22	0.000295	7.16	8009.6	641.56	0.24
3.65	0.2% (500-yr)	53200	893.35	894.1	0.000284	7.52	9927.82	706.99	0.24
3.65	0.133% (750-yr)	58100	895.92	896.59	0.000235	7.22	11821.97	766.14	0.22
3.65	0.1% (1000-yr)	60800	896.3	897	0.000244	7.42	12113.39	774.84	0.23
3.65	0.04% (2500-yr)	72800	901.35	901.94	0.000182	7.05	16301.71	860	0.2
3.764	10% (10-yr)	17200	879.93	880.35	0.000308	5.24	3284.85	202.64	0.23
3.764	2% (50-yr)	30400	886.14	886.75	0.000301	6.42	5462.79	523.84	0.24
3.764	1% (100-yr)	36800	888.47	889.14	0.000294	6.78	6760.42	588.02	0.24
3.764	0.5% (200-yr)	43500	890.7	891.4	0.000286	7.08	8134.4	646.02	0.24
3.764	0.2% (500-yr)	53200	893.53	894.27	0.000277	7.45	10060.52	711.3	0.24
3.764	0.133% (750-yr)	58100	896.07	896.73	0.00023	7.17	11939.14	769.65	0.22
3.764	0.1% (1000-yr)	60800	896.46	897.15	0.000239	7.36	12236.97	778.5	0.23
3.764	0.04% (2500-yr)	72800	901.47	902.05	0.000179	7.01	16402.6	860	0.2
3.874	10% (10-yr)	17200	880.1	880.54	0.000317	5.29	3249.41	201.8	0.23
3.874	2% (50-yr)	30400	886.3	886.94	0.000317	6.55	5172.14	461.56	0.24
3.874	1% (100-yr)	36800	888.63	889.32	0.000308	6.9	6382.27	604.1	0.25
3.874	0.5% (200-yr)	43500	890.86	891.57	0.000293	7.14	7959.37	767.11	0.24
3.874	0.2% (500-yr)	53200	893.73	894.44	0.000271	7.35	10243.8	824.53	0.24
3.874	0.133% (750-yr)	58100	896.26	896.87	0.000218	6.96	12398.1	875.22	0.22
3.874	0.1% (1000-yr)	60800	896.66	897.29	0.000225	7.13	12747.87	883.18	0.22
3.874	0.04% (2500-yr)	72800	901.66	902.16	0.000161	6.64	17385.96	950	0.19
3.984	10% (10-yr)	17200	880.28	880.72	0.000327	5.35	3215.23	200.98	0.24
3.984	2% (50-yr)	30400	886.49	887.13	0.000319	6.54	5252.47	495.56	0.25
3.984	1% (100-yr)	36800	888.81	889.5	0.000312	6.91	6491.46	563.28	0.25
3.984	0.5% (200-yr)	43500	891.02	891.74	0.000302	7.21	7805.79	620	0.25
3.984	0.2% (500-yr)	53200	893.85	894.62	0.000291	7.57	9561.77	620	0.25
3.984	0.133% (750-yr)	58100	896.33	897.03	0.000244	7.32	11102.42	620	0.23
3.984	0.1% (1000-yr)	60800	896.73	897.46	0.000253	7.52	11347.75	620	0.23
3.984	0.04% (2500-yr)	72800	901.66	902.31	0.000198	7.3	14402.34	620	0.21
4.097	10% (10-yr)	17200	880.46	880.94	0.000365	5.56	3091.54	195	0.25
4.097	2% (50-yr)	30400	886.64	887.38	0.000441	6.9	4402.72	231.33	0.28
4.097	1% (100-yr)	36800	888.92	889.78	0.000476	7.44	4948.75	246.57	0.29
4.097	0.5% (200-yr)	43500	891.08	892.05	0.000498	7.92	5522.39	506.68	0.3
4.097	0.2% (500-yr)	53200	893.85	894.94	0.000486	8.42	7894.16	1188.25	0.3
4.097	0.133% (750-yr)	58100	896.34	897.29	0.000385	8.01	11333	1543.36	0.28
4.097	0.1% (1000-yr)	60800	896.74	897.72	0.000394	8.19	11929.05	1543.36	0.28
4.097	0.04% (2500-yr)	72800	901.77	902.45	0.000248	7.28	20005.67	1569.36	0.23
4.0985		Bridge							
4.1	10% (10-yr)	17200	880.51	880.98	0.000362	5.55	3100.16	195.24	0.25

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
4.1	2% (50-yr)	30400	886.72	887.45	0.000436	6.88	4420.49	231.88	0.28
4.1	1% (100-yr)	36800	889.01	889.87	0.00047	7.4	4971.42	247.04	0.29
4.1	0.5% (200-yr)	43500	891.15	892.12	0.000493	7.89	5563.75	545.23	0.3
4.1	0.2% (500-yr)	53200	893.96	895.02	0.000478	8.37	8021.85	1212.3	0.3
4.1	0.133% (750-yr)	58100	896.47	897.4	0.000377	7.95	11519.84	1543.36	0.27
4.1	0.1% (1000-yr)	60800	896.81	897.77	0.00039	8.16	12028.12	1543.36	0.28
4.1	0.04% (2500-yr)	72800	901.89	902.57	0.000243	7.23	20204.04	1569.36	0.23
4.178	10% (10-yr)	17200	880.63	881.16	0.000414	5.83	2950.81	193.6	0.26
4.178	2% (50-yr)	30400	886.88	887.64	0.000438	7.04	4881.52	652.03	0.28
4.178	1% (100-yr)	36800	889.25	890.05	0.000407	7.33	6450.27	668.52	0.28
4.178	0.5% (200-yr)	43500	891.49	892.31	0.000381	7.56	7962.51	681.95	0.27
4.178	0.2% (500-yr)	53200	894.37	895.22	0.000354	7.87	9953.82	696	0.27
4.178	0.133% (750-yr)	58100	896.78	897.55	0.000294	7.6	11631.37	696	0.25
4.178	0.1% (1000-yr)	60800	897.13	897.94	0.000307	7.82	11873.77	696	0.25
4.178	0.04% (2500-yr)	72800	901.96	902.68	0.000235	7.57	15236.22	696	0.23
4.287	10% (10-yr)	17200	880.85	881.42	0.00047	6.06	2837.5	191.06	0.28
4.287	2% (50-yr)	30400	887.14	887.9	0.000462	7.11	5734.73	1209.19	0.29
4.287	1% (100-yr)	36800	889.6	890.29	0.000384	7.03	8757.57	1244.12	0.27
4.287	0.5% (200-yr)	43500	891.92	892.53	0.000324	6.92	11677.34	1276.96	0.25
4.287	0.2% (500-yr)	53200	894.88	895.43	0.000271	6.84	15522.56	1318.7	0.23
4.287	0.133% (750-yr)	58100	897.27	897.72	0.000211	6.39	18677.65	1318.7	0.21
4.287	0.1% (1000-yr)	60800	897.65	898.12	0.000217	6.53	19179.57	1318.7	0.21
4.287	0.04% (2500-yr)	72800	902.46	902.82	0.00015	6.01	25522.75	1318.7	0.18
4.396	10% (10-yr)	17200	881.13	881.69	0.000463	6.03	2851.66	191.52	0.28
4.396	2% (50-yr)	30400	887.39	888.17	0.000463	7.14	5538.85	1207.3	0.29
4.396	1% (100-yr)	36800	889.81	890.52	0.000392	7.1	8492.66	1242.35	0.27
4.396	0.5% (200-yr)	43500	892.08	892.73	0.000334	7.01	11361.79	1275.48	0.25
4.396	0.2% (500-yr)	53200	895.01	895.59	0.000281	6.95	15164.1	1318.7	0.24
4.396	0.133% (750-yr)	58100	897.37	897.85	0.000219	6.49	18276.62	1318.7	0.21
4.396	0.1% (1000-yr)	60800	897.76	898.25	0.000225	6.64	18782.48	1318.7	0.22
4.396	0.04% (2500-yr)	72800	902.53	902.91	0.000156	6.1	25080.35	1318.7	0.18
4.554	10% (10-yr)	16500	881.57	882.05	0.00037	5.55	2974.95	192.68	0.25
4.554	2% (50-yr)	28400	887.87	888.52	0.00037	6.54	4998.51	797.61	0.26
4.554	1% (100-yr)	34100	890.15	890.82	0.000341	6.75	6846.25	819.21	0.25
4.554	0.5% (200-yr)	40100	892.34	893	0.000314	6.89	8657.24	839.84	0.25
4.554	0.2% (500-yr)	48600	895.19	895.85	0.000282	7.04	11096.93	868.04	0.24
4.554	0.133% (750-yr)	53500	897.48	898.07	0.000237	6.82	13085.72	868.7	0.22
4.554	0.1% (1000-yr)	56000	897.86	898.48	0.000245	6.99	13417.58	868.7	0.23
4.554	0.04% (2500-yr)	67000	902.57	903.1	0.000184	6.67	17506.2	868.7	0.2
4.712	10% (10-yr)	16500	881.9	882.35	0.000349	5.35	3084.37	202.74	0.24
4.712	2% (50-yr)	28400	888.26	888.81	0.000307	6.11	5913.77	597.17	0.24
4.712	1% (100-yr)	34100	890.51	891.09	0.000291	6.38	7271.27	608.81	0.24
4.712	0.5% (200-yr)	40100	892.64	893.26	0.000279	6.63	8582.24	619.86	0.23
4.712	0.2% (500-yr)	48600	895.43	896.08	0.000266	6.94	10334.24	635.71	0.23

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
4.712	0.133% (750-yr)	53500	897.66	898.28	0.000234	6.86	11755	639.61	0.22
4.712	0.1% (1000-yr)	56000	898.05	898.7	0.000243	7.06	12001.54	640.28	0.23
4.712	0.04% (2500-yr)	67000	902.68	903.28	0.000195	6.95	14977.51	643.7	0.21
4.869	10% (10-yr)	16500	882.14	882.74	0.000494	6.24	2642.52	176.48	0.28
4.869	2% (50-yr)	28400	888.39	889.2	0.000464	7.31	4260.48	398.99	0.29
4.869	1% (100-yr)	34100	890.6	891.48	0.000447	7.7	5193.46	444.7	0.29
4.869	0.5% (200-yr)	40100	892.71	893.64	0.00043	8.02	6172.49	477.66	0.29
4.869	0.2% (500-yr)	48600	895.47	896.45	0.000405	8.37	7524.75	498.68	0.28
4.869	0.133% (750-yr)	53500	897.68	898.61	0.000353	8.23	8634.7	504.21	0.27
4.869	0.1% (1000-yr)	56000	898.07	899.04	0.000366	8.46	8828.77	505.17	0.27
4.869	0.04% (2500-yr)	67000	902.68	903.56	0.000286	8.24	11174.79	510	0.25
4.907	10% (10-yr)	16500	882.24	882.84	0.000481	6.22	2654.09	174.52	0.28
4.907	2% (50-yr)	28400	888.46	889.31	0.000525	7.38	3845.85	208.34	0.3
4.907	1% (100-yr)	34100	890.64	891.61	0.000557	7.91	4311.16	220.14	0.31
4.907	0.5% (200-yr)	40100	892.69	893.79	0.000572	8.4	4775.86	232.86	0.32
4.907	0.2% (500-yr)	48600	895.37	896.63	0.000574	9.01	5424.43	251.9	0.33
4.907	0.133% (750-yr)	53500	897.52	898.8	0.000516	9.06	5986.41	270.11	0.32
4.907	0.1% (1000-yr)	56000	897.89	899.24	0.000539	9.35	6085.37	273.2	0.32
4.907	0.04% (2500-yr)	67000	902.42	903.77	0.000437	9.38	7563.36	345.1	0.3
4.9105		Bridge							
4.914	10% (10-yr)	16500	882.29	882.88	0.000477	6.2	2662.69	174.79	0.28
4.914	2% (50-yr)	28400	888.54	889.38	0.00052	7.35	3861.71	208.76	0.3
4.914	1% (100-yr)	34100	890.73	891.69	0.000551	7.87	4330.92	220.63	0.31
4.914	0.5% (200-yr)	40100	892.79	893.88	0.000564	8.36	4799.99	233.53	0.32
4.914	0.2% (500-yr)	48600	895.49	896.74	0.000564	8.97	5454.9	252.92	0.33
4.914	0.133% (750-yr)	53500	897.64	898.9	0.000508	9.02	6018.34	271.11	0.31
4.914	0.1% (1000-yr)	56000	898	899.34	0.000531	9.31	6115.48	274.13	0.32
4.914	0.04% (2500-yr)	67000	903.69	904.9	0.000377	8.95	7999.01	347.83	0.28
4.942	10% (10-yr)	16500	882.37	882.96	0.000474	6.15	2683.99	177.87	0.28
4.942	2% (50-yr)	28400	888.67	889.46	0.00044	7.19	4374.3	404.84	0.28
4.942	1% (100-yr)	34100	890.92	891.77	0.000422	7.55	5339.72	451.44	0.28
4.942	0.5% (200-yr)	40100	893.08	893.97	0.000404	7.85	6351.52	479.78	0.28
4.942	0.2% (500-yr)	48600	895.91	896.84	0.000379	8.18	7742.71	499.77	0.28
4.942	0.133% (750-yr)	53500	898.12	899	0.000332	8.06	8855.13	505.3	0.26
4.942	0.1% (1000-yr)	56000	898.52	899.44	0.000343	8.28	9057.22	506.3	0.27
4.942	0.04% (2500-yr)	67000	904.23	905	0.000238	7.74	11967.44	510	0.23
5.108	10% (10-yr)	16500	882.81	883.35	0.000422	5.9	2795.83	180.75	0.26
5.108	2% (50-yr)	28400	889.1	889.83	0.000402	6.94	4457.57	405.57	0.27
5.108	1% (100-yr)	34100	891.35	892.13	0.000383	7.26	5447.64	472.66	0.27
5.108	0.5% (200-yr)	40100	893.51	894.31	0.000363	7.51	6514.1	507.1	0.27
5.108	0.2% (500-yr)	48600	896.33	897.16	0.000337	7.78	7968.58	527.13	0.26
5.108	0.133% (750-yr)	53500	898.5	899.29	0.000297	7.67	9163.79	575.89	0.25
5.108	0.1% (1000-yr)	56000	898.92	899.74	0.000306	7.87	9406.14	585.28	0.25

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
5.108	0.04% (2500-yr)	67000	904.55	905.2	0.000207	7.25	12827.8	609.6	0.21
5.274	10% (10-yr)	16500	883.19	883.73	0.000436	5.88	2805.36	187.43	0.27
5.274	2% (50-yr)	28400	889.5	890.2	0.000425	6.76	4611.28	500.59	0.27
5.274	1% (100-yr)	34100	891.78	892.47	0.000382	6.92	5887.25	611.65	0.27
5.274	0.5% (200-yr)	40100	893.95	894.63	0.000342	6.99	7323.5	706.24	0.26
5.274	0.2% (500-yr)	48600	896.82	897.46	0.000294	7	9524.35	831.21	0.24
5.274	0.133% (750-yr)	53500	898.98	899.54	0.000244	6.73	11413.51	917.63	0.22
5.274	0.1% (1000-yr)	56000	899.42	900	0.000248	6.86	11826.4	935.46	0.23
5.274	0.04% (2500-yr)	67000	904.98	905.38	0.000149	5.99	17149.52	958.6	0.18
5.386	10% (10-yr)	16500	883.46	883.99	0.000427	5.84	2827.08	188.25	0.27
5.386	2% (50-yr)	28400	889.78	890.45	0.000409	6.66	4724.14	598.09	0.27
5.386	1% (100-yr)	34100	892.01	892.7	0.000374	6.87	5787.73	724.74	0.26
5.386	0.5% (200-yr)	40100	894.15	894.85	0.000347	7.05	6817.72	827.96	0.26
5.386	0.2% (500-yr)	48600	896.94	897.66	0.000316	7.26	8195.53	964.3	0.25
5.386	0.133% (750-yr)	53500	899.05	899.73	0.000276	7.14	9242.71	1059.12	0.24
5.386	0.1% (1000-yr)	56000	899.49	900.2	0.000283	7.31	9461.1	1078.9	0.24
5.386	0.04% (2500-yr)	67000	904.94	905.54	0.000196	6.84	12171.96	1108.6	0.21
5.499	10% (10-yr)	16500	883.77	884.23	0.000372	5.42	3045.41	207.28	0.25
5.499	2% (50-yr)	28400	890.07	890.68	0.000346	6.26	4747.6	341.81	0.25
5.499	1% (100-yr)	34100	892.24	892.91	0.000339	6.65	5504.17	356.03	0.25
5.499	0.5% (200-yr)	40100	894.32	895.06	0.000333	7.02	6259.21	369.66	0.26
5.499	0.2% (500-yr)	48600	897.07	897.88	0.000324	7.44	7293.62	380	0.26
5.499	0.133% (750-yr)	53500	899.13	899.94	0.000295	7.47	8077.54	380	0.25
5.499	0.1% (1000-yr)	56000	899.57	900.42	0.000305	7.67	8242.97	380	0.25
5.499	0.04% (2500-yr)	67000	904.94	905.72	0.00023	7.47	10284.66	380	0.23
5.605	10% (10-yr)	16500	883.98	884.44	0.000383	5.47	3015.41	206.38	0.25
5.605	2% (50-yr)	28400	890.26	890.88	0.000357	6.32	4692.48	339.82	0.25
5.605	1% (100-yr)	34100	892.43	893.11	0.000349	6.71	5444.98	354.93	0.26
5.605	0.5% (200-yr)	40100	894.5	895.25	0.000342	7.08	6196.31	368.55	0.26
5.605	0.2% (500-yr)	48600	897.24	898.07	0.000332	7.51	7226.5	380	0.26
5.605	0.133% (750-yr)	53500	899.29	900.11	0.000302	7.53	8004.18	380	0.25
5.605	0.1% (1000-yr)	56000	899.73	900.59	0.000312	7.74	8171.75	380	0.26
5.605	0.04% (2500-yr)	67000	905.06	905.86	0.000236	7.53	10197.62	380	0.23
5.711	10% (10-yr)	16500	884.16	884.69	0.000434	5.83	2829.76	193.26	0.27
5.711	2% (50-yr)	28400	890.4	891.13	0.000431	6.86	4189.87	275	0.28
5.711	1% (100-yr)	34100	892.54	893.37	0.000427	7.34	4865.22	342.65	0.28
5.711	0.5% (200-yr)	40100	894.6	895.51	0.000419	7.74	5586.35	356.57	0.28
5.711	0.2% (500-yr)	48600	897.33	898.32	0.000405	8.19	6578.74	369	0.28
5.711	0.133% (750-yr)	53500	899.36	900.34	0.000367	8.21	7328.63	369	0.27
5.711	0.1% (1000-yr)	56000	899.8	900.84	0.000379	8.43	7491.8	369	0.28
5.711	0.04% (2500-yr)	67000	905.1	906.05	0.000282	8.16	9448.49	369	0.25
5.749	10% (10-yr)	16500	884.26	884.78	0.000425	5.78	2855.41	194.7	0.27
5.749	2% (50-yr)	28400	890.5	891.22	0.000441	6.8	4175.06	228.26	0.28

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
5.749	1% (100-yr)	34100	892.63	893.46	0.000467	7.3	4673.63	239.72	0.29
5.749	0.5% (200-yr)	40100	894.67	895.61	0.000472	7.76	5170.62	250.72	0.3
5.749	0.2% (500-yr)	48600	897.35	898.43	0.000467	8.34	5824.5	265.05	0.3
5.749	0.133% (750-yr)	53500	899.35	900.46	0.000433	8.48	6310.78	275.6	0.29
5.749	0.1% (1000-yr)	56000	899.78	900.96	0.000449	8.73	6416.43	277.9	0.3
5.749	0.04% (2500-yr)	67000	905.07	906.15	0.000345	8.39	8440.74	410	0.27
5.7585		Bridge							
5.768	10% (10-yr)	16500	884.29	884.8	0.000423	5.77	2860.6	194.84	0.27
5.768	2% (50-yr)	28400	890.54	891.25	0.000439	6.79	4183.24	228.45	0.28
5.768	1% (100-yr)	34100	892.67	893.49	0.000464	7.28	4683.63	239.95	0.29
5.768	0.5% (200-yr)	40100	894.72	895.65	0.000469	7.74	5182.04	250.97	0.3
5.768	0.2% (500-yr)	48600	898.35	899.35	0.000407	8.01	6067.9	270.33	0.28
5.768	0.133% (750-yr)	53500	900.79	901.76	0.000361	8.03	6662.53	289.75	0.27
5.768	0.1% (1000-yr)	56000	901.38	902.42	0.000369	8.23	6806.02	336.23	0.27
5.768	0.04% (2500-yr)	67000	906.44	907.41	0.000293	7.97	9001.66	410	0.25
5.795	10% (10-yr)	16500	884.35	884.86	0.000419	5.76	2865.37	194.24	0.26
5.795	2% (50-yr)	28400	890.6	891.31	0.000416	6.79	4244.89	282.89	0.27
5.795	1% (100-yr)	34100	892.75	893.56	0.000412	7.25	4937.29	344.06	0.28
5.795	0.5% (200-yr)	40100	894.83	895.72	0.000404	7.65	5667.29	358.1	0.28
5.795	0.2% (500-yr)	48600	898.53	899.41	0.00034	7.74	7022.51	369	0.26
5.795	0.133% (750-yr)	53500	901	901.85	0.000295	7.64	7936.27	369	0.25
5.795	0.1% (1000-yr)	56000	901.61	902.49	0.000299	7.8	8160.84	369	0.25
5.795	0.04% (2500-yr)	67000	906.61	907.45	0.000239	7.73	10005.55	369	0.23
5.962	10% (10-yr)	16500	884.67	885.31	0.000519	6.42	2568.35	171.06	0.29
5.962	2% (50-yr)	28400	890.88	891.79	0.000538	7.64	3728.56	225.19	0.31
5.962	1% (100-yr)	34100	893	894.04	0.00054	8.22	4283.66	298.02	0.31
5.962	0.5% (200-yr)	40100	895.05	896.21	0.000535	8.7	4931.64	329.56	0.32
5.962	0.2% (500-yr)	48600	898.69	899.84	0.000452	8.81	6182.73	349	0.3
5.962	0.133% (750-yr)	53500	901.13	902.22	0.000391	8.68	7035.46	349	0.28
5.962	0.1% (1000-yr)	56000	901.74	902.87	0.000395	8.85	7247.62	349	0.29
5.962	0.04% (2500-yr)	67000	906.7	907.76	0.000313	8.72	8977.94	349	0.26
6	10% (10-yr)	16500	884.78	885.41	0.000511	6.4	2578.26	170.56	0.29
6	2% (50-yr)	28400	890.99	891.89	0.000547	7.63	3723.3	198.07	0.31
6	1% (100-yr)	34100	893.11	894.16	0.000584	8.21	4153.07	207.46	0.32
6	0.5% (200-yr)	40100	895.14	896.33	0.000616	8.75	4583.66	216.46	0.34
6	0.2% (500-yr)	48600	898.7	899.97	0.00055	9.05	5400.41	247.32	0.32
6	0.133% (750-yr)	53500	901.1	902.36	0.000485	9.04	5994.17	263.82	0.31
6	0.1% (1000-yr)	56000	901.69	903.02	0.000492	9.25	6145.93	267.93	0.31
6	0.04% (2500-yr)	67000	906.64	907.89	0.000382	9.11	8067.65	386	0.28
6.003		Bridge							
6.006	10% (10-yr)	16500	884.85	885.48	0.000504	6.37	2590.73	170.88	0.29
6.006	2% (50-yr)	28400	891.09	891.99	0.000539	7.59	3743.91	198.53	0.31

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
6.006	1% (100-yr)	34100	893.23	894.27	0.000574	8.16	4178.49	208	0.32
6.006	0.5% (200-yr)	40100	895.28	896.46	0.000605	8.69	4614.11	217.08	0.33
6.006	0.2% (500-yr)	48600	899.71	900.87	0.000497	8.65	5648.1	254.29	0.31
6.006	0.133% (750-yr)	53500	902.43	903.55	0.000426	8.52	6443.52	386	0.29
6.006	0.1% (1000-yr)	56000	903.07	904.24	0.000429	8.69	6691.41	386	0.29
6.006	0.04% (2500-yr)	67000	907.59	908.72	0.00035	8.69	8435.08	386	0.27
6.029	10% (10-yr)	16500	884.91	885.54	0.000512	6.4	2580.08	170.96	0.29
6.029	2% (50-yr)	28400	891.16	892.05	0.000512	7.6	3791.83	245.87	0.3
6.029	1% (100-yr)	34100	893.31	894.33	0.000513	8.15	4392.95	307.69	0.31
6.029	0.5% (200-yr)	40100	895.4	896.53	0.000506	8.61	5065.09	333.43	0.31
6.029	0.2% (500-yr)	48600	899.92	900.94	0.000381	8.38	6628.83	349	0.28
6.029	0.133% (750-yr)	53500	902.66	903.61	0.000322	8.19	7585.44	349	0.26
6.029	0.1% (1000-yr)	56000	903.31	904.3	0.000325	8.35	7812.79	349	0.26
6.029	0.04% (2500-yr)	67000	907.79	908.78	0.000279	8.45	9375.54	349	0.25
6.186	10% (10-yr)	16500	885.36	885.96	0.000476	6.23	2647.94	173.37	0.28
6.186	2% (50-yr)	28400	891.62	892.47	0.000489	7.43	3855.31	240.5	0.3
6.186	1% (100-yr)	34100	893.77	894.76	0.000491	7.99	4439.26	301.14	0.3
6.186	0.5% (200-yr)	40100	895.86	896.94	0.000487	8.45	5105.47	332.95	0.31
6.186	0.2% (500-yr)	48600	900.25	901.26	0.000375	8.3	6625.51	349	0.28
6.186	0.133% (750-yr)	53500	902.93	903.88	0.000319	8.14	7562.43	349	0.26
6.186	0.1% (1000-yr)	56000	903.58	904.57	0.000322	8.29	7790.8	349	0.26
6.186	0.04% (2500-yr)	67000	908.02	909.01	0.000278	8.41	9338.74	349	0.25
6.346	10% (10-yr)	16500	885.76	886.39	0.000523	6.37	2589.61	175.3	0.29
6.346	2% (50-yr)	28400	892.05	892.91	0.000557	7.43	3821.21	216.47	0.31
6.346	1% (100-yr)	34100	894.22	895.2	0.000559	7.93	4334.22	261.58	0.32
6.346	0.5% (200-yr)	40100	896.29	897.38	0.000546	8.38	4950.45	325.6	0.32
6.346	0.2% (500-yr)	48600	900.57	901.59	0.000414	8.24	6427.05	349	0.29
6.346	0.133% (750-yr)	53500	903.2	904.17	0.000349	8.07	7345.97	349	0.27
6.346	0.1% (1000-yr)	56000	903.86	904.86	0.000351	8.22	7575.28	349	0.27
6.346	0.04% (2500-yr)	67000	908.26	909.25	0.000298	8.31	9109.27	349	0.25
6.505	10% (10-yr)	16500	886.26	886.84	0.000555	6.13	2691.92	203.57	0.3
6.505	2% (50-yr)	28400	892.65	893.36	0.000486	6.81	4278.6	302.2	0.29
6.505	1% (100-yr)	34100	894.86	895.64	0.000468	7.15	4990.52	339.83	0.29
6.505	0.5% (200-yr)	40100	896.97	897.81	0.000443	7.47	5724.58	357.09	0.29
6.505	0.2% (500-yr)	48600	901.13	901.92	0.000336	7.34	7263.99	375.7	0.26
6.505	0.133% (750-yr)	53500	903.69	904.45	0.000284	7.2	8227.6	375.7	0.24
6.505	0.1% (1000-yr)	56000	904.36	905.14	0.000286	7.33	8477.84	375.7	0.24
6.505	0.04% (2500-yr)	67000	908.71	909.5	0.000243	7.43	10111.25	375.7	0.23
6.663	10% (10-yr)	16500	886.71	887.3	0.000531	6.13	2689.6	195.79	0.29
6.663	2% (50-yr)	28400	893.04	893.79	0.000514	6.93	4129.76	290.79	0.3
6.663	1% (100-yr)	34100	895.24	896.06	0.000496	7.29	4831.55	339.89	0.3
6.663	0.5% (200-yr)	40100	897.32	898.21	0.000471	7.63	5557.13	356.94	0.3
6.663	0.2% (500-yr)	48600	901.39	902.23	0.00036	7.51	7061.5	375.7	0.27
6.663	0.133% (750-yr)	53500	903.91	904.71	0.000305	7.37	8009.05	375.7	0.25

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
6.663	0.1% (1000-yr)	56000	904.58	905.4	0.000305	7.5	8259.5	375.7	0.25
6.663	0.04% (2500-yr)	67000	908.89	909.72	0.000259	7.59	9879.71	375.7	0.24
6.815	10% (10-yr)	16500	887.15	887.68	0.000417	5.82	2833.78	185.94	0.26
6.815	2% (50-yr)	28400	893.43	894.17	0.000428	6.9	4188.51	301.17	0.28
6.815	1% (100-yr)	34100	895.6	896.43	0.000422	7.36	4902.14	343.24	0.28
6.815	0.5% (200-yr)	40100	897.66	898.57	0.000416	7.76	5627.13	360.13	0.28
6.815	0.2% (500-yr)	48600	901.64	902.52	0.000336	7.75	7105.87	375.7	0.26
6.815	0.133% (750-yr)	53500	904.12	904.96	0.000291	7.64	8037.85	375.7	0.25
6.815	0.1% (1000-yr)	56000	904.79	905.65	0.000293	7.78	8288.39	375.7	0.25
6.815	0.04% (2500-yr)	67000	909.07	909.94	0.000254	7.9	9895.3	375.7	0.24
6.967	10% (10-yr)	16500	887.5	888	0.000389	5.67	2908.14	188.25	0.25
6.967	2% (50-yr)	28400	893.8	894.5	0.000397	6.74	4315.67	316.52	0.27
6.967	1% (100-yr)	34100	895.97	896.76	0.000394	7.2	5043.93	346.68	0.27
6.967	0.5% (200-yr)	40100	898.03	898.9	0.00039	7.6	5775.15	363.54	0.27
6.967	0.2% (500-yr)	48600	901.94	902.78	0.00032	7.63	7232.75	375.7	0.25
6.967	0.133% (750-yr)	53500	904.38	905.19	0.000281	7.55	8149.23	375.7	0.24
6.967	0.1% (1000-yr)	56000	905.04	905.89	0.000283	7.68	8400.46	375.7	0.24
6.967	0.04% (2500-yr)	67000	909.29	910.14	0.000247	7.83	9993.97	375.7	0.23
7.119	10% (10-yr)	16500	887.69	888.49	0.000646	7.21	2288.1	148	0.32
7.119	2% (50-yr)	28400	893.9	895.05	0.000716	8.63	3292.14	175.18	0.35
7.119	1% (100-yr)	34100	896.01	897.35	0.000745	9.29	3695.1	232	0.36
7.119	0.5% (200-yr)	40100	898	899.51	0.000744	9.88	4261.11	335.04	0.37
7.119	0.2% (500-yr)	48600	901.88	903.3	0.000589	9.78	5662.87	374	0.34
7.119	0.133% (750-yr)	53500	904.32	905.63	0.000497	9.53	6577.46	374	0.31
7.119	0.1% (1000-yr)	56000	904.99	906.33	0.000496	9.67	6827.53	374	0.31
7.119	0.04% (2500-yr)	67000	909.24	910.52	0.000409	9.61	8418.08	374	0.29
7.271	10% (10-yr)	16500	888.24	888.99	0.000586	6.96	2370.86	150.43	0.31
7.271	2% (50-yr)	28400	894.53	895.61	0.000654	8.34	3403.58	177.94	0.34
7.271	1% (100-yr)	34100	896.67	897.92	0.000668	8.99	3860.28	266.26	0.35
7.271	0.5% (200-yr)	40100	898.69	900.08	0.000665	9.53	4493.42	340.07	0.35
7.271	0.2% (500-yr)	48600	902.41	903.76	0.000547	9.55	5820.5	374	0.33
7.271	0.133% (750-yr)	53500	904.76	906.03	0.00047	9.36	6700.32	374	0.31
7.271	0.1% (1000-yr)	56000	905.43	906.72	0.00047	9.5	6949.96	374	0.31
7.271	0.04% (2500-yr)	67000	909.6	910.84	0.000394	9.5	8508.74	374	0.29
7.309	10% (10-yr)	16500	888.37	889.11	0.000573	6.91	2388.86	150.51	0.31
7.309	2% (50-yr)	28400	894.67	895.74	0.000641	8.3	3422.94	177.53	0.33
7.309	1% (100-yr)	34100	896.82	898.06	0.000691	8.94	3813.31	186.72	0.35
7.309	0.5% (200-yr)	40100	898.81	900.23	0.000739	9.56	4194.6	195.27	0.36
7.309	0.2% (500-yr)	48600	902.42	903.93	0.000696	9.86	4933.72	230.05	0.36
7.309	0.133% (750-yr)	53500	904.7	906.21	0.000633	9.88	5442.7	295.89	0.35
7.309	0.1% (1000-yr)	56000	905.34	906.92	0.000637	10.08	5590.93	337.72	0.35
7.309	0.04% (2500-yr)	67000	909.48	911.02	0.000525	10.1	7258.6	374	0.32
7.312		Bridge							

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
7.315	10% (10-yr)	16500	888.45	889.19	0.000564	6.87	2401.89	150.88	0.3
7.315	2% (50-yr)	28400	894.81	895.86	0.000629	8.24	3446.6	178.1	0.33
7.315	1% (100-yr)	34100	896.97	898.2	0.000676	8.87	3843.12	187.4	0.35
7.315	0.5% (200-yr)	40100	899	900.39	0.000722	9.48	4231.22	196.08	0.36
7.315	0.2% (500-yr)	48600	902.54	904.03	0.000686	9.81	4959.73	232.28	0.36
7.315	0.133% (750-yr)	53500	906.26	907.57	0.000512	9.24	6050.74	374	0.31
7.315	0.1% (1000-yr)	56000	906.97	908.32	0.000509	9.38	6319.08	374	0.31
7.315	0.04% (2500-yr)	67000	910.89	912.25	0.000442	9.55	7782.36	374	0.3
7.345	10% (10-yr)	16500	888.49	889.31	0.000643	7.24	2279.54	144.14	0.32
7.345	2% (50-yr)	28400	894.84	896	0.000722	8.63	3290.73	174.46	0.35
7.345	1% (100-yr)	34100	897.01	898.34	0.000747	9.26	3711.95	246.62	0.36
7.345	0.5% (200-yr)	40100	899.06	900.53	0.000737	9.8	4332.47	338.99	0.36
7.345	0.2% (500-yr)	48600	902.72	904.14	0.000602	9.8	5658.06	374	0.34
7.345	0.133% (750-yr)	53500	906.53	907.66	0.000419	8.95	7080.32	374	0.29
7.345	0.1% (1000-yr)	56000	907.26	908.42	0.000416	9.06	7354.02	374	0.29
7.345	0.04% (2500-yr)	67000	911.17	912.33	0.000366	9.2	8818.38	374	0.28
7.534	10% (10-yr)	16500	889.13	889.98	0.000682	7.41	2226.78	142.1	0.33
7.534	2% (50-yr)	28400	895.55	896.75	0.00075	8.78	3235.36	172.05	0.36
7.534	1% (100-yr)	34100	897.75	899.12	0.000787	9.41	3632.06	205.95	0.37
7.534	0.5% (200-yr)	40100	899.77	901.31	0.000783	10	4165.23	321.8	0.37
7.534	0.2% (500-yr)	48600	903.28	904.81	0.000658	10.11	5405.59	374	0.35
7.534	0.133% (750-yr)	53500	906.9	908.14	0.000466	9.3	6759.17	374	0.3
7.534	0.1% (1000-yr)	56000	907.62	908.89	0.000462	9.41	7031.59	374	0.3
7.534	0.04% (2500-yr)	67000	911.49	912.74	0.000403	9.53	8477.8	374	0.29
7.724	10% (10-yr)	16500	889.82	890.66	0.000667	7.34	2249.06	144.22	0.33
7.724	2% (50-yr)	28400	896.32	897.48	0.000718	8.66	3278.27	172.64	0.35
7.724	1% (100-yr)	34100	898.56	899.89	0.000748	9.28	3683.39	208.82	0.36
7.724	0.5% (200-yr)	40100	900.58	902.08	0.000748	9.88	4210.05	313.09	0.37
7.724	0.2% (500-yr)	48600	903.93	905.46	0.00065	10.1	5381.96	374	0.35
7.724	0.133% (750-yr)	53500	907.34	908.62	0.000476	9.39	6659.11	374	0.31
7.724	0.1% (1000-yr)	56000	908.07	909.36	0.000472	9.5	6929.73	374	0.31
7.724	0.04% (2500-yr)	67000	911.87	913.16	0.000414	9.65	8353.21	374	0.29
7.914	10% (10-yr)	16500	890.49	891.33	0.000676	7.34	2248.68	146.72	0.33
7.914	2% (50-yr)	28400	897.05	898.2	0.000708	8.59	3305.04	175.41	0.35
7.914	1% (100-yr)	34100	899.32	900.63	0.000717	9.18	3749.88	243.11	0.36
7.914	0.5% (200-yr)	40100	901.36	902.82	0.000711	9.74	4351.22	337.38	0.36
7.914	0.2% (500-yr)	48600	904.62	906.1	0.000624	9.97	5508.29	374	0.35
7.914	0.133% (750-yr)	53500	907.83	909.09	0.00047	9.36	6709.98	374	0.31
7.914	0.1% (1000-yr)	56000	908.55	909.83	0.000466	9.47	6979.02	374	0.31
7.914	0.04% (2500-yr)	67000	912.29	913.58	0.000412	9.64	8378.53	374	0.29
8.103	10% (10-yr)	15900	891.56	892.11	0.00084	5.93	2683.53	200.23	0.29
8.103	2% (50-yr)	27100	898.35	898.98	0.000765	6.4	4235.69	263.4	0.28
8.103	1% (100-yr)	32500	900.75	901.41	0.000723	6.58	5633.88	960.27	0.28

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
8.103	0.5% (200-yr)	38100	902.92	903.57	0.000625	6.62	7748.88	990.09	0.26
8.103	0.2% (500-yr)	46000	906.16	906.74	0.000481	6.43	11022.87	1018.11	0.24
8.103	0.133% (750-yr)	51100	909.1	909.58	0.000356	5.99	14029.11	1025	0.21
8.103	0.1% (1000-yr)	53500	909.84	910.31	0.000346	6.02	14783.78	1025	0.21
8.103	0.04% (2500-yr)	64000	913.56	914	0.000285	5.96	18597.25	1025	0.19
8.3	10% (10-yr)	15900	892.42	892.86	0.000609	5.3	3002.52	208.71	0.25
8.3	2% (50-yr)	27100	899.12	899.67	0.000565	5.99	4527.44	355.32	0.25
8.3	1% (100-yr)	32500	901.47	902.06	0.000527	6.21	6260.18	888.42	0.24
8.3	0.5% (200-yr)	38100	903.55	904.14	0.00048	6.32	8174.41	949.86	0.23
8.3	0.2% (500-yr)	46000	906.66	907.19	0.000395	6.25	11348.1	1097.29	0.22
8.3	0.133% (750-yr)	51100	909.48	909.91	0.000297	5.82	14583.95	1161	0.19
8.3	0.1% (1000-yr)	53500	910.22	910.64	0.000288	5.82	15434.43	1161	0.19
8.3	0.04% (2500-yr)	64000	913.9	914.27	0.000232	5.66	19709.31	1161	0.17
8.497	10% (10-yr)	15900	893.05	893.59	0.00075	5.92	2686.65	181.42	0.27
8.497	2% (50-yr)	27100	899.67	900.39	0.000735	6.77	4006.27	621.3	0.28
8.497	1% (100-yr)	32500	902.02	902.67	0.000618	6.7	6711.56	890.4	0.26
8.497	0.5% (200-yr)	38100	904.06	904.69	0.000553	6.75	8614.72	972.46	0.25
8.497	0.2% (500-yr)	46000	907.09	907.64	0.000444	6.57	11635.91	1017.31	0.23
8.497	0.133% (750-yr)	51100	909.81	910.25	0.000339	6.14	14443.24	1049.24	0.2
8.497	0.1% (1000-yr)	53500	910.53	910.97	0.00033	6.16	15202.99	1057.71	0.2
8.497	0.04% (2500-yr)	64000	914.15	914.54	0.000272	6.04	19098.32	1093.24	0.19
8.694	10% (10-yr)	15900	893.78	894.62	0.00112	7.37	2156.06	138.93	0.33
8.694	2% (50-yr)	27100	900.31	901.46	0.00115	8.63	3144.9	489.53	0.35
8.694	1% (100-yr)	32500	902.49	903.61	0.001029	8.76	5798.88	1585.51	0.33
8.694	0.5% (200-yr)	38100	904.54	905.42	0.00081	8.25	9194.66	1708.52	0.3
8.694	0.2% (500-yr)	46000	907.53	908.1	0.000531	7.24	14369.31	1739.82	0.25
8.694	0.133% (750-yr)	51100	910.21	910.58	0.00035	6.26	19050.37	1762.37	0.2
8.694	0.1% (1000-yr)	53500	910.93	911.28	0.000328	6.17	20330.27	1768.49	0.2
8.694	0.04% (2500-yr)	64000	914.53	914.78	0.000235	5.62	26740.49	1798.81	0.17
8.891	10% (10-yr)	15900	895.01	895.63	0.000808	6.32	2515.84	162.71	0.28
8.891	2% (50-yr)	27100	901.64	902.48	0.000786	7.34	3726.01	1006.11	0.29
8.891	1% (100-yr)	32500	903.73	904.48	0.000676	7.28	7896.99	2221.99	0.27
8.891	0.5% (200-yr)	38100	905.49	906.09	0.000552	6.92	11875.07	2289	0.25
8.891	0.2% (500-yr)	46000	908.13	908.52	0.000382	6.18	18020.01	2377.19	0.21
8.891	0.133% (750-yr)	51100	910.59	910.85	0.000254	5.35	23960.51	2444.19	0.18
8.891	0.1% (1000-yr)	53500	911.29	911.53	0.000237	5.25	25680.68	2463.25	0.17
8.891	0.04% (2500-yr)	64000	914.79	914.95	0.000164	4.71	34453.42	2558.24	0.14
9.088	10% (10-yr)	15900	895.87	896.26	0.000448	5.02	3167.71	186	0.21
9.088	2% (50-yr)	27100	902.58	903.13	0.000482	5.99	4546.18	1157	0.23
9.088	1% (100-yr)	32500	904.53	905.06	0.000444	6.05	8761.06	1921.68	0.22
9.088	0.5% (200-yr)	38100	906.08	906.57	0.000413	6.1	11754.67	1949.3	0.22
9.088	0.2% (500-yr)	46000	908.46	908.86	0.000337	5.86	16455.79	1991.91	0.2
9.088	0.133% (750-yr)	51100	910.8	911.09	0.000247	5.31	21144.42	2028.68	0.17
9.088	0.1% (1000-yr)	53500	911.48	911.76	0.000235	5.26	22533.2	2038.5	0.17

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
9.088	0.04% (2500-yr)	64000	914.9	915.12	0.000176	4.89	29597.69	2087.72	0.15
9.274	10% (10-yr)	15400	896.31	896.8	0.000594	5.59	2756.62	168.86	0.24
9.274	2% (50-yr)	25900	903.02	903.68	0.000565	6.52	4025.43	1601.73	0.25
9.274	1% (100-yr)	30800	904.99	905.48	0.000437	6.08	9736.74	1916.64	0.22
9.274	0.5% (200-yr)	36000	906.51	906.95	0.000395	6.03	12687.74	1956.1	0.21
9.274	0.2% (500-yr)	43400	908.82	909.17	0.000319	5.75	17260.66	2003.84	0.19
9.274	0.133% (750-yr)	48300	911.05	911.31	0.000236	5.21	21765.02	2030.84	0.17
9.274	0.1% (1000-yr)	50600	911.72	911.97	0.000225	5.16	23132.74	2049.12	0.17
9.274	0.04% (2500-yr)	60500	915.08	915.27	0.000166	4.76	30074.46	2079.94	0.15
9.46	10% (10-yr)	15400	896.91	897.33	0.000493	5.22	2951.51	174.51	0.22
9.46	2% (50-yr)	25900	903.61	904.19	0.000475	6.16	4438.62	1506.83	0.23
9.46	1% (100-yr)	30800	905.4	905.89	0.000409	6.01	9333.61	1867.42	0.22
9.46	0.5% (200-yr)	36000	906.87	907.33	0.000382	6.04	12093.86	1873.04	0.21
9.46	0.2% (500-yr)	43400	909.1	909.48	0.000322	5.86	16274.65	1881.51	0.2
9.46	0.133% (750-yr)	48300	911.25	911.55	0.000246	5.39	20339.01	1889.72	0.17
9.46	0.1% (1000-yr)	50600	911.91	912.19	0.000236	5.35	21584.8	1892.22	0.17
9.46	0.04% (2500-yr)	60500	915.22	915.44	0.000179	5	27871.93	1900.74	0.15
9.647	10% (10-yr)	15400	897.38	897.86	0.000542	5.53	2785.11	160.13	0.23
9.647	2% (50-yr)	25900	904.05	904.73	0.000561	6.62	3946.4	2043.15	0.25
9.647	1% (100-yr)	30800	905.81	906.32	0.000444	6.19	10170.54	2288.68	0.22
9.647	0.5% (200-yr)	36000	907.29	907.72	0.000394	6.07	13607	2370.63	0.21
9.647	0.2% (500-yr)	43400	909.48	909.8	0.000311	5.69	18877.94	2417.76	0.19
9.647	0.133% (750-yr)	48300	911.56	911.79	0.000228	5.11	23919.21	2428.29	0.16
9.647	0.1% (1000-yr)	50600	912.21	912.43	0.000215	5.03	25496.95	2431.57	0.16
9.647	0.04% (2500-yr)	60500	915.46	915.61	0.000154	4.56	33420.98	2440.73	0.14
9.732	10% (10-yr)	15400	897.45	898.3	0.001037	7.39	2084.51	119.42	0.31
9.732	2% (50-yr)	25900	904	905.24	0.001106	8.94	2898.67	2032.75	0.33
9.732	1% (100-yr)	30800	905.89	906.71	0.000861	7.98	8554.51	2276.91	0.29
9.732	0.5% (200-yr)	36000	907.43	908.05	0.000709	7.49	12142.71	2403.71	0.27
9.732	0.2% (500-yr)	43400	909.65	910.04	0.000504	6.61	17623.57	2486.81	0.23
9.732	0.133% (750-yr)	48300	911.7	911.96	0.00034	5.7	22818.78	2604.29	0.19
9.732	0.1% (1000-yr)	50600	912.35	912.58	0.000313	5.55	24517.18	2657.69	0.18
9.732	0.04% (2500-yr)	60500	915.57	915.72	0.000203	4.8	33464.09	2840.74	0.15
9.734501		Bridge							
9.737	10% (10-yr)	15400	897.49	898.33	0.001031	7.37	2088.88	119.48	0.31
9.737	2% (50-yr)	25900	904.54	905.72	0.001024	8.73	2966.79	2066.27	0.32
9.737	1% (100-yr)	30800	906.3	907	0.000761	7.56	9479.97	2317.03	0.28
9.737	0.5% (200-yr)	36000	907.72	908.28	0.000642	7.19	12853.05	2429.68	0.26
9.737	0.2% (500-yr)	43400	909.75	910.13	0.000488	6.52	17875.25	2487.27	0.22
9.737	0.133% (750-yr)	48300	911.79	912.03	0.000332	5.65	23036.38	2611.67	0.19
9.737	0.1% (1000-yr)	50600	912.42	912.65	0.000307	5.51	24716.59	2663.52	0.18
9.737	0.04% (2500-yr)	60500	915.62	915.76	0.000201	4.78	33588.92	2840.74	0.15

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Energy Grade Slope (ft/ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width (ft)	Channel Froude #
9.87	10% (10-yr)	15400	898.37	898.92	0.000648	5.96	2582.15	145.49	0.25
9.87	2% (50-yr)	25900	905.8	906.26	0.000457	5.86	9518.56	2740.81	0.22
9.87	1% (100-yr)	30800	907.03	907.43	0.000424	5.83	12929.25	2814.95	0.21
9.87	0.5% (200-yr)	36000	908.31	908.65	0.000376	5.68	16573.89	2883.7	0.2
9.87	0.2% (500-yr)	43400	910.16	910.42	0.000305	5.36	21963.58	2931.73	0.18
9.87	0.133% (750-yr)	48300	912.06	912.24	0.000221	4.78	27556.37	2965.58	0.16
9.87	0.1% (1000-yr)	50600	912.67	912.84	0.000207	4.68	29382.28	2976.55	0.15
9.87	0.04% (2500-yr)	60500	915.78	915.89	0.000143	4.17	38711.77	3033.55	0.13

A-2.3 SHUNGANUNGA CREEK

A-2.3.1 INTRODUCTION

As part of the feasibility study, hydrologic and hydraulic analyses were conducted on Shunganunga Creek in Topeka, Kansas. To determine the discharges within the Oakland Levee flood protection works, a watershed analysis was completed using the SWMM (Storm Water Management Model) computer software developed by the U.S. Environmental Protection Agency. The hydraulic investigation was completed to calculate water surface profiles along the Oakland Levee Unit from the mouth of Shunganunga Creek to the 10th Street Bridge. To accomplish this, the HEC-RAS (River Analysis System) computer software developed by the Hydrologic Engineering Center, U.S. Army Corps of Engineers was used. The hydraulic model was developed using 1997 survey data supplemented with 1995 four-foot aerial contour maps supplied by the City of Topeka.

A-2.3.2 PURPOSE

The purpose of this investigation is to develop Shunganunga water surface profiles from the Kansas River to the upstream limit of the flood reduction works reflecting the base (or existing) conditions. The resulting hydraulic model will be used to evaluate a series of alternatives for improving the integrity of the existing flood control system.

A-2.3.3 HYDROLOGY

To determine the discharges along Shunganunga Creek, a computer model was created for the basin using the SWMM (Storm Water Management Model) developed by the U.S. Environmental Protection Agency. Using hypothetical rainfall events, discharges were determined for the 0.2, 0.5, 1, 2, 4, 10, and 50-percent exceedance (500, 200, 100, 50, 25, 10, and 2-yr) flood events at ten different locations within the basin. The following sections describe the components of the hydrologic model: basin topography, development of watershed boundaries, loss rates, rainfall-runoff transformation, routing, and hypothetical rainfall. The last two sections show the resulting discharges used in the feasibility study and the hydrologic uncertainty.

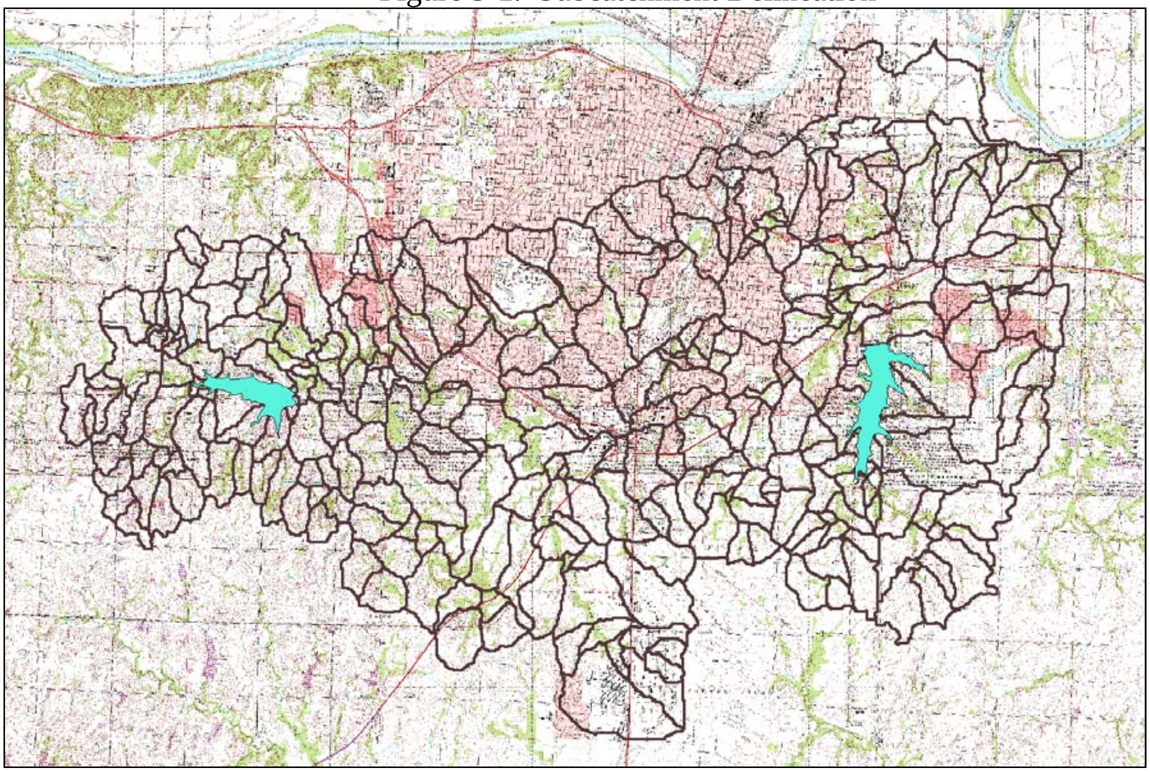
<u>Basin Topography</u>

Shunganunga Creek is a right bank tributary of the Kansas River flowing through Shawnee County, Kansas. The total drainage area of the basin is approximately 75.7 square miles of which 22.5 square miles lie within the city limits of Topeka. The basin is about 20 miles long and 7 miles wide at its widest point. . The land is flat in the lower part of the basin and hilly in the headwater areas. There are four detention dams within the basin. In 1935, Lake Shawnee on Deer Creek, a tributary within the Shunganunga drainage basin, was constructed. However, no provision was made for floodwater storage in this lake. After the disastrous flood of 1951, two more detention basins were constructed. In 1952 and 1953, Burnett Dam on Shunganunga Creek and South Branch Dam on South Branch Shunganunga Creek were constructed. In 1962, Sherwood Lake was constructed upstream from Burnett Dam.

Watershed Boundaries

The watershed was delineated into 299 subcatchments based on surface topography. To complete this task, the computer program HEC-PrePro was used. HEC-PrePro is a developing script for use in ArcView. It is capable of delineating a watershed based on the Digital Elevation Models (DEM) and a given subbasin resolution. Surface topography was obtained from USGS 30-meter DEM's. Figure 1 shows the subcatchment delineation.

Figure 3-1. Subcatchment Delineation



Loss Rates

Loss rates define how much rainfall will be lost to the ground. In this study, the Green-Ampt method was used. This method is dependent on soil characteristics such as initial loss, volume moisture deficit, wetting front suction, and hydraulic conductivity. The soil data for Shawnee County was obtained from the city of Topeka. In the Shunganunga basin, the soils are primarily clay and clay loams with relatively low hydraulic conductivity values.

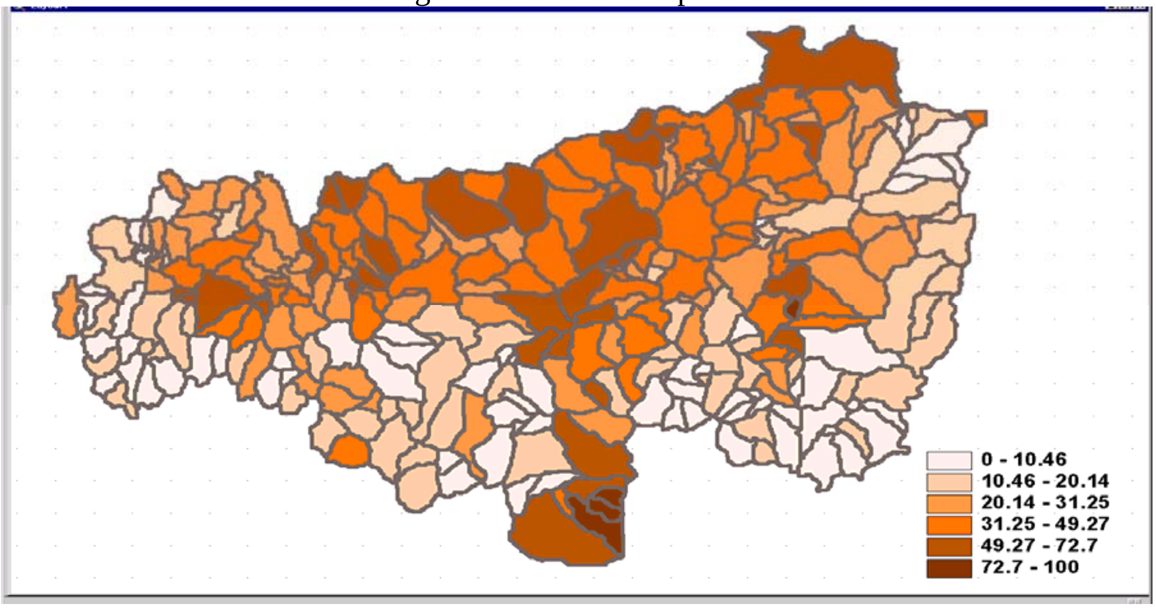
Another important parameter for determining loss rates is the percentage of impervious ground cover such as rooftops and pavement. Percent impervious values were determined from parcel mapping that included land use data. Each land use type was assigned a percent impervious value according to Table 3-1.

Table 3-1 Landuse Percent Impervious Values

Landuse	Fraction Impervious
Agricultural	0.00
Commercial	0.90
Commercial-Office	0.90
Hotel-Motel	0.60
Industrial	0.70
Institutional	0.88
Mobile Home	0.50
Multi-Family (3+)	0.40
None	0.27
Not Codified	0.27
Other Resid. N.E.C.	0.35
Recreational/Open Space	0.15
Single-Family	0.30
Transport-Utility	0.85
Two-Family	0.35
Vacant	0.05
Surface Water	1.00

Parcel polygons were divided according to subcatchment boundaries. Then, for each subcatchment, a composite percent impervious value was calculated based on all the land use parcels it contained. The resulting subcatchment percent impervious values are indicated in Figure 2.

Figure 3-2 Percent Impervious Land



Rainfall-Runoff Transformation

To determine the amount of runoff that results from a particular rainfall event, the Kinematic Wave Routing method was used. This method requires a main channel with one or two overland flow planes defined for each subcatchment. The discharge is calculated using Manning's equation and parameters such as slope, roughness, area, and channel shape and size. Wide, shallow flow is assumed for the overland flow planes. The pervious Manning's roughness coefficient was taken as 0.20, and the impervious Manning's roughness coefficient was taken as 0.014.

Routing

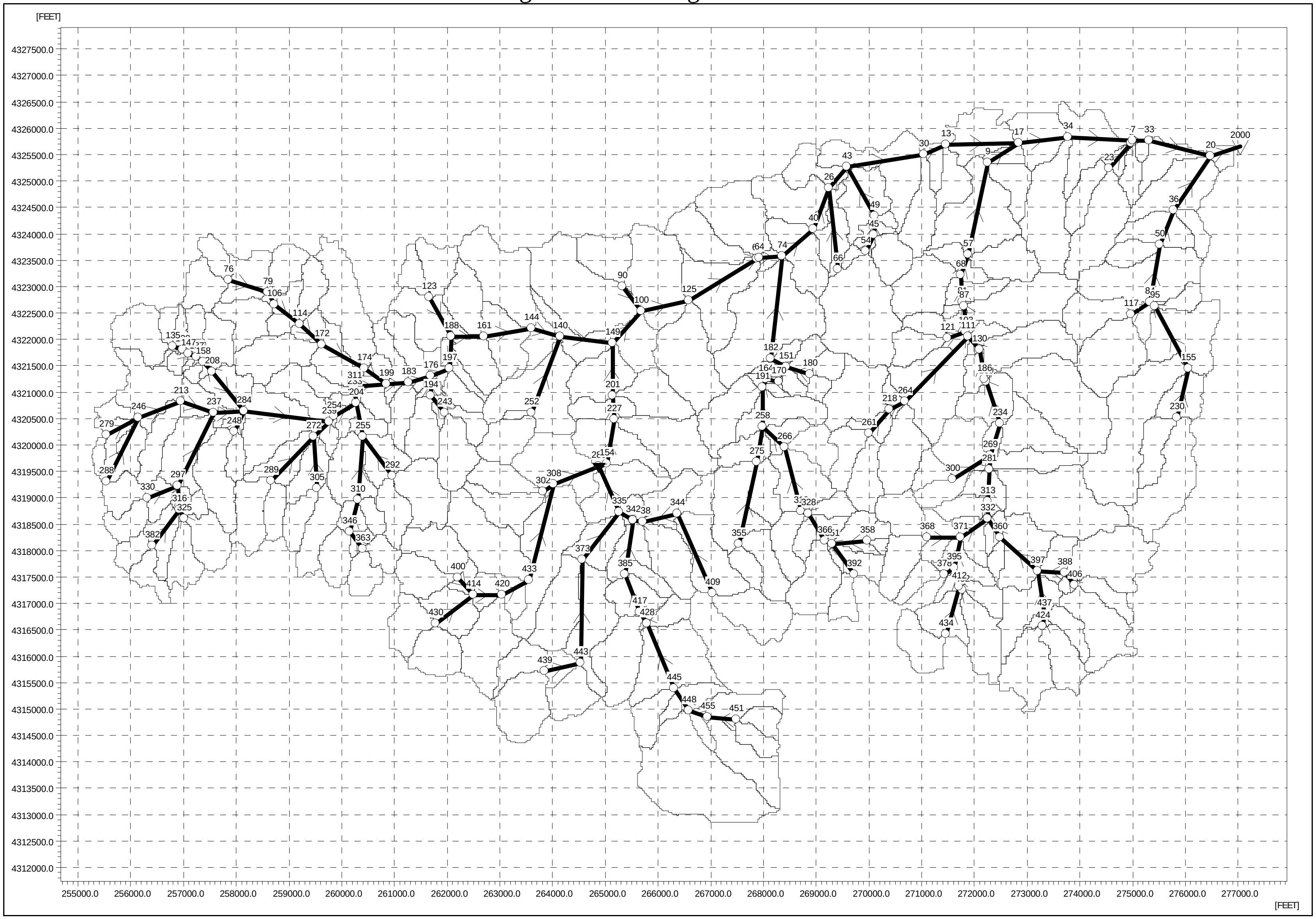
To route the hydrograph from the upstream subcatchments downstream, SWMM EXTRAN was used. EXTRAN is an extremely powerful hydraulic computational engine, which works by finding a complete solution to the St. Venant equations. Consequently, it is capable of simulating backwater effects. EXTRAN is capable of simulating virtually any hydraulic phenomenon including pressurized flow, reverse flow, etc. EXTRAN was chosen for this portion of the model primarily for its ability to simulate the storage and discharge of water in the two dry basins (Burnett Dam and South Branch Dam) within watershed.

The routing component of the model transports the runoff from the individual subareas downstream to the creek and on to the Kansas River. This component of the model consists of a network of channels, or links, which are an attempt to approximate the collection and transport of surface runoff through the Shunganunga Creek tributaries and convey it downstream. Figure 3 on the following page shows the model with the routing network overlaid on the subcatchment boundaries.

The channel links were represented by trapezoidal channels. The majority of the creek's tributaries were represented by channels with bottom widths of 3 feet and side slopes scaled off the USGS quad maps. These channels were given a Manning's roughness value of 0.04. The creek itself was modeled as a trapezoidal channel with a bottom width of roughly 30 feet, 3:1 side slopes, and a roughness of 0.03. This portion of the model was only necessary to propagate peak flows downstream. The creek's actual hydraulic response will be simulated in the HEC-RAS model.

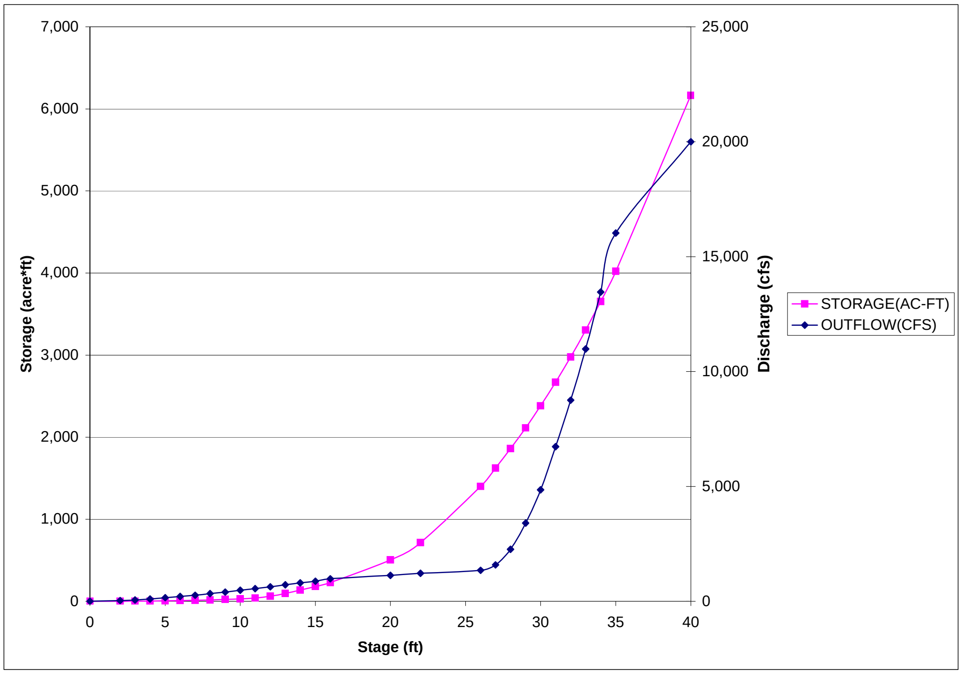
All channel segments in the routing model must start and end at junctions. EXTRAN requires ground and invert elevations at each of these junctions. The junction invert elevations were calculated with ArcView's 3D Spatial Analyst extension. This software package used the USGS DEM's to compute the ground elevations at each node location. These elevations were then assigned to the node invert elevation values in the routing model. The node ground elevations were arbitrarily assigned a value of 30 feet above the inverts, thus giving the channels a maximum flow depth of 30 feet. This depth was never fully utilized.

Figure 3-3 Routing Model Schematic



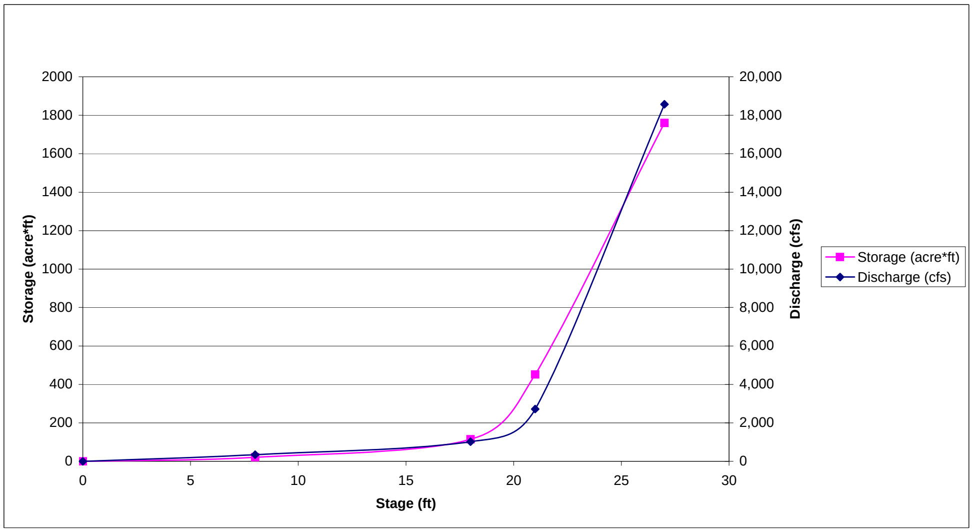
A rating curve of the South Branch dry basin (Figure 4) was taken from the report prepared by White, Martin & Associates in 1993. This curve provided the stage vs. area and discharge vs. stage relationships required to simulate the behavior of this basin.

Figure 3-4 South Branch Dry Basin Rating Curves



For the Burnett dry basin, little information was provided. Therefore the stage vs. area relationship was developed from the USGS contours. For this basin, the dam spillway was assumed to operate like that of the South Branch dam and the rating curve shown in Figure 5 was developed.

Figure 3-5 Burnett Dam Rating Curves



The stage-area relationships were input directly into the EXTRAN model as variable area storage junctions. The discharge spillways were approximated in the model as variable speed pumps whose discharge rates were controlled by the water level in the storage basins. Pump discharge rates vs. water depths were set to approximate the spillways' discharge rating curves.

Hypothetical Rainfall

Finally, synthetic input rainfall hyetographs were developed. These rainfall hyetographs were developed for a range of design storm recurrence intervals ranging from 2 to 500 years. The hyetographs were developed by first selecting 24-hour rainfall totals from the IDF curves (Figure 6). These rainfall totals were then distributed into hourly rainfall volumes according to the SCS Type II rainfall distribution (Figure 7) to develop the synthetic rainfall hyetographs shown in Figure 8.

Figure 3-6 Rainfall IDF Curves

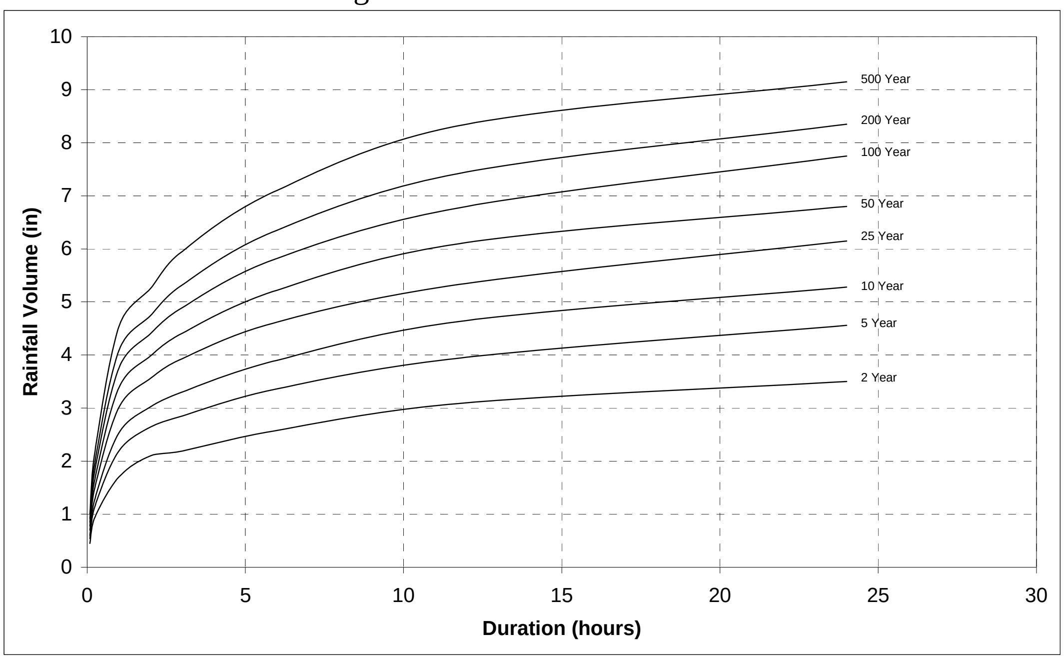


Figure 3-7 SCS Type II Rainfall

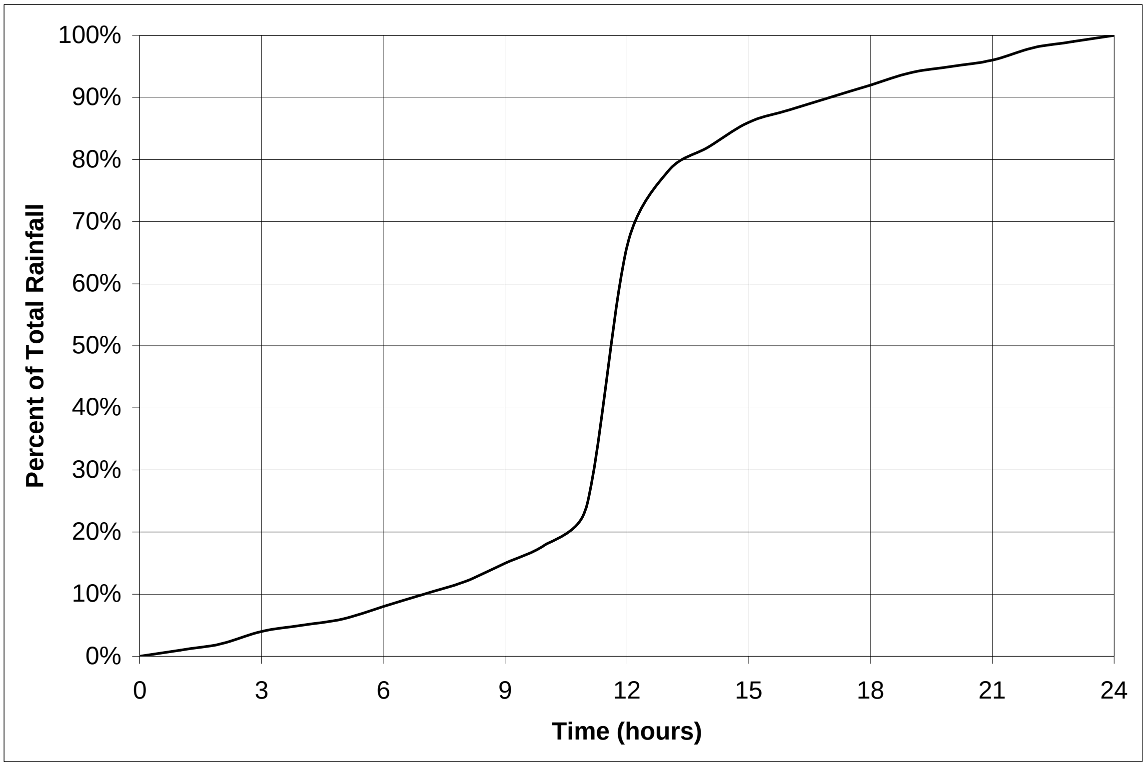
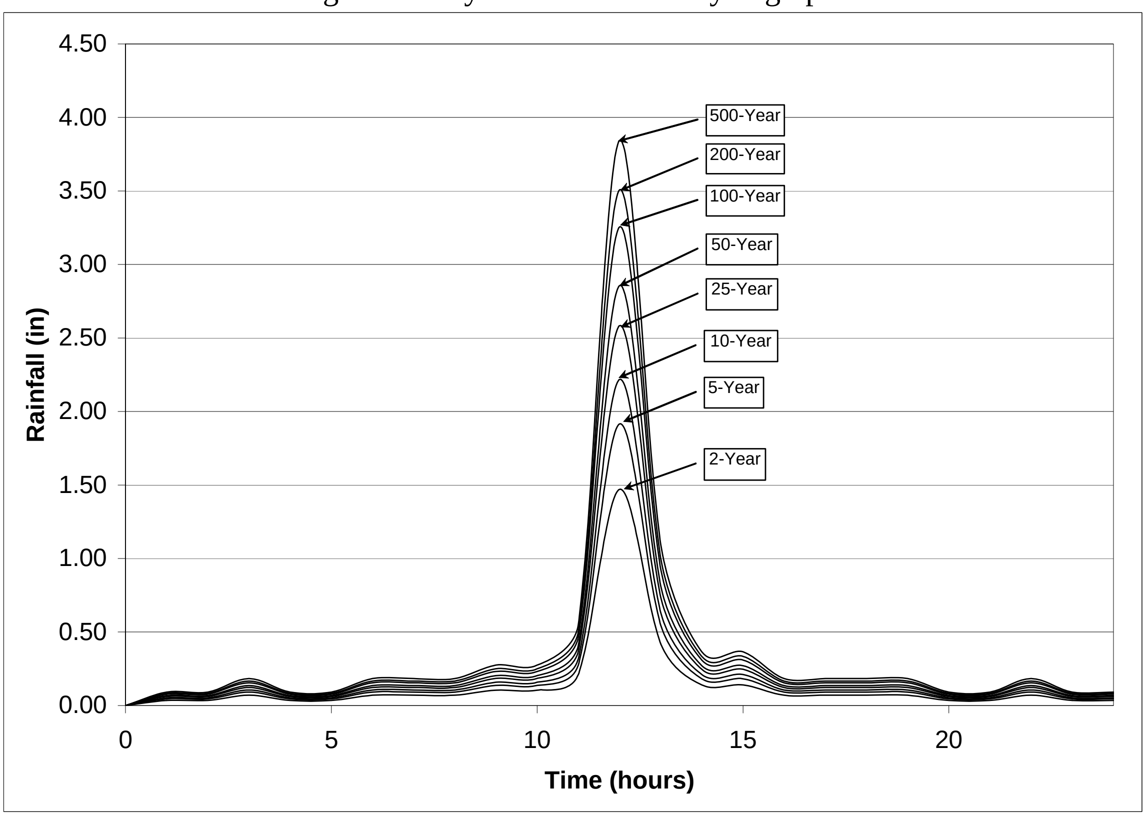


Figure 3-8 Synthetic Rainfall Hyetographs



Feasibility Discharges

By simulating the hypothetical rainfall with the SWMM program, the discharges were determined for the 0.2, 0.5, 1, 2, 4, 10, and 50-percent exceedance (500, 200, 100, 50, 25, 10, and 2-yr) flood events at ten different locations within the basin. The points were consolidated into eight flow change locations in the hydraulic HEC-RAS computer model. The results are shown in Table 3-2.

Table 3-2 Flow Frequency as developed with the SWMM model

Percent Chance of Exceedance	Return Interval (yr)	Discharge (cfs)							
		Station 27054	Station 23003	Station 19198	Station 13895	Station 9689	Station 5659	Station 3210	Station 368
0.2	500	19,400	20,600	21,000	31,100	31,700	32,600	33,500	36,500
0.5	200	17,200	18,200	18,600	27,500	28,000	29,000	29,900	32,100
1	100	15,600	16,500	16,800	24,800	25,300	25,900	26,400	28,900
2	50	13,100	13,900	14,100	20,700	21,000	21,500	22,000	23,900
4	25	11,400	12,100	12,300	17,800	18,100	18,500	18,900	20,600
10	10	9390	9910	10,100	14,500	14,700	15,000	15,400	16,700
20	5	7740	8150	8290	11,800	12,000	12,200	12,500	13,600
50	2	5530	5770	5860	8310	8400	8520	8760	9400

In the Shawnee County, Kansas Flood Insurance Study (FIS) of 1993, the discharges for this entire reach of study for the 10, 50, 100, and 500-year flood events are 10,210 cfs, 17,100 cfs, 20,780 cfs, and 30,750 cfs respectively. At the mouth, the discharges calculated with the SWMM model are higher, and therefore more conservative, than the FIS discharges.

Since flood events above the 0.2% chance exceedance (500-year) event need to be considered in this study, the discharge-frequency curves were extended up to the 0.04% chance exceedance (2500-year) event. To accomplish this, a straight-line extrapolation was used on a log-probability plot of the discharge-frequency events at HEC-RAS river station 3210 (see Plate A2-3-1). The discharges at the other locations were determined by multiplying the results at station 3210 with the average ratio of the known discharges at the area of interest to the discharges at station 3210. Table 3-3 summarizes all of the discharges used on Shunganunga Creek for the existing conditions model.

Table 3-3 Summary of Feasibility Flood Discharges

Percent Chance of Exceedance	Return Interval (yr)	Discharge (cfs)							
		Station 27054	Station 23003	Station 19198	Station 13895	Station 9689	Station 5659	Station 3210	Station 368
0.04	2500	25,200	26,600	27,100	39,400	40,000	41,000	42,000	45,600
0.1	1000	22,200	23,500	23,900	34,700	35,300	36,100	37,000	40,100
0.133	750	21,300	22,500	22,900	33,300	33,800	34,600	35,500	38,500
0.2	500	19,400	20,600	21,000	31,100	31,700	32,600	33,500	36,500
0.5	200	17,200	18,200	18,600	27,500	28,000	29,000	29,900	32,100
1	100	15,600	16,500	16,800	24,800	25,300	25,900	26,400	28,900
2	50	13,100	13,900	14,100	20,700	21,000	21,500	22,000	23,900
10	10	9,390	9,910	10,100	14,500	14,700	15,000	15,400	16,700

Hydrologic Uncertainty

In the past, the Corps of Engineers used freeboard as a factor of safety in designing levees to account for uncertainties in discharge, stage, and other engineering parameters such as geotechnical and structural. Now, the Corps of Engineers has adopted a new methodology called Risk Based Analysis (RBA) for formulating flood risk management projects. This method considers all of the same engineering parameters, but accounts for the uncertainties directly in the analysis in lieu of using freeboard. Using RBA, the project's performance will be expressed as the average return period in years of the largest flood that can be accommodated by the plan under study, with a conditional non-exceedance probability of 90%. The concept of freeboard is no longer used.

To use RBA, the hydrologic uncertainty must be characterized. This information is entered into the computer program HEC-FDA (Flood Damage Analysis), which uses Monte Carlo algorithms to quantify the uncertainties. The uncertainty bands used in this program are based on the effective record lengths used to develop the flow frequency estimates. According to Table 4-5 in EM 1110-2-1619 "Risk Based Analysis for Flood Reduction Studies", the equivalent record length is 15 years for Shunganunga Creek since discharges were estimated with a rainfall-runoff-routing model using textbook parameters.

HEC-FDA calculates the uncertainty either analytically or graphically. For an analytical computation the log Pearson Type III statistics are inputted directly. A graphical approach is used on regulated streams, when the stream gage records are small or incomplete, or when partial duration data is used. For Shunganunga Creek, the discharge-probability curve was defined graphically. HEC-FDA uses the procedures outlined in ETL 1110-2-537 "Uncertainty Estimates for Nonanalytic Frequency Curves" to calculate the error limit curves using order statistics. This is related as standard deviations of the discharge estimate. For the HEC-FDA analysis, an arbitrary index point was selected at HEC-RAS river station 16621 (between Rice and Golden Avenue). Table 3-4 shows the hydrologic uncertainty results at this station.

Table 3-4 Hydrologic Uncertainty on Shunganunga Creek at HEC-RAS river station 16621

Exceedance Probability	Discharge (cfs)	Confidence Limit Curves (standard error) Discharge (cfs)			
		-2 SD	-1 SD	+1 SD	+2 SD
0.999	2260	1370	1760	2900	3720
0.99	2740	1790	2210	3390	4200
0.95	3330	2320	2780	3990	4780
0.9	3730	2680	3160	4390	5170
0.8	4310	3220	3730	4980	5760
0.7	4820	3680	4210	5510	6310
0.5	5860	4560	5170	6640	7530
0.3	7274	5480	6310	8380	9660
0.2	8290	6090	7110	9670	11,290
0.1	10,100	7020	8420	12,110	14,530
0.04	12,300	7990	9910	15,260	18,940
0.02	14,100	8690	11,070	17,960	22,870
0.01	16,800	9650	12,730	22,170	29,260
0.004	18,600	10,240	13,800	25,070	33,800
0.002	21,000	10,970	15,180	29,050	40,190
0.001	23,532	11,700	16,600	33,370	47,320

A-2.3.4 HYDRAULICS

The hydraulic analysis for this report centered on the development of the HEC-RAS computer model for the study reach of Shunganunga Creek at Topeka, Kansas. For this analysis, version 3.0.1 of the HEC-RAS (River Analysis System) developed by the Hydrologic Engineering Center was used. The computer model was calibrated using known water surface elevations and the corresponding discharge. Once calibrated, a series of steady flow water surface profiles were created based on the flood discharges in Table 3-3 above.

<u>Original Design Water Surface Elevations</u>

The elevation of the crown of the existing levee was determined by selecting a design water surface elevation and then adding freeboard to account for uncertainties. For the Oakland Levee Unit the freeboard was three feet. The original design discharges assumed a Kansas River discharge above Soldier Creek of 314,000 cfs and 364,000 cfs below the confluence. The design discharge on Shunganunga creek was 40,000 cfs at the mouth and 27,000 cfs upstream of Deer Creek, which is located at HEC-RAS river station 13895.

<u>Geometric Data</u>

The computer model required cross section geometry along the length of the study reach (see Plate A2-3-2). The information used to create the cross-section geometry was obtained from two sources. The U.S. Army Corps of Engineers provided 1997 cross-section surveys of the channel that covered the entire length of the study reach. The City of Topeka provided four-foot contours, from 1995 aerial mapping that covered the entire study area. In order for the model to more accurately compute friction losses, some of the surveyed sections were copied and modified based on aerial photographs and on-site inspection.

Based on field investigation and review of aerial photography, appropriate Manning's “n” coefficients were selected for each cross section. Values from 0.030 to 0.035 were selected for the channel throughout the entire study reach. Overbank “n” values ranged from 0.040 for well maintained grassy areas to 0.15 for heavily treed areas with dense undergrowth. Higher values of “n” were also used to reduce flow or block out flow in overbanks that were either very wide or contained trees or other obstructions. For the side slopes of the levees, “n” values from 0.035 to 0.045 were used.

The bridge data was obtained from engineering drawings provided by: Kansas Department of Transportation, City of Topeka, Shawnee County, and the Burlington Northern Santa Fe Railroad. The plans for the railroad bridge near the mouth of Shunganunga Creek were not available. The bridge was modeled using plans from a similar bridge upstream of the study limits along with contour data. The plan specifications were used to obtain pier widths and deck thickness, and spot elevations along the railroad track were used to determine the high chord elevation of the bridge deck and embankment. This approximation was deemed satisfactory since this bridge does not significantly affect the water surface profile along the levee during the flood events this study focuses on.

For the cross-sections that did not have a field survey, levee heights were approximated using the “Topeka Flood Protection Project Operation and Maintenance Manual”. There is a well-maintained levee/berm on the right side of Shunganunga Creek, across from the Oakland Levee Unit. It is continuous from the raised Interstate 70 profile, just upstream of the study boundary, through the Branner Street Bridge. Though this levee/berm does not appear pronounced on the contour map, its presence and consistency were verified by on-site inspection.

The lower portion of the study reach, downstream of the levee unit, required some unusual modeling. During the 4% chance and larger events, water is lost over the railroad tracks to the left of the channel. To capture this loss, the railroad berm upstream of Goodell Bridge was modeled as a lateral weir. HEC-RAS calculates the amount of flow spilling over the lateral weir and reduces the downstream flow accordingly. On the cross-sections between Goodell Bridge and the Railroad Bridge, a high ineffective flow area was added at the railroad berm. Therefore, flow to the left of the berm was not considered as contributing flow to the stream. To account for the lateral flow that would be spilling over the berm, the left side of the railroad bridge in the model was coded with the berm elevations. Essentially, the cross-section at the bridge accounted for the lateral flow over the railroad berm between the two bridges.

Starting Water Surface Elevation

Due to the limited amount of gage data available, it was difficult to correlate the peak on Shunganunga Creek with the coincident water surface elevation on the Kansas River. Three different profiles were created on Shunganunga Creek following the illustration of Figure 11-1 in EM 1110-2-1415 “Engineering and Design – Hydrologic Frequency Analysis”. Plate A2-3-3 shows the Shunganunga Creek profiles. The first profile assumed 100-year discharges on both Shunganunga and the Kansas River. The second profile assumed a 2-year water surface elevation on the Kansas River with a 100-yr discharge on Shunganunga Creek. The third profile assumed a 100-year starting water surface elevation on the Kansas River and a 2-year discharge on Shunganunga Creek. As shown in Plate A2-4-3, the majority of the levee is dominated by the discharge on Shunganunga Creek and not the Kansas River starting water surface elevation. Furthermore, upstream of the Rice Bridge at river station 11056, the difference between the first two profiles is diminished to less than half a foot. Since the water surface profile for the majority of the levee is not primarily dependent on the starting water surface elevation, a simplified coincident analysis was used to determine starting water surface elevations.

To simplify the coincident analysis, an empirical table from the Hydraulic Manual from the Texas Department of Highways was used. This relationship is shown in Plate A2-3-4. The empirical table relates annual events based on the relative sizes of the two watersheds up to the 100-year frequency event. Table 3-5 shows the application of the empirical table to the coincident Kansas River flow during a Shunganunga flood event. Above the 100-year frequency events, the Kansas River frequency was estimated. The starting water surface elevation was determined from a rating curve on the Kansas River hydraulic model.

Table 3-5 Coincident Kansas River Discharge And Shunganunga Starting Water Surface Elevation

Shunganunga Creek		Coincident Kansas River		Shunganunga Starting Water Surface Elevation (ft)
Percent Chance of Exceedance	Return Interval (yr)	Percent Chance of Exceedance	Return Interval (yr)	
0.04	2500	0.133	750	877.22
0.1	1000	0.2	500	873.43
0.133	750	0.2	500	873.43
0.2	500	1	100	871.6
0.5	200	2	50	869.01
1	100	10	10	863.02
2	50	20	5	859.4
10	10	50	2	854.14

Calibration

There was limited data available to calibrate the model. Shunganunga Creek only had two short periods with an operating gage. Other than gage readings, no highwater marks with a corresponding discharge could be found. Therefore, the model was calibrated using data from a U.S.G.S. gage that was located at the upstream face of Rice Bridge from May 1980 to September 1981. Other data from a gage located further upstream, from June 1994 to August 1996, were disregarded due to the fact that they were not taken near a surveyed cross section. That is, the geometry at the gage location could not be reproduced accurately enough to calibrate to the relatively low flows recorded by the gage. The calibration discharges were entered as a constant flow throughout the entire length of the model with the downstream boundary condition set to "normal depth."

The calibration of the backwater program to known water surface elevations was accomplished by adjusting the Manning's "n" values for the channel until the profile matches the gage data. In this case, the calibration resulted in "n" values of 0.03 to 0.035 in the channel along the entire study reach.

Table 3-6 presents the results of the calibration. It lists the discharges and water surface elevations from the U.S.G.S. gage data and compares these to the computed water surface profile elevations. Figure 4 shows the calibration discharge profiles and the calibration points.

Table 3-6 Shunganunga Calibration Data

Discharge (cfs)	U.S.G.S. Elevation (ft)	HEC-RAS Model Elevation (ft)
5920	865.24	865.40
2880	860.10	860.95
1280	857.47	857.36
Note: Comparison at HEC-RAS Sta. 12549		

The calibrated backwater model matched the observed stage readings fairly well. However, only one point was used to calibrate the model at three fairly low discharges. Although this is not an ideal calibration, it was the best possible with the limited data available

<u>Shunganunga Creek Existing Condition (Base) Profile</u>

Once the model was calibrated, the existing conditions water surface profiles were generated using the discharges of Table 3-6 above. Plate A2-3-5 shows the 50% non-exceedance probability profiles for the 10, 2, 1, 0.5, 0.2, 0.133, 0.1, and 0.04-percent chance (10, 50, 100, 200, 500, 750, 1000, and 2500-year) flood events. The tabular data is presented in Table 3-7, located at the end of this section.

The HEC-RAS model indicates that the Oakland Levee Unit does not overtop until the water surface elevation reaches the 50% non-exceedance probability stage for the 0.04% chance exceedance (2500-year) event. Discretion should be used when applying profiles higher than the top of the levee. The model used a confined cross sectional area from levee to levee. Essentially, overbank flow beyond the levee height was not taken into consideration. This assumption was made to avoid trying to predict where a levee would fail. Within the Topeka levee systems, there are many different combinations of failure scenarios that could physically occur. Potentially, each could produce a different overbank flow path. HEC-RAS is a one-dimensional steady state model. It is beyond the limitations for HEC-RAS to predict the overbank flow scenarios or to model multi-dimensional flow. Profiles for the rare frequency events that exceed the top of levee are highly speculative and would not necessarily match what would physically happen. These events were produced to formulate frequency-stage curves for economic analyses in the HEC-FDA computer program.

Hydraulic Uncertainty

Uncertainties in computed stage result from two main sources: natural variations in the river and modeling errors. Natural variations include uncertainties in physical factors such as bed forms, debris and other obstructions, channel scour or deposition, sediment transport, and waves. Modeling uncertainty includes factors such as inexact geometry and loss coefficients, variation in hydraulic roughness with season, and error in setting high water marks (EM 1110-2-1619).

In Risk Based Analysis, the stage uncertainty is express as standard deviation (in feet). The total standard deviation depends on the standard deviation based on natural variations and the standard deviation based on model errors according to the formula below:

$$\text{Total Standard Deviation} = \sqrt{S_{natural}{}^{2} + S_{\mathrm{mod}el}{}^{2}}$$

where $S_{natural}$ = standard deviation based on natural variations
S_{model} = standard deviation based on modeling uncertaities

For a ungaged reached, $S_{natural}$ is estimated using Figure 5-3 of the Corps of Engineers Engineering Manual 1110-2-1619 "Risk-Based Analysis for Flood Damage Reduction Studies". This graph shows the stream slope versus the standard deviation of uncertainty for 112 rivers. Based on the graph, $S_{natural}$ for Shunganunga Creek was taken as 0.5 feet.

Table 5-2 in EM 1110-2-1619 quantifies S_{model} based on the quality of topographic data and the reliability of the Manning's n-value. A standard deviation of 1.5 feet was chosen since some of the cross-sections were based on topographical mapping and the Manning's n-values were assumed to have "poor" reliability (due to the limited amount of calibration data available).

Once $S_{natural}$ and S_{model} are known, a total standard deviation can be computed. For this study a total standard deviation of 1.58 ft was computed for the entire discharge set.

A-2.3.5 SUMMARY

First, a hydrologic analysis was completed to determine the expected discharges at the flood reduction works based upon a SWMM computer model of the Shunganunga basin. A hydraulic investigation was conducted on Shunganunga Creek using the HEC-RAS computer software developed by the Hydrologic Engineering Center, U.S. Army Corps of Engineers. The program was used to calculate water surface profiles on approximately the first five miles of Shunganunga Creek adjacent to the Oakland Levee Unit in Topeka, Kansas. The model was calibrated using data from a U.S.G.S. gage that was located at the upstream face of Rice Bridge from May 1980 to September 1981. The 50% non-exceedance probability water surface profiles were then generated for eight different discharge events. These include the 10, 2, 1, 0.5, 0.2, 0.133, 0.1, and 0.04-percent chance (10, 50, 100, 200, 500, 750, 1000, and 2500-year) flood events. The model shows that the existing levees are not overtopped until the 0.04% chance exceedance (2500-year) flood event (with a 50% chance of non-exceedance). Finally, the uncertainty in both stage and discharge were calculated.

Table 3-7 Shunganunga Creek Existing Conditions Water Surface Profiles

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width Width (ft)	Channel Froude #
5	10% 10yr	16700	854.14	856.63	12.92	1541.14	705.03	0.75
5	2% 50yr	23800	859.4	860.58	9.65	3530.66	1250.98	0.45
5	1% 100yr	25900	863.02	863.71	7.62	4972.07	1519.68	0.32
5	0.5% 200yr	23800	869.01	869.26	4.77	7357.16	1687.6	0.17
5	0.2% 500yr	24200	871.6	871.62	1.94	24319.7	1697.8	0.07
5	0.133% 750yr	23500	873.43	873.45	1.61	29763.05	1705.36	0.05
5	0.1% 1000yr	24800	873.43	873.45	1.7	29763.05	1705.36	0.06
5	0.04% 2500yr	20600	877.22	877.23	1.14	36254.5	1720.31	0.04
191	10% 10yr	16700	854	857.87	15.79	1057.66	118.25	0.93
191	2% 50yr	23800	858.13	861.61	15.05	1670.38	669.46	0.75
191	1% 100yr	25900	862.24	864.27	11.75	2586.21	1403.43	0.51
191	0.5% 200yr	23800	868.71	869.45	7.31	4039.2	1479.96	0.27
191	0.2% 500yr	24200	871.55	871.66	3.64	19477.22	1547.21	0.13
191	0.133% 750yr	23500	873.4	873.47	3.07	23462.45	1588.38	0.1
191	0.1% 1000yr	24800	873.39	873.47	3.24	23456.93	1588.31	0.11
191	0.04% 2500yr	20600	877.21	877.24	2.16	29790.3	1773.99	0.07
200				Railroad Bridge				
209	10% 10yr	16700	857.96	859.85	11.05	1511	128.15	0.57
209	2% 50yr	23800	860.98	863.37	12.42	1947.82	909.15	0.58
209	1% 100yr	25900	867.08	868.33	9.13	3129.94	1343.41	0.36
209	0.5% 200yr	23800	871.22	871.4	4.3	15656.38	1370.34	0.15
209	0.2% 500yr	24200	872.96	873.1	3.82	18050.37	1380.04	0.13
209	0.133% 750yr	23500	873.94	874.04	3.48	19395.49	1389.46	0.12
209	0.1% 1000yr	24800	874	874.12	3.65	19489.56	1390.01	0.12
209	0.04% 2500yr	20600	877.24	877.29	2.48	24080.31	1440.25	0.08
368	10% 10yr	16700	858.35	860.16	10.8	1546.93	129.12	0.55
368	2% 50yr	23800	861.36	863.67	12.21	1965.42	152.87	0.56
368	1% 100yr	25900	867.16	868.45	9.24	3036.07	215.16	0.36
368	0.5% 200yr	23800	870.87	871.59	6.99	3904.76	253.03	0.25
368	0.2% 500yr	24200	872.65	873.26	6.49	4371.04	271.18	0.23
368	0.133% 750yr	23500	873.66	874.18	6.02	4650.28	278.51	0.21
368	0.1% 1000yr	24800	873.7	874.27	6.33	4660.92	278.64	0.22
368	0.04% 2500yr	20600	877.08	877.37	4.52	5626.28	294.73	0.14
1575	10% 10yr	15400	860.77	862.03	9.03	1710.43	141.87	0.44

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width Width (ft)	Channel Froude #
1575	2% 50yr	21900	864.03	865.45	9.78	2664.81	362.71	0.43
1575	1% 100yr	23400	868.45	869.16	7.28	4348.31	397.95	0.28
1575	0.5% 200yr	21600	871.54	871.92	5.45	5640.15	440.77	0.2
1575	0.2% 500yr	21200	873.24	873.51	4.72	6974.86	875.3	0.17
1575	0.133% 750yr	20500	874.17	874.39	4.26	7792.07	879.17	0.15
1575	0.1% 1000yr	21700	874.26	874.5	4.47	7873.28	879.55	0.15
1575	0.04% 2500yr	17000	877.38	877.46	2.79	10642.32	907.75	0.09
2414	10% 10yr	15400	861.52	863.35	11.2	1702.19	271.42	0.52
2414	2% 50yr	21900	864.7	866.58	11.87	2696.59	368.06	0.49
2414	1% 100yr	23400	868.73	869.62	8.75	4305.45	416.75	0.33
2414	0.5% 200yr	21600	871.69	872.13	6.4	5548.84	424.91	0.23
2414	0.2% 500yr	21200	873.33	873.65	5.59	6253.14	432.89	0.19
2414	0.133% 750yr	20500	874.24	874.5	5.13	6647.16	443.2	0.17
2414	0.1% 1000yr	21700	874.33	874.63	5.38	6690.83	447.39	0.18
2414	0.04% 2500yr	17000	877.4	877.51	3.42	9441.58	1379.53	0.11
2704	10% 10yr	15400	861.99	863.79	11.06	1682.85	253.25	0.5
2704	2% 50yr	21900	865.07	866.97	11.87	2526.08	405.68	0.49
2704	1% 100yr	23400	868.78	869.85	9.36	3763.17	529.89	0.35
2704	0.5% 200yr	21600	871.69	872.24	6.96	4879.47	564.95	0.25
2704	0.2% 500yr	21200	873.33	873.74	6.08	5539.2	575.25	0.21
2704	0.133% 750yr	20500	874.23	874.57	5.56	5906.21	579.72	0.19
2704	0.1% 1000yr	21700	874.33	874.7	5.83	5945.95	580.2	0.2
2704	0.04% 2500yr	17000	877.4	877.55	3.82	7205.12	595.49	0.12
2809	10% 10yr	15400	862.25	863.95	10.81	1747.54	255.91	0.49
2809	2% 50yr	21900	865.31	867.11	11.6	2636.32	353.74	0.48
2809	1% 100yr	23400	868.91	869.92	9.17	4010.39	400	0.35
2809	0.5% 200yr	21600	871.76	872.28	6.8	5151.06	400	0.24
2809	0.2% 500yr	21200	873.38	873.76	5.94	5798.75	400	0.2
2809	0.133% 750yr	20500	874.27	874.59	5.45	6156.44	400	0.18
2809	0.1% 1000yr	21700	874.38	874.72	5.71	6197.04	400	0.19
2809	0.04% 2500yr	17000	877.41	877.55	3.76	7409.56	400	0.12
2827				Goodell Bridge				
2845	10% 10yr	15400	863.32	864.52	9.11	2145.65	296.89	0.4
2845	2% 50yr	21900	866.34	867.64	9.92	3179.64	370.09	0.4
2845	1% 100yr	23400	869.31	870.18	8.39	4344.73	401.12	0.31

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width Width (ft)	Channel Froude #
2845	0.5% 200yr	21600	871.92	872.4	6.43	5400.54	408.7	0.23
2845	0.2% 500yr	21200	873.48	873.85	5.68	6043.09	412.39	0.2
2845	0.133% 750yr	20500	874.36	874.66	5.23	6403.26	414.44	0.18
2845	0.1% 1000yr	21700	874.47	874.8	5.48	6448.88	414.7	0.18
2845	0.04% 2500yr	17000	877.44	877.59	3.65	7693.05	419.55	0.12
2964	10% 10yr	15400	863.47	864.64	8.98	2191.75	297.77	0.39
2964	2% 50yr	21900	866.44	867.75	9.91	3101.16	315.4	0.4
2964	1% 100yr	23400	869.34	870.25	8.54	4055.6	335.72	0.32
2964	0.5% 200yr	21800	871.91	872.45	6.7	4928.43	343.23	0.24
2964	0.2% 500yr	21400	873.47	873.89	5.99	5465.74	346.89	0.21
2964	0.133% 750yr	20800	874.34	874.69	5.56	5768.17	348.94	0.19
2964	0.1% 1000yr	21900	874.44	874.84	5.83	5805.81	349.2	0.2
2964	0.04% 2500yr	17500	877.43	877.6	3.99	6856.46	354.09	0.13
3210	10% 10yr	15400	863.8	864.85	8.62	2411.6	368.87	0.37
3210	2% 50yr	21900	866.85	867.96	9.32	3630.42	422.16	0.37
3210	1% 100yr	23800	869.62	870.39	8.05	4804.99	424.31	0.3
3210	0.5% 200yr	23200	872.03	872.52	6.6	5828.89	426.18	0.23
3210	0.2% 500yr	23600	873.53	873.94	6.1	6470.31	427.33	0.21
3210	0.133% 750yr	23600	874.38	874.74	5.79	6830.88	427.98	0.2
3210	0.1% 1000yr	24800	874.49	874.88	6.05	6879.65	428.07	0.2
3210	0.04% 2500yr	22400	877.41	877.63	4.65	8132.69	430.58	0.15
4150		Lateral Weir - Spill over Railroad						
4428	10% 10yr	15000	864.71	865.8	8.64	2228.01	536.62	0.37
4428	2% 50yr	21500	867.84	868.8	8.84	4153.44	669.57	0.35
4428	1% 100yr	25900	870.21	870.95	8.24	5768.5	688.07	0.3
4428	0.5% 200yr	29000	872.36	872.92	7.5	7259.07	701.66	0.27
4428	0.2% 500yr	32600	873.77	874.3	7.46	8259.72	711.48	0.26
4428	0.133% 750yr	34600	874.57	875.08	7.42	8828.66	717	0.25
4428	0.1% 1000yr	36100	874.7	875.25	7.66	8925.57	717.93	0.26
4428	0.04% 2500yr	41000	877.47	877.91	7.14	10943.65	748.76	0.23
5659	10% 10yr	15000	865.85	866.62	7.46	2918.14	529.24	0.31
5659	2% 50yr	21500	868.77	869.52	7.92	4534.2	582.46	0.31
5659	1% 100yr	25900	870.84	871.52	7.85	5794.42	621.24	0.29
5659	0.5% 200yr	29000	872.77	873.38	7.61	7083.53	796.61	0.27
5659	0.2% 500yr	32600	874.13	874.75	7.84	8284.29	999.1	0.27

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width Width (ft)	Channel Froude #
5659	0.133% 750yr	34600	874.91	875.5	7.71	9081.41	1018.54	0.26
5659	0.1% 1000yr	36100	875.07	875.69	7.92	9242.9	1021.26	0.27
5659	0.04% 2500yr	41000	877.77	878.22	7.11	12078.05	1084.84	0.23
6926	10% 10yr	14700	866.44	867.43	8.21	2053.9	178.72	0.34
6926	2% 50yr	21000	869.1	870.49	9.85	2627.1	302.24	0.38
6926	1% 100yr	25300	871	872.46	10.36	3405.35	515.43	0.38
6926	0.5% 200yr	28000	872.92	874.15	9.84	4583.9	673.82	0.35
6926	0.2% 500yr	31700	874.32	875.48	9.86	5547.97	706.47	0.34
6926	0.133% 750yr	33800	875.07	876.2	9.85	6087.76	724.12	0.34
6926	0.1% 1000yr	35300	875.23	876.42	10.14	6206.07	727.86	0.34
6926	0.04% 2500yr	40000	877.88	878.74	9.16	8347.7	894.61	0.3
7069	10% 10yr	14700	866.54	867.52	8.15	2073.27	180.18	0.34
7069	2% 50yr	21000	869.24	870.6	9.75	2670.95	318.17	0.38
7069	1% 100yr	25300	871.16	872.58	10.23	3488.35	533.09	0.38
7069	0.5% 200yr	28000	873.04	874.24	9.73	4665.38	676.64	0.34
7069	0.2% 500yr	31700	874.43	875.57	9.77	5626.72	709.07	0.34
7069	0.133% 750yr	33800	875.18	876.28	9.76	6164.04	726.54	0.33
7069	0.1% 1000yr	35300	875.35	876.5	10.05	6288.05	730.45	0.34
7069	0.04% 2500yr	40000	877.95	878.8	9.11	8413.49	898.91	0.29
7091				Oakland Expressway				
7113	10% 10yr	14700	867.19	868.14	8.04	2098.88	168.57	0.33
7113	2% 50yr	21000	869.85	871.24	9.83	2602.33	223.52	0.38
7113	1% 100yr	25300	871.29	872.88	10.72	3297.25	679.74	0.4
7113	0.5% 200yr	28000	873.12	874.41	10.05	4609.11	755.95	0.36
7113	0.2% 500yr	31700	874.49	875.67	9.99	5670.85	785.19	0.35
7113	0.133% 750yr	33800	875.23	876.36	9.91	6265.26	824.4	0.34
7113	0.1% 1000yr	35300	875.42	876.59	10.17	6419.16	847.72	0.35
7113	0.04% 2500yr	40000	878.07	878.87	8.97	8903.09	995.94	0.29
7338	10% 10yr	14700	867.34	868.27	7.96	2125.2	169.63	0.33
7338	2% 50yr	21000	870.06	871.41	9.71	2651.05	265.71	0.37
7338	1% 100yr	25300	871.53	873.07	10.56	3294.67	691	0.39
7338	0.5% 200yr	28000	873.22	874.57	10.2	4166.19	761.35	0.36
7338	0.2% 500yr	31700	874.52	875.86	10.4	4853.85	785.54	0.36
7338	0.133% 750yr	33800	875.22	876.56	10.48	5240.24	823.24	0.36

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width Width (ft)	Channel Froude #
7338	0.1% 1000yr	35300	875.4	876.8	10.79	5341.68	845.26	0.37
7338	0.04% 2500yr	40000	877.97	879.06	9.95	7089.58	994.42	0.32
8114	10% 10yr	14700	867.79	868.75	8.11	2146.51	202.48	0.34
8114	2% 50yr	21000	870.75	871.96	9.38	2861.39	393.73	0.36
8114	1% 100yr	25300	872.27	873.67	10.2	3271.32	728.1	0.38
8114	0.5% 200yr	28000	873.74	875.09	10.18	3883.79	867.89	0.37
8114	0.2% 500yr	31700	875	876.38	10.5	4583.09	1018.68	0.37
8114	0.133% 750yr	33800	875.69	877.07	10.61	4987.83	1146.58	0.37
8114	0.1% 1000yr	35300	875.89	877.34	10.91	5104.84	1172.8	0.37
8114	0.04% 2500yr	40000	878.3	879.49	10.29	6512.62	1278.03	0.34
9323	10% 10yr	14700	868.67	869.62	8.11	2094.98	180.53	0.34
9323	2% 50yr	21000	871.69	872.94	9.46	2674.92	202.93	0.36
9323	1% 100yr	25300	873.25	874.76	10.48	3031.95	271.66	0.39
9323	0.5% 200yr	28000	874.62	876.13	10.6	3374.96	339.11	0.38
9323	0.2% 500yr	31700	875.84	877.46	11.11	3685.62	398.66	0.39
9323	0.133% 750yr	33800	876.49	878.18	11.4	3853.7	985.17	0.4
9323	0.1% 1000yr	35300	876.72	878.51	11.74	3913.77	1002.69	0.41
9323	0.04% 2500yr	40000	878.83	880.58	11.8	4477.14	1107.64	0.39
9503	10% 10yr	14700	868.64	869.99	9.72	1740.5	187.65	0.45
9503	2% 50yr	21000	871.69	873.3	10.81	2270.61	219.84	0.46
9503	1% 100yr	25300	873.29	875.13	11.63	2547.97	247.12	0.47
9503	0.5% 200yr	28000	874.62	876.5	11.8	2778.88	258.43	0.46
9503	0.2% 500yr	31700	875.8	877.87	12.45	2982.72	268.82	0.47
9503	0.133% 750yr	33800	876.42	878.61	12.8	3090.98	728.1	0.48
9503	0.1% 1000yr	35300	876.64	878.96	13.21	3128.69	801.25	0.49
9503	0.04% 2500yr	40000	878.67	881.06	13.43	3481.63	1200.27	0.48
9520				Croco Bridge				
9537	10% 10yr	14700	868.79	870.49	11	1704.62	157.02	0.47
9537	2% 50yr	21000	871.88	874.07	12.73	2210.85	197.3	0.5
9537	1% 100yr	25300	873.52	876.02	13.75	2492.14	216.3	0.52
9537	0.5% 200yr	28000	878.9	880.49	11.25	3417.93	659.91	0.38
9537	0.2% 500yr	31700	879.22	881.2	12.54	3473.75	708.89	0.42
9537	0.133% 750yr	33800	879.35	881.57	13.28	3495.93	728.35	0.45
9537	0.1% 1000yr	35300	879.43	881.83	13.82	3509.63	740.36	0.46

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width Width (ft)	Channel Froude #
9537	0.04% 2500yr	40000	879.42	882.5	15.67	3506.74	737.83	0.53
9689	10% 10yr	14700	869.1	870.76	10.89	1768.77	186.15	0.46
9689	2% 50yr	21000	872.41	874.38	12.2	2446.79	227.07	0.47
9689	1% 100yr	25300	874.07	876.35	13.24	2874.33	288.93	0.5
9689	0.5% 200yr	28000	879.52	880.71	10.15	4878.45	783.06	0.34
9689	0.2% 500yr	31700	880.07	881.47	11.09	5093.13	1008.54	0.37
9689	0.133% 750yr	33800	880.37	881.88	11.6	5207.37	1019.93	0.38
9689	0.1% 1000yr	35300	880.67	882.19	11.74	7017.58	1031.69	0.39
9689	0.04% 2500yr	40000	881.28	882.99	12.59	7660.43	1056.24	0.41
11056	10% 10yr	14500	871.36	872.04	6.83	2662.29	295.35	0.27
11056	2% 50yr	20700	874.9	875.7	7.68	3762.74	319.11	0.28
11056	1% 100yr	24800	876.8	877.77	8.51	4507.06	513.25	0.3
11056	0.5% 200yr	27500	880.83	881.44	7.17	6686.97	665.6	0.23
11056	0.2% 500yr	31100	881.62	882.31	7.7	7123.39	672.5	0.25
11056	0.133% 750yr	33300	882.04	882.78	8.02	7355.01	674.91	0.26
11056	0.1% 1000yr	34700	882.33	883.1	8.2	7517.43	676.6	0.26
11056	0.04% 2500yr	39400	883.13	884.01	8.86	7958.18	681.19	0.28
11935	10% 10yr	14500	871.83	872.5	6.86	2492.66	182.67	0.27
11935	2% 50yr	20700	875.31	876.23	8.11	3227.62	240.07	0.29
11935	1% 100yr	24800	877.27	878.32	8.76	3788.64	402.29	0.3
11935	0.5% 200yr	27500	881.04	881.82	7.81	4986.78	552.68	0.25
11935	0.2% 500yr	31100	881.84	882.75	8.46	5249.43	562.34	0.27
11935	0.133% 750yr	33300	882.27	883.25	8.85	5390.44	567.48	0.28
11935	0.1% 1000yr	34700	882.57	883.6	9.08	5488.71	571.04	0.29
11935	0.04% 2500yr	39400	883.38	884.59	9.9	5761.23	581.52	0.31
12191	10% 10yr	14500	871.93	872.66	7.07	2390.86	176.23	0.28
12191	2% 50yr	20700	875.44	876.4	8.27	3051.11	199.71	0.3
12191	1% 100yr	24800	877.4	878.51	8.95	3461.33	235	0.31
12191	0.5% 200yr	27500	881.08	881.98	8.24	4289.26	419.69	0.27
12191	0.2% 500yr	31100	881.87	882.94	8.96	4470.6	441.46	0.29
12191	0.133% 750yr	33300	882.3	883.46	9.4	4566.89	452.64	0.3
12191	0.1% 1000yr	34700	882.59	883.82	9.66	4633.47	460.37	0.31
12191	0.04% 2500yr	39400	883.39	884.85	10.57	4816.37	481.6	0.33
12209				Rice Bridge				

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width Width (ft)	Channel Froude #
12227	10% 10yr	14500	872.7	873.44	7.24	2429.73	182.65	0.28
12227	2% 50yr	20700	876.16	877.14	8.51	3087.45	208.42	0.3
12227	1% 100yr	24800	878.1	879.22	9.19	3502.35	236.78	0.32
12227	0.5% 200yr	27500	881.74	882.64	8.38	4333.78	425.44	0.27
12227	0.2% 500yr	31100	882.54	883.59	9.11	4517.33	443.34	0.29
12227	0.133% 750yr	33300	882.97	884.12	9.55	4615.65	452.93	0.3
12227	0.1% 1000yr	34700	883.26	884.47	9.81	4683.18	459.51	0.31
12227	0.04% 2500yr	39400	884.07	885.52	10.72	4869.7	477.7	0.33
12549	10% 10yr	14500	872.85	873.64	7.43	2381.86	185.2	0.29
12549	2% 50yr	20700	876.32	877.36	8.71	3089.87	225.18	0.31
12549	1% 100yr	24800	878.29	879.48	9.37	3566.26	257.86	0.32
12549	0.5% 200yr	27500	881.85	882.81	8.63	4608.04	336.46	0.28
12549	0.2% 500yr	31100	882.68	883.79	9.35	4893.76	358.76	0.3
12549	0.133% 750yr	33300	883.12	884.33	9.79	5055.58	370.8	0.31
12549	0.1% 1000yr	34700	883.43	884.69	10.03	5170.7	377.72	0.32
12549	0.04% 2500yr	39400	884.28	885.74	10.87	5500.22	395.53	0.34
13895	10% 10yr	14500	873.71	874.64	8.22	2128.8	170.64	0.33
13895	2% 50yr	20700	877.24	878.47	9.59	2942.07	385.76	0.36
13895	1% 100yr	24800	879.3	880.55	9.96	3766.73	419.03	0.36
13895	0.5% 200yr	27500	882.64	883.54	8.76	5460.88	637.06	0.29
13895	0.2% 500yr	31100	883.66	884.59	9.11	6117.04	650.43	0.3
13895	0.133% 750yr	33300	884.23	885.18	9.32	6486.53	658.09	0.3
13895	0.1% 1000yr	34700	884.6	885.56	9.42	6734.52	663.44	0.31
13895	0.04% 2500yr	39400	885.68	886.69	9.84	7458.06	679.09	0.31
14931	10% 10yr	10100	874.88	875.28	5.55	2255.6	201.96	0.22
14931	2% 50yr	14100	878.71	879.14	6	3229.46	278.62	0.22
14931	1% 100yr	16800	880.76	881.19	6.18	4120.88	758.67	0.21
14931	0.5% 200yr	18600	883.69	883.95	5.21	6981.71	1049.71	0.17
14931	0.2% 500yr	21000	884.76	885.01	5.25	8115.65	1077.23	0.17
14931	0.133% 750yr	22900	885.36	885.61	5.38	8765.21	1095.43	0.17
14931	0.1% 1000yr	23900	885.74	886	5.39	9191.14	1105.66	0.17
14931	0.04% 2500yr	27100	886.89	887.13	5.47	10479.87	1151.52	0.17
16621	10% 10yr	10100	875.51	876.01	6.16	2014.92	175.63	0.25
16621	2% 50yr	14100	879.26	879.81	6.63	2696.36	198.95	0.25
16621	1% 100yr	16800	881.26	881.9	7.29	4535.26	1557.08	0.26

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width Width (ft)	Channel Froude #
16621	0.5% 200yr	18600	884.03	884.35	5.73	9607.96	1952.64	0.19
16621	0.2% 500yr	21000	885.09	885.38	5.65	11696.78	1966.2	0.19
16621	0.133% 750yr	22900	885.7	885.98	5.72	12901.02	1985.35	0.19
16621	0.1% 1000yr	23900	886.09	886.36	5.71	13671.77	2004.87	0.19
16621	0.04% 2500yr	27100	887.23	887.48	5.69	15971.79	2026.1	0.18
17813	10% 10yr	10100	876.08	876.86	7.8	1734.05	229.57	0.34
17813	2% 50yr	14100	879.82	880.46	7.48	2713.46	291.32	0.3
17813	1% 100yr	16800	882.06	882.46	6.68	7034.95	2122.56	0.25
17813	0.5% 200yr	18600	884.44	884.65	5.36	12211.96	2216.07	0.19
17813	0.2% 500yr	21000	885.47	885.66	5.33	14503.46	2244.95	0.19
17813	0.133% 750yr	22900	886.07	886.26	5.41	15854.2	2261.8	0.19
17813	0.1% 1000yr	23900	886.45	886.64	5.41	16714.46	2272.35	0.18
17813	0.04% 2500yr	27100	887.57	887.75	5.42	19286.67	2302.61	0.18
18194	10% 10yr	10100	876.39	877.19	7.82	1622.88	171.48	0.34
18194	2% 50yr	14100	879.97	880.77	8.09	2278.63	197	0.32
18194	1% 100yr	16800	882.06	882.75	7.93	5736.92	1833.77	0.3
18194	0.5% 200yr	18600	884.45	884.8	6.22	10321.55	1998.7	0.22
18194	0.2% 500yr	21000	885.49	885.79	6.12	12406.26	2042.07	0.21
18194	0.133% 750yr	22900	886.09	886.39	6.2	13642.18	2081.84	0.21
18194	0.1% 1000yr	23900	886.47	886.76	6.17	14437.5	2095.95	0.21
18194	0.04% 2500yr	27100	887.58	887.88	6.43	16866	2271.15	0.22
18212				Golden Bridge				
18230	10% 10yr	10100	876.61	877.27	7.3	1774.18	171.47	0.3
18230	2% 50yr	14100	880.13	880.83	7.81	2582.83	1454.61	0.3
18230	1% 100yr	16800	882.24	882.77	7.39	5875.59	1680.7	0.27
18230	0.5% 200yr	18600	884.47	884.83	6.45	9963.19	1955.14	0.22
18230	0.2% 500yr	21000	885.5	885.82	6.37	11992.6	1987.32	0.22
18230	0.133% 750yr	22900	886.1	886.42	6.45	13197.42	2020.31	0.22
18230	0.1% 1000yr	23900	886.48	886.79	6.43	13966.09	2028.49	0.22
18230	0.04% 2500yr	27100	887.58	887.91	6.78	16279.8	2177.15	0.22
18571	10% 10yr	10100	876.81	877.57	7.72	1641.31	156.75	0.32
18571	2% 50yr	14100	880.28	881.11	8.35	2647.34	1618.22	0.32
18571	1% 100yr	16800	882.45	882.95	7.24	6541.98	1901.85	0.26
18571	0.5% 200yr	18600	884.73	884.94	5.38	11067.61	2066.32	0.19
18571	0.2% 500yr	21000	885.75	885.92	5.17	13183.79	2090.43	0.18

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width Width (ft)	Channel Froude #
18571	0.133% 750yr	22900	886.35	886.52	5.16	14453.18	2104.77	0.17
18571	0.1% 1000yr	23900	886.73	886.89	5.1	15243.36	2113.64	0.17
18571	0.04% 2500yr	27100	887.87	888.01	4.96	17677.3	2138.34	0.16
19198	10% 10yr	10100	877.31	877.99	7.31	1807.41	166.19	0.31
19198	2% 50yr	14100	880.72	881.51	8.12	2777.42	873.29	0.31
19198	1% 100yr	16800	882.54	883.36	8.53	4233.19	1253.64	0.31
19198	0.5% 200yr	18600	884.73	885.18	6.97	7849.84	1516.3	0.25
19198	0.2% 500yr	21000	885.73	886.14	6.9	9404.35	1568.97	0.24
19198	0.133% 750yr	22900	886.34	886.73	6.97	10356.21	1598.58	0.24
19198	0.1% 1000yr	23900	886.71	887.09	6.96	10956.01	1618.17	0.24
19198	0.04% 2500yr	27100	887.85	888.2	6.9	12833.69	1668.9	0.23
19815	10% 10yr	9900	877.59	878.35	7.56	1614.8	153.27	0.31
19815	2% 50yr	13900	880.99	881.86	8.27	2191.9	190.46	0.32
19815	1% 100yr	16500	882.8	883.71	8.62	2603.59	251.82	0.32
19815	0.5% 200yr	18200	884.89	885.39	7.16	8092.17	1656.25	0.25
19815	0.2% 500yr	20600	885.87	886.35	7.26	9740.76	1708.5	0.25
19815	0.133% 750yr	22500	886.46	886.95	7.44	10760.9	1741.17	0.25
19815	0.1% 1000yr	23500	886.82	887.3	7.47	11401.01	1760.67	0.25
19815	0.04% 2500yr	26600	887.95	888.41	7.53	13408.86	1810.78	0.25
20002	10% 10yr	9900	877.7	878.46	7.54	1600.38	145.56	0.31
20002	2% 50yr	13900	881.08	881.97	8.36	2135.17	179.36	0.32
20002	1% 100yr	16500	882.87	883.82	8.77	2535.33	248.76	0.32
20002	0.5% 200yr	18200	884.98	885.45	7.02	8384.17	1726.04	0.25
20002	0.2% 500yr	20600	885.99	886.42	7.03	10076.79	1762.57	0.24
20002	0.133% 750yr	22500	886.58	887.02	7.2	11214.94	1821.19	0.25
20002	0.1% 1000yr	23500	886.94	887.36	7.22	11870.09	1841.06	0.24
20002	0.04% 2500yr	26600	888.05	888.46	7.28	13986.99	1987.05	0.24
20020				Pedestrian Bridge				
20039	10% 10yr	9900	877.78	878.54	7.52	1606.08	146.19	0.31
20039	2% 50yr	13900	881.14	882.03	8.35	2139.49	179.24	0.32
20039	1% 100yr	16500	882.96	883.89	8.73	2544.67	246.33	0.32
20039	0.5% 200yr	18200	885.03	885.5	7.05	7811.32	1732.43	0.25
20039	0.2% 500yr	20600	886.03	886.46	7.02	10299.84	1799.24	0.24
20039	0.133% 750yr	22500	886.63	887.06	7.16	11386.36	1840.48	0.24

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width Width (ft)	Channel Froude #
20039	0.1% 1000yr	23500	886.99	887.41	7.19	12048.84	1884.34	0.24
20039	0.04% 2500yr	26600	888.12	888.51	7.22	14220.97	1996.72	0.24
20230	10% 10yr	9900	877.87	878.65	7.61	1580.64	146.54	0.32
20230	2% 50yr	13900	881.24	882.15	8.43	2126.56	185.12	0.32
20230	1% 100yr	16500	883.06	884	8.76	2551.64	254.45	0.32
20230	0.5% 200yr	18200	885.04	885.59	7.44	6373.31	1881.8	0.26
20230	0.2% 500yr	20600	886.12	886.53	6.92	10833.86	1954.06	0.24
20230	0.133% 750yr	22500	886.72	887.12	7.02	12020	1985.04	0.24
20230	0.1% 1000yr	23500	887.09	887.48	7.02	12747.42	2003.76	0.24
20230	0.04% 2500yr	26600	888.21	888.57	6.99	15043.04	2065.89	0.23
20776	10% 10yr	9900	878.12	879.04	8.21	1488.69	142.04	0.36
20776	2% 50yr	13900	881.48	882.52	8.97	2005.69	843.13	0.35
20776	1% 100yr	16500	883.24	884.39	9.57	2561.3	1686.15	0.36
20776	0.5% 200yr	18200	884.87	886.02	9.69	3692.68	2105	0.35
20776	0.2% 500yr	20600	886.14	886.79	8.15	10646.27	2178.26	0.29
20776	0.133% 750yr	22500	886.76	887.37	8.17	12000.38	2209.9	0.29
20776	0.1% 1000yr	23500	887.13	887.71	8.11	12828.12	2229.28	0.28
20776	0.04% 2500yr	26600	888.28	888.78	7.91	15416.66	2285.43	0.27
21730	10% 10yr	9900	878.81	879.89	8.8	1359.43	149.56	0.41
21730	2% 50yr	13900	882.09	883.3	9.54	1955.31	235.64	0.4
21730	1% 100yr	16500	883.89	885.13	9.85	2483.46	1305.04	0.4
21730	0.5% 200yr	18200	885.67	886.64	9.06	3281.7	2058.22	0.35
21730	0.2% 500yr	20600	886.57	887.2	8.06	8488.28	2097.26	0.3
21730	0.133% 750yr	22500	887.21	887.77	7.9	9828.39	2124.27	0.29
21730	0.1% 1000yr	23500	887.57	888.09	7.78	10597.35	2138.48	0.29
21730	0.04% 2500yr	26600	888.69	889.11	7.41	13033.03	2186.81	0.27
21939	10% 10yr	9900	879.13	880.1	8.43	1450.44	158.62	0.39
21939	2% 50yr	13900	882.47	883.49	8.89	2021.2	181.4	0.37
21939	1% 100yr	16500	884.3	885.31	9.13	3294.82	1700.93	0.37
21939	0.5% 200yr	18200	886.11	886.78	7.98	6615.22	1911.85	0.31
21939	0.2% 500yr	20600	886.62	887.32	8.38	7586.09	1933.63	0.32
21939	0.133% 750yr	22500	887.22	887.89	8.41	8770.53	1989.71	0.31
21939	0.1% 1000yr	23500	887.57	888.21	8.35	9474.54	2004.91	0.31
21939	0.04% 2500yr	26600	888.68	889.23	8.09	11734.48	2061	0.29

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width Width (ft)	Channel Froude #
21962				4th Street Bridge				
21985	10% 10yr	9900	879.24	880.19	8.34	1468.31	159.56	0.39
21985	2% 50yr	13900	882.61	883.6	8.8	2045.46	182.24	0.37
21985	1% 100yr	16500	884.46	885.43	8.99	3570.62	1782.86	0.36
21985	0.5% 200yr	18200	886.23	886.87	7.84	6845.53	1917.04	0.3
21985	0.2% 500yr	20600	886.75	887.42	8.22	7840.33	1939.29	0.31
21985	0.133% 750yr	22500	887.34	887.98	8.27	9008.75	1994.87	0.31
21985	0.1% 1000yr	23500	887.69	888.3	8.21	9702.06	2009.8	0.3
21985	0.04% 2500yr	26600	888.77	889.3	7.99	11920.57	2064.5	0.29
22032	10% 10yr	9900	879.29	880.24	8.07	1369.65	144.85	0.4
22032	2% 50yr	13900	882.63	883.65	8.53	1885.13	164.06	0.38
22032	1% 100yr	16500	884.42	885.5	8.87	2187.92	1438.35	0.37
22032	0.5% 200yr	18200	885.93	887.04	9.02	2465.98	1942.87	0.36
22032	0.2% 500yr	20600	886.61	887.52	8.69	6717.69	1991.95	0.34
22032	0.133% 750yr	22500	887.21	888.08	8.71	7927.47	2017.97	0.34
22032	0.1% 1000yr	23500	887.56	888.39	8.66	8636.97	2033.09	0.34
22032	0.04% 2500yr	26600	888.67	889.38	8.43	10906.26	2080.89	0.32
22062	10% 10yr	9900	879.51	880.29	7.29	1512.58	146.15	0.35
22062	2% 50yr	13900	882.8	883.69	7.92	2024.89	165.05	0.34
22062	1% 100yr	16500	884.58	885.54	8.33	2327.08	1500.66	0.34
22062	0.5% 200yr	18200	886.37	887.1	7.64	6350.58	1981.56	0.3
22062	0.2% 500yr	20600	886.72	887.55	8.25	7056.84	1996.87	0.32
22062	0.133% 750yr	22500	887.3	888.11	8.34	8215.65	2021.75	0.32
22062	0.1% 1000yr	23500	887.64	888.42	8.33	8893.41	2036.18	0.31
22062	0.04% 2500yr	26600	888.7	889.4	8.21	11092.37	2082.45	0.3
22195	10% 10yr	9900	879.62	880.39	7.25	1534.75	147.98	0.34
22195	2% 50yr	13900	882.89	883.78	7.93	2052.75	168.85	0.34
22195	1% 100yr	16500	884.69	885.63	8.29	2733.47	1356.4	0.34
22195	0.5% 200yr	18200	886.36	887.21	8.02	3944.19	1903.04	0.31
22195	0.2% 500yr	20600	886.67	887.69	8.86	4168.77	1914.79	0.34
22195	0.133% 750yr	22500	887.17	888.29	9.31	4533.33	1933.83	0.35
22195	0.1% 1000yr	23500	887.46	888.61	9.51	4745.39	1944.89	0.36
22195	0.04% 2500yr	26600	888.4	889.65	10.03	5432.52	1980.77	0.37
23003	10% 10yr	9900	880.12	881.28	9.33	1289.33	140.42	0.43

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width Width (ft)	Channel Froude #
23003	2% 50yr	13900	883.34	884.57	9.88	1778.64	163.91	0.41
23003	1% 100yr	16500	885.17	886.35	10.03	2738.54	1110.11	0.4
23003	0.5% 200yr	18200	886.81	887.76	9.37	4302.05	1531.97	0.36
23003	0.2% 500yr	20600	887.22	888.35	10.27	4718.26	1639.55	0.39
23003	0.133% 750yr	22500	887.61	889.1	11.52	5151.96	1915.11	0.43
23003	0.1% 1000yr	23500	887.98	889.4	11.44	5602.65	1938.33	0.42
23003	0.04% 2500yr	26600	889.15	890.34	11	7040.76	2058.6	0.4
23632	10% 10yr	9400	881.07	882.19	9.05	1248.36	134.99	0.42
23632	2% 50yr	13100	884.14	885.38	9.79	1695.9	164.23	0.41
23632	1% 100yr	15600	885.77	887.19	10.58	2043.96	263.15	0.42
23632	0.5% 200yr	17200	887.19	888.51	10.39	2487.45	1418.12	0.4
23632	0.2% 500yr	19400	887.69	889.19	11.2	2682.74	1906.28	0.43
23632	0.133% 750yr	21300	888.34	890	11.82	3033.55	2850.43	0.44
23632	0.1% 1000yr	22200	888.67	890.26	11.75	3244.27	2889.13	0.44
23632	0.04% 2500yr	25200	889.72	891.12	11.49	3907.16	2956.5	0.42
23801	10% 10yr	9400	881.35	882.37	8.7	1301.02	133.79	0.4
23801	2% 50yr	13100	884.4	885.54	9.42	1741.5	155.02	0.39
23801	1% 100yr	15600	886.15	887.36	9.83	2022.52	165.83	0.39
23801	0.5% 200yr	17200	887.46	888.65	9.88	2244.19	1450.17	0.38
23801	0.2% 500yr	19400	887.91	889.35	10.85	2324.58	1902.19	0.41
23801	0.133% 750yr	21300	888.62	890.17	11.31	2453.46	2719.9	0.42
23801	0.1% 1000yr	22200	888.8	890.43	11.64	2485.73	2818.21	0.43
23801	0.04% 2500yr	25200	889.54	891.41	12.55	2621.07	2942.38	0.46
23832				Branner Bridge				
23863	10% 10yr	9400	881.64	882.73	8.96	1308.07	135.2	0.41
23863	2% 50yr	13100	884.65	885.91	9.86	1746.15	158.52	0.41
23863	1% 100yr	15600	886.25	887.65	10.51	2015.14	178.15	0.42
23863	0.5% 200yr	17200	887.55	888.95	10.61	2256.79	1546.71	0.41
23863	0.2% 500yr	19400	888.03	889.69	11.59	2352.13	2548.44	0.44
23863	0.133% 750yr	21300	888.75	890.52	12.06	2497.59	2735.17	0.45
23863	0.1% 1000yr	22200	888.94	890.81	12.4	2535.86	2764.89	0.46
23863	0.04% 2500yr	25200	889.72	891.85	13.35	2693.86	2841.02	0.49
23998	10% 10yr	9400	881.76	882.85	8.94	1325.3	136.03	0.4
23998	2% 50yr	13100	884.76	886.03	9.9	1762.92	159.81	0.41
23998	1% 100yr	15600	886.35	887.78	10.57	2033.05	179.38	0.42

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width Width (ft)	Channel Froude #
23998	0.5% 200yr	17200	887.63	889.07	10.7	2273.5	1608.33	0.41
23998	0.2% 500yr	19400	888.13	889.83	11.69	2371.83	2569.6	0.44
23998	0.133% 750yr	21300	888.83	890.68	12.28	2515.67	2752.55	0.46
23998	0.1% 1000yr	22200	889.01	890.97	12.65	2555.06	2771.07	0.47
23998	0.04% 2500yr	25200	889.74	892.05	13.79	2716.43	2843.45	0.51
24276	10% 10yr	9400	881.89	883.3	10.26	1137.82	126.68	0.48
24276	2% 50yr	13100	884.88	886.42	11.02	1545.36	146.44	0.47
24276	1% 100yr	15600	886.5	888.15	11.55	1792.51	382.28	0.47
24276	0.5% 200yr	17200	887.78	889.41	11.59	1999.49	1714.04	0.45
24276	0.2% 500yr	19400	888.31	890.22	12.59	2088.02	2495.63	0.49
24276	0.133% 750yr	21300	889.03	891.08	13.14	2210.5	2645.98	0.5
24276	0.1% 1000yr	22200	889.23	891.39	13.5	2245.02	2656.48	0.51
24276	0.04% 2500yr	25200	890.03	892.5	14.52	2385.7	2768.65	0.54
24317				6th Street Bridge				
24358	10% 10yr	9400	882.37	883.65	9.84	1183.64	129.84	0.45
24358	2% 50yr	13100	885.3	886.71	10.62	1591.65	148.98	0.45
24358	1% 100yr	15600	886.98	888.48	11.07	1852.5	1025.45	0.45
24358	0.5% 200yr	17200	888.27	889.75	11.1	2063.46	2489.82	0.43
24358	0.2% 500yr	19400	888.94	890.63	11.91	2177.62	2565.27	0.46
24358	0.133% 750yr	21300	889.75	891.54	12.36	2316.85	2682.2	0.46
24358	0.1% 1000yr	22200	890.02	891.89	12.64	2363.56	2766.07	0.47
24358	0.04% 2500yr	25200	891.03	893.1	13.4	2543.15	2907.28	0.49
24516	10% 10yr	9400	882.57	883.86	9.88	1199.31	129.55	0.45
24516	2% 50yr	13100	885.45	886.91	10.77	1596.2	146.33	0.45
24516	1% 100yr	15600	887.11	888.68	11.3	1847.55	156.07	0.45
24516	0.5% 200yr	17200	888.37	889.94	11.39	2048.94	163.03	0.44
24516	0.2% 500yr	19400	889.04	890.84	12.26	2159	166.15	0.47
24516	0.133% 750yr	21300	889.83	891.76	12.77	2292	170.69	0.48
24516	0.1% 1000yr	22200	890.1	892.12	13.09	2338.65	172.65	0.49
24516	0.04% 2500yr	25200	891.09	893.36	13.98	2512.21	179.15	0.51
25468	10% 10yr	9400	883.74	885.27	10.92	1105.32	128.21	0.5
25468	2% 50yr	13100	886.55	888.21	11.73	1490.45	146.54	0.49
25468	1% 100yr	15600	888.19	889.93	12.18	1739.74	157.07	0.49
25468	0.5% 200yr	17200	889.38	891.1	12.22	1930.47	163.79	0.48
25468	0.2% 500yr	19400	890.18	892.09	12.98	2062.98	168.31	0.5

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width Width (ft)	Channel Froude #
25468	0.133% 750yr	21300	891.02	893.03	13.4	2206.99	173.08	0.5
25468	0.1% 1000yr	22200	891.33	893.41	13.66	2260.7	174.82	0.51
25468	0.04% 2500yr	25200	892.45	894.71	14.35	2460.09	180.66	0.52
26024	10% 10yr	9400	885.03	886.11	9.42	1290	148.6	0.47
26024	2% 50yr	13100	887.78	888.97	10.13	1720.21	166	0.45
26024	1% 100yr	15600	889.39	890.66	10.55	1996.37	176.91	0.45
26024	0.5% 200yr	17200	890.49	891.78	10.68	2196.14	184.4	0.44
26024	0.2% 500yr	19400	891.4	892.81	11.27	2365.88	190.53	0.45
26024	0.133% 750yr	21300	892.21	893.76	11.89	2525.17	208.37	0.47
26024	0.1% 1000yr	22200	892.52	894.16	12.24	2591.97	219.86	0.48
26024	0.04% 2500yr	25200	893.78	895.49	12.64	2876.23	228.42	0.48
26165	10% 10yr	9400	885.4	886.29	8.71	1413.92	160.39	0.43
26165	2% 50yr	13100	888.15	889.14	9.36	1874.92	176.6	0.42
26165	1% 100yr	15600	889.76	890.82	9.77	2168.4	187.66	0.41
26165	0.5% 200yr	17200	890.85	891.93	9.91	2377.74	195.2	0.41
26165	0.2% 500yr	19400	891.79	892.97	10.45	2563.5	201.74	0.42
26165	0.133% 750yr	21300	892.69	893.94	10.77	2748.77	207.65	0.42
26165	0.1% 1000yr	22200	893.07	894.34	10.94	2826.74	210.04	0.42
26165	0.04% 2500yr	25200	894.23	895.67	11.71	3077.67	228.59	0.44
26339	10% 10yr	9400	885.65	886.49	8.48	1453.47	161.63	0.41
26339	2% 50yr	13100	888.36	889.32	9.19	1913.26	178.08	0.41
26339	1% 100yr	15600	889.97	890.99	9.61	2207.4	189.08	0.4
26339	0.5% 200yr	17200	891.05	892.09	9.77	2416.01	196.57	0.4
26339	0.2% 500yr	19400	892	893.14	10.3	2605.15	203.18	0.41
26339	0.133% 750yr	21300	892.9	894.11	10.62	2791.9	208.98	0.41
26339	0.1% 1000yr	22200	893.28	894.52	10.79	2870.9	211.38	0.42
26339	0.04% 2500yr	25200	894.52	895.85	11.24	3138.28	239.23	0.42
26382				10th Street Bridge				
26425	10% 10yr	9400	886.02	886.68	7.19	1682.71	178.36	0.33
26425	2% 50yr	13100	888.75	889.51	7.89	2189.11	192.32	0.34
26425	1% 100yr	15600	890.37	891.19	8.3	2506.53	213.5	0.34
26425	0.5% 200yr	17200	891.46	892.29	8.44	2726.67	239.93	0.33
26425	0.2% 500yr	19400	892.44	893.36	8.87	2930.2	251.39	0.34
26425	0.133% 750yr	21300	893.37	894.34	9.15	3123.14	254.98	0.35

HEC-RAS River Station	Profile	Q Total (cfs)	Water Surface Elevation (ft)	Energy Grade Elevation (ft)	Average Channel Velocity (ft/s)	Flow Area (sq ft)	Top Width Width (ft)	Channel Froude #
26425	0.1% 1000yr	22200	893.76	894.76	9.3	3204.55	256.49	0.35
26425	0.04% 2500yr	25200	895.04	896.12	9.74	3474.3	261.44	0.35
		0						
26593	10% 10yr	9400	886.17	886.78	6.95	1708.78	179.11	0.32
26593	2% 50yr	13100	888.9	889.61	7.61	2217.83	193.08	0.32
26593	1% 100yr	15600	890.48	891.28	8.22	2533.24	217.03	0.33
26593	0.5% 200yr	17200	891.54	892.38	8.51	2777.97	241.41	0.34
26593	0.2% 500yr	19400	892.54	893.45	8.9	3027.01	251.76	0.34
26593	0.133% 750yr	21300	893.49	894.43	9.07	3268.73	255.45	0.34
26593	0.1% 1000yr	22200	893.91	894.85	9.16	3374.7	257.05	0.34
26593	0.04% 2500yr	25200	895.23	896.22	9.46	3718.61	262.18	0.34
		0						
26772	10% 10yr	9400	886.27	886.88	6.95	1682.84	178.7	0.32
26772	2% 50yr	13100	889	889.71	7.66	2200.18	203.2	0.33
26772	1% 100yr	15600	890.61	891.38	8.08	2541.61	221.68	0.33
26772	0.5% 200yr	17200	891.7	892.49	8.23	2792.38	235.2	0.32
26772	0.2% 500yr	19400	892.72	893.56	8.59	3034.61	241.88	0.33
26772	0.133% 750yr	21300	893.66	894.53	8.78	3264.66	245.9	0.33
26772	0.1% 1000yr	22200	894.06	894.95	8.89	3364.3	247.62	0.33
26772	0.04% 2500yr	25200	895.38	896.33	9.2	3695.21	253.26	0.33
		0						
27054	10% 10yr	9400	886.39	887.06	7.25	1615.54	177.75	0.34
27054	2% 50yr	13100	889.11	889.88	7.88	2129.91	197.07	0.34
27054	1% 100yr	15600	890.73	891.54	8.19	2454.14	203.41	0.33
27054	0.5% 200yr	17200	891.83	892.65	8.3	2679.62	207.7	0.33
27054	0.2% 500yr	19400	892.83	893.73	8.73	2890.48	213.45	0.34
27054	0.133% 750yr	21300	893.76	894.7	9.01	3090.9	219.11	0.34
27054	0.1% 1000yr	22200	894.16	895.13	9.15	3178.52	221.54	0.34
27054	0.04% 2500yr	25200	896.11	896.48	6.69	6734.73	2104.36	0.24

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CHAPTER A-2

PLATES

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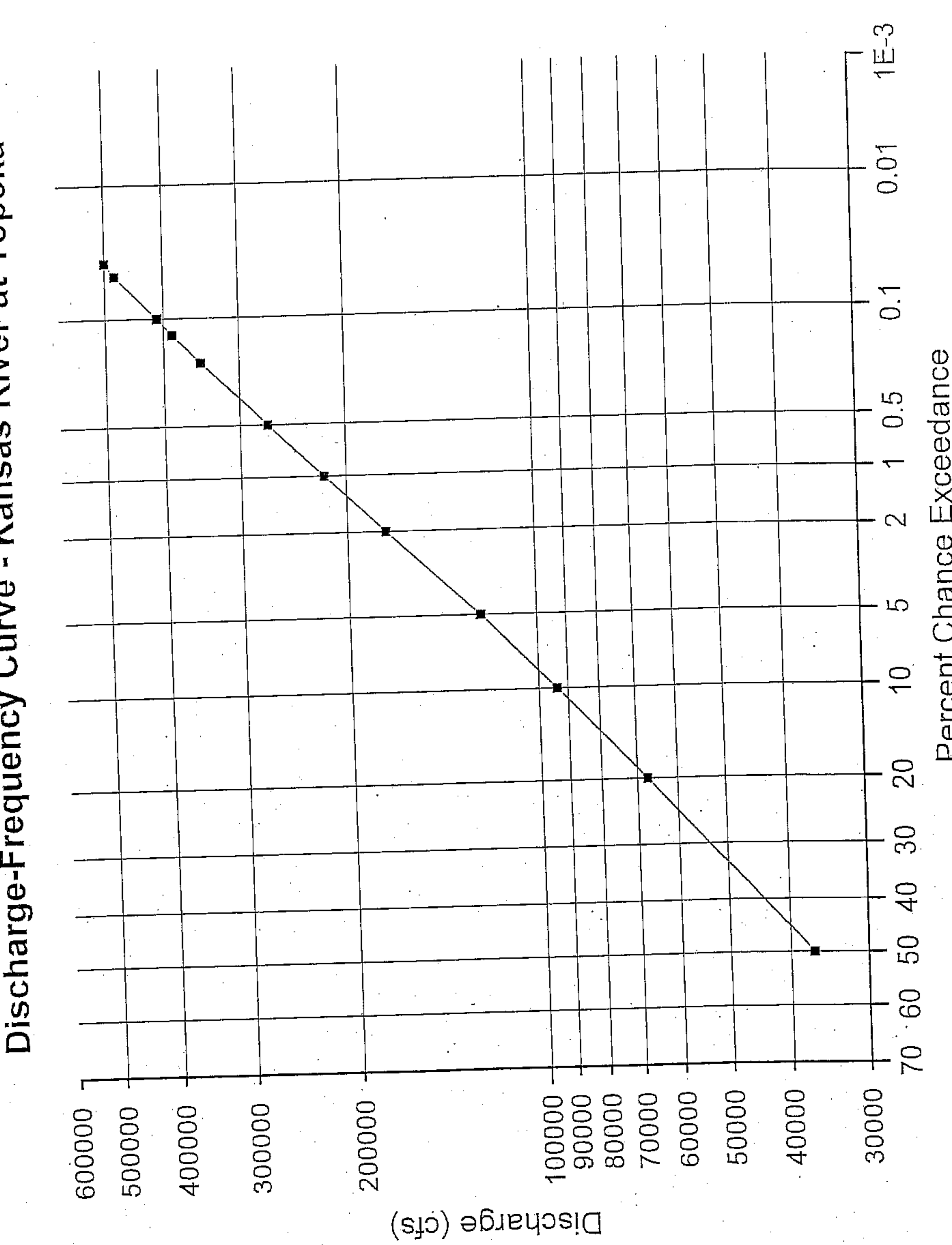
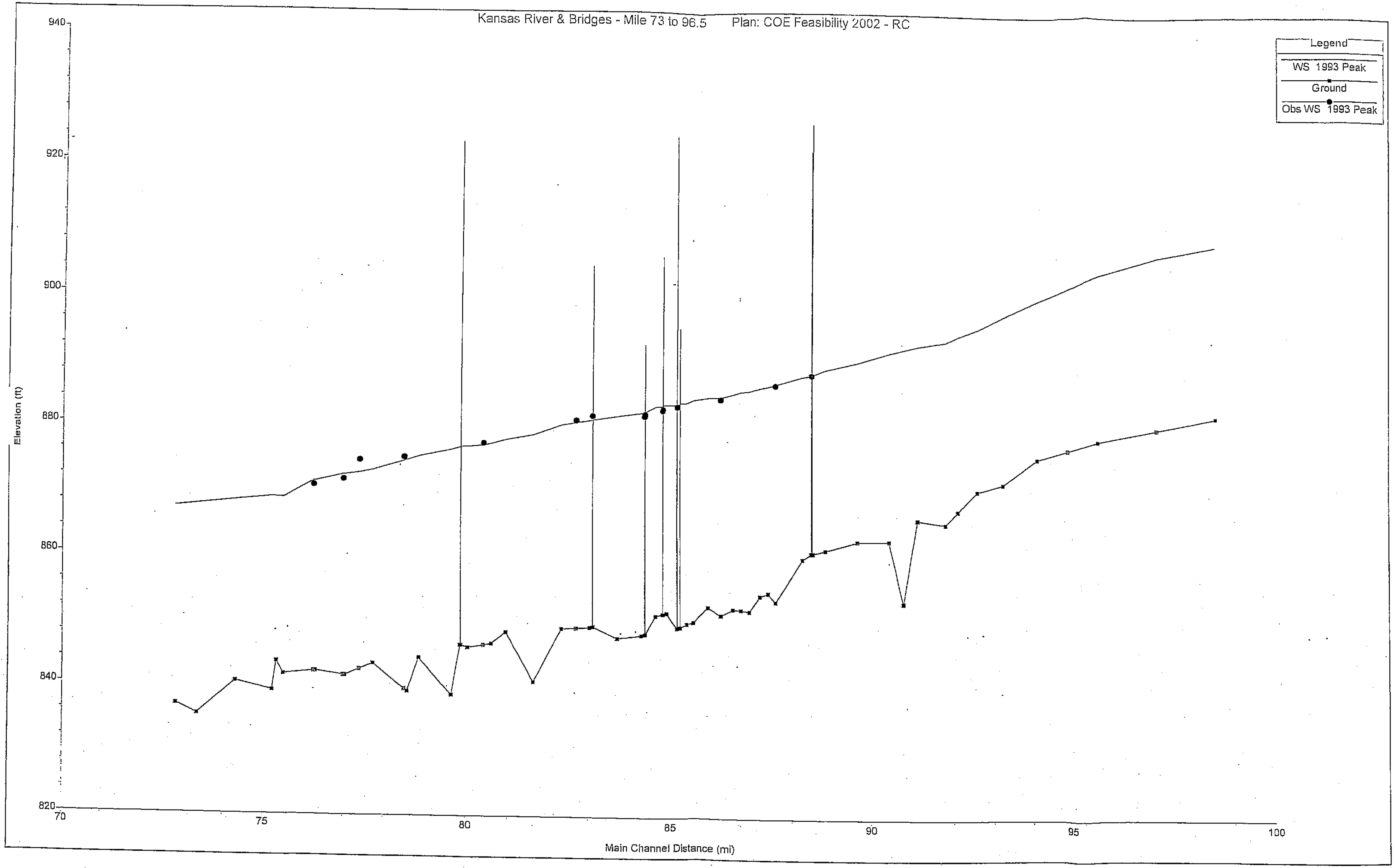


Discharge-Frequency Curve - Kansas River at Topeka
Discharge (cfs)
600000
500000
400000
300000
200000
100000
90000
80000
70000
60000
50000
40000
30000
Percent Chance Exceedance
70
60
50
40
30
20
10
5
2
1
0.5
0.1
0.01
1E-3

Plate A2-1-1


Kansas River & Bridges - Mile 73 to 96.5 Plan: COE Feasibility 2002 - RC
Legend
WS 1993 Peak
Ground
Obs WS 1993 Peak
Elevation (ft)
940
920
900
880
860
840
820
70
75
80
85
90
95
100
Main Channel Distance (mi)

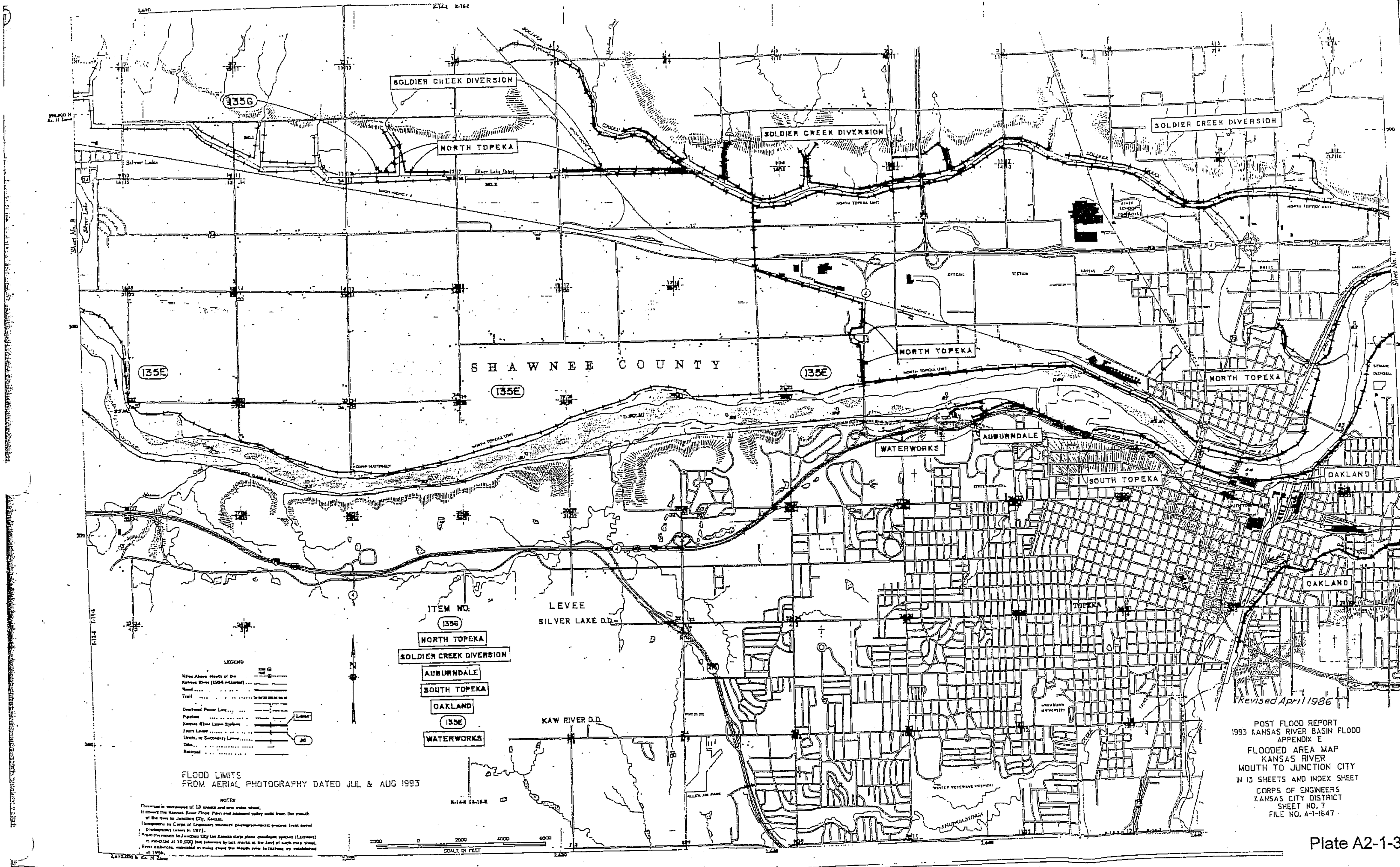
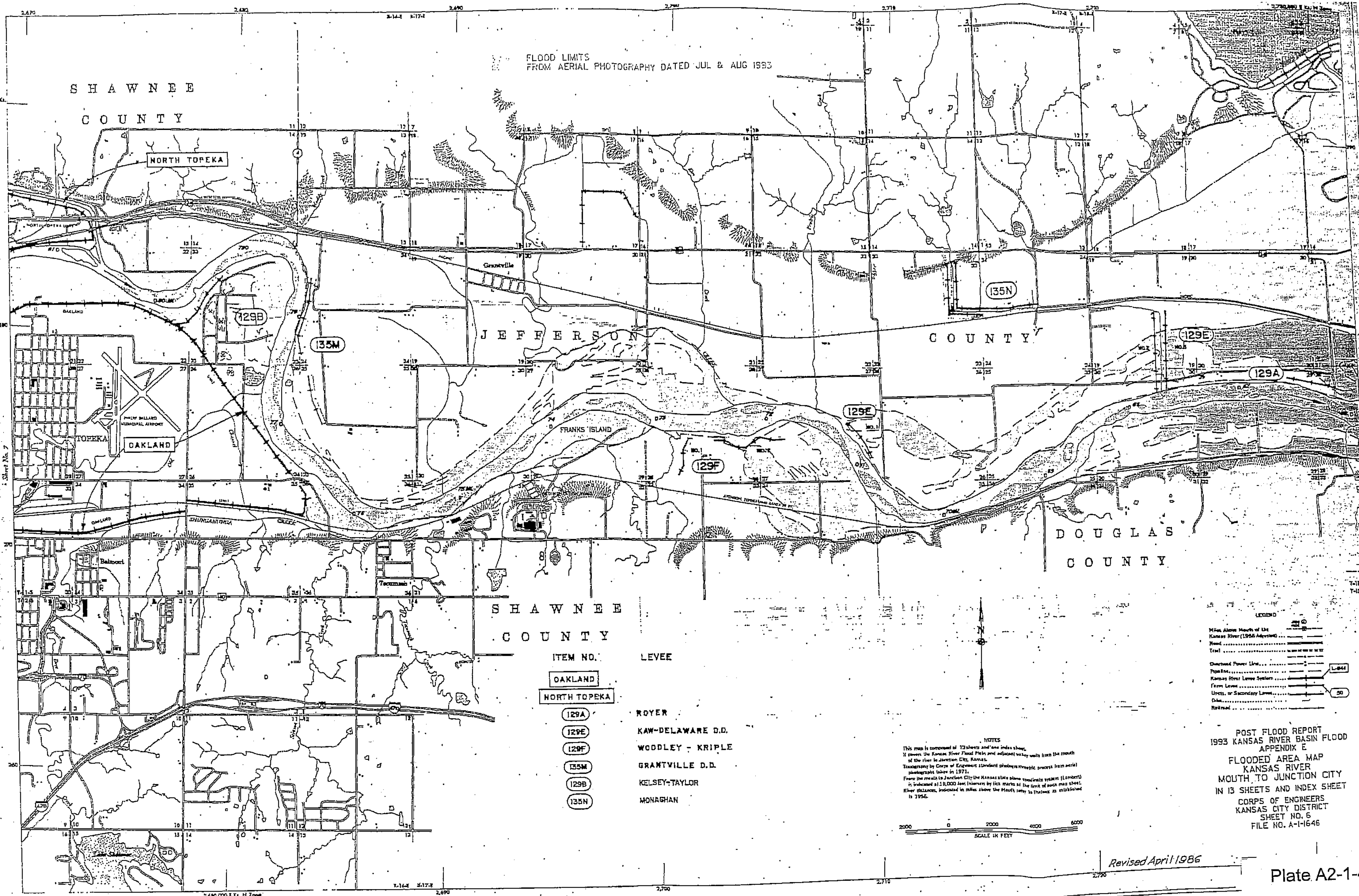
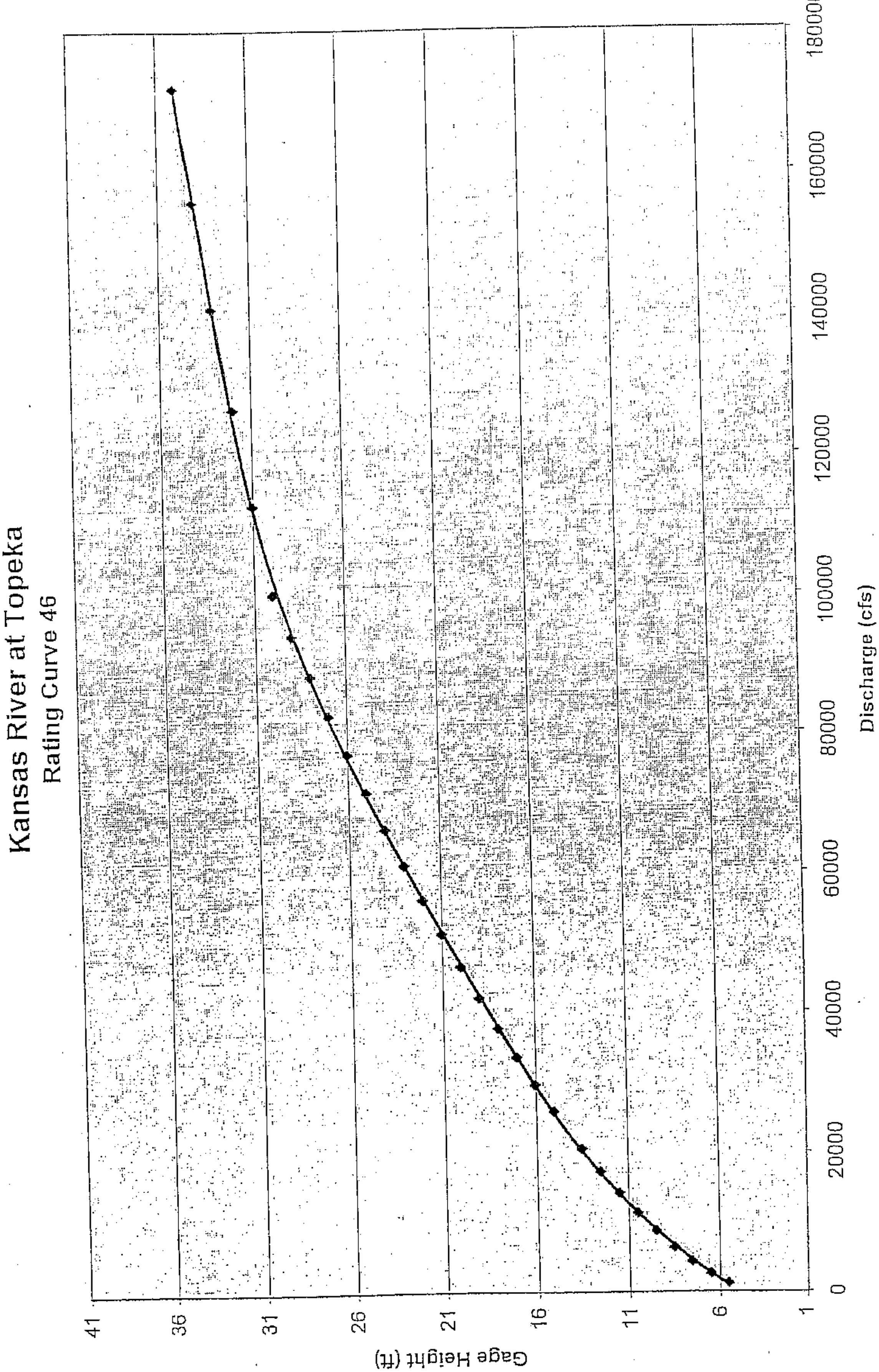

SOLDIER CREEK DIVERSION
135G
NORTH TOPEKA
SOLDIER CREEK DIVERSION
SOLDIER CREEK DIVERSION
NORTH TOPEKA
SHAWNEE COUNTY
135E
135E
135E
NORTH TOPEKA
WATERWORKS
AUBURNDALE
SOUTH TOPEKA
OAKLAND
OAKLAND
TOPEKA
ITEM NO.
135G
NORTH TOPEKA
SOLDIER CREEK DIVERSION
AUBURNDALE
SOUTH TOPEKA
OAKLAND
135E
WATERWORKS
LEVEE
SILVER LAKE D.D.
KAW RIVER D.D.
LEGEND
FLOOD LIMITS
FROM AERIAL PHOTOGRAPHY DATED JUL & AUG 1993
NOTES
SCALE IN FEET
Revised April 1986
POST FLOOD REPORT
1993 KANSAS RIVER BASIN FLOOD
APPENDIX E
FLOODED AREA MAP
KANSAS RIVER
MOUTH TO JUNCTION CITY
IN 13 SHEETS AND INDEX SHEET
CORPS OF ENGINEERS
KANSAS CITY DISTRICT
SHEET NO. 7
FILE NO. A-1-1647

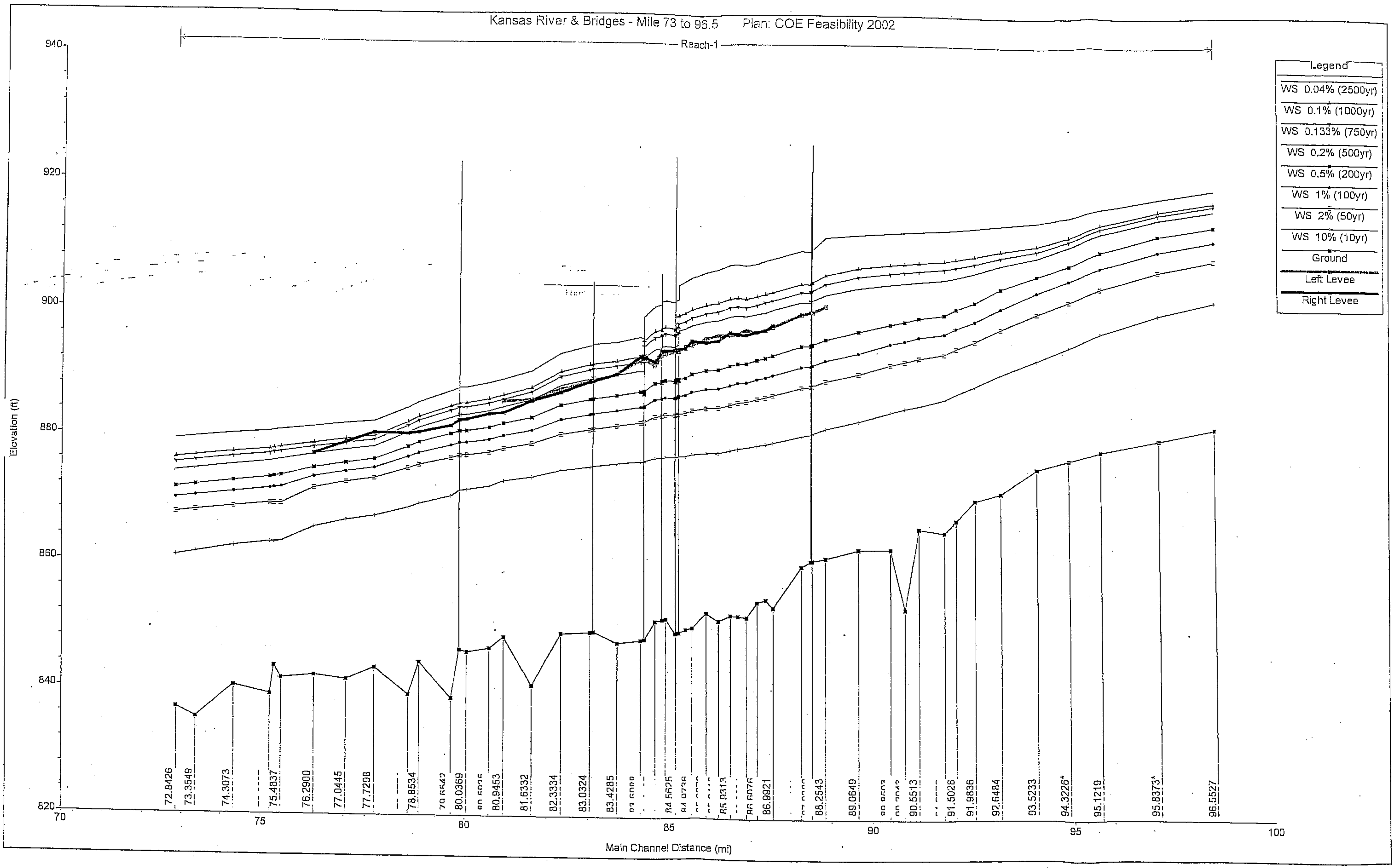
Plate A2-1-3



FLOOD LIMITS
FROM AERIAL PHOTOGRAPHY DATED JUL & AUG 1993
SHAWNEE
COUNTY
NORTH TOPEKA
OAKLAND
TOPEKA
Grantville
JEFFERSON
COUNTY
FRANKS ISLAND
SHUNGANUNGA CREEK
DOUGLAS
COUNTY
SHAWNEE
COUNTY
Tecumseh
ITEM NO.
LEVEE
OAKLAND
NORTH TOPEKA
129A
ROYER
129E
KAW-DELAWARE D.D.
129F
WOODLEY - KRIPLE
135M
GRANTVILLE D.D.
129B
KELSEY-TAYLOR
135N
MONAGHAN
129B
135M
135N
129E
129A
129F
LEGEND
NOTES
SCALE IN FEET
POST FLOOD REPORT
1993 KANSAS RIVER BASIN FLOOD
APPENDIX E
FLOODED AREA MAP
KANSAS RIVER
MOUTH TO JUNCTION CITY
IN 13 SHEETS AND INDEX SHEET
CORPS OF ENGINEERS
KANSAS CITY DISTRICT
SHEET NO. 6
FILE NO. A-1-1646
Revised April 1986

Plate A2-1-4


Kansas River at Topeka
Rating Curve 46
Gage Height (ft)
41
36
31
26
21
16
11
6
1
Discharge (cfs)
0
20000
40000
60000
80000
100000
120000
140000
160000
180000



Kansas River & Bridges - Mile 73 to 96.5 Plan: COE Feasibility 2002
Reach-1
Legend
WS 0.04% (2500yr)
WS 0.1% (1000yr)
WS 0.133% (750yr)
WS 0.2% (500yr)
WS 0.5% (200yr)
WS 1% (100yr)
WS 2% (50yr)
WS 10% (10yr)
Ground
Left Levee
Right Levee
Elevation (ft)
Main Channel Distance (mi)

Plate A2-1-6

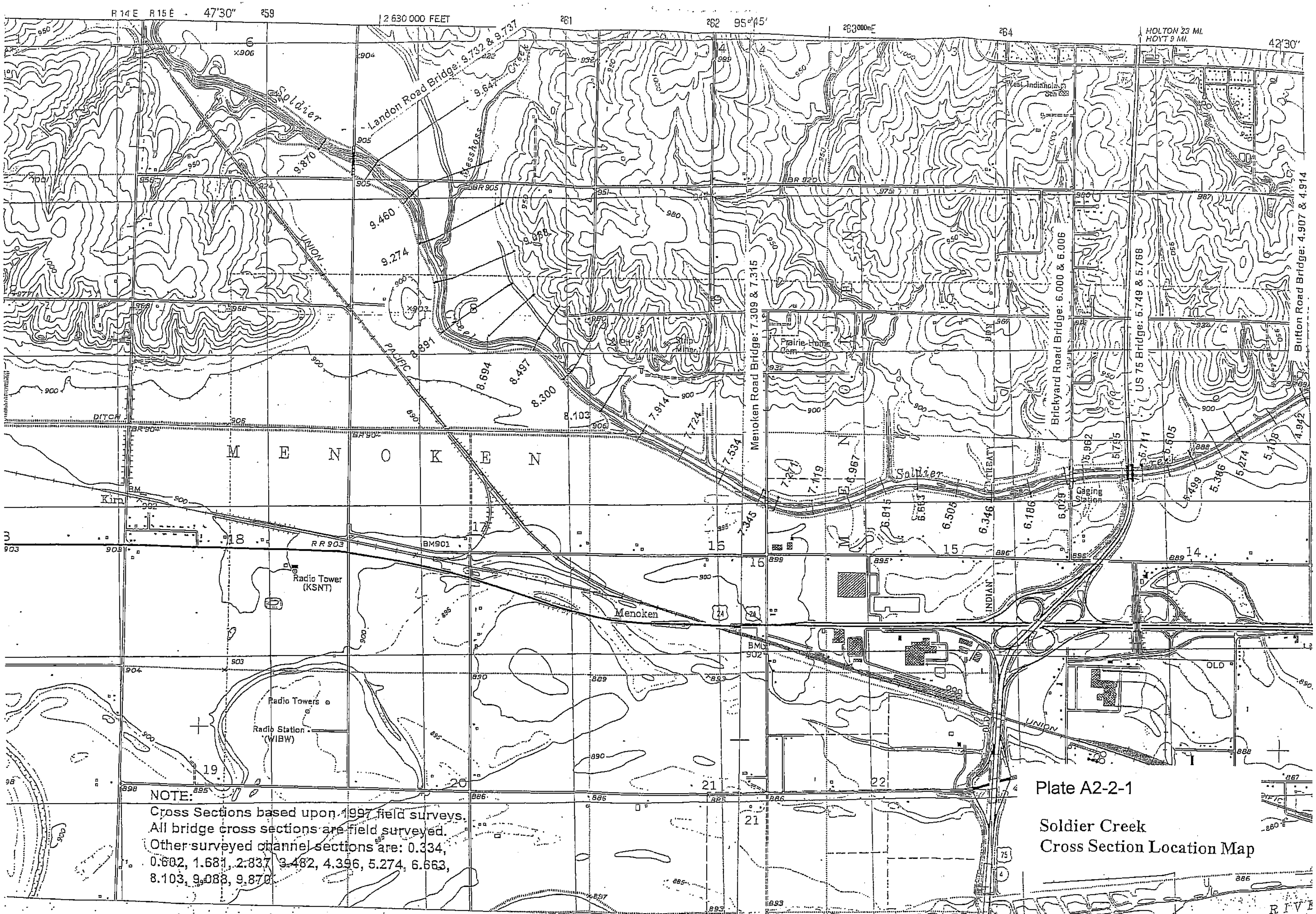


Plate A2-2-1

Soldier Creek
Cross Section Location Map

NOTE:
Cross Sections based upon 1997 field surveys.
All bridge cross sections are field surveyed.
Other surveyed channel sections are: 0.334, 0.602, 1.681, 2.837, 3.482, 4.396, 5.274, 6.663, 8.103, 9.088, 9.870

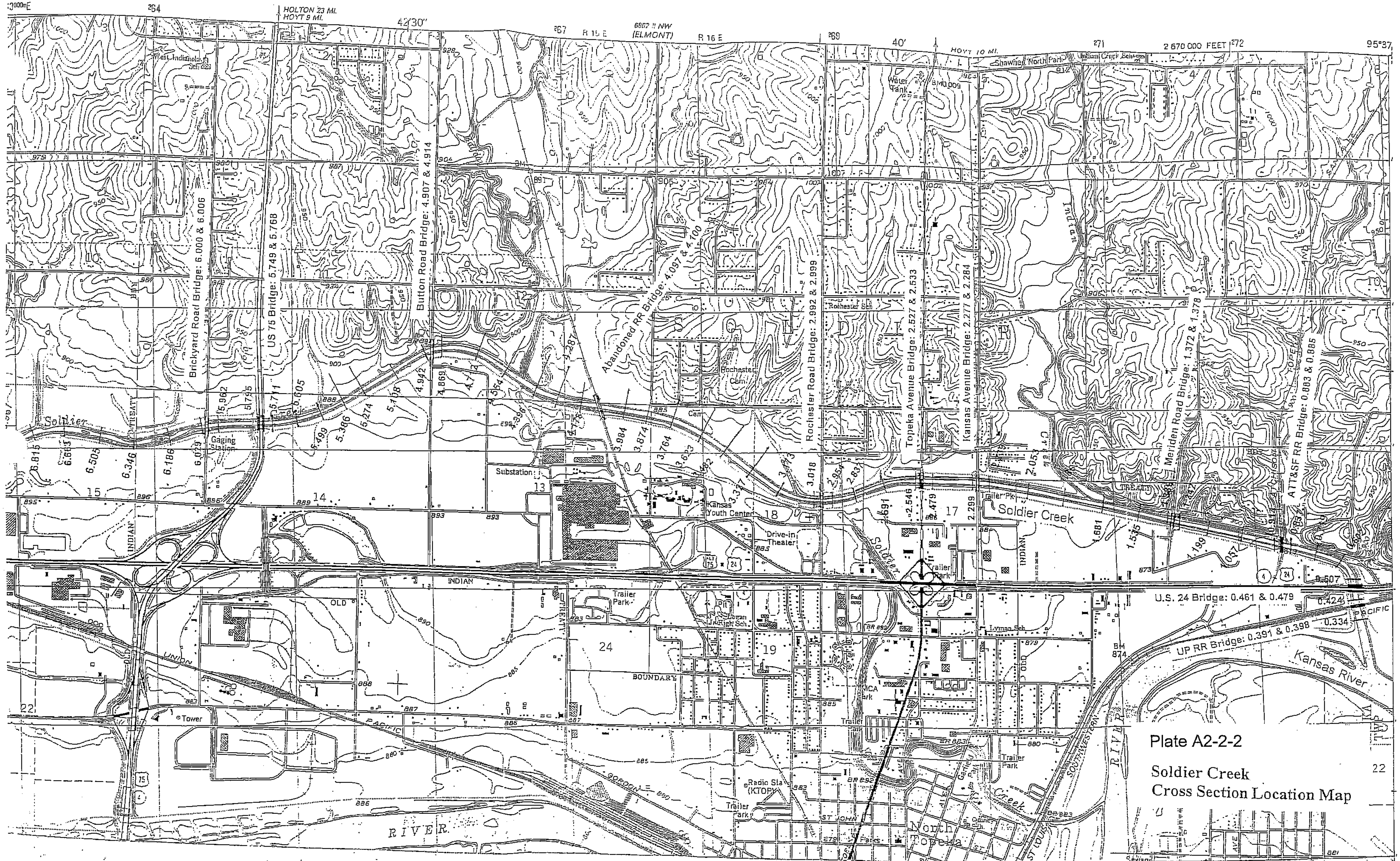
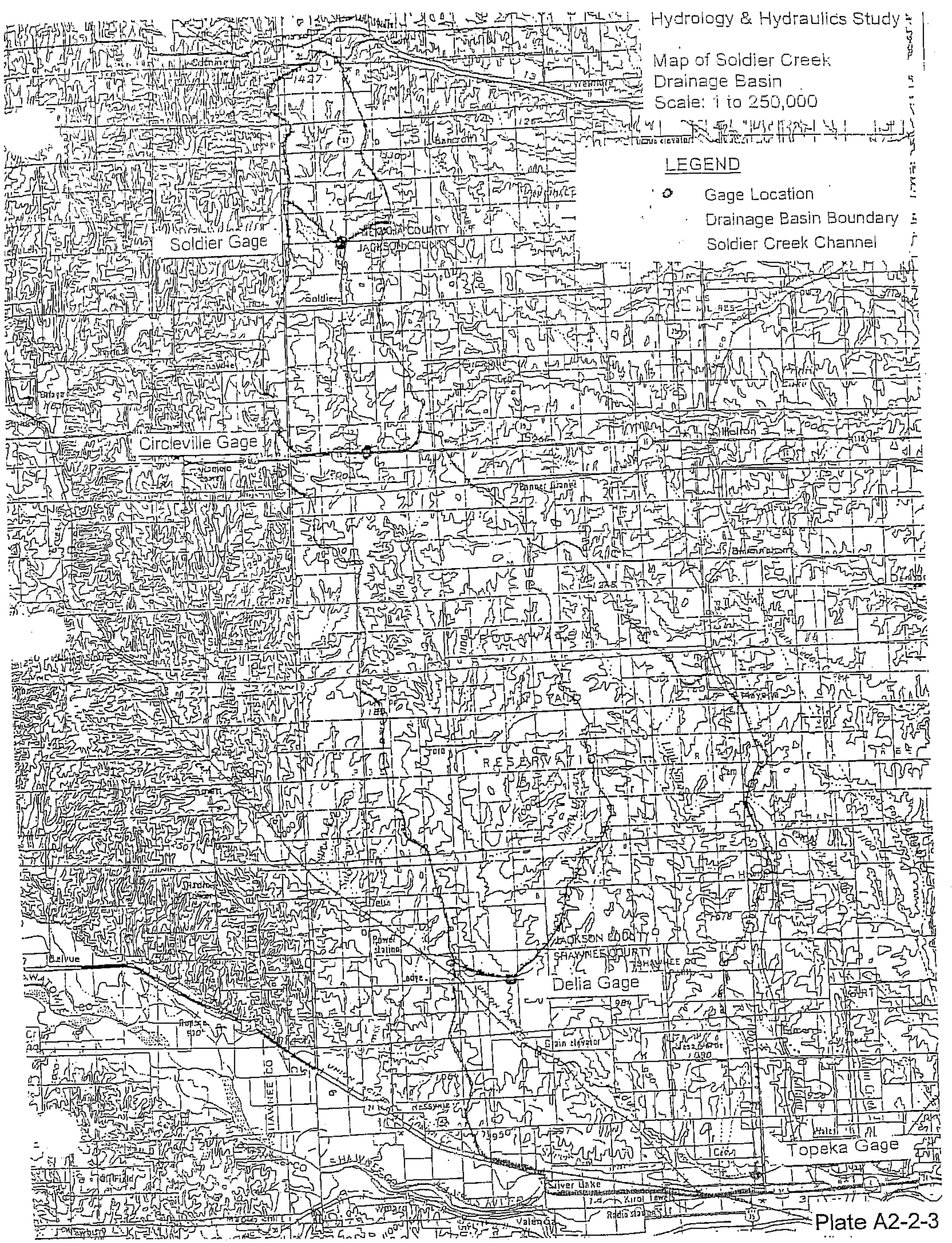


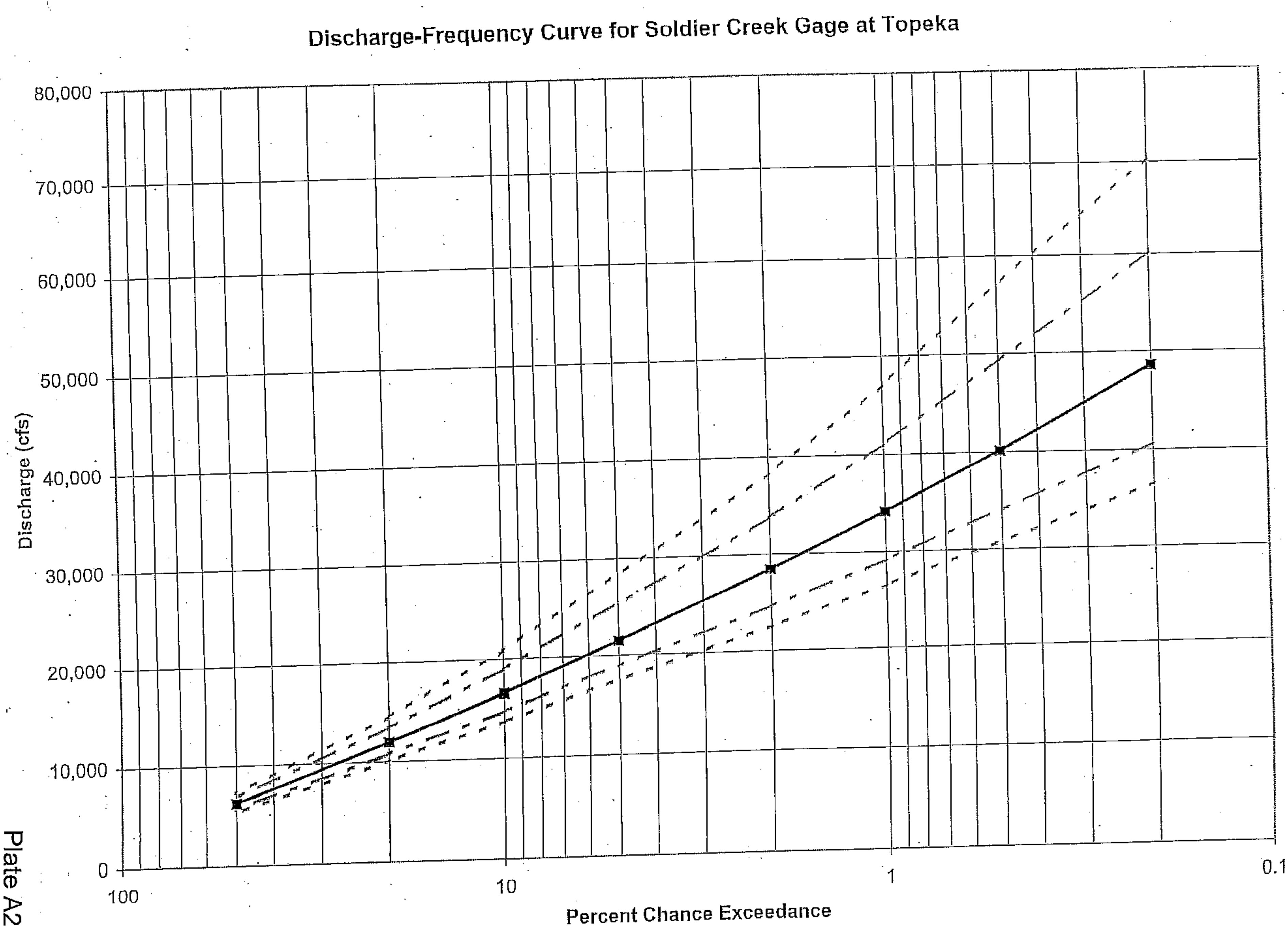
Plate A2-2-2

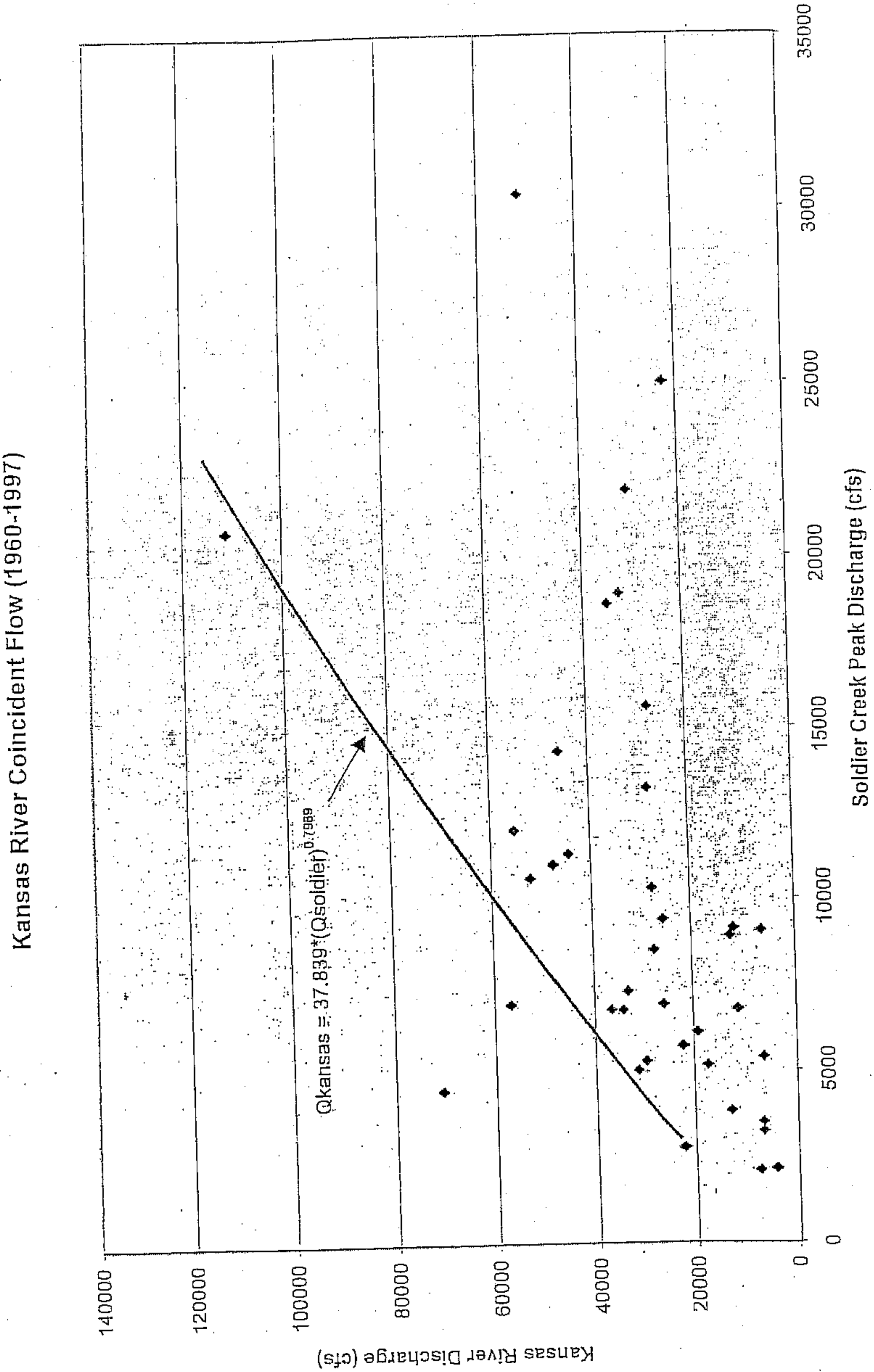
Soldier Creek
Cross Section Location Map



Hydrology & Hydraulics Study
Map of Soldier Creek
Drainage Basin
Scale: 1 to 250,000
LEGEND
Gage Location
Drainage Basin Boundary
Soldier Creek Channel
Soldier Gage
Circleville Gage
Delia Gage
Topeka Gage
NEMAHA COUNTY
JACKSON COUNTY
SHAWNEE COUNTY
Bancroft
Soldier
Delia
Silver Lake

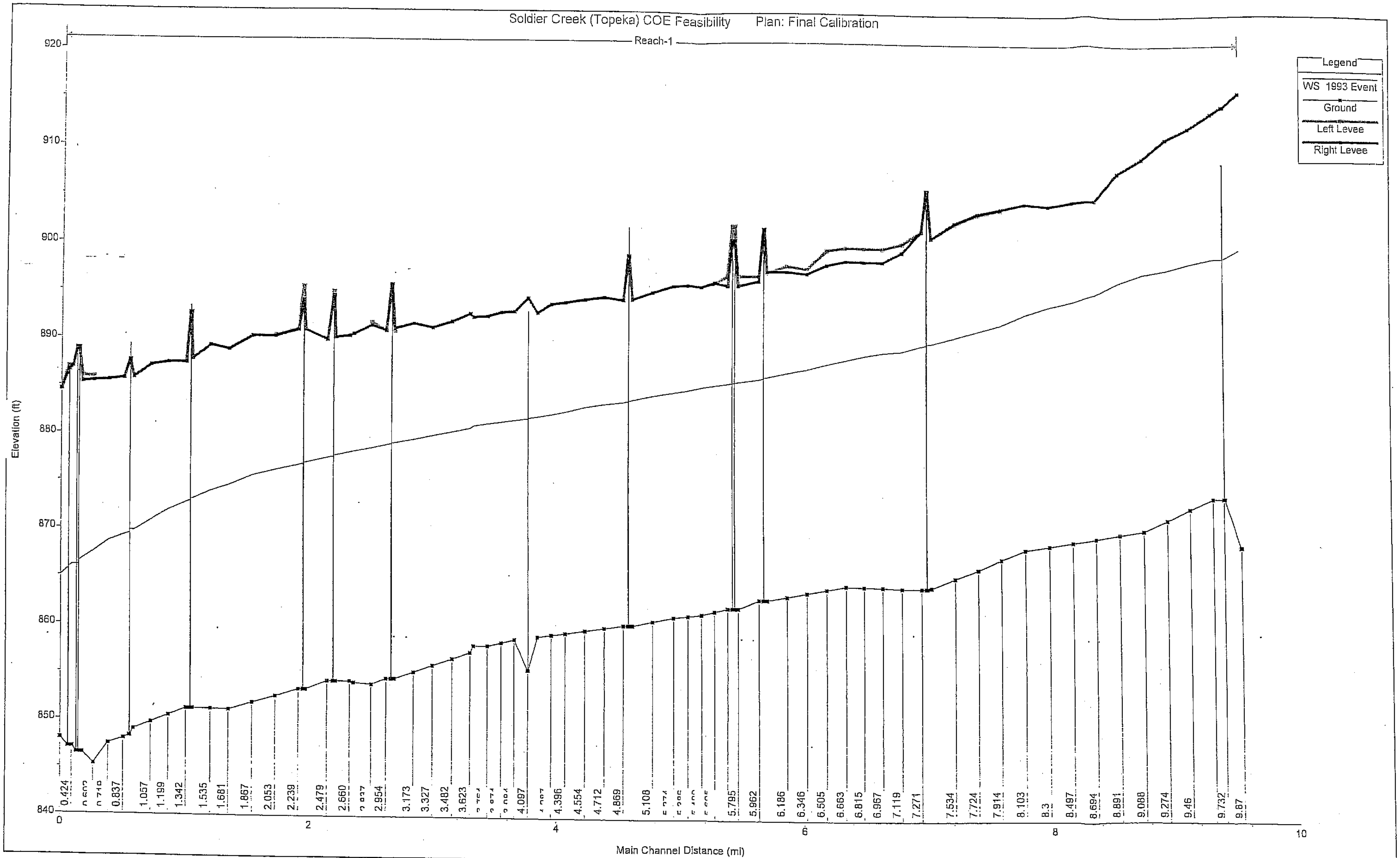
Plate A2-2-3

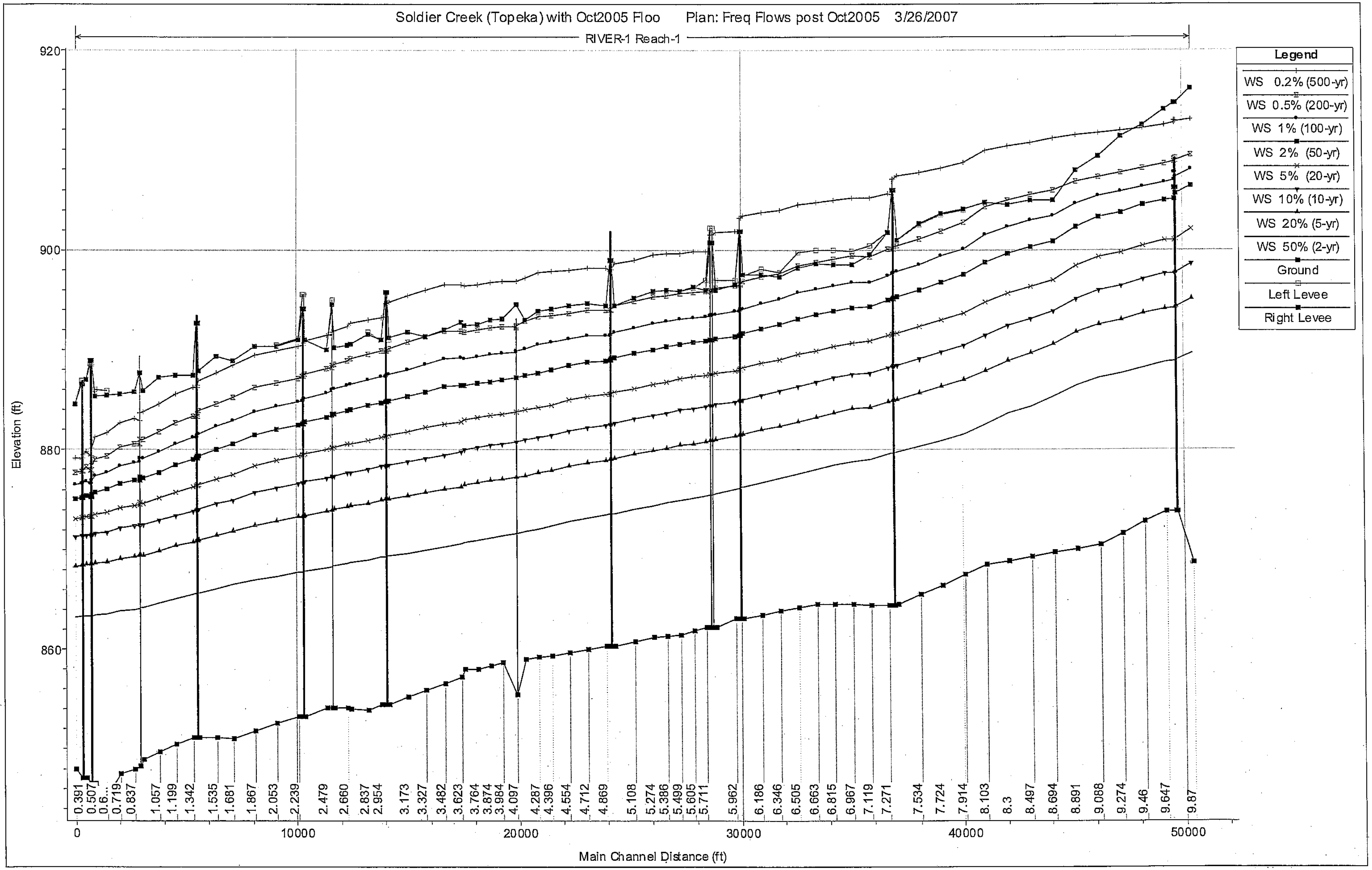

Discharge-Frequency Curve for Soldier Creek Gage at Topeka
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
Discharge (cfs)
100
10
1
0.1
Percent Chance Exceedance



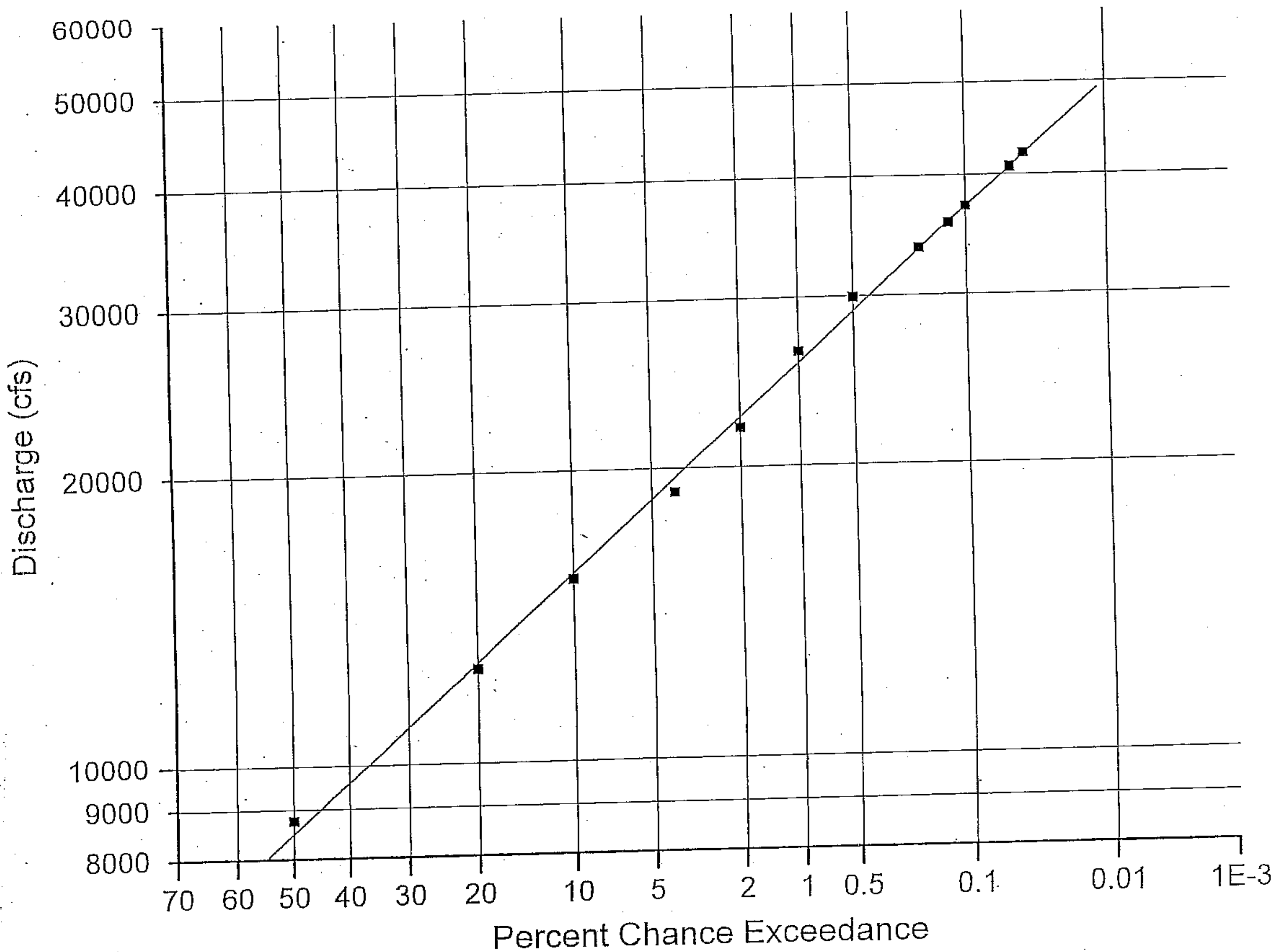
Kansas River Coincident Flow (1960-1997)
Qkansas = 37.839*(Qsoldier)^0.7989
Kansas River Discharge (cfs)
Soldier Creek Peak Discharge (cfs)
0
20000
40000
60000
80000
100000
120000
140000
0
5000
10000
15000
20000
25000
30000
35000

Plate A2-2-5


Soldier Creek (Topeka) COE Feasibility Plan: Final Calibration
Reach-1
Legend
WS 1993 Event
Ground
Left Levee
Right Levee
Elevation (ft)
920
910
900
890
880
870
860
850
840
0
2
4
6
8
10
Main Channel Distance (mi)
0.424
0.837
1.057
1.199
1.342
1.535
1.681
1.867
2.053
2.239
2.479
2.660
2.954
3.173
3.327
3.482
3.623
4.097
4.396
4.554
4.712
4.869
5.108
5.795
5.962
6.186
6.346
6.505
6.663
6.815
6.957
7.119
7.271
7.534
7.724
7.914
8.103
8.3
8.497
8.694
8.891
9.088
9.274
9.46
9.732
9.87

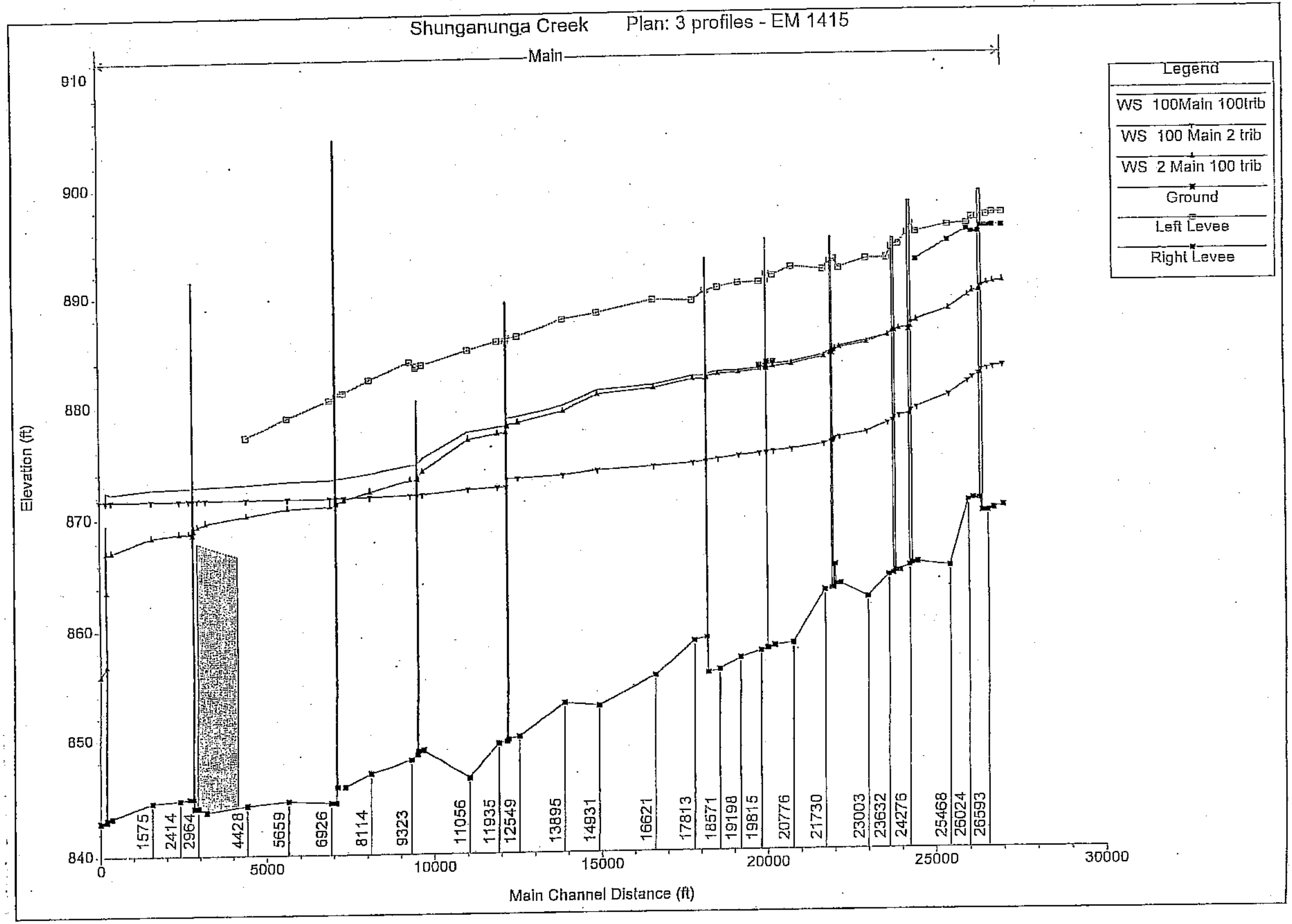

Soldier Creek (Topeka) with Oct2005 Floo Plan: Freq Flows post Oct2005 3/26/2007
RIVER-1 Reach-1
Legend
WS 0.2% (500-yr)
WS 0.5% (200-yr)
WS 1% (100-yr)
WS 2% (50-yr)
WS 5% (20-yr)
WS 10% (10-yr)
WS 20% (5-yr)
WS 50% (2-yr)
Ground
Left Levee
Right Levee
Elevation (ft)
Main Channel Distance (ft)

Discharge-Frequency Curve - Shunganunga at Station 3210




N
20776
21939
22195
23003
23998
25468
26165
26339
27054
14931
13895
8114
5659
4428

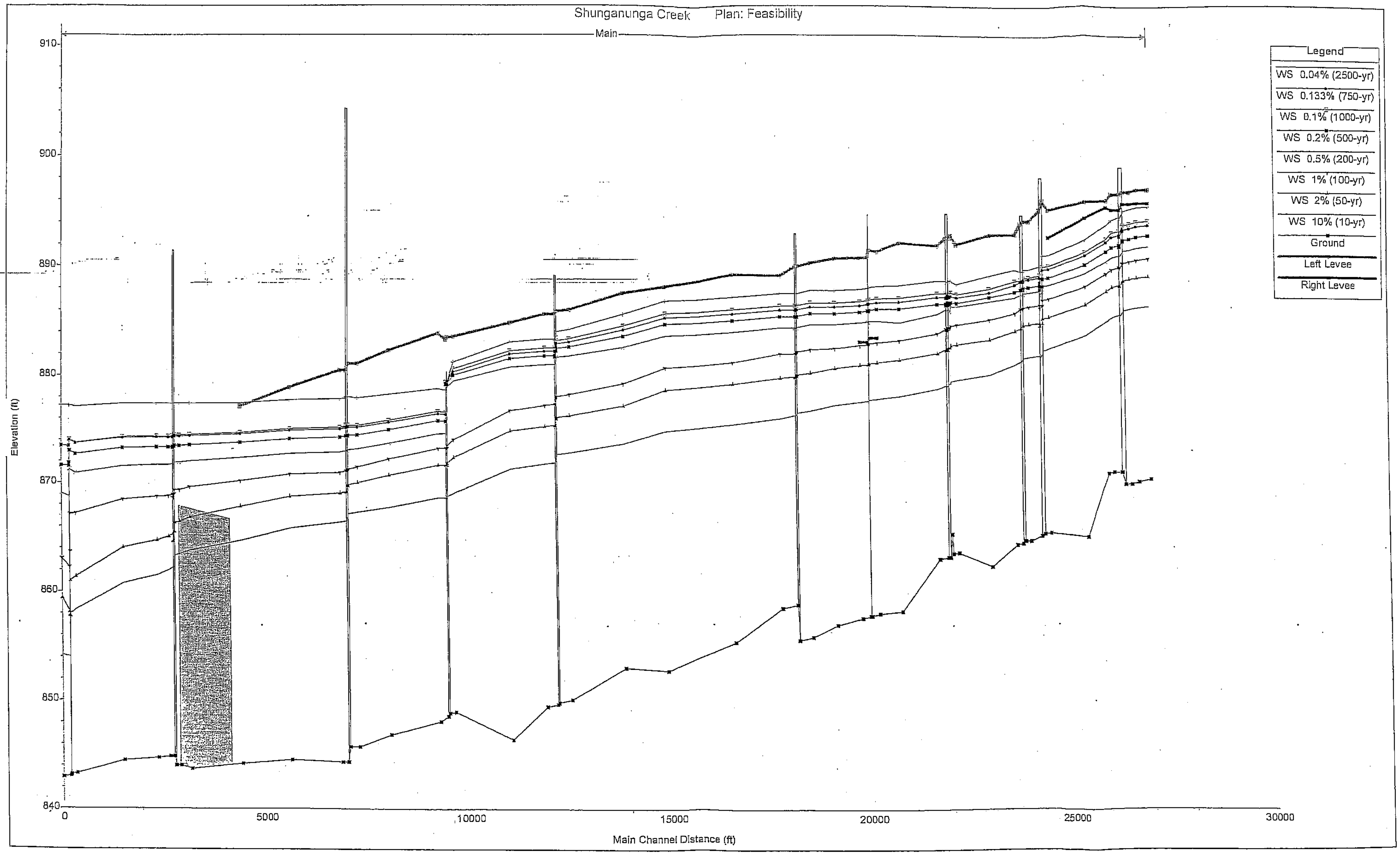
Plate A2-3-2



Shunganunga Creek Plan: 3 profiles - EM 1415
Main
Legend
WS 100Main 100trib
WS 100 Main 2 trib
WS 2 Main 100 trib
Ground
Left Levee
Right Levee
Elevation (ft)
910
900
890
880
870
860
850
840
Main Channel Distance (ft)
0
5000
10000
15000
20000
25000
30000
1575
2414
2964
4428
5659
6926
8114
9323
11056
11935
12549
13895
14931
16621
17813
18571
19198
19815
20776
21730
23003
23632
24276
25468
26024
26593

Plate A2-3-3

U.S. Army Corps of Engineers Frequencies for Coincidental Occurrence						
Area Ratio	10 Year Design		50 Year Design		100 Year Design	
	Main Stream	Tributary	Main Stream	Tributary	Main Stream	Tributary
10000 to 1	1	10	1	50	2	100
	10	1	50	1	100	2
1000 to 1	2	10	5	50	10	100
	10	2	50	5	100	10
100 to 1	5	10	10	50	25	100
	10	5	50	10	100	25
10 to 1	10	10	25	50	50	100
	10	10	50	25	100	50
1 to 1	10	10	50	50	100	100
	10	10	50	50	100	100



Shunganunga Creek Plan: Feasibility
Main
Legend
WS 0.04% (2500-yr)
WS 0.133% (750-yr)
WS 0.1% (1000-yr)
WS 0.2% (500-yr)
WS 0.5% (200-yr)
WS 1% (100-yr)
WS 2% (50-yr)
WS 10% (10-yr)
Ground
Left Levee
Right Levee
Elevation (ft)
910
900
890
880
870
860
850
840
0
5000
10000
15000
20000
25000
30000
Main Channel Distance (ft)

Plate A2-3-5

Chapter A-3

GEOTECHNICAL DESIGN ANALYSIS

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Topeka, Kansas
Flood Risk Management Study
Appendix A – Engineering
Chapter 3 – Geotechnical Analysis

TABLE OF CONTENTS

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A-3 GEOTECHNICAL ANALYSIS

A-3.1 Existing Conditions

A-3.1.1 Introduction

This section presents the results of the geotechnical evaluation of the existing conditions performed as part of the feasibility flood study of the Topeka Flood Protection Project at Topeka, Kansas. The flood risk management project within the study area was designed by the Kansas City District U. S. Army Corps of Engineers, and was constructed under its supervision. The unit is operated and maintained by two local sponsors as follows: a) the North Topeka Drainage District operating and maintaining the Soldier Creek and North Topeka units, and b) The City of Topeka maintaining and operating the Waterworks Unit, Auburndale Unit, South Topeka Unit and Oakland Unit.

The primary goal of this phase of the geotechnical evaluation is to gather and review all available data and develop an assessment of the existing conditions of each levee units by identifying the critical reaches for each unit and their probability of failure for different river stages. Additionally, the past performance of the levee system is evaluated. This information is to assist in an assessment of the future performance of the levee during flood events. In particular, the following tasks were performed for this study:

- Review of existing sources of information.
- Description of each existing levee unit including design features and subsurface conditions.
- Reliability analyses of each unit and identification of critical reaches of each unit.

The evaluation of the existing condition was based on the subsurface investigation performed for the design of the project supplemented with the additional investigation performed for this feasibility study, such as cone penetrometer tests and laboratory testing performed on selected samples collected from borings drilled in some areas considered critical.

A-3.1.2 Sources of Information

The primary sources of information include the references listed in Section 12 (References) of this Appendix.

A-3.1.3 Description of the Levee Units

The Topeka Flood Protection Project consists of six (6) flood risk management units along the Kansas River and its tributaries, protecting the city of Topeka, Kansas. The project includes approximately 40 miles of levees along the Kansas River and approximately 3 miles of tie back levees, 0.7 miles of floodwall, 9.2 miles of improved channel on Soldier Creek, 5.5 miles of improved channel on Shunganunga Creek, and 2.6 miles of improved and enlarged channel along the Kansas River. The project also includes pumping plants, gated outlets for drainage structures, sandbag gaps and ponding areas. Flood risk management units forming portions of the Topeka Flood Protection Project are described in the following paragraphs

Soldier Creek Unit

The Soldier Creek Unit is located along Soldier Creek, beginning at Kansas River mile 81.9 and extending northwesterly to the vicinity of the Silver Lake channels and levees. The purpose of this unit is to provide flood risk management for north Topeka against a peak discharge of approximately 50,000 cfs. The Soldier Creek unit includes 17.9 miles of levee, 9.2 miles of channel improvement, approximately 4.3 miles of tributary tie back levees along the left bank of Soldier Creek, and 35 drainage structures. The project was designed in 1958 and constructed between the years 1958 and 1962.

North Topeka Unit

The North Topeka Unit is located along the left bank of the Kansas River beginning on Soldier Creek and extending upstream along the left bank of the Kansas Rive r to approximate river mile 82. The flood risk management unit includes 9.3 miles of earthen levee, 3 relief wells, 3 pumping plants, 15 drainage structures, one sandbag gap, and one stoplog gap. The North Topeka Unit was designed in 1961 and constructed between 1964 and 1967 for the purpose of protecting the North Topeka area.

Waterworks Unit

The Waterworks Unit is located along the right bank of the Kansas River to provide flood risk management for the western side of Topeka. The levee unit includes 1,998 feet of earthen levee and 1,662 feet of floodwall with 9 relief wells for underseepage control, 4 drainage structures for the interior drainage control, and 1 sandbag and 4 stoplog gaps. The project was designed in 1957 and constructed during 1959.

Auburndale Unit

The Auburndale Unit is located east of the Waterworks unit along the right bank of the Kansas River. The unit uses the Interstate I-70 embankment in lieu of a right bank levee between the Waterworks Unit at the upper end and the South Topeka Unit at the lower end. This unit also includes the Waite Street Levee and an 850-foot tie back levee, which serves as the upstream boundary for a ponding area. The entire length of the earthen levee section is 1.3 miles and includes 15 relief wells for underseepage control, 2 pumping plants and 4 drainage structures for interior drainage control and discharge of the relief well system, and one sandbag gap. The unit was designed in 1958 and constructed between the years 1961 and 1962.

South Topeka Unit

The South Topeka Unit is located along the right bank of the Kansas River between the Auburndale Unit at the west upper end (river mile 85.5) and Santa Fe Railroad bridge at mile

83.8 at the lower end. The unit consists of 1.4 miles of earthen levee, 1,944 feet of floodwall and includes 2 stoplog gaps. Underseepage is controlled by 27 relief wells with the water collected from the relief well system and interior drainage discharged into the Kansas River by 5 pumping plants and 15 drainage structures. The unit was designed in 1966 and constructed between the years of 1970 and 1973.

Oakland Unit

The Oakland Unit is located along the Kansas River downstream of South Topeka Unit and continuing along left bank of Shunganunga Creek. The unit consists of 10 miles of earthen levee, one sandbag gap, and 5.5 miles of channel improvement. Underseepage is controlled by underseepage berms and 22 relief wells. The collected interior drainage and relief well water is discharged into the Kansas River by 2 pumping plants and 48 drainage structures. The Oakland Unit was designed in 1960 and constructed during the period between 1965 and 1969.

A-3.1.4 Subsurface Conditions

Assessments of the subsurface conditions along the project are derived from the Design Memoranda (DMs) referenced later in this Appendix and from additional subsurface investigation performed for this feasibility study.

The Topeka area is located within the Eudora-Muir soils association. A review of available geological information indicates that part of the study area is situated in an area of alluvial deposition and erosion at the confluences of Soldier Creek with the Kansas River and Shunganunga Creek with the Kansas River. The efforts to control the flooding are done with a series of upstream flood control dams and levees. Subsurface investigations performed during the design of the subject flood risk management project and the additional subsurface investigation performed for this feasibility study indicate that the composition and thickness of the natural blanket in the Topeka area generally conforms to that found elsewhere in Kansas River Valley. The natural surface impervious blanket consists of sandy silts from 10 to 20 feet thick overlaying a deposit of sands and gravels 40 to 80 feet thick, which become coarser with depth. In a few reaches along the river the impervious blanket is absent requiring a constructed underseepage protection system. A fairly consistent weak layer of organic material has been found along Soldier Creek, near the base of the excavated channel. The consistency and thickness of the impervious blanket shown on the record drawings have been used for the evaluation of the existing underseepage condition for each levee unit.

Local bedrock in the project area is comprised of the Upper Pennsylvanian limestone and shale formation which may be found at approximate depths of 60 to 80 feet below existing natural ground surface.

A-3.1.5 Levee Design Features

Basic Levee Section

The basic levee section was constructed with a 10' crown width, with generally 1V on 3H riverside and landside slopes. Underseepage and stability berms were added when necessary in certain reaches. The following table presents the average and maximum height for each levee unit.

Table 1 - Levee Embankment Characteristics

	Soldier Creek	North Topeka	Waterworks	Auburndale	South Topeka	Oakland
Average Height (ft)	15	16	12	15	13	12
Maximum Height (ft)	17	20	19	26	16	25

The levee embankment consists of compacted earthen material placed in random and impervious zones. Riprap protection is provided on the riverside slopes where needed and around the inlets and outlets of drainage structures. All other sloped surfaces are protected by established grasses. The levee crown, turnouts, and ramps are surfaced with 6 inches of aggregate surfacing.

Seepage Control Measures

Seepage control measures consist of underseepage berms, relief wells and area fill where necessary. Typical locations of existing underseepage controls are located where the natural blanket is thin in a localized area.

Stability Berms

Levee sections were designed to provide a minimum factor of safety of 1.25 for riverward submerged toe case, and 1.5 for the steady seepage case. Typically stability berms were used for levee sections over 10 feet. For the existing soil conditions, this appears to be the limiting height, or spring point.

A-3.1.6 Assessment of Levee Integrity

The current levee system is in good condition with no presently identifiable problem areas. The entire levee system has performed well during past flood events. The seepage and stability berms have performed as designed over the years. A partial top of levee survey was provided to the Corps of Engineers by the City of Topeka. Additional cross sections were surveyed as part of this feasibility study.

A-3.1.7 Uncertainty Analysis

Geotechnical failure in this study is defined as failure of the embankment slope resulting from the river flowing to landside areas of the levee with resulting economic damages or due to a sudden drawdown of the water elevation from the maximum level, considered at the levee crest, to the normal operating level. Further, geotechnical failure may occur when river stages reach an elevation at or below the top of the levee. Within this range, geotechnical failure modes are excessive seepage leading to a piping condition and slope instability.

Uncertainty analyses were performed to define the existing condition of the Topeka Flood Protection system. The probability of failure was evaluated by assessing the foundation and embankment materials and assigning values for the probability moments of the random variables considered in the analyses. The First-Order-Second-Moment (FOSM) method, as recommended in ETL 1110-2-556, "Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies" dated 28 May 1999, was followed during the evaluation of the existing conditions of each levee unit. In this approach, the uncertainty in performance is taken to be a function of the uncertainty in model parameters. The standard deviations of a performance function were estimated based on the expected values (means) and the standard deviation of the random variable means. The performance functions considered were slope stability and underseepage piping stability. The final result of the FOSM is a reliability index, Beta (β), representing the amount of standard deviation of the performance function by which the expected value exceeds the limit state. The limit state for the slope stability and underseepage piping stability was defined using a factor of safety of 1.0. The standard deviation and variance of the performance function are calculated from the standard deviation and variance of the foundation and embankment parameters using the Taylor's series method based on a Taylor's series expansion of the performance function about the expected values. The partial derivatives were calculated numerically using an increment of plus and minus one standard deviation centered on the expected value. The variance of the performance function was obtained by summing the products of the partial derivatives of the performance function considering the variance of the corresponding parameters. For the existing condition of the levee, the probability of slope or underseepage piping failure (Pr_f) was expressed as a function of the river water elevation and other factors including soil strengths, permeabilities, and subsurface stratification. Reliability (R) is defined as:

$$R = (1-Pr_f)$$

A set of conditional-probability-of-failure versus floodwater-elevation graphs were developed as related to underseepage piping stability and slope stability for the long-term seepage or sudden drawdown condition.

The probability of geotechnical failure of a levee is conditional on the uncertainties associated with hydrologic and hydraulic aspects of determining the water surface profile during a flood. These uncertainties can be combined with the geotechnical uncertainties and in the @RISK model. This is accomplished, for economic purposes, through estimation of two index elevations for each levee reach within the study area. These index elevations are defined as follows:

The Probable Non-Failure Point (PNP) is the water elevation below which it is highly likely that the levee would not fail.
The Probable Failure Point (PFP) is the water elevation above which it is highly likely that the levee would fail.

The terms "highly likely that the levee would fail" is defined by the ETL as having 85% probability of occurrence. Therefore, the probability of failure at the PNP is 15% and the probability of failure at the PFP is 85%. A linear distribution is assumed in the economic model between the PNP and PFP.

A-3.1.8 Underseepage Reliability

Underseepage analyses were performed for every levee unit. Subsurface conditions were developed based on past investigations conducted for the design of each levee unit and on additional Cone Penetration Tests (CPT) performed at selected locations for this feasibility study. The impervious blanket thickness, soil type (for determination of the permeability ratio), and aquifer thickness were determined for each characteristic reach of every levee system. The standard deviation and the coefficient of variation of the blanket thickness for each reach and for the entire levee unit are provided as enclosure 1: Underseepage Analysis of this appendix. Underseepage analysis was performed using the Kansas City District method as approved by Corps of Engineers Missouri River Division Conference, 27 November 1962. A 50% relief well efficiency is assumed to determine the amount of artesian pressure to be used between relief wells. Critical area was determined based on the blanket thickness and material and levee height. The standard deviation for the blanket thickness and levee height was calculated for typical reaches on each levee unit and was used in underseepage reliability evaluation. Critical reach was determined for each levee unit by calculating the underseepage factor of safety for the existing conditions at the toe of the levee. The underseepage factor of safety is defined as the ratio between the actual gradient at the levee toe obtained by analysis and the computed critical gradient ($FS = i_0/i_{cr}$). If the factor of safety was deemed unsatisfactory, i.e. had a factor of safety of less than 1.0, an uncertainty analysis was performed for that particular reach. In the uncertainty analysis, the maximum exit gradient at the landside toe of the levee was considered as the performance function and the value of the critical gradient, assumed to be 0.84, considered the limit state. The foundation sand gradient obtained during the underseepage analyses was used in the stability analyses to assist in defining the steady state condition of the landside slope or the rapid drawdown condition of the riverside slope if the critical surface passed through the aquifer layer.

Reliability analysis was performed using Taylor's Series Method. In the Taylor method, random variables are quantified by their expected values, standard deviations, and correlation coefficients. These variables were used in the generalized equation for underseepage analysis as follows:

$$i_0 = \frac{H\left(\frac{K_f}{K_{bL}} D_f D_{bL}\right)^{1/2}}{D_{bo}\left[C_R \frac{e^{2L_R}-1}{e^{2L_R}+1} + 6H + 10 + \left(\frac{K_f}{K_{bL}} D_f D_{bL}\right)^{1/2}\right]}$$

$$P(F) = P(c_{ritical} < i_o)$$

Thus, an equation is used to calculate seepage gradient for a range of water levels on the riverside of the levee. From previous studies, the Taylor Series method appears to be more conservative and appropriate for a reconnaissance level investigation.

Permeability ratios of the blanket landside (K_L) and riverside (K_R) values were obtained by studying the classification information listed on the available boring logs and CPT. The Kansas City District Corps of Engineers correlations between soil classifications and K_L values for soils in this region were used to determine the K_L values for this study

Details of the underseepage analyses for each unit are shown on Figures 1 through 5 at the end of this section. A summary of underseepage evaluation for each levee unit is provided below.

Soldier Creek

The unit consists of the improved Soldier Creek channel and levees on both banks to contain the designed flood event, and tie back levees on the left bank of the creek. Foundation soils consist of a natural blanket with an average thickness of 23 feet overlaying a deposit of poorly graded sand averaging 20 feet in thickness. The composition of the natural blanket varies from clays (CL, CH) to silty sands, but primarily of lean clays. A weak layer of fat clay was mapped between stations 180+00 and 213+00 as substantiated by slides along the original channel. An extensive cinder fill overlaying the impervious blanket between stations 222+00 and 245+00 required the construction of a riverside seepage cut-off trench. Landside underseepage berms exist between station 397+50 and the levee end, relief wells for an existing Goodyear Plant between stations 205+00 and 206+00, and the existence of the thick impervious blanket indicates that underseepage instability was expected for this unit during initial design.

North Topeka Unit

This unit, constructed along the left bank of the Kansas River, includes 9.3 miles of earthen levee with heights varying between 2 feet and 21 feet. The natural blanket for the entire levee unit, consisting predominantly silt, varies in thickness from 1 to 23 feet, with an average thickness of 12 feet. The coefficient of variation in the thickness of the natural blanket has been calculated to be 39.4% with a standard deviation of 4.8 feet. Underseepage is controlled by landside underseepage berms between stations 83+00 and 220+00. Cut-off trenches are present between stations 205+00 and 462+50 at locations where the blanket is overlain by a sand layer or by existing pervious fill. Three (3) relief wells were placed at station 392+05 where the natural impervious blanket had been excavated for the basement of a warehouse building. Underseepage analyses for the reaches between stations 165+00 and 180+00 and between stations 205+00 to 298+00 evaluating the existing conditions indicate piping safety factors less than 1.0 for a river stage at the existing levee crest and were considered critical for reliability evaluation. The assumed soil material parameters and the details of the uncertainty analyses performed for these two reaches are shown on Figures 1 and 2 at the end of this section.

The critical water stage for 85 percent probability of failure for the reach between stations 165+00 and 180+00 is elevation 891 feet and 892 feet for the reach between stations 205+00 and 298+00.

Waterworks Unit.

The Waterworks Unit, located on the right bank of the Kansas River, consists of 1,998 feet of earthen levee and 1,662 feet of floodwall. The floodwall is constructed on a foundation soil consisting of an impervious blanket varying in thickness from 9 to 13 feet, overlaying a layer of very fine sand, which becomes progressively coarser with depth. The average impervious blanket thickness is 9.6 feet with a coefficient of variation of 28.2% and a standard deviation of 2.7 feet. Nine (9) relief wells provide underseepage control along the floodwall reach. A landside fill controls the underseepage along the levee embankment reach. Underseepage analyses considering the existing conditions indicated factors of safety less than 1.0 for a river stage at the levee crest for the reaches between stations 33+00 and 40+00. The assumed soil material parameters and the details of the uncertainty analyses performed at this reach are shown on Figure 3 at the end of this section.

The critical water stage for an 85 percent probability of failure within this reach is elevation 892.5 feet.

Auburndale Unit.

The Auburndale Unit is located along the right bank of the Kansas River east of the Waterworks Unit. The Interstate I-70 embankment is used as the right bank levee between the Waterworks Unit at the upper end and the South Topeka Unit at the lower end. Foundation soils below the levee embankment consist of an impervious blanket of silt or sandy silts varying in thickness between 8 and 14 feet. Near the bluff line, a clay blanket overlays the poorly graded foundation sand to a depth of up to 45 feet. A layer of impervious fill was placed on the highway landside slope to control through seepage in the embankment. Fifteen (15) relief wells are located between stations 2+00 and 17+50. A riverside impervious cut-off trench was keyed 1 foot into the impervious blanket between stations 80+00 and 137+00. Due to the high level of underseepage control and thickness of blanket, risk and uncertainty analyses were not considered to be required.

South Topeka Unit.

The South Topeka Unit is located along the right bank of the Kansas River and consists of 1.4

miles of earthen levee, and 1,944 feet of floodwall founded on an impervious blanket varying in thickness between 5 and 24 feet, with an average of 15.5 feet. The standard deviation of the blanket thickness is 5 feet and the coefficient of variation 32.4%. The blanket consisting of silty clays and silty sands overlays a sand deposit more than 80 feet thick. Fill placed on the top of the natural blanket between station 50+00 and 74+30 contains debris, rock, rubble, and sand requiring the construction of riverside cut-off trenches to reduce seepage. Between station 74+30 and 93+90, a 6 to 7 foot thick layer of debris required construction of 27 relief wells for underseepage control. The blanket beneath this fill averages only a few feet in thickness and appears to be entirely missing between stations 77+50 and 80+50. A seepage interceptor drain and relief wells were placed between stations 74+05 and 93+25. The interceptor was designed to control underseepage flow along a void detected at the base of the pile cap. The void was measured as 1/16" at the sheet pile cut-off wall and 3/4" at the toe. Underseepage analyses considering the existing conditions and a factor of safety less than 1.0 was computed for a river stage at the levee crest for the reaches between stations 0+00 and 72+20 where no relief wells exist. The assumed soil material parameters and the details of the uncertainty analysis performed for this reach is shown in Figure 4 at the end of this section.

Oakland Unit.

The Oakland Unit is located along the Kansas River downstream of the South Topeka Unit and along left bank of Shunganunga Creek. The Oakland Unit consists of 10 miles of earthen levee and 5.5 miles of channel improvements. Foundation soils of this flood risk management unit contain an impervious blanket that can be divided into three general areas considering blanket material and blanket thickness. The blanket in the upper reach, between stations 0+00 to 60+00, consists of clay-type material varying from silty clay to fat clay. Blanket thickness ranges between 20 and 30 feet. The middle reach, between stations 60+00 and 285+00, is overlain by an impervious silt blanket having a thickness of between 2 and 30 feet. The blanket thickness between stations 200+00 and 245+00 is very thin; having a thickness of between 0 and 4 feet. The reach along Shunganunga creek, from station 285+00 to the end, has a substantial blanket consisting of lean to fat clays with a thickness of between 20 and 35 feet. Underlying foundation sands possess a thickness ranging between 10 and 60 feet. Sands vary in grain size from very fine to medium in the upper half of the aquifer to coarser near the top of bedrock. The entire foreshore area between station 0+00 and approximate station 40+00 contains deposits of fill material consisting of waste material, debris, cinders, and rubble. A riverside cut-off trench exists between stations 0+00 and 523+20, constructed to reduce the seepage through the levee foundation. Relief wells between stations 205+00 and 237+50 control the underseepage. Underseepage analyses indicate factors of safety less than 1.0 for the reaches between stations 60+00 and 85+55 with a river stage at the levee crest. A relief well between stations 200+00 and 245+00, considering 50 percent efficiency, increases the underseepage stability to an acceptable level of greater than 1.0. The assumed soil material parameters and the results of the uncertainty analyses performed for the reach between stations 60+00 and 85+55 is shown on Figure 5 at the end of this section.

The critical water stage for an 85 percent probability of failure for the reach between stations 64+00 and 80+00 is elevation 880.5 feet.

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FIGURE 1 - UNDERSEEPAGE ANALYSIS
NORTH TOPEKA Section I
Station 172+00

Crest width (feet)	10.00	H	design water head		
Horizontal to vertical slope ratio	3.00	Hwt	head above tailwater at levee toe without berm	L2	levee base width
		H'wt	head above tailwater at 1/2 berm	Le	landside effective length
K-r riverside permeability ratio		H'o	head above tailwater at levee toe (w/ berm)	Lt	total effective length
K-L landside permeability ratio		i-c	critical seepage gradient	t'	underseepage berm thickness at toe
Dbr riverside blanket thickness		Wt	berm width		
DbL landside blanket thickness		i-o	seepage gradient		
Dbo blanket thickness under levee footprint		Wt	landside berm width		
Df pervious foundation thickness		Cr	riverside effective length coefficient		
Lr length of riverside blanket		Cl	landside effective length coefficient		
Ll length of landside blanket		L1	riverside effective length		

Station	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety Factor	L't	Hwt	H'wt	i-o	Cr	Cl	L1	L2	Le	Lt	H'o	t'
70+00	300	300	12.3	12.3	12.3	85.0	12.0	0.840	1500	0	1.84	1197	5.6	5.6	0.46	560	560	555	82	560	1197	NA	NA
105+00	300	300	4.0	4.0	4.0	20.0	15.9	0.840	500	250	1.65	540	4.6	2.0	0.51	155	155	154	105	155	415	7.17	3.16
150+00	400	400	12.0	12.0	12.0	30.0	15.3	0.840	350	25	1.38	769	7.5	7.3	0.61	379	379	276	102	379	757	6.76	0.03
172+00	300	300	4.8	4.8	4.8	40.0	17.6	0.840	250	190	0.90	637	6.63	4.5	0.93	240	240	187	116	240	542	8.05	3.44
190+00	300	300	6.5	6.5	6.5	40.0	14.5	0.840	200	150	1.10	623	6.5	5.0	0.76	279	279	172	97	279	548	7.17	2.27
248+00	300	300	6.7	6.7	6.7	70.0	13.7	0.840	50	10	0.58	522	9.8	9.7	1.45	375	375	50	92	375	517	8.68	3.16

Station 172+00

Head = 17.60
X2 = 116

	Kf/Kb	z	d	X3	s	ho	I	Variance Component	Percent of Variance
Mean	300	6.70	70.0	375	334.56	9.3	1.388468		
	180	6.70	70.0	291	318.36	8.4	1.253442	0.0131303	8.1367
	420	6.70	70.0	444	342.53	9.9	1.482617		
	300	4.60	70.0	311	323.14	8.6	1.875816	0.1446893	89.662
	300	8.80	70.0	430	341.17	9.8	1.115055		
	300	6.70	55.0	332	327.56	8.9	1.323249	0.0035521	2.2012
	300	6.70	85.0	413	339.40	9.7	1.442447		
							Total	0.1613717	100

E[I] = 1.3885
Var[I]= 0.16137
sigma[I]= 0.40171
V(I) = 0.2893

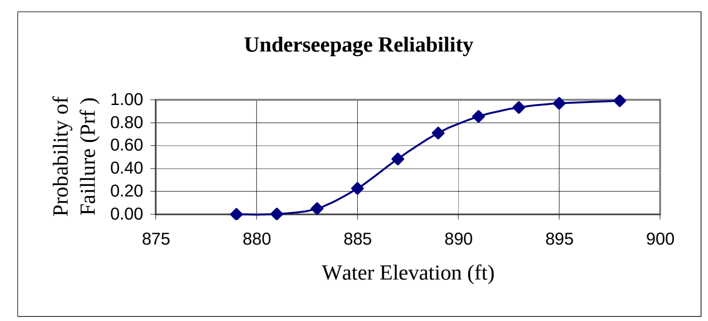
E[ln I] = 0.28801
sigma [ln I] = 0.283525

I crit = 0.840
ln(I crit) = -0.17435

F(z) = 0.05147
Pr(f) = 94.85295

Table 1 : Random Variables for North Topeka

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Blanket z	6.7	2.08	31.22
Perm Ratio	300	120	40.00
Fdn Sand	70	15	20.00



Head	Elev.	Prf
3	879	0.00000
5	881	0.00240
7	883	0.05118
9	885	0.22750
11	887	0.48432
13	889	0.70880
15	891	0.85420
17	893	0.93268
19	895	0.97051
22	898	0.99192

FIGURE 2 - UNDERSEEPAGE ANALYSIS

NORTH TOPEKA
Station 248+00

Crest width (feet)		10.00
Horizontal to vertical slope ratio		3.00
K-r	riverside permeability ratio	
K-L	landside permeability ratio	
Dbr	riverside blanket thickness	
DbL	landside blanket thickness	
Dbo	blanket thickness under levee footprint	
Df	pervious foundation thickness	
Lr	length of riverside blanket	
Ll	length of landside blanket	

H	design water head
Hwt	head above tailwater at levee toe without berm
H'wt	head above tailwater at 1/2 berm
H'o	head above tailwater at levee toe (w/ berm)
i-c	critical seepage gradient
Wt	berm width
i-o	seepage gradient
Wt	landside berm width
Cr	riverside effective length coefficient
Cl	landside effective length coefficient
L1	riverside effective length

L2	levee base width
Le	landside effective length
Lt	total effective length
t'	underseepage berm thickness at toe

Station	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety Factor	L't	Hwt	H'wt	i-o	Cr	Cl	L1	L2	Le	Lt	H'o	t'
70+00	300	300	12.3	12.3	12.3	85.0	12.0	0.840	1500	0	1.84	1197	5.6	5.6	0.46	560	560	555	82	560	1197	NA	NA
105+00	300	300	4.0	4.0	4.0	20.0	15.9	0.840	500	250	1.65	540	4.6	2.0	0.51	155	155	154	105	155	415	7.17	3.16
150+00	400	400	12.0	12.0	12.0	30.0	15.3	0.840	350	25	1.38	769	7.5	7.3	0.61	379	379	276	102	379	757	6.76	0.03
172+00	300	300	4.8	4.8	4.8	40.0	17.6	0.840	250	190	0.90	637	6.63	4.5	0.93	240	240	187	116	240	542	8.05	3.44
190+00	300	300	6.5	6.5	6.5	40.0	14.5	0.840	200	150	1.10	623	6.5	5.0	0.76	279	279	172	97	279	548	7.17	2.27
248+00	300	300	6.7	6.7	6.7	70.0	16.0	0.840	50	10	0.51	536	11.2	11.1	1.65	375	375	50	106	375	531	9.87	3.92

Station 248+00

Head = 16.00
X2 = 106

z = Dbl
d = Df
X3 = Cr

ho = Hwt

	Kf/Kb	z	d	X3	s	ho	I	Variance Component	Percent of Variance
Mean	300	6.70	70.0	375	155.71	11.3	1.687549		
	180	6.70	70.0	291	155.51	10.4	1.555504	0.0112464	4.3943
	420	6.70	70.0	444	155.79	11.8	1.767602		
	300	4.60	70.0	311	155.57	10.7	2.317993	0.2417777	94.471
	300	8.80	70.0	430	155.78	11.7	1.334575		
	300	6.70	55.0	332	155.63	10.9	1.626674	0.0029048	1.1350
	300	6.70	85.0	413	155.76	11.6	1.734467		
							Total	0.2559289	100

E[I] = 1.6875
Var[I]= 0.25593
sigma[I]= 0.50589
V(I) = 0.2998

E[ln I] = 0.48025

sigma [ln I] = 0.293354

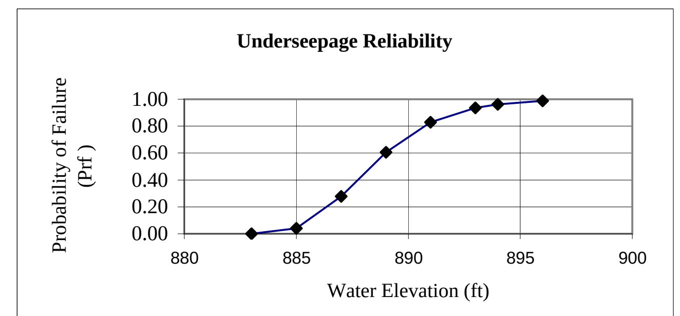
I crit = 0.840

ln(I crit) = -0.17435

F(z) =	0.01283
Pr(f) =	98.71739

Table 1 : Random Variables for North Topeka

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Blanket z	6.7	2.08	31.22
Perm Ratio	300	120	40.00
Fdn Sand depth	70	15	21.40



Head	Elev.	Prf
3	883	0.00026
5	885	0.04140
7	887	0.27874
9	889	0.60646
11	891	0.83000
13	893	0.93620
14	894	0.96215
16	896	0.98717

FIGURE 3 - UNDERSEEPAGE ANALYSIS
WATERWORKS
Station 172+00

Crest width (feet)	10.00	H	design water head		
Horizontal to vertical slope ratio	3.00	Hwt	head above tailwater at levee toe without berm	L2	levee base width
		H'wt	head above tailwater at 1/2 berm	Le	landside effective length
K-r riverside permeability ratio		H'o	head above tailwater at levee toe (w/ berm)	Lt	total effective length
K-L landside permeability ratio		i-c	critical seepage gradient	t'	underseepage berm thickness at toe
Dbr riverside blanket thickness		Wt	berm width		
DbL landside blanket thickness		i-o	seepage gradient		
Dbo blanket thickness under levee footprint		Wt	landside berm width		
Df pervious foundation thickness		Cr	riverside effective length coefficient		
Lr length of riverside blanket		Cl	landside effective length coefficient		
Ll length of landside blanket		L1	riverside effective length		

Station	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety Factor	L't	Hwt	H'wt	i-o	Cr	Cl	L1	L2	Le	Lt	H'o	t'
10+00	600	600	8.0	8.0	8.0	40.0	10.0	0.840	200	0	0.98	640	6.8	6.8	0.86	438	438	187	15	438	640	NA	NA
16+20	500	500	7.6	7.6	7.6	50.0	9.5	0.840	150	0	0.92	595	7.0	7.0	0.92	436	436	144	15	436	595	NA	NA
19+00	500	500	7.2	7.2	7.2	40.0	10.5	0.840	250	0	1.02	672	5.9	5.9	0.82	379	379	219	73	379	672	NA	NA
33+50	500	500	7.3	7.3	7.3	40.0	12.0	0.840	70	0	0.71	533	8.60	8.6	1.18	382	382	69	82	382	533	NA	NA

Station 33+50

Head = 14.00
X2 = 82

	Kf/Kb	z	d	X3	s	ho	I	Variance Component	Percent of Variance
Mean	500	7.30	40.0	382	151.23	10.0	1.374005		
	300	7.30	40.0	296	150.72	9.3	1.270705	0.0068789	8.8793
	700	7.30	40.0	452	151.45	10.5	1.436583		
	500	5.80	40.0	341	151.03	9.7	1.672249	0.0632867	81.690
	500	8.80	40.0	420	151.36	10.3	1.169112		
	500	7.80	30.0	342	151.04	9.7	1.245085	0.0073057	9.4302
	500	7.30	50.0	427	151.38	10.3	1.416032		
							Total	0.0774713	100

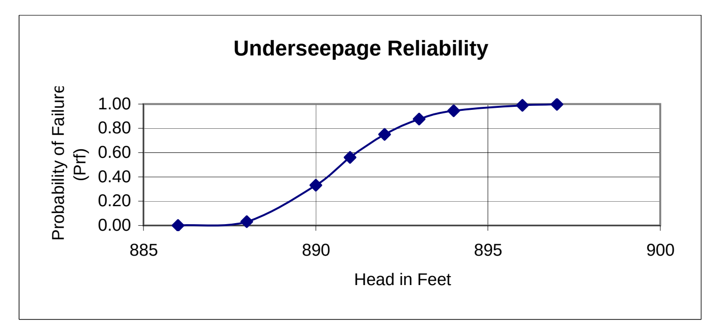
E[I] = 1.3740 E[ln I] = 0.29762
Var[I]= 0.07747
sigma[I]= 0.27834 sigma [ln I] = 0.200540
V(I) = 0.2026

I crit = 0.840 ln(I crit) = -0.17435

F(z) =	0.00930
Pr(f) =	99.07017

Table 1 : Random Variables for Waterworks

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Blanket z	7.3	1.5	20.80
Perm Ratio	600	240	40.00
Fdn Sand d	40	10	25.00



Head	Elev.	Prf
4	886	0.00005
6	888	0.03063
8	890	0.33105
9	891	0.55974
10	892	0.75038
11	893	0.87512
12	894	0.94350
14	896	0.99070
15	897	0.99651

FIGURE 4 - UNDERSEEPAGE ANALYSIS

SOUTH TOPEKA
Station 58+70

Crest width (feet)	10.00	H	design water head			
Horizontal to vertical slope ratio	3.00	Hwt	head above tailwater at levee toe without berm	L2	levee base width
		H'wt	head above tailwater at 1/2 berm	Le	landside effective length
K-r riverside permeability ratio		H'o	head above tailwater at levee toe (w/ berm)	Lt	total effective length
K-L landside permeability ratio		i-c	critical seepage gradient	t'	underseepage berm thickness at toe
Dbr riverside blanket thickness		Wt	berm width		
DbL landside blanket thickness		i-o	seepage gradient		
Dbo blanket thickness under levee footprint		Wt	landside berm width		
Df pervious foundation thickness		Cr	riverside effective length coefficient		
Lr length of riverside blanket		Cl	landside effective length coefficient		
Ll length of landside blanket		L1	riverside effective length		

											Factor									Le	Lt	H'o	t'
58+70	400	400	11.3	11.3	11.3	80.0	11.7	0.840	10	0	0.93	692	10.2	10.2	0.90	601	601	10	80	601	692	NA	NA
75+84	300	300	15.5	15.5	15.5	80.0	12.0	0.840	20	0	1.15	647	11.3	11.3	0.73	610	610	20	17	610	647	NA	NA
78+40	300	300	16.0	16.0	16.0	80.0	12.0	0.840	20	0	1.19	657	11.3	11.3	0.71	620	620	20	17	620	657	NA	NA
87+50	300	300	14.0	14.0	14.0	80.0	12.0	0.840	20	0	1.04	617	11.3	11.3	0.81	580	580	20	17	580	617	NA	NA

Station 58+70

Head = 16.00
X2 = 80

	Kf/Kb	z	d	X3	s	ho	I	Variance Component	Percent of Variance
Mean	400	11.30	80.0	601	89.9991	13.9	1.231600		
	560	11.30	80.0	712	89.9993	14.2	1.256937	0.0012351	0.4020
	240	11.30	80.0	466	89.9985	13.4	1.186649		
	400	6.70	80.0	463	89.9984	13.4	1.999435	0.3057468	99.511
	400	15.90	80.0	713	89.9993	14.2	0.893548		
	400	11.30	64.0	538	89.9988	13.7	1.212962	0.0002684	0.0874
	400	11.30	96.0	659	89.9992	14.1	1.245730		
							Total	0.3072504	100

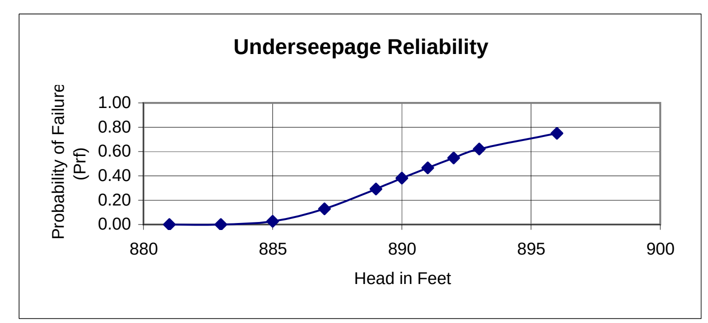
E[I] = 1.2316 E[ln I] = 0.11609
Var[I]= 0.30725
sigma[I]= 0.55430 sigma [ln I] = 0.429479
V(I) = 0.4501

F(z) = 0.24944

I crit = 0.840 ln(I crit) = -0.17435 Pr(f) = 75.05637

Table 1 : Random Variables for South Topeka Sta. 58+70

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Blanket z	11.3	4.6	40.80
Perm Ratio	400	160	40.00
Fdn Sand	80	16	20.00



Head	Elev.	Prf
1	881	0.00000
3	883	0.00094
5	885	0.02753
7	887	0.12819
9	889	0.29122
10	890	0.38038
11	891	0.46710
12	892	0.54777
13	893	0.62035
16	896	0.75056

FIGURE 5 - UNDERSEEPAGE ANALYSIS

OAKLAND

Station 78+00

Crest width (feet)	10.00	H	design water head
Horizontal to vertical slope ratio	3.00	Hwt	head above tailwater at berm end
		H'wt	head above tailwater at 1/2 berm
K-r	riverside permeability ratio	H'o	head above tailwater at levee toe (w/ berm)
K-L	landside permeability ratio	i-c	critical seepage gradient
Dbr	riverside blanket thickness	Wt	berm width
DbL	landside blanket thickness	i-o	seepage gradient
Df	pervious foundation thickness	Cr	riverside effective length coefficient
Lr	length of riverside blanket	Cl	landside effective length coefficient
Ll	length of landside blanket	L1	riverside effective length

L2	levee base width
Le	landside effective length
Lt	total effective length
t'	underseepage berm thickness at toe
Sc	= calculated slope of underseepage berm
tu	= used thickness of underseepage berm

Station	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety Factor	L't	Hwt	H'wt	i-o	Cr	Cl	L1	L2	Le	Lt	H'o	t'
78+00	600	600	7.0	7.0	7.0	40.0	16.0	0.850	10	0	0.48	526	12.5	12.5	1.78	410	410	10	106	410	526	NA	NA
78+00	600	600	7.0	7.0	7.0	40.0	15.0	0.850	10	0	0.51	526	11.7	11.7	1.67	410	410	10	106	410	526	NA	NA
78+00	600	600	7.0	7.0	7.0	40.0	14.0	0.850	10	0	0.55	526	10.9	10.9	1.56	410	410	10	106	410	526	NA	NA
78+00	600	600	7.0	7.0	7.0	40.0	13.0	0.850	10	0	0.59	526	10.1	10.1	1.45	410	410	10	106	410	526	NA	NA
78+00	600	600	7.0	7.0	7.0	40.0	12.0	0.850	10	0	0.64	526	9.4	9.4	1.34	410	410	10	106	410	526	NA	NA
78+00	600	600	7.0	7.0	7.0	40.0	11.0	0.850	10	0	0.69	526	8.6	8.6	1.22	410	410	10	106	410	526	NA	NA
78+00	600	600	7.0	7.0	7.0	40.0	10.0	0.850	10	0	0.76	526	7.8	7.8	1.11	410	410	10	106	410	526	NA	NA
78+00	600	600	7.0	7.0	7.0	40.0	8.0	0.850	10	0	0.95	526	6.2	6.2	0.89	410	410	10	106	410	526	NA	NA
78+00	600	600	7.0	7.0	7.0	40.0	6.0	0.850	10	0	1.27	526	4.7	4.7	0.67	410	410	10	106	410	526	NA	NA
78+00	600	600	7.0	7.0	7.0	40.0	3.0	0.850	10	0	2.54	526	2.3	2.3	0.33	410	410	10	106	410	526	NA	NA
78+00	600	600	7.0	7.0	7.0	40.0	1.0	0.850	10	0	7.63	526	0.8	0.8	0.11	410	410	10	106	410	526	NA	NA

Station 220+00

Head = 10.00

X2 = 100

ho= 4.9 (no relief wells)

ho=2.4 (100% relief wells efficiency)

ho=3.6 (50% relief wells effiency)

9

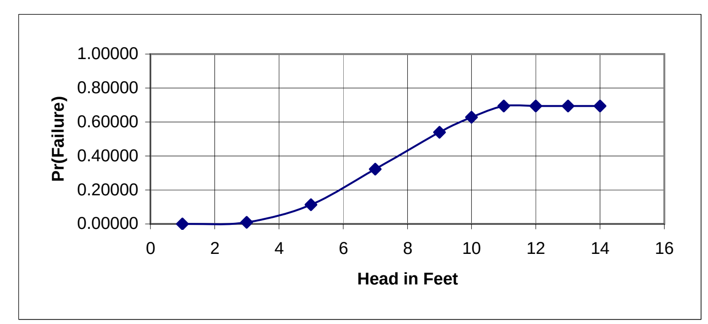
	Kf/Kb	z	d	X3	s	ho	I	Variance Component	Percent of Variance
Mean	50	3.00	60.0	95	109.96	4.6	1.543844		
	30	3.00	60.0	73	109.94	4.0	1.335429	0.0045853	1.1399
	70	3.00	60.0	112	109.97	3.6	1.200000		
	50	1.60	60.0	69	109.93	3.9	2.416188	0.3937432	97.888
	50	4.40	60.0	115	109.97	5.1	1.161209		
	50	3.00	51.0	87	109.96	4.4	1.476782	0.0039106	0.9722
	50	3.00	69.0	102	109.97	4.8	1.601851		
							Total	0.4022391	100

E[I] =	1.5438	E[ln I] =	0.35630
Var[I]=	0.40224		
sigma[I]=	0.63422	sigma [ln I] =	0.394900
V(I) =	0.4108		
I crit =	0.850	ln(I crit) =	-0.16252

F(z) =	0.09446
Pr(f) =	90.55439

Table 1 : Random Variables for Oakland Levee Sta. 71+25

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Blanket z	3	1.4	47.14
Perm Ratio	50	20	40.00
Fdn Sand	60	9	15.00



Head	Prf
1	0.00000
3	0.00958
5	0.11369
7	0.32296
9	0.53945
10	0.62851
11	0.69512
12	0.69512
13	0.69512
14	0.69512

A-3.1.9 Slope Stability Reliability

A risk analysis was performed on a basic typical section of the levee embankment for each unit, at reaches considered critical due to the levee height or foundation conditions. A sensitivity study was done to determine which three parameters in the slope stability calculations were most influential. For this study, those variables are soil strength in the embankment, soil strength in the foundation material such as cohesive soils and cohesionless soils. Statistical descriptors for these three variables were determined using available site-specific information and published statistical data as in the underseepage study. Details and results of the slope stability analysis are shown in Figures 6 through 10 at the end of this section.

Cases of Stability Analyses

Conditions analyzed for stability analyses considered long-term conditions having a steady state seepage condition along the landside slope for levees located on the Kansas River or rapid drawdown of the channel water for the riverside slope of projects located along Soldier Creek and Shunganunga Creek. When steady state conditions were analyzed, the water pressure in the sand layer underlying the natural impervious blanket was computed by underseepage analysis for every flood stage considered in calculations.

Soil Strength Parameters

Soil Strength Parameters used in the stability analyses were the drained soil parameters used for the original flood control project design. The only new subsurface investigation performed to refine the understanding of existing conditions involved cone penetration testing (CPT) at selected locations. The coefficient of variation for soil strength parameters were obtained using methodologies outlined in ETL 1110-2-556. The coefficient of variation of the blanket thickness was determined using all existing subsurface data.

Method of Stability Analysis

The limit equilibrium computer program "UTEXAS3" was used to perform the stability analyses. Circular failure surfaces were assumed and the embankment was modeled as homogeneous. All analyses consisted of running a search routine to identify the critical failure surface using the Spencer's Method. Three random variables were defined for each unit. Stability analyses were performed for different assumed river stages. Results of the stability analyses are summarized in the following paragraphs.

Probability Analysis

The Probability of Failure of a slope ($P_{rf(Failure)}$) is defined as the probability that the critical failure surface could be loaded to the limit equilibrium state. This infers the slope is loaded to its maximum capacity. For this study, the variables for slope stability were not assumed to be correlated to the parameters for underseepage analyses.

Results of Stability Analyses by Unit

Soldier Creek Unit. The Soldier Creek Unit was analyzed for a rapid drawdown condition in the channel. The critical section on Soldier Creek was considered to be the channel excavation between stations 13+00 and 113+00 where the channel slope is approximately 39 feet in height. The sand layer within this reach extends 56 feet below the top of the levee. The levee is located adjacent to the riverbank. Original design soil properties and those determined from the uncertainty analyses are shown on Figure 6. The probability that the factor of safety for slope stability could be less than 1.0 for increasing river levels for a reach between stations 13+00 and 133+00 is also shown on Figure 6. The 85% probability of failure corresponds to water elevation of 886 feet.

North Topeka Unit. The North Topeka Unit was analyzed assuming steady state seepage conditions and that the aquifer layer under the impervious blanket is being pressurized by the hydraulic gradient determined during underseepage analyses for different river stage elevations and different blanket thicknesses. The critical reach was considered to be located between levee stations 246+00 and 250+00. Impervious blanket thickness is 5 feet or less in thickness. Original design soil properties and those determined from the uncertainty analyses are provided in Figure 7. The probability that the factor of safety for slope stability is less than 1.0 for increasing river stages is shown by the curve presented in Figure 7.

Waterworks Unit**.** The Waterworks Unit was analyzed for the steady state condition considering the aquifer layer underneath the impervious blanket as being pressurized by the hydraulic gradient developed during underseepage analyses for different river stage elevations and different blanket thicknesses. The critical section for stability was considered to be between stations 7+00 and 73+00 where the impervious layer thickness is less than 7 feet thick. The original design soil properties and those determined from the uncertainty analyses are also provided in Figure 8. The probability that the factor of safety for slope stability is be less than 1.0 for increasing river stages is indicated by the curve presented in Figure 8. The elevation corresponding to 85 % probability of failure is 893 feet.

Auburndale Unit. The Auburndale Unit is located along the right bank of the Kansas River east of the Waterworks Unit. No stability analyses were performed for this levee unit since the foundation conditions and the height of the levee did not give any indication of any weak reaches. The impervious blanket is thicker than 8 feet throughout and consists of silt or sandy silts having an internal friction angle of 26.5 degrees, as recommended for the original design. The levee height does not exceed 15 feet, with the crest elevation varying between 897.23 feet at the upper end and 895.75 at the lower end. Critical failure surfaces for steady state seepage conditions will not penetrate the impervious blanket. Considering all these conditions, no instabilities were deemed to exist within this unit.

South Topeka Unit. The South Topeka Levee Unit was analyzed for steady state seepage conditions considering the aquifer layer underneath the impervious blanket as being pressurized by the hydraulic gradient determined during underseepage analyses for different river stage elevations. The critical section for stability was identified as the reach between stations 50+00 and 73+00 where the impervious blanket layer thickness is less than seven feet. Original design soil properties and the variations used in the uncertainty analysis are also provided in Figure 9.The probability that the factor of safety for slope stability is be less than 1.0 for increasing river stages is indicated by the curve presented in Figure 9.

Oakland Unit. The Oakland Levee Unit was analyzed for the steady state seepage condition considering the aquifer layer underneath the impervious blanket as being pressurized by the hydraulic gradient determined during underseepage analyses for different river stage elevations. The critical section for stability was identified as being between stations 64+00 and 80+00 where the impervious blanket layer thickness is less than 8 feet. Original design soil properties and those determined from the uncertainty analyses are provided in Figure 10. The probability that the factor of safety for slope stability is be less than 1.0 for increasing river stages is indicated by the curve presented in Figure 10.

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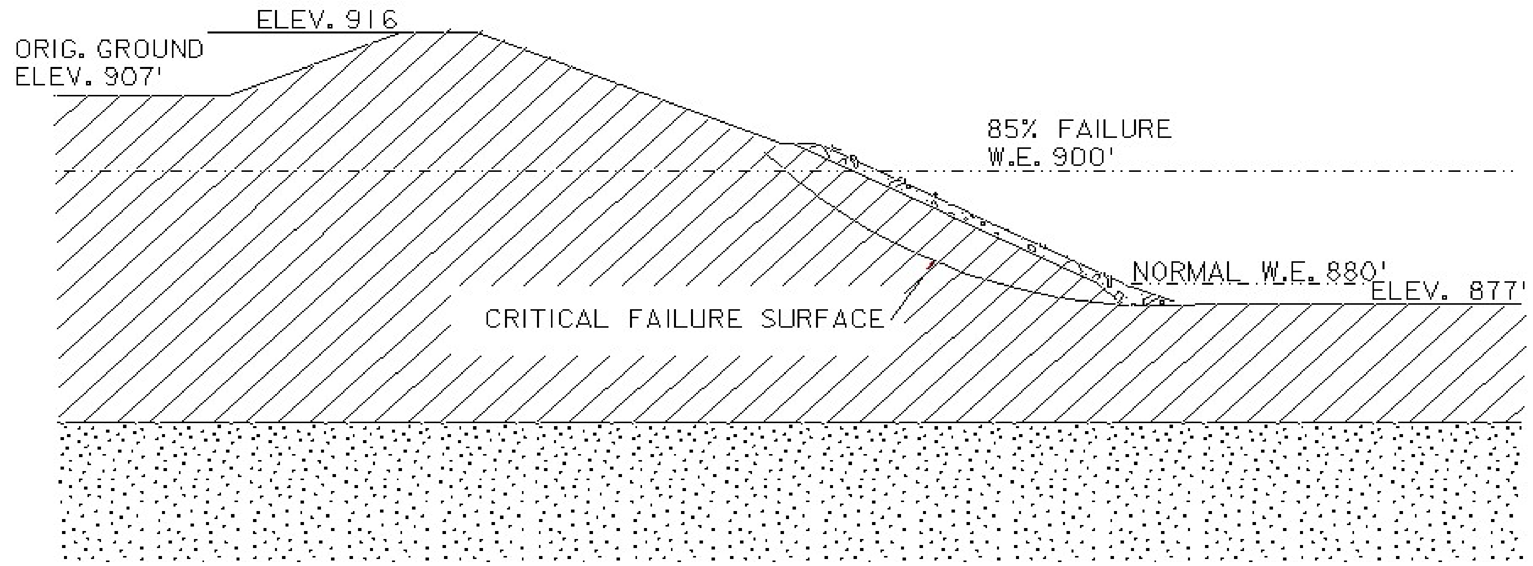
FIGURE 6 - STABILITY ANALYSIS **Reliability Analysis**

Soldier Creek	Sta.	Rapid
Unit	13+00	Drawdown



Head =	23.00
Water Elev.	900
Levee Crest	916

	Phi Clay Mat.	Phi Sand	Gamma Clay	FS		
Mean	26.50	32.0	Clay Mat.	0.609		
	23.80	32.00	107.00	0.569		
	29.20	32.00	107.00	0.682		
	26.50	28.20	107.00	0.609		
	26.50	35.80	107.00	0.609		
	26.50	32.00	98.00	0.572		
	26.50	32.00	116.00	0.617		

	26.5	2.70	
	32	3.80	
	107	9.00	

	Required
	1.0
	Pr(f)
877	0.00590
880	0.22558
883	0.59073
886	0.85000
890	0.96361
900	1.00000

100

E[FS] =	0.60900	E[ln FS] =	-0.50090
Var[FS]=	0.00370		
sigma[FS]=	0.06082	sigma [ln FS] =	0.099613

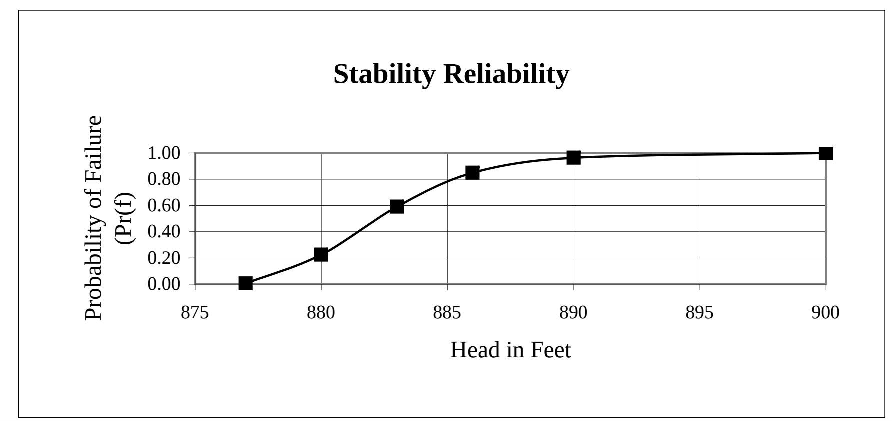
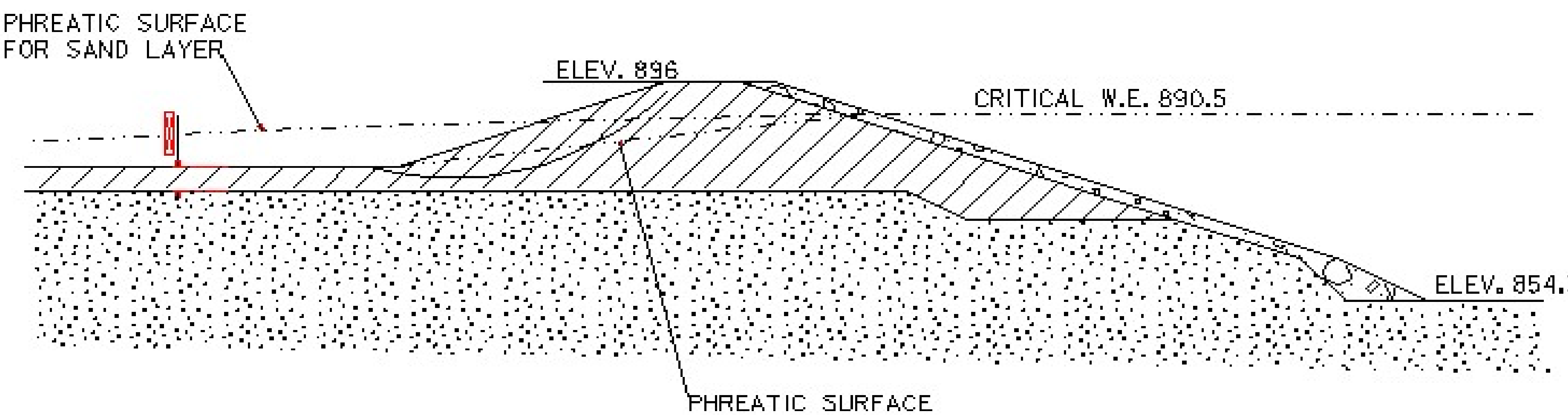


FIGURE 7 - STABILITY ANALYSIS **Reliability Analysis**

North Topeka Sta. 248+00 Steady State



Head =	16.00
W.E.	896.0

	Reliability Analysis of Critical Slide				Variance Component	Percent of Variance
	Ph Clay	Phi Sand	Clay Depth Elev.	FS		
Mean	26.50	32.0	873.30	0.950		
	23.80	32.00	873.30	0.840	0.0126563	10.5258
	29.20	32.00	873.30	1.065		
	26.50	28.00	873.30	0.950	0.0000000	0.000
	26.50	36.00	873.30	0.950		
	26.50	32.00	875.30	0.294	0.1075840	89.4742
	26.50	32.00	871.20	0.950		
				1		
				Total	0.1202403	100

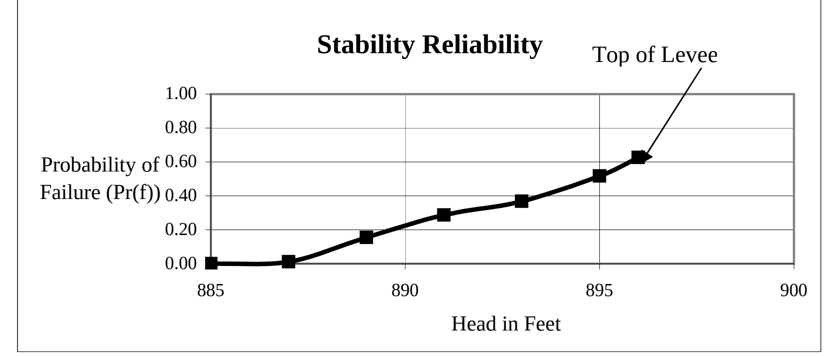
E[FS] =	0.9500	E[ln FS] =	-0.11383
Var[FS]=	0.12024		
sigma[FS]=	0.34676	sigma [ln FS] =	0.353655
V(FS) =	0.3650		
FS req'd =	1.000	ln(FS req'd) =	0.00000

Beta =	-0.321865
F(z) =	0.62622
Pr(f) =	62.622259

Pr(f) = Probability of a stability factor of safety less than one

Table 1 : Random Variables for North Topeka

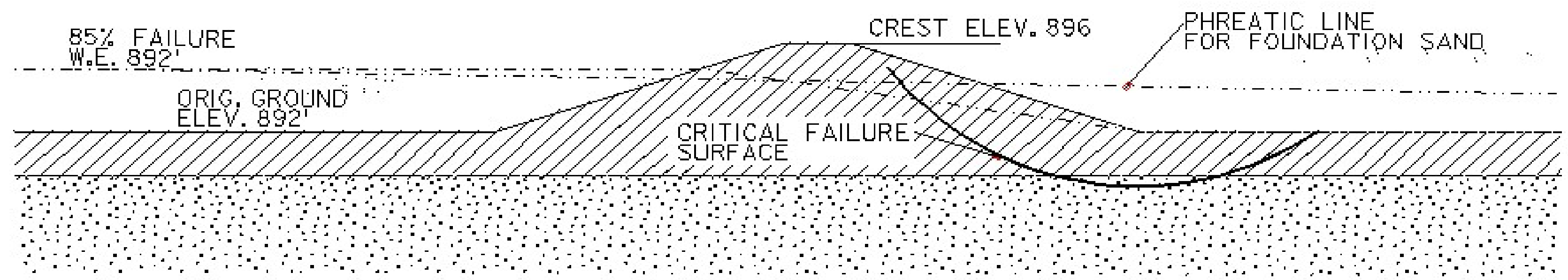
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Impervious Phi	26.5	1.700	10.00
Foundation Sand Phi	32	3.200	12.00
Clay Blanket Thickness	6.7	2.000	21.60



Head	Required 1.0 Pr(f)
885	0.0005
887	0.01217
889	0.15412
891	0.28620
893	0.36752
895	0.51674
896	0.62622

FIGURE 8 - STABILITY ANALYSIS **Reliability Analysis**

Waterworks	Sta.	Steady
Unit	33+00	State



Head =	12.00
Water Elev.	894
Levee Crest	896

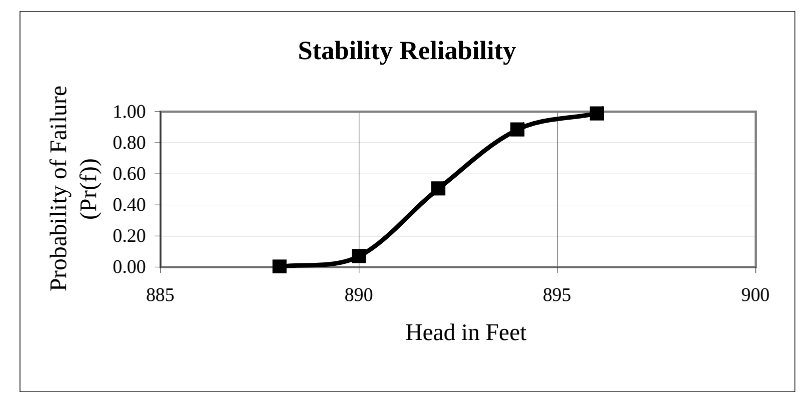
	Reliability Analysis of Critical Slide					
	Phi Clay Mat.	Phi Sand	Blanket Thickness	FS	Variance Component	Percent of Variance
Mean	26.50	32.0	7.00	0.759		
	23.80	32.00	7.00	0.626	0.0163840	41.2560
	29.20	32.00	7.00	0.882		
	26.50	28.20	7.00	0.793	0.0023040	5.802
	26.50	35.80	7.00	0.697		
	26.50	32.00	5.50	0.708	0.0210250	52.9424
	26.50	32.00	8.50	0.998		
				1		
					0.0397130	100

E[FS] =	0.7590	E[ln FS] =	-0.30909
Var[FS]=	0.03971		
sigma[FS]=	0.19928	sigma [ln FS] =	0.258194
V(FS) =	0.2626		
FS req'd =	1.000	ln(FS req'd) =	0.00000

Beta =	-1.197105
F(z) =	0.88437
Pr(f) =	88.436727

Table 1 : Random Variables for Waterworks

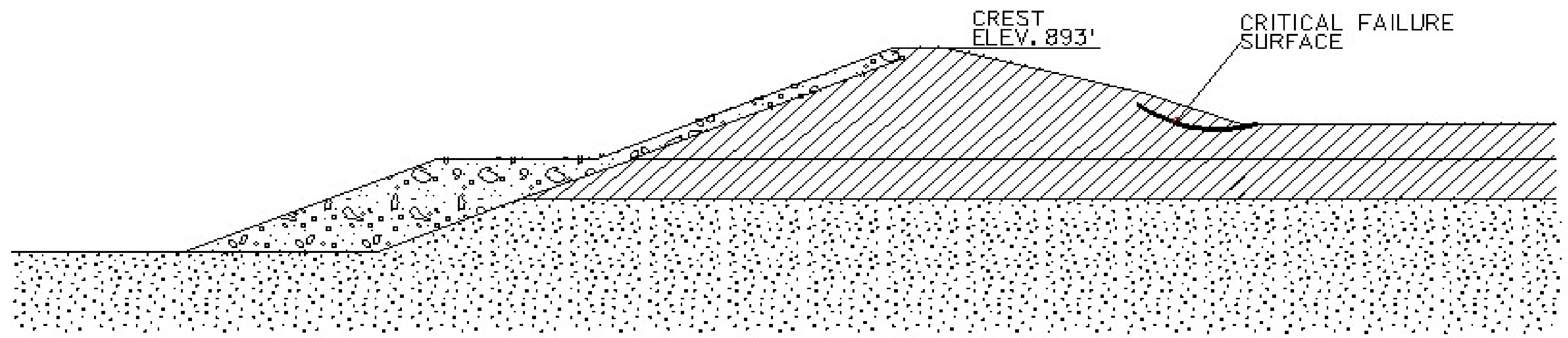
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Phi Clay Material	26.5	2.70	10.00
Phi Foundation Sand	32	3.80	12.00
Blanket Thickness	7	1.50	21.40



Head	Required 1.0 Pr(f)
888	0.00178
890	0.06908
892	0.50438
894	0.88437
896	0.98845

FIGURE 9 - STABILITY ANALYSIS **Reliability Analysis**

South Topeka Unit	Sta. 56+00	Steady State



Head =	13.00
W.E.	893
Crest	893

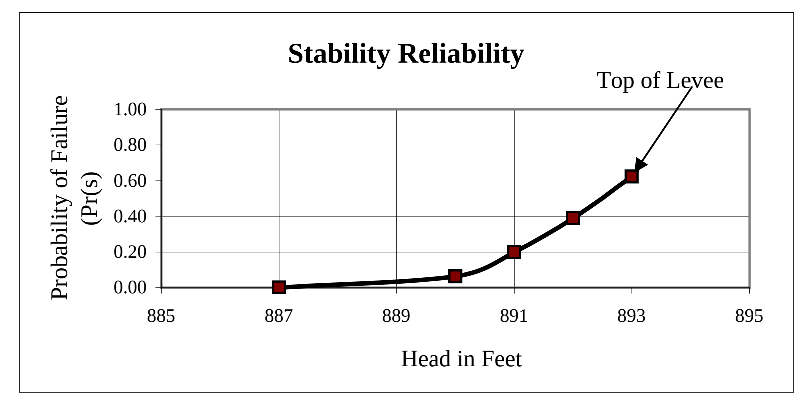
	Reliability Analysis of Critical Slide				Variance Component	Percent of Variance
	Phi Exist. Fill	Phi New Fill	Phi Found. Clay	FS		
Mean	24.00	26.50	Clay Mat.	0.963		
	21.60	26.50	22.00	0.857	0.0219040	88.6299
	26.40	26.50	22.00	1.153		
	24.00	28.20	22.00	0.961	0.0000010	0.004
	24.00	35.80	22.00	0.963		
	24.00	26.50	19.80	0.963	0.0028090	11.3660
	24.00	26.50	24.20	1.069		
				1		
				Total	0.0247140	100

E[FS] =	0.9630	E[ln FS] =	-0.05085
Var[FS]=	0.02471		
sigma[FS]=	0.15721	sigma [ln FS] =	0.162175
V(FS) =	0.1632		
FS req'd =	1.000	ln(FS req'd) =	0.00000

Beta =	-0.313564
F(z) =	0.62307
Pr(s) =	62.307394

Table 1 : Random Variables for South Topeka Levee

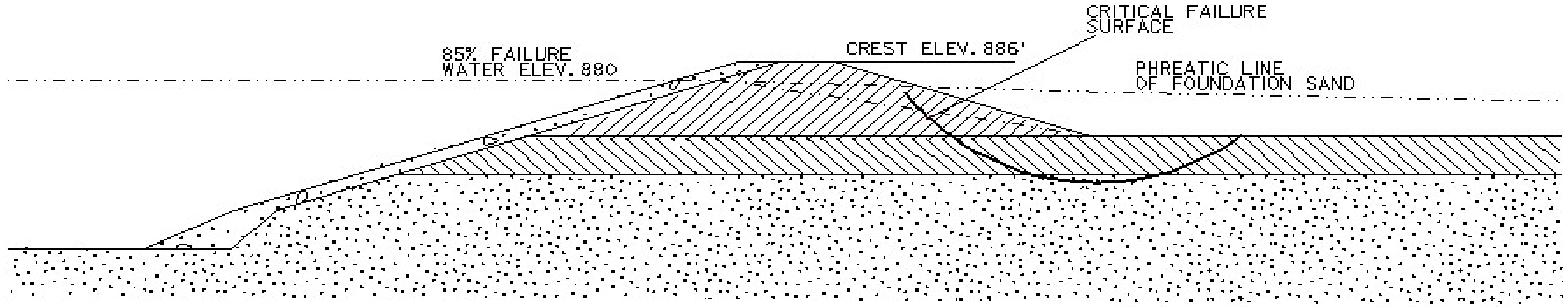
Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Phi Existing Fill	24	2.40	10.00
Phi Embakment Fill	26.5	2.70	10.00
Phi Foundation Clay	22	2.20	10.00



Head	Required 1.0 Pr(s)
887	0.00000
890	0.06228
891	0.19843
892	0.38931
893	0.62307

FIGURE 10 - STABILITY ANALYSIS **Reliability Analysis**

Oakland Unit Sta. 78+00 Steady State



Head = 14.00
W.E. 884

Reliability Analysis of Critical Slide					Variance Component	Percent of Variance
	Phi Embank.	Phi Found.	Pfhi Sand	FS		
Mean	26.50	19.0	32.00	0.460		
	24.00	19.00	32.00	0.455	0.0000250	0.1129
	29.00	19.00	32.00	0.465		
	26.50	17.00	32.00	0.342	0.0129960	58.696
	26.50	21.00	32.00	0.570		
	26.50	19.00	28.00	0.347	0.0091203	41.1912
	26.50	19.00	36.00	0.538		
				Total	0.0221413	100

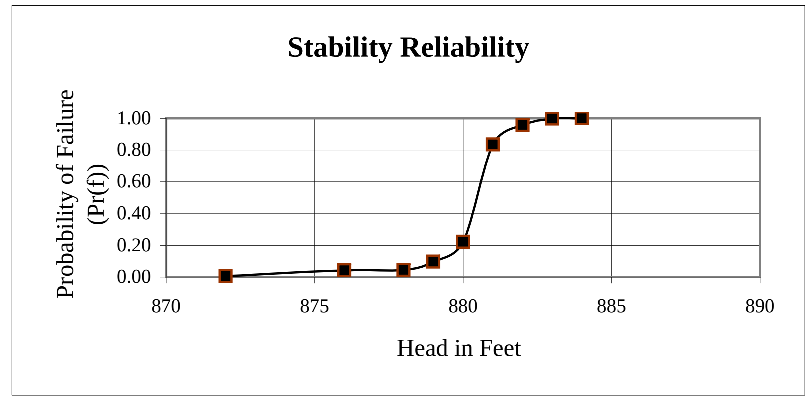
E[FS] = 0.4600 E[ln FS] = -0.82629
Var[FS]= 0.02214
sigma[FS]= 0.14880 sigma [ln FS] = 0.315463
V(FS) = 0.3235

FS req'd = 1.000 ln(FS req'd) = 0.00000

Beta =	-2.619283
F(z) =	0.99559
Pr(u) =	99.559425

Table 1 : Random Variables for Oakland Levee

Parameter	Expected Value	Standard Deviation	Coefficient of Variation, %
Embankment Phi	26.5	2.500	10.00
FoundationClay Phi	19	2.000	10.00
Foundation Sand Phi	32	4.000	12.00



Head	Required 1.0 Prf
872	0.00648
876	0.04207
878	0.04425
879	0.09584
880	0.22161
881	0.83448
882	0.95774
883	0.99854
884	0.99945

A-3.1.10 Conclusions of the Uncertainty Analysis

The total conditional probability of failure as a function of floodwater elevation has been developed by combining the probability of failure functions for two failure modes; underseepage piping and slope instability. The reliability is the probability of no failure due to each mode considered in the calculations. The total probabilities of failure function computed for each critical levee unit are indicated in the following figures. The combined probability curves are shown on Figures 11 through 15 at the end of this section.

Soldier Creek Levee Unit.

The combined probability of failure along the Soldier Creek Channel between stations 13+00 and 130+00 is as shown in Figure 11. The 85 percent probability of having a localized channel slope failure for the Soldier Creek Unit between stations 13+00 and 130+00 occurs for a flood stage of 886 feet, a water level of between 6 and 13 feet above the bottom of the existing channel. This channel reach does have an established history of bank slides. In 1967, near station 40+00, an emergency rehabilitation contract was required to repair a major bank failure into the extended toe of the levee. Without emergency repair, the levee embankment could have been lost. No other bank slides have directly threatened the levee integrity in this area. No underseepage piping has been considered critical for these analyses. The levee crest elevation along Soldier Creek varies between 919 and 886 feet and the Soldier Creek Channel bottom varies between elevations 880 and 873 feet. As determined during stability analyses, channel side slopes fail in this area due to sudden drawdown conditions. This creates the possibility of a progressive failure of the channel and failure of the levee if repair of the channel banks are not accomplished shortly after the initial signs of distress are observed. However, since the failures are due to sudden drawdown of the water elevation in the Soldier Creek, after the water reaches a very low elevation, the risk of flood damages of the protected area are not existent if the riverside slope is repaired before the next flood occurs. Consequently, the probability of failure of the riverside slope due to sudden drawdown should not be included in the risk analysis since the repairs can be done between two consequent floods and the damages are limited to the riverbank slope. The damages described in Table 12 are limited to the riverbank and can be repaired if they occurred after a flood event.

North Topeka Levee Unit.

The combined probability of failure for the critical sections between stations 246+00 and 250+00 is illustrated in Figure 12. The 85 percent probability of failure for this reach occurs for a flood stage of elevation 890.5 feet. The levee crest elevation varies within this reach between elevations 895.6 and 896.0 feet.

Waterworks Levee Unit.

The combined probability of failure for the critical section between stations 16+62 and 33+50 is illustrated by the curve shown in Figure 13. The 85 percent probability of failure for this reach occurs for a flood stage of elevation 892 feet. The levee crest elevation varies between 897.0 and 897.6 feet.

South Topeka Levee Unit.

The combined probability of failure for the critical section between stations 0+00 and 73+00 is illustrated in Figure 14. The 85 percent probability of failure for this reach occurs for a flood stage of elevation 893 feet corresponding to the elevation of the levee crest.

Oakland Levee Unit.

The combined probability of failure for the critical section between stations 64+00 and 80+00 is illustrated by the curve shown in Figure 15. The 85 percent probability of failure for this reach occurs at a flood stage of elevation 880 feet. The levee crest elevation varies within this reach between 886 and 887 feet.

Figure 11
Topeka - Soldier Creek   Station 13+00 to 113+00
Top Elev. 916

Flood Water Elevation	Pr(f)	R	Pr(f)	R	Pr(f)	R
	Underseepage		Slope Stability		Combined	
877	0.00000	1.00000	0.00590	0.994100	0.00590	0.99410
880	0.00000	1.00000	0.22558	0.774420	0.22558	0.77442
883	0.00000	1.00000	0.59073	0.409270	0.59073	0.40927
886	0.00000	1.00000	0.85000	0.150000	0.85000	0.15000
890	0.00000	1.00000	0.96361	0.036390	0.96361	0.03639
900	0.00000	1.00000	1.00000	0.000000	1.00000	0.00000

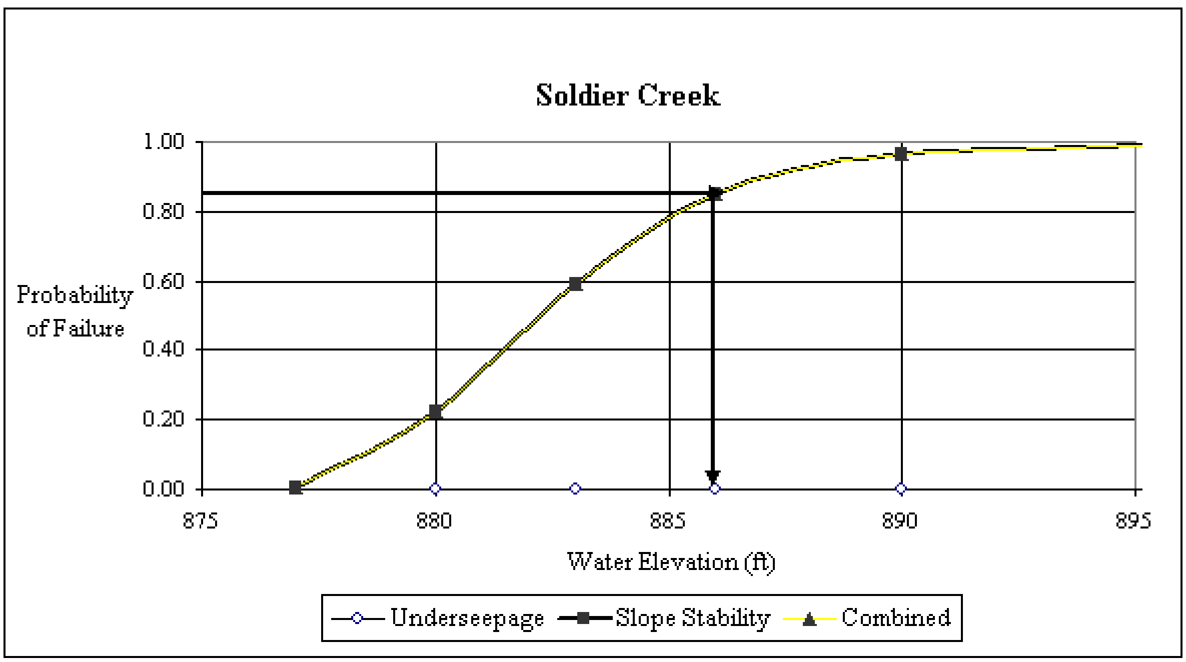


Figure 12
Topeka - North Topeka Unit
Station 246+00 to 260+00
Levee Crest Elev. 896.5

Flood Water Elevation	Underseepage		Slope Stability		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R
	Underseepage		Slope Stability		Combined	
883	0.00026	0.99974	0.00000	1.00000	0.00026	0.99974
885	0.04140	0.95860	0.00050	0.99950	0.04188	0.95812
887	0.27874	0.72126	0.01217	0.98783	0.28752	0.71248
889	0.60646	0.39354	0.15412	0.84588	0.66711	0.33289
891	0.83000	0.17000	0.28620	0.71380	0.87865	0.12135
893	0.93620	0.06380	0.36752	0.63248	0.95965	0.04035
896	0.98717	0.01283	0.62622	0.37378	0.99520	0.00480

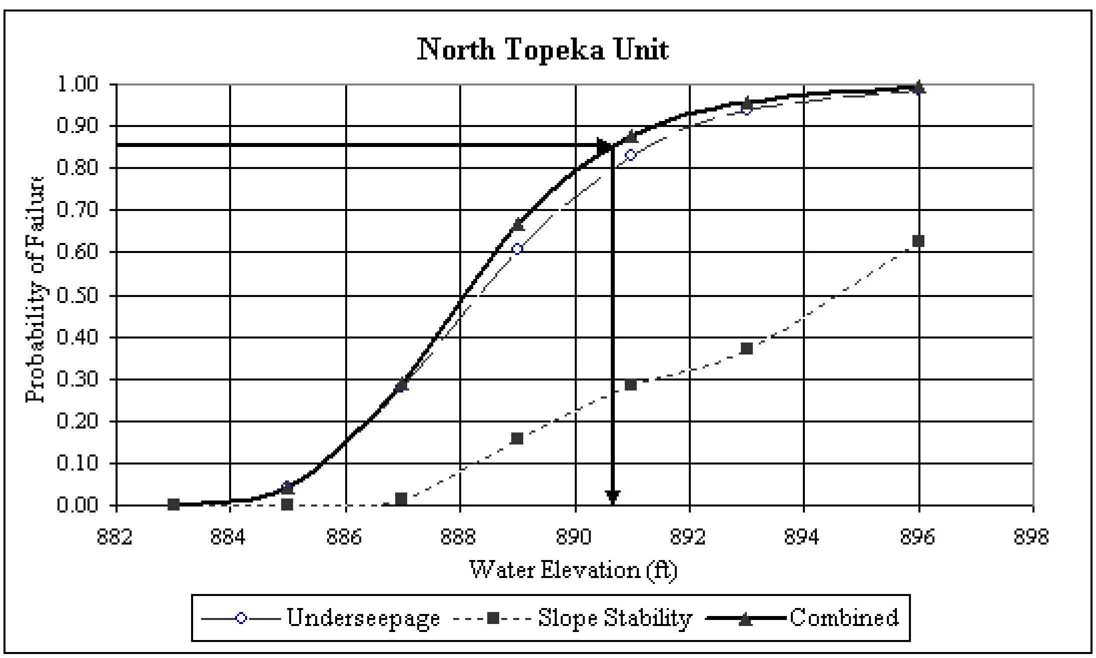


Figure 13

Waterworks Levee Unit Station 16+62 and 33+50

Top of Levee 896'

Flood Water Elevation	Underseepage		Slope Stability		Combined	
	Pr(f)	**R**	**Pr(f)**	**R**	**Pr(f)**	**R**
	Underseepage		**Slope Stability**		**Combined**	
888	0.03063	0.96937	0.00178	0.998220	0.032355	0.967645
890	0.33105	0.66895	0.06908	0.930920	0.377261	0.622739
892	0.75038	0.24962	0.50438	0.495620	0.876283	0.123717
894	0.94350	0.0565	0.88437	0.115630	0.993467	0.006533
896	0.99070	0.00930	0.98845	0.011550	0.999893	0.000107

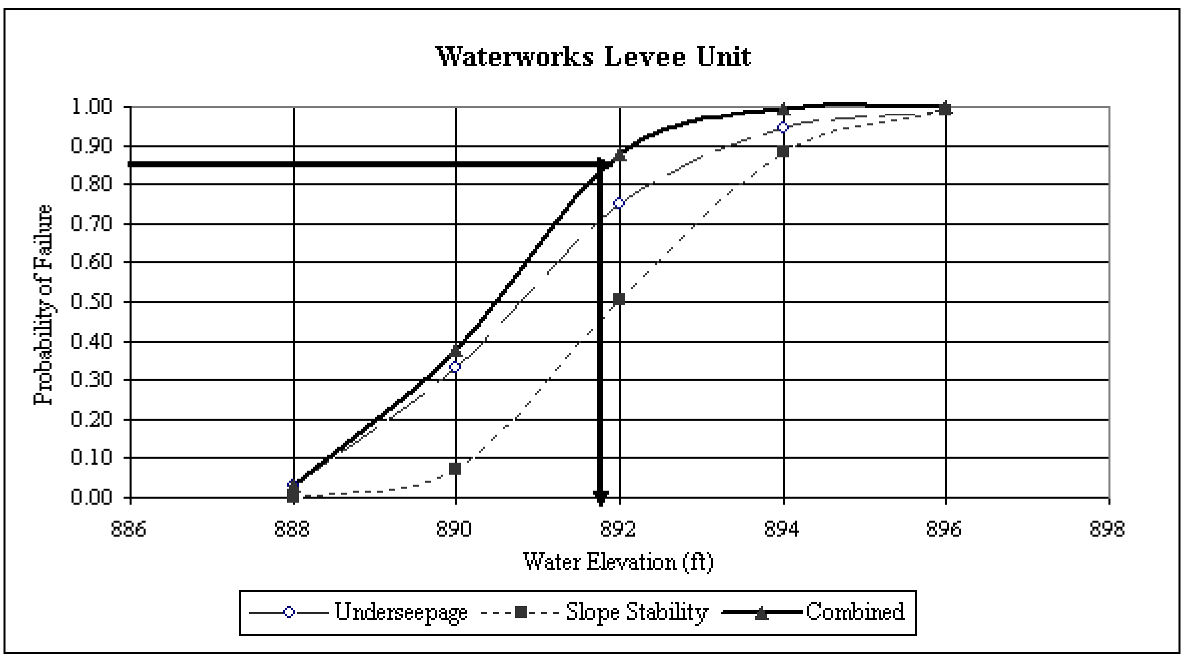


Figure 14
Topeka - South Topeka
Unit **Station 0+00 to 73+00**

Levee Crest Elev. 893'

Flood Water Elevation	Underseepage		Slope Stability		Combined	
	Pr(f)	R	Pr(f)	R	Pr(f)	R
	Underseepage		Slope Stability		Combined	
881	0.00000	1.00000	0.00000	1.000000	0.00000	1.00000
883	0.00094	0.99906	0.00000	1.000000	0.00094	0.99906
885	0.02753	0.97247	0.00000	1.000000	0.02753	0.97247
887	0.12819	0.87181	0.00000	1.000000	0.12819	0.87181
890	0.38038	0.61962	0.06228	0.937720	0.41897	0.58103
891	0.46710	0.5329	0.20843	0.791570	0.57817	0.42183
892	0.54777	0.45223	0.38931	0.610690	0.72383	0.27617
893	0.62035	0.37965	0.62307	0.376930	0.85690	0.14310

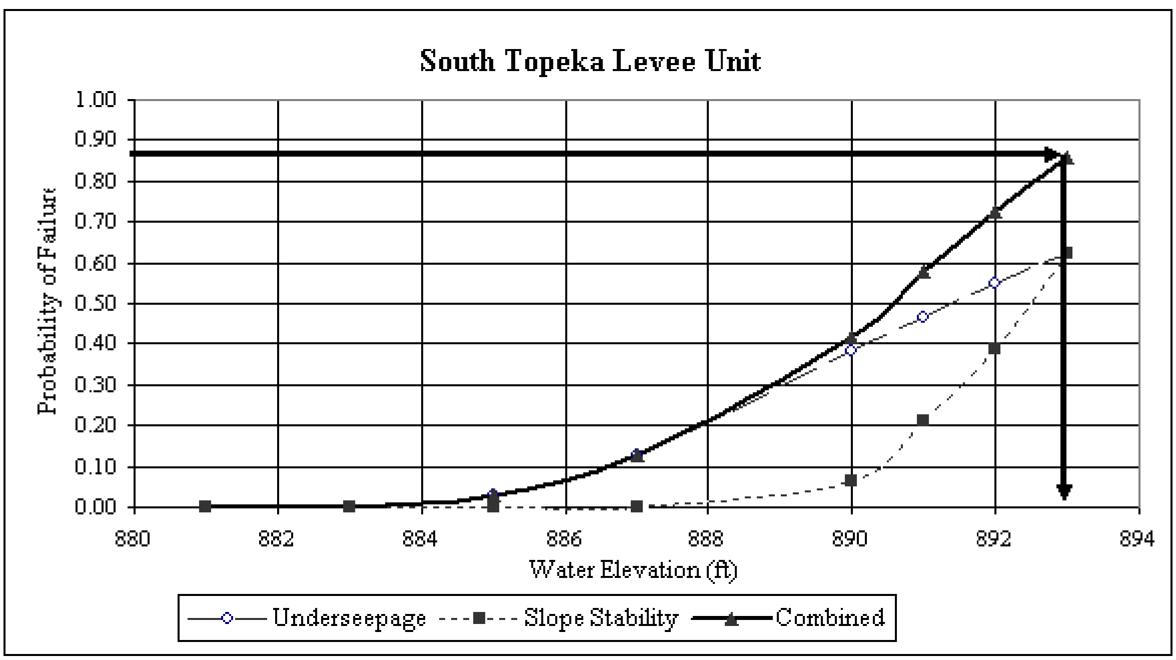
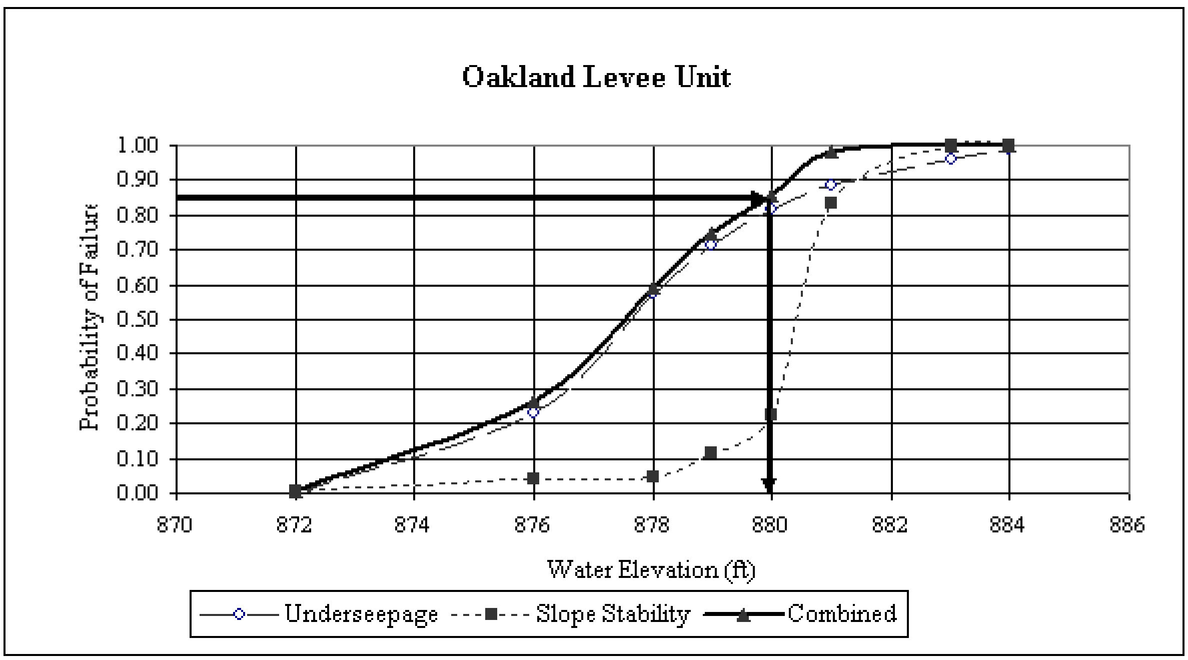


Figure 15
Topeka - Oakland Unit   Station 64+00 to 80+00
Levee Crest 886'

Flood Water Elevation	Underseepage		Slope Stability		Combined	
	Pr(f)	**R**	**Pr(f)**	**R**	**Pr(f)**	**R**
	Underseepage		**Slope Stability**		**Combined**	
872	0.00000	1.00000	0.00648	0.99352	0.00648	0.99352
876	0.23152	0.76848	0.04207	0.95793	0.26385	0.73615
878	0.57522	0.42478	0.04425	0.95575	0.59402	0.40598
879	0.71492	0.28508	0.11584	0.88416	0.74794	0.25206
880	0.81754	0.18246	0.22161	0.77839	0.85797	0.14203
881	0.88725	0.11275	0.83448	0.16552	0.98134	0.01866
883	0.95954	0.04046	0.99854	0.00146	0.99994	0.00006
884	0.98990	0.01010	0.99945	0.00055	0.99999	0.00001



A-3.1.11 Levee System Reliability Summary

Based on the uncertainty analyses of the individual units of the Topeka Flood Protection System, critical reaches of the Topeka levee system have been identified and are summarized in Table 2. The geotechnical order of risk based on the combined risk and uncertainty analysis is shown in Table 3.

Table 2 - Critical Reaches for Topeka Levee System

Levee Unit	Critical Station Range	Average Levee Crest Elevation	Flood Stage for 85% Probability of Failure	Freeboard Distance to Levee Crest @ 85% Failure Probability Stage
North Topeka	246+00 to 250+00	896.0	890.5	5.5
Waterworks	16+62 to 33+50	897.0	892.0	5.0
Auburndale	N/A	N/A	N/A	N/A
South Topeka	0+00 to 73+00	893.0	893.0	0.0
Oakland	64+00 to 80+00	886.50	880.0	6.5

Table 3 - Combined Geotechnical Risk and Uncertainty Analysis

Order of Risk (high to low)	Levee Unit Reach	Nature of Risk	**Damages**	Nature of Cost
1. North Topeka	246+00 to 260+00	• Slope Failure • Loss of Levee	• Property • Loss of Lives	• Dollars • Loss of Lives
2. Waterworks	16+62 to 33+50	• Slope Failure • Loss of Levee	• Loss of water plant • Loss of Lives	• Utility Loss • River Contamination • Loss of Lives
3. Oakland	64+00 to 80+00	• Potential loss of full levee	• Property • Loss of Lives	• Flooding of Oakland area • Flood Fighting Costs •Levee Repair Costs
4. South Topeka	0+00 to 73+00	• Levee Toe Slide • Complete loss of Levee Toe	• Property • Loss of Life	• Levee Repair Costs • Loss of Life
5. Soldier Creek	13+00 to 130+00	• Bank slides	• Uncontrolled Revision of Channel • Channel Flow Impacts • Opposite Bank Scour	• Repair of Flood Damages on the Riverbank

A-3.2 Future Conditions

A-3.2.1 Introduction

Future conditions were modeled and recommendations are made to improve underseepage conditions during flood conditions. This section presents the geotechnical evaluation and results for five of the six units of Topeka levee system.

A-3.2.2 Future Flooding Concerns

Observations after the completion of the Existing Conditions analysis has resulted in refinements to proposed areas of concern. The areas of concern outlined in Table 4 reflect a reduced scope based on observations by the Geotechnical Design Section of Engineering and Construction Division of the Kansas City District.

Table 4. Levee Unit Areas of Concern

Levee Unit	Area of Concern
North Topeka	165+00 to 189+00
	245+75 to 249+50
Oakland	64+00 to 80+00
South Topeka	22+00 to 48+00
Waterworks	64+00 to 80+00

Area Site Characterization

Boring logs located in the as-built drawings serve as the basis for the characterization of the foundation for each berm analyzed.

Underseepage Analysis

The underseepage analysis is modeled after consideration of the types of soils landward of the levee, the consistency of the thickness of the soil blanket clays or silts, the thickness and type of sand deposit below the levee blanket materials, the lateral extent of the blanket landside and riverward of the levee, the effects of the location of the Kansas river, and the height of the existing levee. All of these variables were considered during the development of the model to characterize the representative reaches along the alignment of the levee.

Underseepage can lead to piping. Piping of the blanket materials could lead to subsequent piping of sand grains toward the river entrance, leading to ultimate collapse of the levee section due to the foundation voids caused by piping. Piping occurs when soil begins moving in the blanket. Soil can become mobilized when the pressure in a vertical column of material changes and exceeds the weight of the material bearing on the location where the pressure change occurs. Because pressure typically decreases from depth to the surface, a diagram of the change in pressure typically produces a sloping line or “gradient”. The underseepage design aims to assure that the weight of the soil column at any depth exceeds the upward gradient by a factor of safety.

Levee Loading Conditions

An analysis was performed to evaluate existing seepage conditions. Analysis is based on rationale and formulas presented in the Kansas City District’s Guidance link on the Geotechnical Section Home Page:

http://www.nwk.usace.army.mil/local_protection/guidance.html.

Deficient conditions are determined by checking the factor of safety (FS) for piping to occur under different river elevations. One condition exists when the river is at the top of the levee, known as full head (FH). Deficiency under these conditions is defined when the FSFH is less than 1.1. The other condition exists when the river is three feet below the top of levee. Deficiency under this condition is defined at any FSFH-3 less than 1.5. The threshold value for FSFH-3 is higher than FSFH because the likelihood of the water reaching three feet below the top of levee, and maintaining that level, is greater than the likelihood of water reaching the top of levee and maintaining that level for a period of time.

The Kansas City District method of estimating the underseepage gradient and the required FS deviates somewhat from the method presented in the EM-1110-2-1913. The Kansas City District’s traditional empirical approach has been extensively used and has proven effective in providing adequate underseepage control for most reaches within the Topeka Levee System. This method is based on conclusions of a Corps of Engineers conference, held in Omaha in November, 1962.

Underseepage results will be verified at PED based on ETL 1110-2-569 (1 May 2005).

Input

Permeability parameters were assigned to the blanket materials based on the content of silt, clay, or sand. Only areas that contained a blanket thickness of least 1/4 the height of the levee were considered meaningful in the underseepage model. The traditionally assumed permeability ratios for blanket materials are shown in Table 5. Table 6 shows design assumptions for each unit analyzed.

Table 5. Permeability Ratios for Blanket Materials

Blanket Material	Assigned Permeability Ratio
SM: Silty Sand	100
ML: Silt	200-400
ML-CL: Silt and Clay	400
CL: Low Plasticity Clay	400-600
CH: High Plasticity Clay	800-1000

Table 6. Assumptions for Design

Unit	Max. Water Head at Top of Levee, ft	Ave. Blanket Thickness, ft	Material Type
Oakland	10.75	7	Silt and Clay
North Topeka, sta. 165+00	16.7	6.7	Silt
North Topeka, sta. 246+00	16	6.7	Silt
South Topeka	17	10.6	Silt and Clay

Mitigation Strategies

Berm design was considered only when the area landside of the levee was available for construction. If area for a berm was not available, a buried collector system was considered. In areas that exhibited a blanket thickness of less than 5 feet, relief wells were considered appropriate to provide the underseepage control.

Calculations

The calculations of the underseepage factors of safety used in the underseepage analysis are shown below:

The gradient piping factor of safety is defined as:

$$\text{FSi} = \frac{i_c}{i_o}$$

where: ic = critical (or maximum) gradient through blanket = (γs – γw) / γw
io = actual gradient = ho / DbL

The actual gradient, io, is the change in head from the base of the blanket to the top of the blanket. The reference datum is set at the top of the blanket because the movement of the soil grain will begin at the top of the blanket. Actual gradient, io, is defined as the head above the tailwater at the landside levee toe, ho, divided by the depth of the blanket on the landside, DbL. The head above tailwater on the landside, ho, is defined by the following equation:

$$h_0 = \frac{H * L_e}{L_1 + L_2 + L_e}$$

where: H = total head on levee
Le = distance from the landside toe of the assumed impervious section to the effective

seepage exit.
L1= effective length of the riverside blanket
L2= base width of the assumed impervious fill and natural blanket beneath it

The effective length of riverside blanket, L1, is defined by the following equation:

$$L_1 = C_r * \frac{e^{\left(\frac{2L_r}{C_r}\right)} - 1}{e^{\left(\frac{2L_r}{C_r}\right)} + 1}$$

where: Lr= actual length of the riverside natural blanket
Cr= effective length of the pervious foundation of infinite length covered by a natural impervious blanket

$$= \sqrt{D_{fr} * D_{br} * \left(\frac{k_{fr}}{k_{br}}\right)}$$

where: Dfr =depth of pervious riverside foundation
Dbr= depth of impervious riverside natural blanket
kfr= permeability of pervious riverside foundation
kbr= permeability of impervious riverside natural blanket

A-3.2.3 Recommendations

The original designers considered underseepage berms, buried collector, and relief wells for the area being considered. No underseepage control measures were adopted due to marginal safety concerns. The constructed levee section did include a riverside cutoff trench through any unknown upper sand lens layers and a landside sand blanket above the existing ground surface to control any underseepage infiltrating beyond the riverside cutoff trench. The area was to be monitored closely during high water, and future consideration for underseepage control measures were to be based on the monitoring of these reaches.

Geotechnical concerns are related to underseepage beneath the levee which may occur during high flow events. If uncontrolled underseepage is allowed to surface on the landside during a flood, it can create a failure of the levee foundation by piping. Underseepage pressures can be countered using either landside underseepage berms (additional soil placed on the ground surface) to prevent flow to the surface, or by pressure relief wells that provide a controlled path for the underseepage. Berms are the preferred method based on lower installation cost and maintenance needs, but require more real estate for installation and borrow areas. In locations where real estate is not available, relief wells can be installed.

Table 7 shows conclusions from the Existing Conditions Analysis and this Future Conditions

Analysis. The first row is shaded to highlight that it is taken from Table 3.

Table 7. Existing Analysis Summarized with Future Conditions Analysis

	Order of Risk					
	(1) North Topeka		(2) Waterworks	(3) Oakland	(4) South Topeka	(5) Soldier Creek
Existing Conditions Analysis (Table 3) Geotechnical Risk Extents		246+00 to 260+00	16+62 to 33+50	64+00 to 80+00	0+00 to 73+00	13+00 to 130+00
Levee Reaches Analyzed in Future Conditions Analysis	165+00 to 189+00	245+75 to 249+50	No improvements recommended	64+00 to 80+00	22+00 to 48+00	None.
Remedy	Landside underseepage berm	New pressure relief wells	No geotechnical action.	Landside underseepage berm	Land side underseepage berm	No geotechnical action.
Changes from Existing Conditions Analysis	Updated design parameters resulted in adding the proposed length of improvement	Updated design parameters resulted in adjusting the proposed length of improvement.	An existing berm placed by others after construction of unit is not identified on as-built drawings. Need for further action to be identified during PED phase.	No change	Updated design parameters resulted in adjusting the proposed length of improvement.	Low risk. No loss of life or property impacts due to bank slides during falling river phases.

The following is list of the specific modifications proposed for the Topeka Levee system by unit and location:

Oakland Unit

From stations 64+00 to 80+00, install new land side underseepage berm. Dimensions would be

6.5 feet thickness of fill at levee toe sloping to three feet thick at end of berm, and 240 ft. wide. Total borrow required would be 84,500 cy, which includes an additional 25% to account for volume change during excavation and compaction.

Oakland Berm
Station 64+00 to 80+00 = 1600 ft of levee
Berm width: 240 ft landward
Thickness at levee toe: 6.5 ft.
Thickness at end of berm: 3.0 ft.
Average berm thickness: 4.75 ft.

(1600' x 240' x 4.75')/27 = 67,600 cy + 25% = 84,500 cy

North Topeka Unit

Approximately from stations 165+00 to 189+00, install new land side underseepage berm. Dimensions would be seven feet thickness of fill at the levee toe sloping to three feet thick at end of berm, and 220 feet wide. Total borrow required would be 122,250 cy, including an additional 25% required due to volume change during excavation and compaction.

North Topeka Berm
Station 165+00 to 189+00 = 2400 ft of levee
Berm width: 220 ft. landward
Thickness at levee toe: 7 ft
Thickness at end of berm: 3 ft
Average berm thickness: 5 ft

(2400' x 220' x 5')/27 = 97,800 cy + 25% = 122,250 cy

From station 245+75 to 249+50, install new pressure relief wells. Install six wells spaced at 75 feet, each to a depth of 75 feet. The wells are to drain to a central manhole using a buried header system; the total discharge of the system is to be one cfs per well or six cfs total (2700 GPM). The drainage district will be required to pump the water down one foot below existing ground when the river is near the top of levee. A pad should be constructed on the slope for access. The railroad has a series of tracks just outside of the toe of the levee. Work may need to be done inside of the footprint (temporary excavation for drilling access, header pipe system and manhole installation). Civil and mechanical engineers should be consulted to determine the number of manholes required.

Soldier Creek Unit

Most damage to the Soldier Creek Unit is estimated to be from bank slides that would occur after the river rapidly drops then rises again. No loss of life or property impacts are projected to occur. Therefore, no mitigation is considered for this unit.

South Topeka Unit

Approximately from stations 22+00 to 48+00, install new land side underseepage berm. Dimensions would be five feet thickness of fill at levee toe sloping to three feet thick at end of berm, and 100 feet wide. Total borrow would be 48,150 cy, including an additional 25% required due to volume change during excavation and compaction. The calculations are shown below:

South Topeka Berm
Station 22+00 to 48+00 = 2600 ft of levee
Berm width: 100 ft landward
Thickness at levee toe: 5 ft.
Thickness at end of berm: 3 ft
Average berm thickness: 4 ft.

(2600' x 100' x 4')/27 = 38,520 cy + 25% = 48,150 cy

Waterworks Unit

Seepage at this unit was determined not to be a concern after it was discovered fill has been placed where an underseepage berm would have been recommended. The preconstruction, engineering, and design (PED) phase should include analysis of existing conditions to verify assumptions.

A-3.2.4 Borrow Sources

Local sources on the riverside of the levee are probable candidates for borrow material. The PED phase will further evaluate borrow sources with borings, testing, and characterization to determine if the borrow material is suitable. Requirements for underseepage berm material dictate the berm material have a permeability equal to or greater than the underlying soil. It is anticipated all borrow material will be the same and is expected to meet the permeability requirements. Borrow material will be stripped below existing grade before construction of the underseepage berm. Strippage will be replaced as a cap for the completed underseepage berm and serve as topsoil.

A-3.3 References

The following documents prepared by the Kansas City District, U. S. Army Corps of Engineers were used in this study:

Topeka, Kansas, Design Memorandum No. 13, South Topeka Unit, dated May, 1966.

Topeka, Kansas, Design Memorandum No. 3, Waterworks Unit, dated July, 1957.

Topeka, Kansas, Design Memorandum No. 11, Oakland Unit, dated September, 1960.

Topeka, Kansas, Design Memorandum No. 15, North Topeka Unit, dated June, 1961.

Topeka, Kansas, Design Memorandum No. 7, Auburndale Unit, dated September, 1958.

Topeka, Kansas, Design Memorandum No. 2, Soldier Creek Diversion Unit, dated July 1956.

Operation and Maintenance Manual, Flood Protection Project, Topeka, Kansas, Volume Three, Auburndale Unit, Appendix I

Operation and Maintenance Manual, Flood Protection Project, Topeka, Kansas, Volume Two, Soldier Creek Diversion Unit, Appendix I, dated January, 1963.

Topeka, Kansas, Waterworks Unit, Flood Control Project Construction Plans for Relief Well System, Levee, and Appurtenances, dated April 1958.

Operation and Maintenance Manual, Flood Protection Project, Topeka, Kansas, Oakland Unit, Volume Four, Appendix I, dated January 1965.

Operation and Maintenance Manual, Flood Protection Project, Topeka, Kansas, North Topeka Unit, Volume Five, Appendix I

Operation and Maintenance Manual, Flood Protection Project, Topeka, Kansas, Volume Three, Auburndale Unit, Appendix I

Master Operations and Maintenance Manual, Flood Protection Project, Kansas River Basin, Volume 8, Topeka Kansas", dated April 1978.

In addition, the following documents were used in this study:

Design and Construction of Levees, EM 1110-2-1913, prepared by the Department of the Army, Office of the Chief of Engineers, dated April 2000.

Duncan, J. M., Buchignani, A. L., "Geotechnical Engineering: An Engineering Manual for

Slope Stability Studies" Department of Civil Engineering, University of California, Berkeley. March 1975.

Reliability Assessment of Existing Levees for Benefit Determination, ETL 1110-2-328, prepared by the U. S. Army Corps of Engineers, dated March 22, 1993.

Shackelford, C. D., Nelson, P. P., Roth, M. J. S., Uncertainty in the Geologic Environment: From Theory to Practice, Geotechnical Special Publication No. 58 (1996), ASCE, New York, New York.

Risk Based Analysis for Evaluation of the Hydrology/Hydraulics, Geotechnical Stability, and Economics in Flood Damage Reduction Studies, ER 1105-2-101, prepared by the U. S. Army Corps of Engineers, dated March 1996

Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies. EC 1110-2-554, prepared by the U. S. Army Corps of Engineers, dated February 1998.

HEC-FDA, Flood Damage Reduction Analysis, Users Manual, prepared by the U. S. Army Corps of Engineers, dated March 1998

J. Michael Duncan, Michael Navin, and Katherine Patterson, “Manual for Geotechnical Engineering Reliability Calculations”, Department of Civil Engineering, Virginia Polytechnic Institute and State University, December 1999.

J. Michael Duncan, Hon M., “Factors of Safety and Reliability in Geotechnical Engineering” Department of Civil Engineering, Virginia Polytechnic Institute and State University, paper submitted for publication in ASCE Geotechnical Journal, May 1999

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

Chapter A-4

CIVIL DESIGN ANALYSIS

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A-4 TOPEKA CIVIL DESIGN ANALYSIS

A-4.1 INTRODUCTION

This chapter presents the results of the civil design evaluation performed as part of the existing conditions analysis for the Topeka Local Flood Protection Project. The area of civil design encompasses utility relocations, bridges, and other infrastructure items affected by proposed work. Sanitary, Gas, and water lines were analyzed for Auburndale, N. Topeka, S. Topeka, Oakland, Soldier Creek, and Waterworks.

A-4.2 BRIDGE CLEARANCES

S. Topeka floodwall from sta.74+41 to 93+86 will be replaced due to structural risks detailed in the structural portion of this report. Kansas Avenue Bridge is directly above floodwall. This feasibility doesn't modify the access road or the wall elevations.

A-4.3 REAL ESTATE

A Preliminary Attorney's Opinion of Compensability has been prepared and used for the purpose of completing the study. Final opinions and final relocation determinations will later occur as required by paragraph 12-22 of Engineering Regulation 405-1-12. Any conclusion or categorization contained in this appendix that an item is a utility or facility relocation would result in work to be performed at the cost of the nonfederal sponsor as part of LERRD responsibilities and is preliminary only. The Government will make a final determination of the relocations necessary for the construction, operation or maintenances of the project after further analysis and completion and approval of Final Attorney's Opinions of Compensability for each of the impacted utilities and facilities. For further details on all real estate issues, see the Real Estate Appendix included as part of the main Engineering Feasibility Report.

A-4.4 UTILITY RELOCATIONS

A review of the Kansas City District's criteria for utility lines was performed and a criteria document was developed. See attached document Exhibit A-4.3 Topeka Utility Crossing Guidance. This document was used in determining the disposition of existing utility lines crossing the levee.

A-4.4.1 UTILITY CROSSINGS

N. Topeka Unit

UL 2: Sta 9+35, 24 in Corrugated metal Pipe (CMP). Approximately 6' below top of levee. **Replace with Reinforced Concrete Pipe (RCP).**

UL 3: Sta 275+50, 21 in gravity CMP, Approximately 20 ft below top of levee. **Replace with Reinforced Concrete Pipe (RCP).**

UL 4: Sta 303+60, 18 in waterline CIP, Approximately 16 ft below top of levee. **No action**

Oakland Unit

UL 5: Sta 300+81, 6 in classification unknown , unknown depth. **Investigate during PED.**

UL 7: Sta 516+85, 6 in water, Approximately 12 ft below top of levee. **Relocate up and over levee or provide positive closure.**

Soldier Creek Unit

UL 8: Sta 114+60, 4 in steel gas , Approximately 4 ft below top of levee. **Relocate up and over levee.**

Waterworks Unit

UL 10: Sta 14+90, 2300 Volt powerline in , Approximately 12 ft below top of levee. **Relocate or provide positive closure.**

UL 11: Sta 33+75, 18 in water CIP, Approximately 24 ft below top of levee. **Relocate or provide positive closure.**

UL 12: Sta 35+90, 18 in water CIP, Approximately 24 ft below top of levee. **Relocate or provide positive closure.**

A-4.4.2 Power Lines

No levee raises are anticipated as a result of this feasibility study. As such, modifying powerlines for clearances aren't required.

A-4.4.3 Utility Uplift

The study of uplift on existing utilities was conducted to estimate costs for relocation or removal of functioning or abandoned utilities. Regions were identified for utility uplift concern, based on geotechnical and structural criteria. The region is 500 feet landward of the levee centerline and corresponds with the "critical zone" of the levee.

The attached spreadsheets are titled “Pipe Uplift” (Exhibit A-4.3) and are labeled for the various pipe locations. The civil designer provided the expected types of piping and depths that may be anticipated for the existing piping.

HDR Inc provided a review of the existing project dated January 2000. The HDR report was assumed to have most current data. The references used to determine pipe uplift were taken from 1) HDR reports or 2) Topeka operational and maintenance manual 1978 or 3) Topeka various supplemental designs. Pipe types and sizes, and related comments were taken from the HDR reports. Top of levee and supporting information were taken from the operational manual. Necessary information not found in these two data sets were obtained from the supplemental designs.

In some cases, depths of utilities were not available. Assumptions of 2 ft of cover for gravity lines and 3 ft for water lines were made. In cases where geotechnical data wasn’t available, the pressure head (H’O) was assumed. These assumption need to be verified during PED.

Acceptable uplift conditions are calculated under extreme conditions as provided for in ETL 1110-2-307. The uplift factor of safety under this condition is 1.1. Utilities that don’t meet this condition fail and require corrective course of actions. A general characterization has also been used for utilities, i.e., if a 10 in pipe failed uplift with 4 ft of cover, then a 6 in pipe with similar cover and soil properties would also fail with no uplift calculations needed.

Acceptable uplift conditions:

These utilities are considered acceptable for uplift and are shown as ‘OK” in the action column below.
2, 5, 14, 15, 16,17, 26

Unacceptable uplift conditions:

These utilities are considered unacceptable for uplift and are shown as ‘NG” in the action column below. Uplift calculations were not performed on each utility but were grouped by similar grouping characteristics.
4, 6,7,8,22,24,25,27,28

Investigate during PED:

These utilities don’t have enough information to be analyzed properly. In some cases, the utilities are shown on The HDR inventory list but not on operational drawings. The ground survey work, which will be done at PED, will provide the information necessary to determine their uplift condition.

In the cases where H'O has been assumed, utilities 18,19,20,21 have failed uplift considerations. H'O needs to be verified once proper geotechnical data is available. For cost purposes, an average utility relocation will be applied to 50% of the total amount of utilities that need to be investigated during PED. These utilities will need further investigation.
1,3, 9,10,11,12,13,18,19,20,21, 23,29

A-4.4.3.1 Auburndale

a. Six utilities were reviewed on this system. Two uplift calculations were performed. The row heading Pipe Line Item No. refers to the 2nd column of spreadsheet exhibit A-4.5 UPLIFT SUMMARY. The results are as follows:

Pipe Line Item No.	Action
1	unknown. Dia. investigate PED
2	Assume OK for grouted pipe
3	unknown. Dia. investigate PED
4	Uplift calc #1 NG
5	Uplift calc #2 OK
6	Uplift NG based on calc #1

Missing information for Pipes 1 and 3 require further investigation during PED. Uplift calculations show failure for uplift on pipe 4 and 6.

A-4.4.3.2 N. Topeka

a. Four utilities were reviewed on this system. Two uplift calculations were performed. The results are as follows:

Pipe Line Item No.	Action
7	Uplift NG
8	Uplift NG
9	Investigate PED
10	Investigate PED

Uplift Calculations for No. 7 and No. 8 show failing uplift conditions. These pipes require investigation during PED.

A-4.4.3.3 Oakland

Not enough information was available to determine uplift.

Pipe Line Item No.	Action
11	Investigate PED
12	Investigate PED
13	Investigate PED

A-4.4.3.4 Soldier Creek

a. Eight utilities were reviewed on this system. Four uplift calculations were performed. The results are as follows:

Pipe Line Item No.	Action
14	Uplift OK based on calc #9
15	Uplift OK, based on calc no. 10
16	Uplift OK, Calc no. 9
17	Uplift OK based on calc #9
18	Uplift NG, Calc No. 11
19	Uplift NG, based on calc #11
20	Uplift NG, Calc No. 12
21	Uplift NG based on calc #12

Reliable geotechnical data wasn't available for the Soldier creek analysis for the above soil types. As such, pressure heads (H'O) were assumed to be at levee top elevations (worst case conditions).

A-4.4.3.5 S. Topeka

a. Four utilities were reviewed on this system. Three uplift calculations were performed. The results are as follows:

Pipe Line Item No.	Action
22	Uplift NG, Calc No. 5
23	Investigate PED
24	Uplift NG, Calc No. 7
25	Uplift NG , Calc No. 8

Pipe Line item 22 is in the floodwall section that will be replaced.

A-4.4.3.6 Waterworks

a. Four utilities were reviewed on this system. Two uplift calculations were performed on the worst cases. The results are as follows:

Pipe Line Item No.	Action
26	Uplift OK, Calc No. 6
27	Uplift NG, Calc No. 13
28	Uplift NG, based on Calc No. 13
29	Investigate PED

A-4.4.3.7 REFFERENCE

The following documents were used in this study:

Topeka, Kansas, HDR reconnaissance study, Topeka Units, dated Sep, 1997.

Topeka, Kansas, Operation and Maintenance Manual Volume III, dated August, 1978.

Topeka, Kansas, Operation and Maintenance Manual Volume III, Auburndale Unit, dated July 1963.

Topeka, Kansas, Operation and Maintenance Manual, Volume VI, South Topeka Unit, dated April 1974

Topeka Kansas, Operation and Maintenance Manual, Section I, Oakland Unit, dated Dec 1961

Topeka, Kansas, Operations and Maintenance Manual, Volume 5, N. Topeka Unit, dated Dec 1968

Topeka, Kansas, Design Memorandum No. 3, Waterworks Unit, dated July, 1957.

Engineering Technical Letter (ETL) 1110-2-307, Flotation Stability Criteria for Concrete Hydraulic Structures, Department of the Army, dated August 1987

CHAPTER A-4

EXHIBITS

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Exhibit A-4.1
Topeka Uplift

Assumptions:
Utilities are on landside of levee units
All lines shall be lowered 2 feet to alleviate uplift concern
Manholes shall be replaced with new
Blanket thicknesses are assumed to be 2 ft.
gravity lines assumed to have 2 ft of cover unless stated otherwise
Pressure lines assumed to have 3 ft of cover unless stated otherwise
Pipes not found on drawings are assumed to be 300 ft in length

Auburndale

Utility No.	line size	Material	Type	Length (ft)	Headwalls	depth of cover (ft)
4	18	cmp	gravity	300		2
6	18	cmp	gravity	300		2
N. Topeka						
7	24	cmp	gravity	300		4
8	12	steel	pressure	300		2
S. Topeka						
22	15x24	rcb	pressure	40	2	3
24	27x43	rcb	gravity	200	2	2
25	8	pvc	gravity	300		2
Waterworks						
27	10	cmp	gravity	300		2
28	8	cmp	gravity	300		2

Exhibit A-4.2
Topeka Utility Levee Crossings

Station	line size	Material	Type	Length (ft)	Headwalls	depth of cover (ft)
N. Topeka Unit						
UL 2, 9+35	24	cmp	gravity	100	2	6
UL 3, 275+50	21	cmp	gravity	50	2	20
Oakland Unit						
UL 7, 516+85	6	ci	pressure	86		12
Soldier Creek						
UL 8, 114+60	4	steel	pressure	400		4
Water Works						
UL 10, 14+90	2300 V		Power	86		12
UL 11, 33+75	18	CIP	Water	400		24
UL 12, 35+90	18	CIP	Water	350		24

Exhibit A-4.3
Topeka Utility Crossing Guidance

LEVEE AND FLOODWALL GRAVITY AND UTILITY PIPELINE GUIDANCE

PURPOSE

The purpose of this document is to provide specific guidance as to the disposition of existing utilities and drainage structures within the sections of levee and floodwall to be raised. This guidance will be used for the feasibility level of effort in order to develop reasonable costs associated with the modification of drainage structures and the relocation of utilities.

Uplift of utilities within the critical zone of the levee or floodwall will be addressed in accordance with COE criteria. Uplift is not addressed in this KCL guidance.

REFERENCES

	Local Protection – Web page guidance
	Local Protection - Guidebook on web page
EM 1110-2-1913	Design and Construction of Levees
EM 1110-2-2902	Conduits, Culverts, and Pipes
EM 1110-2-3102	General Principles of Pumping Station Design and Layout
EM 1110-2-3104	Structural and Architectural Design of Pumping Stations
EM 1110-2-3105	Mechanical and Electrical Design of Pumping Stations (Changes 1 of 2)

GRAVITY PIPELINES

Existing pipelines crossing the levee that do not meet current COE criteria shall be replaced with pipelines that are compliant. Existing pipelines that meet current COE criteria shall remain with the following exceptions:

> Any Corrugated Metal Pipe (CMP) with a diameter greater than 36” shall be replaced with a minimum diameter 48” Reinforced Concrete Pipe (RCP).
>
> Any pipe inadequate to handle the drainage shall be replaced with a minimum diameter 48” RCP.
>
> Any pipe known to have joints that are not watertight shall be replaced with a minimum diameter 48” RCP.

For new pipe installations, CMP will not be allowed.

Pipe strengths, unless otherwise known, will be assumed to be that required by Corps criteria at the time of their installation. Pipe condition shall be determined by field assessment.

GATEWELLS AND POSITIVE CLOSURES

In areas where levee raises are performed, positive closure will be provided for all drainage and utility lines crossing the levee. EM 1110-2-1913 states that gravity lines that penetrate the embankment or foundation of a levee must be provided with devices to assure positive closure. This criteria also states that gravity lines should be provided with flap-type or slide-type service gates on the riverside of the levee. Because the KS River and MO River are not fast rising rivers, a flap gate will not be recommended on existing outfalls where sluice gates are present but no flap gate. For new outfall structures, however, flap gates will generally we installed.

Emergency means of closure is suggested for gravity lines in addition to the positive closure device. Historically, a flap gate on the end of the pipe has acted as this second closure device. However, it is possible to use sandbags or concrete to fill a gatewell as a means of emergency closure during a flood situation, although this is not the recommended alternative.

All gatewells within the Kansas City Levee study area are considered confined spaces. OSHA regulations and Corp EM 385-1-1 require anyone entering a confined space to comply with specific confined space entry requirements. New or modified gatewells will be designed so that these confined space entry requirements can be met. For example, space will be provided above the gatewell opening so that a tripod can be set to facilitate non-entry rescue.

NON-GRAVITY PIPELINES CROSSING THROUGH OR UNDER LEVEES

It is preferable for all non-gravity pipes or conduits to cross over the levee rather than penetrate the embankment or foundation materials. This includes pipes carrying fiber optic, pressurized gas or pressurized liquid. Where raises are made to the levee, non-gravity pipelines should be relocated over the crest of the new levee raise. See detail “Typical Utility Crossing Levee Raise”.

Pressure pipe
All pipes allowed to penetrate the embankment or foundation of a levee must be provided with devices to assure positive closure. These valves shall be placed at various locations that can be closed rapidly to prevent gas or fluid from escaping within or beneath a levee should the pipe rupture within these areas. Provisions for closure of pressure pipes on the water side must also be provided to prevent backflow of floodwater into the protected area should the pipe rupture.

Casing Pipes and Conduits Crossing Through or Under Levees (Telecommunications)

It is preferred that conduits or casing pipes cross up and over the levee. However, where it is not possible to go over the levee, casing pipes or conduits must be installed in accordance with COE criteria. This criteria states that the conduit crossing through or under a levee must end in an encasement to prevent a preferred seepage path (both external and internal to the conduit). EM 1110-2-1913.

ABANDONED PIPELINES

Pipelines which are currently abandoned and grouted in accordance with COE criteria under or through the levee will not be disturbed. Pipes that have been abandoned and do not meet criteria or it is unknown if they meet criteria shall be removed or filled with grout. Pipelines that are currently active but are to be abandoned as part of this project will be removed or grouted full.

Removal

For feasibility purposes only, the following guidance is used in determining if an abandoned pipeline will be removed or abandoned in-place in accordance with Corps criteria.

Where levee heights are less than 10 feet and when an abandoned utility is buried less than 5 feet below the base of the levee, the abandoned utility crossing under the levee should be removed unless special circumstances warrant a different approach.

Exploration Trench

For cost estimating purposes during feasibility, all known pipes are assumed to be located as shown on maps and plans or as located in the field during feasibility site visits.

No exploration trenches will be specified during feasibility. However, it is noted that during PED phase, it may be determined that exploration trenches will be needed during construction in order to find some utilities or to verify that some utilities do not exist as shown on the drawings.

Grouting Abandoned Pipelines

In accordance with Local Protection guidance, if removal of piping system is not feasible, (i.e. line is too deep for removal) the pipes should be filled with a grout based substance, cement-bentonite, or flowable fill. The grout or flowable fill mix should be approved by the Corps of Engineers. The grout shall be fluid enough, and pumped in the up-slope direction so that the pipe will be completely filled leaving no voids. Points of access need to be made into the pipe at sufficient intervals to accomplish the grouting. See detail "Typical Utility Abandonment – Left in Place" for additional details regarding abandoning a utility in place.

OTHER CONSIDERATIONS

Other considerations will be given to whether a pipe crosses over or under levee on a case by case basis when HTRW concerns or real estate issues exist. HTRW concerns exist in various locations along the Kansas City Seven Levee system. When it is desirable to not disturb the existing ground due to HTRW concerns, the final recommendation for relocating an existing utility will weigh the risks involved with disturbing the ground against leaving an existing utility in place. When real estate issues exist, the final recommendation will consider how real estate is affected.

SUMMARY OF RECOMMENDATIONS

For sections of levee or floodwall to be raised or modified, current Corps requirements will be extended to all components of that levee section, including any pipes and closure structures therein. When it is not practical to meet Corps requirement, each utility will be evaluated on a case-by-case basis.

EXHIBIT A-4.4

UPLIFT SUMMARY

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EXHIBIT A-4.4 UPLIFT SUMMARY

	Item No.	uplift calc #	Sta (from hdr)	flow type	conduit type	conduit size (in)	function	depth below flood protection	source	note	findings
Auburndale											
	1		1 - 10						hdr spreadsheet misc structures.xls	possible exp for water treatment, not enough information to asses	need to investigate during design
	2		15.96	force		6	water		hdr spreadsheet misc structures.xls		Abandoned/grouted in place
	3		23 - 36				water		hdr spreadsheet misc structures.xls		need to investigate during design
	4	1	27.70		cmp	18	storm		hdr spreadsheet misc structures.xls	No depth provided assumed 2.5' cover for drainage	uplift ng
	5	2	28.30			3 - 42"			hdr spreadsheet misc structures.xls	discharge pipes for ward martin	uplift ok
	6		31.00	gravity	cmp	18			hdr spreadsheet misc structures.xls		uplift ng based on calc #1
N Topeka	7	3	277.00	gravity	cmp	24	sand	4	hdr spreadsheet floodprotection.xls	field located, not on drawings, geot info 50' away	uplift ng
	8	4	295.00	pressure	steel	12	sand plant suction	2	hdr spreadsheet floodprotection.xls	field located, not on drawings	uplift ng
	9		sta 12+50	pressure		16	water		hdr spreadsheet misc structures.xls	not on drawings, hdr notes	need to investigate during design
	10		sta 82+50	pressure	dip	18	water		hdr spreadsheet misc structures.xls	not on drawings, hdr notes	need to investigate during design
Oakland											
	11		sta 168		steel pipe	6	magnolia steel pipe		hdr spreadsheet oakfloodprotection.xls		need to investigate during design
	12		sta 185+65	pressure		6			hdr spreadsheet oakfloodprotection.xls		need to investigate during design
	13		sta 300+08	pressure		6	magnolia steel pipe		hdr spreadsheet oakfloodprotection.xls		need to investigate during design
Soldier Creek											
	14		sta 294	pressure	dip	12	water	6.5	hdr spreadsheet soldier floodprotection.xls	no geotechnical parameters provided for uplift	uplift ok based on calc #9
	15	10	sta 317+33	gravity	cip	6	sanitary	2.2	hdr spreadsheet soldier floodprotection.xls	no geotechnical parameters provided, assumed Ho at top of levee, blanket, bedrock assumed	uplift ok
	16	9	sta 356	pressure	dip	12	water	5.3	hdr spreadsheet soldier floodprotection.xls	no geotechnical parameters provided, assumed Ho at top of levee, blanket, bedrock assumed	uplift ok
	17		sta 410	gravity	cmp	12	storm	7	hdr spreadsheet soldier floodprotection.xls	no geotechnical parameters provided for uplift	uplift ok based on calc #9
	18	11	sta 8+85	gravity	cmp	30	storm	8.9	hdr spreadsheet soldier floodprotection.xls	Ho assumed at levee top	need to investigate during design
	19		sta 10+33	gravity	cmp	24	storm	7.8	hdr spreadsheet soldier floodprotection.xls	no geotechnical parameters provided for uplift	need to investigate based on uplift #11
	20	12	sta 6+50	gravity	cmp	12	storm	9	hdr spreadsheet soldier floodprotection.xls	no geotechnical parameters provided, assumed Ho at top of levee, blanket, bedrock assumed	need to investigate
	21		sta 14	gravity	cmp	18	storm	9	hdr spreadsheet soldier floodprotection.xls	no geotechnical parameters provided for uplift	need to investigate based on #12
S Topeka											
	22	5	sta 75+74	pressure	cip	15, 24	discharge piping	3	hdr spreadsheet south topeka floodprotection.xls		uplift ng
	23		sta 2+40	gravity	rcb	15x7			hdr spreadsheet south topeka floodprotection.xls	not found on drawings,	need to investigate
	24	7	sta 39+50	gravity	rcb	27 x 43"	storm		hdr spreadsheet south topeka misc structures.xls	300' ls	uplift ng
	25	8	sta 61+50	gravity	pvc	8"	sanitary		hdr spreadsheet south topeka misc structures.xls	250' ls, 2 ft cover assumed	uplift ng <.95
Waterworks											
	26	6	sta 17+49	gravity	steel pipe	20	storm	6.8	hdr spreadsheet waterworks floodprotection.xls		uplift ok
	27	13	sta 0+60		cmp	10	storm		hdr spreadsheet waterworks floodprotection.xls	not found on drawings assumed 2' of cover	uplift ng
	28		sta 1+20			8	storm		hdr spreadsheet waterworks floodprotection.xls	not found on drawings assumed 2' of cover	uplift ng based on calc # 13
	29		sta 11+20 to 13+00			36	interceptor		hdr spreadsheet waterworks floodprotection.xls	not found on drawings, assumed 2' of cover	uplift ng based on calc # 13

Assumptions:

EXHIBIT A-4.5

UPLIFT CALCULATION TABLES

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EXHIBIT A - 12.5 UPLIFT CALCULATIONS

Auburndale uplift #1	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
																							feet	
	500	500	9	9	9	40	16.85	0.84	800	0	0.99	405	100	424	929	929	7.69	7.69	7.69	0.85	424	424	0	0.000

Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineer, Scott Loehr, and is used to develop the Hydraulic Gradient at various distances from the toe. Verification of these numbers is done separately.

Lr 800 | Levee Width, L2 100

Levee Elev. 898.5

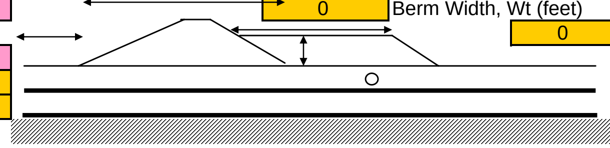
0 Berm Width, Wt (feet)

0 Berm Height, t (feet)

Ground Elev. 881.65

Blanket Base Elev. 872.65

Bedrock Elev. 832.65



3.5 Pipe Dept (ft)

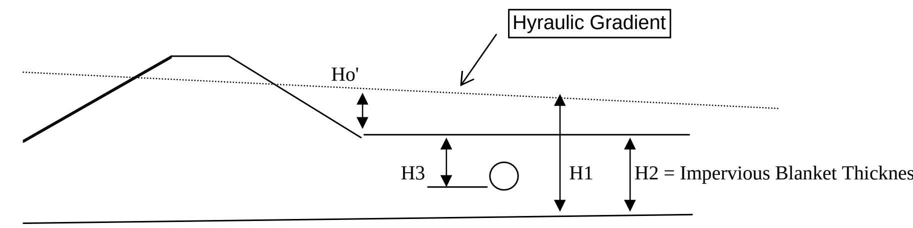
878.15 Pipe Elev.

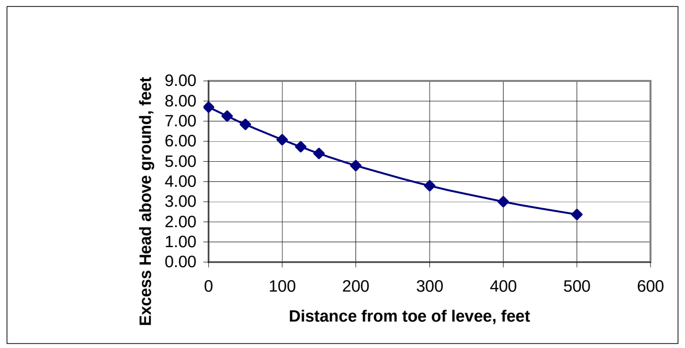
18 Pipe Diameter (in)

cl Soil Type

120 Soil Unit Weight, γsoil (pcf)

Ground Elev = Landside Average Ground Surface Used as Reference Datum





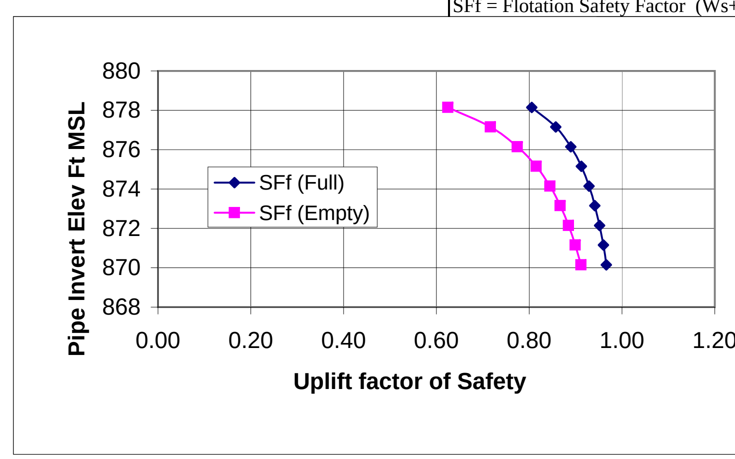
d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	7.69	878.15	872.65	881.65	3.5	9	16.7	405.1	20	110	360	608	0	0.81	0.62
25	7.25	878.15	872.65	881.65	3.5	9	16.3	394.4	20	110	360	592	0	0.83	0.64
50	6.84	878.15	872.65	881.65	3.5	9	15.8	384.3	20	110	360	576	0	0.85	0.66
100	6.08	878.15	872.65	881.65	3.5	9	15.1	365.9	20	110	360	549	0	0.89	0.69
125	5.73	878.15	872.65	881.65	3.5	9	14.7	357.4	20	110	360	536	0	0.91	0.71
150	5.40	878.15	872.65	881.65	3.5	9	14.4	349.5	20	110	360	524	0	0.93	0.72
200	4.80	878.15	872.65	881.65	3.5	9	13.8	334.9	20	110	360	502	0	0.97	0.76
300	3.79	878.15	872.65	881.65	3.5	9	12.8	310.4	20	110	360	466	0	1.05	0.81
400	3.00	878.15	872.65	881.65	3.5	9	12.0	291.1	20	110	360	437	0	1.12	0.87
500	2.37	878.15	872.65	881.65	3.5	9	11.4	275.8	20	110	360	414	0	1.18	0.92

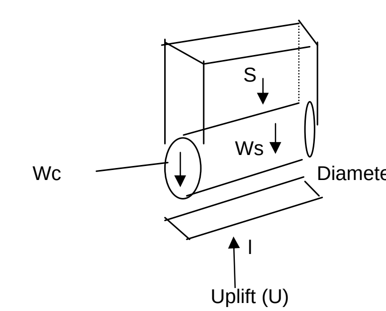
Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 881.65		Distance from toe (ft)		0		
						Uplift Safety Extreme Case (1.1 REQ'D)	
		Ws = 20		Wc = 110 Full			
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	881.65	0		20	110		
3.5	878.15	360	608	20	110	0.81	0.62
4.5	877.15	540	781	20	110	0.86	0.72
5.5	876.15	720	955	20	110	0.89	0.77
6.5	875.15	900	1128	20	110	0.91	0.81
7.5	874.15	1080	1302	20	110	0.93	0.84
8.5	873.15	1260	1476	20	110	0.94	0.87
9.5	872.15	1440	1649	20	110	0.95	0.89
10.5	871.15	1620	1823	20	110	0.96	0.90
11.5	870.15	1800	1996	20	110	0.97	0.91

Ws Calc
19.5
Wc Calc
110

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 19.5 lb per ft 18-inch Diameter cmp
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*(18/12)^2) *1*62.4 = 110 pl
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)'*1'*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)



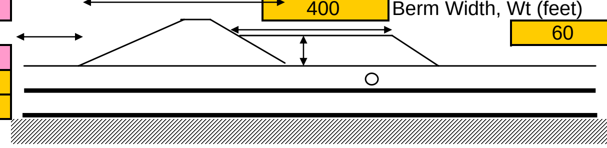


U = P3 * diameter * l
Using l = 1

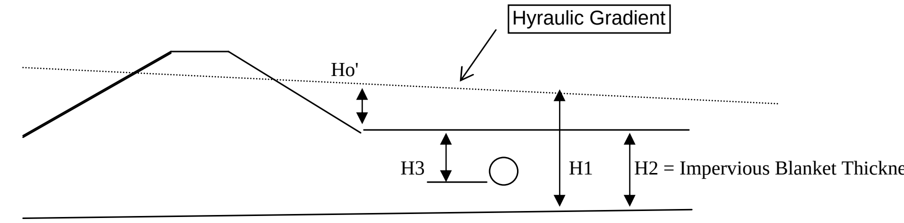
Auburndale	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
uplift #2																							feet	
	500	500	9	9	9	40	12	0.84	800	0	1.38	405	100	424	929	929	5.48	5.48	5.48	0.61	424	424	0	0.000

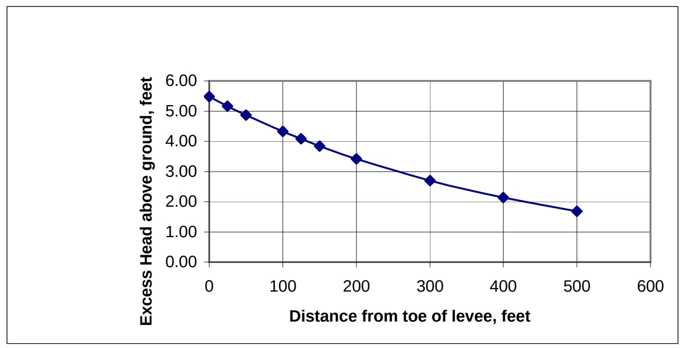
Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineer, Scott Loehr, and is used to develop the Hydraulic Gradient at various distances from the toe. Verification of these numbers is done separately.

Lr	800	
Levee Width, L2	100	
Levee Elev.	896.75	
Berm Width, Wt (feet)	400	
Berm Height, t (feet)	60	
Ground Elev.	882.5	
Blanket Base Elev.	873.5	
Bedrock Elev.	833.5	
Pipe Dept (ft)	17.5	
Pipe Elev.	865	
Pipe Diameter (in)	42	
Soil Type	cl	
Soil Unit Weight, γsoil (pcf)	120	



Ground Elev = Landside Average Ground Surface Used as Reference Datum





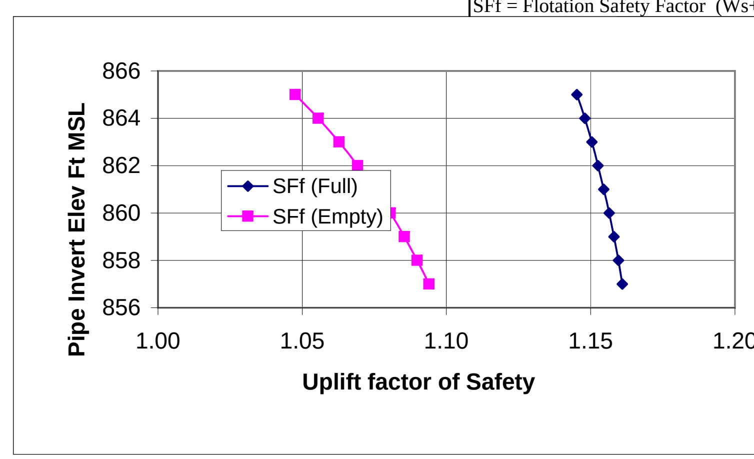
d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 42 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	5.48	865	873.5	882.5	17.5	9	14.5	1756.6	561	600	5880	6148	0	1.15	1.05
25	5.16	865	873.5	882.5	17.5	9	14.2	1718.6	561	600	5880	6015	0	1.17	1.07
50	4.87	865	873.5	882.5	17.5	9	13.9	1682.7	561	600	5880	5890	0	1.20	1.09
100	4.33	865	873.5	882.5	17.5	9	13.3	1617.1	561	600	5880	5660	0	1.24	1.14
125	4.08	865	873.5	882.5	17.5	9	13.1	1587.0	561	600	5880	5555	0	1.27	1.16
150	3.85	865	873.5	882.5	17.5	9	12.8	1558.7	561	600	5880	5455	0	1.29	1.18
200	3.42	865	873.5	882.5	17.5	9	12.4	1506.8	561	600	5880	5274	0	1.34	1.22
300	2.70	865	873.5	882.5	17.5	9	11.7	1419.7	561	600	5880	4969	0	1.42	1.30
400	2.13	865	873.5	882.5	17.5	9	11.1	1350.9	561	600	5880	4728	0	1.49	1.36
500	1.69	865	873.5	882.5	17.5	9	10.7	1296.5	561	600	5880	4538	0	1.55	1.42

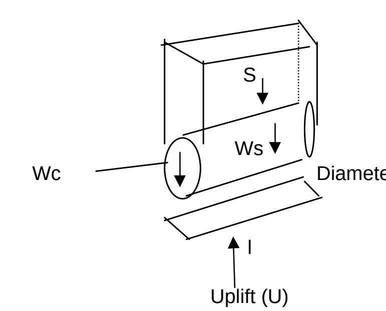
Ws Calc
561
Wc Calc
600

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 500 lb per ft 42-inch Diameter rcp
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*(42/12)^2) *1*62.4 = 600 pl
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 882.5	Distance from toe (ft)		0			
		Ws = 561		Wc = 600 Full		Uplift Safety Extreme Case (1.1 REQ'D)	
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	882.5	0		561	600		
17.5	865	5880	6148	561	600	1.15	1.05
18.5	864	6300	6500	561	600	1.15	1.06
19.5	863	6720	6851	561	600	1.15	1.06
20.5	862	7140	7202	561	600	1.15	1.07
21.5	861	7560	7554	561	600	1.15	1.08
22.5	860	7980	7905	561	600	1.16	1.08
23.5	859	8400	8256	561	600	1.16	1.09
24.5	858	8820	8607	561	600	1.16	1.09
25.5	857	9240	8959	561	600	1.16	1.09





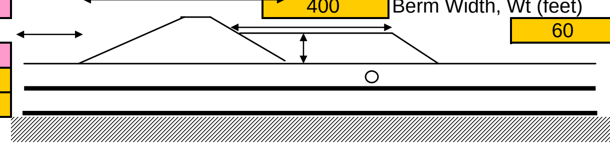
U = P3 * diameter * l
Using l = 1

n. topeka	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
uplift #3																							feet	
	300	300	14	14	14	56	11.6	0.84	350	0	1.77	300	58	485	843	843	6.68	6.68	6.68	0.48	485	485	0	0.000

Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineer, Scott Loehr, and is used to develop the Hydraulic Gradient at various distances from the toe. Verification of these numbers is done separately.

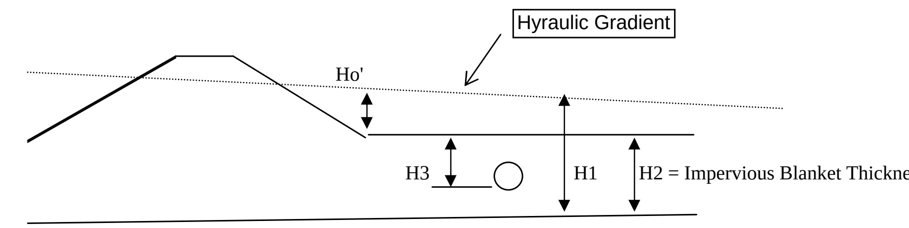
Lr Levee Width, L2
350 58

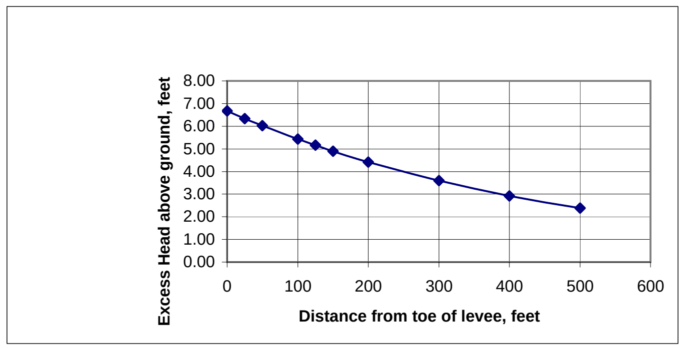
Levee Elev.	895.6
Ground Elev.	884
Blanket Base Elev.	870
Bedrock Elev.	814



400	Berm Width, Wt (feet)
60	Berm Height, t (feet)
4	Pipe Dept (ft)
880	Pipe Elev.
24	Pipe Diameter (in)
ml	Soil Type
120	Soil Unit Weight, γsoil (pcf)

Ground Elev = Landside Average Ground Surface Used as Reference Datum





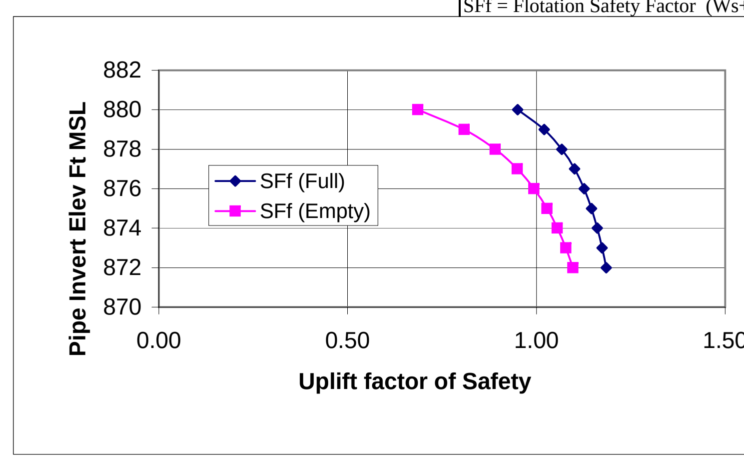
d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 24 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	6.68	880	870	884	4	14	20.7	368.6	26	195	480	737	0	0.95	0.69
25	6.34	880	870	884	4	14	20.3	362.6	26	195	480	725	0	0.97	0.70
50	6.02	880	870	884	4	14	20.0	357.0	26	195	480	714	0	0.98	0.71
100	5.43	880	870	884	4	14	19.4	346.4	26	195	480	693	0	1.01	0.73
125	5.16	880	870	884	4	14	19.2	341.6	26	195	480	683	0	1.03	0.74
150	4.90	880	870	884	4	14	18.9	337.0	26	195	480	674	0	1.04	0.75
200	4.42	880	870	884	4	14	18.4	328.4	26	195	480	657	0	1.07	0.77
300	3.60	880	870	884	4	14	17.6	313.7	26	195	480	627	0	1.12	0.81
400	2.93	880	870	884	4	14	16.9	301.8	26	195	480	604	0	1.16	0.84
500	2.38	880	870	884	4	14	16.4	292.1	26	195	480	584	0	1.20	0.87

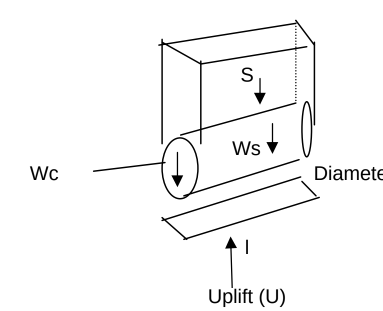
Ws Calc	25.5
Wc Calc	195

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 25 lb per ft 24-inch Diameter cmp
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*(24/12)^2) *1*62.4 = 195 pl
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)'*1'*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 884		Distance from toe (ft)	0		Uplift Safety Extreme Case (1.1 REQ'D)	
		Ws = 26		Wc = 195 Full			
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	884	0		26	195		
4	880	480	737	26	195	0.95	0.69
5	879	720	922	26	195	1.02	0.81
6	878	960	1106	26	195	1.07	0.89
7	877	1200	1290	26	195	1.10	0.95
8	876	1440	1474	26	195	1.13	0.99
9	875	1680	1659	26	195	1.15	1.03
10	874	1920	1843	26	195	1.16	1.06
11	873	2160	2027	26	195	1.17	1.08
12	872	2400	2212	26	195	1.18	1.10





U = P3 * diameter * l
Using l = 1

n. topeka	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
uplift #4																							feet	
	300	300	14	14	14	56	4	0.84	350	0	5.13	300	58	485	843	843	2.30	2.30	2.30	0.16	485	485	0	0.000

Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineer, Scott Loehr, and is used to develop the Hydraulic Gradient at various distances from the toe. Verification of these numbers is done separately.

Lr 350 — Levee Width, L2 58

Levee Elev. 894 — 400 Berm Width, Wt (feet)

60 Berm Height, t (feet)

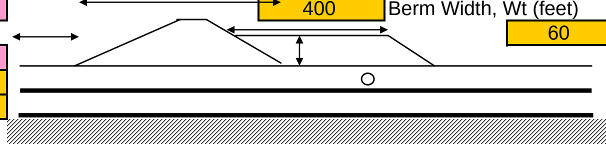
Ground Elev. 890 — 2 Pipe Dept (ft)

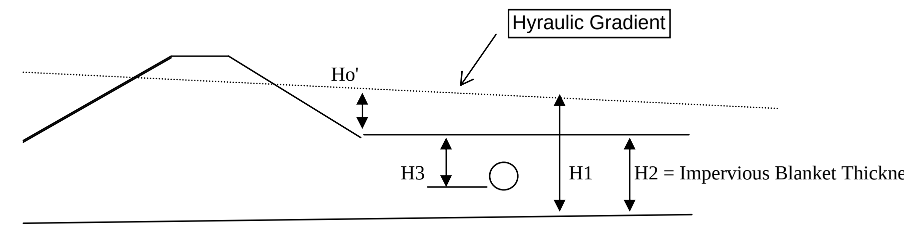
Blanket Base Elev. 876 — 888 Pipe Elev.

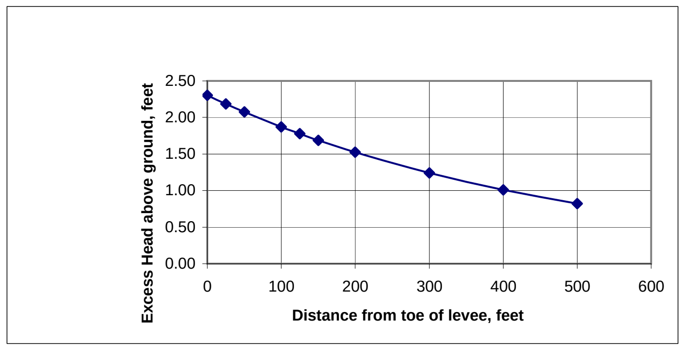
Bedrock Elev. 820 — 12 Pipe Diameter (in)

ml Soil Type

Ground Elev = Landside Average Ground Surface Used as Reference Datum — 120 Soil Unit Weight, γsoil (pcf)







d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 12 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	2.30	888	876	890	2	14	16.3	145.3	50	49	120	145	0	1.51	1.17
25	2.19	888	876	890	2	14	16.2	144.3	50	49	120	144	0	1.52	1.18
50	2.08	888	876	890	2	14	16.1	143.3	50	49	120	143	0	1.53	1.19
100	1.87	888	876	890	2	14	15.9	141.5	50	49	120	141	0	1.55	1.20
125	1.78	888	876	890	2	14	15.8	140.7	50	49	120	141	0	1.56	1.21
150	1.69	888	876	890	2	14	15.7	139.9	50	49	120	140	0	1.57	1.22
200	1.52	888	876	890	2	14	15.5	138.4	50	49	120	138	0	1.58	1.23
300	1.24	888	876	890	2	14	15.2	135.9	50	49	120	136	0	1.61	1.25
400	1.01	888	876	890	2	14	15.0	133.8	50	49	120	134	0	1.64	1.27
500	0.82	888	876	890	2	14	14.8	132.1	50	49	120	132	0	1.66	1.29

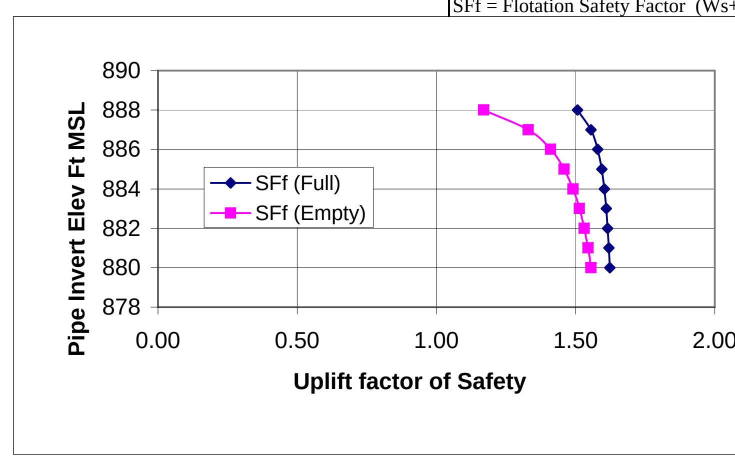
Ws Calc
50

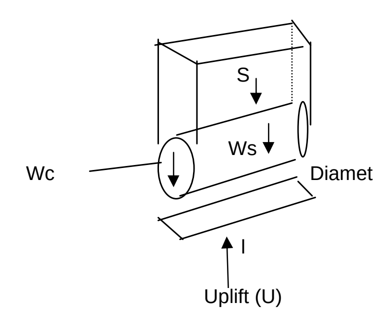
Wc Calc
49

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 50 lb per ft 12-inch Diameter st
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*(6/12)^2) *1*62.4 = 49 pl
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)'*1'*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 890		Distance from toe (ft)	0		Uplift Safety Extreme Case (1.1 REQ'D)	
		Ws = 50		Wc = 49 Full			
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	890	0		50	49		
2	888	120	145	50	49	1.51	1.17
3	887	240	218	50	49	1.56	1.33
4	886	360	291	50	49	1.58	1.41
5	885	480	363	50	49	1.59	1.46
6	884	600	436	50	49	1.60	1.49
7	883	720	509	50	49	1.61	1.51
8	882	840	581	50	49	1.62	1.53
9	881	960	654	50	49	1.62	1.54
10	880	1080	727	50	49	1.62	1.56





U = P3 * diameter * l
Using l = 1

s. topeka	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
uplift #5																							feet	
	300	300	15.5	15.5	15.5	80	12	0.84	20	0	1.50	20	17	610	647	843	8.68	8.68	8.68	0.56	610	610	0	0.000

Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineer, Scott Loehr, and is used to develop the Hydraulic Gradient at various distances from the toe. Verification of these numbers is done separately.

Lr Levee Width, L2

20 17

Levee Elev. 892 Berm Width, Wt (feet)

Berm Height, t (feet)

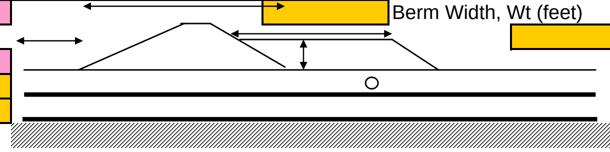
Ground Elev. 890 | 7 Pipe Dept (ft)

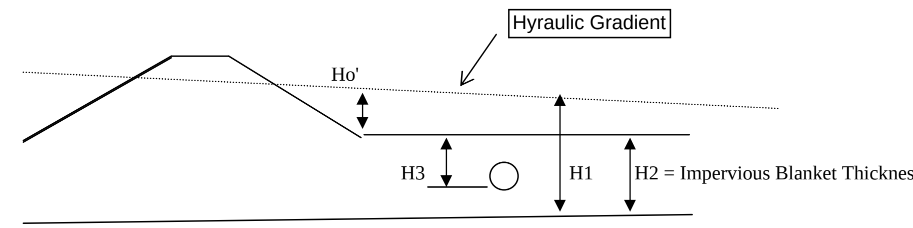
Blanket Base Elev. 874.5 | 883 Pipe Elev.

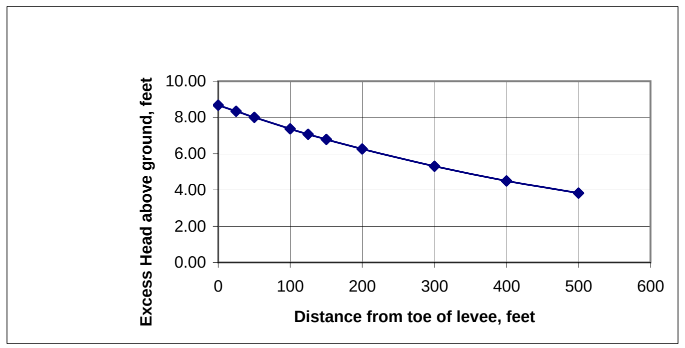
Bedrock Elev. 794.5 | 24 Pipe Diameter (in)

ml Soil Type

Ground Elev = Landside Average Ground Surface Used as Reference Datum | 120 Soil Unit Weight, γsoil (pcf)







d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 24 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	8.68	883	874.5	890	7	15.5	24.2	681.5	8	195	1200	1363	0	1.03	0.89
25	8.33	883	874.5	890	7	15.5	23.8	671.6	8	195	1200	1343	0	1.04	0.90
50	8.00	883	874.5	890	7	15.5	23.5	662.2	8	195	1200	1324	0	1.06	0.91
100	7.37	883	874.5	890	7	15.5	22.9	644.5	8	195	1200	1289	0	1.09	0.94
125	7.07	883	874.5	890	7	15.5	22.6	636.1	8	195	1200	1272	0	1.10	0.95
150	6.79	883	874.5	890	7	15.5	22.3	628.1	8	195	1200	1256	0	1.12	0.96
200	6.25	883	874.5	890	7	15.5	21.8	613.1	8	195	1200	1226	0	1.14	0.98
300	5.31	883	874.5	890	7	15.5	20.8	586.4	8	195	1200	1173	0	1.20	1.03
400	4.51	883	874.5	890	7	15.5	20.0	563.8	8	195	1200	1128	0	1.24	1.07
500	3.82	883	874.5	890	7	15.5	19.3	544.6	8	195	1200	1089	0	1.29	1.11

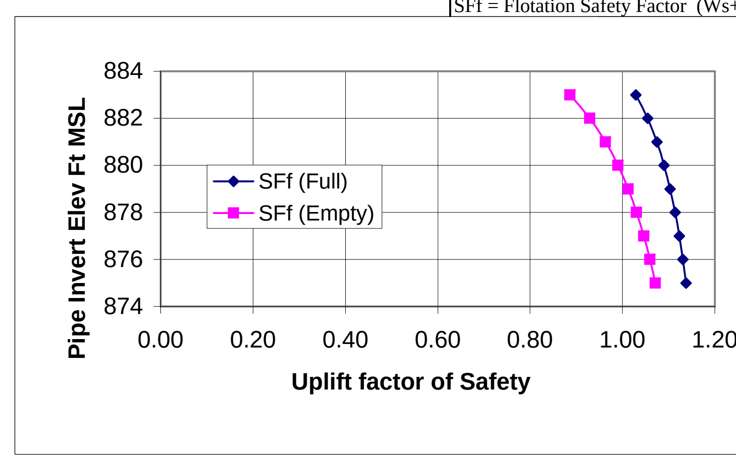
Ws Calc
7.68

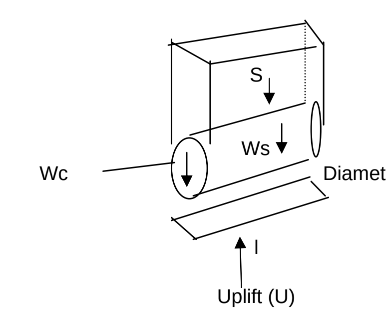
Wc Calc
195

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 7.7 lb per ft 15-inch Diameter cip
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*(24/12)^2) *1*62.4 = 195 pl
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 890		Distance from toe (ft)	0		Uplift Safety Extreme Case (1.1 REQ'D)	
		Ws = 8		Wc = 195 Full			
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	890	0		8	195		
7	883	1200	1363	8	195	1.03	0.89
8	882	1440	1558	8	195	1.05	0.93
9	881	1680	1752	8	195	1.07	0.96
10	880	1920	1947	8	195	1.09	0.99
11	879	2160	2142	8	195	1.10	1.01
12	878	2400	2336	8	195	1.11	1.03
13	877	2640	2531	8	195	1.12	1.05
14	876	2880	2726	8	195	1.13	1.06
15	875	3120	2921	8	195	1.14	1.07





U = P3 * diameter * l
Using l = 1

Waterworks uplift #6	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
																							feet	
	500	500	7.6	7.6	7.6	50	9.5	0.84	150	0	0.92	144	15	436	595	595	6.96	6.96	7.00	0.92	436	436	0	0.000

Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineer, Scott Loehr, and is used to develop the Hydraulic Gradient at various distances from the toe. Verification of these numbers is done separately.

Lr 150 | Levee Width, L2 15

Levee Elev. 896.5 | Berm Width, Wt (feet) 400

Berm Height, t (feet) 60

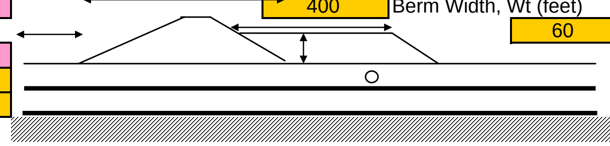
Ground Elev. 879 | Pipe Dept (ft) 6.8

Blanket Base Elev. 871.4 | Pipe Elev. 872.2

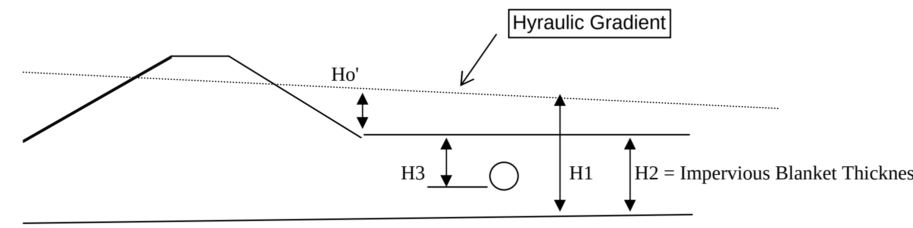
Bedrock Elev. 821.4 | Pipe Diameter (in) 20

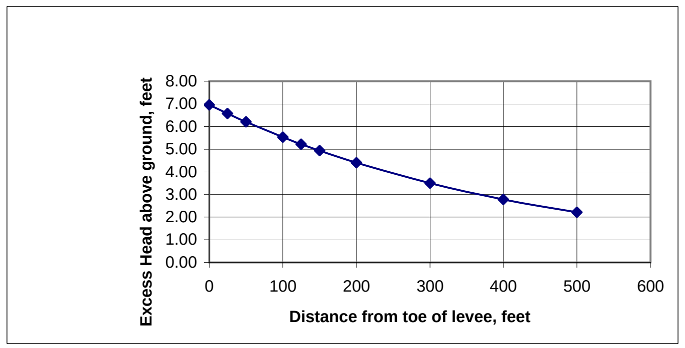
Soil Type cl

Soil Unit Weight, γsoil (pcf) 120



Ground Elev = Landside Average Ground Surface Used as Reference Datum





d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 20 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	6.96	872.2	871.4	879	6.8	7.6	14.6	813.0	78	136	1027	1355	0	0.92	0.82
25	6.57	872.2	871.4	879	6.8	7.6	14.2	791.3	78	136	1027	1319	0	0.94	0.84
50	6.21	872.2	871.4	879	6.8	7.6	13.8	770.9	78	136	1027	1285	0	0.97	0.86
100	5.53	872.2	871.4	879	6.8	7.6	13.1	733.3	78	136	1027	1222	0	1.02	0.90
125	5.23	872.2	871.4	879	6.8	7.6	12.8	716.1	78	136	1027	1194	0	1.04	0.93
150	4.93	872.2	871.4	879	6.8	7.6	12.5	699.8	78	136	1027	1166	0	1.06	0.95
200	4.40	872.2	871.4	879	6.8	7.6	12.0	670.0	78	136	1027	1117	0	1.11	0.99
300	3.50	872.2	871.4	879	6.8	7.6	11.1	619.6	78	136	1027	1033	0	1.20	1.07
400	2.78	872.2	871.4	879	6.8	7.6	10.4	579.6	78	136	1027	966	0	1.28	1.14
500	2.21	872.2	871.4	879	6.8	7.6	9.8	547.8	78	136	1027	913	0	1.36	1.21

Ws Calc: 78

Wc Calc: 136

See Sample Calculations for list of abbreviations and sample calculations

Ws = weight of structure per foot of length = 78 lb per ft 20-inch Diameter st

Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*(24/12)^2) *1*62.4 = 195 pl

S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil

P3 = H3*(H1/H2)*γwater

U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)'*1'*P3

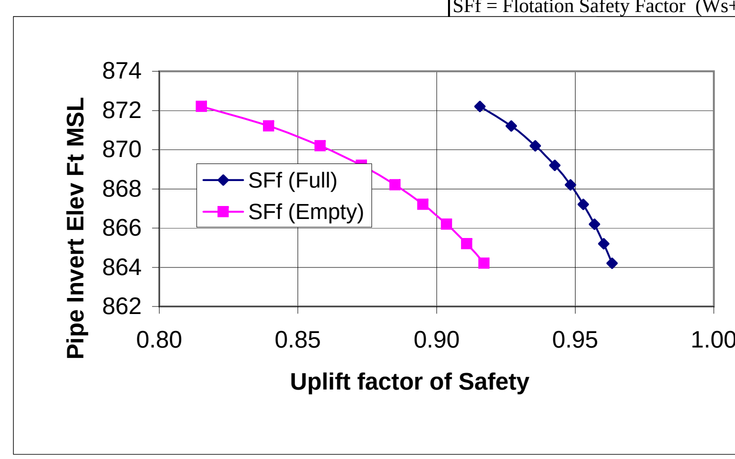
Wg = weight of surcharge water above top surface of structure control by gravity flow

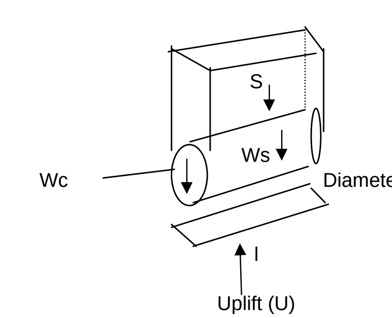
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

Use this table to determine the distance from toe at which the Safety Factor is me

Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 879		Distance from toe (ft)		0		
		Ws = 78		Wc = 136 Full		Uplift Safety Extreme Case (1.1 REQ'D)	
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	879	0		78	136		
6.8	872.2	1027	1355	78	136	0.92	0.82
7.8	871.2	1227	1554	78	136	0.93	0.84
8.8	870.2	1427	1753	78	136	0.94	0.86
9.8	869.2	1627	1953	78	136	0.94	0.87
10.8	868.2	1827	2152	78	136	0.95	0.89
11.8	867.2	2027	2351	78	136	0.95	0.90
12.8	866.2	2227	2551	78	136	0.96	0.90
13.8	865.2	2427	2750	78	136	0.96	0.91
14.8	864.2	2627	2949	78	136	0.96	0.92





U = P3 * diameter * l

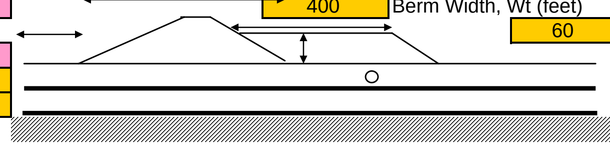
Using l = 1

s. Topeka uplift #7	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t feet	s
	400	400	11.3	11.3	11.3	80	11.7	0.84	10	0	0.94	10	80	601	691	692	10.17	10.17	10.17	0.90	601	601	0	0.000

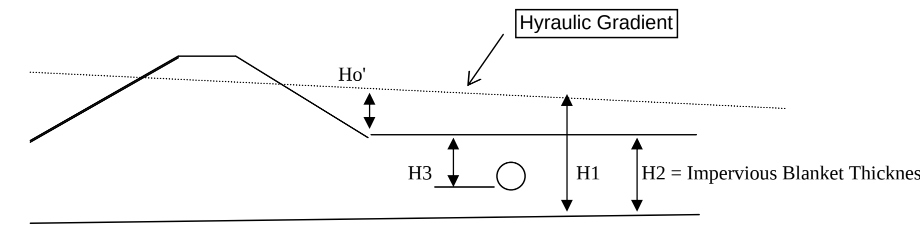
Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineer, Scott Loehr, and is used to develop the Hydraulic Gradient at various distances from the toe. Verification of these numbers is done separately.

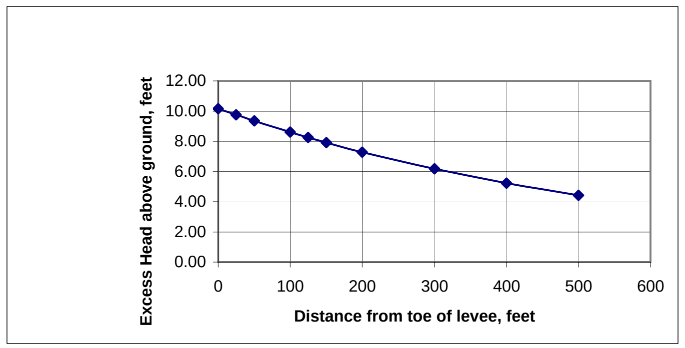
Lr Levee Width, L2

Lr	10	
Levee Width, L2	80	
Levee Elev.	893.8	
Berm Width, Wt (feet)	400	
Berm Height, t (feet)	60	
Ground Elev.	890	
Blanket Base Elev.	878.7	
Bedrock Elev.	798.7	
Pipe Dept (ft)	12.5	
Pipe Elev.	877.5	
Pipe Diameter (in)	38	
Soil Type	ml-cl	
Soil Unit Weight, γsoil (pcf)	120	



Ground Elev = Landside Average Ground Surface Used as Reference Datum





d=Landside — No Berm Calcs

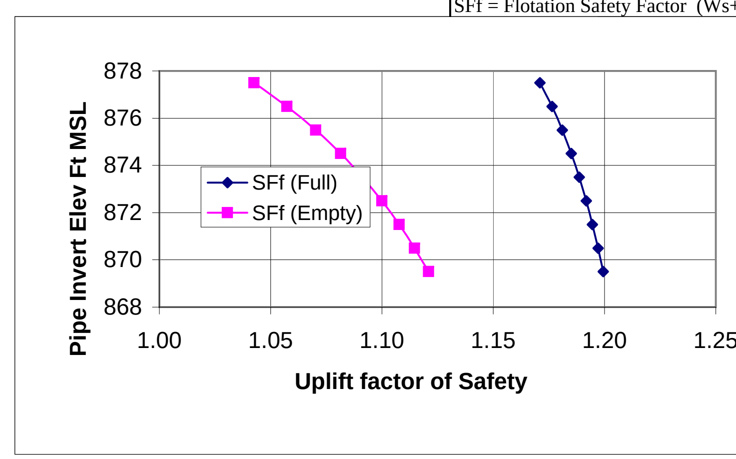
Distance from Toe Feet	Excess Head Feet, Ho'	Invert Elev of pipe 38 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	10.17	877.5	878.7	890	12.5	11.3	21.5	1481.8	435	491	3547	4692	0	0.95	0.85
25	9.75	877.5	878.7	890	12.5	11.3	21.1	1453.2	435	491	3547	4602	0	0.97	0.87
50	9.36	877.5	878.7	890	12.5	11.3	20.7	1425.8	435	491	3547	4515	0	0.99	0.88
100	8.61	877.5	878.7	890	12.5	11.3	19.9	1374.3	435	491	3547	4352	0	1.03	0.91
125	8.26	877.5	878.7	890	12.5	11.3	19.6	1350.1	435	491	3547	4275	0	1.05	0.93
150	7.92	877.5	878.7	890	12.5	11.3	19.2	1326.9	435	491	3547	4202	0	1.06	0.95
200	7.29	877.5	878.7	890	12.5	11.3	18.6	1283.2	435	491	3547	4064	0	1.10	0.98
300	6.17	877.5	878.7	890	12.5	11.3	17.5	1206.1	435	491	3547	3819	0	1.17	1.04
400	5.23	877.5	878.7	890	12.5	11.3	16.5	1140.8	435	491	3547	3613	0	1.24	1.10
500	4.43	877.5	878.7	890	12.5	11.3	15.7	1085.6	435	491	3547	3438	0	1.30	1.16

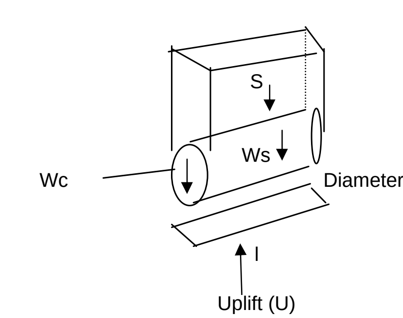
Ws Calc
435
Wc Calc
491

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 435 lb per ft38-inch equivalent Diameter rcp
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*(24/12)^2) *1*62.4 = 195 pl
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)'*1'*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 890		Distance from toe (ft)		300		
		Ws = 435		Wc = 491 Full		Uplift Safety Extreme Case **(1.1 REQ'D)**	
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	890	0		435	491		
12.5	877.5	3547	3819	435	491	1.17	1.04
13.5	876.5	3927	4125	435	491	1.18	1.06
14.5	875.5	4307	4431	435	491	1.18	1.07
15.5	874.5	4687	4736	435	491	1.19	1.08
16.5	873.5	5067	5042	435	491	1.19	1.09
17.5	872.5	5447	5347	435	491	1.19	1.10
18.5	871.5	5827	5653	435	491	1.19	1.11
19.5	870.5	6207	5958	435	491	1.20	1.11
20.5	869.5	6587	6264	435	491	1.20	1.12



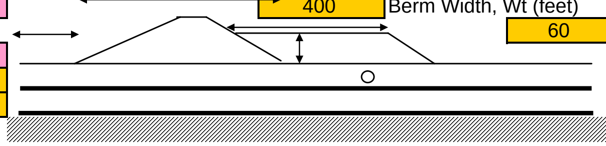


U = P3 * diameter * l
Using l = 1

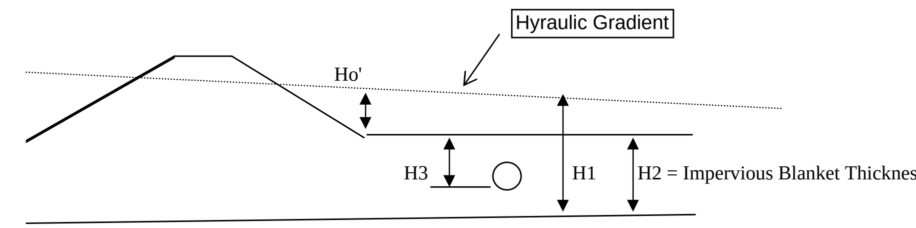
n. topeka	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
uplift #8																							feet	
	400	400	11.3	11.3	11.3	80	11.7	0.84	10	0	0.94	10	80	601	691	691	10.18	10.18	10.18	0.90	601	601	0	0.000

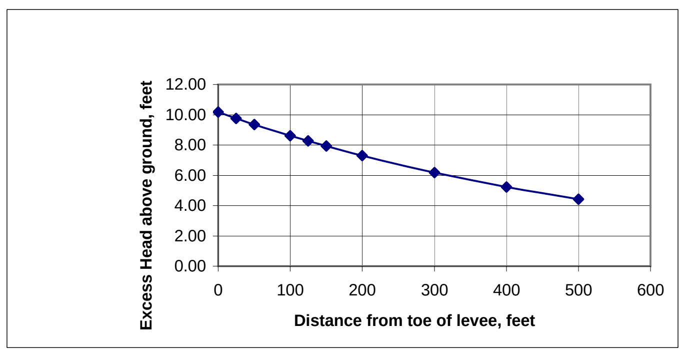
Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineer, Scott Loehr, and is used to develop the Hydraulic Gradient at various distances from the toe. Verification of these numbers is done separately.

Lr	10	
Levee Width, L2	80	
Levee Elev.	893.8	
Berm Width, Wt (feet)	400	
Berm Height, t (feet)	60	
Ground Elev.	892	
Pipe Dept (ft)	3	
Blanket Base Elev.	880.7	
Pipe Elev.	889	
Bedrock Elev.	800.7	
Pipe Diameter (in)	8	
Soil Type	ml-cl	
Soil Unit Weight, γsoil (pcf)	120	



Ground Elev = Landside Average Ground Surface Used as Reference Datum





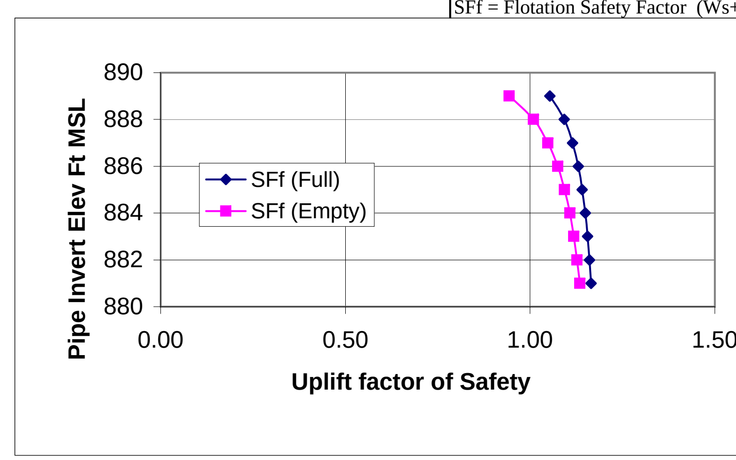
d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 8 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	10.18	889	880.7	892	3	11.3	21.5	355.8	1	22	187	237	0	0.88	0.79
25	9.76	889	880.7	892	3	11.3	21.1	348.9	1	22	187	233	0	0.90	0.81
50	9.36	889	880.7	892	3	11.3	20.7	342.3	1	22	187	228	0	0.92	0.82
100	8.62	889	880.7	892	3	11.3	19.9	330.0	1	22	187	220	0	0.95	0.85
125	8.27	889	880.7	892	3	11.3	19.6	324.2	1	22	187	216	0	0.97	0.87
150	7.93	889	880.7	892	3	11.3	19.2	318.6	1	22	187	212	0	0.99	0.88
200	7.30	889	880.7	892	3	11.3	18.6	308.1	1	22	187	205	0	1.02	0.91
300	6.18	889	880.7	892	3	11.3	17.5	289.6	1	22	187	193	0	1.09	0.97
400	5.23	889	880.7	892	3	11.3	16.5	273.9	1	22	187	183	0	1.15	1.03
500	4.43	889	880.7	892	3	11.3	15.7	260.6	1	22	187	174	0	1.21	1.08

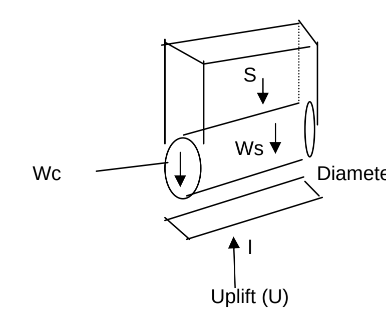
Ws Calc	Wc Calc
1	22

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 1 lb per ft 8-inch Diameter pvc
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*(8/12)^2) *1*62.4 = 22 pl
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)'*1'*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 892	Distance from toe (ft)		250			
		Ws = 1		Wc = 22 Full		Uplift Safety Extreme Case (1.1 REQ'D)	
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	892	0		1	22		
3	889	187	199	1	22	1.05	0.94
4	888	267	265	1	22	1.09	1.01
5	887	347	332	1	22	1.11	1.05
6	886	427	398	1	22	1.13	1.07
7	885	507	464	1	22	1.14	1.09
8	884	587	531	1	22	1.15	1.11
9	883	667	597	1	22	1.16	1.12
10	882	747	663	1	22	1.16	1.13
11	881	827	730	1	22	1.16	1.13





U = P3 * diameter * l
Using l = 1

Soldier Crk	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
uplift #9																							feet	
		300	15	15	15	40	1.2	0.84	50	0	#DIV/0!	#DIV/0!	125	424	#DIV/0!	#DIV/0!	1.20	#DIV/0!	#DIV/0!	#DIV/0!	0	424	0	0.000

Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineer, Scott Loehr, and is used to develop the Hydraulic Gradient at various distances from the toe. Verification of these numbers is done separately.

Lr 50 | Levee Width, L2 125

Levee Elev. 889.7 | Berm Width, Wt (feet) 400

Berm Height, t (feet) 60

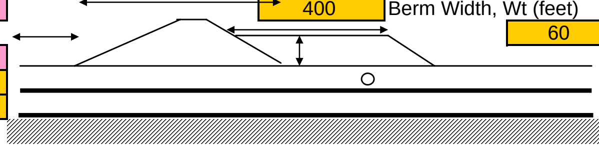
Ground Elev. 888.5 | Pipe Dept (ft) 5.3

Blanket Base Elev. 873.5 | Pipe Elev. 883.2

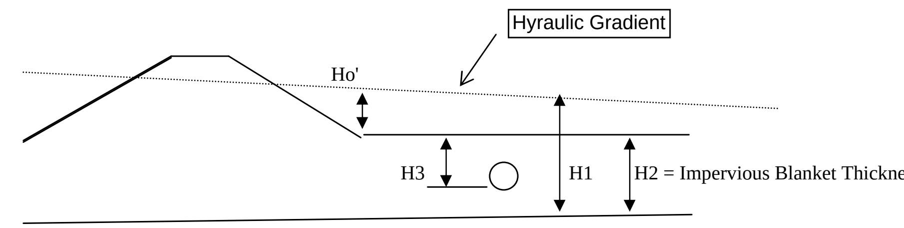
Bedrock Elev. 833.5 | Pipe Diameter (in) 12

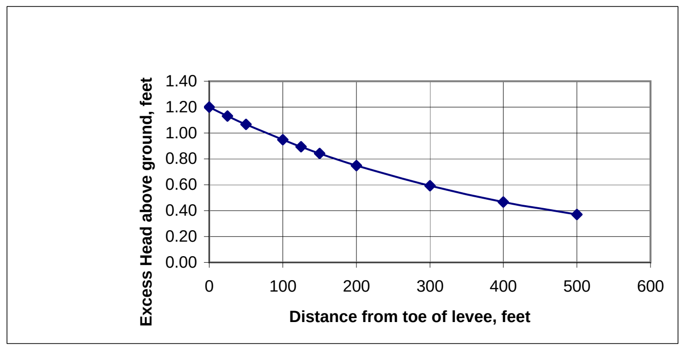
Soil Type ml

Soil Unit Weight, γsoil (pcf) 120



Ground Elev = Landside Average Ground Surface Used as Reference Datum





d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 12 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	1.20	883.2	873.5	888.5	5.3	15	16.2	357.2	3	49	516	357	0	1.59	1.45
25	1.13	883.2	873.5	888.5	5.3	15	16.1	355.7	3	49	516	356	0	1.60	1.46
50	1.07	883.2	873.5	888.5	5.3	15	16.1	354.2	3	49	516	354	0	1.60	1.46
100	0.95	883.2	873.5	888.5	5.3	15	15.9	351.6	3	49	516	352	0	1.61	1.47
125	0.89	883.2	873.5	888.5	5.3	15	15.9	350.4	3	49	516	350	0	1.62	1.48
150	0.84	883.2	873.5	888.5	5.3	15	15.8	349.3	3	49	516	349	0	1.62	1.48
200	0.75	883.2	873.5	888.5	5.3	15	15.7	347.2	3	49	516	347	0	1.63	1.49
300	0.59	883.2	873.5	888.5	5.3	15	15.6	343.8	3	49	516	344	0	1.65	1.51
400	0.47	883.2	873.5	888.5	5.3	15	15.5	341.0	3	49	516	341	0	1.66	1.52
500	0.37	883.2	873.5	888.5	5.3	15	15.4	338.9	3	49	516	339	0	1.67	1.53

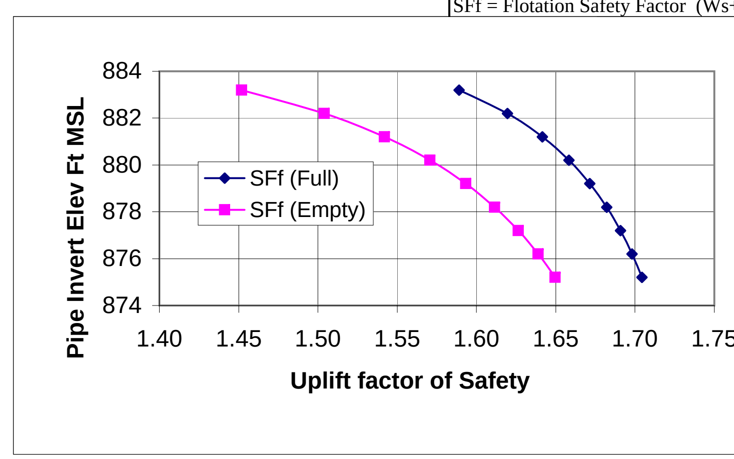
Ws Calc
2.57

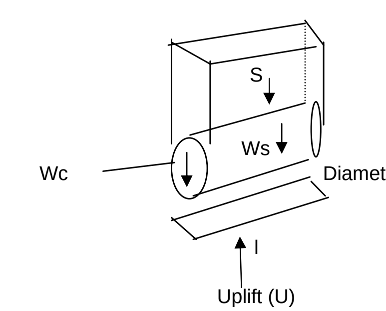
Wc Calc
49

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 2.57 lb per ft 12-inch Diameter dip
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*(12/12)^2) *1*62.4 = 49 pl
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)*1*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 888.5		Distance from toe (ft)	0			
						Uplift Safety Extreme Case (1.1 REQ'D)	
		Ws = 3		Wc = 49 Full			
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	888.5	0		3	49		
5.3	883.2	516	357	3	49	1.59	1.45
6.3	882.2	636	425	3	49	1.62	1.50
7.3	881.2	756	492	3	49	1.64	1.54
8.3	880.2	876	559	3	49	1.66	1.57
9.3	879.2	996	627	3	49	1.67	1.59
10.3	878.2	1116	694	3	49	1.68	1.61
11.3	877.2	1236	762	3	49	1.69	1.63
12.3	876.2	1356	829	3	49	1.70	1.64
13.3	875.2	1476	896	3	49	1.70	1.65



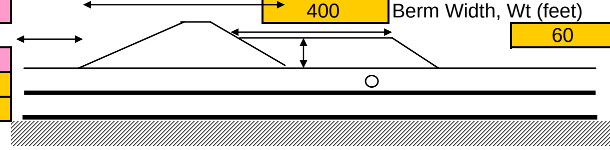


U = P3 * diameter * l
Using l = 1

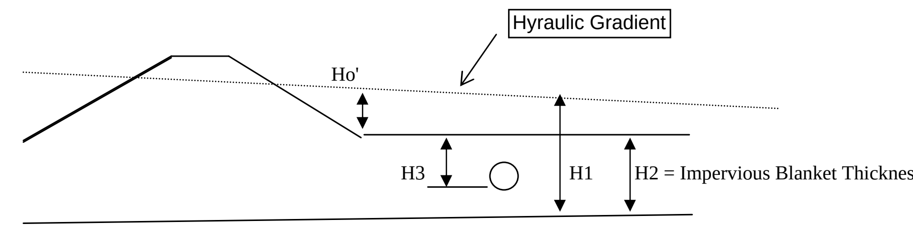
Soldier Crk	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
uplift #10																							feet	
	300	300	15	15	15	65	7.2	0.84	50	0	2.32	50	125	541	716	716	7.20	5.44	5.44	0.36	541	541	0	0.000

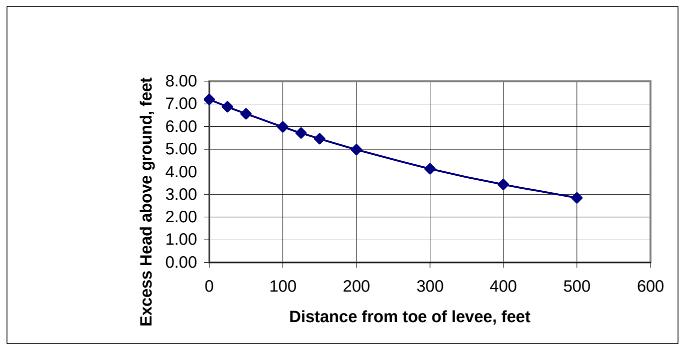
Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineer, Scott Loehr, and is used to develop the Hydraulic Gradient at various distances from the toe. Verification of these numbers is done separately.

Lr	50	
Levee Width, L2	125	
Levee Elev.	889.2	
Berm Width, Wt (feet)	400	
Berm Height, t (feet)	60	
Ground Elev.	882	
Blanket Base Elev.	867	
Bedrock Elev.	802	
Pipe Dept (ft)	-5.2	
Pipe Elev.	887.2	
Pipe Diameter (in)	6	
Soil Type	ml	
Soil Unit Weight, γsoil (pcf)	120	



Ground Elev = Landside Average Ground Surface Used as Reference Datum





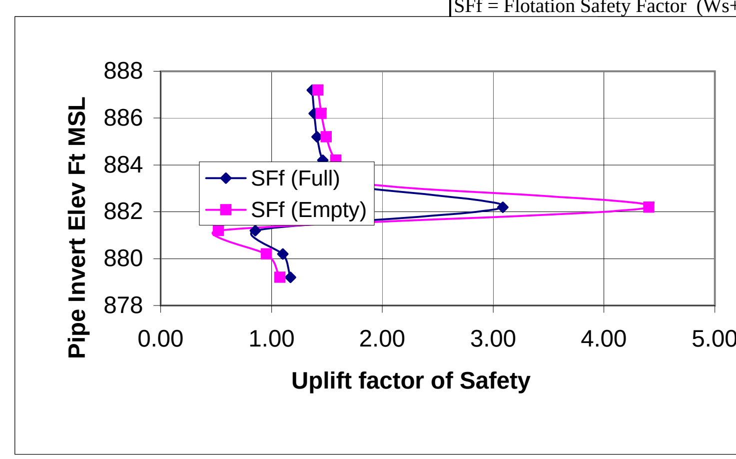
d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 6 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	7.20	887.2	867	882	-5.2	15	22.2	-480.2	1	12	-342	-240	0	1.37	1.42
25	6.87	887.2	867	882	-5.2	15	21.9	-473.2	1	12	-342	-237	0	1.39	1.44
50	6.56	887.2	867	882	-5.2	15	21.6	-466.5	1	12	-342	-233	0	1.41	1.46
100	5.98	887.2	867	882	-5.2	15	21.0	-453.9	1	12	-342	-227	0	1.45	1.50
125	5.71	887.2	867	882	-5.2	15	20.7	-448.1	1	12	-342	-224	0	1.47	1.52
150	5.46	887.2	867	882	-5.2	15	20.5	-442.5	1	12	-342	-221	0	1.48	1.54
200	4.97	887.2	867	882	-5.2	15	20.0	-432.1	1	12	-342	-216	0	1.52	1.58
300	4.13	887.2	867	882	-5.2	15	19.1	-413.9	1	12	-342	-207	0	1.59	1.65
400	3.44	887.2	867	882	-5.2	15	18.4	-398.8	1	12	-342	-199	0	1.65	1.71
500	2.86	887.2	867	882	-5.2	15	17.9	-386.3	1	12	-342	-193	0	1.70	1.76

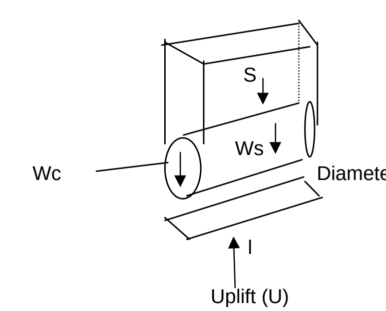
Ws Calc	1.3
Wc Calc	12.2

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 1.3 lb per ft 6-inch Diameter dip
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*(12.25/12)^2) *1*62.4 = 12.2 pl
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)'*1'*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 882	Distance from toe (ft)		0		Uplift Safety Extreme Case (1.1 REQ'D)	
		Ws = 1		Wc = 12 Full			
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	882	0		1	12		
-5.2	887.2	-342	-240	1	12	1.37	1.42
-4.2	886.2	-282	-194	1	12	1.38	1.45
-3.2	885.2	-222	-148	1	12	1.41	1.49
-2.2	884.2	-162	-102	1	12	1.46	1.58
-1.2	883.2	-102	-55	1	12	1.60	1.82
-0.2	882.2	-42	-9	1	12	3.09	4.41
0.8	881.2	18	37	1	12	0.85	0.52
1.8	880.2	78	83	1	12	1.10	0.95
2.8	879.2	138	129	1	12	1.17	1.08





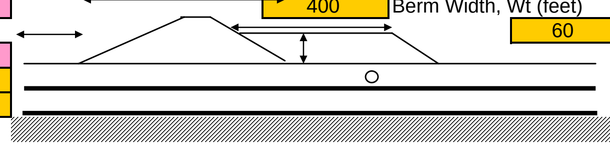
U = P3 * diameter * l
Using l = 1

Soldier Crk	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
uplift #11																							feet	
	400	400	11.3	11.3	11.3	65	19.9	0.84	10	0	0.56	10	80	542	632	632	17.07	17.07	17.07	1.51	542	542	0	0.000

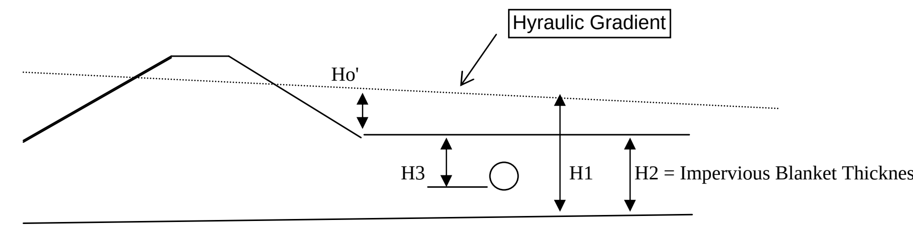
Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineer, Scott Loehr, and is used to develop the Hydraulic Gradient at various distances from the toe. Verification of these numbers is done separately.

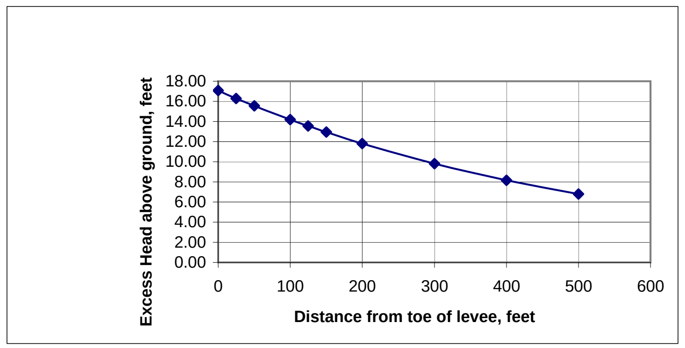
Lr Levee Width, L2

Lr	10
Levee Width, L2	80
Levee Elev.	919
Berm Width, Wt (feet)	400
Berm Height, t (feet)	60
Ground Elev.	899.1
Blanket Base Elev.	887.8
Bedrock Elev.	822.8
Pipe Dept (ft)	8.9
Pipe Elev.	890.2
Pipe Diameter (in)	30
Soil Type	ml-cl
Soil Unit Weight, γsoil (pcf)	120



Ground Elev = Landside Average Ground Surface Used as Reference Datum





d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 30 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	17.07	890.2	887.8	899.1	8.9	11.3	28.4	1394.1	31	306	1920	3485	0	0.65	0.56
25	16.30	890.2	887.8	899.1	8.9	11.3	27.6	1356.3	31	306	1920	3391	0	0.67	0.58
50	15.56	890.2	887.8	899.1	8.9	11.3	26.9	1320.2	31	306	1920	3301	0	0.68	0.59
100	14.19	890.2	887.8	899.1	8.9	11.3	25.5	1252.8	31	306	1920	3132	0	0.72	0.62
125	13.55	890.2	887.8	899.1	8.9	11.3	24.9	1221.4	31	306	1920	3053	0	0.74	0.64
150	12.94	890.2	887.8	899.1	8.9	11.3	24.2	1191.4	31	306	1920	2978	0	0.76	0.66
200	11.80	890.2	887.8	899.1	8.9	11.3	23.1	1135.3	31	306	1920	2838	0	0.80	0.69
300	9.81	890.2	887.8	899.1	8.9	11.3	21.1	1037.6	31	306	1920	2594	0	0.87	0.75
400	8.16	890.2	887.8	899.1	8.9	11.3	19.5	956.4	31	306	1920	2391	0	0.94	0.82
500	6.78	890.2	887.8	899.1	8.9	11.3	18.1	888.8	31	306	1920	2222	0	1.02	0.88

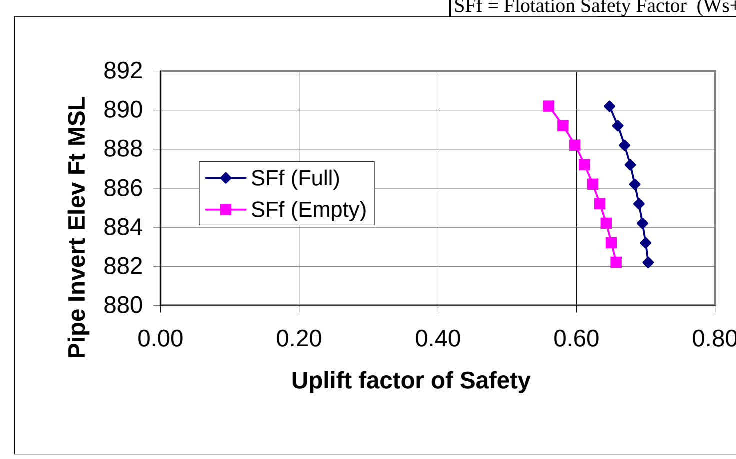
Ws Calc
31

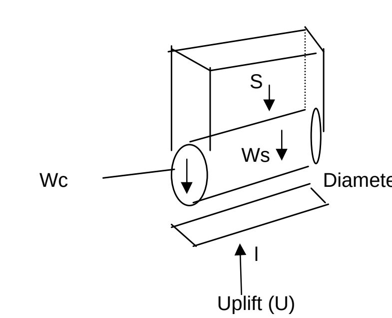
Wc Calc
306

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 31 lb per ft 30-inch Diameter cmp
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*(30/12)^2) *1*62.4 = 306 pl
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)'*1'*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 899.1		Distance from toe (ft)	0		Uplift Safety Extreme Case (1.1 REQ'D)	
		Ws = 31		Wc = 306 Full			
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	899.1	0		31	306		
8.9	890.2	1920	3485	31	306	0.65	0.56
9.9	889.2	2220	3877	31	306	0.66	0.58
10.9	888.2	2520	4269	31	306	0.67	0.60
11.9	887.2	2820	4660	31	306	0.68	0.61
12.9	886.2	3120	5052	31	306	0.68	0.62
13.9	885.2	3420	5443	31	306	0.69	0.63
14.9	884.2	3720	5835	31	306	0.70	0.64
15.9	883.2	4020	6227	31	306	0.70	0.65
16.9	882.2	4320	6618	31	306	0.70	0.66



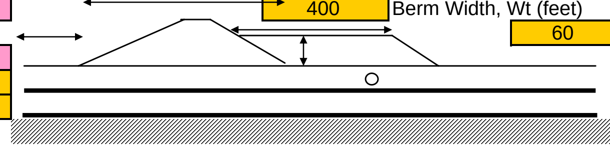


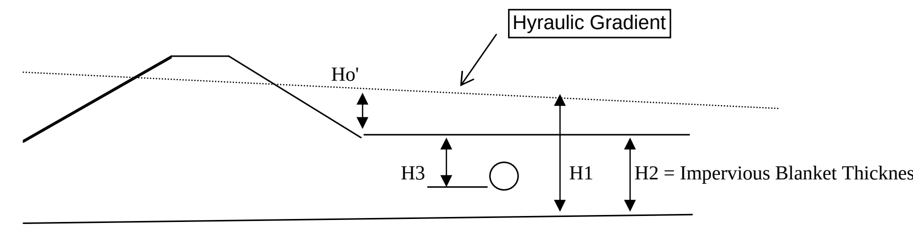
U = P3 * diameter * l
Using l = 1

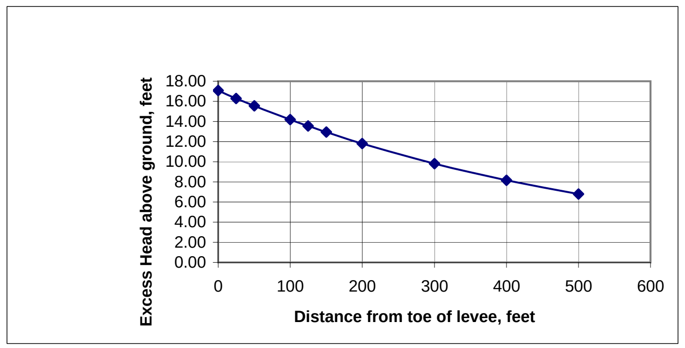
Soldier Crk uplift #12	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t feet	s
	400	400	11.3	11.3	11.3	65	19.9	0.84	10	0	0.56	10	80	542	632	632	17.07	17.07	17.07	1.51	542	542	0	0.000

Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineer, Scott Loehr, and is used to develop the Hydraulic Gradient at various distances from the toe. Verification of these numbers is done separately.

Lr Levee Width, L2
10 80
Levee Elev. 919
400 Berm Width, Wt (feet)
60 Berm Height, t (feet)
Ground Elev. 899.1 — 9.1 Pipe Dept (ft)
Blanket Base Elev. 887.8 — 890 Pipe Elev.
Bedrock Elev. 822.8 — 12 Pipe Diameter (in)
ml-cl Soil Type
Ground Elev = Landside Average Ground Surface Used as Reference Datum — 120 Soil Unit Weight, γsoil (pcf)







d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 12 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	17.07	890	887.8	899.1	9.1	11.3	28.4	1425.4	16	49	972	1425	0	0.73	0.69
25	16.30	890	887.8	899.1	9.1	11.3	27.6	1386.8	16	49	972	1387	0	0.75	0.71
50	15.56	890	887.8	899.1	9.1	11.3	26.9	1349.9	16	49	972	1350	0	0.77	0.73
100	14.19	890	887.8	899.1	9.1	11.3	25.5	1281.0	16	49	972	1281	0	0.81	0.77
125	13.55	890	887.8	899.1	9.1	11.3	24.9	1248.8	16	49	972	1249	0	0.83	0.79
150	12.94	890	887.8	899.1	9.1	11.3	24.2	1218.1	16	49	972	1218	0	0.85	0.81
200	11.80	890	887.8	899.1	9.1	11.3	23.1	1160.8	16	49	972	1161	0	0.89	0.85
300	9.81	890	887.8	899.1	9.1	11.3	21.1	1060.9	16	49	972	1061	0	0.98	0.93
400	8.16	890	887.8	899.1	9.1	11.3	19.5	977.9	16	49	972	978	0	1.06	1.01
500	6.78	890	887.8	899.1	9.1	11.3	18.1	908.8	16	49	972	909	0	1.14	1.09

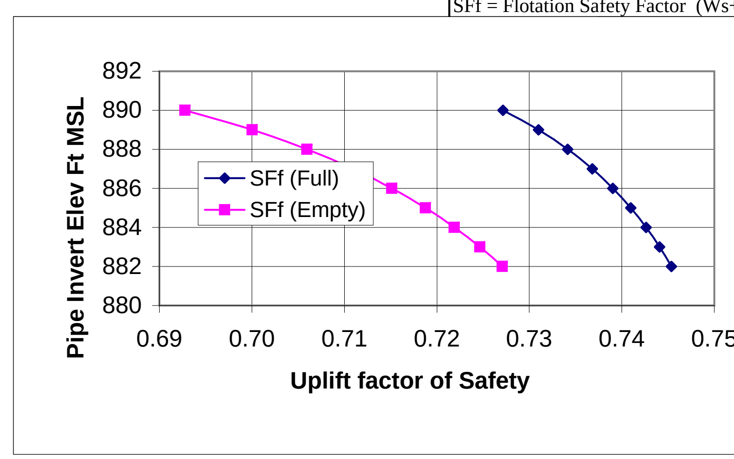
Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

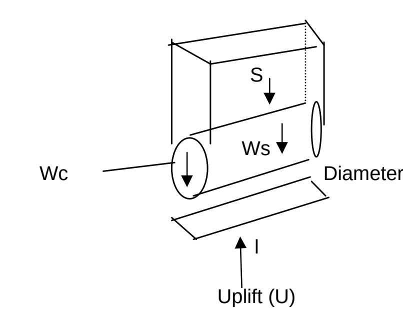
	Ground Elev 899.1	Distance from toe (ft)	0			Uplift Safety Extreme Case (1.1 REQ'D)	
		Ws = 16		Wc = 49 Full			
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	899.1	0		16	49		
9.1	890	972	1425	16	49	0.73	0.69
10.1	889	1092	1582	16	49	0.73	0.70
11.1	888	1212	1739	16	49	0.73	0.71
12.1	887	1332	1895	16	49	0.74	0.71
13.1	886	1452	2052	16	49	0.74	0.72
14.1	885	1572	2209	16	49	0.74	0.72
15.1	884	1692	2365	16	49	0.74	0.72
16.1	883	1812	2522	16	49	0.74	0.72
17.1	882	1932	2679	16	49	0.75	0.73

Ws Calc
15.5

Wc Calc
49

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 78 lb per ft 12-inch Diameter cmp
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*(12/12)^2) *1*62.4 = 49 pl
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)'*1'*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)





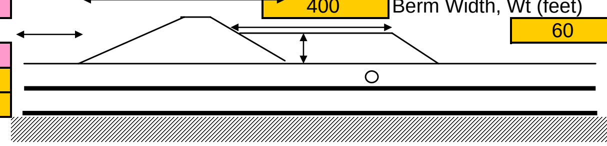
U = P3 * diameter * l
Using l = 1

Waterworks	K-r	K-L	DbL	Dbo	Dbr	Df	H	i-c	Lr	Wt	Safety	L1	L2	Le	Lt	L't	Ho'	Hwt/2	Hwt	i-o	Cr	Cl	t	s
uplift #13																							feet	
	600	600	12.75	12.75	12.75	65	10.1	0.84	50	0	1.33	50	125	705	880	880	9.88	8.09	8.09	0.63	705	705	0	0.000

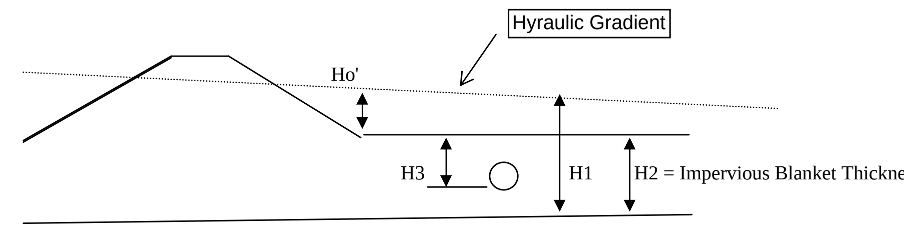
Geotechnical data (above) presented in this spreadsheet was provided by Geotechnical Engineer, Scott Loehr, and is used to develop the Hydraulic Gradient at various distances from the toe. Verification of these numbers is done separately.

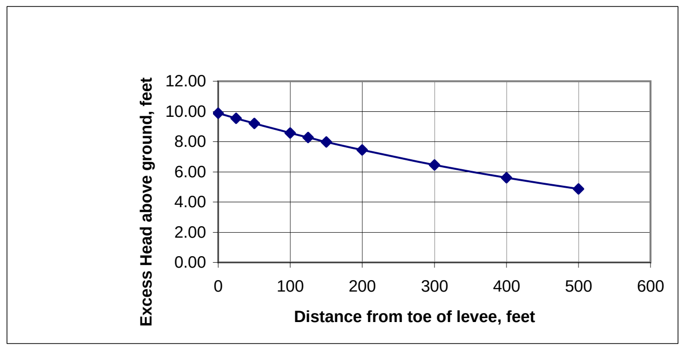
	Lr	Levee Width, L2
	50	125
Levee Elev.	898.1	
	400	Berm Width, Wt (feet)
	60	Berm Height, t (feet)
Ground Elev.	888	
Blanket Base Elev.	875.25	
Bedrock Elev.	810.25	

3	Pipe Dept (ft)
885	Pipe Elev.
10	Pipe Diameter (in)
ml	Soil Type
120	Soil Unit Weight, γsoil (pcf)



Ground Elev = Landside Average Ground Surface Used as Reference Datum





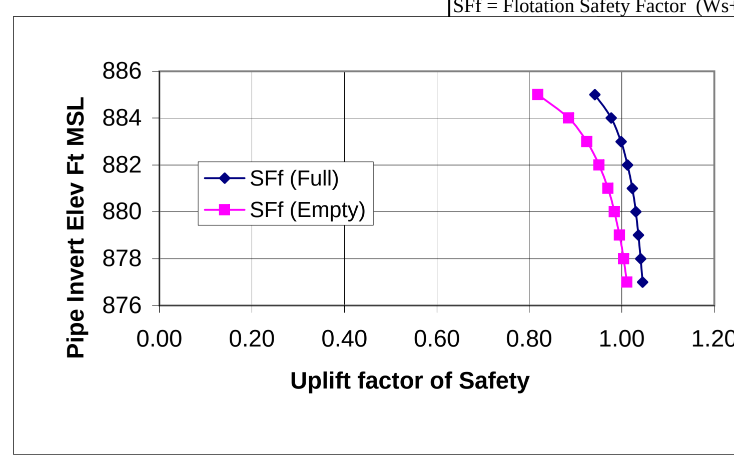
d=Landside Distance from Toe Feet	No Berm Calcs Excess Head Feet, Ho'	Invert Elev of pipe 10 -in steel	Design Base Blanket Feet, MSL	Ground Elev Feet, MSL	H3 Feet	H2 Feet	H1 Feet	P3 Psf	Ws lb	Wc lb	S lb	U lb	Wg lb	SFf (Full)	SFf (Empty)
0	9.88	885	875.25	888	3	12.75	22.6	332.3	10	34	217	277	0	0.94	0.82
25	9.54	885	875.25	888	3	12.75	22.3	327.2	10	34	217	273	0	0.96	0.83
50	9.20	885	875.25	888	3	12.75	22.0	322.3	10	34	217	269	0	0.97	0.84
100	8.57	885	875.25	888	3	12.75	21.3	313.1	10	34	217	261	0	1.00	0.87
125	8.28	885	875.25	888	3	12.75	21.0	308.7	10	34	217	257	0	1.01	0.88
150	7.99	885	875.25	888	3	12.75	20.7	304.5	10	34	217	254	0	1.03	0.89
200	7.44	885	875.25	888	3	12.75	20.2	296.4	10	34	217	247	0	1.06	0.92
300	6.46	885	875.25	888	3	12.75	19.2	282.0	10	34	217	235	0	1.11	0.96
400	5.60	885	875.25	888	3	12.75	18.4	269.5	10	34	217	225	0	1.16	1.01
500	4.86	885	875.25	888	3	12.75	17.6	258.6	10	34	217	215	0	1.21	1.05

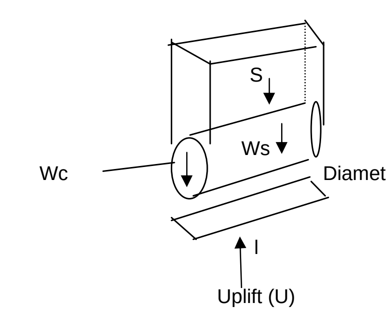
Ws Calc
10
Wc Calc
34

See Sample Calculations for list of abbreviations and sample calculations
Ws = weight of structure per foot of length = 10 lb per ft 10-inch Diameter rcp
Wc = weight of water contained in the structure = pi * r^2 * 1 = 3.1416*(10/12)^2) *1*62.4 = 34 pl
S = surcharge loads = weight of satuarted soils above = (H3-Pipe Diameter/12)*(Pipe Diameter/12)*1*γsoil
P3 = H3*(H1/H2)*γwater
U = Uplift force on the project area of structure = Area of pipe * P3 = (Pipe Diameter/12)'*1'*P3
Wg = weight of surcharge water above top surface of structure control by gravity flow
SFf = Flotation Safety Factor (Ws+Wc+S)/(U-Wg)

Use this table to determine the distance from toe at which the Safety Factor is me
Enter the Distance from toe and change the value until the SFf is equal to that required

	Ground Elev 888		Distance from toe (ft)		0		
		Ws = 10		Wc = 34 Full		Uplift Safety Extreme Case (1.1 REQ'D)	
Depth Below Ground	Pipe Inv Elev, Ft	Surcharge Load, S, lbs/ft	Uplift (U) Force, lbs/ft	Pipe Weight, Lbs/ft	Full water Wt, Lbs / ft	SFf Full	SFf Empty
	888	0		10	34		
3	885	217	277	10	34	0.94	0.82
4	884	317	369	10	34	0.98	0.88
5	883	417	461	10	34	1.00	0.92
6	882	517	554	10	34	1.01	0.95
7	881	617	646	10	34	1.02	0.97
8	880	717	738	10	34	1.03	0.98
9	879	817	831	10	34	1.04	1.00
10	878	917	923	10	34	1.04	1.00
11	877	1017	1015	10	34	1.04	1.01





U = P3 * diameter * l
Using l = 1

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Chapter A-5

STRUCTURAL ANALYSIS

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A-5 TOPEKA STRUCTURAL RELIABILITY ANALYSIS

A-5.1 INTRODUCTION

An input requirement of the HEC-FDA program model is the reliability or probability of failure for flood risk management features with water at various elevations. The structural features of the levee units included in this study consist of floodwalls, pump stations, closure structures for openings in levees and floodwalls, gatewells, reinforced box culverts, drainage structures, and retaining walls integral to the integrity of the levee system. The structural analysis involved an assessment of the existing condition of the structures. The assessment was based on visual observation, dated construction plans, historical data, discussions with the Corps of Engineers and Levee District personnel (those familiar with and involved in the inspection, operation, and maintenance of the levee units), detailed engineering analysis, and engineering judgment. The results of this portion of the study will be used in the development of probabilities of failure as required for input into the HEC-FDA model. Probability of failure analysis will not be used for design.

A-5.2 STRUCTURAL RELIABILITY METHODOLOGY

The following structural methodology was developed by the Kansas City District during the course of the Phase 1 – Kansas Citys Levees Feasibility Study. The subsequent criterion was accepted by representatives of the U.S. Army Corps of Engineers Headquarters in the Fall of 2005. The approved structural reliability methodology used in the course of this study is summarized below.

A-5.3 DETERMINISTIC CRITERIA

A series of screening criteria are used to determine if a probabilistic analysis is necessary for a given structure. Summarized below are the current stability and strength criterion developed from and based on current design standards. If analysis shows the existing structure to meet the criterion (derived from current design criterion), the structure is assumed reliable and a 99.8% reliability is assigned. If the structure does not meet this criterion a reliability analysis is performed.

A-5.3.1 Stability Requirements

Structural stability criterion can be seen in Table A-5-1. It is based upon the EM 110-2-2100– *Stability Analysis of Concrete Structures*, dated 01 December 2005, with the exception of the extreme load condition. There is some concern with the extreme load condition categories as specified in EM 110-2-2100. The Missouri River L-142 Design Criteria Issue Resolution Paper (2002) addressed these issues and put forth more stringent guidelines for recommended extreme load condition stability criteria. That criterion is used herein.

Table A-5-1: Stability Criterion

Recommended Sliding Stability Factor of Safety		
Load Condition Category	**Return Period**	**Factor of Safety**
Usual	10 yrs	2
Unusual	300 yrs	1.5
Extreme	Top of Levee	1.3*

Recommended Rotational Stability Percent of Base in Compression		
Load Condition Category	**Return Period**	**Percent of Base in Compression**
Usual	10 yrs	100%
Unusual	300 yrs	75%
Extreme	Top of Levee	25% *

Recommended Maximum Allowable Bearing Capacity % Increase in Allowable Bearing Capacity		
Load Condition Category	**Return Period**	**% Increase in Allowable Bearing Capacity**
Usual	10 yrs	0%
Unusual	300 yrs	15%
Extreme	Top of Levee	50%
Recommended Flotation Stability Factor of Safety		
Load Condition Category	**Return Period**	**Factor of Safety**
Usual	10 yrs	1.3
Unusual	300 yrs	1.2
Extreme	Top of Levee	1.1

* Stability requirements increased from value in EM 110-2-2100

A-5.3.2 Strength Requirements

a. Unfactored loads and unreduced strengths were used in the analysis. Factored loads and reduced strengths are used for design and are not appropriate for a probability of failure analysis. This implies that if an existing structure has a calculated Factor of Safety of less then 1.0 (Capacity/Demand), the structure has ceased to function as designed.

b. For new structures designed with the Strength Design Method, loads are increased by multiplying service loads by appropriate load factors and nominal strengths are decreased by corresponding strength reduction factors. Load factors required by EM 1110-2-2104, *Strength Design for Reinforced-Concrete Hydraulic Structures* include a dead and live load factor (LF) of 1.7 and a hydraulic factor (HF) of 1.3. Combining these gives a total load factor (TF) of 2.2. The strength reduction factor for flexure (ϕ), the typical controlling failure mechanism, is 0.90. Dividing the load factor by the strength reduction factor gives an overall factor of safety of about 2.45 for a new design.

c. When considering an allowable factor of safety for existing structures, several allowable reductions can be taken into account. EM 1110-2-2104 allows for a 25% reduction in load for short duration loads with a low probability of occurrence (SD = 0.75), which would apply to flood events with a return period of greater than 300 years. A "performance" factor (PF) is proposed to take into account the successful response of the existing structure to design or near design loads. If an existing structure has performed well under load and not shown visible signs of distress, a 15% reduction in factor of safety is acceptable as a threshold for requiring an upgrade to the structure. Combining the design load factors with the frequency and performance factors [((LF x HF) / ϕ) x SD x PF)] produces an approximate 1.5 Factor of Safety for existing hydraulic structures under extreme loading conditions.

d. For structures subjected to earthen loads without extreme water loadings, such as unsubmerged box culverts and gatewells, the hydraulic load and extreme loading reduction factors would not apply. The resulting allowable factor of safety would include a 1.7 live load factor (LF), a 0.90 flexural strength reduction factor (ϕ), and a 15% factor of safety reduction for known performance of existing structures (PF = 0.85). Combining these load factors and strength reductions [(LF/ϕ) x PF] would result in a 1.6 allowable factor of safety for existing structures under normal (non-hydraulic) load conditions.

Table A-5-2: Strength Criterion

Recommended Minimum Strength Factors of Safety		
Load Condition Category	**Return Period**	**Factor of Safety**
Non-Hydraulic	N/A	1.6
Extreme	Top of Levee	1.5

A-5.4 UNCERTAINTY ANALYSIS

a. For structural features not meeting deterministic strength and stability criterion, a risk and uncertainty analysis was performed. The method adopted for calculating a probability of failure is that outlined for geotechnical engineering in "Factors of Safety and Reliability in Geotechnical Engineering", by J. Michael Duncan, published in the Journal of Geotechnical and Geoenvironmental Engineering, April 2000. The use of this method provides consistency between the structural and geotechnical analyses.

b. To produce a probability of failure curve, the critical section of each feature not meeting criteria was analyzed (factor of safety determined) using mean material strengths and/or mean soil properties. Next, the parameters were varied to plus and minus one standard deviation from the mean one at a time and the factor of safety was recomputed. A Taylor Series expansion was then used to compute a probability of failure. A 2% probability of failure was used as an appropriate non-failure threshold. If a probability of failure greater than 2% resulted, then the water elevation was lowered in 1-foot increments and the feature was reanalyzed until the probability of failure obtained was less than 2%.

c. The Taylor Series Method (TSM) of analysis was used in the calculation of structural risk and uncertainty. The TSM is appropriate when data is normally distributed, when parameters display a linear relationship, and when degradation over time is not a consideration. Because of the limited availability of data and with no information to suggest otherwise, an assumption of normal distributions for input data is reasonable and consistent with guidance provided in ETL 1110-2-547 (paragraph B-6.c). Examples of non-linear behavior for which the TSM should not be used include overturning stability analysis when the resultant is outside the kern of the base. Examples of degradation over time would include scour around piles, reactive concrete, sliding movement, and deteriorating drainage systems that affect uplift. All available historic data, site inspections, and engineering judgment do not show time dependent deterioration of structures to be a concern for the Topeka Levee Systems.

A-5.4.1 Risk Calculation

a. For strength calculations, uncertainty is measured by applying a mean and standard deviation to the concrete and steel strengths. The selected mean and normal standard deviation are based on engineering judgment and information published in *Reliability Based Design in Civil Engineering* by Milton E. Harr.

b. For stability calculations, uncertainty is considered by applying a mean and standard deviation to the soil unit weight and shear strength, and is based on values provided by the geotechnical engineers. The uncertainty inherent in determining the soil parameters provides a means to find a probability of failure. From experience on the Missouri River Levee Project L-142 Criteria Study (KCD-COE), it was determined through analysis that the unit weight and the soil shear strength have a noticeable effect on a floodwall's factor of safety. Varying the concrete density has only a minor effect on the factor of safety.

c. Failure is defined as the capacity to demand ratio (factor of safety) less than 1.0, or in other words when the demand (loads) exceed the capacity (structural or geotechnical).

A-5.4.2 Structural Material Properties

a. For the screening portion of the Topeka Levee Systems feasibility study the following structural properties were used. The American Concrete Institute recommended the use of a 3,000 psi concrete strength around the 1940's through 1960's, the typical timeframe of construction for most of the levee structures in the study. For earlier concrete strengths little information exists. It is currently assumed that 2500 psi concrete strengths are appropriate. If additional research information is discovered this value will be updated.

b. Knowing the time period of construction (~1940's – 1960's) and based upon the Portland Cement Association's pamphlet *Engineered Concrete Structures*, 1997, an assumed reinforcing steel design yield strength, F_y, of 40 ksi is used for most computations, unless known or stated otherwise. For earlier structures (~1900's), the Concrete Reinforcing Steel Institute in *Engineering Data Report 48* suggests 33 ksi steel is typical.

c. Based on FEMA 310, the mean strength (or expected strength) for Risk and Uncertainty calculations shall be taken as 125% of the design strength. For reinforced concrete structures Harr suggests a 14% standard deviation.

Concrete Strength Variation (14%)
1940's-1950's: $\mu - \sigma = 3225$, $\mu = 3750$, $\mu + \sigma = 4275$ (3000 psi min)
1900's-1920's: $\mu - \sigma = 2150$, $\mu = 2500$, $\mu + \sigma = 2850$ (2000 psi min)
Steel Strength Variation (14%)
1940's-1950's: $\mu - \sigma = 43$, $\mu = 50$, $\mu + \sigma = 57$ (40 ksi min)
1900's-1920's: $\mu - \sigma = 35.5$, $\mu = 41.25$, $\mu + \sigma = 47.0$ (33 ksi min)

A-5.4.3 Soil Material Properties

a. The soil properties used to compute loads on structures for the Topeka Feasibility study are located in Table A-5-3. The values posted were obtained from the *Topeka Feasibility Study Phase I – Existing Conditions Geotechnical Appendix* in consultation with the geotechnical engineers of record. These simplified values were generalized conservatively for use in typical structural calculations.

Table A-5-3: Soil Properties

Parameter	Soldier Creek Unit	North Topeka Unit	Waterworks Unit	South Topeka Unit	Oakland Unit	Auburndale Unit
Friction Angle	26.5	26.5	26.5	22.0	19.0	26.5
Cohesion	0	0	0	0	0	0
Moist unit wt.	120	120	120	120	120	120
Saturated unit wt.	120	120	120	120	120	120

Note: Soil to structure friction and cohesion interaction were typically neglected for stability and strength calculations.

b. Geotechnical members of the project team provided standard deviations of 8% and 10% of the mean for both soil unit weight and soil shear strength respectively.

A-5.5 STRUCTURAL ANALYSIS

The following structural features were analyzed for the Topeka feasibility study. Features specific to only one levee unit are mentioned below briefly and are described in greater detail in the section relating to the levee unit in which the feature is located. Features unique to a levee unit and analyzed in a manner different then described below are also more thoroughly discussed in the related levee unit section.

A-5.5.1 Floodwalls on Spread Footings

a. Spread footing floodwalls were analyzed for sliding, bearing capacity and overturning stability, along with wall stem and foundation strengths. Each floodwall cross-section was analyzed using the Corps of Engineers CASE project program CTWALL. CTWALL computes a sliding factor of safety, percent base in compression, and maximum bearing pressure. Sliding factors of safety and percent base in compression were compared to required design minimums. The ratio of bearing pressure to allowable soil bearing capacity as supplied from geotechnical team members was compared to allowable maximums.

b. CTWALL output includes a free body diagram detailing the horizontal and vertical forces acting on the wall cross section. These forces were entered into a MathCAD worksheet developed by the Kansas City District to check shear and flexural strengths. The failure of floodwall stems or foundations was based on a capacity/demand ratio of less than one.

c. For floodwalls not meeting the minimum strength and stability factors of safety, a reliability analysis was conducted for the floodwall cross section displaying the lowest (controlling) factor of safety. The resulting reliability curve for the critical cross section is then defined as the representative curve for the entire reach of floodwall. (For example, a hypothetical floodwall has 5 different cross sections, Sections A through E. Section C has the lowest factor of safety. The resulting reliability curve for Section C would be used to define the reliability of the entire hypothetical floodwall.) Failure was based on a capacity/demand ratio (structural or geotechnical) of less than one.

A-5.5.2 Retaining Walls

Retaining walls located in the line of flood risk management and critical to the function of the levee were analyzed in a method consist with that of spread footing floodwalls.

A-5.5.3 Stoplog and Sandbag Closure Structures

a. All stoplog closure structures in the Topeka levees system have spread footing foundations. Stoplog closure structures were analyzed in a simplified manner similar to spread footing floodwalls. All Topeka stoplog gaps are one gap wide and do not have intermediate posts. Stoplog structure stability was analyzed using CTWALL in conjunction with a typical stoplog wall cross section. Free body pressures from CTWALL are used to check reinforcing steel in the closure structure foundation and stem walls. Because these simplified strength calculations revealed no foundation or wall stem strength concerns, and because no levee raises are purposed, foundation rigidity, stoplog strengths and stoplog post slots were not checked.

b. Routine levee inspections of sandbag gaps have revealed no foundation slab issues for the Topeka units. Strength and stability calculations were not performed for sandbag closure structures. If strength or uplift concerns are experienced during flood events, it can reasonably be assumed that flood fighting efforts (additional sandbags) would be successful in addressing any uplift problems.

A-5.5.4 Floodwalls on Piles

The only floodwall on piles is located in the South Topeka unit. A more detailed description of the analysis of floodwalls on piles is located in the South Topeka section of this report.

A-5.5.5 Pump Stations

The structural evaluation of pump stations focused on floatation stability along with foundation wall and floor strengths. The potential for pump station uplift was computed using a Kansas City developed MathCAD worksheet based on EM 110-2-2100. Stability Analysis of Concrete Structures taking into consideration site specific hydraulic grade lines supplied by Geotechnical project team members. Foundation wall and floor capacities were calculated using MathCAD worksheets based on plate mechanics and each component’s length to width aspect ratios. The CASE project program CASTR was used when combined axial bending capacity computations were necessary. For pump stations not meeting strength and floatation factors of safety, reliability calculations were performed.

A-5.5.6 Gatewells, Reinforced Concrete Boxes, and Drainage Structures

Gatewells, reinforced concrete boxes, and other drainage closure structures were all analyzed in a manner similar to the pump station evaluations. MathCAD worksheets evaluated floatation stability and structural component strengths. Because of the length to width aspect ratios of these structures, plate mechanics were not used. Instead wall and floor component capacities were assessed using one-way beam analysis. For structures not meeting the minimum factors of safety, reliability analysis was conducted.

A-5.6 SUMMARY OF RESULTS BY LEVEE UNIT

The following structural features were analyzed for the Topeka feasibility study.

A-5.6.1 Auburndale Unit

a. Located on the right bank of the Kansas River, the Auburndale unit begins at the intersection of Interstate Highway No. 70 and the end of the Waterworks unit levee. The levee extends eastward along the highway, with the highway fill acting as levee, gradually diverging from the highway and stretching east, southeast to the intersection of the Ward-Martin Creeks Pumping Plant. The levee is incorporated into the access road from the intersection of the access road and Ward-Martin Creeks Pump Plant until approximately levee Sta. 30+00, where it again transitions back into a zoned portion of the Highway 70 embankment fill, continuing on to Sta. 58+80, the beginning of the South Topeka unit. The unit was designed in 1958 and constructed between the years 1961 and 1962.

b. Auburndale structures considered for this study included four gatewell closure structures, one large multi-box reinforced concrete box running through the levee, and two pump stations.

A-5.6.1.1 Gatewell Closure Structures

Four Auburndale gatewells were analyzed for water to top of structure. All required factors of safety were meet and a 99.8% reliability was assigned.

Table A-5-4: Auburndale Gatewell Reliability

Auburndale Gatewells with water to top of levee				
Station	Uplift Factor of Safety (> 1.1 Req'd)	Strength Factor of Safety (> 1.5 Req'd)	Controlling Structural Mechanism	Assigned Reliability
1+90	1.383 (dry)	1.559	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
25+10	1.339 (dry)	1.706	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
37+20	1.413 (dry)	1.801	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
40+00	1.446 (dry)	2.885	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%

A-5.6.1.2 Reinforced Concrete Box

A quadruple (4-14.5’x12’) reinforced concrete box draining Ward Martin creek runs under Interstate No. 70 through the line of flood risk management. Analysis results are based on water to top of levee with no water in the box and are summarized below. All required factors of safety were meet and a 99.8% reliability was assigned.

Table A-5-5: Auburndale Reinforced Box Culvert

Auburndale Reinforced Box Culvert with water to top of levee				
Station	Uplift Factor of Safety (> 1.1 Req'd)	Strength Factor of Safety (> 1.5 Req'd)	Controlling Structural Mechanism	Assigned Reliability
28+30	1.43 (dry)	2.63	Shear in Roof Slab	99.8%

A-5.6.1.3 Pump Stations

a. Two Auburndale pump stations were constructed as part of the Federal project in the 1960's. The Waite Street Station (as builts dated 1970) and Ward Martin Creek Station (as builts dated 1963) both handle interior drainage only.

b. Table A-5-6 below summarizes reliability criteria findings for the two stations. All results are computed with water to top of levee. Column three displays uplift factors of safety with no water in the wet well. Column four shows the level of water required in the wet well to meet the minimum 1.1 required uplift factor of safety. Based on each pump stations individual pump station shutoff elevations, column five shows the actual minimum level of water likely to present in the well.

Table A-5-6: Auburndale Pump Stations

Auburndale Pump Stations with water at top of levee						
Station	Station Name	Uplift Factor of Safety (> 1.1 Req'd)	Water Req'd to meet 1.1 Uplift Factor of Safety (ft)	Water Available (ft)	Strength Factor of Safety (> 1.5 Req'd)	Comments
22+60	Waite Street	0.97 (Dry)	1.5	2	2	**No Corrective Measures Necessary**
28+30	Ward Martin Creek	0.84 (Dry)	8.5	6.5	1.74	**No Corrective Measures at this Time. To be investigated further at time of Plans and Specifications.**

c. The Waite Street Pump Station requires 1.5ft of water in the wet well to meet a required 1.1 minimum factor of safety. Based on pump shut off data, 2ft of water could be available. Consequently, no corrective measures are assumed necessary and a 99.8% reliability is assigned.

d. The Ward Martin Creek Pump Station requires 8.5ft of water in the wet well area to meet the 1.1 minimum required uplift factor of safety, yet based on the station pump operating curves only 6.5 ft of water is guaranteed to be present at any given time. Uplift calculations with 6.5ft of water in the wet well generated an uplift factor of safety greater then 1.0 so at this point it is assumed no action is necessary. At time of plans and specifications the pump station operating curves may be reanalyzed to determine if 8.5 ft of water can be stored in the wet well without impacts to interior ponding and flooding.

A-5.6.2 Oakland Unit

a. The Oakland Unit is located along the Kansas River downstream of South Topeka Unit and continuing along the left bank of Shunganunga Creek. Flood risk management features consist of 10 miles of earthen levee, one sandbag gap, and 5.5 miles of channel improvement. The Oakland Unit was designed in 1960 and constructed during the period between 1965 and 1969.

b. Oakland unit structures considered included thirty-four gatewell closure structures, twenty-six manhole and drop inlet structures, one sandbag closure gap, one floodwall section, one landside toe retaining wall, and one pump stations.

A-5.6.2.1 Gatewell Closure Structures

Analysis results are based on water to top of levee with no water in the gatewell and are summarized below. All required factors of safety were meet and a 99.8% reliability was assigned.

Table A-5-7: Oakland Gatewell Reliability

Oakland Gatewells with water to top of levee				
Station	Uplift Factor of Safety (> 1.1 Req'd)	Strength Factor of Safety (> 1.5 Req'd)	Controlling Structural Mechanism	Assigned Reliability
2+08	1.4 (dry)	1.52	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
5+45	1.4 (dry)	1.67	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
7+46	1.3 (dry)	1.74	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
7+58	1.4 (dry)	1.50	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
22+93	1.3 (dry)	1.60	Pos. Wall Reinforcing Steel w/ Yielded End Supports	99.8%
40+68	1.4 (dry)	1.55	Neg. Wall Reinforcing Steel w/ Yielded End Supports	99.8%
62+61	1.2 (dry)	1.90	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
70+75	1.3 (dry)	1.83	Pos. Wall Reinforcing Steel w/ Yielded End Supports	99.8%
118+87	1.4 (dry)	1.80	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
144+65	1.3 (dry)	1.64	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
220+00	1.3 (dry)	1.94	Pos. Wall Reinforcing Steel w/ Yielded End Supports	99.8%
241+30	1.4 (dry)	1.96	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
264+91	1.4 (dry)	2.49	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%

309+54	1.3 (dry)	1.66	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
345+97	1.4 (dry)	1.59	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
378+15	1.4 (dry)	3.41	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
398+00	1.4 (dry)	1.97	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
429+00	1.3 (dry)	1.74	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
442+30	1.4 (dry)	2.18	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
448+64	1.4 (dry)	2.19	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
458+44	1.4 (dry)	1.93	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
462+13	1.4 (dry)	1.85	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
462+26	1.4 (dry)	2.38	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
468+41	1.4 (dry)	1.99	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
474+75	1.5 (dry)	3.35	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
479+23	1.4 (dry)	3.03	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
484+80	1.3 (dry)	1.56	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
493+06	1.3 (dry)	1.58	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
500+60	1.4 (dry)	2.37	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
505+88	1.3 (dry)	1.97	Pos. Wall Reinforcing Steel w/ Yielded End Supports	99.8%
512+48	1.3 (dry)	2.27	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
517+54	1.5 (dry)	3.50	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
521+68	1.5 (dry)	3.55	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%

A-5.6.2.2 Drop Inlet and Manhole Collector Structures

a. A series of some twenty-four drop inlet and manhole collector box structures located just landward of the levee toe were analyzed. Analysis results are based on water to top of levee (full HGL) with no water in the gatewell and are summarized below.

Table A-5-8: Oakland Manholes and Drop Inlets

Oakland Manholes and Drop Inlets with water to top of levee				
Station	Uplift Factor of Safety (> 1.1 Req'd)	Strength Factor of Safety (> 1.5 Req'd)	Controlling Structural Mechanism	Assigned Reliability
0+00	1.4 (dry)	4.8	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
1+50	1.3 (dry)	4.5	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
2+06	1.4 (dry)	4.5	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
3+11	1.4 (dry)	4.7	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
5+81	1.2 (dry)	2.2	Pos. Wall Reinforcing Steel w/ Yielded End Supports	99.8%
7+07	1.4 (dry)	3.8	Neg. Wall Reinforcing Steel w/ Yielded End Supports	99.8%
7+46	1.3 (dry)	2.2	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
8+36	1.4 (dry)	4.8	Pos. Wall Reinforcing Steel w/ Yielded End Supports	99.8%
9+07	1.4 (dry)	5.1	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
40+83	1.8 (dry)	15.8	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
75+50	0.93 (dry)	1.9	Pos. Wall Reinforcing Steel w/ Yielded End Supports	3.75 ft of Water Req'd for 1.1 Uplift Factor Of Safety
118+87	1.1 (dry)	2.2	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
343+95	1.1 (dry)	2.7	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
442+30	1.1 (dry)	2.1	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
448+64	1.1 (dry)	2.3	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
458+44	1.4 (dry)	4.1	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
462+13	1.2 (dry)	1.9	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
462+26	1.2 (dry)	2.4	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
468+41	1.3 (dry)	3.4	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%

474+75	1.3 (dry)	3.1	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
479+23	1.1 (dry)	1.5	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
484+80	1.5 (dry)	2.2	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
500+60	1.4 (dry)	1.8	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
505+88	1.4 (dry)	1.8	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
512+48	1.6 (dry)	4.8	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
521+68	1.4 (dry)	4	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%

b. The 5.5'x 4.5' drop inlet buried to a 6.5 ft depth at station 75+50 fails to meet uplift criteria. Almost 4ft of water would be necessary to meet the minimum required 1.1 uplift factor of safety, while only 1.5ft are required for a factor of safety greater then 1.0. Because the 4ft water requirement may be unreasonable, a cost has been included for the addition of foundation heel extensions. Because of the relatively minor cost contribution of the repair (<$25K), and the rather significant probability of failure curve developed for the East Oakland Pump Station, a reliability curve for the drop inlet was not developed for economic input.

A-5.6.2.3 Pump Stations

a. The East Oakland Pump Station was constructed as part of the Federal Project (as builts dated 1970) to handle interior drainage.

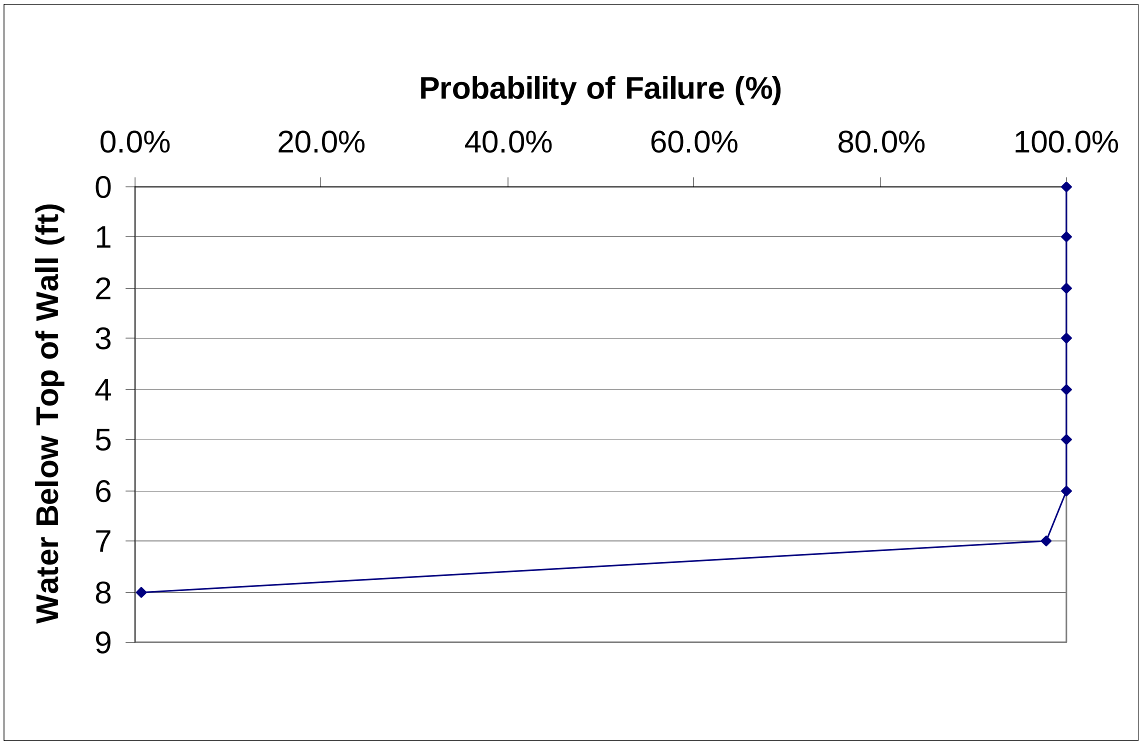
b. Table A-5-9 below summarizes reliability criteria findings. All results are computed with water to top of levee. Column three displays uplift factors of safety with no water in the wet well. Column four shows the level of water required in the wet well to meet the minimum 1.1 required uplift factor of safety. Based on the pump stations pump shutoff elevation, column five shows the actual minimum level of water likely to present in the well.

Table A-5-9: East Oakland Pump Station

East Oakland Pump Station with water at top of levee						
Station	Station Name	Uplift Factor of Safety (> 1.1 Req'd)	Water Req'd to meet 1.1 Uplift Factor of Safety (ft)	Water Available (ft)	Strength Factor of Safety (> 1.5 Req'd)	Comments
220+00	East Oakland	0.76 (Dry)	7.25	2	1.68	**Foundation Heel Extensions Req'd**

c. The East Oakland Pump station fails to meet a minimum uplift factor of safety. Using a varying hydraulic gradeline (based on possible variations in the foundation blanket thickness, blanket permeability, and foundation permeability) supplied by geotechnical team members, the reliability curve below was developed for input into HEC-FDA. To correct possible uplift concerns, extensive temporary excavation will be required to facilitate the addition of foundation heel extensions to allow for additional soil loading to counteract uplift pressures.

FIGURE 1 – East Oakland Pump Station Probability of Failure



A-5.6.2.4 Spread Footing Floodwall and Retaining Wall

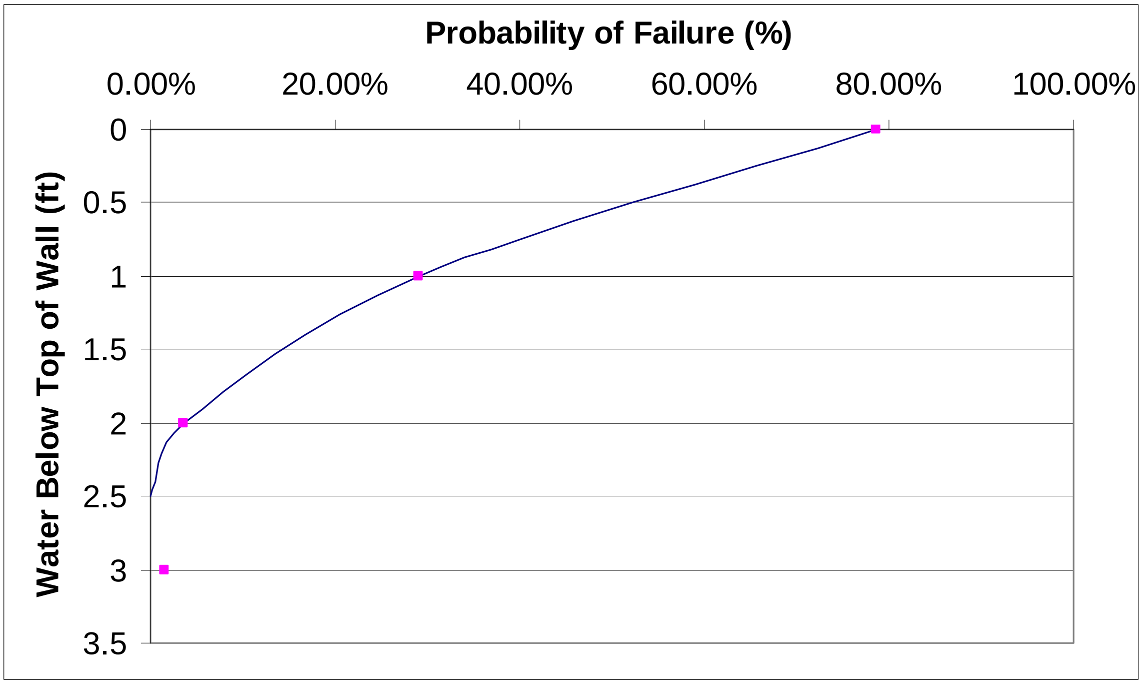
a. One floodwall runs from station 485+86 to 491+01. Exposed wall heights vary from 7ft to 9ft. Wall strength and stability calculations are summarized below.

Table A-5-10: Oakland Spread Footing Floodwall

Oakland Spread Footing Floodwall with water to top of levee						
Station	Wall Cross Section	Overturning % Base in Compression (> 25% Req'd)	% Bearing Capacity (Demand/Capacity) (150% Max Increase)	Sliding Factor of Safety (>1.3 Req'd)	Wall Strength Factor of Safety (>1.5 Req'd)	Comments
489+50	Sec B-B	38.0 %	43.5 %	0.76	1.95	2 ft of Additional Fill Req'd Behind Floodwall to meet sliding requirements
489+81	Sec A-A	45.8 %	43.5 %	0.85	1.56	2 ft of Additional Fill Req'd Behind Floodwall to meet sliding requirements

b. Both wall sections failed to meet sliding stability. Wall cross section B-B (see Table A-5-10 Sta 489+50) was determined to be the critical wall cross section (lowest factor of safety) for which a probability of failure was calculated. The risk and uncertainty analysis (using the procedure described earlier in this chapter) yielded the curve shown below. The graph in FIGURE 2 was used for HEC-FDA input data for probability of failure vs. water elevation. (The squares on the graph are the actual data point used to develop the curve.)

FIGURE 2 - Oakland Floodwall Probability of Failure



c. Subsequent analysis of the critical section revealed two feet of additional fill behind the wall would be sufficient to meet minimum sliding requirements for both wall section types. Site visits showed sufficient landside real estate is available for placement of the two feet of additional fill from approximately station 485+86 to 491+01.

A-5.6.2.5 Retaining Walls

One retaining wall located at the toe of the levee runs from station 0+51 to 1+75. Wall strength and stability calculations are summarized below. Results are based on water to top of levee. All factors of safety are met and a 99.8% reliability is assigned.

Table A-5-11: Oakland Retaining Wall

Oakland Retaining Wall with water to top of levee						
Station	Wall Cross Section	Overturning % Base in Compression (> 25% Req'd)	% Bearing Capacity (Demand/Capacity) (150% Max Increase)	Sliding Factor of Safety (>1.3 Req'd)	Wall Strength Factor of Safety (>1.5 Req'd)	Comments
1+00	Sec A-A	100 %	24.2 %	1.58	1.7	99.8 % Reliability

A-5.6.2.6 Levee Opening Closure Structures

One sandbag gap is located at Station 337+87. No deficiencies were observed and a 99.8% reliability is assumed.

A-5.6.3 North Topeka Unit

a. The North Topeka Unit is located along the left bank of the Kansas River beginning on Soldier Creek and extending upstream along the left bank of the Kansas River to approximate river mile 82. The flood risk management unit includes 9.3 miles of earthen levee. The North Topeka Unit was designed in 1961 and constructed between 1964 and 1967 for the purpose of protecting the North Topeka area.

b. North Topeka structures considered included fourteen gatewell closure structures, two reinforced concrete box structures, one sandbag closure gap, one floodwall section with stoplog gap, and three pump stations.

A-5.6.3.1 Gatewell Closure Structures

Information was available for only eight of the fourteen gatewell closure structures located along the North Topeka unit. Because no problems were discovered with any of the other North Topeka gatewells (or any other Topeka unit's gatewells) and site inspections revealed on issues, it is assumed that the gatewells for which information was not located are also acceptable. Non-Destructive Testing may be necessary to determine material strengths, reinforcing details, and wall thicknesses to validate this assumption when plans and specifications are prepared.

Table A-5-12: North Topeka Gatewell Reliability

North Topeka Gatewells with water to top of levee				
Station	Uplift Factor of Safety (> 1.1 Req'd)	Strength Factor of Safety (> 1.5 Req'd)	Controlling Structural Mechanism	Assigned Reliability
81+50	1.2 (dry)	1.51	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
104+00	No Information			
172+00	1.42	2.03	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
177+41	No Information			
210+00	No Information			
215+50	1.33	1.78	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
260+88	No Information			
277+00	No Information			
295+75	1.49	3.49	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
299+20	1.35	1.64	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
325+15	1.3	1.6	Pos. Wall Reinforcing Steel w/ Yielded End Supports	99.8%
364+60	1.27	2.75	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
375+00	No Information			
493+70	1.2	1.6	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%

A-5.6.3.2 Reinforced Concrete Boxes

One active and one abandoned reinforced concrete box crosses the North Topeka unit. Results shown in Table A-5-13 are based on water to top of levee with no water in the box. All factors of safety are met and a 99.8% reliability is assumed.

Table A-5-13: North Topeka Reinforced Box Culverts

North Topeka Reinforced Box Culvert with water to top of levee				
Station	Uplift Factor of Safety (> 1.1 Req'd)	Strength Factor of Safety (> 1.5 Req'd)	Controlling Structural Mechanism	Assigned Reliability
92+68	Abandoned, Filled in Place			99.8%
392+05	2	1.5	Pos. Wall Reinforcing Steel w/ Yielded End Supports	99.8%

A-5.6.3.3 Pump Stations

a. Three pump stations are located along the North Topeka unit. The exact date of construction for the Fairchild Pump station is unknown (probably 1920's) and the station is no longer used. Quincy and Soldier Creek pump stations were constructed as part of the Federal project in the 1960's (as-builts dated 1969) for interior drainage.

b. Table A-5-14 below summarizes reliability criteria findings. All results are computed with water at the top of levee. Column three displays uplift factors of safety with no water in the wet well. Column four shows the level of water required in the wet well to meet the minimum 1.1 required uplift factor of safety. Based on each pump stations individual pump station shutoff elevations, column five shows the actual minimum level of water likely to present in the well.

Table A-5-14: North Topeka Pump Stations

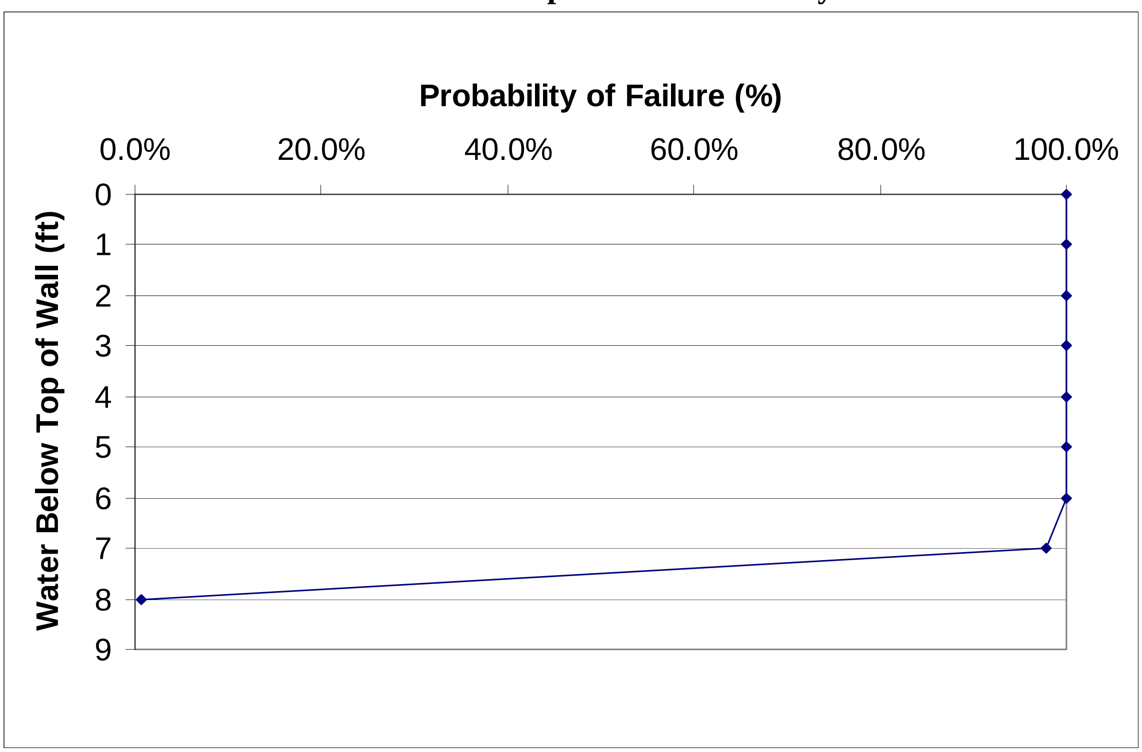
North Topeka Pump Stations with water at top of levee						
Station	Station Name	Uplift Factor of Safety (> 1.1 Req'd)	Water Req'd to meet 1.1 Uplift Factor of Safety (ft)	Water Available (ft)	Strength Factor of Safety (> 1.5 Req'd)	Comments
325+15	Soldier Creek	0.93 (Dry)	4.25	4.05	1.57	**No Corrective Measures Necessary**
364+60	Fairchild	0.72 (Dry)	9.4	No Information Available		**Station to be abandoned in place. Fill substructure and outlet lines with flowable fill.**
392+05	Quincy	1.13 (Dry)	N/A	N/A	1.53	**No Corrective Measures Necessary**

c. Without water in the wet well, the Solider Creek pump station fails to meet uplift criteria. Calculations show approximately 4.25 ft of water in the wet well would be required to achieve a 1.1 uplift Factor of Safety. The pumps shutoff when water drops to 4 ft of water in the well, and with 4ft of water in the wet well, an uplift factor of safety greater then 1.0 is calculated. At time of plans and specifications the pump station operating curves may be reanalyzed to determine if 4.25ft of water can be stored in the wet well without impacts to interior ponding and flooding.

d. Little information has been located for the Fairchild pump station. Pump station uplift calculations are based on field measurements of exterior footprint dimensions, interior sump dimensions, and assumptions for floor member thickness. Using these dimensions and varying hydraulic gradelines (based on possible variations in blanket thickness, blanket permeability, and foundation permeability) supplied by geotechnical team members, very low reliabilities were calculated for the Fairchild pump station for water at any elevation on the levee. The Fairchild pump station is no longer in use and will be abandoned in place by filling both the pump station substructure and outlet works with flowable fill.

e. The relatively small estimated cost for the fix (~$40K) can be justified by the prevention of only minimal damages. Consequently, a refined reliability curve has not been developed for the Fairchild pump station. Instead the reliability curve developed for the more reliable East Oakland pump station will also be used to define the Fairchild station.

FIGURE 3 – Fairchild Pump Station Probability of Failure



A-5.6.3.4 Spread Footing Floodwall

A floodwall starts at station 300+28 and extends to station 301+06 with exposed wall heights up to approximately 7feet. Wall strength and stability calculations are summarized below. All required factors of safety were met and a 99.8% reliability was assigned.

Table A-5-15: North Topeka Spread Footing Floodwall

Spread Footing Floodwall with water to top of levee						
Station	Wall Cross Section	Overturning % Base in Compression (> 25% Req'd)	% Bearing Capacity (Demand/Capacity) (150% Max Increase)	Sliding Factor of Safety (>1.3 Req'd)	Wall Strength Factor of Safety (>1.5 Req'd)	Comments
300+28	Sec B	86.4 %	27.8 %	3.87	2.08	99.8 % Assigned Reliability

A-5.6.3.5 Levee Opening Closure Structures

A single railroad stoplog closure gap structure is located in the North Topeka floodwall. Results are summarized below. All required factors of safety were met and a 99.8% reliability was assigned.

Table A-5-16: North Topeka Levee Opening Closure Structures

North Topeka Closure Structures with Water to Top of Wall					
Station	Overturning % Base in Compression (> 25% Req'd)	% Bearing Capacity (Demand/Capacity) (150% Max Increase)	Sliding Factor of Safety (>1.3 Req'd)	Wall Strength Factor of Safety (>1.5 Req'd)	Comments
29+55	Sandbag Closure Gap, No Deficiencies Observed				99.8% Assigned Reliability
300+68	86.4 %	27.8 %	3.45	2.09	99.8 % Reliability

A-5.6.4 Soldier Creek Unit

a. The Soldier Creek Unit is located along Soldier Creek, beginning at Kansas River mile 81.9 and extending northwesterly to the vicinity of the Silver Lake channels and levees. The purpose of this unit is to provide flood risk management for north Topeka against a peak Soldier Creek discharge of approximately 50,000 cfs. The Soldier Creek unit includes 17.9 miles of levee, 9.2 miles of channel improvement, and approximately 4.3 miles of tributary tie back levees along the left bank of Soldier Creek. The project was designed in 1958 and constructed between the years 1958 and 1962.

b. Nineteen gatewell closure structures were considered.

A-5.6.4.1 Gatewell Closure Structures

a. Analysis results are based on water to top of levee with no water in the gatewell and are summarized below. All required factors of safety were met and a 99.8% reliability was assigned.

Table A-5-17 – Soldier Creek Gatewell Reliability

Soldier Creek Gatewells with water to top of levee				
Station	Uplift Factor of Safety (> 1.1 Req'd)	Strength Factor of Safety (> 1.5 Req'd)	Controlling Structural Mechanism	Assigned Reliability
Right Bank				
-1+75	1.4 (dry)	2.27	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
39+80	1.5 (dry)	3.94	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
62+15	1.5 (dry)	3.94	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
75+20	1.5 (dry)	3.43	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
93+55	1.5 (dry)	3.16	Neg. Wall Reinforcing Steel w/ Yielded End Supports	99.8%
115+00	1.5 (dry)	4.42	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
138+49	1.5 (dry)	3.95	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
162+90	1.5 (dry)	3.49	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
191+50	1.4 (dry)	1.92	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
210+82	1.2 (dry)	2.18	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
244+96	1.4 (dry)	1.88	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
264+10	1.4 (dry)	3.57	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
344+65	1.4 (dry)	2.04	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%

375+50	1.4 (dry)	2.36	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
399+20	1.4 (dry)	1.97	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
Left Bank				
138+05	1.4 (dry)	2.31	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
277+90	1.4 (dry)	2.12	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
330+25	1.4 (dry)	1.84	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
Tieback				
2+87 TB3	No Information			99.8% Assumed

b. Information was not available for one of the nineteen gatewell closure structures located along the Soldier Creek unit. Because no problems were discovered with any of the other Soldier Creek gatewells (or any other Topeka unit gatewells) and site inspections revealed no issues, it is assumed that the gatewells for which information was not located are also acceptable. Non-Destructive Testing may be necessary to determine material strengths, reinforcing details, and wall thicknesses to validate this assumption when plans and specifications are prepared.

A-5.6.5 South Topeka Unit

a. The South Topeka Unit is located along the right bank of the Kansas River between the Auburndale Unit at the west upper end (river mile 85.5) and Santa Fe Railroad bridge at mile 83.8 at the lower end. The unit consists of 1.4 miles of earthen levee, 1,944 feet of pile founded floodwall and two stoplog gaps. The unit was designed in 1966 and constructed between the years of 1970 and 1973, though incorporated portions of the unit predate the 1940's.

b. South Topeka structures considered for this study included eight gatewell closure structures, six riverside closure gates, forty-six manhole and drop inlet structures, two reinforced concrete box structures, four pump stations, one pile founded floodwall, and two stoplog closure gaps, and associated spread footing transition walls.

A-5.6.5.1 Gatewell Closure Structures

Information was only available for four of the eight gatewell closure structures located along the South Topeka unit. The four gatewells for which information could not be found are located in the pile founded floodwall. Significant concerns about the floodwall reliability have led to a recommendation that the floodwall be removed and replaced. Because the four gatewells are integral with the floodwall, in the process of replacing the floodwall, the four gatewells will also require replacement.

Table A-5-18: South Topeka Gatewell Reliability

South Topeka Gatewells with water to top of levee				
Station	Uplift Factor of Safety (> 1.1 Req'd)	Strength Factor of Safety (> 1.5 Req'd)	Controlling Structural Mechanism	Assigned Reliability
16+07	1.3 (dry)	1.8	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
19+81	1.2 (dry)	1.5	Pos Wall Reinforcing Steel w/ Yielded End Supports	99.8%
67+55	1.3 (dry)	1.7	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%
69+22	No Information, To Be Replaced with Floodwall			
75+62	No Information, To Be Replaced with Floodwall			
86+09	No Information, To Be Replaced with Floodwall			
86+55	No Information, To Be Replaced with Floodwall			
88+09	1.5 (dry)	3.4	Neg. Wall Reinforcing Steel w/ Fixed End Supports	99.8%

A-5.6.5.2 Manholes and Drop Inlets

a. An elaborate system of some forty-six manholes, drop inlets, and relief wells form an underseepage relief system along the South Topeka levee unit. A summary of results follows.

Table A-5-19: South Topeka Manholes

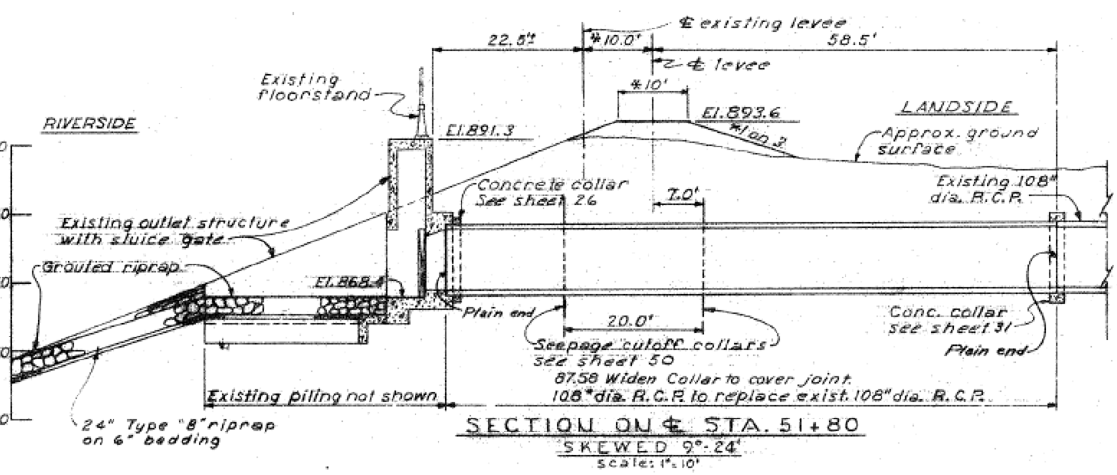
South Topeka Manholes with Water at Top of Levee				
Station	Uplift Factor of Safety (> 1.1 Req'd)	Water Req'd to meet 1.1 Uplift Factor of Safety (ft)	Strength Factor of Safety (> 1.5 Req'd)	Comments
16+07	0.84 (Dry)	11 ft	4.4	**Heel Extension Required**
73+69	1.47 (Dry)		2.1	
73+98	1.38 (Dry)		2.4	
74+02	1.47 (Dry)		2.7	
74+16	1.51 (Dry)		1.8	
75+01	1.37 (Dry)		2.5	
75+10	1.24 (Dry)		2.1	
75+13	1.32 (Dry)		2.4	
75+20	1.43 (Dry)		1.6	
75+48	1.68 (Dry)		2.1	
75+84	1.19 (Dry)		2.2	
76+09	1.33 (Dry)		2.2	
76+25	1.32 (Dry)		1.7	
76+32	1.10 (Dry)		2.1	
76+75	1.10 (Dry)		2.0	
77+22	1.97 (Dry)		2.0	
77+50	1.10 (Dry)		2.0	
77+91	1.10 (Dry)		1.9	
78+20	1.31 (Dry)		2.5	
78+25	1.10 (Dry)		2.1	
79+18	1.00 (Dry)	1.9 ft	1.8	
79+25	0.99 (Dry)	3.1 ft	1.9	
79+25a	1.32 (Dry)		2.5	
80+15	1.33 (Dry)		2.6	
80+25	1.06 (Dry)	1 ft	2.0	
81+04	1.20 (Dry)		2.6	
81+30	1.06 (Dry)	1.1 ft	1.9	
84+10	0.89 (Dry)	8.2 ft	2.1	**Heel Extension Required**
84+10a	0.88 (Dry)	8 ft	2.2	**Heel Extension Required**
85+57	0.96 (Dry)	4.7 ft	1.5	**Heel Extension Required**
86+34	0.99 (Dry)	3.0 ft	1.1	
86+65	1.01 (Dry)	1.5 ft	2.0	
87+15	1.01 (Dry)	1.5 ft	2.1	
87+65	1.01 (Dry)	1.5 ft	2.1	
88+15	1.01 (Dry)	1.5 ft	2.2	
88+60	1.01 (Dry)	1.5 ft	2.2	
88+69	0.99 (Dry)	2.5 ft	2.5	
89+15	1.00 (Dry)	1.5 ft	2.5	
89+66	1.00 (Dry)	1.4 ft	2.5	
89+73	1.03 (Dry)	1.7 ft	2.0	
90+15	1.01 (Dry)	1.4 ft	2.6	
90+65	1.00 (Dry)	1.4 ft	2.7	
91+02	1.23 (Dry)		1.6	
91+29	1.00 (Dry)	1.3 ft	3.0	
91+40	0.98 (Dry)	2.8 ft	2.0	
93+30	1.00 (Dry)	2.8 ft	1.7	

b. Uplift calculations are based on fifty percent relief well efficiency as supplied by geotechnical engineers. It is assumed that up to three feet of water will be allowed in the collector system to meet uplift requirements. Four manhole boxes fail to meet uplift criteria with three foot of standing water (Sta. 16+07, 84+10, 84+10a, and 85+57). Costs have been included for adding foundation heel extensions to each manhole. Because of the high probabilities of failure developed for the South Topeka floodwall and due to the relatively low cost of repair (<$25K), reliability curves were not developed for manhole uplifts.

A-5.6.5.3 Riverside Sluice Gates

a. The South Topeka unit has six riverside closure gates consisting of manually operated sluice gates located in gatewells riverside of the centerline of levee. A typical gate (Sta 51+80) is shown below.

FIGURE 4 – Typical South Topeka Closure Gate



b.

Table A-5-20 below summarizes reliability criteria findings. All results are computed with water to top of levee. Column two displays uplift factors of safety with no water in the outlet structure. Column three shows the level of water required in the structure to meet the minimum 1.1 required uplift factor of safety.

Table A-5-20: South Topeka Riverside Sluice Gates

South Topeka Riverside Sluice Gates with Water at Top of Levee					
Station	Uplift Factor of Safety (> 1.1 Req'd)	Water Req'd to meet 1.1 Uplift Factor of Safety (ft)	Strength Factor of Safety (> 1.5 Req'd)	Comments	Reliability
19+81	**See RCB Calculations**				
22+71	**Previously Abandoned in place with Grout**				
46+74	**See RCB Calculations**				
51+80	1.08	1 ft	Not Calculated	Limited Information	**Assumed 99.8 %**
88+69	Located in Floodwall (To be Replaced)				
91+02	Located in Floodwall (To be Replaced)				

c. Of the six closure gates studied, one was previously abandoned in place with grout. No further action is recommended for this structure. Two of the riverside gatewell closure structures are located in the pile founded floodwall. Significant concerns have been determined for the floodwall foundation and it is recommended the floodwall be removed and replaced. As a result, to facilitate construction of the floodwall, the two riverside gates will also need to be removed and replaced with gatewells. Reliabilities were not calculated for the two closure structures. Uplift calculations were performed for a fourth closure structure, but because of insufficient information, strength calculations were not conducted. Because only 1ft of water is required in the structure to meet uplift criteria with water to top of levee and no other closure structures studied exhibited strength concerns, the gate structure is considered sufficient and assigned a 99.8% reliability. A summary of calculations for the other two structures is included in the Reinforced Concrete Box portion of this report.

A-5.6.5.4 Reinforced Concrete Boxes

Two reinforced concrete boxes cross the South Topeka unit. Analysis results are based on water to top of levee with no water in the box. All minimum required factors of safety are met and a 99.8% reliability was assigned.

Table A-5-21: South Topeka Reinforced Box Culverts

South Topeka Reinforced Box Culvert with Water to Top of Levee					
Station	Uplift Factor of Safety (> 1.1 Req'd)	Water Req'd to meet 1.1 Uplift Factor of Safety (ft)	Strength Factor of Safety (> 1.5 Req'd)	Controlling Structural Mechanism	Assigned Reliability
19+81	1.3 (dry)	N/A	1.7	Floor Flexural Steel	99.8%
46+74	1.4 (dry)	N/A	4.3	Floor Flexural Steel	99.8%

A-5.6.5.5 Pump Stations

a. Four pump stations are located along the South Topeka Levee Unit. City Park pump station predates the Federal project and handles interior drainage. The Kansas Avenue pump station was constructed with the Federal Project (as-builts dated 1974) to pump intercepted flows from the collector system and relief wells. Morrell pump station predates the Federal project (possible original construction in 1947) and was modified in the Federal project to intercept collector system and relief well flows. Madison Street pump station was constructed by the City of Topeka, but was closely coordinated with the late '60s/early 70's Federal project to handle interior drainage.

b. Table A-5-22 below summarizes reliability criteria findings. All results are computed with water to top of levee. Column three displays uplift factors of safety with no water in the wet well. Column four shows the level of water required in the wet well to meet the minimum 1.1 required uplift factor of safety. Based on each pump stations individual pump station shutoff elevations, column five shows the actual minimum level of water likely to present in the well.

Table A-5-22: South Topeka Pump Stations

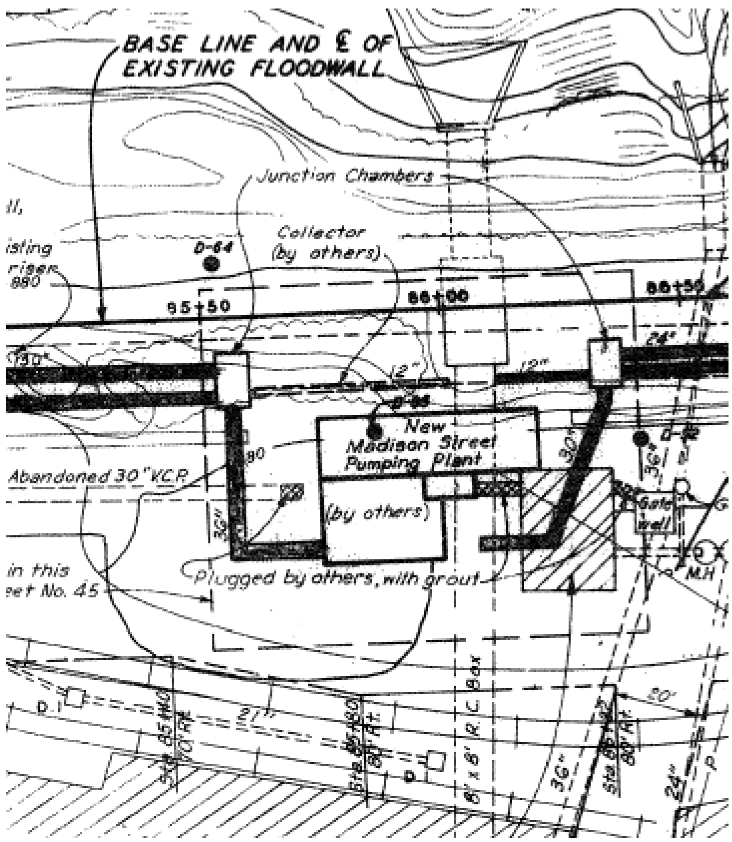
South Topeka Pump Stations with Water at Top of Levee						
Station	Station Name	Uplift Factor of Safety (> 1.1 Req'd)	Water Req'd to meet 1.1 Uplift Factor of Safety (ft)	Water Available (ft)	Strength Factor of Safety (> 1.5 Req'd)	Comments
68+85	City Park	1.48 (Dry)	N/A	N/A	1.57	**No Corrective Measures Necessary**
75+84	Kansas Avenue	1.2 (Dry)	N/A	N/A	0.90	**31% Prob. Of Failure: Wall Stiffener Required**
84+07	Morrell	0.87 (Dry)	4	No Information		**Assumed Sufficient No Corrective Measures Necessary**
86+00	Madison Street	1.0 (Dry)	1.5	3.5	1.52	**No Corrective Measures Necessary**

c. The City Park Pump Station meets uplift and strength requirements and a 99.8% reliability is assumed. At the Kansas Avenue pump station, the exterior foundation wall vertical steel fails to meet the required 1.5 factor of safety. Adding a mid-span stiffener wall will reduce the effective length by half, successfully reducing the edge plate moments to within an allowable range. Because of the high probabilities of failure developed for the South Topeka floodwall and due to the relatively low cost of repair (~ <$50K) for the pump station, a reliability curve was not developed for the Kansas Avenue Pump Station.

d. Uplift calculations for the Morrell Pump station are based on field measurements of foundation wall thicknesses and assumed floor thickness and super structure weights. Computations show 4ft of required in the wet well to meet uplift criteria. Pump station operation curves have not been found, but it is assumed 4ft of water is reasonable. No information is available for strength calculations but site visits revealed no foundation cracking or distress and reinforcing is assumed sufficient. At the time of plans and specifications preparation, these assumptions may require verification.

e. The Madison Street Pump Station meets uplift and strength screening criteria and a 99.8% reliability was assigned. The Madison Street Pump Station uplift factor of safety including 3.5ft of water in the wet well and a conservative value for skin friction (μ=0.15) was calculated as 1.2. The availability of 3.5 ft of water in the well is based on the existing pump shut off when water drops to 3.5ft.

FIGURE 5 – Madison Street Pump Station



A-5.6.5.6   Spread Footing Floodwall

Two sections of transition floodwalls for stoplog gaps are located along the South Topeka Unit from stations 1+34 to 3+33 and 50+72 to 51+70 with typical exposed stem wall heights of 9ft. Wall section properties with water to top of levee are summarized below. All factors of safety are met and a 99.8% reliability was assigned.

Table A-5-23: South Topeka Spread Footing Floodwall

South Topeka Spread Footing Floodwall with Water to Top of Levee						
Station	Wall Cross Section	Overturning % Base in Compression (> 25% Req'd)	% Bearing Allowable (Demand/Capacity) (150% Max Increase)	Sliding Factor of Safety (>1.3 Req'd)	Wall Strength Factor of Safety (>1.5 Req'd)	Comments
1+45	Sec A	72.0 %	23.6 %	2.22	1.7	99.8 % Reliability
51+00	Sec A	70.6 %	38.6 %	2.37	2.05	99.8 % Reliability

A-5.6.5.7 Levee Opening Closure Structures

a. Two stoplog closure structures are located in the floodwall reaches and were analyzed with water to top of wall. Results are summarized below.

Table A-5-24: South Topeka Levee Opening Closure Structures

South Topeka Closure Structures with Water to Top of Wall					
Station	Overturning % Base in Compression (> 25% Req'd)	% Bearing Allowable (Demand/Capacity) (150% Max Increase)	Sliding Factor of Safety (>1.3 Req'd)	Wall Strength Factor of Safety (>1.5 Req'd)	Comments
2+30	91.2 %	27.3 %	2.3	1.5	99.8 % Reliability
51+20	Stoplog gap integrated into RR Bridge Abutment. Stability not an issue.			4.26	99.8 % Reliability

b. Both stoplog gaps meet all deterministic criteria and a 99.8 % reliability was assumed.

A-5.6.5.8 Pile Founded Floodwall

a. The timber pile founded floodwall extends from approximately station 74+41 to 93+86. The concrete wall and the timber piles were analyzed and found to be reliable for strength. However, the timber piles were found to have unacceptable reliability due to their soil based axial capacity.

The complete discussion of this floodwall can be found in Chapter A-6. Background information, discussion of structural and geotechnical evaluations, and the results for the wall are included. See Chapter A-6 Exhibit 11 for the reliability results.

b. A potential timber pile axial failure could result in excessive floodwall deflections, water infiltration through opened wall joints, scour around the openings, and rapid wall failure. Based on review of existing information and preliminary analysis, it has been concluded that the South Topeka floodwall is unreliable and cannot reasonably be made reliable by modifications to the existing structure. Consequently, a new floodwall will be required to replace the existing floodwall to address reliability concerns. Due to real estate constraints and the extensive landside underseepage collector system, it is recommended to construct the new wall in the same footprint as the old wall. See Chapter A-6 for a discussion of the required sequence for in-line replacement.

A-5.6.6 Waterworks Unit

a. The Waterworks Unit is located along the right bank of the Kansas River to provide flood risk management for the western side of Topeka. The levee unit includes 1,998 feet of earthen levee and 1,662 feet of floodwall. The project was designed in 1957 and constructed during 1959.

b. Waterworks structures considered for this study included one gatewell closure, one sandbag closure gap, four stoplog closure gaps, and fifteen different floodwall cross sections making up the floodwall.

A-5.6.6.1 Gatewell Closure Structures

One Waterworks gatewell was analyzed for water to top of structure. All required factors of safety were met and a 99.8% reliability was assigned.

Table A-5-25: Waterworks Gatewell Reliability

Waterworks Gatewell with water to top of levee				
Station	Uplift Factor of Safety (> 1.1 Req'd)	Strength Factor of Safety (> 1.5 Req'd)	Controlling Structural Mechanism	Assigned Reliability
1+90	1.2 (dry)	1.51	Wall Shear Strength	99.8%

A-5.6.6.2 Spread Footing Floodwall

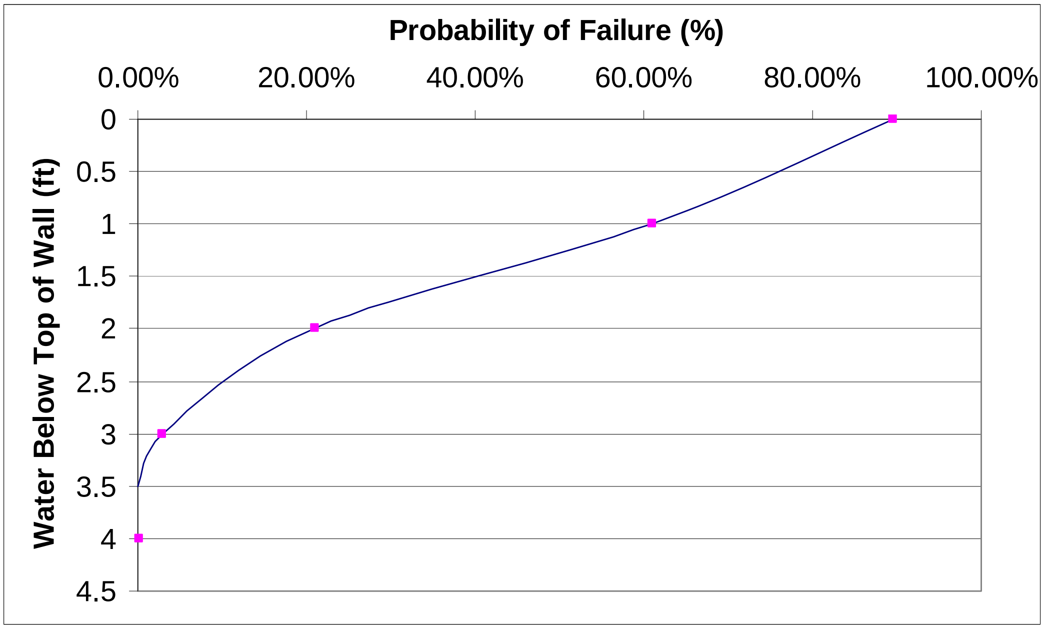
a. The 1600 linear foot spread footing floodwall has an average exposed wall height of between eight and twelve feet. A sheetpile cutoff wall is embedded in the heel of the floodwall along with a relief well system. Wall section properties are summarized below.

Table A-5-26: Waterworks Spread Footing Floodwall

Waterworks Spread Footing Floodwall with water to top of levee						
Station	Wall Cross Section	Overturning % Base in Compression (> 25% Req'd)	% Bearing Allowable (Demand/Capacity) (150% Max Increase)	Sliding Factor of Safety (>1.3 Req'd)	Wall Strength Factor of Safety (>1.5 Req'd)	Comments
0+58	Sec P	100 %	46.0 %	3.13	1.82	99.8 % Assigned Reliability
1+50	Sec N	69.4 %	81.6 %	0.78	1.72	2 ft of Additional Fill Req'd Behind Floodwall to meet sliding requirements
2+75	Sec M	85.7 %	81.6 %	1.37	1.76	99.8 % Assigned Reliability
6+64	Sec B	93.4 %	65.2 %	1.28	2.32	99.8 % Assigned Reliability
8+50	Sec L	92.0 %	65.0 %	2.42	1.71	99.8 % Assigned Reliability
9+30	Sec K	100 %	53.0 %	1.86	2.25	99.8 % Assigned Reliability
10+70	Sec H	100 %	43.2 %	1.67	1.53	99.8 % Assigned Reliability
11+50	Sec G	94.9%	53.4 %	1.87	2.97	99.8 % Assigned Reliability
12+00	Sec U	84.9 %	93.6 %	1.69	1.38	99.8% Calc. Reliability
12+58	Sec T	85.9 %	84.5 %	1.55	1.58	99.8 % Assigned Reliability
13+20	Sec F	94.2 %	53.6 %	1.15	2.14	2 ft of Fill Behind Floodwall
14+50	Sec A	96.0 %	52.2 %	1.05	2.15	2 ft of Fill Behind Floodwall
15+10	Sec E	95.3 %	53.2 %	1.06	2.11	2 ft of Fill Behind Floodwall
15+50	Sec S	90.3 %	52.9 %	0.96	2.14	2 ft of Fill Behind Floodwall
16+15	Sec C	89.7 %	53.9 %	0.97	2.1	2 ft of Fill Behind Floodwall

b. Six analyzed floodwall cross sections failed to meet sliding stability. Section N (Station 1+50) was determined to be the critical wall cross section (lowest factor of safety for sliding) for which a probability of failure was calculated. The risk and uncertainty analysis (using the procedure described earlier in this chapter) yielded the curve shown below. The displayed data points represent the calculated probabilities versus water elevation while the continuous line represents values input into the HEC-FDA model.

FIGURE 6 - Waterworks Floodwall Probability of Failure



c. Subsequent analysis of the critical section revealed two feet of additional fill behind the wall would be sufficient to meet minimum sliding requirements. Based on site visits two foot of fill extended for a distance of 5ft from centerline of floodwall and then tapered at a 1 on 3 slope can easily be placed behind the floodwall. Fill was assumed required between stations 0+78 to 7+00 and 10+00 to 16+50 for a total of 1272 linear feet. At time of plans and specifications more exact stationing will be determined.

A-5.6.6.3 Levee Opening Closure Structures

a. Four stoplog closure structures in the Waterworks floodwall were analyzed with water to top of wall. Results are summarized below.

Table A-5-27: Waterworks Levee Opening Closure Structures

Waterworks Closure Structures with water to top of wall					
Station	Overturning % Base in Compression (> 25% Req'd)	% Bearing Allowable (Demand/Capacity) (150% Max Increase)	Sliding Factor of Safety (>1.3 Req'd)	Wall Strength Factor of Safety (>1.5 Req'd)	Comments
11+10	Sandbag Closure Gap, No Deficiencies Observed				
0+58	69.4 %	69.0 %	0.78	1.73	Stoplog Gap has been filled Wall Cross Section same as Sec. N
9+30	75.7 %	85.9 %	1.04	1.7	Wall Alignment Restrains Sliding Potential
13+07	68.6 %	90.2 %	0.8	1.3	Minimal Gap, Backfill behind gap sidewalls to address sliding stability. 99.8% Calculated Strength Reliability.
15+95	56.7 %	86.8 %	0.75	1.6	Backfill behind gap sidewalls to address sliding stability.

b. The stoplog gap at station 0+58 was filled in the past, and has a similar geometry to floodwall cross Section N.   Two feet of additional will be added behind the wall to address stability concerns. The stoplog gap at station 9+30 is configured such that an adjoining floodwall monolith forms a ninety degree at the end of the stop log monolith, effectively acting as a stiffener preventing lateral movement of the wall. The gap at 13+07 is of such small size that sufficient length of approach wall is available to allow for placement of 2 feet of backfill behind the floodwall monolith. The Gap at 15+95 is located in an L-shaped monolith, also allowing for enough length to effectively place two additional feet of fill behind the wall. Because Section N had a lower sliding factor of safety then the gap at station 13+07, the probability of failure curve developed for Section N was used to define the reliability of the entire wall, including all the stoplog gaps located in the wall.

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Chapter A-6

SOUTH TOPEKA FLOODWALL

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A-6 SOUTH TOPEKA – PILE FOUNDED FLOODWALL (Sta. 74+41 to 93+86)

A-6.1 FLOODWALL BACKGROUND INFORMATION

The existing South Topeka pile-founded floodwall was constructed in 1939. The wall is a concrete cantilever type supported on creosote treated timber piles with a 15-foot steel sheet pile cutoff under the riverside edge of the base. The floodwall is approximately 2,000 feet long and extends 11 to 13 feet above the landside surface. This floodwall replaced a pile founded floodwall constructed in 1908. 1938 construction drawings provide detailed cross-sections of the floodwall; however, the drawings do not explicitly provide the pile diameter, pile depth, or material properties.

A-6.1.1 Floodwall "Roofing"

a. Portions of the original 1908 floodwall were "encompassed" (buried) in the 1939 levee construction. Construction of the City Park Pump Station (Sta. 68+84) in 1956 uncovered a portion of the buried floodwall and discovered significant "roofing" under the old pile foundation system, suggesting the possibility of void areas and seepage paths in the levee (Exhibit 3).

b. Construction photos (possibly of the Kansas Avenue Pump Station, ~1968) and three Corps of Engineers' test pit excavations mentioned in the 8 September 1964 meeting minutes reference similar roofing concerns for the 1939 floodwall (Photo 1 & Exhibit 1).

Photo 1 – Floodwall "Roofing"

c. To address the underseepage concerns associated with the roofing issue, a 23-24 March 1964 meeting suggested the ideas of either a new/reconditioned toe drain or some form of subsurface grouting (either a grout curtain wall or grouting the roofing void) (Exhibit 4). By the time of a 15 November 1966 meeting a series of seepage interceptors, relief wells and pumped wells were recommended to address wall underseepage and nearby basement uplift concerns (Exhibit 5).

A-6.1.2 Floodwall Sheetpile Cutoff Wall and Uplift Assumptions

The 8 September 1964 meeting refers to the sheetpile condition and size. "The sheet pile wall is reasonably impervious. The interlocks being corroded and filled in the past 25 years." (Exhibit 1).

Uplift due to full river head was assumed at the sheet piling and static head from the bottom of the base to the ground surface was assumed at the landside toe, plus 2 additional feet of head to provide for losses into the toe drain and for discharging slightly above ground surface manholes in the toe drain. Based on the previously described roofing void, uplift under the base from the sheet pile to the landward toe is assumed equal. Since the void is in direct communication with the toe drain, pressure in the void cannot be higher then the landside uplift described above.

A-6.2 STRUCTURAL ANALYSIS

A-6.2.1 Floodwall Analysis Assumptions

a. Several assumptions were made for the analysis based on the drawings, correspondence, historical construction practices, and engineering judgment. The assumptions were as follows:

- **12 in. Pile Diameter.** This assumption was largely based on drawn to scale construction drawing sections. In addition, it is supported by photographic evidence of piles exposed during a 1960's excavation (Photo 2) and is consistent with the expected 10"-12" range for 1930's era construction.
- **25 ft. Pile Length.** Typical driven depths for timber piles of this era were anywhere from 20 to 35 feet. The manufactured length of timber piles was documented in the range of 30 to 60 feet. The construction drawings note a minimum penetration of 20 ft, and therefore, 25 ft was assumed.
- **2500 psi Concrete and 33 ksi Reinforcing Steel.** Based on historical material information.
- **Southern Pine Piles.** Southern pine was chosen as a typical pile species and to provide middle ground values of common pile species.
- **No degradation of the piles.**
- **Sheet Pile Loading.** The sheet piles were assumed to carry no load.
- **Pinned pile heads.**

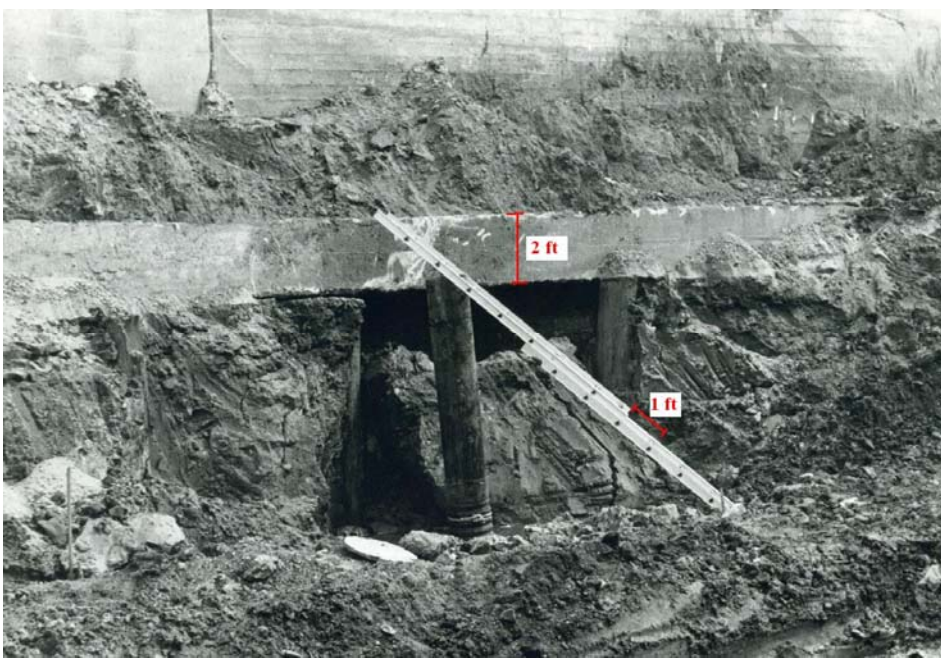

Photo 2 –Photo with exposed pile

A-6.2.2 Wall and Pile Load Analysis

Four floodwall cross sections were examined representing all the floodwall section geometries. A Mathcad sheet was used to determine the loads acting on the pile group and to verify the pile loads calculated using the Corps' Computer-Aided Structural Engineering (CASE) computer program Pile Group Analysis (CPGA). The loads on the pile cap or pile group were input into CPGA to provide pile loads and check the piles for combined axial and bending loads. In addition, Mathcad was used to check the capacity of the stem and pile cap.

A-6.2.3 Reliability Analysis Methodology

The reliability of a typical pile founded floodwall would ideally be based on the following evaluations:

a. Concrete Strength
    1) Concrete compressive strength
    2) Concrete shear strength
    3) Reinforcing steel strength
b. Piles
    1) Pile Normal Stress (Combined Bending & Axial) & Pile Shear Stress
    2) Axial Capacity (soil based)
    3) Pile Lateral Deflection

For each mechanism listed above, the reliability is usually analyzed in the method described in the Chapter A-5.2 to 5.4. The following paragraphs explain how these failure modes were addressed specifically for this floodwall.

a. Concrete Strength
See Chapter A-5

b. Pile Normal Stress (Combined Bending & Axial) & Pile Shear Stress

The pile structural capacity was evaluated initially based on the traditional allowable load methodology using CPGA to compute axial, horizontal, and bending loads on the piles. The analysis found that shear loads exceeded the allowable shear stresses for one section. By exceeding the allowable stresses, the piles did not meet the factor of safety inherent to allowable loads which suggested a reliability concern. Therefore, a reliability analysis was performed.

For reliability, failure was defined as a Factor of Safety (capacity to demand ratio) less than one as described in Chapter A-5. Capacity was based on LRFD reference strength values. Strength reduction values (ϕ) were not used, but design adjustment factors and a time effect factor were included (see paragraph material properties paragraph below for more information). The demand, D, was be computed using unfactored loads.

<u>Material properties for pile stress reliability calculations:</u>

- Reference shear strength of 300 psi was taken from LRFD Manual for Wood Construction. The values in the LRFD manual are derived according to the principles of ASTM D5457. Strength reduction values were not used.
- The LRFD reference strength was multiplied by a time effect factor, λ, of 0.9 to account for the load duration. This value was chosen for a flood to represent a

length of loading that falls somewhere between a short duration load (wind, λ= 1.0) and an intermediate load duration (snow, λ= 0.8).

- Design adjustment factors for shear, C_t and C_u (for temperature and treatment), were equal to one.
- The Standard Deviation was found using the standard deviation from ASTM D 2555 Clear Wood Strength Values factored by the ratio of LRFD Strength over Clear Wood Strength from ASTM D 2555.

c. Axial Capacity

The ultimate axial capacity was computed and compared to the unfactored load to produce a factor of safety. A factor of safety of less than one was considered failure. An in-depth discussion and results for this failure mechanism is given in section A-6.3.

d. Lateral Deflection

The lateral deflection of the floodwall was not investigated due to the limited knowledge of foundation parameters and the piles as well as the poor results of the axial capacity analysis. A lateral deflection analysis would likely show significant deflections for the loading condition with water to the top of the wall.

A-6.2.4 Results

Results are provided in section A-6.4.

A-6.3 GEOTECHNICAL ANALYSIS

A-6.3.1 Axial Capacity of Timber Piles for Floodwall

A-6.3.1.1 Introduction

An analysis of the existing pile founded floodwall which comprises part of the flood protection in the South Topeka unit was investigated for the axial load capacity of the existing timber piles. The relief wells installed in 1969/1970 were considered in the calculation of underseepage pressures for the loading condition analyzed.

A-6.3.1.2 Foundation conditions

The floodwall comprises the flood protection for South Topeka from approximately station 74+40 AH to the downstream end of the protection at approximately station 94+00. As noted in boring logs and previous analyses, the floodwall is founded on a significant amount of highly variable fill materials over a relatively thin natural blanket. In fact, over some reaches the natural blanket is believed to be non-existent. The fill and blanket materials range in thickness from approximately 20 to 25 feet. The heterogeneous fill consists of everything from cinders and bricks to lean clays to organic materials. The top of bedrock is relatively constant through this reach at elevation 810, so the aquifer varies in thickness from approximately 45 to 50 feet. See **Exhibit 6** for excerpts from the November 1969 Construction Drawings that provide a plan and profile along the floodwall.

A-6.3.1.3 Underseepage Control Modifications

Underseepage control was added to the project in the late 1960's and early 1970's, apparently due to historic underseepage problems during the 1951 flood and may also be related to the revised protection design discharge (which effectively raised the level of protection to the top of the wall). The underseepage control included relief wells and a buried collector system along the entire reach of the wall. There are a total of 27 fully penetrating relief wells. The buried collector system is located at the landside toe of the floodwall and is intended to intercept any seepage through any pervious fill material that may exist in the heterogeneous fill under the floodwall. Both the wells and the collector system drain underground to the Madison Street pump plant which was constructed at the same time period. See **Exhibit 6** for relief well locations and buried collector system details.

A-6.3.1.4 Deterministic Axial Pile Capacity Analysis for Design Loading

a. Analyzed Sections

Evaluation of the foundation identified a reach from station 83+00 to 87+00 that had a relatively consistent 15 feet of CL and ML material overlying 10 feet of OL and other organic materials. These materials were considered to comprise the blanket. This reach was considered to be the critical reach with respect to the axial capacity of the piles because this is the thickest reach of blanket and due to the relative low strength of the OL materials compared to other material comprising the fill and blanket. In addition, no relief wells exist between stations 81+30 and station 86+65 so the underseepage pressures in the aquifer are also the highest through the reach of the South Topeka floodwall. A second, more typical section that is similar to most of the rest of the floodwall foundation was evaluated at station 89+00 for comparison. For station 89+00 a 25-foot thick blanket comprised of only CL and ML materials was estimated. The blanket and fill materials at both sections were assumed to be impervious relative to the aquifer, though in some areas this may not be the case.

b. Hydraulic Grade Lines

Because of the sand aquifer, the hydraulic grade line is required at the sections being analyzed to determine the pore pressures developed in the foundation soils during the design loading condition. The hydraulic grade lines at the two sections were computed taking into account the relief wells by using the method of image wells. The approach was in accordance with EM 1110-2-1914, Design, Construction, and Maintenance of Relief Wells. The section at station 84+50 is located midway between two wells spaced over 500 feet apart, and the section at station 89+00 is located in a reach of wells at the more common spacing of about 50 feet. The excess head above the landside toe ground surface at station 84+50 was computed to be 10 feet and at station 89+00 to be 6 feet. See **Exhibit 7** for plots from the analysis.

c. Effect of Buried Collector

In addition to evaluating the effects of the relief wells, a rough flow net was drawn to determine the effect of the buried collector system on the hydraulic grade line since it is in contact with the aquifer. The flow net was evaluated only qualitatively, but revealed only a minor effect on the overall flow regime, and the effect was localized to the buried collector location. Because of this, the effect of the buried collector was ignored. A copy of this work is not included in this document.

d. Timber Piles

The exact details of the wooden piles are unknown. Through research, old photographs, the original construction drawings and evaluation of construction practices in the 1930's, a reasonable estimate of what was constructed was determined. Research showed that timber piles were typically manufactured in 30 to 60 foot lengths, had a top diameter of about 12 inches, and tapered about 0.1 inches per foot. It had been previously deduced during the feasibility study that the wooden piles were 25 feet in length. The modulus of elasticity for the piles was taken as 1.5×10^6 psi.

e. Soil Parameters

The soil parameters used for this analysis were taken from previous work in the feasibility study except for the organic blanket material which was estimated based upon typical values. A detailed discussion on the evaluation of available data for the undrained shear strength of the CL-ML material can be found in section A-6.3.3. The following table summarizes the parameters used in the analysis.

Table A-6-1: Soil Parameters

Soil	Saturated Unit Weight	Shear Strength			
		Undrained		Drained	
	γ (pcf)	c (psf)	ϕ (degrees)	c' (psf)	ϕ' (degrees)
CL-ML	110	600	0	0	24
Organics	100	400	0	0	17
Sand	115	N/A	N/A	0	33

The effective unit weight of the blanket materials was computed using the hydraulic grade line at the bottom of the blanket with water to the top of the wall for both sections, and the results are as follows. Calculations are provided in **Exhibit 8**.

Table A-6-2: Effective Unit Weights of Soils under Steady Seepage Conditions

Soil	Effective Unit Weight	
	Station 84+50	Station 89+00
	γ (pcf)	γ (pcf)
CL-ML	22.6	32.6
Organics	12.6	N/A
Sand	52.6	52.6

f. **Analysis**

The axial capacity analysis of the piles was performed using the Corps of Engineers EM 1110-2-2906, Design of Pile Foundations. Because of the taper of the piles, the capacity of the pile in tension was reduced by 50% from the computed capacity in compression. Allowable axial capacities were computed from guidance also found in EM 1110-2-2906, however one could easily argue that due to the significant unknowns of the foundation materials that the required factors of safety should be even higher. All hand calculations are provided in **Exhibit 8**. The results of the analysis are provided in

Table A-6-3 and table A-6-4. It is unknown what the original design criteria were. The drained loading condition is the controlling loading condition. The wall section at station 83+75 (type B) was used to evaluate the reach between station 83+00 and 87+00 (It should be noted that the station is referred to as 84+50 in the exhibits, but was changed to 83+75 in the text to correlate to the actual wall section used in the analysis). It was considered to be the most critical wall section because the footing was constructed at a higher elevation than other wall types through this reach. Given the thickness of the blanket and fill was assumed to be constant through this reach, these piles would have the shortest lengths in the aquifer and subsequently the lowest axial capacity. In addition, this location has the highest hydraulic gradient in the selected reach.

Table A-6-3: Axial Capacity for Undrained Loading Conditions

Station	Ultimate Axial Capacity (kips)		Allowable Axial Capacity (kips)			
			Unusual Loading (FS = 2.25)		Extreme Loading (FS = 1.7)	
	Compression	Tension	Compression	Tension	Compression	Tension
83+75	39.1	19.6	17.4	8.7	23.0	11.5
89+00	53.8	43.7	23.9	12.0	31.6	15.8

Table A-6-4: Axial Capacity for Drained Loading Conditions

Station	Ultimate Axial Capacity (kips)		Allowable Axial Capacity (kips)			
			Unusual Loading (FS = 2.25)		Extreme Loading (FS = 1.7)	
	Compression	Tension	Compression	Tension	Compression	Tension
83+75	28.0	14.0	12.4	6.2	16.5	8.2
89+00	43.7	21.8	19.4	9.7	25.7	12.8

It should be noted that the conditions between station 83+00 and 87+00 could be marginally improved with the installation of additional relief wells to reduce the excess pore pressures developed during the design loading condition.

g. Applied Loads

Loads on the piles were computed using the computer program CPGA and are shown in Table A-6-4. the most landward pile has the highest axial compression load and the most riverward pile has the highest axial tension load.

Table A-6-4: Maximum Loads in Piles

Station	Max Axial Load in Compression (kips)	Max Axial Load in Tension (kips)	Lateral Load (kips)	Maximum Bending Moment (in-k)
83+75	23.0	4.0	9.0	202
89+00	25.0	3.0	9.0	220

h. Results

The drained loading condition gave the lower axial capacity for the piles at both sections. The maximum axial load on the floodwall at station 89+00 meets the minimum factor of safety for the extreme loading condition, so the wall is probably acceptable at this location. The maximum axial load on the floodwall at station 83+75 does exceed the minimum factor of safety for the extreme loading condition in compression (FS = 1.2), and approaches the ultimate capacity of the pile. The tension capacity, however, is acceptable. Because the results of this analysis show the wall at station 83+75 does not meet the minimum factor of safety for the design loading condition with water to the top of the wall, a reliability analysis was performed on a single pile for axial loading at this location.

A-6.3.2 Reliability Analysis for Axial Capacity

A-6.3.2.1 Design Parameters

The design parameters varied in the reliability analysis included shear strength of soils, blanket thickness, and pile length. Initially the soil unit weight was also varied in the analyses; however it was found to have a very small effect and was subsequently removed. Because there was no data to determine either expected values or statistical parameters, the values determined for the deterministic analysis were considered expected values. Published values for coefficient of variation (COV) from ETL 1110-2-561 Table D-2, dated July 2005, were utilized for the statistical parameters, except the published coefficient of variation values were judgmentally increased to account for the additional uncertainty of the input data. Published values for the coefficient of variation were not available for blanket thickness or pile length, so these parameters were also estimated based upon judgment. Table A-6-5 lists the parameters used in the analysis.

Table A-6-5: Parameters used in the Reliability Analysis

Parameter	Expected Value E[V]	COV	E[V](1-COV)	E[V](1+COV)
Sand Shear Strength	33^{o}	15%	28^{o}	38^{o}
CL-ML Shear Strength	600 psf	60%	240 psf	960 psf
Organic Soil Shear Strength	400 psf	60%	160 psf	640 psf
CL-ML Blanket Thickness	15 feet*	25%	11.25 feet	18.75 feet
Organic Soil Blanket Thickness	10 feet	25%	7.5 feet	12.5 feet
Pile Length	25 feet	20%	20 feet	30 feet

*The top of the pile is a constant 7.5 feet below the ground surface.

A-6.3.2.2 Analysis Method

The pile reliability analysis procedure started at the design loading condition of water at the top of the floodwall, and for each subsequent loading condition the water level was decreased in 1-foot intervals until the computed probability of failure was negligible. The general analysis procedure is as follows:

For each water loading condition the hydraulic grade line was determined considering the effects of the relief wells.

The effective unit weight of the blanket materials was computed using the excess pore pressures determined from the hydraulic grade line. It should be noted that the hydraulic grade line for each loading condition was computed only for the expected value of blanket thickness, and not recalculated for the case of varying the blanket thicknesses. The effect was considered to be minor.

For each loading condition a series of axial capacity calculations were performed, with one run using expected values and the remaining runs each varying one parameter by plus or minus one standard deviation. Again, axial capacity calculations were made utilizing EM 1110-2-2906.

Including the expected values, this amounted to 13 sets of calculations for each loading condition. Miscellaneous hand calculations are provided in **Exhibit 9** and the axial capacity runs for all the loading conditions are provided in **Exhibit 10**. A total of nine analyses for each of the loading conditions were performed in this manner, from water at the top of the wall to water 8 feet below the top of the wall.

A-6.3.2.3 Analysis Results

The reliability relationship vs. loading condition from the above analysis was plotted and is provided in **Exhibit 11**. The calculations show nearly a 45% probability of failure for the loading condition of water to the top of the wall. The probability of failure drops to 0% with water approximately 5 feet below the top of the wall. The largest effect on reliability for the analysis was the pile length.

A-6.3.3 South Topeka Feasibility Study Soils Investigation Review

A-6.3.3.1 Introduction

After completion of a reliability analysis for the axial capacity of a single timber pile, some recent soils data was discovered. This data was evaluated to validate assumptions in the previous analysis, namely the shear strength of the blanket material. Documentation of this evaluation is provided in **Exhibit 12**.

A-6.3.3.2 South Topeka Information

Only one boring was drilled in the South Topeka Levee Unit, DU-89, located at station 64+00 in the levee reach of the unit. It was drilled through the crest of the levee at this location. The levee is approximately 9 feet tall at this location. There was very limited data from this boring, Atterberg limits and field torvane tests.

1) Atterberg Limits

The soils tested from this boring had liquid limits ranging from 22 to 47, and plasticity indices ranging from 5 to 31. All soils classified as CL or ML to a depth of 30 feet.

2) Field Torvane Tests

Three torvane tests were performed on undisturbed samples in the field, which provide a direct indication of undrained shear strength. The samples were taken under the centerline of the levee section of South Topeka. The results ranged from 800 psf to 1800 psf, with an average of 1300 psf. These samples came from under the footprint of the levee and through consolidation have probably increased the shear strength since construction of the levee. These field test results provide a relative measure of shear strength, however probably overestimate actual blanket strengths at the floodwall. These data probably also reflect an increase in shear strength due to the consolidation of the blanket materials under the weight of the levee.

A-6.3.3.3 Other Topeka Information

The subsurface investigation was not limited to the South Topeka unit for the feasibility study, and significant information was collected for the other units in the Topeka flood protection system. In conjunction with this investigation was laboratory testing of the soils. An attempt was made to only compare testing of materials similar to those found in the South Topeka unit.

a. Unconfined Compression Tests

Three unconfined compression tests were performed for the study, one from the Soldier Creek unit and two from the Oakland unit. The Soldier Creek unit is on a tributary to the Kansas River and on the North side of the Kansas unit. The Oakland unit is on the South side of the Kansas River and adjacent to the South Topeka unit to the East. The undrained shear strengths from the tests ranged from 540 psf to 910 psf, with an average of 725 psf. It should be noted, however, that the two samples from the Oakland unit were CH material.

b. Cone Penetration Test (CPT) Investigation

In addition to traditional drilling and sampling, numerous cone penetration holes were pushed in the Oakland unit. Pile capacity can be directly obtained from CPT data, unfortunately though the majority of the holes were only pushed 15 to 20 feet deep. The blanket in the reach being evaluated was also much thinner than in the South Topeka reach, on the order of 5 to 12 feet thick. Due to the limited depth of the investigation, tip resistance could not be determined, so the evaluation was limited to sleeve friction in the blanket. The evaluation was limited to a reach between station 80+00 and 105+00. This part of the levee is located adjacent to the river similar to the Topeka unit.

There are two methods to compute the ultimate skin friction for a pile from CPT sleeve friction data published by the Federal Highway Administration, the Nottingham and Schmertmann method and the Laboratoire des Ponts et Chaussees (LPC) method. The only difference between the methods is the former applies an α' factor to the measured sleeve friction and the latter does not. The equation for the Nottingham and Schmertmann method is:

$$Q_s = \alpha' f_s A_s$$

Q_s = ultimate skin friction resistance (pounds)
α' = ratio of the pile shaft resistance to cone sleeve friction (from figure 9.23)
f_s = sleeve friction measured from the CPT test (pounds per square foot)
A_s = surface area of pile (square feet)

This equation is similar to the ultimate undrained pile capacity for a cohesive soil.

$Q_s = \alpha S_u A_s$

α = adhesion factor
Su = undrained shear strength (pounds per square foot)

A comparison was even made between α' and α for the two methods, and the numerical values are very similar (see **Exhibit 12)**. Since the ultimate pile capacities should theoretically be the same, it follows that the measured sleeve friction f_s and the undrained shear strength S_u can be directly compared. From the CPT pushed evaluated, the measured sleeve friction values in the blanket ranged from 150 to 730 psf with an average value of 435 psf. The expected undrained shear strength value used in the reliability analysis was 600 psf. If the comparison was made using the LPC method (no reduction factor), the results would probably agree better.

A-6.3.3.4 Soils Investigation Review Conclusion

Based upon the investigation of the recent field and test data for Topeka, it is concluded that the undrained shear strength used for the reliability analysis is reasonable. The strength may be on the low side of and expected value if actual test data were available, however the difference would not likely be enough to significantly change the results of the analysis.

A-6.4 OVERALL RESULTS

The results shown in Table A-6-6 are based on calculations with water to the top of the wall. The given factors of safety are based on unfactored loads and expected strength values, and the probabilities of failure are based on calculations including the variability of those values.

Table A-6-6: Results with Water to Top of Wall

Station		Pile Strength (Meets Allowable?)		Concrete Strength (FS)	Axial Capaicty (FS)	Comments
Begin	End	Normal Stress	Shear Stress			
80+42	83+78	Y	Y	1.7	1.2	45% (axial capacity)
84+20	84+62	Y	Y	1.9	See Note 3	See Note 3
84+62	85+04	Y	N	1.8		
92+60	93+86	Y	Y	1.8		

Table A-6-6 Notes:

1. Screening Critieria:
   a) Pile Strength: Allowable Loads $\geq$ Service Loads
   b) Concrete Strength: FS $\geq$ 1.5
   c) Pile Axial Capacity: FS $\geq$ 1.7
2. Reliability analyses were run only for those failure mechanisms that did not meet the screening criterion.
3. Based on load, soil conditions, and top of pile elevations, 80+42 to 83+78 was found to be the critical case for axial capacity without requiring analyses at other locations. The 45% reliability based on axial capacity noted for 80+42 to 83+78 was found to be the lowest for all failure mechanisms considering all locations. This governing reliability was provided to represent the whole wall in the economics analysis.

A-6.4.1 Uncertainty

Based upon the above analysis it appears that the existing floodwall at station 83+75 in the South Topeka unit has a significant probability of failure at the design loading condition. This is somewhat contingent, however, upon the accuracy of the parameters used in the analysis. The pile length was the most significant parameter in the analysis, and better information concerning the length would have benefited the analysis greatly. This is also contingent upon the idea that overloading of a single pile in a pile group leads to failure. A pile deformation analysis should accompany this work to estimate the magnitude of wall movement due to the applied loads, as a barometer for the measure of failure. Due to the limited knowledge of the soil parameters, however, an analysis of this type would probably not be useful at this time.

It is strongly recommended for PED that better information be obtained. The investigative effort should include a comprehensive drilling, sampling and testing program to better define the geotechnical parameters and subsurface conditions. In addition to that, a field investigation to determine the condition and length of the existing piles should be implemented. It is likely that the timber piles are nearing the end of their dependable life based upon the environment. This fact should be seriously considered in any future analysis.

A-6.4.2 Life Expectancy of Timber Piles

The existing piles for the floodwall are creosote impregnated timber piles of unknown specie. The Timber Piling Council suggests that the life expectancy of a treated timber pile partially above the groundwater is 100 years or longer (http://www.timberpilingcouncil.org/durability.html). The flood protection at South Topeka is at least 70 years old. For this evaluation of existing conditions it was assumed the piles were in perfect condition although this is unlikely.

A-6.4.3 Floodwall Remedy

a. A potential timber pile axial failure could result in excessive floodwall deflections, water infiltration through opened wall joints, scour around the openings, and rapid wall failure. Computed probabilities of failure range from a maximum of 45% with water at the top of the wall to essentially 0% with water 5 feet below the top of wall. It has been concluded that the South Topeka floodwall should be replaced rather than modified because of the high cost associated with adding a row of piles and the timber pile life expectancy. Consequently, a new floodwall will be required to replace the existing floodwall to address reliability concerns.

b. Because of real estate constraints and the extensive landside underseepage collector system, it is recommended to construct the new wall in the same footprint as the old wall. Flood risk management must be maintained during construction of the new floodwall. Cost estimates for the new wall are based on the following construction sequence.

- Stockpile sufficient fill material on site or within access to fill four floodwall monolith openings. (Approximately ~ 5500 cy)
- Demolish one monolith section (~84ft) to allow ease of riverside access. This monolith will be rebuilt at the completion of the project.
- Construct riverside construction and haul road to serve as working platform.
- Demolish three additional floodwall monoliths.
- Drive foundation piles, form and place the two monolith pile caps.
- The following five sequential construction steps will be repeated until the length of the wall has been replaced. See Figure 2 to match the step number with the corresponding monolith.

1. Construct floodwall stem (completing monolith)
2. Demolish next floodwall monolith. (No more then four monoliths will be open at any one time, the first monolith open to allow haul road access and three monoliths in the line of construction sequence.)

3. Drive pile foundation system. (There is always a separation of at least one monolith (~84ft) between piles being driven and freshly poured “green” concrete.)
4. Pour monolith pile cap.
5. Repeat Steps 1-4.

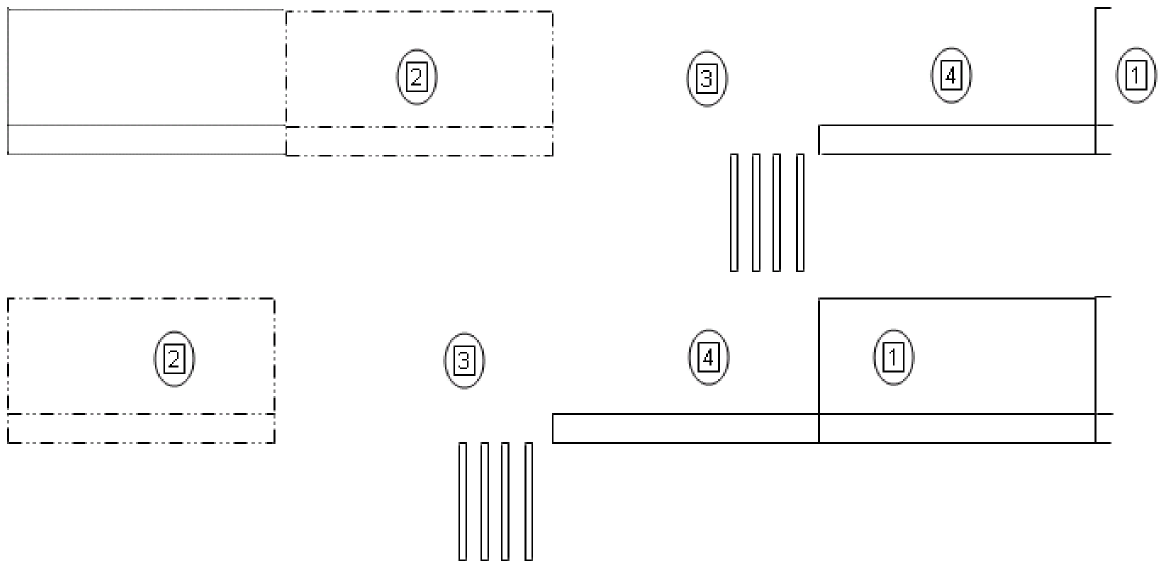

Figure 2 –Floodwall Replacement Construction Sequence

A-6.4.4 Summary of Findings

Based on pile axial failure, computed probabilities of failure range from a maximum of 45% with water at the top of the wall to essentially 0% with water 5 feet below the top of wall. Because of the extensive landside underseepage and relief well system and the aging condition of the timber piles, replacing the floodwall in the footprint of the existing floodwall was determined to be most economical.

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CHAPTER A-6

EXHIBITS

Exhibit 1 – Letter of 9 September 1964

Exhibit 2 – Letter of 19 July 1963

Exhibit 3 – Letter of 23 March 1956

Exhibit 4 – Letter of 2 April 1964

Exhibit 5 – Letter of 15 November 1966

Exhibit 6 - Project Plan View and Soil Profile

Exhibit 7 - Hydraulic Grade Lines for Design Loading Condition Station 84+50 and Station 89+00

Exhibit 8 – Hand Calculations for Design Loading Condition Station 84+50 and Station 89+00

Exhibit 9 – Reliability Analysis Miscellaneous Hand Calculations

Exhibit 10 – Pile Axial Capacity Reliability Analysis Excel Files to Determine Probability of Failure

Exhibit 11 – Probability of Failure Curve for the Axial Capacity of a Single Pile

Exhibit 12 - Review of Topeka Feasibility Study Subsurface Investigation and Soils Test Data

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Exhibit 1

9 September 1964, Memorandum for File
Subject: MRD Conference, Floodwall, South Topeka Unit,
Topeka, KS

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Mr. Tomlinson thru Mr. Gillis RLG.

Mr. Gillis/2753 and
Mr. Spiegel/3080/jc
9 September 1964

MRKED-FL

MEMORANDUM FOR FILES

SUBJECT: MRD Conference, Floodwall, South Topeka Unit, Topeka, Kansas

1. A conference was held in MRD on 8 September 1964, to review the Design Memorandum studies and proposed treatment for the floodwall reach of the South Topeka Unit, located between Kansas Avenue and the beginning of the Oakland Unit (the River Road). This meeting was a followup on the meeting held in this office on 23-24 March 1964, at which these same problems were discussed in less detail. Those in attendance:

MRD	Kansas City District
Asa Shannon	Ralph L. Gillis
Ken Lane	J. H. Tomlinson
Paul Wohlt	R. J. Spiegel
Walt Bruner	

2. The meeting was opened by Mr. Spiegel. He presented topographic print of the downtown area on the south side of the river showing the whole South Topeka Unit. The 1827 and 1854 bank lines had been superimposed on this print showing "old Felix Island" and the chute that formerly went down about Crane Street and back out to the river just downstream of Kansas Avenue. This tended to show some of the problems further exposed by our recent drilling. The general condition of the foundation in this wall area was described with the following being the salient features:

a. There is a layer of fill generally 15 feet or more in thickness under the floodwall. This fill decreases in thickness landward.

b. The natural blanket (predominately silt) shows reasonable continuity landward except in one short reach of the floodwall. It is thin under the floodwall and increases in thickness landward. The natural blanket slopes up in a landward direction from the floodwall.

c. Recent drilling shows the natural blanket to be very thin where intercepted and absent in several reaches riverward of the floodwall (under the sheet pile cutoff).

d. Base of blanket in the vicinity of the basement areas rises to within 2 or 3 feet of basement floor elevations in several instances.

e. The fill under the floodwall and above the natural blanket is a heterogeneous mass of pervious lenses and impervious lenses of all descriptions.

f. The sheet pile wall is reasonably impervious. The interlocks being corroded and filled in the past 25 years.

3. After much discussion, the following areas of agreement were reached in regard to the seepage and foundation problem:

a. The district's pervious drain in a deep trench landward of the wall extending to make contact with the buried natural blanket is necessary and should remain in our consideration.

b. Separate headers for the relief well system and collector drain will be used as proposed. The headers to drain to the pumping plant.

c. The new plant proposed by the city should be used. No adaptation of either existing plant into our system. Contribution to the cities expense of construction based on relative Q from wells and seepage to planned plant Q appears reasonable.

d. We should discuss in the DM why the wells alone will not suffice. (In the meeting we pretty well proved the reasoning, but Asa felt it should be discussed in the DM.)

e. Need more borings landward around the basement areas to pin down the "base of blanket" which is critical to the well design and basement stability.

f. We should present as on alternate relief well plan the case of a minimum number of wells are located around the basement areas rather than along the floodwall. This includes the two unit pumped wells.

g. Discuss the relative dependability of the two relief well plans in the DM.

4. The existing floodwall was discussed as follows:

a. The wall is a concrete cantilever type supported on creosoted timber piles with a 15-foot steel sheet pile cutoff under the riverside edge of the base. It is approximately 2,000 feet long and extends 12 to 13 feet above the landside ground surface.

b. Three test pits dug by the Corps of Engineers and excavation for a sewer under the wall by local interests have revealed the presence of a void under the wall base. The void varies from a hairline crack up to approximately 3/4-inch in thickness. The diverse spacing of the test pits and sewer excavation indicates that the void probably extends the full length of the wall.

c. Two cases of the most severe loading considered possible were discussed. Both cases assume the earth on the riverside of the wall is scoured away to the bottom of the base and the pumping plant to which the toe drain is connected is not working. Uplift due to full river head is assumed at the sheet piling and due to static head from the bottom of the base to the ground surface at the landside toe, plus 2 additional feet of head to provide for losses into the toe drain and for discharging slightly aboveground surface at manholes in the toe drain. For the case of loading considered most likely to prevail, uplift under the base from the sheet pile to the landward toe is assumed equal to the landside tail water as described above. This seems reasonable since pressure from the small amount of seepage through and around the sheet pile will be dissipated in the void under the wall. Since the void is in direct communication with the toe drain, pressure in the void cannot be higher than the landside uplift described above. The resultant of wall loads falls 3.72 feet inside the landside toe (1.72 feet inside the landward pile) and results in compression on the four timber piles varying from 25.8K to 4.0K landward to riverward and tension in the sheet pile amounting to 69 pounds per square foot of sheet pile area. Tension in the sheet pile is not required for wall stability, however, and if it is neglected the load will be taken by the three most landward piles with compression of 31.8K, 18.4K and 4.0K, respectively, on the piles, landward to riverward. The other case discussed was developed to determine the effect on wall stability and pile loads under the most adverse conditions of loading. It assumes that the uplift on the wall base reduces uniformly from full river head at the sheet pile to landside uplift described above at the toe. This assumption would be valid only if the foundation soil were in full and intimate contact with the wall base; a condition that the void under the wall belies. It is, however, in agreement with current design criteria from OCE which permits no drop in uplift pressure under wall bases at steel sheet pile cutoff walls. The resultant of wall loads for this case falls 0.44 feet inside the landside toe (1.56' outside the most landward file), and requires that the sheet pile cutoff wall develop tension of 125 pounds per square foot of surface area to prevent rotation of the wall about the landside toe. Compression in the four timber bearing piles varies from 24.6K to 0.1K landward to riverward.

d. It was explained that by adding 2 feet of concrete on top of the riverside base the resultant of wall loads for the latter case would fall 2.26± ft. inside the landside toe (0.26' inside the landside pile), and tension in the sheet pile would be reduced from 125 p.s.f. to 96 p.s.f. Tension is not required in the sheet pile for stability of the wall and if it is neglected, the load will be carried on the two most landward piles with 42.0K and 3.4K, respectively, on the landward and riverward piles.

e. Computations show that earth at rest, based on a submerged soil weight of 57-1/2 lbs./cu. ft., tan $\phi = 0.5$, K=0.5, will produce

108 p.s.f. friction on the sheet pile and cohesion in the nominal amount of 0.05 T/sq. ft. will produce 100 p.s.f. resistance to pull out, making a total of 208 p.s.f. resistance to pull out.

f. It was the consensus of those present that the sheet pile could develop tension in the amounts required for the assumed loading conditions and that no modifications to the wall were required in order to reduce or eliminate this tension.

R. L. GILLIS
Chief, Design Branch

Copies furnished:
Mr. Leinbach - ED-RH
Mr. McCann - ED-PL
Mr. Tomlinson - ED-DS

R. J. SPIEGEL
Chief, Levee Section

Exhibit 2

19 July 1963
Letter Regarding Madison Street Pump Plant

D. S. Leinbach/dr/2769

MRKED-RH

19 July 1963

Mr. Abram Pratt
City Engineer
Topeka City Hall
Topeka, Kansas

Dear Mr. Pratt:

Reference is made to your letter, dated 26 June 1963, regarding the proposed Madison Street Flood Pumping Plant, Topeka, Kansas.

The flood protection system on the south side of the Kansas River that we refer to as the South Topeka Unit was constructed by the Corps of Engineers and completed in 1939. Improvements that we are presently planning consist of raising the leveed portion of the unit to grades equal to floodwall grades and making any structural changes necessary to accomplish this raise.

The proposed Madison Street Pumping Station will serve the storm sewer requirements in connection with the urban renewal project. Our studies of drainage in the area indicate there are three existing storm-water pumping stations serving this area and their total capacity is adequate to serve the total area including the urban renewal sewer. Economic studies might prove that the cost of interconnecting sewers is less than the proposed Madison Street Pumping Station. Since the need for the pumping station arises out of the urban renewal project, it is our opinion that all costs of this improvement should be borne by local interests.

Representatives from this office will meet with City officials early this fall to discuss the proposed plan of construction for the South Topeka Unit.

Your continuing cooperation with members of my staff is appreciated.

Sincerely yours,

Fish

Monsees

LOUIS G. FEIL
Chief, Engineering Division

Copies furnished:
Reading File
Mr. Fish
ED-E
ED-DS
ED-PL

Hydro Planning Section

Exhibit 3

23 March 1956, Memorandum for File
Subject: Inspection of Old Flood Wall, City Park Pumping Plant, Topeka, Kansas

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MRDGL 23 March 1956

MEMORANDUM FOR THE FILES

SUBJECT: Inspection of Old Flood Wall, City Park Pumping Plant, Topeka, Kansas.

1. Reference is made to the memorandum of 28 February relative to the condition of the old flood wall encompassed by the levee construction in 1939 at the City Park Pumping Station, Topeka, Kansas.

2. I received a phone call at 10:00 a.m. 22 March from Mr. E. L. Smith, Construction Division, stationed at Forbes Air Force Base, who has apparently been delegated the Corps of Engineers' responsibility for inspection of the construction operations of the City Park Pumping Station. Mr. Smith stated that Mr. Padden, of Padden and Bartlet (which firm has the architect-engineer services for this pumping plant) had called him requesting to have a conference at 2:00 p.m. at Topeka relative to the conditions referred to in the above referenced memorandum, and that I be present. Accordingly, Mr. Tomlinson, of the Structures Section, and I met with Mr. Padden, Mr. Pratt, City Engineer, Mr. Tom Bruce, City Engineer's Office, Mr. Smith, and the contractor's superintendent.

3. During my preliminary inspection, I had noted "roofing" conditions under the old wall but had not made a close inspection. A detailed inspection disclosed the "roofing" condition to be even more pronounced than at first indicated. From a point within the excavation beneath the wall, an open space approximately 1" wide can be noted on either side of the excavation for the full width of the base of the wall. Mr. Pratt said that he felt this possibly was due to drying out of the material under the wall. However, a small diameter stick was pushed back into the opening approximately 6 feet before wet soil was noted on the end of the stick. This condition is even more apparent along the landside of the wall where it is exposed by the pumping plant excavation. The material here has slumped away from the base of the wall approximately 4" at the edge of the wall base. The contractor's superintendent stated that the void already existed when the excavation reached this point. However, it is my opinion that the void opened up almost as fast as the adjacent support was removed during the excavation process giving the appearance that the void existed before the excavation started. The material under the wall consists of refuse, organic material, willows and old dump material. This "roofing" condition is apparent on both sides of the excavation under the wall and extends over most of the width of the pumping plant excavation.

4. In view of the above noted conditions I believe that it would be extremely difficult to provide any type of backfill which would effectively seal this space beneath the base of the wall even though the actual opening for the outfall conduit was filled completely with

concrete. Mr. Fadden appeared to be in considerable disagreement with the suggestion that a section of the old wall be removed in order that proper backfill can be placed. I told them that an alternate method could probably be worked out which, although not equal to the recommended scheme, might be a satisfactory substitute. This would involve drilling through the base all along the exposed portion of the wall, inserting grout pipes, and after backfill has been completed above the base of the wall, to pressure grout the full length of the exposed portion of the wall. Under this plan, however, it would still be necessary to remove considerable material adjacent to the wall in order to place the grout pipes. I also explained to both Mr. Pratt and Mr. Fadden the importance of sloping the pumping excavation as previously recommended in order to provide sufficient working space to properly compact the backfill material. Under the present conditions, almost 100% of the backfill will have to be placed by hand or pneumatic equipment while the recommended slopes would permit most of the backfill to be placed by power equipment. I also raised the question of the source of the impervious material proposed for backfill of the plant. They indicated that they had one source in mind and Mr. Smith stated that he would have the source checked for suitability of material.

5. In view of the foregoing conditions, I advised Mr. Pratt and Mr. Fadden that my opinion had not altered relative to the advisability of removing the old flood wall within the excavation area. If, after consideration they wished to investigate some alternate plan, such as the grouting method, this office would be pleased to discuss it with them in more detail. Under these conditions I do not believe any method of filling the excavation with mass concrete under head would result in satisfactory backfill.

R. J. SPIEGEL
Head, Soil Section

RJS/iga

cc: Mr. R. L. Gillis
Mr. J. M. McCann
Mr. K. B. Lane

Exhibit 4

2 April 1964
Subject: South Topeka Unit, Topeka, KS
Flood Protection Project – Conference with MRD to Review Preliminary Studies Prior to Submission of Design Memorandum

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U. S. ARMY ENGINEER DISTRICT, KANSAS CITY
CORPS OF ENGINEERS
1800 Federal Office Building
Kansas City, Missouri 64106

MRKED-PL

2 April 1964

SUBJECT: South Topeka Unit, Topeka, Kansas, Flood Protection Project - Conference with MRD to Review Preliminary Studies Prior to Submission of Design Memorandum

1. A conference relative to the South Topeka Unit, Topeka, Kansas, Flood Protection Project was held in the Kansas City District office 23-24 March 1964, with the following in attendance:

A. Shannon, *MRD
K. S. Lane, MRD
W. Breuer, MRD
A. Harrison, MRD
Col. Wachendorf, *KCD
L. G. Feil, KCD
C. R. Van Orman, KCD
R. C. Gillis, KCD
J. F. Redlinger, KCD
J. M. McCann, KCD
D. S. Leinbach, KCD
R. J. Spiegel, KCD
W. N. Doyle, KCD
Lee Nelson, KCD
P. D. Barber, KCD
P. E. Morris, KCD
J. Tomlinson, KCD
R. L. Browning, KCD
G. L. Audsley, KCD

*Attended 24 March 1964

2. Points resolved:

a. It was agreed that the existing hand placed riprap on the South Topeka Unit should dictate the approximate limits of slope protection proposed. New riprap will consist of repair to the toe and at the top of the proposed levee raise, and some modifications and alterations at drainage structures. Rockfill appearing in the estimate is required for levee stability and not for protection against streamflows.

b. The hydrologic features were considered satisfactory.

c. The area fill proposed upstream of the C.R.I.& P. Railroad bridge and the design sections between the C.R.I.& P. Railroad bridge and Kansas Avenue were considered satisfactory. Additional studies will be made in connection with stability berms.

d. In connection with modifications and/or alterations of the existing floodwall the following was agreed:

(1) The existing floodwall is considered stable under design conditions, regardless of uplift pressures.

(2) Either a new or reconditioned toe drain is considered a necessity.

(3) Additional explorations are considered necessary to better define the problem in the wall area.

(4) Objections were raised relative to relief wells around private buildings. A "line of wells" system next to the wall will be designed.

(5) The use of a header and manhole system for relief wells is satisfactory.

(6) Utilization of existing pumping facilities is satisfactory.

(7) It was agreed that a reasonable amount of pull-out resistance could be assumed for the steel sheet pile cutoff-wall. The amount of resistance allowed should be based on a study of the soil type and values recommended by authorities as a result of pull-out piles.

(8) Tension cannot be assumed for timber bearing piles.

(9) A reasonable reduction can be assumed in the uplift pressures under the wall because of the sheet pile cutoff wall, and the proposed pressure relief well system and the toe drain rehabilitation.

2. <u>Additional studies</u>:

a. If it appears that grouting is economical, consideration should be given to test grouting through the present floodwall base to form cutoff at existing sheet pile cutoff. This would be accomplished by Government forces.

b. Mr. Shannon seemed to favor the proposed excavated trench on the riverside to form an impervious cutoff. A firm proposal for the cutoff will be made after correlation with data obtained from the experiment described in a. above.

c. The advisability of grouting the roofing void under the wall was questioned. This will be discussed and resolved in the Design Memorandum.

d. Mr. Lane appeared to favor the idea of a deep toe collector near the top of blanket in lieu of either the cutoff or grouting.

3. Discussion: Hydraulic Design of Drainage Structures was discussed with Mr. Harrison. The specific items discussed were: (a) use of rounded inlet ends on pipes and conduits to reduce inlet losses where full flow exists and inlet ends are submerged. Mr. Harrison agreed to send Kansas City District details of rounded inlet ends used by Omaha District (b) The use of timber piles to support outlet ends of C.M. pipes and flap gates. It was pointed out to Mr. Harrison that comparative cost estimates for both 24 and 48-inch pipe indicated that the conventional concrete outlet structure used by Kansas City District was cheaper than the pile-supported outlet.

Prepared in
Kansas City District
Corps of Engineers
2 April 1964

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Exhibit 5

15 November 1966, Memorandum for Files
Subject: Underseepage Treatment behind Floodwall in South Topeka Unit

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MRKED-FL 15 November 1966

MEMORANDUM FOR FILES

SUBJECT: Underseepage Treatment Behind Floodwall in the South Topeka Unit

1. On Tuesday, 8 November 1966, a meeting was held in this office concerning the underseepage treatment for the floodwall portion of the South Topeka Unit. The following were in attendance.

Office, Chief of Engineers

R. O. Barron

Missouri River Division

K. S. Lane
P. E. Wohlt

Kansas City District

J. F. Redlinger
R. E. Spiegel
R. L. Browning
W. H. Cook

2. Mr. Spiegel presented the recommended plan as shown in the D.M. along with several factors that would influence this plan. Mr. Barron was very concerned about the low safety factors in the basements located behind the floodwall. He also stated that seepage safety factors considered adequate for agricultural levee design are not considered adequate in a highly industrial area such as this. He feels that a flood fight in the basements is highly undesirable if not impossible.

3. Photographs of the basements were studied and it was agreed that it would be impractical to place fill in the basements which was suggested as a possibility by OCE and MRD.

4. In order to obtain higher safety factors in the basements, two relief well systems were suggested to be studied.

a. One plan would keep the basic line of protection (relief wells) along the wall and supplement this as required with pumped wells located around the basements.

b. The other plan would have only the minimum number of wells needed to protect the floodwall located along the wall and a proportionately greater number of pumped wells located around the basements to provide the desired safety factors. (This is the plan favored by Mr. J. O. Ackerman, MRD, by telephone 11-15-66.)

6. Originally, Mr. Barron indicated that a safety factor of 1.5 would be desirable in all the basements. In the discussion that followed, Mr. Lane suggested that we might try for safety factors of 1.5 between the wells located along the wall and 1.0 in the basements. In the largest basement, we could go as low as 0.8 without ponded water. Mr. Barron said he would be satisfied if we had a 1.5 S.F. with basements flooded.

7. The power supply for the pump units and the possibility of an electrical failure at the pumping plant were also discussed. An auxiliary power supply could be located at the proposed pumping plant or portable pumps could be provided to pump from individual wells. In the latter case provision should be made to store them in a warehouse on the site.

8. Mr. Barron said that he did not want a cross connection between the gravity header system and the collector system. He felt that if it became necessary to pump the artesian wells, the cross connection would not be desirable.

9. After we have analyzed the above plans, we will forward a supplemental design memorandum to OCE through MRD. After sufficient time for review, another conference will be scheduled with all parties represented.

R. J. SPIEGEL
Chief, Levee Section

Copies furnished:
ED-FL
ED-DG thru Mr. Gillis
ED-DM (Mr. Heavey)

Exhibit 6

Project Plan View
and Soil Profile

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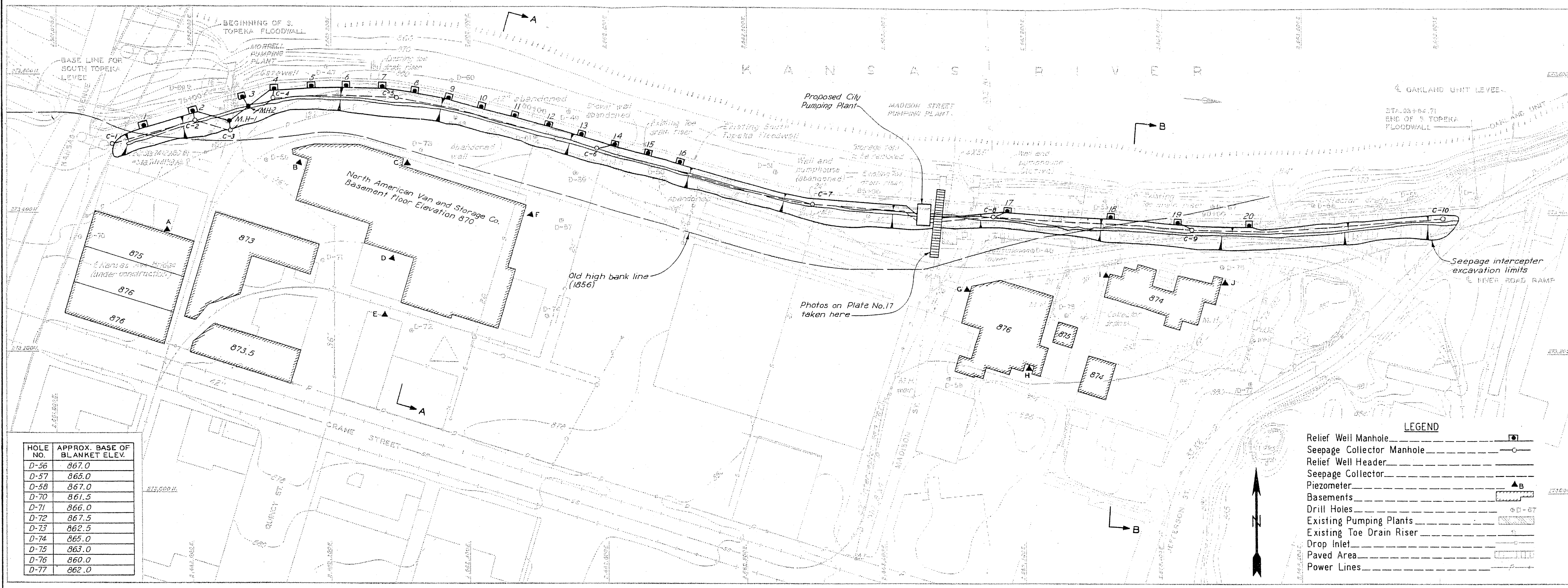
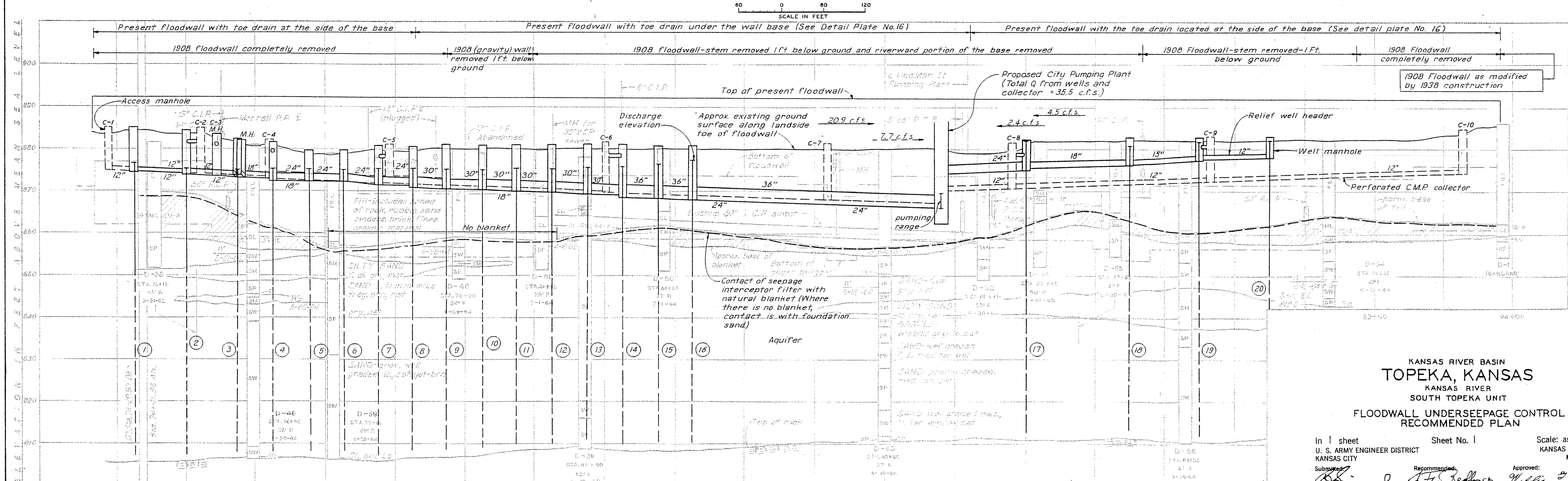


HOLE NO.	APPROX. BASE OF BLANKET ELEV.
D-56	867.0
D-57	865.0
D-58	867.0
D-70	861.5
D-71	866.0
D-72	867.5
D-73	862.5
D-74	865.0
D-75	863.0
D-76	860.0
D-77	862.0

PLAN

SCALE IN FEET



KANSAS RIVER BASIN

TOPEKA, KANSAS

KANSAS RIVER

SOUTH TOPEKA UNIT

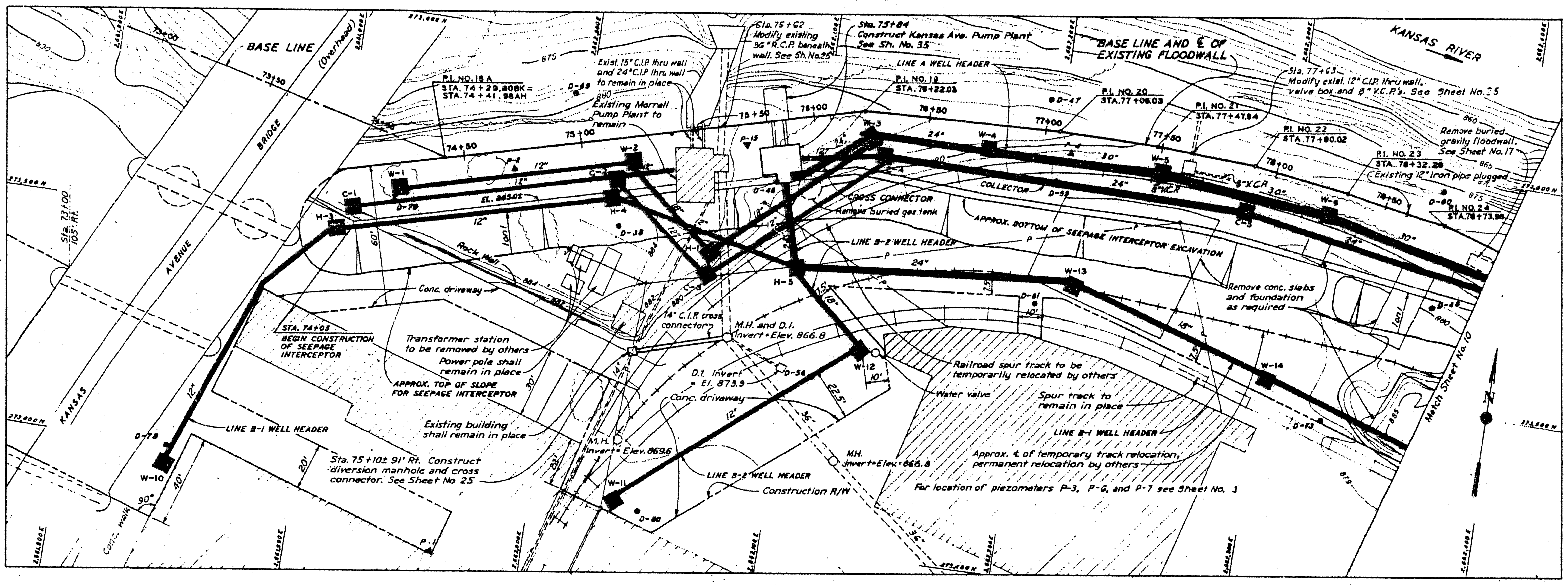
FLOODWALL UNDERSEEPAGE CONTROL RECOMMENDED PLAN

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

U. S. ARMY ENGINEER DISTRICT KANSAS CITY — KANSAS CITY, MO. MAY 1966

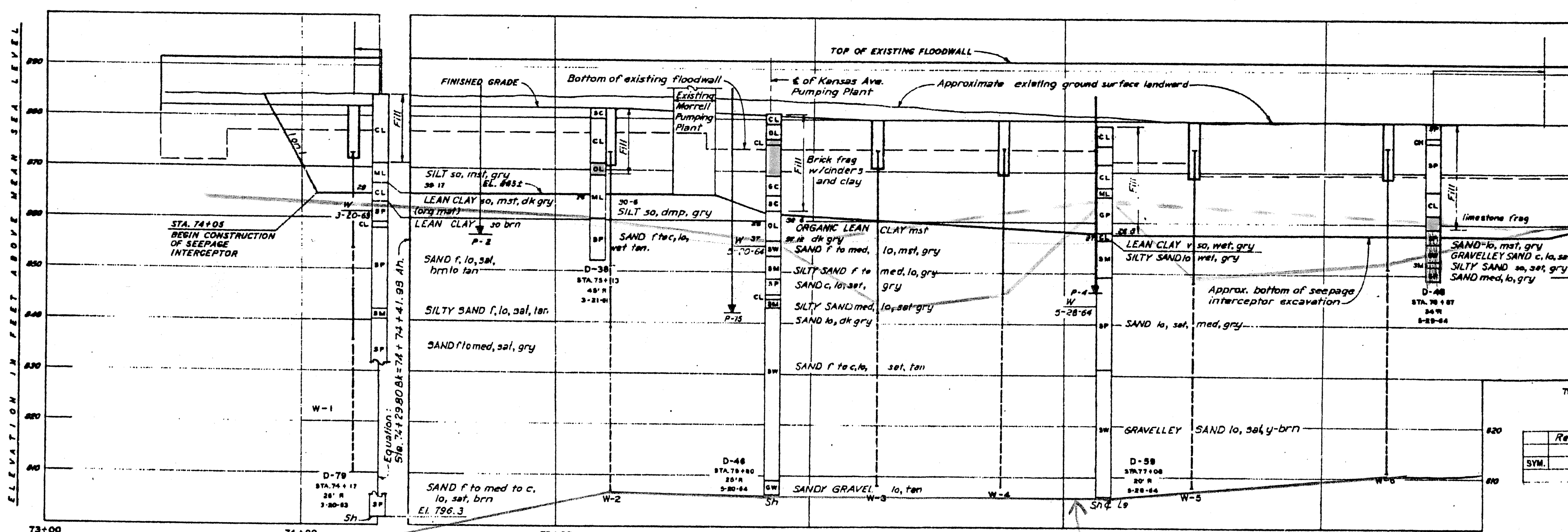
Submitted: CHIEF, LEVEE SECTION — Recommended: CHIEF, FOUNDATIONS & MATLS. BR. — Approved: CHIEF, ENGINEERING DIVISION

DRAWN BY: W.H.C. — TRACED BY: M.M.W. — CHECKED BY: R.L.B. — DM 13 — FILE NO. A-12-1123



PLAN

POOR QUALITY FILL IN BLANKET



PROFILE

TOP OF BEDROCK

12" Fill (CL)

SP

Notes:

1. For legend of underground explorations and logs of drill holes adjacent to Line B well headers, see Sheet No. 13.
2. For legend of existing and construction features, see Sheet No. 10.
3. For relief well and piezometer data, see Sheet No. 14.
4. For seepage interceptor details and collector profile, see Sheet No. 15.
5. For well header profiles, see Sheet No. 16.
6. For manhole details, see Sheet Nos. 46, 47, 48
7. For details of levee embankment and floodwall tie, see Sheet No. 8.

RECORD DRAWING

AUGUST 1971

CONTRACT NO. DACW41-70-C-0041

SYM.	DESCRIPTION	DATE	APP'D.
	Revised for "As Built conditions"	4-9-74	

REVISIONS

KANSAS RIVER, KANSAS

TOPEKA, KANSAS

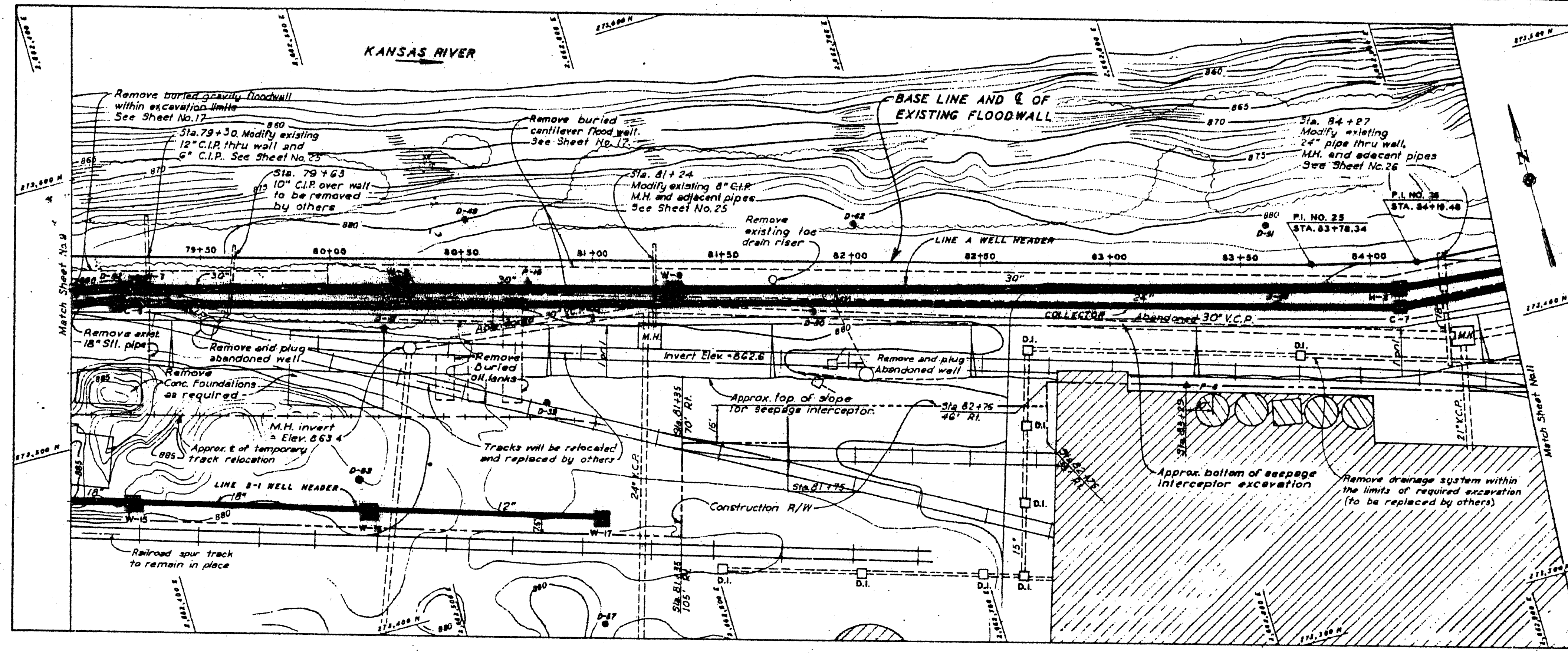
SOUTH TOPEKA UNIT

RELIEF WELL AND SEEPAGE INTERCEPTOR SYSTEMS PLAN AND PROFILE STATION 74+00 TO 79+00

In 55 sheets | Sheet No. 9 | Scale: as shown

CORPS OF ENGINEERS KANSAS CITY DISTRICT | U. S. ARMY

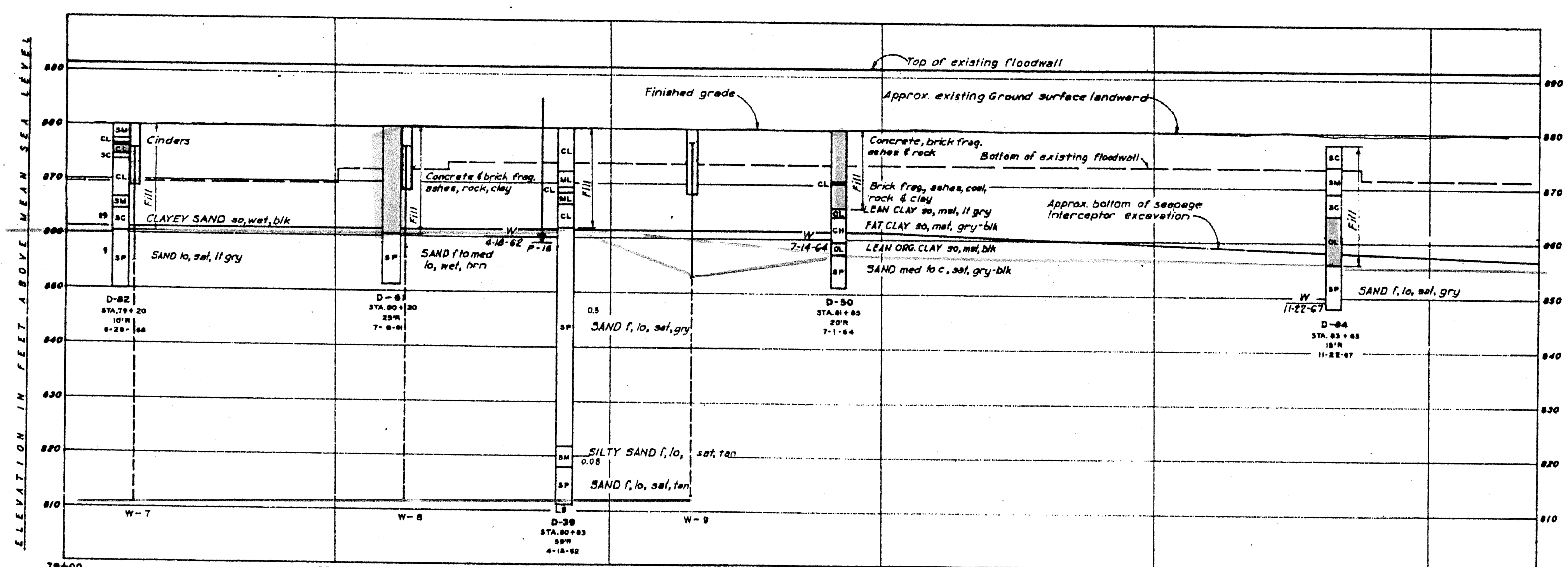
A-12-1259



PLAN

SCALE IN FEET

POOR QUALITY FILL COMPRISING BLANKET



PROFILE

LEGEND

Base line and ℄ of existing floodwall
Existing Underground Line
Existing Manhole — MH
Existing Drop Inlet — D.I.
Construct Well Manhole — W-6
Construct Header Manhole — H-2
Construct Collector Manhole — C-6
Construct Header Pipe — 30"
Construct Collector Pipe — 24"
Landside slope for seepage interceptor (riverside slope not shown)
Construct Piezometer — P-6
Overhead Powerlines — P
Drill Holes — D-61
Relief Well Manhole in Profile
Riser
Screen
Piezometer in profile

Notes:

1. For legend of underground explorations and logs of drill holes adjacent to Line B well headers, see Sheet No. 13
2. For relief well and piezometer data, see Sheet No. 15
3. For seepage interceptor details and collector profile, see Sheet No. 16
4. For well header profile, see See Sheet No. 17
5. For manhole details, see Sheet Nos. 46, 47, & 48.
6. For modification of existing pipelines located within the excavation limits of the seepage interceptor, see Sheets No. 24 and No. 25.
7. Where abandoned 30-inch V.C.P. sewer line falls within the seepage interceptor excavation it shall be removed, and the ends of the portions to remain in place shall be plugged with concrete.

RECORD DRAWING

AUGUST 1971

CONTRACT NO. DACW41-70-C-0041

SYM.	DESCRIPTION	DATE	APP'D.
	Revised for "As Built" conditions	4-9-74	

REVISIONS

KANSAS RIVER, KANSAS

TOPEKA, KANSAS

SOUTH TOPEKA UNIT

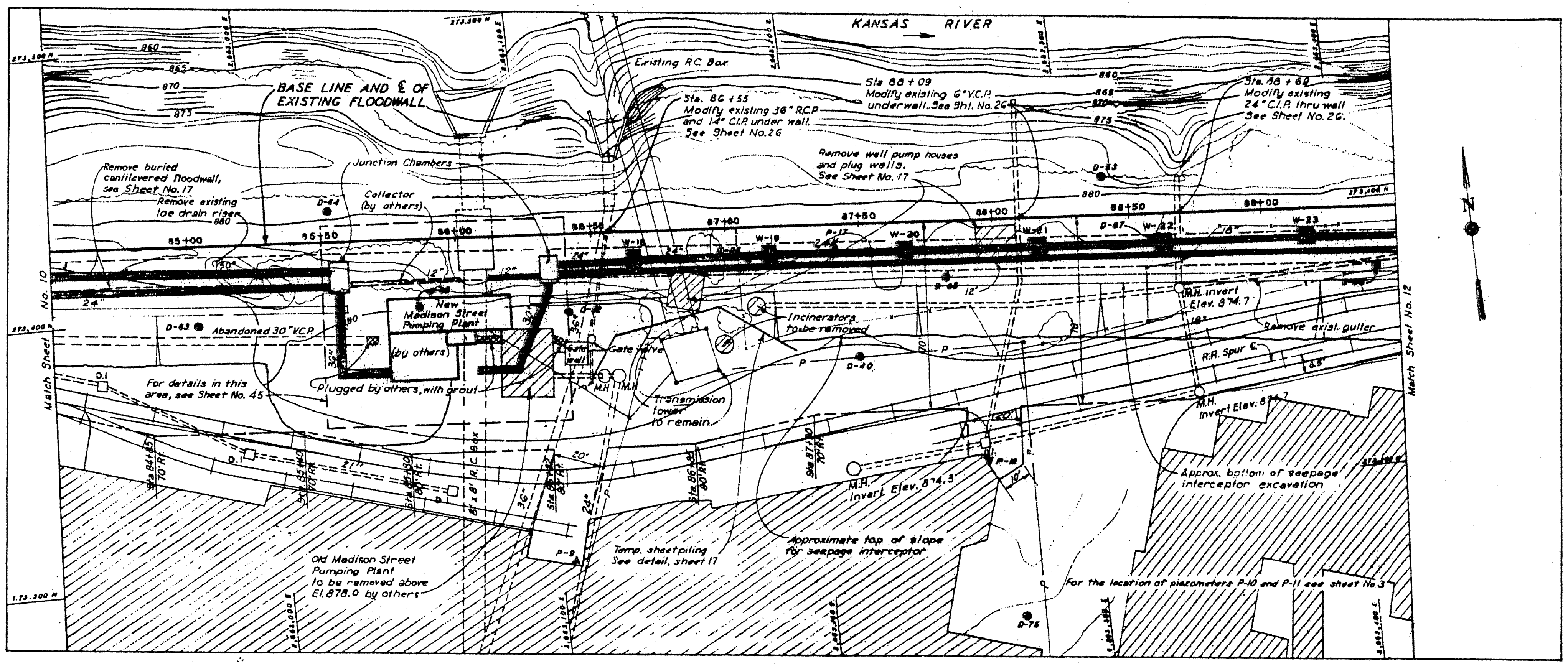
RELIEF WELL AND SEEPAGE INTERCEPTOR SYSTEMS PLAN AND PROFILE STATION 79+00 TO 84+50

In 55 sheets — Sheet No. 10 — Scale: as shown

CORPS OF ENGINEERS KANSAS CITY DISTRICT — U. S. ARMY NOVEMBER 1969

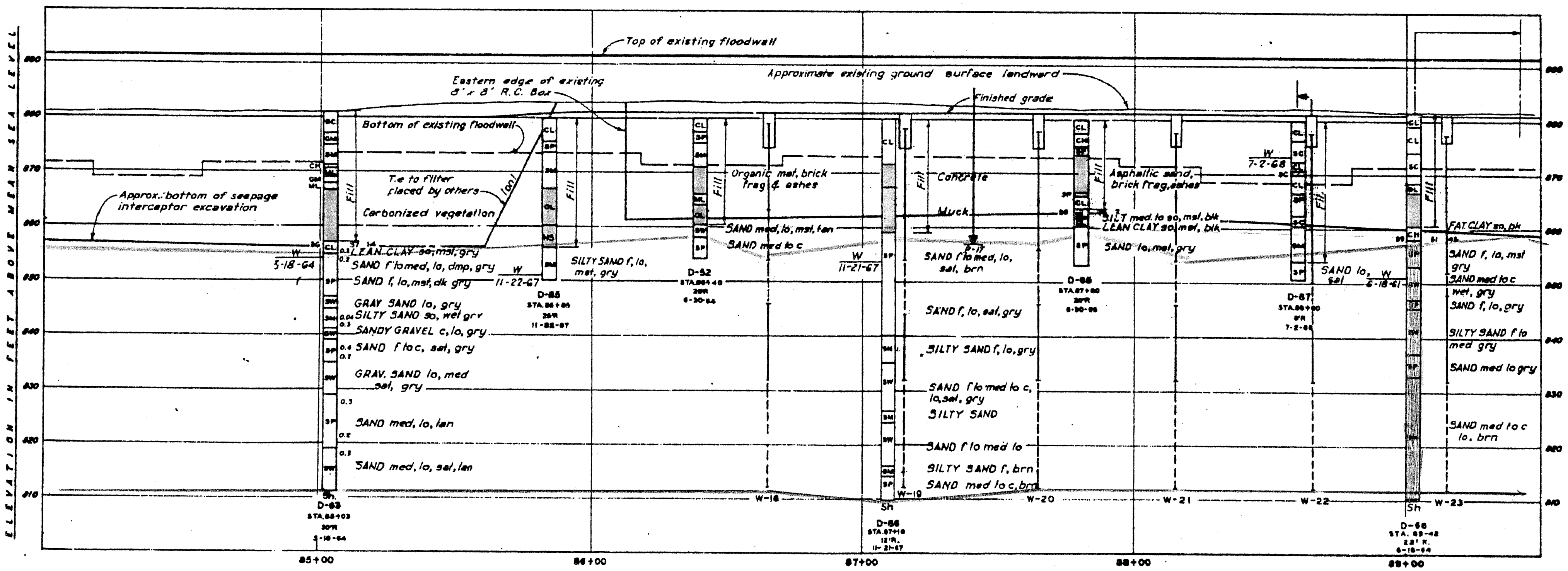
Submitted: CHIEF, DESIGN SECTION — Recommended: CHIEF, FOUNDATIONS & MATLS. BR. — Approved: CHIEF, ENGINEERING DIVISION

COMPILED BY: W.H.C. — DRAWN BY: J.S.D. — CHECKED BY: R.L.B. — FILE NO. A-12-1280



PLAN

POOR QUALITY FILL COMPRISING BLANKET



PROFILE

Notes:

1. For legend of underground explorations, see Sheet No. 13.
2. For legend of existing and construction features, see Sheet No. 10
3. For relief well and piezometer data, see Sheet No. 14.
4. For seepage interceptor details and collector profile, see Sheet No. 15.
5. For well header profiles, see Sheet No. 16.
6. For manhole details, see Sheet No. 46, 47 & 48
7. Where abandoned 30-inch V.C.P. sewer line falls within the seepage interceptor excavation it shall be removed, and the ends of the portions to remain in place shall be plugged with concrete.

RECORD DRAWING

AUGUST 1971

CONTRACT NO. DACW41-70-C-0041

SYM.	DESCRIPTION	DATE	APP'D.
	Revised for "As Built" conditions	4-9-74	

REVISIONS

KANSAS RIVER, KANSAS

TOPEKA, KANSAS

SOUTH TOPEKA UNIT

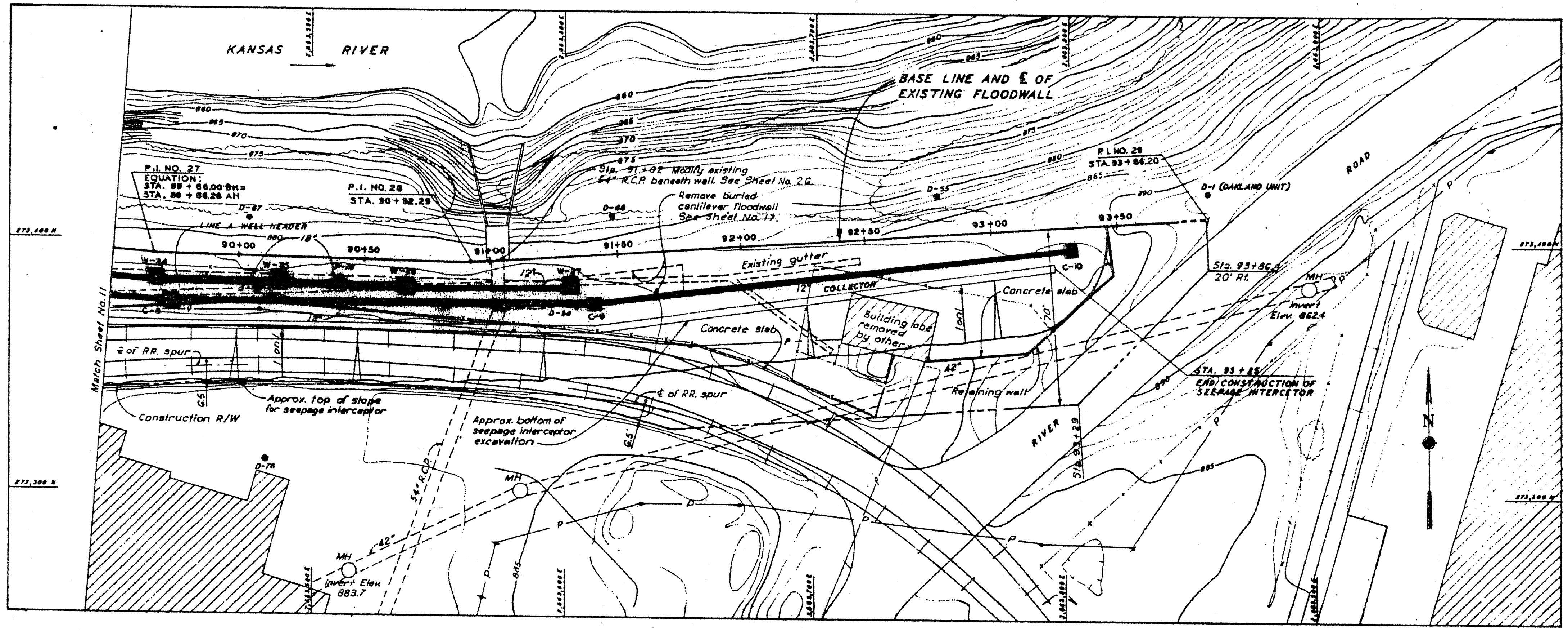
RELIEF WELL AND SEEPAGE INTERCEPTOR SYSTEMS
PLAN & PROFILE
STATION 84+50 TO 89+50

In 55 sheets — Sheet No. 11 — Scale: as shown

CORPS OF ENGINEERS KANSAS CITY DISTRICT — U. S. ARMY — NOVEMBER 1969

W.H.C. — J.B.D. — R.L.B.

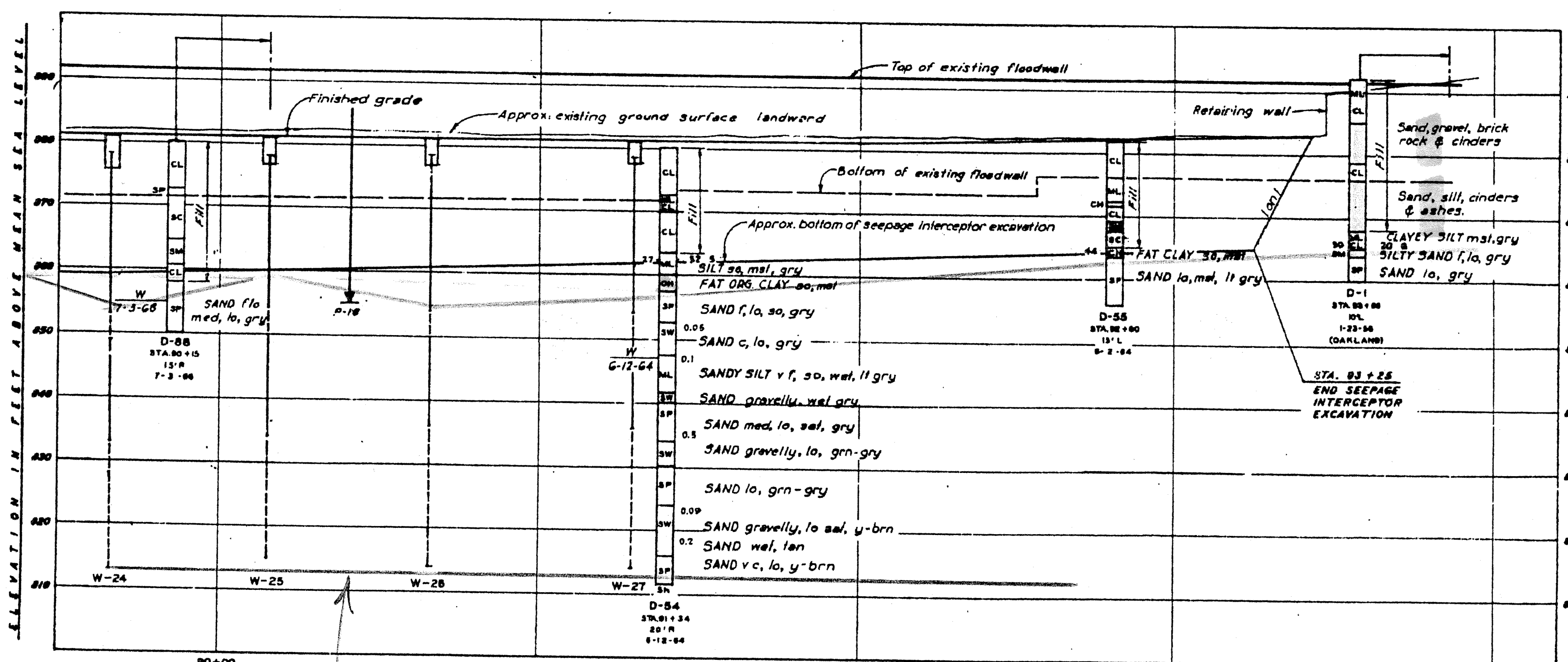
A-12-1261



PLAN

SCALE IN FEET

POOR QUALITY FILL COMPRISING BLANKET



PROFILE

TOP OF BEDROCK

Notes:
1. For legend of underground explorations, see Sheet No. 13.
2. For legend of existing and construction features, see Sheet No. 10.
3. For relief well and piezometer data, see Sheet No. 14.
4. For seepage interceptor details and collector profile, see Sht. 15.
5. For well header profiles, see Sheet No. 16.
6. For Manhole details, see Sheets No. 46, 47 & 48.

RECORD DRAWING

AUGUST 1971

CONTRACT NO. DACW41-70-C-0041

SYM.	DESCRIPTION	DATE	APP'D.
	Revised for "As Built" conditions	4-9-74	

REVISIONS

KANSAS RIVER, KANSAS

TOPEKA, KANSAS

SOUTH TOPEKA UNIT

RELIEF WELL AND SEEPAGE INTERCEPTOR SYSTEMS
PLAN AND PROFILE STATION 89+50 TO 94+00

In 55 sheets | Sheet No. 12 | Scale: as shown

CORPS OF ENGINEERS
KANSAS CITY DISTRICT

U. S. ARMY
NOVEMBER 1966

COMPILED BY: W.H.C. DRAWN BY: C.F.L. CHECKED BY: R.L.S.

FILE NO. A-12-1262

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Exhibit 7

Hydraulic Grade Lines for Design Loading Condition
Station 84+50 and Station 89+00

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South Topeka Floodwall Analysis

UNDERSEEPAGE ANALYSIS WITHOUT WELLS

Water to top of Flood Protection

Blanket Unit Weight = 105.0 pcf (saturated) (Average Value)

Station	Top of Levee Elevation (msl)	River Side Ground Elevation (msl)	Land Side Ground Elevation (msl)	River Blanket Bottom Elevation (msl)	Land Blanket Bottom Elevation	Top of Bedrock Elevation (msl)	Driving Head (ft) H	Permeability Ratio River Side k_f/k_{br}	Permeability Ratio Land Side k_f/k_{bl}	Impervious Blanket Thickness (ft) River Side z_{br}	Impervious Blanket Thickness (ft) Levee z_{bo}	Impervious Blanket Thickness (ft) Land Side z_{bl}	Pervious Blanket Thickness (ft) d	Seepage Length (ft) River Side L_1	Seepage Length (ft) Levee L_2	Seepage Length (ft) Land Side L_3	Factor c_r	Factor c_l	Effective Seepage Length (ft) River Side x_1	Effective Seepage Length (ft) Land Side x_3	Head at Toe (ft) h_o	Computed Hydraulic Gradient i_o	Critical Hydraulic Gradient i_c	Factor of Safety for Piping (i_c/i_o)	Remarks
83+00 to 87+00	892.0	880	880	855	855	810	12.0	300	300	25	25	25	45	70	10	1000	0.00172	0.00172	69.66	580.95	10.55	0.42	0.68	1.62	

1/5

RELIEF WELL ANALYSIS

k =	0.0036	ft/s
D =	45	ft
h_0 =	10.55	ft
TOL =	892	ft elevation
Landside =	880.0	ft elevation
Bottom Blanket =	855	ft elevation
blanket =	25.0	ft
z, Landside Toe =	80	ft
r_w =	1	ft

γ_{sat} =	110	pcf
i_c =	0.76	
FS_{req} =	1.6	
Efficiency =	0.8	
Total Flow =	14.75	cfs

real well locations						
well	x	y	discharge el	Q_w (cfs)	v_w (ft/s)	H_w (ft)
W-7	90	7925	876.0	0.85	0.27	1.00
W-15	170	7925	870.0	1.01	0.32	1.00
W-16	175	8015	870.7	0.92	0.29	1.00
W-8	90	8025	876.3	0.78	0.25	1.00
W-17	180	8104	872.3	0.88	0.28	1.00
W-9	90	8130	877.5	0.77	0.24	1.00
W-18	90	8665	879.1	0.66	0.21	1.00
W-19	90	8715	878.1	0.66	0.21	1.00
W-20	90	8765	877.6	0.66	0.21	1.00
W-21	90	8815	877.3	0.66	0.21	1.00
W-22	90	8855	877.5	0.65	0.21	1.00
W-23	90	8915	877.7	0.65	0.21	1.00
W-24	90	8966	877.9	0.64	0.20	1.00
W-25	90	9015	878.2	0.64	0.20	1.00
W-26	90	9065	878.5	0.66	0.21	1.00
W-27	90	9129	878.6	0.72	0.23	1.00
17				0.00	0.00	0.00
18				0.00	0.00	0.00
19				0.00	0.00	0.00
20				0.00	0.00	0.00
						1.00

<---Input H_w AVG after any changes are made to well parameters

image well locations		
well	x'	y'
W-7	-90	7925
W-15	-170	7925
W-16	-175	8015
W-8	-90	8025
W-17	-180	8104
W-9	-90	8130
W-18	-90	8665
W-19	-90	8715
W-20	-90	8765
W-21	-90	8815
W-22	-90	8855
W-23	-90	8915
W-24	-90	8966
W-25	-90	9015
W-26	-90	9065
W-27	-90	9129
17	0	0
18	0	0
19	0	0
20	0	0

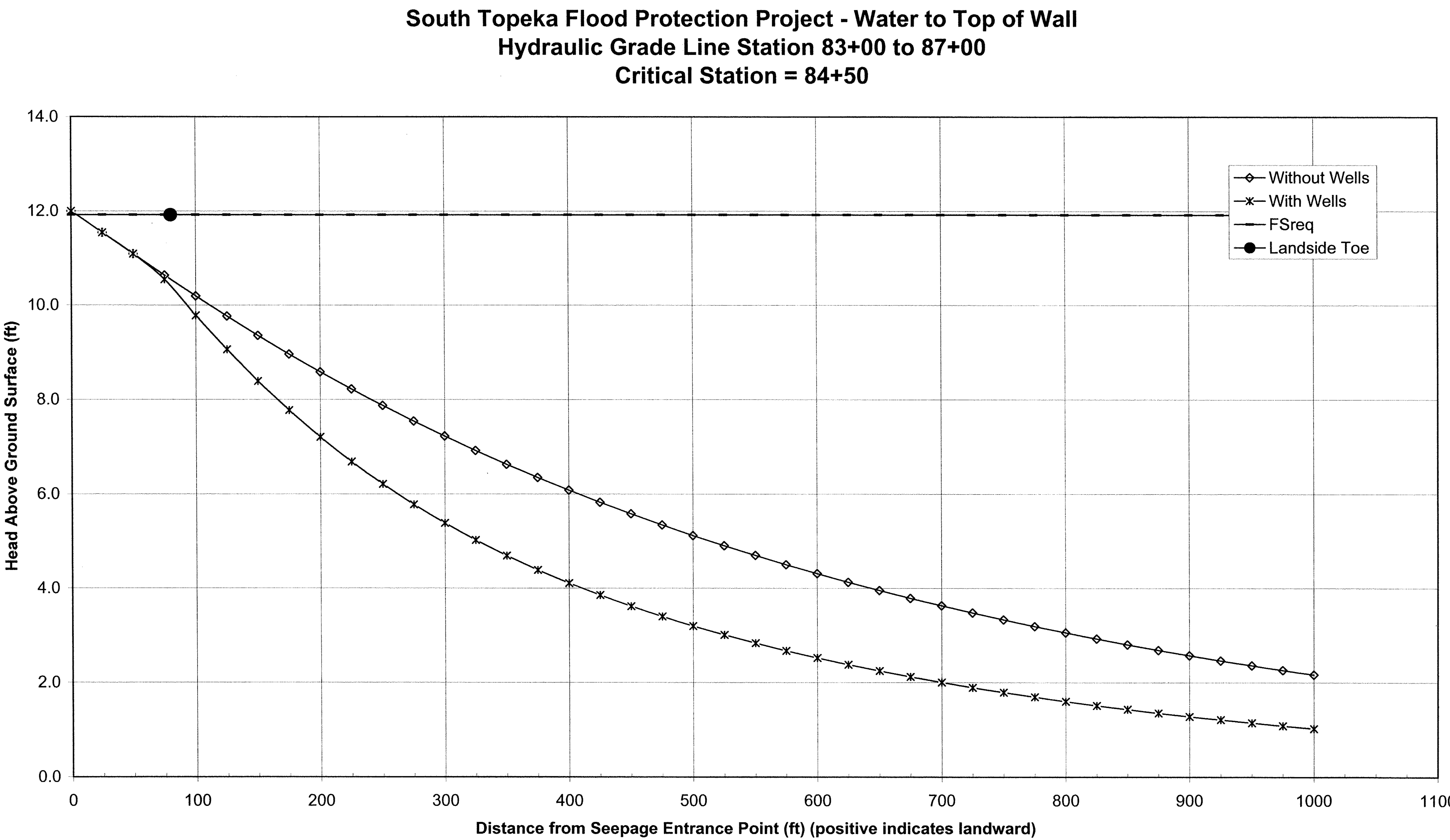
Change y_p in this table to change stationing of HGL Plot Perpendicular to Levee

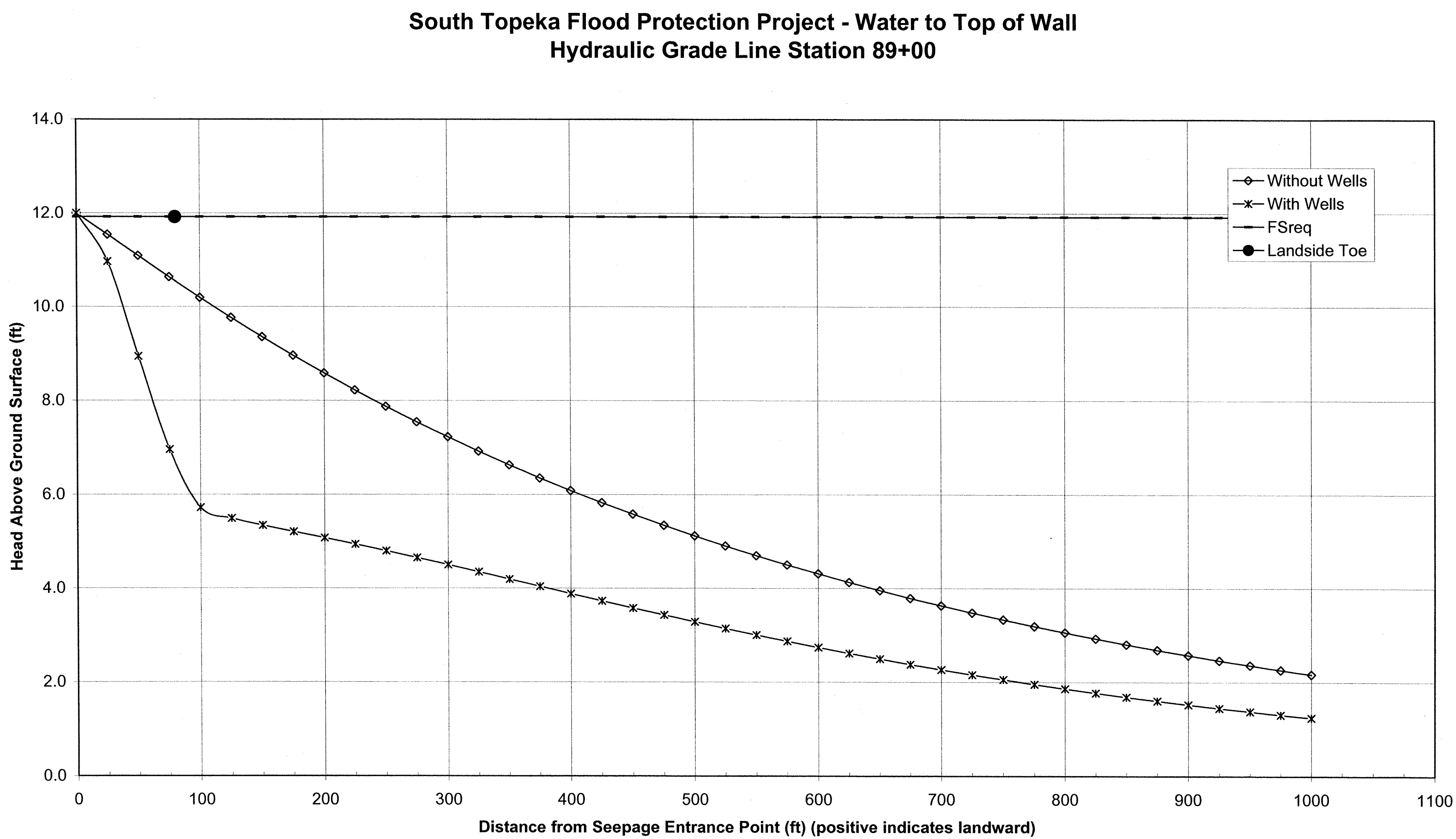
Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	0	8450	12.0	0.0	12.0	11.92	1.00	0.48	1.59
2	25	8450	11.5	0.4	11.5	11.92	1.00	0.46	1.65
3	50	8450	11.1	0.7	11.1	11.92	1.00	0.44	1.72
4	75	8450	10.6	1.1	10.5	11.92	1.00	0.42	1.81
5	100	8450	10.2	1.4	9.8	11.92	1.00	0.39	1.95
6	125	8450	9.8	1.7	9.1	11.92	1.00	0.36	2.10
7	150	8450	9.4	2.0	8.4	11.92	1.00	0.34	2.27
8	175	8450	9.0	2.2	7.8	11.92	1.00	0.31	2.45
9	200	8450	8.6	2.4	7.2	11.92	1.00	0.29	2.65
10	225	8450	8.2	2.5	6.7	11.92	1.00	0.27	2.85
11	250	8450	7.9	2.7	6.2	11.92	1.00	0.25	3.07
12	275	8450	7.5	2.8	5.8	11.92	1.00	0.23	3.30
13	300	8450	7.2	2.8	5.4	11.92	1.00	0.22	3.54
14	325	8450	6.9	2.9	5.0	11.92	1.00	0.20	3.80
15	350	8450	6.6	2.9	4.7	11.92	1.00	0.19	4.07
16	375	8450	6.4	3.0	4.4	11.92	1.00	0.18	4.35
17	400	8450	6.1	3.0	4.1	11.92	1.00	0.16	4.64
18	425	8450	5.8	3.0	3.9	11.92	1.00	0.15	4.95
19	450	8450	5.6	3.0	3.6	11.92	1.00	0.14	5.27
20	475	8450	5.3	2.9	3.4	11.92	1.00	0.14	5.61
21	500	8450	5.1	2.9	3.2	11.92	1.00	0.13	5.97
22	525	8450	4.9	2.9	3.0	11.92	1.00	0.12	6.34
23	550	8450	4.7	2.9	2.8	11.92	1.00	0.11	6.73
24	575	8450	4.5	2.8	2.7	11.92	1.00	0.11	7.14
25	600	8450	4.3	2.8	2.5	11.92	1.00	0.10	7.57
26	625	8450	4.1	2.8	2.4	11.92	1.00	0.10	8.02
27	650	8450	4.0	2.7	2.2	11.92	1.00	0.09	8.49
28	675	8450	3.8	2.7	2.1	11.92	1.00	0.08	8.99
29	700	8450	3.6	2.6	2.0	11.92	1.00	0.08	9.52
30	725	8450	3.5	2.6	1.9	11.92	1.00	0.08	10.07
31	750	8450	3.3	2.5	1.8	11.92	1.00	0.07	10.65
32	775	8450	3.2	2.5	1.7	11.92	1.00	0.07	11.27
33	800	8450	3.1	2.5	1.6	11.92	1.00	0.06	11.91
34	825	8450	2.9	2.4	1.5	11.92	1.00	0.06	12.60
35	850	8450	2.8	2.4	1.4	11.92	1.00	0.06	13.32
36	875	8450	2.7	2.3	1.4	11.92	1.00	0.05	14.08
37	900	8450	2.6	2.3	1.3	11.92	1.00	0.05	14.88
38	925	8450	2.5	2.3	1.2	11.92	1.00	0.05	15.73
39	950	8450	2.4	2.2	1.1	11.92	1.00	0.05	16.62
40	975	8450	2.3	2.2	1.1	11.92	1.00	0.04	17.57
41	1000	8450	2.2	2.1	1.0	11.92	1.00	0.04	18.57

Change y_p and x_p in this table to change stationing of HGL Plot Parallel to Levee

Point of Interest	x_p	y_p	H_{HGL} (ft)	Drawdown (ft)	h_p (ft)	h_a (ft)	H_w (ft)	i	FS_i
1	90	7805	10.4	2.1	9.2	11.92	1.00	0.37	2.07
2	90	7840	10.4	2.7	8.6	11.92	1.00	0.35	2.21
3	90	7875	10.4	3.6	7.8	11.92	1.00	0.31	2.45
4	90	7910	10.4	5.1	6.3	11.92	1.00	0.25	3.02
5	90	7945	10.4	5.3	6.0	11.92	1.00	0.24	3.16
6	90	7980	10.4	5.1	6.3	11.92	1.00	0.25	3.05
7	90	8015	10.4	6.1	5.3	11.92	1.00	0.21	3.59
8	90	8050	10.4	5.3	6.1	11.92	1.00	0.24	3.14
9	90	8085	10.4	4.8	6.6	11.92	1.00	0.26	2.88
10	90	8120	10.4	5.3	6.1	11.92	1.00	0.24	3.12
11	90	8155	10.4	4.1	7.3	11.92	1.00	0.29	2.61
12	90	8190	10.4	3.0	8.4	11.92	1.00	0.33	2.28
13	90	8225	10.4	2.4	9.0	11.92	1.00	0.36	2.12
14	90	8260	10.4	2.0	9.4	11.92	1.00	0.38	2.03
15	90	8295	10.4	1.7	9.7	11.92	1.00	0.39	1.97
16	90	8330	10.4	1.5	9.9	11.92	1.00	0.40	1.93
17	90	8365	10.4	1.4	10.0	11.92	1.00	0.40	1.90
18	90	8400	10.4	1.3	10.1	11.92	1.00	0.40	1.89
19	90	8435	10.4	1.3	10.1	11.92	1.00	0.40	1.89
20	90	8470	10.4	1.3	10.1	11.92	1.00	0.40	1.90
21	90	8505	10.4	1.4	9.9	11.92	1.00	0.40	1.92
22	90	8540	10.4	1.6	9.8	11.92	1.00	0.39	1.96
23	90	8575	10.4	1.9	9.4	11.92	1.00	0.38	2.02
24	90	8610	10.4	2.5	8.9	11.92	1.00	0.36	2.15
25	90	8645	10.4	3.6	7.8	11.92	1.00	0.31	2.44
26	90	8680	10.4	4.5	6.9	11.92	1.00	0.27	2.78
27	90	8715	10.4	6.6	4.7	11.92	1.00	0.19	4.04
28	90	8750	10.4	5.3	6.1	11.92	1.00	0.24	3.13
29	90	8785	10.4	5.4	6.0	11.92	1.00	0.24	3.18
30	90	8820	10.4	6.3	5.1	11.92	1.00	0.20	3.75
31	90	8855	10.4	7.2	4.2	11.92	1.00	0.17	4.58
32	90	8890	10.4	5.2	6.2	11.92	1.00	0.25	3.08
33	90	8925	10.4	5.6	5.8	11.92	1.00	0.23	3.30
34	90	8960	10.4	5.8	5.5	11.92	1.00	0.22	3.45
35	90	8995	10.4	5.1	6.2	11.92	1.00	0.25	3.06
36	90	9030	10.4	5.0	6.3	11.92	1.00	0.25	3.01
37	90	9065	10.4	6.4	5.0	11.92	1.00	0.20	3.80
38	90	9100	10.4	4.0	7.4	11.92	1.00	0.30	2.58
39	90	9135	10.4	4.3	7.1	11.92	1.00	0.28	2.70
40	90	9170	10.4	2.5	8.9	11.92	1.00	0.36	2.15
41	90	9205	10.4	1.8	9.6	11.92	1.00	0.38	1.99

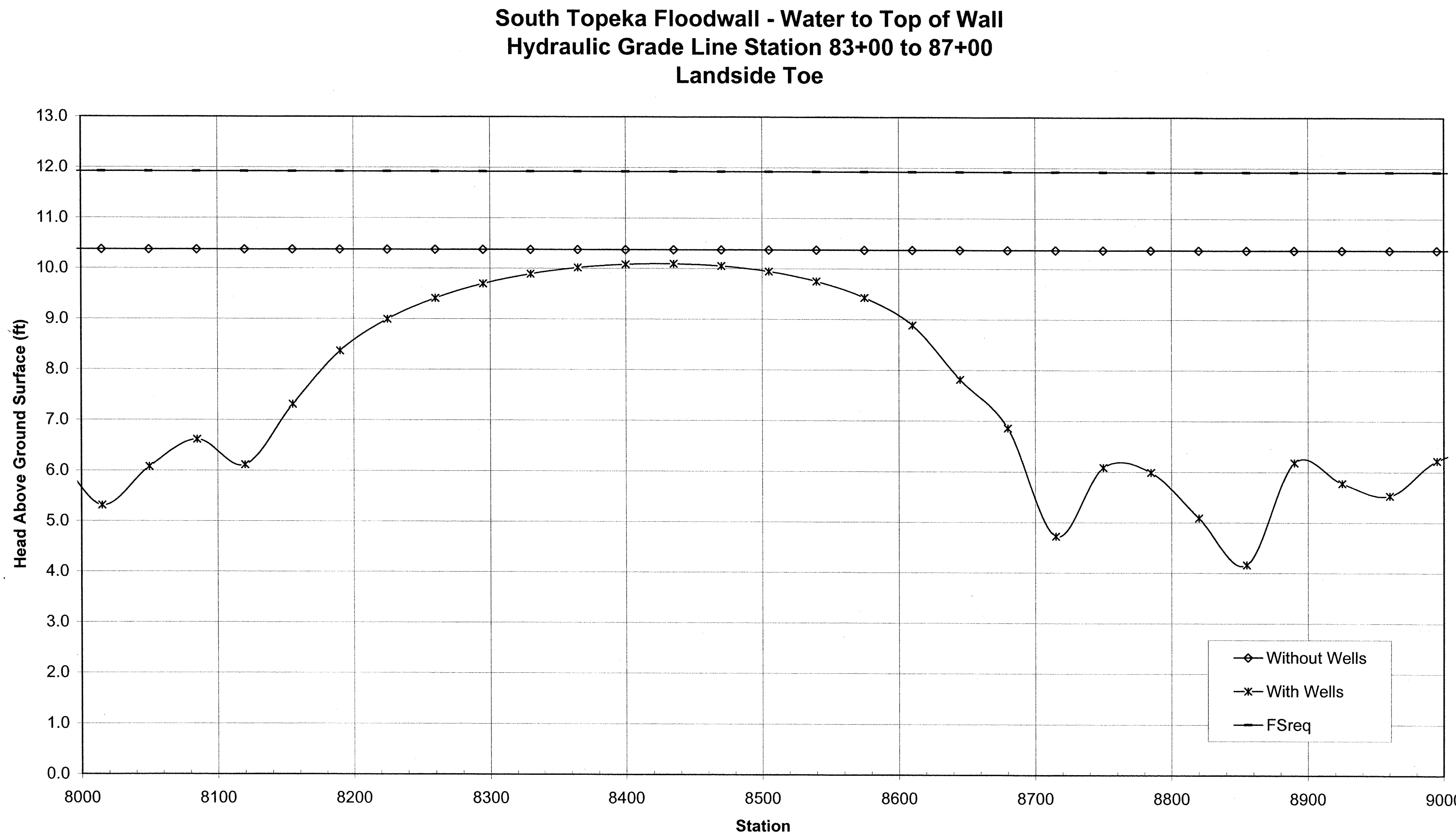
2/5


South Topeka Flood Protection Project - Water to Top of Wall
Hydraulic Grade Line Station 83+00 to 87+00
Critical Station = 84+50
Head Above Ground Surface (ft)
Distance from Seepage Entrance Point (ft) (positive indicates landward)
Without Wells
With Wells
FSreq
Landside Toe
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0
100
200
300
400
500
600
700
800
900
1000
1100



South Topeka Flood Protection Project - Water to Top of Wall
Hydraulic Grade Line Station 89+00
Head Above Ground Surface (ft)
14.0
12.0
10.0
8.0
6.0
4.0
2.0
0.0
0
100
200
300
400
500
600
700
800
900
1000
1100
Distance from Seepage Entrance Point (ft) (positive indicates landward)
Without Wells
With Wells
FSreq
Landside Toe

4/5


South Topeka Floodwall - Water to Top of Wall
Hydraulic Grade Line Station 83+00 to 87+00
Landside Toe
Head Above Ground Surface (ft)
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
8000
8100
8200
8300
8400
8500
8600
8700
8800
8900
9000
Station
Without Wells
With Wells
FSreq

5/5

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Exhibit 8

Hand Calculations for Design Loading Condition
Station 84+50 and Station 89+00

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SOUTH TOPEKA FLOODWALL - BUILT BY THE CORPS IN 1930'S

PILE FOUNDATION ANALYSIS

THE CRITICAL SECTION APPEARS TO BE BETWEEN STA 83+00 AND 87+00

- BLANKET THICKNESS : 25'
- AQUIFER THICKNESS : AVG APP: 50'

LOWER PART OF BLANKET CONTAINS ≈ 10' OF ORGANIC MATERIAL

- THERE IS A BURIED COLLECTOR CONTINUOUS ALONG THE LANDSIDE TOE OF THE WALL. THE COLLECTOR PENETRATES THE BLANKET. THE SYSTEM DRAINS INTO THE MADISON STREET PUMPING PLANT

SOIL STRENGTH PARAMETERS

- PHASE I GEOTECHNICAL REPORT

FILL : DRAINED $\phi' = 24°$, $c = 0$

EMB : DRAINED $\phi' = 26.5°$, $c = 0$

FND CLAY: DRAINED $\phi' = 22°$, $c = 0$

SAND : $\phi' = 33°$

COULD NOT FIND UNDRAINED STRENGTHS IN PREVIOUS DESIGN DOCUMENTATION

USE VALUES FROM FEB 2006 PILE ANALYSIS

FND CLAY: 600 psf

FOR FILL ?? VARIES SIGNIFICANTLY, FOR ORGANICS, USE $S_U = 400$ psf

UNIT WEIGHTS - FROM FEB 2006 PILE ANALYSIS

FND CLAY : $\gamma_{SAT} = 110$ pcf

SAND SP-SM: $\gamma_{SAT} = 115$ pcf

FOR FILL ?? VARIES SIGNIFICANTLY, FOR ORGANICS, USE $\gamma_{SAT} = 100$ pcf

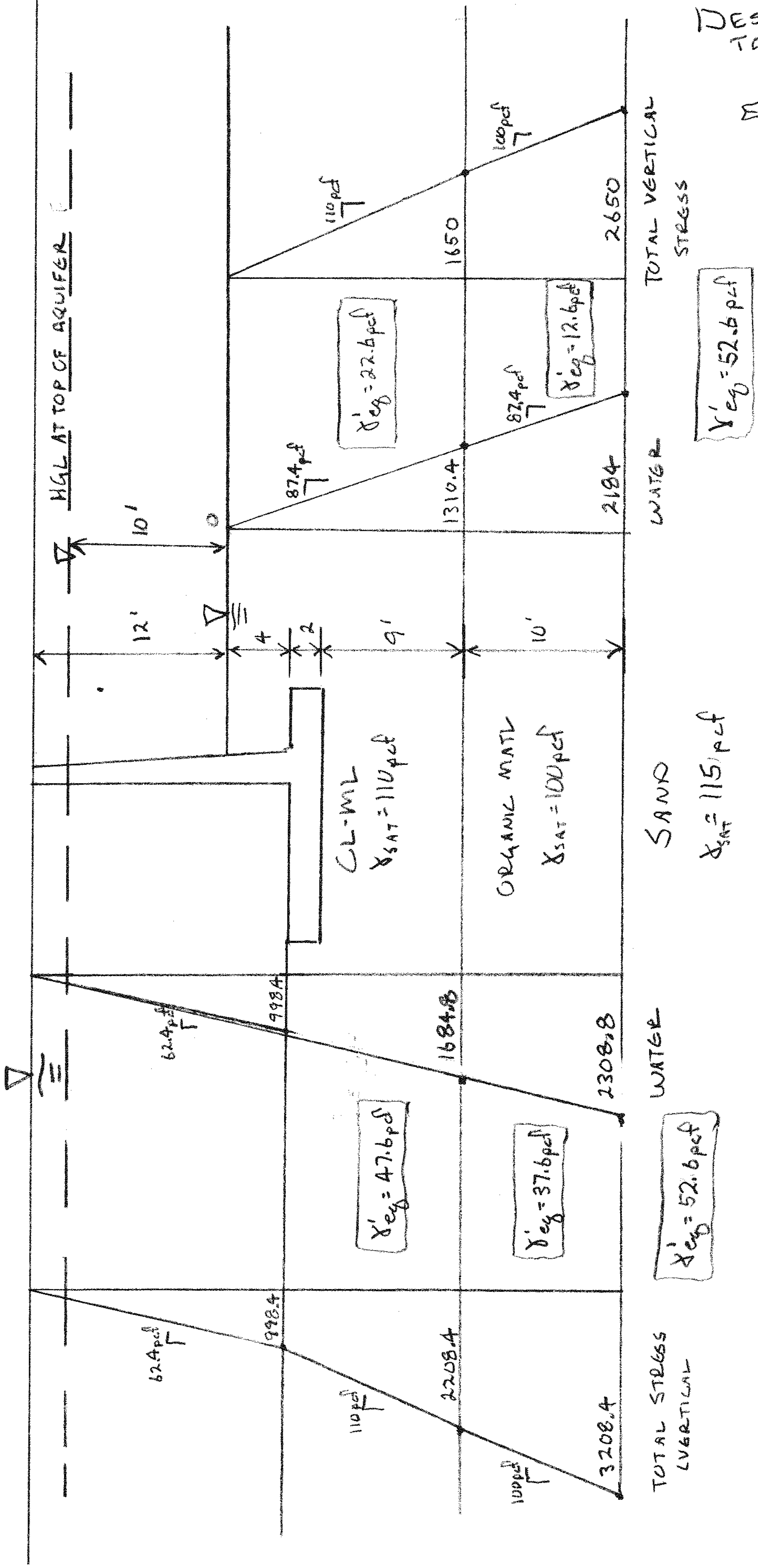
SOUTH TOPEKA FLOODWALL - STA 84+50 6 OCT 2007

3/8

- USE COMPUTED HGL WITH WELLS CASE TO DETERMINE EFFECTIVE UNIT WEIGHTS OF SOIL.

DESIGN LOADING WATER TO TOP OF WALL.

* USE THE LANDSIDE EQUIVALENT UNIT WEIGHTS FOR THE A PILE ANALYSIS.



SOUTH TOPEKA FLOODWALL ANALYSIS

- AFTER THE INITIAL RELIABILITY ANALYSIS SHOWED A VERY HIGH PROBABILITY OF FAILURE (92%) WITH WATER TO THE TOP OF THE WALL, THE ASSUMED PARAMETERS WERE SCRUTINIZED CLOSELY.

IN ADDITION TO THIS, THE 1938 CONSTRUCTION DRAWINGS FOR THE FLOODWALL WERE FOUND, PROVIDING PILE SPACING INFORMATION WHICH WAS PREVIOUSLY UNKNOWN.

BASED UPON FURTHER INVESTIGATIONS, THE FOLLOWING CHANGES WILL BE MADE:

1) REVISE PILE DIAMETER
   ORIGINAL: 10" AT TOP AND 4" AT TIP
   NEW: 12" AT TOP AND 9" AT TIP.

2) REVISE K USED IN SAND DUE TO CLOSE SPACING OF PILES NOTED IN THE CONSTRUCTION DRAWINGS THAT WERE FOUND.
   ORIGINAL. $K = 1.0$
   NEW: $K = 1.5$

3) REVISE FOOTING DEPTH BASED UPON CONSTRUCTION DRAWINGS
   ORIGINAL: $D = 6'$
   NEW: $D = 7.5'$?

* NOTE: HAND CALCULATIONS WERE PERFORMED AND THE DRAINED ANALYSIS CONTROLS. USE DRAINED ANALYSIS FOR RELIABILITY CALCS.

SINCE THE DRAINED ANALYSIS CONTROLS, NEED DRAINED STRENGTHS OF CLAY AND ORGANICS:
FROM THE RELIABILITY ANALYSIS DONE FOR THE SOUTH TOPEKA LEVEE

FILL & ?: $\phi' = 24°$

ASSUME FOR ORGANICS: $\phi' = 17°$

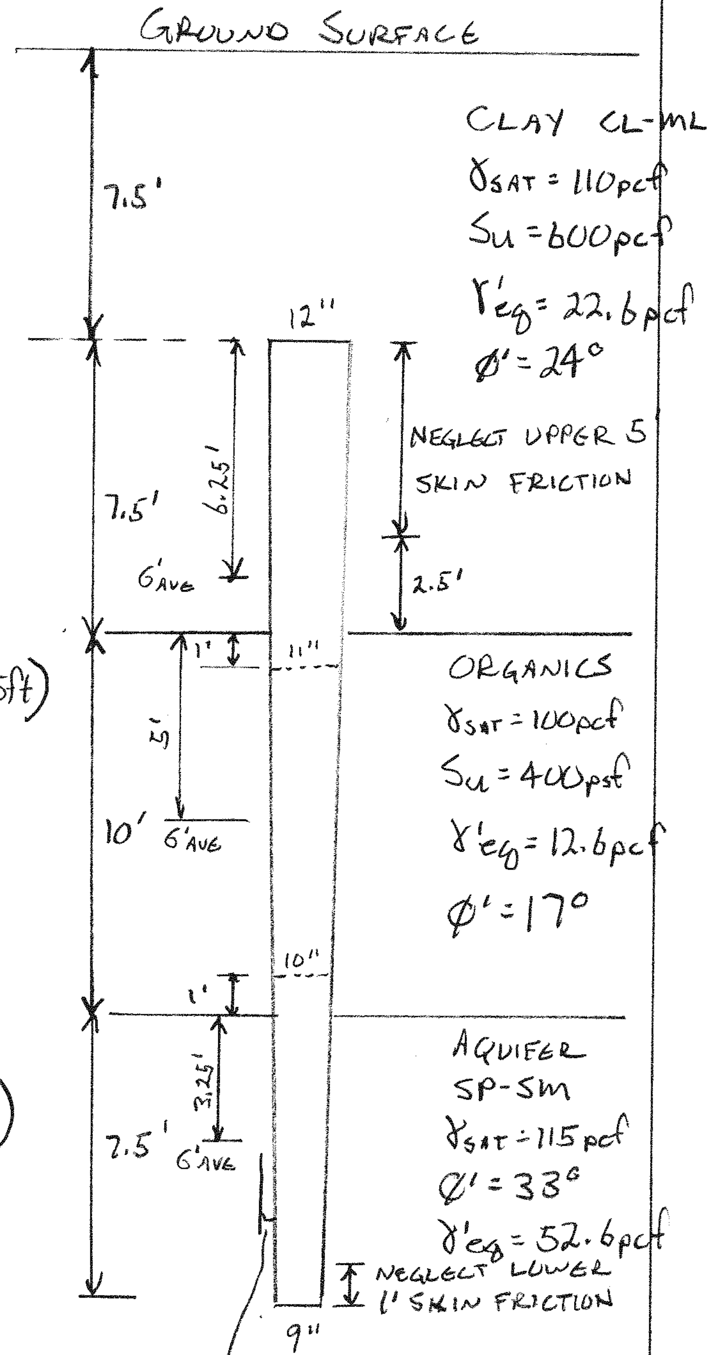
SOUTH TOPEKA FLOODWALL — 8 JAN 2008

STA 84+50 - AXIAL CAPACITY PILE ANALYSIS WITH WATER TO THE TOP OF WALL — RSK — 1/8

HEIGHT OF WALL = 12'
HGL W/ WELLS = 10'

UNDRAINED ANALYSIS

(USE AVERAGE DIAMETER)



SIDE FRICTION

CLAY

$Q_s = \alpha S_u A_s$

FROM FIG 4-5a, EM 1110-2-2906

$\alpha = 0.9$

$Q_s = (0.9)(600\text{psf})(\pi)(11.25/12\text{ft})(2.5\text{ft})$

$Q_s = \underline{3.98}$ kips

ORGANICS

$Q_s = \alpha S_u A_s$

.75 $\alpha = 1.0$

$Q_s = (1.0)(400\text{psf})(\pi)(\frac{10.5''}{12}\text{ft})(10\text{ft})$

$Q_s = \underline{11.0}$ kips

AQUIFER

$Q_s = f_s A_s \quad f_s = K\sigma_v' \tan\delta$

FROM EM 2906 $\delta = 0.9\phi' = 0.9(33°) = 30°$ — $\Theta = 0.28°$

FROM EM, $1.0 < K < 2.0$ IN SANDS — (TOTAL TAPER ≈ 0.1'/10' LENGTH)
DUE TO CLOSELY SPACED PILES, USE $K = 1.5$

$\sigma_v' = (15\text{ft})(22.6\text{pcf}) + (10\text{ft})(12.6\text{pcf}) + (3.25\text{ft})(52.6\text{pcf}) = 635.9\text{psf}$

$Q_s = (1.5)(635.9\text{psf})\tan 30° (\pi)(\frac{9.5\text{in}}{12}\text{ft})(6.5\text{ft})$

$Q_s = 8.90$ kips

TOTAL SKIN FRICTION:

BLANKET: $3.98^k + 11.0^k =$ 15.0^k

SAND: 8.9^k

TOTAL: 23.9^{kips}

TIP CAPACITY IN SAND

$Q_p = \sigma_v' N_q A_p$

FROM EM 2906 FIG 4.4 ; $N_q = 40$

$\sigma_v' = (15')(22.6pcf) + (10ft)(12.6pcf) + (7.5ft)(52.6pcf) = 859.5\ psf$

$Q_p = (859.5psf)(40)\left(\frac{\pi}{4}\right)\left(\frac{9in}{12ft}\right)^2$

$Q_p = 15.2^{kips}$

TOTAL CAPACITY FOR UNDRAINED LOADING: $23.9^{kips} + 15.2^{kips} = \boxed{39.1^{kips}}$

DRAINED ANALYSIS

SIDE FRICTION

CLAY

$Q_s = f_s A_s \qquad f_s = K\sigma_v' \tan\delta$

FROM EM 2906

$\delta = 0.9\phi = 0.9(24°) = 21.6°$

$K = 1.0$

$\sigma_v' = (7.5' + 6.25ft)(22.6pcf) = 310.75\,psf$

$Q_s = (1.0)(310.75\,psf)\tan 21.6° (\pi)(11.25in/12\,ft)(2.5ft)$

$Q_s = \underline{0.9^{kips}}$

ORGANICS

$Q_s = f_s A_s \qquad f_s = K\sigma_v' \tan\delta$

$\delta = 0.9\phi' = 0.9(17°) = 15.3°$

$K = 1.0$

$\sigma_v' = (15ft)(22.6\,pcf) + (5ft)(12.6pcf) = 402\,psf$

$Q_s = (1.0)(402.0psf)\tan 15.3° (\pi)(10.5in/12\,ft)(10ft)$

$Q_s = \underline{3.02^{kips}}$

TOTAL SKIN FRICTION:

BLANKET: $0.9^k + 3.0^k = 3.9^{kips}$

SAND (FROM PREVIOUS) $= 8.9^{kips}$

$\underline{\text{TOTAL} \quad 12.8^k}$

ULTIMATE DRAINED LOADING CAPACITY

$Q_T = 12.8^k + 15.2^k$ (FROM PREVIOUS) $= \boxed{28.0^k}$

* THE DRAINED LOADING CONDITION CONTROLS

STA 89+00 - AXIAL PILE ANALYSIS WITH WATER TO THE TOP OF WALL.

THIS ANALYSIS IS BEING PERFORMED BECAUSE THIS SECTION IS FELT TO BE REPRESENTATIVE BUT CONSERVATIVE OF THE "REST OF THE WALL" WITH RESPECT TO FOUNDATION CONDITIONS.

HEIGHT OF WALL = 12'
HGL WITH WELLS = 6'

UNDRAINED ANALYSIS

CLAY

$Q_s = \alpha S_u A_s$

$\alpha = 0.9$

$Q_s = (0.9)(600 psf)(\pi)\left(\frac{10.5 in}{12} ft\right)(12.5 ft)$

$\underline{Q_s = 18.6 \text{ kips}}$

AQUIFER

$Q_s = f_s A_s \quad f_s = K \sigma_v' \tan \delta$

FROM PREVIOUS CALCS

$\delta = 30° \; ; \; K = 1.5$

$\sigma_v' = 25 ft (32.6 pcf) + (3.25 ft)(52.6 pcf)$

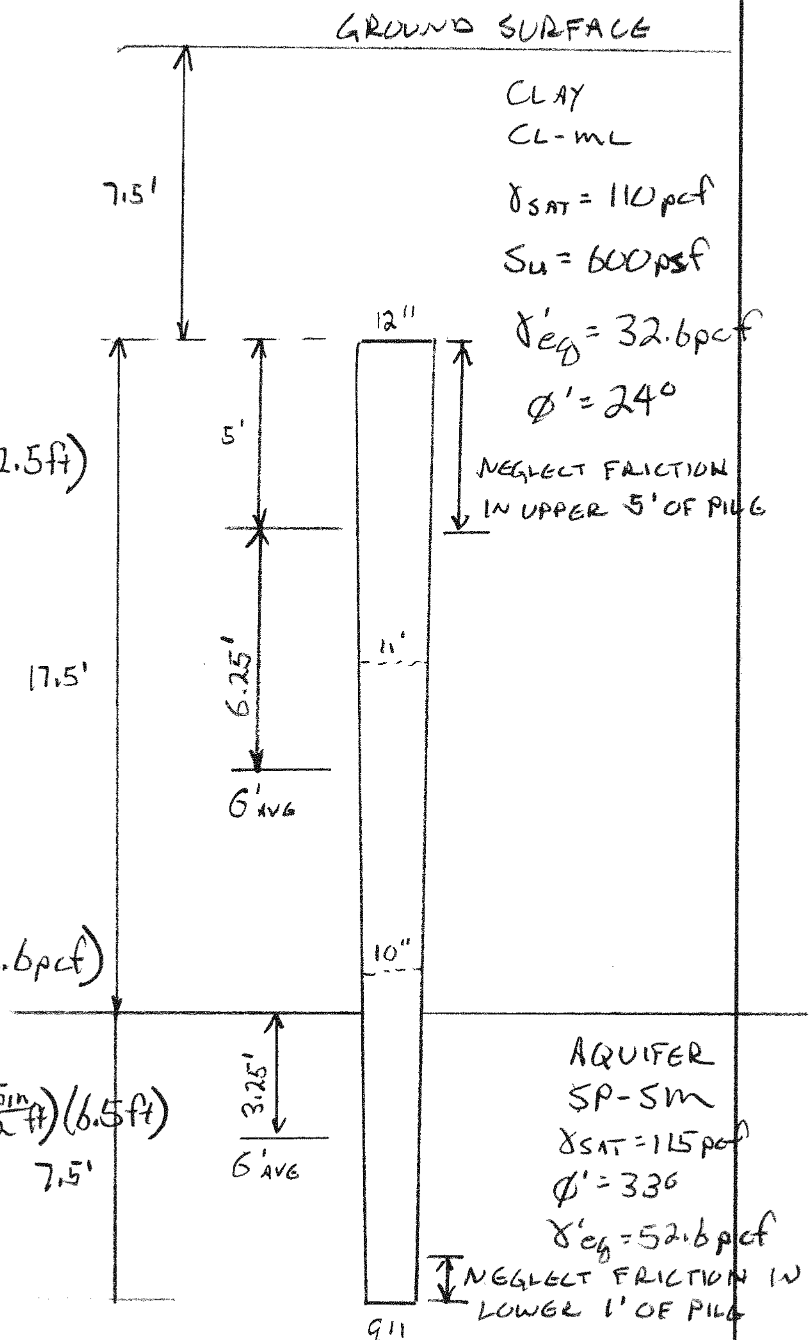
$\sigma_v' = 986.0 psf$

$Q_s = (1.5)(986 psf) \tan 30° (\pi)\left(\frac{9.5 in}{12} ft\right)(6.5 ft)$

$\underline{Q_s = 13.8 \text{ kips}}$

TOTAL: $18.6^k + 13.8^k =$

$\underline{Q_s = 32.4^{kips}}$



TIP CAPACITY IN SAND

$Q_p = \sigma_v' N_q A_p$

$N_q = 40$ FROM PREVIOUS CALCULATIONS

$\sigma_v' = (25ft)(32.6pcf) + (7.5ft)(52.6pcf) = 1209.5psf$

$Q_p = (1209.5psf)(40)\left(\frac{\pi}{4}\right)\left(\frac{9in}{12}ft\right)^2$

$Q_p = 21.4$ kips

ULTIMATE CAPACITY FOR UNDRAINED LOADING:

$Q_{ULT} = 32.4^{kips} + 21.4^{kips} = \boxed{53.8^{kips}}$

DRAINED LOADING CONDITION
(ONLY HAVE TO COMPUTE SIDE FRICTION IN CLAY)

CLAY

$Q_s = f_s A_s \quad f_s = K \sigma_v' \tan \delta$

FROM PREVIOUS CALCULATIONS

$K = 1.0 \quad ; \quad \delta = 21.6°$

$\sigma_v' = (18.75ft)(32.6pcf) = 611.25psf$

$Q_s = (1.0)(611.25psf) \tan 21.6° \, (\pi)\left(\frac{10.75''}{12}ft\right)(12.5ft)$

$Q_s = 8.51$ kips

TOTAL SKIN FRICTION:

$8.51^k + 13.8^{kips} = 22.3^{kips}$

ULTIMATE CAPACITY FOR DRAINED LOADING

$Q_{ULT} = 22.3^k + 21.4^k = \boxed{43.7^{kips}}$

∴ AGAIN, THE DRAINED LOADING CONDITION CONTROLS

COMPUTE ALLOWABLE AXIAL CAPACITIES USING EM 1110-2-2906 CRITERIA.

FOR UNUSUAL LOADING ; FS = 2.25

FOR EXTREME LOADING ; FS = 1.7

$$Q_{ALL} = \frac{Q_{ULT}}{FS}$$

WATER TO THE TOP OF THE WALL IS CONSIDERED EXTREME LOADING

* BASED UPON PILE TAPER; ASSUME ULTIMATE CAPACITY IN TENSION IS 50% OF THE ULTIMATE CAPACITY IN COMPRESSION.

ULTIMATE AXIAL CAPACITY

	COMPRESSION		TENSION	
STA	UNDRAINED (kips)	DRAINED (kips)	UNDRAINED (kips)	DRAINED (kips)
84+50	39.1	28.0	19.6	14.0
89+00	53.8	43.7	26.9	21.8

ALLOWABLE AXIAL CAPACITY FOR EXTREME LOADING

	COMPRESSION		TENSION	
STA	UNDRAINED (kips)	DRAINED (kips)	UNDRAINED (kips)	DRAINED (kips)
84+50	23.0	16.5	11.5	8.2
89+00	31.6	25.7	15.8	12.8

ALLOWABLE AXIAL CAPACITY FOR UNUSUAL LOADING

	COMPRESSION		TENSION	
STA	UNDRAINED (KIPS)	DRAINED (KIPS)	UNDRAINED (KIPS)	DRAINED (KIPS)
84+50	17.4	12.4	8.7	6.2
89+00	23.9	19.4	12.0	9.7

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Exhibit 9

Reliability Analysis
Miscellaneous Hand Calculations

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PARAMETERS FOR RELIABILITY ANALYSIS

SHEAR STRENGTH

SAND (ϕ') $\mu_x = 33°$; COV = 15%

LOWER BOUND; $33° - [(0.15)(33°)] = 28°$

UPPER BOUND; $33° + [(0.15)(33°)] = 38°$

CL-ML (FILL) $\mu_x = 24°$

EM 1110-2-556 RECOMMENDS COV ≈ 10%; DUE TO UNCERTAINTY USE 30%

LOWER BOUND; $24° - [(0.30)(24°)] = 16.8°$

UPPER BOUND; $24° + [(0.30)(24°)] = 31.2°$

ORGANICS; $\mu_x = 17°$

AGAIN, USE COV = 30°

LOWER BOUND; $17° - [(0.30)(17°)] = 11.9°$

UPPER BOUND; $17° + [(0.30)(17°)] = 22.1°$

PILE LENGTH $\mu_x = 25'$

CONSTRUCTION DRAWINGS STATE A MINIMUM DRIVE LENGTH OF 20 FT.

ASSUME COV = 20%

LOWER BOUND; $25' - [(0.2)(25')] = 20'$

UPPER BOUND; $25' + [(0.2)(25')] = 30'$

27 Nov. 2007

BLANKET THICKNESS

ASSUME COV = 25%

CL-ML ; M_x = 15' (7.5' PILE PENETRATION

LOWER BOUND ; 15' − [(0.25)(15')] = 11.25'

UPPER BOUND ; 15' + [(0.25)(15')] = 18.75'

ORGANIC MATL ; M_x = 10'

LOWER BOUND ; 10' − [(0.25)(10')] = 7.5'

UPPER BOUND ; 10' + [(0.25)(10')] = 12.5'

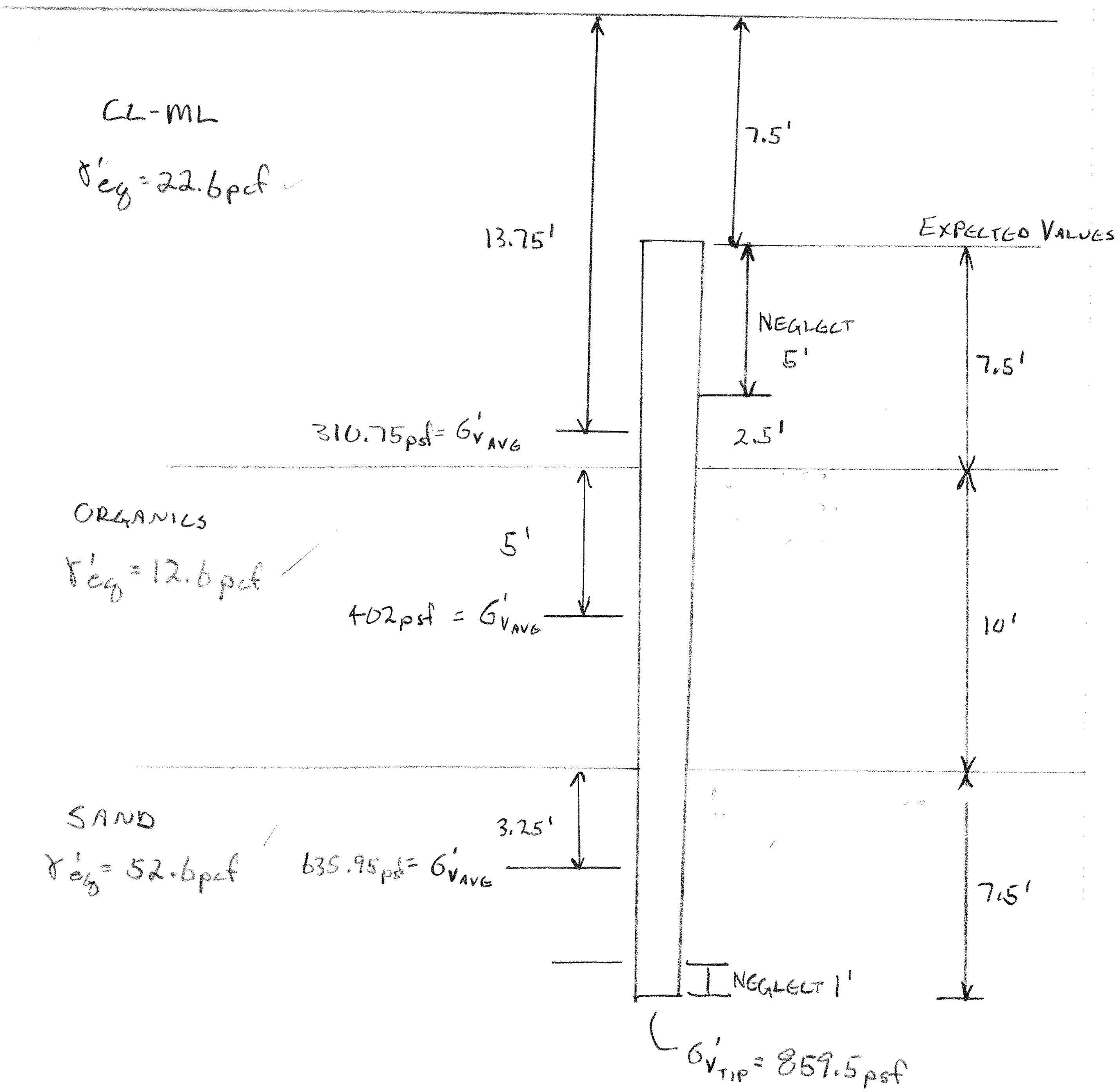
SUMMARY

VALUE	SHEAR STRENGTH			BLANKET THICKNESS		PILE LENGTH
	SAND	CL-ML	ORG	CL-ML	ORG	
EXPECTED	33	24	17	15'	10	25
LOW	28	16.8	11.9	11.25'	7.5	20
HIGH	38	31.2	22.1	18.75'	12.5	30

1 Dec 2007

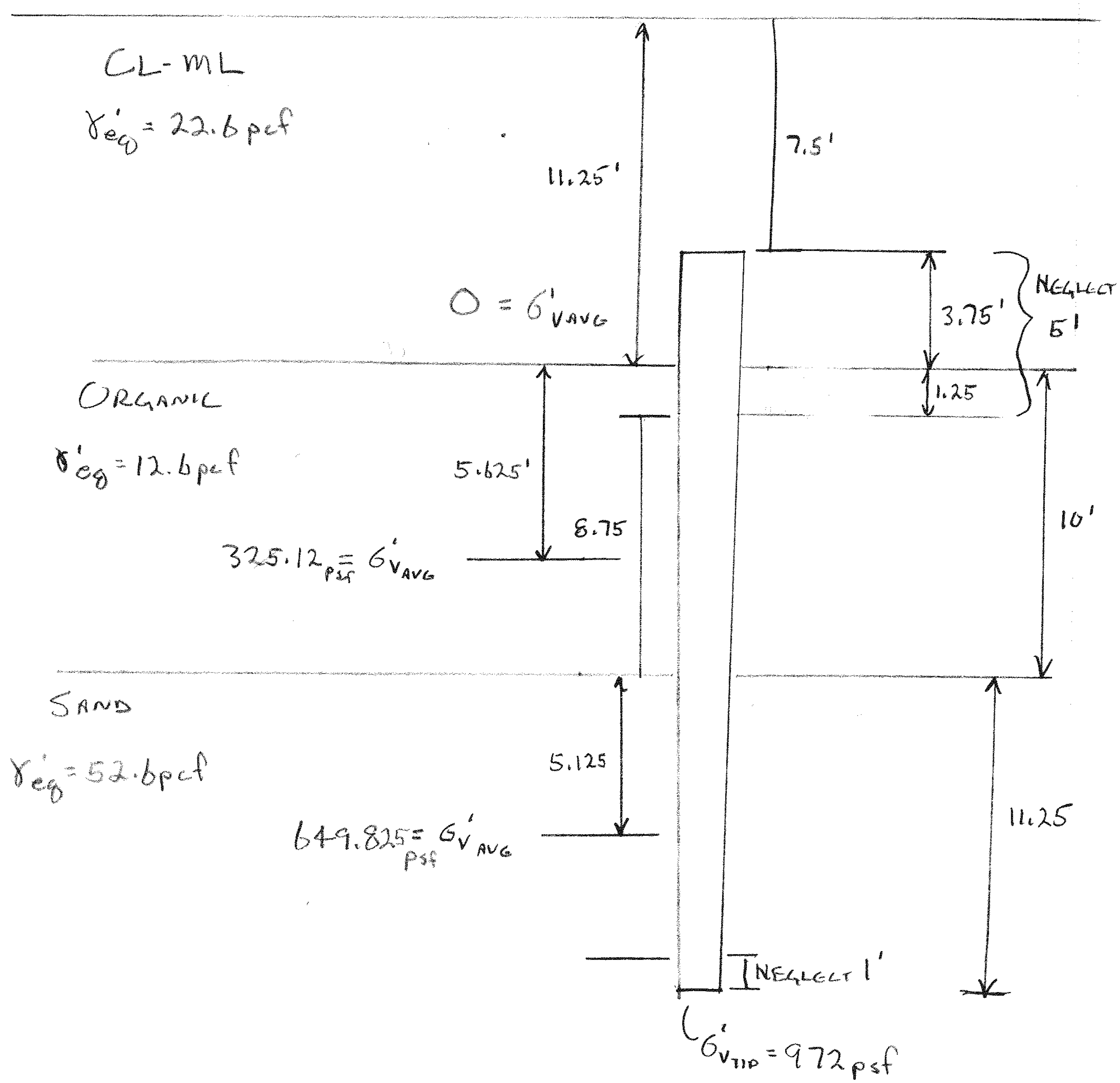
4/8

EXPECTED VALUE OF BLANKET THICKNESS - WATER TO TOP OF WALL
HGL CONSIDERS EFFECTS OF THE WELLS; SEE RELIEF WELL CALCULATIONS



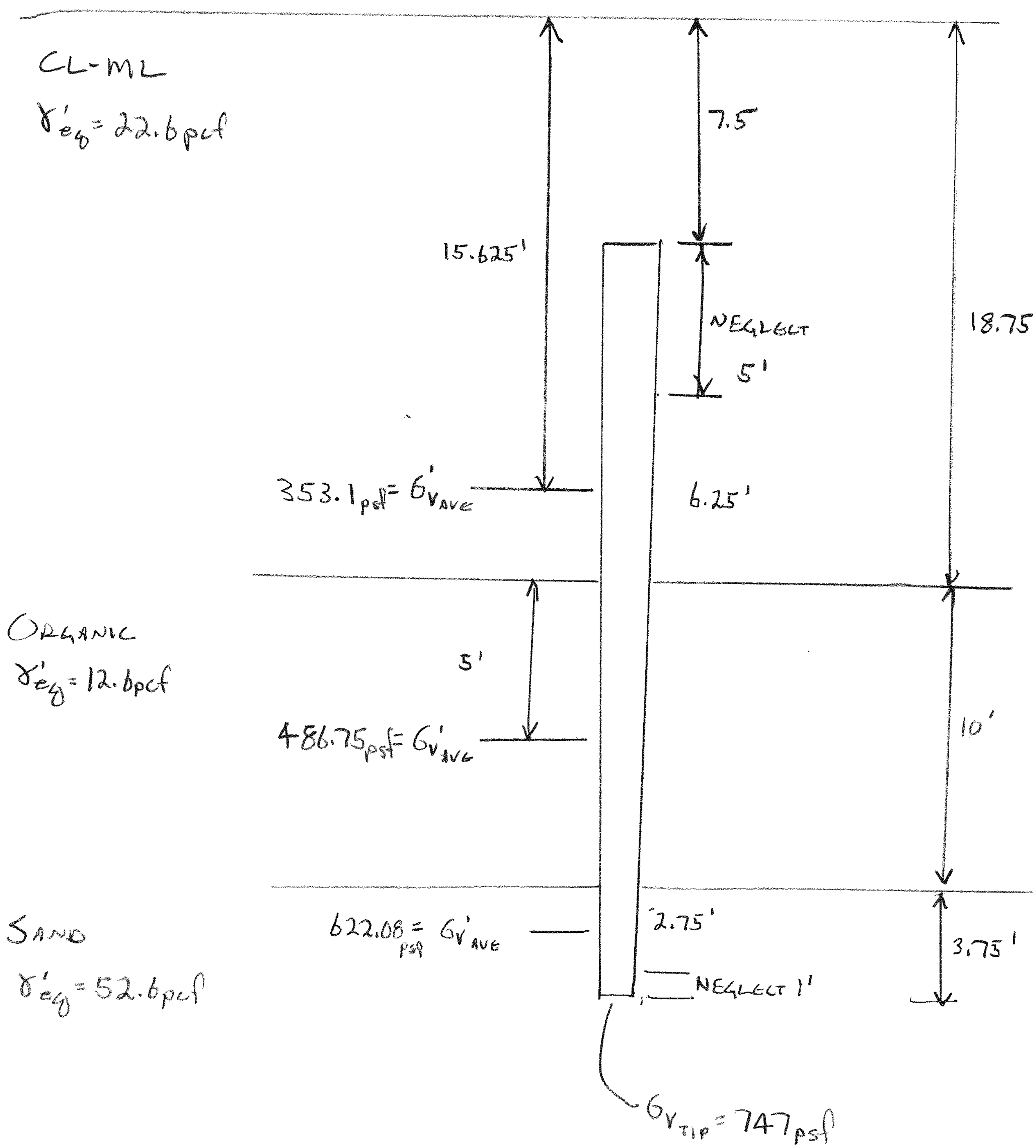
1 Dec 2007

LOWER COV CL-ML THICKNESS - WATER TO TOP OF WALL 5/8



1 Dec 2007

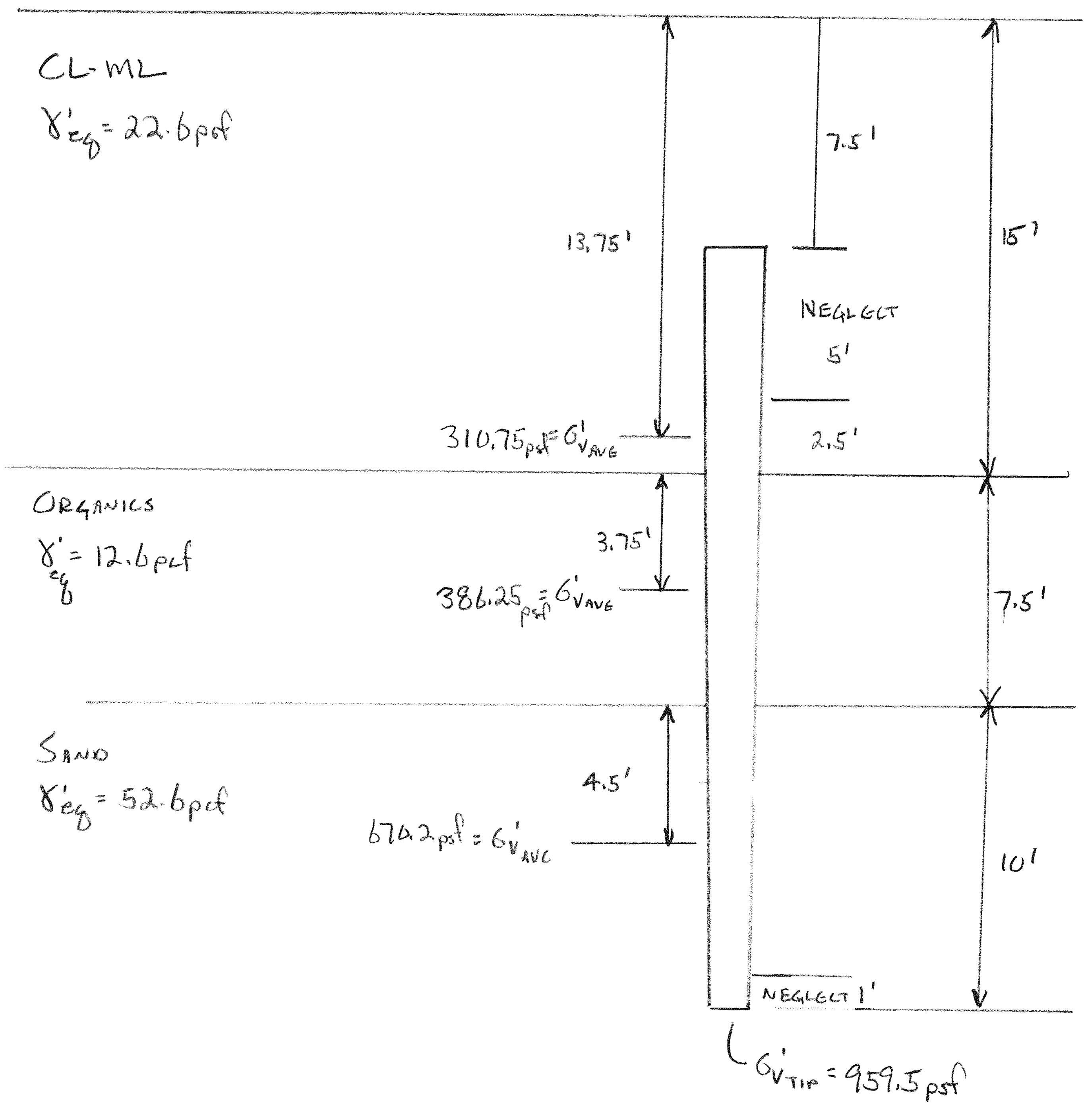
UPPER COV CL-ML THICKNESS - WATER TO TOP OF WALL



1 DEC 2009

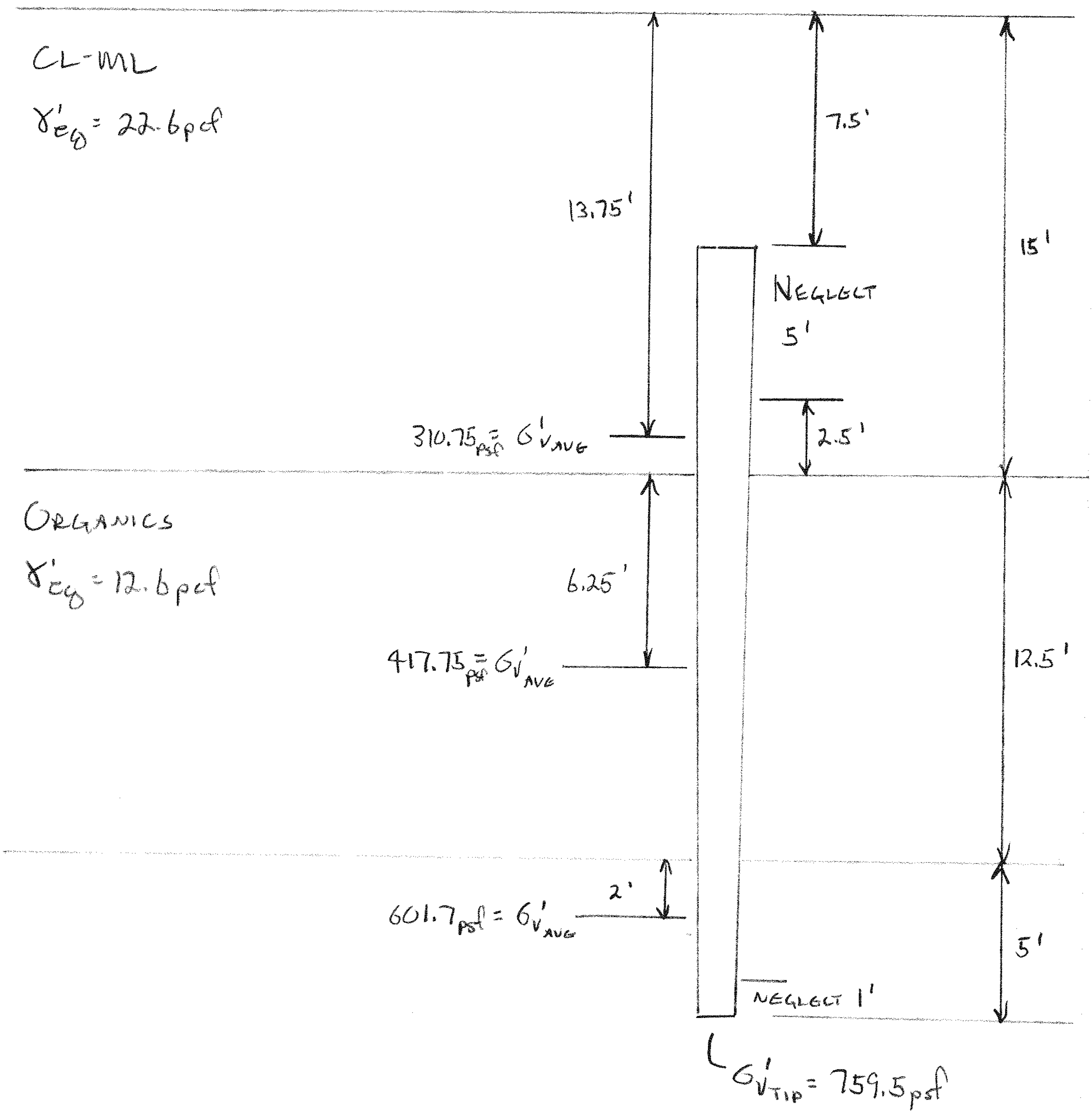
7/8

LOW COV ORGANIC THICKNESS - WATER TO TOP OF WALL



1 Dec 2007
RSK

HIGH COV ORGANIC THICKNESS - WATER TO TOP OF WALL



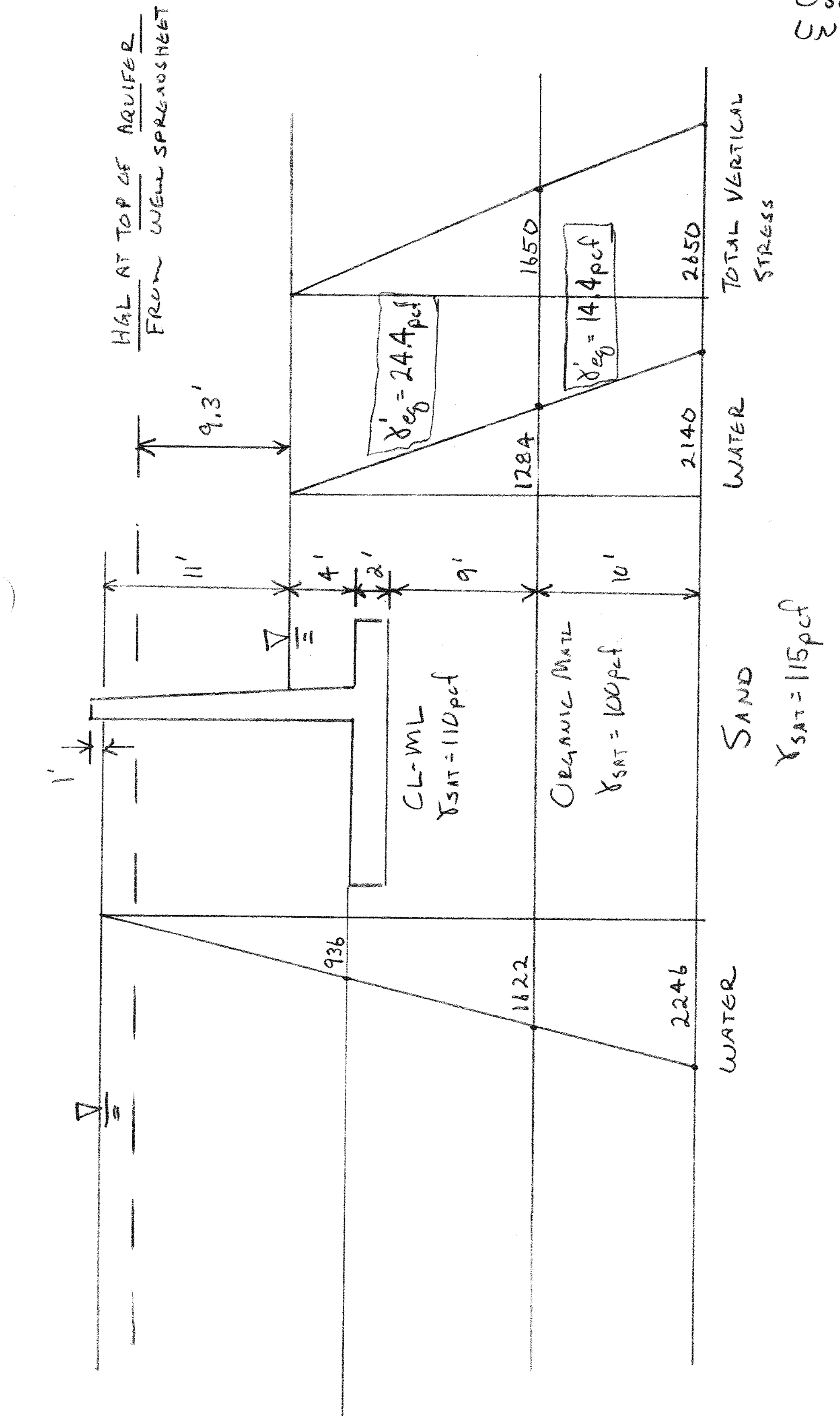
SOUTH TOPEKA FLOODWALL - STA 84+50 20 OCT 2007

RSK

8/

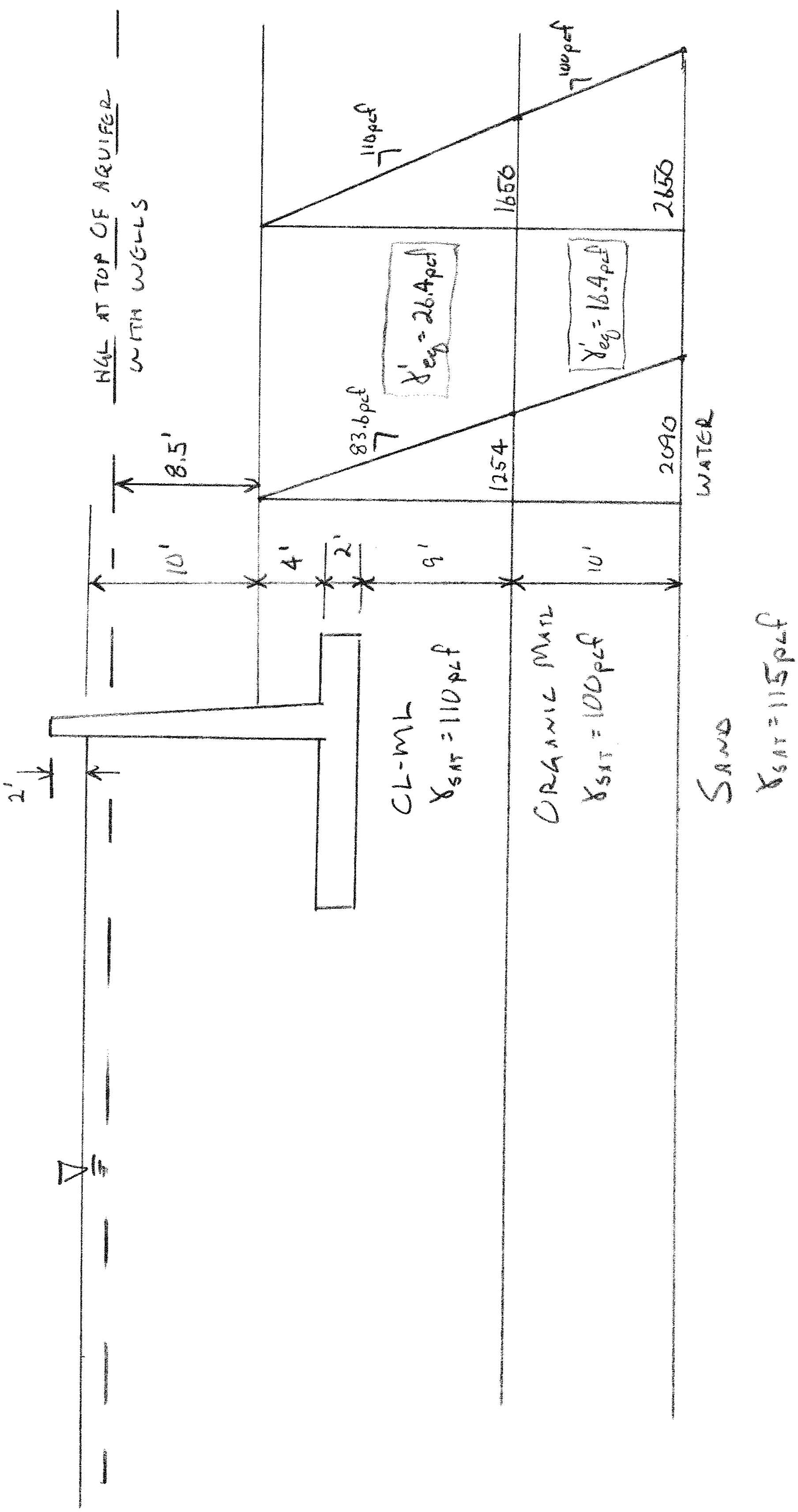
- USE COMPUTED HGL WITH WELLS CASE TO DETERMINE EFFECTIVE UNIT WEIGHTS OF SOILS
- WATER 1 FT BELOW TOP OF WALL

USE THE LANDSIDE EQUIVALENT UNIT WEIGHTS FOR APILE



SOUTH TOPEKA FLOODWALL STA 84+50 20 OCT 2007 10/

- USE COMPUTED HGL WITH WELLS CASE TO DETERMINE EFFECTIVE UNIT WEIGHTS OF SOILS
- WATER 2' BELOW TOP OF WALL

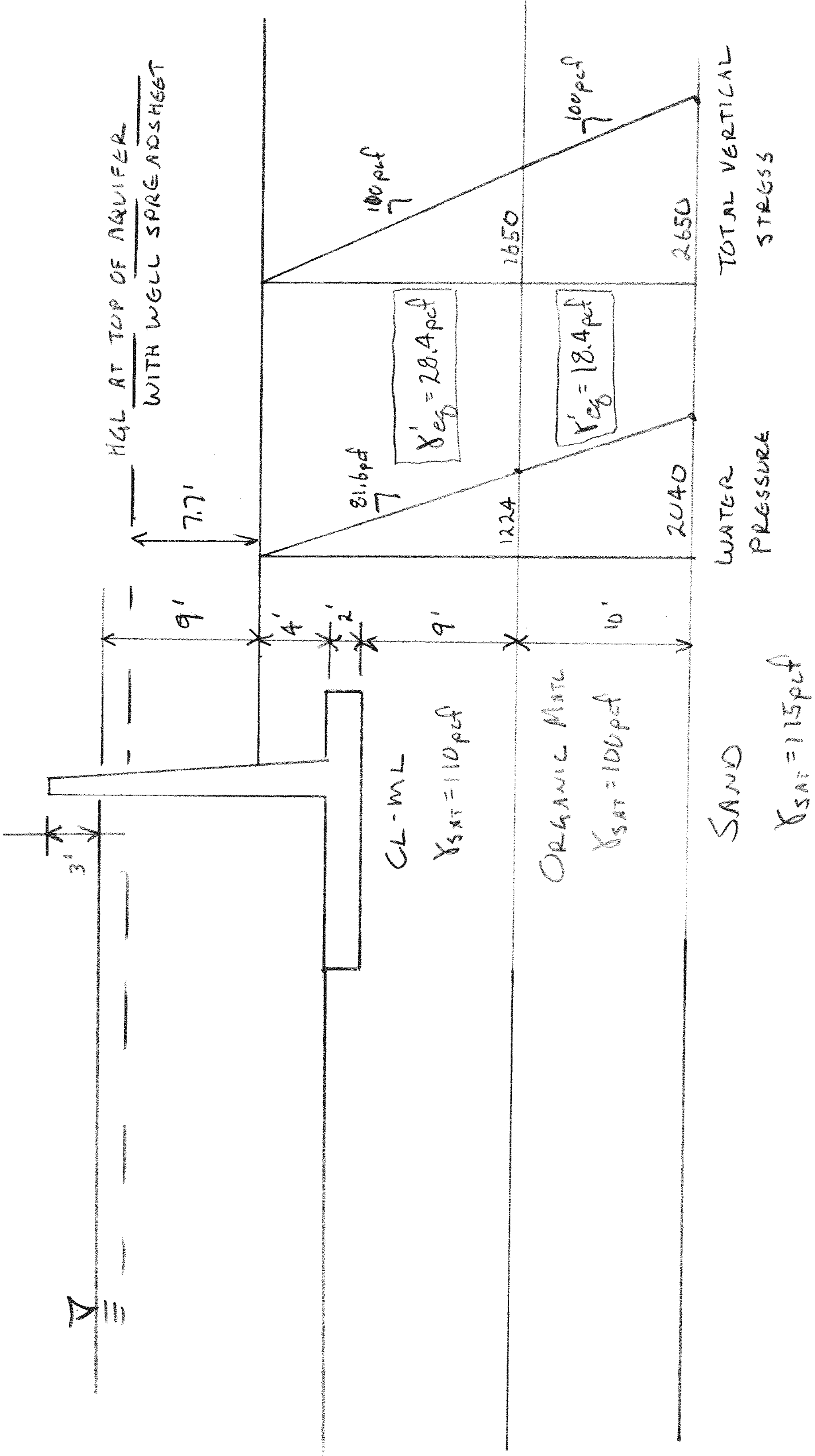


SOUTH TOPEKA FLOODWALL STA 84+50 20 OCT 2007

12/

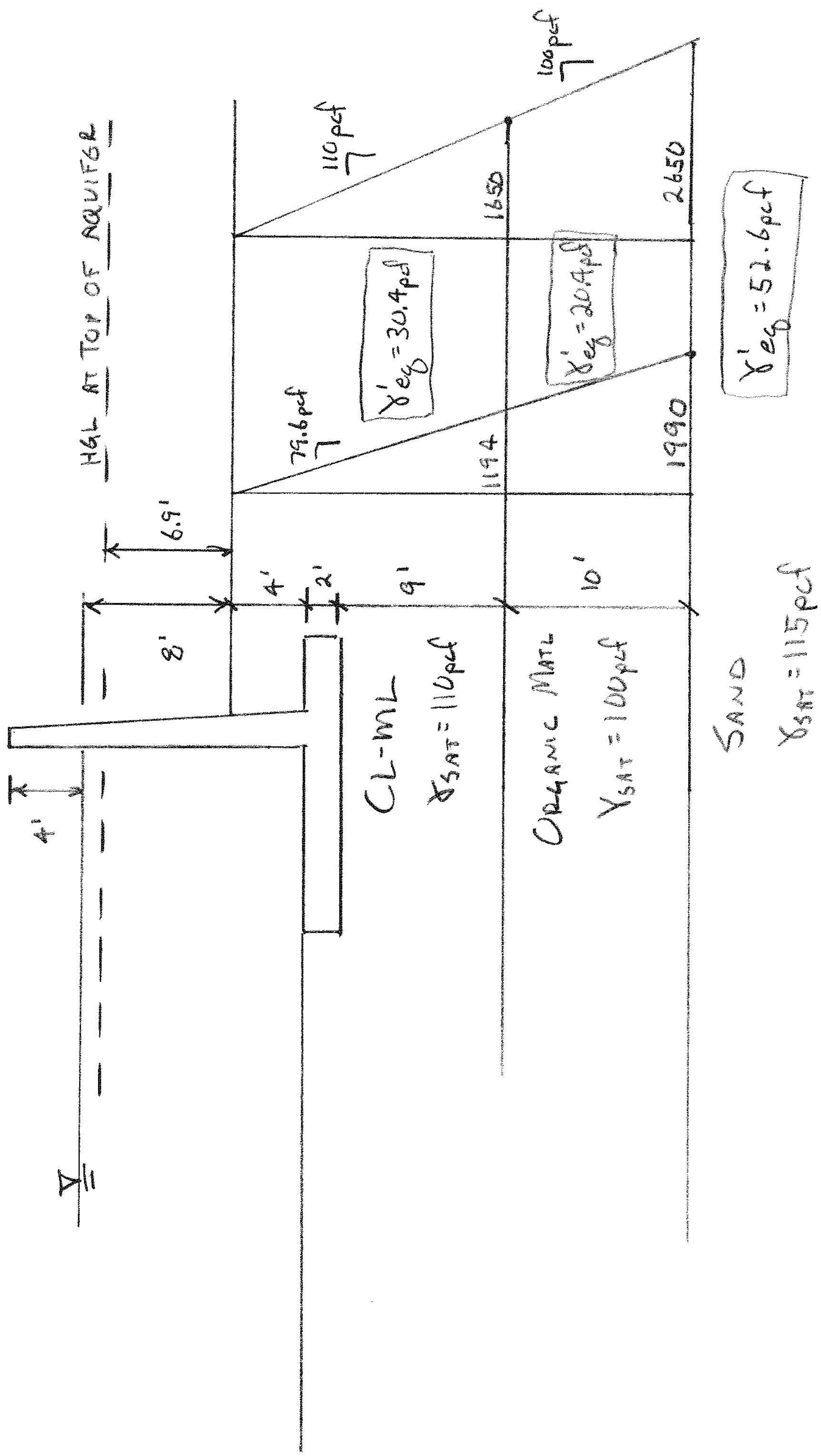
- USE COMPUTED HGL WITH WELLS CASE TO DETERMINE EFFECTIVE UNIT WEIGHT OF SOILS
- WATER 3' BELOW TOP OF WALL

FOR INPUT INTO APILE



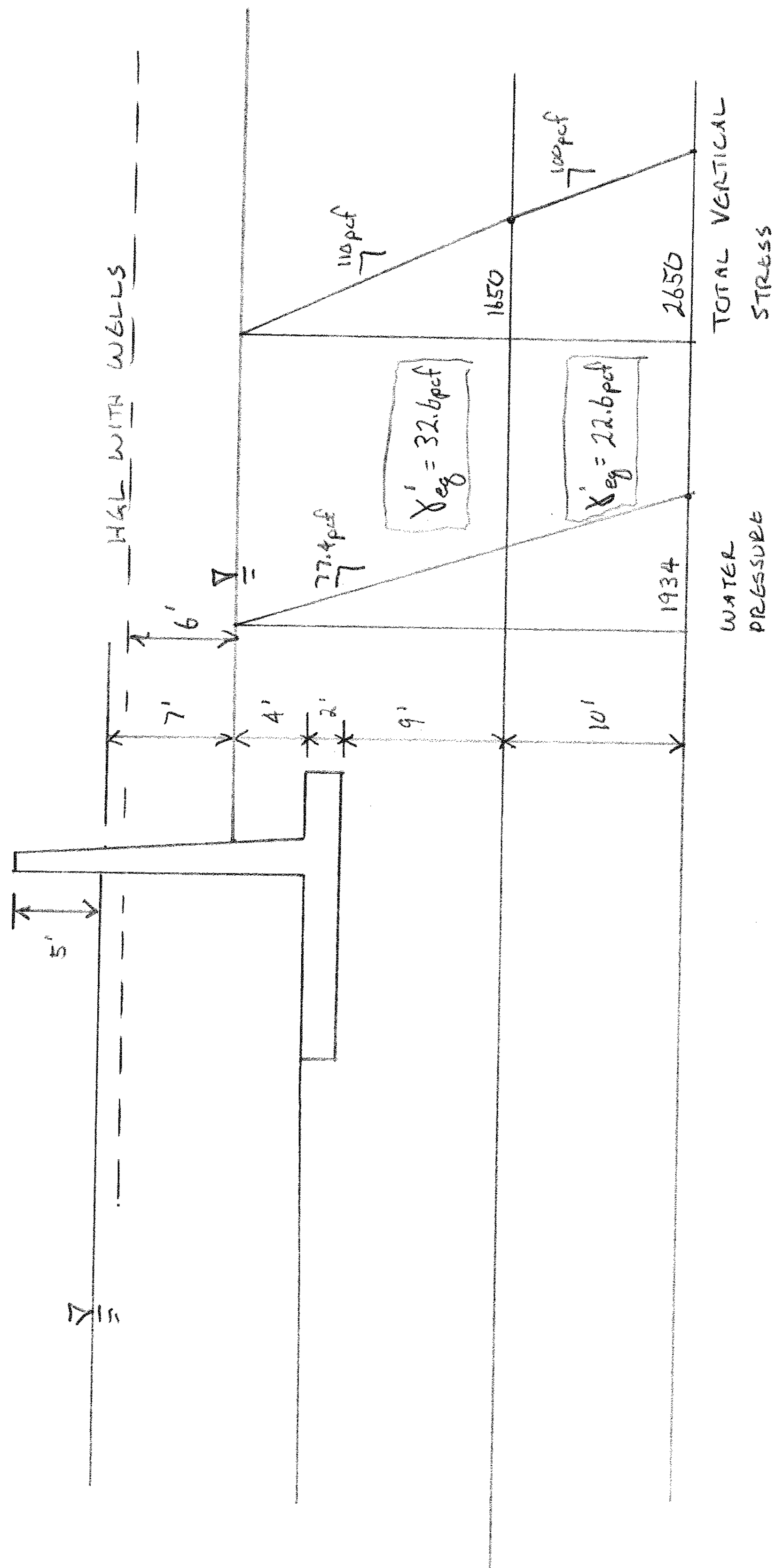
SOUTH TOPEKA FLOODWALL 84+50 20 OCT 2007
RSK

- USE COMPUTED HGL WITH WELLS CASE TO DETERMINE EFFECTIVE UNIT WEIGHT OF SOILS
- WATER 4' BELOW TOP OF WALL



SOUTH TOPEKA FLOODWALL 84+50 21 OCT 2007

- USE COMPUTED HGL WITH WELLS TO DETERMINE RSK 10/
EFFECTIVE UNIT WEIGHT OF SOILS
- WATER 5' BELOW TOP OF WALL:

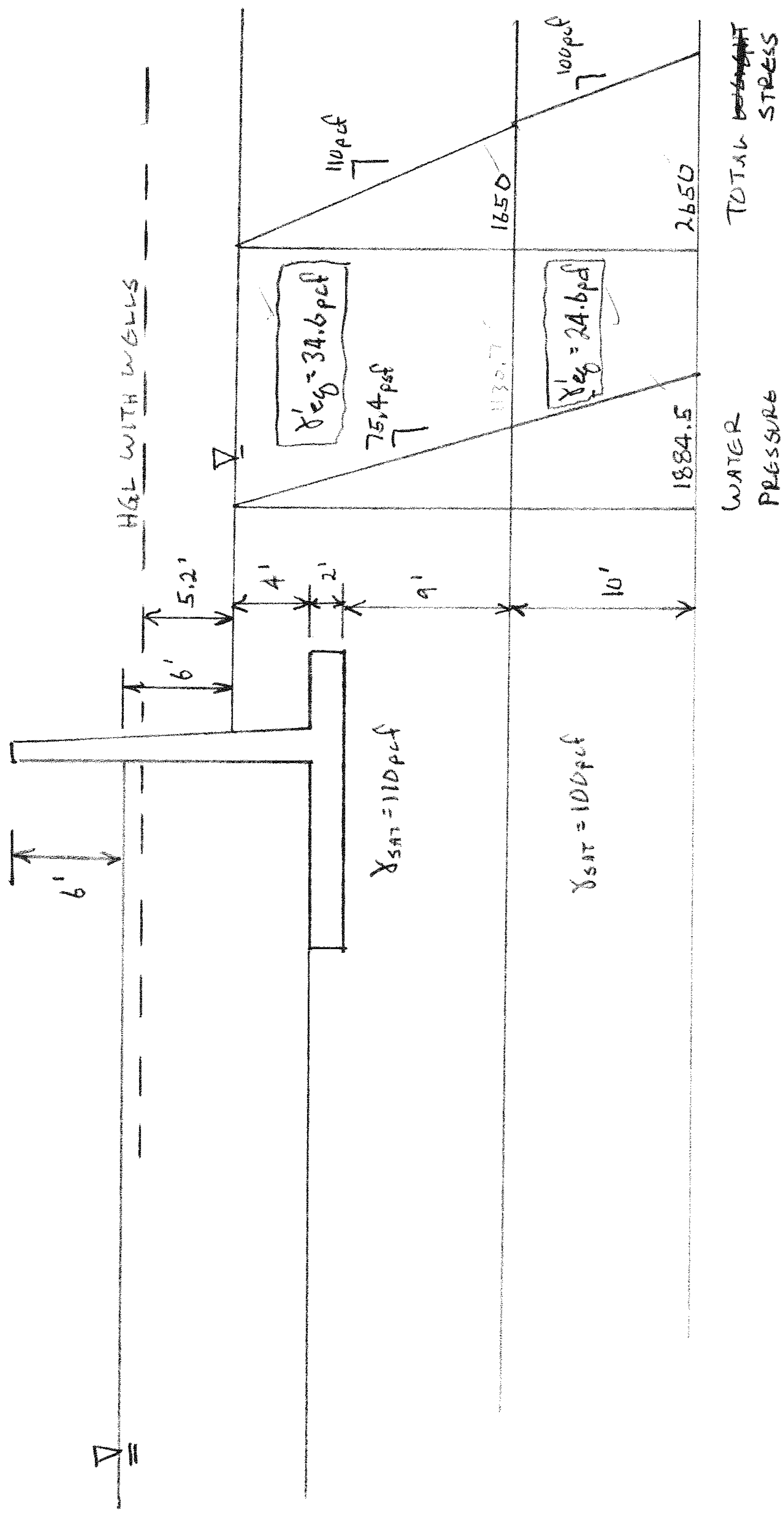


South Topeka Floodwall 84+50

- Use computed HGL with wells to determine effective unit weight of soils
- Water 6' below top of walls

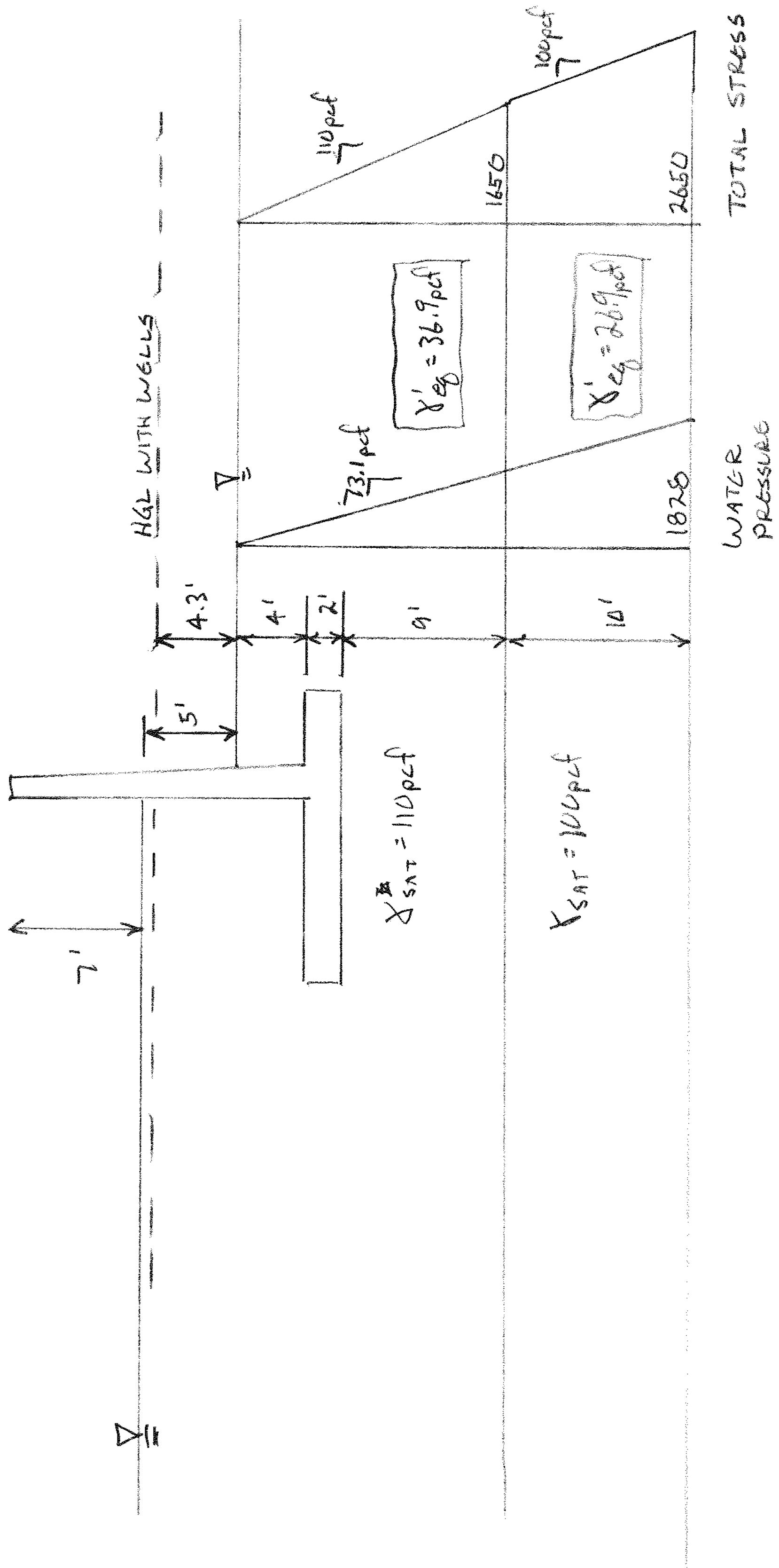
6 Dec 2007
RSK

Review: GMB
12/20/07



SOUTH TOPEKA FLOODWALL 84+50 6 DEC 2007

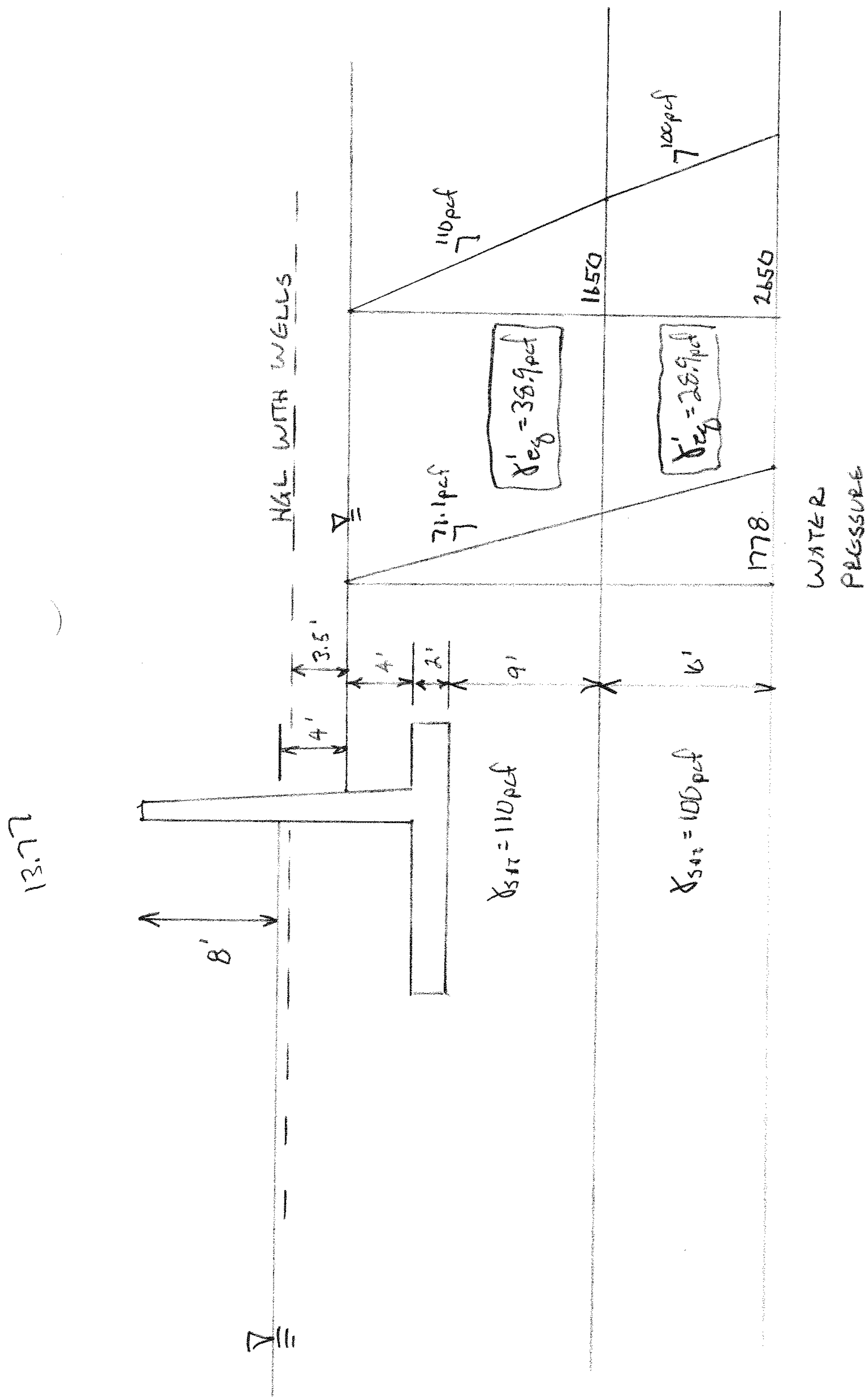
- USE COMPUTED HGL WITH WELLS TO DETERMINE EFFECTIVE UNIT WEIGHT OF SOILS RSK
- WATER 7' BELOW TOP OF WALLS



SOUTH TOPEKA FLOODWALL 84+50 6 DEC 2007

RSK

- USE COMPUTED HGL WITH WELLS TO DETERMINE EFFECTIVE UNIT WEIGHT OF SOILS
- WATER 8' BELOW TOP OF WALLS



South Topeka Floodwall Feasibility Study
Calculation of equivalent effective unit weight of the blanket materials

Landside Ground Surface Elevation:	880 ft msl	Coefficient of Variation of CL-ML Thickness:	25%
Unit Weight of Water:	62.4 pcf	Coefficient of Variation of Organics Thickness:	25%
Saturated Unit Weight of CL-ML:	110 pcf		
Saturated Unit Weight of Organics:	100 pcf		

	River Level	River Elevation (ft msl)	Height of Water on Wall (ft)	HGL Above Ground (feet)	HGL Elevation* (ft msl)	Water Pressure at Ground Surface (psf)	CL-ML Thickness (ft)	Organic Thickness (ft)	Total Blanket Thickness (ft)	Water Pressure at Bottom of Blanket (psf)	Change in Water Pressure Through Blanket (psf/ft)	Equivalent Effective Unit Weight CL-ML (pcf)	Equivalent Effective Unit Weight Organics (pcf)
Expected Value	Top of Wall	892	12	10	890	748.8	15	10	25	2184	87.36	22.6	12.6
[EV] - COV							11.25	7.50	18.75	1794	95.68	14.3	4.3
[EV] + COV							18.75	12.50	31.25	2574	82.37	27.6	17.6
Expected Value	TOW - 1	891	11	9.3	889.3	686.4	15	10	25	2140.32	85.61	24.4	14.4
[EV] - COV							11.25	7.50	18.75	1750.32	93.35	16.6	6.6
[EV] + COV							18.75	12.50	31.25	2530.32	80.97	29.0	19.0
Expected Value	TOW - 2	890	10	8.5	888.5	624	15	10	25	2090.4	83.62	26.4	16.4
[EV] - COV							11.25	7.50	18.75	1700.4	90.69	19.3	9.3
[EV] + COV							18.75	12.50	31.25	2480.4	79.37	30.6	20.6
Expected Value	TOW - 3	889	9	7.7	887.7	561.6	15	10	25	2040.48	81.62	28.4	18.4
[EV] - COV							11.25	7.50	18.75	1650.48	88.03	22.0	12.0
[EV] + COV							18.75	12.50	31.25	2430.48	77.78	32.2	22.2
Expected Value	TOW - 4	888	8	6.9	886.9	499.2	15	10	25	1990.56	79.62	30.4	20.4
[EV] - COV							11.25	7.50	18.75	1600.56	85.36	24.6	14.6
[EV] + COV							18.75	12.50	31.25	2380.56	76.18	33.8	23.8
Expected Value	TOW - 5	887	7	6	886	436.8	15	10	25	1934.4	77.38	32.6	22.6
[EV] - COV							11.25	7.50	18.75	1544.4	82.37	27.6	17.6
[EV] + COV							18.75	12.50	31.25	2324.4	74.38	35.6	25.6
Expected Value	TOW - 6	886	6	5.2	885.2	374.4	15	10	25	1884.48	75.38	34.6	24.6
[EV] - COV							11.25	7.50	18.75	1494.48	79.71	30.3	20.3
[EV] + COV							18.75	12.50	31.25	2274.48	72.78	37.2	27.2
Expected Value	TOW - 7	885	5	4.3	884.3	312	15	10	25	1828.32	73.13	36.9	26.9
[EV] - COV							11.25	7.50	18.75	1438.32	76.71	33.3	23.3
[EV] + COV							18.75	12.50	31.25	2218.32	70.99	39.0	29.0
Expected Value	TOW - 8	884	4	3.5	883.5	249.6	15	10	25	1778.4	71.14	38.9	28.9
[EV] - COV							11.25	7.50	18.75	1388.4	74.05	36.0	26.0
[EV] + COV							18.75	12.50	31.25	2168.4	69.39	40.6	30.6

*HGL computed using effect of the relief wells

Exhibit 10

Pile Axial Capacity Reliability Analysis
Excel Files to Determine Probability of Failure

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South Topeka Floodwall
Wall Data from Construction Drawings Dated March 1938 showing plan, profile and sections of all wall types
Landside Ground Surface Data From Record Drawings Dated November 1969
Compression load on landward most pile at different water levels
Station 80+42 to 83+78 (Wall station 6+72 to 10+08) Type B Wall Type

Distance to Water Surface from Top of Wall (ft)	Axial Load in Compression (kips)
0	23
1	20
2	17
3	14
4	12
5	10
6	9
7	8
8	6

South Topeka Axial Pile Capacity Reliability Analysis - November 2007
Reference: Corps of Engineers EM 1110-2-2906 dated 15 January 1991

Analysis: **Water to the top of the wall** Elevation: 892.00
Maximum Axial Load on Pile: **23 kips**

Top of Pile Below Ground Surface: 27 7.5 feet Bearing Capacity Factor in Sand (Nq): 40
Friction between soil and timber pile (δ): 0.9

Neglect skin friction on upper part of pile: 5 feet
Neglect skin friction on lower part of pile 1 feet
Earth Pressure Coefficients
CL-ML: 1.0
Organics: 1.0
Sand: 1.5
Expected Length of Pile 25 feet
Average Diameter of Pile
CL-ML: 0.95 feet (11.5 inches)
Organics: 0.875 feet (10.5 inches)
Sand: 0.8 feet (9.5 inches)
Pile Tip: 0.75 feet (9 inches)

	CL-ML Blanket Material Shaft Capacity							Organic Blanket Material Shaft Capacity					
Analysis	Coefficient of Variance (%)	Effective Unit Weight (pcf)	Thickness of Layer (ft)	Length of Pile in Layer (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Unit Weight (pcf)	Thickness of Layer (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)
Expected Value	N/A	22.6	15	7.5	24	310.75	7.46	**0.92**	12.6	10	17	402.00	27.49
Lower COV Strength CL-ML	30%	22.6	15	7.5	**16.8**	310.75	7.46	**0.63**	12.6	10	17	402.00	27.49
Upper COV Strength CL-ML	30%	22.6	15	7.5	**31.2**	310.75	7.46	**1.24**	12.6	10	17	402.00	27.49
Lower COV Strength Organics	30%	22.6	15	7.5	24	310.75	7.46	**0.92**	12.6	10	**11.9**	402.00	27.49
Upper COV Strength Organics	30%	22.6	15	7.5	24	310.75	7.46	**0.92**	12.6	10	**22.1**	402.00	27.49
Lower COV Strength Sand	15%	22.6	15	7.5	24	310.75	7.46	**0.92**	12.6	10	17	402.00	27.49
Upper COV Strength Sand	15%	22.6	15	7.5	24	310.75	7.46	**0.92**	12.6	10	17	402.00	27.49
Lower COV Blanket Thickness CL-ML	25%	22.6	**11.25**	3.75	24	0	0.00	**0.00**	12.6	10	17	325.13	24.05
Upper COV Blanket Thickness CL-ML	25%	22.6	**18.75**	11.25	24	353.125	18.65	**2.61**	12.6	10	17	486.75	27.49
Lower COV Blanket Thickness Organic	25%	22.6	15	7.5	24	310.75	7.46	**0.92**	12.6	**7.5**	17	386.25	20.62
Upper COV Blanket Thickness Organic	25%	22.6	15	7.5	24	310.75	7.46	**0.92**	12.6	**12.5**	17	417.75	34.36
Lower COV Pile Length	20%	22.6	15	7.5	24	310.75	7.46	**0.92**	12.6	10	17	402.00	27.49
Upper COV Pile Length	20%	22.6	15	7.5	24	310.75	7.46	**0.92**	12.6	10	17	402.00	27.49

	Aquifer Shaft Capacity						Aquifer Tip Capacity				Total Capacity			
Analysis	Ultimate Shaft Capacity (kips)	Effective Unit Weight (pcf)	Length of Pile in Sand (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Vertical Stress at Tip (psf)	Area of Pile Tip (psf)	Ultimate Tip Capacity (kips)	Total Ultimate Pile Capacity (kips)	Factor of Safety	Variance Component	Percent of Variance
Expected Value	**3.02**	52.6	7.5	33	635.95	16.34	**8.89**	859.5	0.44	**15.19**	**28.02**	1.22		
Lower COV Strength CL-ML	**3.02**	52.6	7.5	33	635.95	16.34	**8.89**	859.5	0.44	**15.19**	**27.73**	1.21		
Upper COV Strength CL-ML	**3.02**	52.6	7.5	33	635.95	16.34	**8.89**	859.5	0.44	**15.19**	**28.34**	1.23	0.000176	0.040237%
Lower COV Strength Organics	**2.09**	52.6	7.5	33	635.95	16.34	**8.89**	859.5	0.44	**15.19**	**27.09**	1.18		
Upper COV Strength Organics	**4.00**	52.6	7.5	33	635.95	16.34	**8.89**	859.5	0.44	**15.19**	**28.99**	1.26	0.001721	0.393036%
Lower COV Strength Sand	**3.02**	52.6	7.5	**28.05**	635.95	16.34	**7.35**	859.5	0.44	**15.19**	**26.48**	1.15		
Upper COV Strength Sand	**3.02**	52.6	7.5	**37.95**	635.95	16.34	**10.57**	859.5	0.44	**15.19**	**29.70**	1.29	0.004914	1.122611%
Lower COV Blanket Thickness CL-ML	**2.14**	52.6	11.25	33	649.825	25.76	**14.32**	972	0.44	**17.18**	**33.64**	1.46		
Upper COV Blanket Thickness CL-ML	**3.66**	52.6	3.75	33	622.075	6.91	**3.68**	747	0.44	**13.20**	**23.15**	1.01	0.052015	11.882365%
Lower COV Blanket Thickness Organic	**2.18**	52.6	10	33	670.2	22.62	**12.97**	959.5	0.44	**16.96**	**33.02**	1.44		
Upper COV Blanket Thickness Organic	**3.93**	52.6	5	33	601.7	10.05	**5.18**	759.5	0.44	**13.42**	**23.44**	1.02	0.043380	9.909628%
Lower COV Pile Length	**3.02**	52.6	**2.5**	33	504.45	3.77	**1.63**	596.5	0.44	**10.54**	**16.11**	0.70		
Upper COV Pile Length	**3.02**	52.6	**12.5**	33	767.45	28.90	**18.98**	1122.5	0.44	**19.84**	**42.76**	1.86	0.335546	76.652123%

SUM: 0.437752 100.000000%

Variance $Var[_{FS}]$ = 0.437752

Standard of Deviation σ_{FS} = 0.661628

Coefficient of Variation COV_{FS} = 0.543123

Reliability Index β_{FS} = 0.133981

Probability of Failure P_f = 44.67%

Notes:

1. The average pile diameters were not changed with the varying stratigraphy. This simplification is not exactly correct, however variations in pile diameter would be very minor.

South Topeka Axial Pile Capacity Reliability Analysis - November 2007
Reference: Corps of Engineers EM 1110-2-2906 dated 15 January 1991

Analysis: **Water 1 Foot below the top of the wall** Elevation: 891.00
Maximum Axial Load on Pile: **20 kips**

Top of Pile Below Ground Surface: 7.5 feet Bearing Capacity Factor in Sand (Nq): 40
Friction between soil and timber pile (δ): 0.9

Neglect skin friction on upper part of pile: 5 feet
Neglect skin friction on lower part of pile 1 feet
Earth Pressure Coefficients
CL-ML: 1.0
Organics: 1.0
Sand: 1.5
Expected Length of Pile 25 feet
Average Diameter of Pile
CL-ML: 0.95 feet (11.5 inches)
Organics: 0.875 feet (10.5 inches)
Sand: 0.8 feet (9.5 inches)
Pile Tip: 0.75 feet (9 inches)

	CL-ML Blanket Material Shaft Capacity							**Organic Blanket Material Shaft Capacity**						**Aquifer Shaft Capacity**						**Aquifer Tip Capacity**				**Total Capacity**			
Analysis	Coefficient of Variance (%)	Effective Unit Weight (pcf)	Thickness of Layer (ft)	Length of Pile in Layer (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Unit Weight (pcf)	Thickness of Layer (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Unit Weight (pcf)	Length of Pile in Sand (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Vertical Stress at Tip (psf)	Area of Pile Tip (psf)	Ultimate Tip Capacity (kips)	Total Ultimate Pile Capacity (kips)	Factor of Safety	Variance Component	Percent of Variance
Expected Value	N/A	24.4	15	7.5	24	335.5	7.46	**0.99**	14.4	10	17	438.00	27.49	**3.29**	52.6	7.5	33	680.95	16.34	**9.52**	904.5	0.44	**15.98**	**29.79**	1.49		
Lower COV Strength CL-ML	30%	24.4	15	7.5	**16.8**	335.5	7.46	**0.68**	14.4	10	17	438.00	27.49	**3.29**	52.6	7.5	33	680.95	16.34	**9.52**	904.5	0.44	**15.98**	**29.47**	1.47		
Upper COV Strength CL-ML	30%	24.4	15	7.5	**31.2**	335.5	7.46	**1.34**	14.4	10	17	438.00	27.49	**3.29**	52.6	7.5	33	680.95	16.34	**9.52**	904.5	0.44	**15.98**	**30.13**	1.51	0.000272	0.044287%
Lower COV Strength Organics	30%	24.4	15	7.5	24	335.5	7.46	**0.99**	14.4	10	**11.9**	438.00	27.49	**2.28**	52.6	7.5	33	680.95	16.34	**9.52**	904.5	0.44	**15.98**	**28.77**	1.44		
Upper COV Strength Organics	30%	24.4	15	7.5	24	335.5	7.46	**0.99**	14.4	10	**22.1**	438.00	27.49	**4.36**	52.6	7.5	33	680.95	16.34	**9.52**	904.5	0.44	**15.98**	**30.85**	1.54	0.002701	0.440567%
Lower COV Strength Sand	15%	24.4	15	7.5	24	335.5	7.46	**0.99**	14.4	10	17	438.00	27.49	**3.29**	52.6	7.5	**28.05**	680.95	16.34	**7.87**	904.5	0.44	**15.98**	**28.14**	1.41		
Upper COV Strength Sand	15%	24.4	15	7.5	24	335.5	7.46	**0.99**	14.4	10	17	438.00	27.49	**3.29**	52.6	7.5	**37.95**	680.95	16.34	**11.32**	904.5	0.44	**15.98**	**31.59**	1.58	0.007451	1.215339%
Lower COV Blanket Thickness CL-ML	25%	24.4	**11.25**	3.75	24	0	0.00	**0.00**	14.4	10	17	355.50	24.05	**2.34**	52.6	11.25	33	688.075	25.76	**15.17**	1010.25	0.44	**17.85**	**35.36**	1.77		
Upper COV Blanket Thickness CL-ML	25%	24.4	**18.75**	11.25	24	381.25	18.65	**2.82**	14.4	10	17	529.50	27.49	**3.98**	52.6	3.75	33	673.825	6.91	**3.98**	798.75	0.44	**14.12**	**24.90**	1.24	0.068386	11.153957%
Lower COV Blanket Thickness Organic	25%	24.4	15	7.5	24	335.5	7.46	**0.99**	14.4	**7.5**	17	420.00	20.62	**2.37**	52.6	10	33	710.7	22.62	**13.75**	1000	0.44	**17.67**	**34.79**	1.74		
Upper COV Blanket Thickness Organic	25%	24.4	15	7.5	24	335.5	7.46	**0.99**	14.4	**12.5**	17	456.00	34.36	**4.29**	52.6	5	33	651.2	10.05	**5.60**	809	0.44	**14.30**	**25.17**	1.26	0.057727	9.415365%
Lower COV Pile Length	20%	24.4	15	7.5	24	335.5	7.46	**0.99**	14.4	10	17	438.00	27.49	**3.29**	52.6	**2.5**	33	549.45	3.77	**1.77**	641.5	0.44	**11.34**	**17.39**	0.87		
Upper COV Pile Length	20%	24.4	15	7.5	24	335.5	7.46	**0.99**	14.4	10	17	438.00	27.49	**3.29**	52.6	**12.5**	33	812.45	28.90	**20.09**	1167.5	0.44	**20.63**	**45.01**	2.25	0.476575	77.730485%

Variance $Var_{[FS]}$ = 0.613112

Standard of Deviation σ_{FS} = 0.783015

SUM: 0.613112 100.000000%

Coefficient of Variation COV_{FS} = 0.525752

Reliability Index β_{FS} = 0.559276

Probability of Failure P_f = **28.80%**

Notes:

1. The average pile diameters were not changed with the varying stratigraphy. This simplification is not exactly correct, however variations in pile diameter would be very minor.

South Topeka Axial Pile Capacity Reliability Analysis - November 2007
Reference: Corps of Engineers EM 1110-2-2906 dated 15 January 1991

Analysis: **Water 2 Feet below the top of the wall** Elevation: 890.00
Maximum Axial Load on Pile: **17 kips**

Top of Pile Below Ground Surface: 7.5 feet
Friction between soil and timber pile (δ): 0.9

Bearing Capacity Factor in Sand (Nq): 40

Neglect skin friction on upper part of pile: 5 feet
Neglect skin friction on lower part of pile 1 feet
Earth Pressure Coefficients
CL-ML: 1.0
Organics: 1.0
Sand: 1.5
Expected Length of Pile 25 feet
Average Diameter of Pile
CL-ML: 0.95 feet (11.5 inches)
Organics: 0.875 feet (10.5 inches)
Sand: 0.8 feet (9.5 inches)
Pile Tip: 0.75 feet (9 inches)

	CL-ML Blanket Material Shaft Capacity							Organic Blanket Material Shaft Capacity						Aquifer Shaft Capacity						Aquifer Tip Capacity				Total Capacity			
Analysis	Coefficient of Variance (%)	Effective Unit Weight (pcf)	Thickness of Layer (ft)	Length of Pile in Layer (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Unit Weight (pcf)	Thickness of Layer (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Unit Weight (pcf)	Length of Pile in Sand (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Vertical Stress at Tip (psf)	Area of Pile Tip (psf)	Ultimate Tip Capacity (kips)	Total Ultimate Pile Capacity (kips)	Factor of Safety	Variance Component	Percent of Variance
Expected Value	N/A	26.4	15	7.5	24	363	7.46	**1.07**	16.4	10	17	478.00	27.49	**3.59**	52.6	7.5	33	730.95	16.34	**10.22**	954.5	0.44	**16.87**	**31.75**	1.87		
Lower COV Strength CL-ML	30%	26.4	15	7.5	**16.8**	363	7.46	**0.73**	16.4	10	17	478.00	27.49	**3.59**	52.6	7.5	33	730.95	16.34	**10.22**	954.5	0.44	**16.87**	**31.41**	1.85		
Upper COV Strength CL-ML	30%	26.4	15	7.5	**31.2**	363	7.46	**1.44**	16.4	10	17	478.00	27.49	**3.59**	52.6	7.5	33	730.95	16.34	**10.22**	954.5	0.44	**16.87**	**32.12**	1.89	0.000440	0.048708%
Lower COV Strength Organics	30%	26.4	15	7.5	24	363	7.46	**1.07**	16.4	10	**11.9**	478.00	27.49	**2.49**	52.6	7.5	33	730.95	16.34	**10.22**	954.5	0.44	**16.87**	**30.64**	1.80		
Upper COV Strength Organics	30%	26.4	15	7.5	24	363	7.46	**1.07**	16.4	10	**22.1**	478.00	27.49	**4.75**	52.6	7.5	33	730.95	16.34	**10.22**	954.5	0.44	**16.87**	**32.91**	1.94	0.004453	0.492966%
Lower COV Strength Sand	15%	26.4	15	7.5	24	363	7.46	**1.07**	16.4	10	17	478.00	27.49	**3.59**	52.6	7.5	**28.05**	730.95	16.34	**8.45**	954.5	0.44	**16.87**	**29.98**	1.76		
Upper COV Strength Sand	15%	26.4	15	7.5	24	363	7.46	**1.07**	16.4	10	17	478.00	27.49	**3.59**	52.6	7.5	**37.95**	730.95	16.34	**12.15**	954.5	0.44	**16.87**	**33.69**	1.98	0.011883	1.315648%
Lower COV Blanket Thickness CL-ML	25%	26.4	**11.25**	3.75	24	0	0.00	**0.00**	16.4	10	17	389.25	24.05	**2.56**	52.6	11.25	33	730.575	25.76	**16.10**	1052.75	0.44	**18.60**	**37.27**	2.19		
Upper COV Blanket Thickness CL-ML	25%	26.4	**18.75**	11.25	24	412.5	18.65	**3.05**	16.4	10	17	577.00	27.49	**4.34**	52.6	3.75	33	731.325	6.91	**4.32**	856.25	0.44	**15.13**	**26.84**	1.58	0.094033	10.410586%
Lower COV Blanket Thickness Organic	25%	26.4	15	7.5	24	363	7.46	**1.07**	16.4	**7.5**	17	457.50	20.62	**2.58**	52.6	10	33	755.7	22.62	**14.62**	1045	0.44	**18.47**	**36.74**	2.16		
Upper COV Blanket Thickness Organic	25%	26.4	15	7.5	24	363	7.46	**1.07**	16.4	**12.5**	17	498.50	34.36	**4.69**	52.6	5	33	706.2	10.05	**6.07**	864	0.44	**15.27**	**27.10**	1.59	0.080450	8.906763%
Lower COV Pile Length	20%	26.4	15	7.5	24	363	7.46	**1.07**	16.4	10	17	478.00	27.49	**3.59**	52.6	**2.5**	33	599.45	3.77	**1.93**	691.5	0.44	**12.22**	**18.82**	1.11		
Upper COV Pile Length	20%	26.4	15	7.5	24	363	7.46	**1.07**	16.4	10	17	478.00	27.49	**3.59**	52.6	**12.5**	33	862.45	28.90	**21.33**	1217.5	0.44	**21.52**	**47.51**	2.79	0.711984	78.825330%

Variance $Var_{[FS]}$ = 0.903243

Standard of Deviation σ_{FS} = 0.950391

SUM: 0.903243 100.000000%

Coefficient of Variation COV_{FS} = 0.508856

Reliability Index β_{FS} = 1.061925

Probability of Failure P_f = **14.41%**

Notes:

1. The average pile diameters were not changed with the varying stratigraphy. This simplification is not exactly correct, however variations in pile diameter would be very minor.

South Topeka Axial Pile Capacity Reliability Analysis - November 2007
Reference: Corps of Engineers EM 1110-2-2906 dated 15 January 1991

Analysis: **Water 3 Feet below the top of the wall** Elevation: 889.00
Maximum Axial Load on Pile: **14 kips**

Top of Pile Below Ground Surface: 7.5 feet Bearing Capacity Factor in Sand (Nq): 40
Friction between soil and timber pile (δ): 0.9

Neglect skin friction on upper part of pile: 5 feet
Neglect skin friction on lower part of pile 1 feet
Earth Pressure Coefficients
CL-ML: 1.0
Organics: 1.0
Sand: 1.5
Expected Length of Pile 25 feet
Average Diameter of Pile
CL-ML: 0.95 feet (11.5 inches)
Organics: 0.875 feet (10.5 inches)
Sand: 0.8 feet (9.5 inches)
Pile Tip: 0.75 feet (9 inches)

	CL-ML Blanket Material Shaft Capacity							**Organic Blanket Material Shaft Capacity**						**Aquifer Shaft Capacity**						**Aquifer Tip Capacity**				**Total Capacity**			
Analysis	Coefficient of Variance (%)	Effective Unit Weight (pcf)	Thickness of Layer (ft)	Length of Pile in Layer (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Unit Weight (pcf)	Thickness of Layer (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Unit Weight (pcf)	Length of Pile in Sand (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Vertical Stress at Tip (psf)	Area of Pile Tip (psf)	Ultimate Tip Capacity (kips)	Total Ultimate Pile Capacity (kips)	Factor of Safety	Variance Component	Percent of Variance
Expected Value	N/A	28.4	15	7.5	24	390.5	7.46	**1.15**	18.4	10	17	518.00	27.49	**3.90**	52.6	7.5	33	780.95	16.34	**10.92**	1004.5	0.44	**17.75**	**33.72**	2.41		
Lower COV Strength CL-ML	30%	28.4	15	7.5	**16.8**	390.5	7.46	**0.79**	18.4	10	17	518.00	27.49	**3.90**	52.6	7.5	33	780.95	16.34	**10.92**	1004.5	0.44	**17.75**	**33.35**	2.38		
Upper COV Strength CL-ML	30%	28.4	15	7.5	**31.2**	390.5	7.46	**1.55**	18.4	10	17	518.00	27.49	**3.90**	52.6	7.5	33	780.95	16.34	**10.92**	1004.5	0.44	**17.75**	**34.12**	2.44	0.000751	0.053031%
Lower COV Strength Organics	30%	28.4	15	7.5	24	390.5	7.46	**1.15**	18.4	10	**11.9**	518.00	27.49	**2.69**	52.6	7.5	33	780.95	16.34	**10.92**	1004.5	0.44	**17.75**	**32.51**	2.32		
Upper COV Strength Organics	30%	28.4	15	7.5	24	390.5	7.46	**1.15**	18.4	10	**22.1**	518.00	27.49	**5.15**	52.6	7.5	33	780.95	16.34	**10.92**	1004.5	0.44	**17.75**	**34.97**	2.50	0.007710	0.544655%
Lower COV Strength Sand	15%	28.4	15	7.5	24	390.5	7.46	**1.15**	18.4	10	17	518.00	27.49	**3.90**	52.6	7.5	**28.05**	780.95	16.34	**9.02**	1004.5	0.44	**17.75**	**31.82**	2.27		
Upper COV Strength Sand	15%	28.4	15	7.5	24	390.5	7.46	**1.15**	18.4	10	17	518.00	27.49	**3.90**	52.6	7.5	**37.95**	780.95	16.34	**12.98**	1004.5	0.44	**17.75**	**35.78**	2.56	0.020001	1.412902%
Lower COV Blanket Thickness CL-ML	25%	28.4	**11.25**	3.75	24	0	0.00	**0.00**	18.4	10	17	423.00	24.05	**2.78**	52.6	11.25	33	773.075	25.76	**17.04**	1095.25	0.44	**19.35**	**39.18**	2.80		
Upper COV Blanket Thickness CL-ML	25%	28.4	**18.75**	11.25	24	443.75	18.65	**3.28**	18.4	10	17	624.50	27.49	**4.70**	52.6	3.75	33	788.825	6.91	**4.66**	913.75	0.44	**16.15**	**28.79**	2.06	0.137740	9.730073%
Lower COV Blanket Thickness Organic	25%	28.4	15	7.5	24	390.5	7.46	**1.15**	18.4	**7.5**	17	495.00	20.62	**2.79**	52.6	10	33	800.7	22.62	**15.50**	1090	0.44	**19.26**	**38.70**	2.76		
Upper COV Blanket Thickness Organic	25%	28.4	15	7.5	24	390.5	7.46	**1.15**	18.4	**12.5**	17	541.00	34.36	**5.09**	52.6	5	33	761.2	10.05	**6.55**	919	0.44	**16.24**	**29.03**	2.07	0.119438	8.437165%
Lower COV Pile Length	20%	28.4	15	7.5	24	390.5	7.46	**1.15**	18.4	10	17	518.00	27.49	**3.90**	52.6	**2.5**	33	649.45	3.77	**2.09**	741.5	0.44	**13.10**	**20.25**	1.45		
Upper COV Pile Length	20%	28.4	15	7.5	24	390.5	7.46	**1.15**	18.4	10	17	518.00	27.49	**3.90**	52.6	**12.5**	33	912.45	28.90	**22.56**	1267.5	0.44	**22.40**	**50.01**	3.57	1.129975	79.822174%

Variance $Var_{[FS]}$ = 1.415615

Standard of Deviation σ_{FS} = 1.189796

SUM: 1.415615 100.000000%

Coefficient of Variation COV_{FS} = 0.494052

Reliability Index β_{FS} = 1.646990

Probability of Failure P_f = **4.98%**

Notes:

1. The average pile diameters were not changed with the varying stratigraphy. This simplification is not exactly correct, however variations in pile diameter would be very minor.

South Topeka Axial Pile Capacity Reliability Analysis - November 2007
Reference: Corps of Engineers EM 1110-2-2906 dated 15 January 1991

Analysis: **Water 4 Feet below the top of the wall** Elevation: 888.00
Maximum Axial Load on Pile: **12 kips**

Top of Pile Below Ground Surface: 7.5 feet
Bearing Capacity Factor in Sand (Nq): 40
Friction between soil and timber pile (δ): 0.9

Neglect skin friction on upper part of pile: 5 feet
Neglect skin friction on lower part of pile 1 feet
Earth Pressure Coefficients
CL-ML: 1.0
Organics: 1.0
Sand: 1.5
Expected Length of Pile 25 feet
Average Diameter of Pile
CL-ML: 0.95 feet (11.5 inches)
Organics: 0.875 feet (10.5 inches)
Sand: 0.8 feet (9.5 inches)
Pile Tip: 0.75 feet (9 inches)

	CL-ML Blanket Material Shaft Capacity							Organic Blanket Material Shaft Capacity						Aquifer Shaft Capacity						Aquifer Tip Capacity				Total Capacity			
Analysis	Coefficient of Variance (%)	Effective Unit Weight (pcf)	Thickness of Layer (ft)	Length of Pile in Layer (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Unit Weight (pcf)	Thickness of Layer (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Unit Weight (pcf)	Length of Pile in Sand (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Vertical Stress at Tip (psf)	Area of Pile Tip (psf)	Ultimate Tip Capacity (kips)	Total Ultimate Pile Capacity (kips)	Factor of Safety	Variance Component	Percent of Variance
Expected Value	N/A	30.4	15	7.5	24	418	7.46	**1.23**	20.4	10	17	558.00	27.49	**4.20**	52.6	7.5	33	830.95	16.34	**11.61**	1054.5	0.44	**18.63**	**35.68**	2.97		
Lower COV Strength CL-ML	30%	30.4	15	7.5	**16.8**	418	7.46	**0.84**	20.4	10	17	558.00	27.49	**4.20**	52.6	7.5	33	830.95	16.34	**11.61**	1054.5	0.44	**18.63**	**35.29**	2.94		
Upper COV Strength CL-ML	30%	30.4	15	7.5	**31.2**	418	7.46	**1.66**	20.4	10	17	558.00	27.49	**4.20**	52.6	7.5	33	830.95	16.34	**11.61**	1054.5	0.44	**18.63**	**36.11**	3.01	0.001171	0.057245%
Lower COV Strength Organics	30%	30.4	15	7.5	24	418	7.46	**1.23**	20.4	10	**11.9**	558.00	27.49	**2.90**	52.6	7.5	33	830.95	16.34	**11.61**	1054.5	0.44	**18.63**	**34.38**	2.87		
Upper COV Strength Organics	30%	30.4	15	7.5	24	418	7.46	**1.23**	20.4	10	**22.1**	558.00	27.49	**5.55**	52.6	7.5	33	830.95	16.34	**11.61**	1054.5	0.44	**18.63**	**37.03**	3.09	0.012178	0.595422%
Lower COV Strength Sand	15%	30.4	15	7.5	24	418	7.46	**1.23**	20.4	10	17	558.00	27.49	**4.20**	52.6	7.5	**28.05**	830.95	16.34	**9.60**	1054.5	0.44	**18.63**	**33.67**	2.81		
Upper COV Strength Sand	15%	30.4	15	7.5	24	418	7.46	**1.23**	20.4	10	17	558.00	27.49	**4.20**	52.6	7.5	**37.95**	830.95	16.34	**13.81**	1054.5	0.44	**18.63**	**37.88**	3.16	0.030822	1.506988%
Lower COV Blanket Thickness CL-ML	25%	30.4	**11.25**	3.75	24	0	0.00	**0.00**	20.4	10	17	456.75	24.05	**3.01**	52.6	11.25	33	815.575	25.76	**17.98**	1137.75	0.44	**20.11**	**41.09**	3.42		
Upper COV Blanket Thickness CL-ML	25%	30.4	**18.75**	11.25	24	475	18.65	**3.51**	20.4	10	17	672.00	27.49	**5.05**	52.6	3.75	33	846.325	6.91	**5.00**	971.25	0.44	**17.16**	**30.73**	2.56	0.186245	9.106275%
Lower COV Blanket Thickness Organic	25%	30.4	15	7.5	24	418	7.46	**1.23**	20.4	**7.5**	17	532.50	20.62	**3.00**	52.6	10	33	845.7	22.62	**16.37**	1135	0.44	**20.06**	**40.66**	3.39		
Upper COV Blanket Thickness Organic	25%	30.4	15	7.5	24	418	7.46	**1.23**	20.4	**12.5**	17	583.50	34.36	**5.48**	52.6	5	33	816.2	10.05	**7.02**	974	0.44	**17.21**	**30.95**	2.58	0.163682	8.003062%
Lower COV Pile Length	20%	30.4	15	7.5	24	418	7.46	**1.23**	20.4	10	17	558.00	27.49	**4.20**	52.6	**2.5**	33	699.45	3.77	**2.26**	791.5	0.44	**13.99**	**21.67**	1.81		
Upper COV Pile Length	20%	30.4	15	7.5	24	418	7.46	**1.23**	20.4	10	17	558.00	27.49	**4.20**	52.6	**12.5**	33	962.45	28.90	**23.80**	1317.5	0.44	**23.28**	**52.51**	4.38	1.651143	80.731008%

SUM: 2.045241 100.000000%

Variance $Var_{[FS]}$ = 2.045241

Standard of Deviation σ_{FS} = 1.430119

Coefficient of Variation COV_{FS} = 0.480984

Reliability Index β_{FS} = 2.160571

Probability of Failure P_f = **1.54%**

Notes:

1. The average pile diameters were not changed with the varying stratigraphy. This simplification is not exactly correct, however variations in pile diameter would be very minor.

South Topeka Axial Pile Capacity Reliability Analysis - November 2007
Reference: Corps of Engineers EM 1110-2-2906 dated 15 January 1991

Analysis: **Water 5 Feet below the top of the wall** Elevation: 887.00
Maximum Axial Load on Pile: **10 kips**

Top of Pile Below Ground Surface: 7.5 feet Bearing Capacity Factor in Sand (Nq): 40
Friction between soil and timber pile (δ): 0.9

Neglect skin friction on upper part of pile: 5 feet
Neglect skin friction on lower part of pile 1 feet
Earth Pressure Coefficients
CL-ML: 1.0
Organics: 1.0
Sand: 1.5
Expected Length of Pile 25 feet
Average Diameter of Pile
CL-ML: 0.95 feet (11.5 inches)
Organics: 0.875 feet (10.5 inches)
Sand: 0.8 feet (9.5 inches)
Pile Tip: 0.75 feet (9 inches)

	CL-ML Blanket Material Shaft Capacity							Organic Blanket Material Shaft Capacity						Aquifer Shaft Capacity						Aquifer Tip Capacity				Total Capacity			
Analysis	Coefficient of Variance (%)	Effective Unit Weight (pcf)	Thickness of Layer (ft)	Length of Pile in Layer (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Unit Weight (pcf)	Thickness of Layer (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Unit Weight (pcf)	Length of Pile in Sand (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Vertical Stress at Tip (psf)	Area of Pile Tip (psf)	Ultimate Tip Capacity (kips)	Total Ultimate Pile Capacity (kips)	Factor of Safety	Variance Component	Percent of Variance
Expected Value	N/A	32.6	15	7.5	24	448.25	7.46	**1.32**	22.6	10	17	602.00	27.49	**4.53**	52.6	7.5	33	885.95	16.34	**12.38**	1109.5	0.44	**19.61**	**37.84**	3.78		
Lower COV Strength CL-ML	30%	32.6	15	7.5	**16.8**	448.25	7.46	**0.90**	22.6	10	17	602.00	27.49	**4.53**	52.6	7.5	33	885.95	16.34	**12.38**	1109.5	0.44	**19.61**	**37.42**	3.74		
Upper COV Strength CL-ML	30%	32.6	15	7.5	**31.2**	448.25	7.46	**1.78**	22.6	10	17	602.00	27.49	**4.53**	52.6	7.5	33	885.95	16.34	**12.38**	1109.5	0.44	**19.61**	**38.30**	3.83	0.001939	0.061744%
Lower COV Strength Organics	30%	32.6	15	7.5	24	448.25	7.46	**1.32**	22.6	10	**11.9**	602.00	27.49	**3.13**	52.6	7.5	33	885.95	16.34	**12.38**	1109.5	0.44	**19.61**	**36.44**	3.64		
Upper COV Strength Organics	30%	32.6	15	7.5	24	448.25	7.46	**1.32**	22.6	10	**22.1**	602.00	27.49	**5.99**	52.6	7.5	33	885.95	16.34	**12.38**	1109.5	0.44	**19.61**	**39.30**	3.93	0.020411	0.650015%
Lower COV Strength Sand	15%	32.6	15	7.5	24	448.25	7.46	**1.32**	22.6	10	17	602.00	27.49	**4.53**	52.6	7.5	**28.05**	885.95	16.34	**10.24**	1109.5	0.44	**19.61**	**35.69**	3.57		
Upper COV Strength Sand	15%	32.6	15	7.5	24	448.25	7.46	**1.32**	22.6	10	17	602.00	27.49	**4.53**	52.6	7.5	**37.95**	885.95	16.34	**14.73**	1109.5	0.44	**19.61**	**40.19**	4.02	0.050453	1.606763%
Lower COV Blanket Thickness CL-ML	25%	32.6	**11.25**	3.75	24	0	0.00	**0.00**	22.6	10	17	493.88	24.05	**3.25**	52.6	11.25	33	862.325	25.76	**19.01**	1184.5	0.44	**20.93**	**43.19**	4.32		
Upper COV Blanket Thickness CL-ML	25%	32.6	**18.75**	11.25	24	509.375	18.65	**3.76**	22.6	10	17	724.25	27.49	**5.45**	52.6	3.75	33	909.575	6.91	**5.38**	1034.5	0.44	**18.28**	**32.87**	3.29	0.266244	8.479041%
Lower COV Blanket Thickness Organic	25%	32.6	15	7.5	24	448.25	7.46	**1.32**	22.6	**7.5**	17	573.75	20.62	**3.24**	52.6	10	33	895.2	22.62	**17.32**	1184.5	0.44	**20.93**	**42.82**	4.28		
Upper COV Blanket Thickness Organic	25%	32.6	15	7.5	24	448.25	7.46	**1.32**	22.6	**12.5**	17	630.25	34.36	**5.92**	52.6	5	33	876.7	10.05	**7.54**	1034.5	0.44	**18.28**	**33.07**	3.31	0.237472	7.562753%
Lower COV Pile Length	20%	32.6	15	7.5	24	448.25	7.46	**1.32**	22.6	10	17	602.00	27.49	**4.53**	52.6	**2.5**	33	754.45	3.77	**2.43**	846.5	0.44	**14.96**	**23.24**	2.32		
Upper COV Pile Length	20%	32.6	15	7.5	24	448.25	7.46	**1.32**	22.6	10	17	602.00	27.49	**4.53**	52.6	**12.5**	33	1017.45	28.90	**25.16**	1372.5	0.44	**24.25**	**55.27**	5.53	2.563507	81.639683%

SUM: 3.140026 100.000000%

Variance $Var[_{FS}]$ = 3.140026

Standard of Deviation σ_{FS} = 1.772012

Coefficient of Variation COV_{FS} = 0.468281

Reliability Index β_{FS} = 2.766110

Probability of Failure P_f = **0.28%**

Notes:

1. The average pile diameters were not changed with the varying stratigraphy. This simplification is not exactly correct, however variations in pile diameter would be very minor.

South Topeka Axial Pile Capacity Reliability Analysis - November 2007
Reference: Corps of Engineers EM 1110-2-2906 dated 15 January 1991

Analysis: **Water 6 Feet below the top of the wall** Elevation: 886.00
Maximum Axial Load on Pile: **9 kips**

Top of Pile Below Ground Surface: 7.5 feet
Friction between soil and timber pile (δ): 0.9

Bearing Capacity Factor in Sand (Nq): 40

Neglect skin friction on upper part of pile: 5 feet
Neglect skin friction on lower part of pile 1 feet
Earth Pressure Coefficients
CL-ML: 1.0
Organics: 1.0
Sand: 1.5
Expected Length of Pile 25 feet
Average Diameter of Pile
CL-ML: 0.95 feet (11.5 inches)
Organics: 0.875 feet (10.5 inches)
Sand: 0.8 feet (9.5 inches)
Pile Tip: 0.75 feet (9 inches)

	CL-ML Blanket Material Shaft Capacity								Organic Blanket Material Shaft Capacity						Aquifer Shaft Capacity						Aquifer Tip Capacity			Total Capacity			
Analysis	Coefficient of Variance (%)	Effective Unit Weight (pcf)	Thickness of Layer (ft)	Length of Pile in Layer (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Unit Weight (pcf)	Thickness of Layer (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Unit Weight (pcf)	Length of Pile in Sand (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Vertical Stress at Tip (psf)	Area of Pile Tip (psf)	Ultimate Tip Capacity (kips)	Total Ultimate Pile Capacity (kips)	Factor of Safety	Variance Component	Percent of Variance
Expected Value	N/A	34.6	15	7.5	24	475.75	7.46	**1.41**	24.6	10	17	642.00	27.49	**4.83**	52.6	7.5	33	935.95	16.34	**13.08**	1159.5	0.44	**20.49**	**39.81**	4.42		
Lower COV Strength CL-ML	30%	34.6	15	7.5	**16.8**	475.75	7.46	**0.96**	24.6	10	17	642.00	27.49	**4.83**	52.6	7.5	33	935.95	16.34	**13.08**	1159.5	0.44	**20.49**	**39.36**	4.37		
Upper COV Strength CL-ML	30%	34.6	15	7.5	**31.2**	475.75	7.46	**1.89**	24.6	10	17	642.00	27.49	**4.83**	52.6	7.5	33	935.95	16.34	**13.08**	1159.5	0.44	**20.49**	**40.29**	4.48	0.002696	0.065707%
Lower COV Strength Organics	30%	34.6	15	7.5	24	475.75	7.46	**1.41**	24.6	10	**11.9**	642.00	27.49	**3.34**	52.6	7.5	33	935.95	16.34	**13.08**	1159.5	0.44	**20.49**	**38.32**	4.26		
Upper COV Strength Organics	30%	34.6	15	7.5	24	475.75	7.46	**1.41**	24.6	10	**22.1**	642.00	27.49	**6.38**	52.6	7.5	33	935.95	16.34	**13.08**	1159.5	0.44	**20.49**	**41.36**	4.60	0.028658	0.698389%
Lower COV Strength Sand	15%	34.6	15	7.5	24	475.75	7.46	**1.41**	24.6	10	17	642.00	27.49	**4.83**	52.6	7.5	**28.05**	935.95	16.34	**10.81**	1159.5	0.44	**20.49**	**37.54**	4.17		
Upper COV Strength Sand	15%	34.6	15	7.5	24	475.75	7.46	**1.41**	24.6	10	17	642.00	27.49	**4.83**	52.6	7.5	**37.95**	935.95	16.34	**15.56**	1159.5	0.44	**20.49**	**42.28**	4.70	0.069516	1.694087%
Lower COV Blanket Thickness CL-ML	25%	34.6	**11.25**	3.75	24	0	0.00	**0.00**	24.6	10	17	527.63	24.05	**3.47**	52.6	11.25	33	904.825	25.76	**19.94**	1227	0.44	**21.68**	**45.10**	5.01		
Upper COV Blanket Thickness CL-ML	25%	34.6	**18.75**	11.25	24	540.625	18.65	**3.99**	24.6	10	17	771.75	27.49	**5.80**	52.6	3.75	33	967.075	6.91	**5.72**	1092	0.44	**19.30**	**34.81**	3.87	0.326516	7.957080%
Lower COV Blanket Thickness Organic	25%	34.6	15	7.5	24	475.75	7.46	**1.41**	24.6	**7.5**	17	611.25	20.62	**3.45**	52.6	10	33	940.2	22.62	**18.20**	1229.5	0.44	**21.73**	**44.78**	4.98		
Upper COV Blanket Thickness Organic	25%	34.6	15	7.5	24	475.75	7.46	**1.41**	24.6	**12.5**	17	672.75	34.36	**6.32**	52.6	5	33	931.7	10.05	**8.01**	1089.5	0.44	**19.25**	**35.00**	3.89	0.295170	7.193179%
Lower COV Pile Length	20%	34.6	15	7.5	24	475.75	7.46	**1.41**	24.6	10	17	642.00	27.49	**4.83**	52.6	**2.5**	33	804.45	3.77	**2.59**	896.5	0.44	**15.84**	**24.67**	2.74		
Upper COV Pile Length	20%	34.6	15	7.5	24	475.75	7.46	**1.41**	24.6	10	17	642.00	27.49	**4.83**	52.6	**12.5**	33	1067.45	28.90	**26.40**	1422.5	0.44	**25.14**	**57.77**	6.42	3.380914	82.391559%

SUM: 4.103472 100.000000%

Variance $Var_{[FS]}$ = 4.103472

Standard of Deviation σ_{FS} = 2.025703

Coefficient of Variation COV_{FS} = 0.458013

Reliability Index β_{FS} = 3.188800

Probability of Failure P_f = **0.07%**

Notes:

1. The average pile diameters were not changed with the varying stratigraphy. This simplification is not exactly correct, however variations in pile diameter would be very minor.

South Topeka Axial Pile Capacity Reliability Analysis - November 2007
Reference: Corps of Engineers EM 1110-2-2906 dated 15 January 1991

Analysis: **Water 7 Feet below the top of the wall** Elevation: 885.00
Maximum Axial Load on Pile: **8 kips**

Top of Pile Below Ground Surface: 7.5 feet — Bearing Capacity Factor in Sand (Nq): 40
Friction between soil and timber pile (δ): 0.9

Neglect skin friction on upper part of pile: 5 feet
Neglect skin friction on lower part of pile 1 feet
Earth Pressure Coefficients
CL-ML: 1.0
Organics: 1.0
Sand: 1.5
Expected Length of Pile 25 feet
Average Diameter of Pile
CL-ML: 0.95 feet (11.5 inches)
Organics: 0.875 feet (10.5 inches)
Sand: 0.8 feet (9.5 inches)
Pile Tip: 0.75 feet (9 inches)

	CL-ML Blanket Material Shaft Capacity								Organic Blanket Material Shaft Capacity						Aquifer Shaft Capacity						Aquifer Tip Capacity			Total Capacity			
Analysis	Coefficient of Variance (%)	Effective Unit Weight (pcf)	Thickness of Layer (ft)	Length of Pile in Layer (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Unit Weight (pcf)	Thickness of Layer (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Unit Weight (pcf)	Length of Pile in Sand (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Vertical Stress at Tip (psf)	Area of Pile Tip (psf)	Ultimate Tip Capacity (kips)	Total Ultimate Pile Capacity (kips)	Factor of Safety	Variance Component	Percent of Variance
Expected Value	N/A	36.9	15	7.5	24	507.375	7.46	**1.50**	26.9	10	17	688.00	27.49	**5.17**	52.6	7.5	33	993.45	16.34	**13.89**	1217	0.44	**21.51**	**42.06**	5.26		
Lower COV Strength CL-ML	30%	36.9	15	7.5	**16.8**	507.375	7.46	**1.02**	26.9	10	17	688.00	27.49	**5.17**	52.6	7.5	33	993.45	16.34	**13.89**	1217	0.44	**21.51**	**41.59**	5.20		
Upper COV Strength CL-ML	30%	36.9	15	7.5	**31.2**	507.375	7.46	**2.02**	26.9	10	17	688.00	27.49	**5.17**	52.6	7.5	33	993.45	16.34	**13.89**	1217	0.44	**21.51**	**42.59**	5.32	0.003881	0.070110%
Lower COV Strength Organics	30%	36.9	15	7.5	24	507.375	7.46	**1.50**	26.9	10	**11.9**	688.00	27.49	**3.58**	52.6	7.5	33	993.45	16.34	**13.89**	1217	0.44	**21.51**	**40.47**	5.06		
Upper COV Strength Organics	30%	36.9	15	7.5	24	507.375	7.46	**1.50**	26.9	10	**22.1**	688.00	27.49	**6.84**	52.6	7.5	33	993.45	16.34	**13.89**	1217	0.44	**21.51**	**43.73**	5.47	0.041654	0.752443%
Lower COV Strength Sand	15%	36.9	15	7.5	24	507.375	7.46	**1.50**	26.9	10	17	688.00	27.49	**5.17**	52.6	7.5	**28.05**	993.45	16.34	**11.48**	1217	0.44	**21.51**	**39.66**	4.96		
Upper COV Strength Sand	15%	36.9	15	7.5	24	507.375	7.46	**1.50**	26.9	10	17	688.00	27.49	**5.17**	52.6	7.5	**37.95**	993.45	16.34	**16.52**	1217	0.44	**21.51**	**44.70**	5.59	0.099124	1.790573%
Lower COV Blanket Thickness CL-ML	25%	36.9	**11.25**	3.75	24	0	0.00	**0.00**	26.9	10	17	566.44	24.05	**3.73**	52.6	11.25	33	953.7	25.76	**21.02**	1275.875	0.44	**22.55**	**47.29**	5.91		
Upper COV Blanket Thickness CL-ML	25%	36.9	**18.75**	11.25	24	576.5625	18.65	**4.26**	26.9	10	17	826.38	27.49	**6.21**	52.6	3.75	33	1033.2	6.91	**6.11**	1158.125	0.44	**20.47**	**37.05**	4.63	0.410086	7.407783%
Lower COV Blanket Thickness Organic	25%	36.9	15	7.5	24	507.375	7.46	**1.50**	26.9	**7.5**	17	654.38	20.62	**3.69**	52.6	10	33	991.95	22.62	**19.20**	1281.25	0.44	**22.64**	**47.03**	5.88		
Upper COV Blanket Thickness Organic	25%	36.9	15	7.5	24	507.375	7.46	**1.50**	26.9	**12.5**	17	721.63	34.36	**6.78**	52.6	5	33	994.95	10.05	**8.56**	1152.75	0.44	**20.37**	**37.21**	4.65	0.376488	6.800864%
Lower COV Pile Length	20%	36.9	15	7.5	24	507.375	7.46	**1.50**	26.9	10	17	688.00	27.49	**5.17**	52.6	**2.5**	33	861.95	3.77	**2.78**	954	0.44	**16.86**	**26.31**	3.29		
Upper COV Pile Length	20%	36.9	15	7.5	24	507.375	7.46	**1.50**	26.9	10	17	688.00	27.49	**5.17**	52.6	**12.5**	33	1124.95	28.90	**27.82**	1480	0.44	**26.15**	**60.64**	7.58	4.604647	83.178225%

Variance $Var_{[FS]}$ = 5.535880

Standard of Deviation σ_{FS} = 2.352845

SUM: 5.535880 100.000000%

Coefficient of Variation COV_{FS} = 0.447475

Reliability Index β_{FS} = 3.671420

Probability of Failure P_f = **0.01%**

Notes:

1. The average pile diameters were not changed with the varying stratigraphy. This simplification is not exactly correct, however variations in pile diameter would be very minor.

South Topeka Axial Pile Capacity Reliability Analysis - November 2007
Reference: Corps of Engineers EM 1110-2-2906 dated 15 January 1991

Analysis: **Water 8 Feet below the top of the wall** Elevation: 884.00
Maximum Axial Load on Pile: **6 kips**

Top of Pile Below Ground Surface: 7.5 feet Bearing Capacity Factor in Sand (Nq): 40
Friction between soil and timber pile (δ): 0.9

Neglect skin friction on upper part of pile: 5 feet
Neglect skin friction on lower part of pile 1 feet
Earth Pressure Coefficients
CL-ML: 1.0
Organics: 1.0
Sand: 1.5
Expected Length of Pile 25 feet
Average Diameter of Pile
CL-ML: 0.95 feet (11.5 inches)
Organics: 0.875 feet (10.5 inches)
Sand: 0.8 feet (9.5 inches)
Pile Tip: 0.75 feet (9 inches)

	CL-ML Blanket Material Shaft Capacity							**Organic Blanket Material Shaft Capacity**						**Aquifer Shaft Capacity**						**Aquifer Tip Capacity**				**Total Capacity**			
Analysis	Coefficient of Variance (%)	Effective Unit Weight (pcf)	Thickness of Layer (ft)	Length of Pile in Layer (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Unit Weight (pcf)	Thickness of Layer (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Unit Weight (pcf)	Length of Pile in Sand (ft)	ϕ' (deg)	Average Effective Vertical Stress (psf)	Pile Surface Area (sq ft)	Ultimate Shaft Capacity (kips)	Effective Vertical Stress at Tip (psf)	Area of Pile Tip (psf)	Ultimate Tip Capacity (kips)	Total Ultimate Pile Capacity (kips)	Factor of Safety	Variance Component	Percent of Variance
Expected Value	N/A	38.9	15	7.5	24	534.875	7.46	**1.58**	28.9	10	17	728.00	27.49	**5.47**	52.6	7.5	33	1043.45	16.34	**14.58**	1267	0.44	**22.39**	**44.03**	7.34		
Lower COV Strength CL-ML	30%	38.9	15	7.5	**16.8**	534.875	7.46	**1.08**	28.9	10	17	728.00	27.49	**5.47**	52.6	7.5	33	1043.45	16.34	**14.58**	1267	0.44	**22.39**	**43.53**	7.25		
Upper COV Strength CL-ML	30%	38.9	15	7.5	**31.2**	534.875	7.46	**2.13**	28.9	10	17	728.00	27.49	**5.47**	52.6	7.5	33	1043.45	16.34	**14.58**	1267	0.44	**22.39**	**44.58**	7.43	0.007668	0.073805%
Lower COV Strength Organics	30%	38.9	15	7.5	24	534.875	7.46	**1.58**	28.9	10	**11.9**	728.00	27.49	**3.78**	52.6	7.5	33	1043.45	16.34	**14.58**	1267	0.44	**22.39**	**42.34**	7.06		
Upper COV Strength Organics	30%	38.9	15	7.5	24	534.875	7.46	**1.58**	28.9	10	**22.1**	728.00	27.49	**7.24**	52.6	7.5	33	1043.45	16.34	**14.58**	1267	0.44	**22.39**	**45.79**	7.63	0.082913	0.798029%
Lower COV Strength Sand	15%	38.9	15	7.5	24	534.875	7.46	**1.58**	28.9	10	17	728.00	27.49	**5.47**	52.6	7.5	**28.05**	1043.45	16.34	**12.06**	1267	0.44	**22.39**	**41.50**	6.92		
Upper COV Strength Sand	15%	38.9	15	7.5	24	534.875	7.46	**1.58**	28.9	10	17	728.00	27.49	**5.47**	52.6	7.5	**37.95**	1043.45	16.34	**17.35**	1267	0.44	**22.39**	**46.79**	7.80	0.194405	1.871123%
Lower COV Blanket Thickness CL-ML	25%	38.9	**11.25**	3.75	24	0	0.00	**0.00**	28.9	10	17	600.19	24.05	**3.95**	52.6	11.25	33	996.2	25.76	**21.96**	1318.375	0.44	**23.30**	**49.20**	8.20		
Upper COV Blanket Thickness CL-ML	25%	38.9	**18.75**	11.25	24	607.8125	18.65	**4.49**	28.9	10	17	873.88	27.49	**6.57**	52.6	3.75	33	1090.7	6.91	**6.45**	1215.625	0.44	**21.48**	**38.99**	6.50	0.724172	6.970063%
Lower COV Blanket Thickness Organic	25%	38.9	15	7.5	24	534.875	7.46	**1.58**	28.9	**7.5**	17	691.88	20.62	**3.90**	52.6	10	33	1036.95	22.62	**20.07**	1326.25	0.44	**23.44**	**48.99**	8.16		
Upper COV Blanket Thickness Organic	25%	38.9	15	7.5	24	534.875	7.46	**1.58**	28.9	**12.5**	17	764.13	34.36	**7.18**	52.6	5	33	1049.95	10.05	**9.03**	1207.75	0.44	**21.34**	**39.14**	6.52	0.673831	6.485537%
Lower COV Pile Length	20%	38.9	15	7.5	24	534.875	7.46	**1.58**	28.9	10	17	728.00	27.49	**5.47**	52.6	**2.5**	33	911.95	3.77	**2.94**	1004	0.44	**17.74**	**27.74**	4.62		
Upper COV Pile Length	20%	38.9	15	7.5	24	534.875	7.46	**1.58**	28.9	10	17	728.00	27.49	**5.47**	52.6	**12.5**	33	1174.95	28.90	**29.05**	1530	0.44	**27.04**	**63.15**	10.52	8.706761	83.801442%

SUM: 10.389751 100.000000%

Variance $Var_{[FS]}$ = 10.389751

Standard of Deviation σ_{FS} = 3.223314

Coefficient of Variation COV_{FS} = 0.439255

Reliability Index β_{FS} = 4.535094

Probability of Failure P_f = **0.00%**

Notes:

1. The average pile diameters were not changed with the varying stratigraphy. This simplification is not exactly correct, however variations in pile diameter would be very minor.

Exhibit 11

Probability of Failure Curve for the Axial Capacity of a Single Pile

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Summary

Elevation of Water Surface	Probability of Failure	
892.00	44.67%	Top of Wall
891.00	28.80%	
890.00	14.41%	
889.00	4.98%	
888.00	1.54%	
887.00	0.28%	
886.00	0.07%	
885.00	0.01%	
884.00	0.00%	

South Topeka Floodwall Station 84+50 - Top of Wall Elev. 892, Landside Elev. 880
Probability of Failure for Axial Pile Capacity

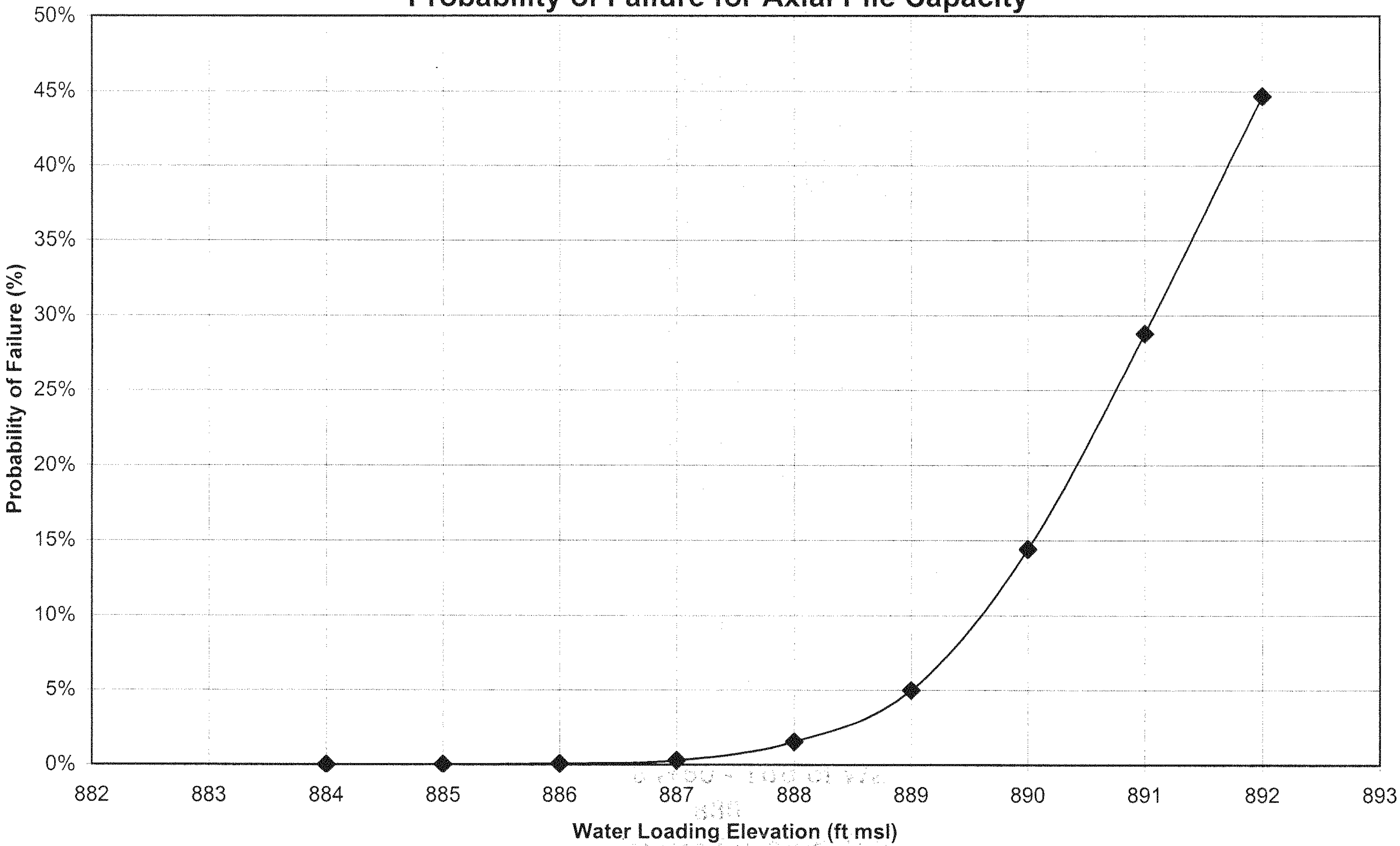

Exhibit 12

Review of Topeka Feasibility Study
Subsurface Investigation and Soils Test Data

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SOUTH TOPEKA - DRILLING AND SAMPLING

DU-89 ; STA 64+00 ℄ CREST OF LEVEE

LEVEE HEIGHT APPROX 9'

9.5' BLANKE

3.5' SAND

BELOW THAT SAND & SILT LAYERS.

TORVANE

9' ________________ TOP OF GROUND

11' - AVG S_u = 0.67 tsf

13.5 - AVG S_u = 0.9 tsf

17' - AVG S_u = 0.4 tsf

LAB TESTING

ATTERBURG LIMITS: $47 \leq LL \leq 22$

$31 \leq PI \leq 5$

MOSTLY CLASSIFIED AS CL OR ML TO 30'

ALL Q TESTS FOR TOPEKA FEASIBILITY STUDY, APRIL 2001

- SOLDIER CREEK, STA 190+00 ; 7.5-9.5', CL S_u = 910 psf
- OAKLAND, STA 399+00 ; 5.0-7.0', CH (VERY FAT) S_u = 540 psf
- OAKLAND ; STA 518+00 ; 5.0'-7.0' CH S_u = 730 psf

TOPEKA FEASIBILITY STUDY - CPT DATA OAKLAND UNIT

OAK1CITY THRU OAK7CITY → STA 80+00 TO 95+00
- THESE ARE CLOSEST TO SOUTH TOPEKA UNIT.

• THE APPARENT THICKNESS OF BLANKET CORRELATES PRETTY WELL WITH RECORD DRAWINGS:

5 TO 10' BLANKET, WITH SOME PERCHED SAND LENSES

• PUSHES ARE BETWEEN 15' TO 20'; CANNOT READILY OBTAIN PILE CAPACITY FOR A 25' PILE WITH PILE HEAD 3' TO 5' BELOW GROUND.

HOWEVER; COMPUTING PILE SKIN FRICTION FROM CPT TESTS IS SIMILAR TO USING UNDRAINED SHEAR STRENGTH FOR UNDRAINED LOADING IN CLAY

a) BASED UPON UNDRAINED SHEAR STRENGTH

$Q_s = f_s A_s$; $f_s = \alpha S_u$ $\quad 1 \leq \alpha \leq 0.5$ DEPENDING UPON S_u; AS S_u DECREASES, α INCREASES

b) BASED UPON CPT SLEEVE FRICTION

$Q_s = \alpha' \bar{f}_s A_s$ $\quad \bar{f}_s$ = MEASURED SLEEVE FRICTION

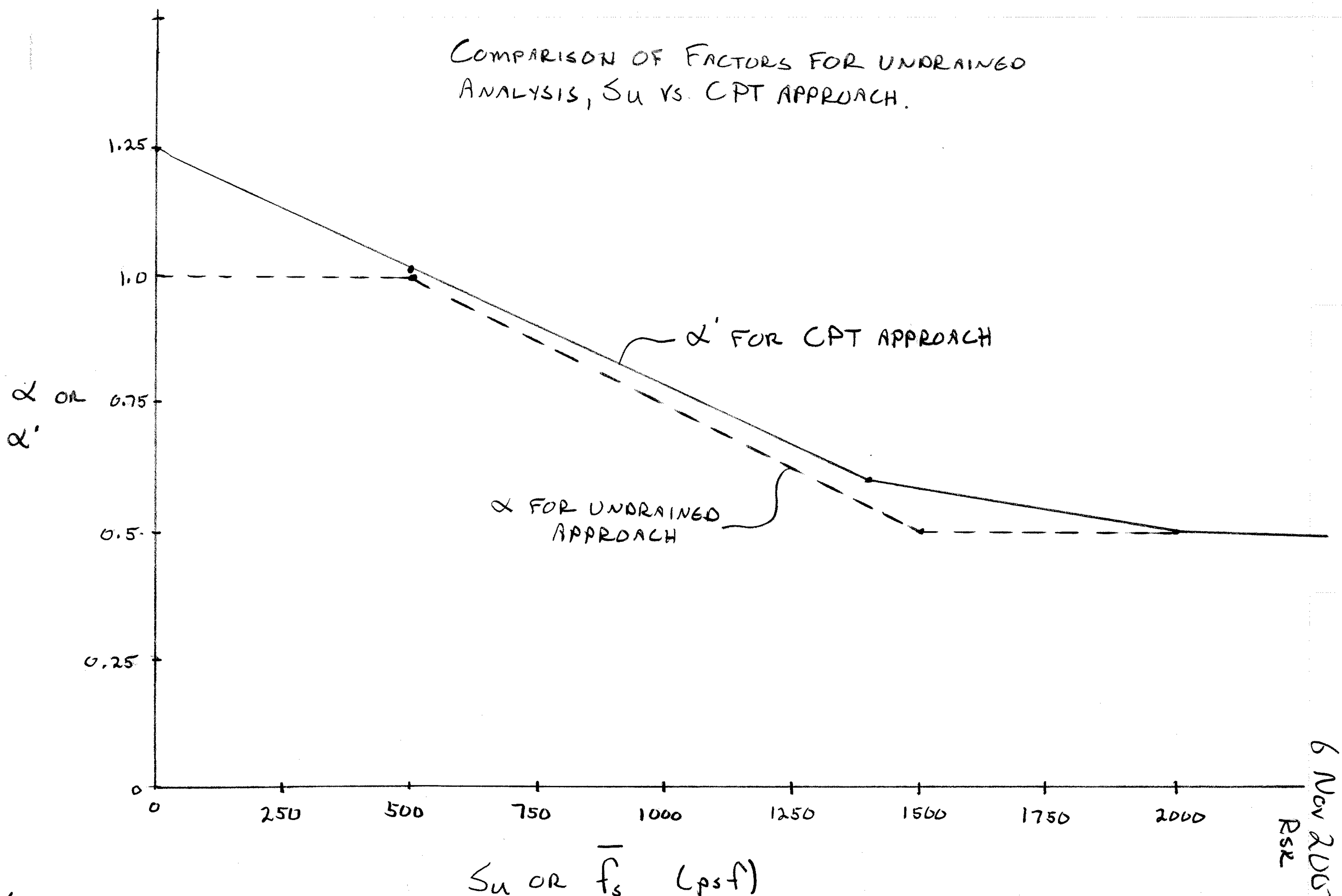
$1.2 \leq \alpha' \leq 0.4$, DEPENDING UPON $\bar{f}_s$; AS $\bar{f}_s$ DECREASES, α' INCREASES

SEE α / α' COMPARISON PLOT PAGE 3

SO:

$$\boxed{S_u \approx \bar{f}_s}$$

COMPARE $\bar{f}_s$ FROM CPT TO EXPECTED VALUE OF S_u USED IN AXIAL CAPACITY CALCULATIONS:



6 Nov 2007
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EXPECTED VALUE OF $S_u = 600\,psf$ FOR CL-ML

- IGNORE UPPER 5' OF PILE

	$f_{s\,AVG}$
OAK1 CITY, STA 80+00	700 psf
OAK2CITY; STA 82+00	275 psf
AVG: $\dfrac{(200psf)(2ft) + 300psf(4ft)}{6ft}$	
OAK3CITY; STA 85+00	600 psf
OAK4 CITY; STA 86+00	300 psf
OAK5 CITY; STA 90+00	500 psf
OAK6 CITY; STA 91+00	730 psf
AVG; $\dfrac{500psf(2ft) + 800psf(7ft)}{9ft}$	
OAK7CITY; STA 92+00	300 psf
OAK1HAL; STA 95+00	150 psf
OAK2HAL; STA 96+00	
AVG $\dfrac{700psf(3ft) + 400psf(6ft)}{9ft}$	500 psf
OAK1ROE; STA 102+00	
AVG $\dfrac{150psf(4.5ft) + 600psf(3.5ft)}{8ft}$	350 psf
OAK 2ROE; STA 105+00	
AVG = $\dfrac{260psf(2ft) + 500psf(3ft)}{5ft}$	400 psf
	AVG: 435 psf

4

RSK

6 Nov 2007

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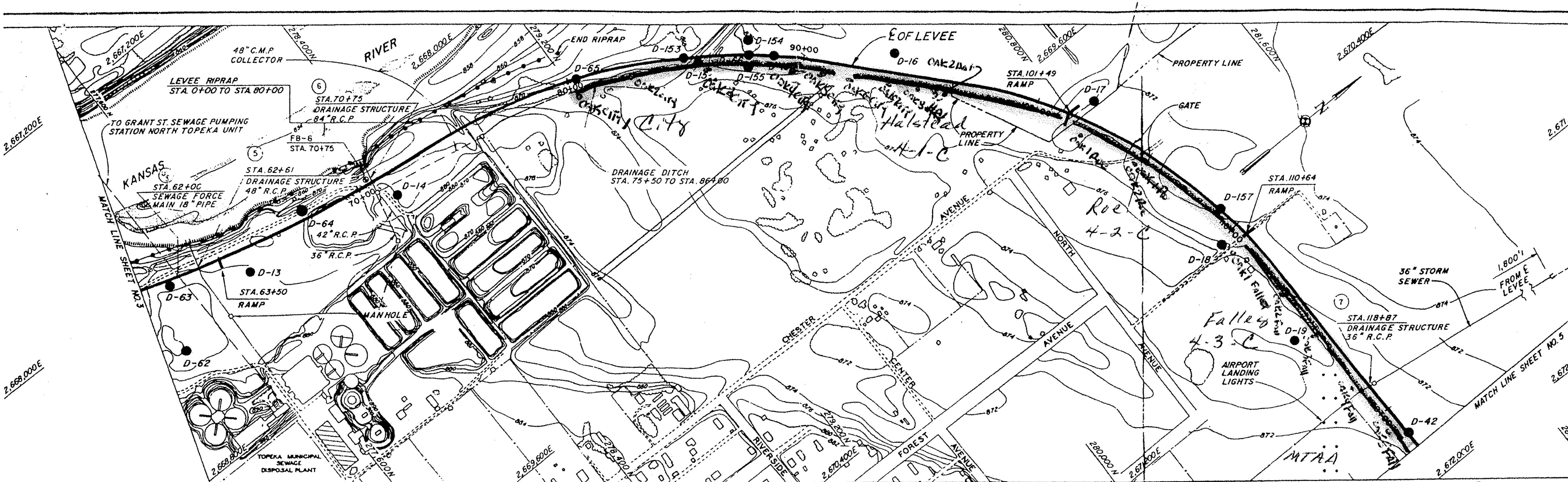
SUMMARY

FOR THE BLANKET MATERIALS

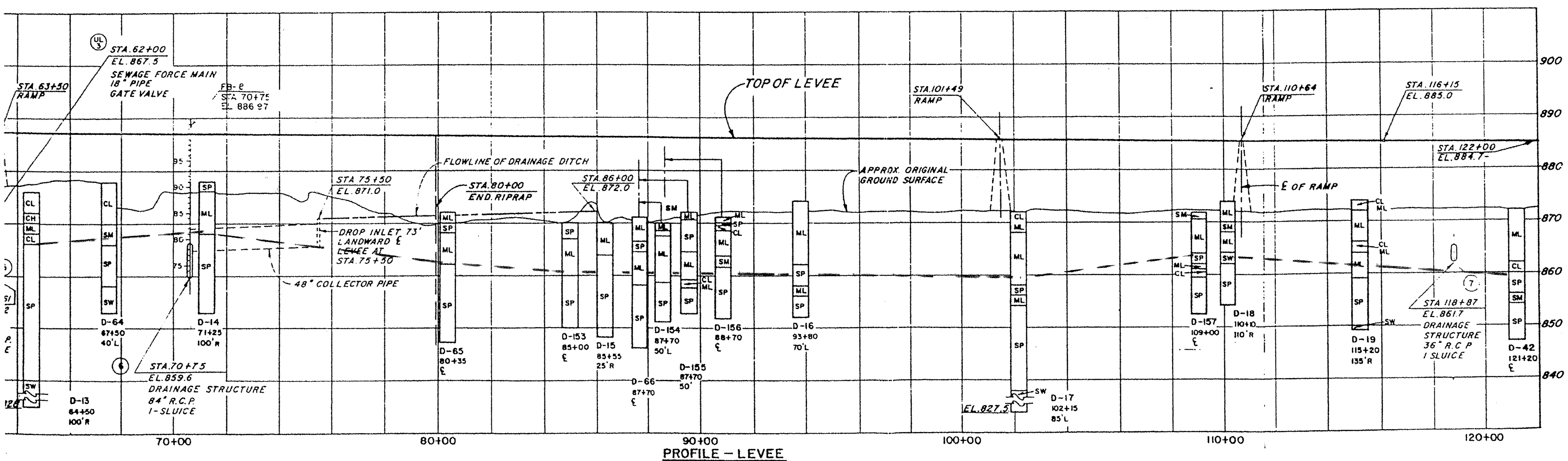
· BASED UPON THE PREVIOUS COMPARISON; IT APPEARS THE UNDRAINED SHEAR STRENGTH USED IN THE CALCULATIONS WAS REASONABLE.



PLAN

CPT TESTING
STATIONING

80+00 to 100+00
100+00 to 110+00
110+00 to 122+00



PROFILE - LEVEE

SYM.	DESCRIPTION
	REVISIONS

OPERATIONAL D
TOPEKA, KANSAS FLOOD PROT
OAKLAND

PLAN, PROFILE AND UNDERGR
LEVEE STA. 60+00 TO

In 21 sheets
Sheet No. 4

CORPS OF ENGINEERS
KANSAS CITY DIST
SEPTEMBER 1

DESIGNED BY: DRAWN BY: J.E.B. CHECKED BY:

INCLOSURE 6

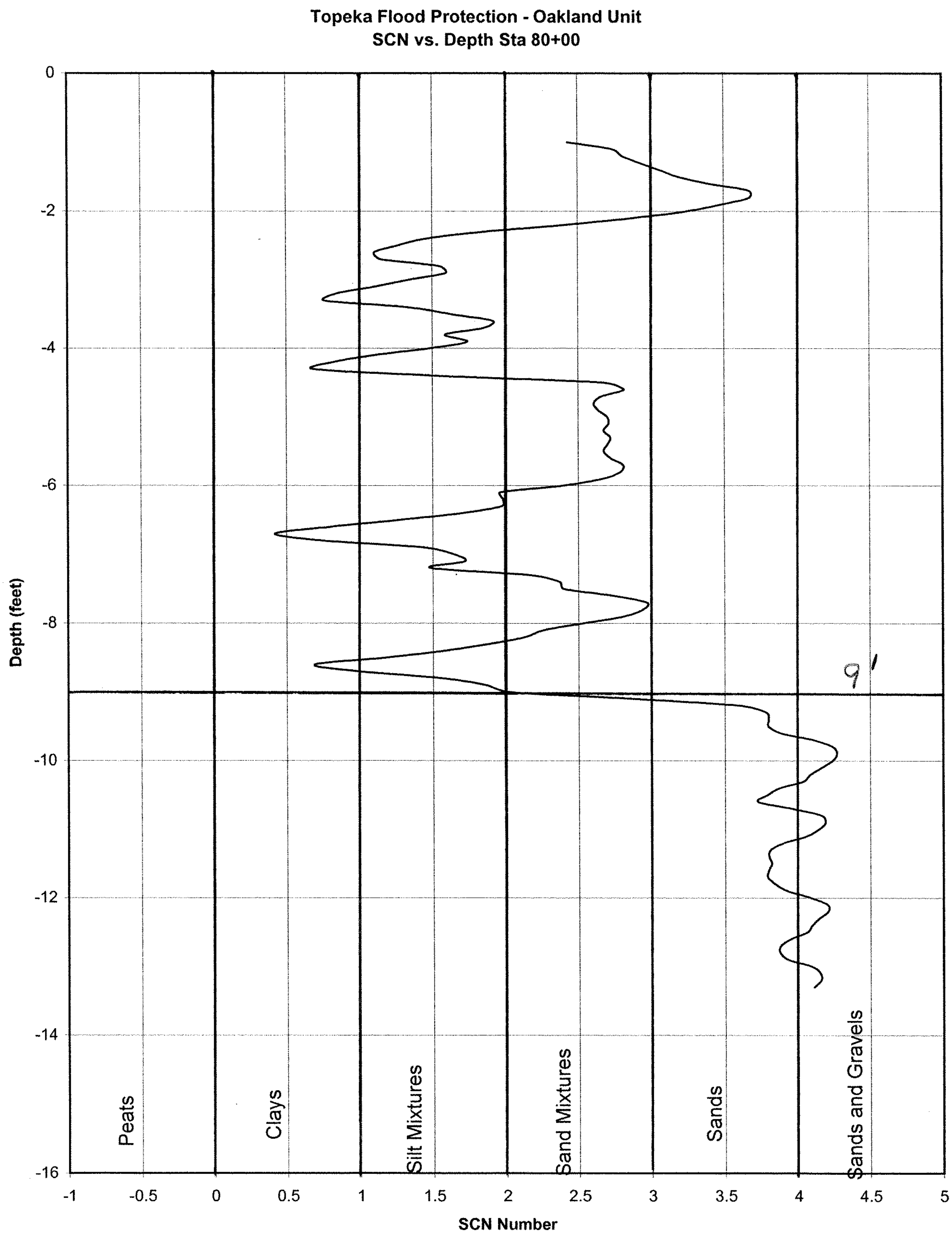
"CITY" AND "HAL"
TEST IF POSSIBLE 8 HOLES @ 250' spacing

TEST "ROE"
5 holes @ 200' spacing

TEST "FALL"
5 holes @ 200' spacing

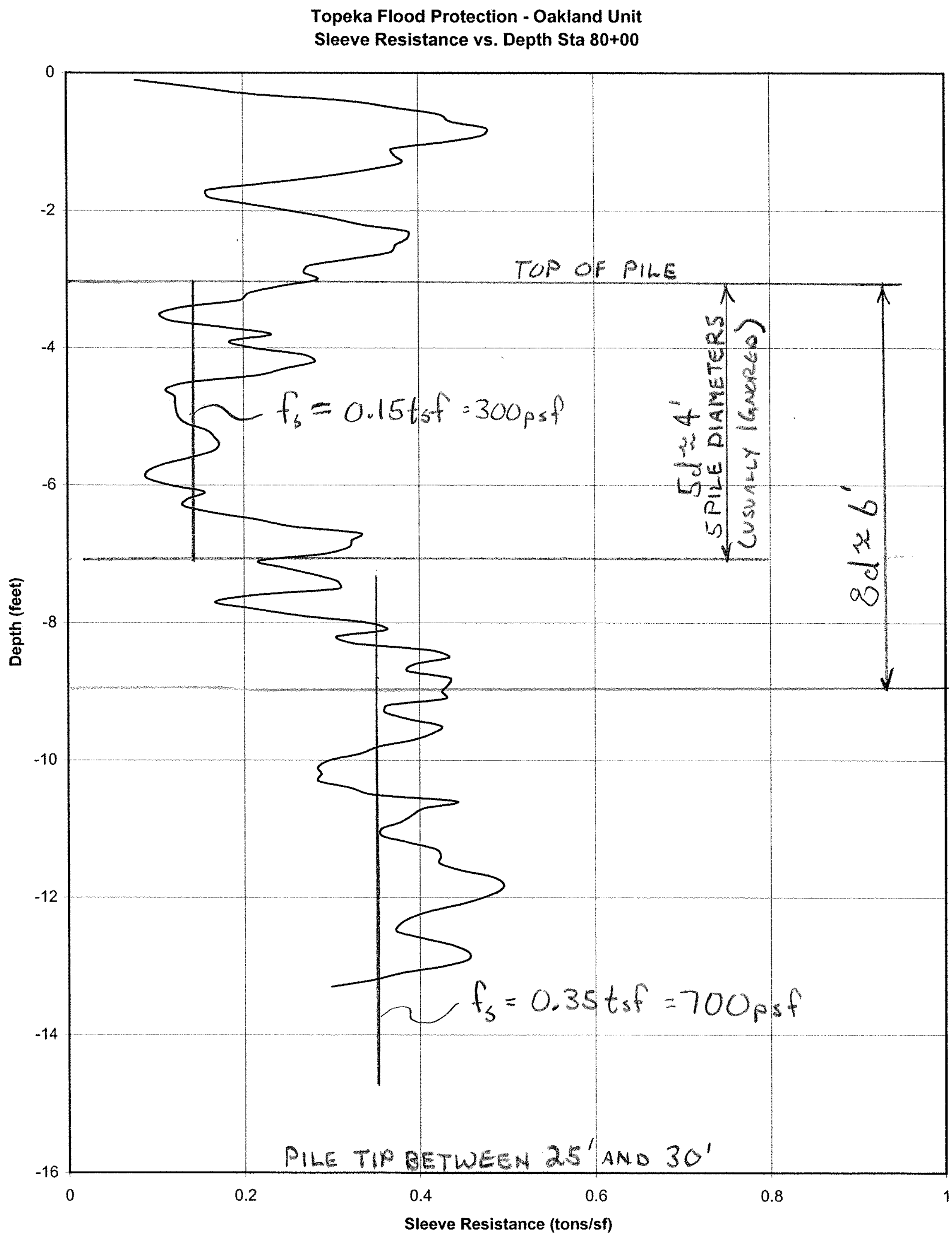
10/1/99 EC-GL

OAK I CITY



Topeka Flood Protection - Oakland Unit
SCN vs. Depth Sta 80+00
Depth (feet)
0
-2
-4
-6
-8
-10
-12
-14
-16
9'
Peats
Clays
Silt Mixtures
Sand Mixtures
Sands
Sands and Gravels
-1
-0.5
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
SCN Number

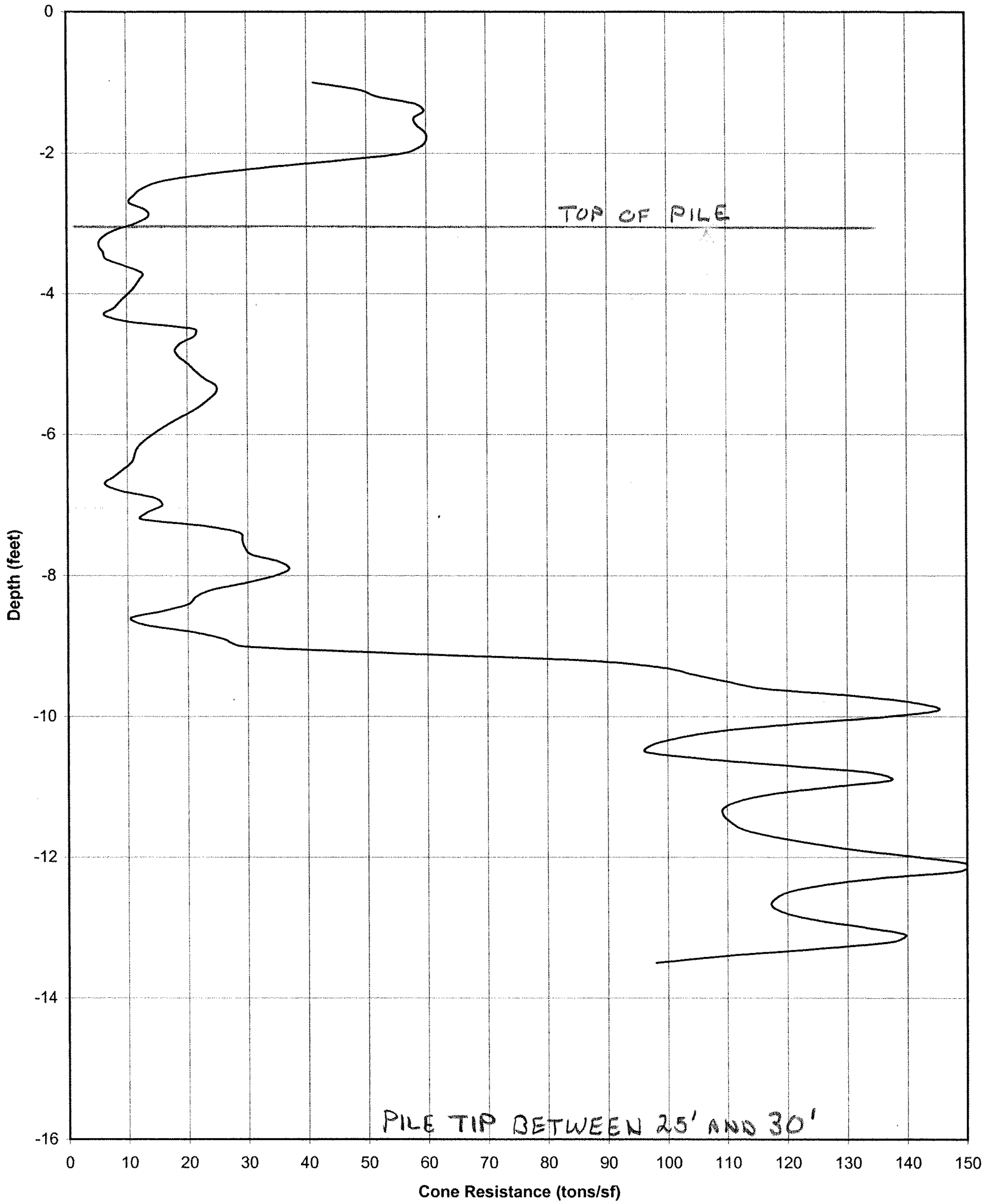
7

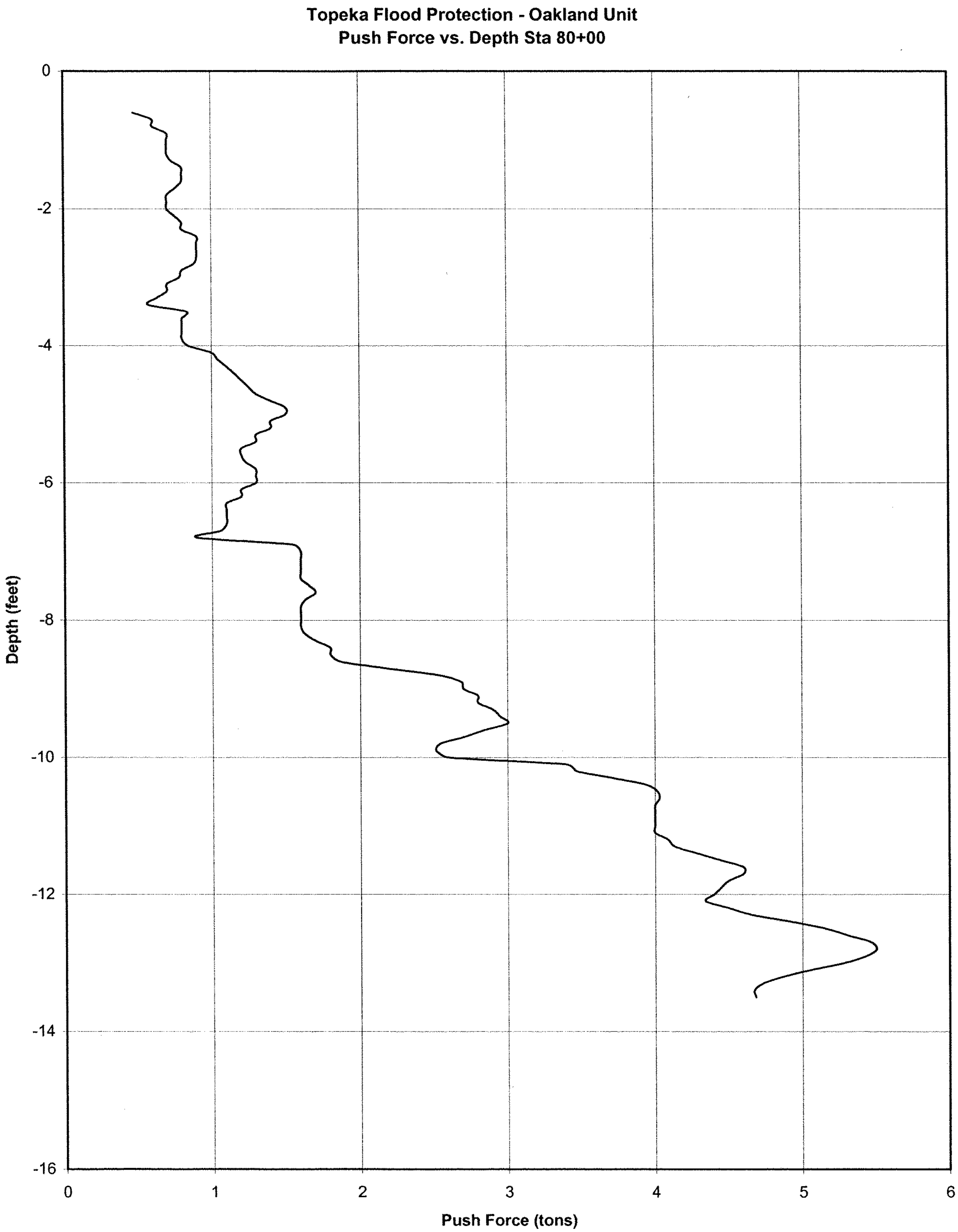


Topeka Flood Protection - Oakland Unit
Sleeve Resistance vs. Depth Sta 80+00
Depth (feet)
Sleeve Resistance (tons/sf)
0
-2
-4
-6
-8
-10
-12
-14
-16
0
0.2
0.4
0.6
0.8
1
TOP OF PILE
f_s = 0.15 tsf = 300 psf
5d ≈ 4'
5 PILE DIAMETERS
(USUALLY IGNORED)
8d ≈ 6'
f_s = 0.35 tsf = 700 psf
PILE TIP BETWEEN 25' AND 30'

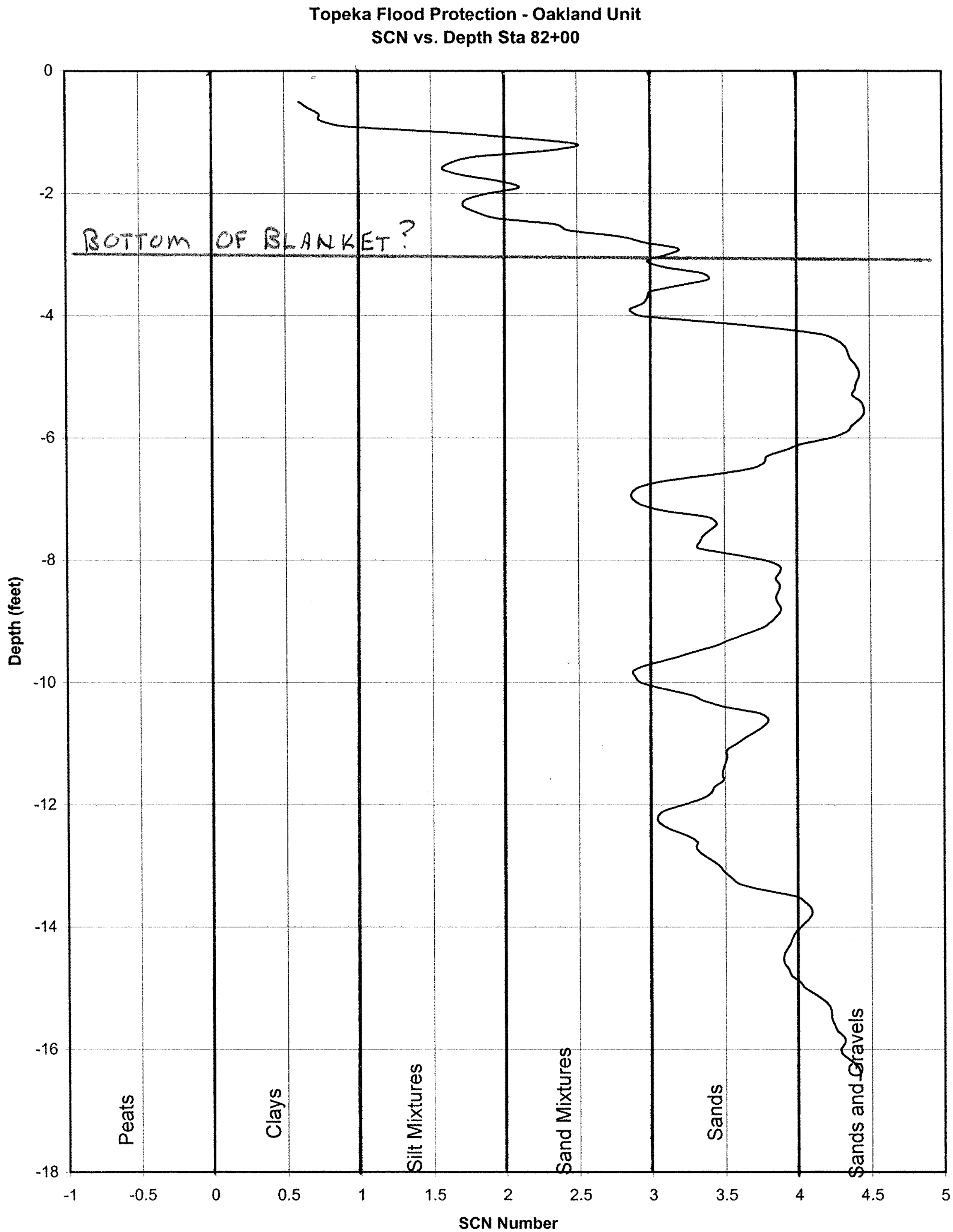
8

Topeka Flood Protection - Oakland Unit
Cone Resistance vs. Depth Sta 80+00



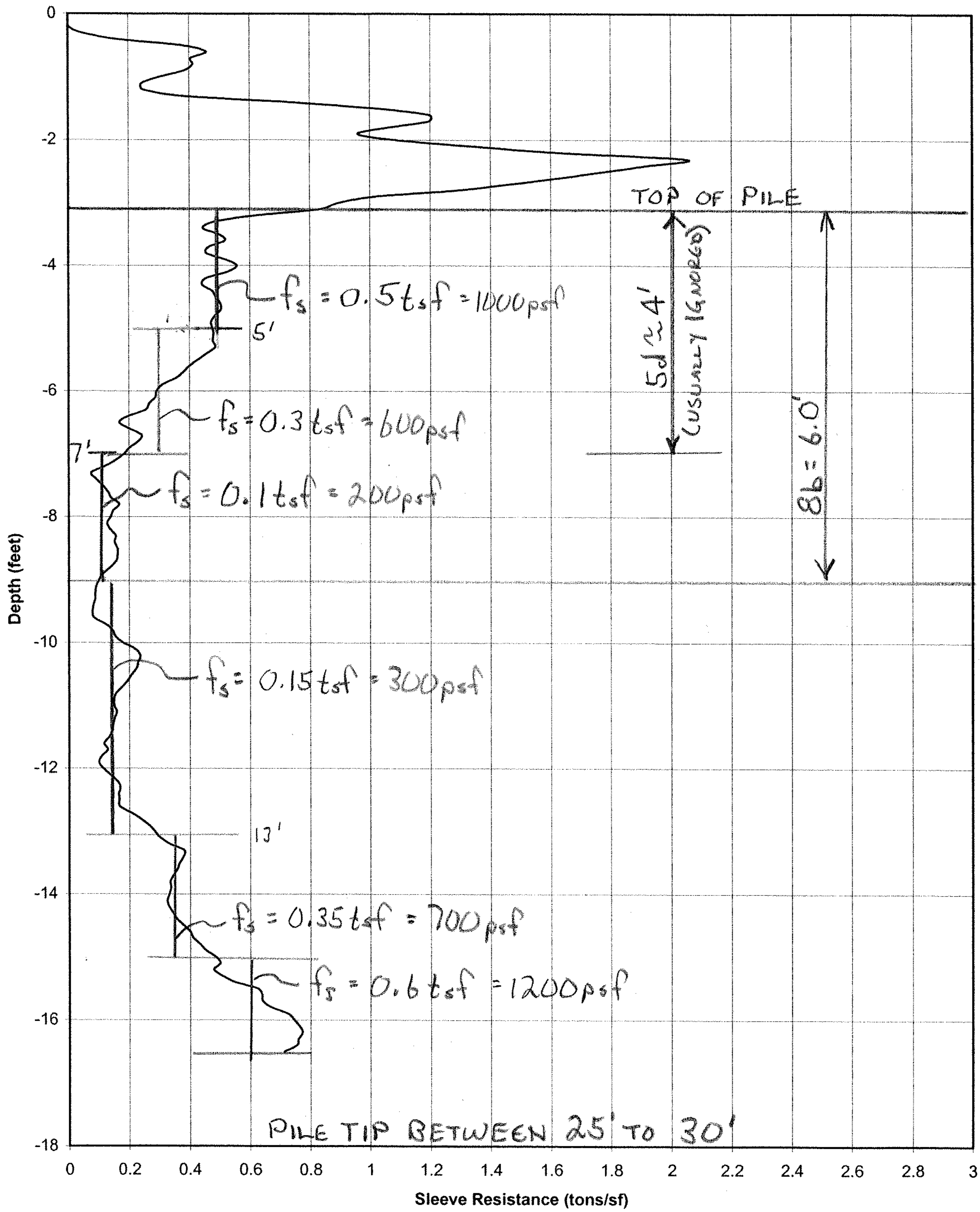

Topeka Flood Protection - Oakland Unit
Push Force vs. Depth Sta 80+00
0
-2
-4
-6
-8
-10
-12
-14
-16
Depth (feet)
0
1
2
3
4
5
6
Push Force (tons)

OAK2CITY

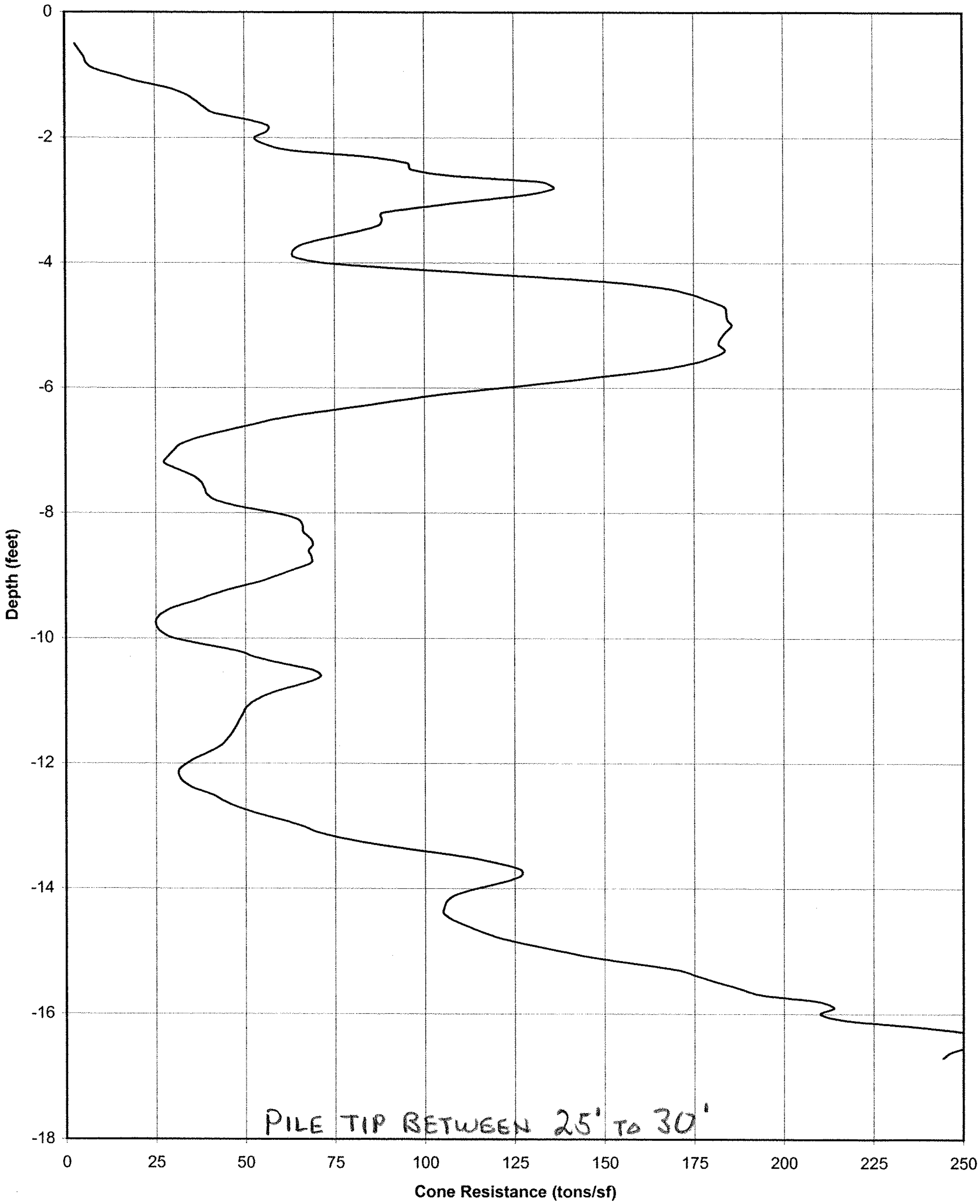


Topeka Flood Protection - Oakland Unit
SCN vs. Depth Sta 82+00
BOTTOM OF BLANKET?
Depth (feet)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
Peats
Clays
Silt Mixtures
Sand Mixtures
Sands
Sands and Gravels
-1
-0.5
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
SCN Number

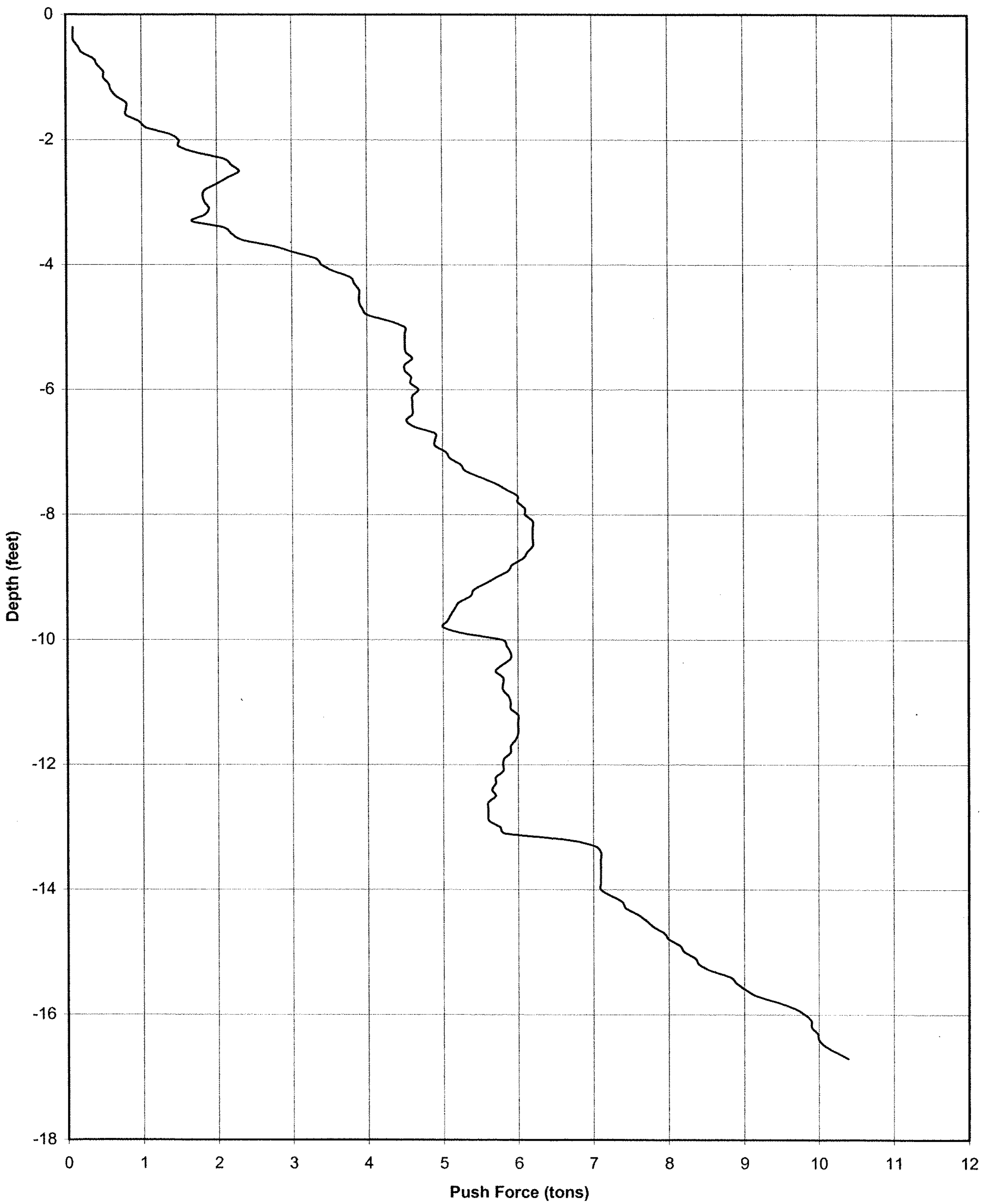
Topeka Flood Protection - Oakland Unit
Sleeve Resistance vs. Depth Sta 82+00



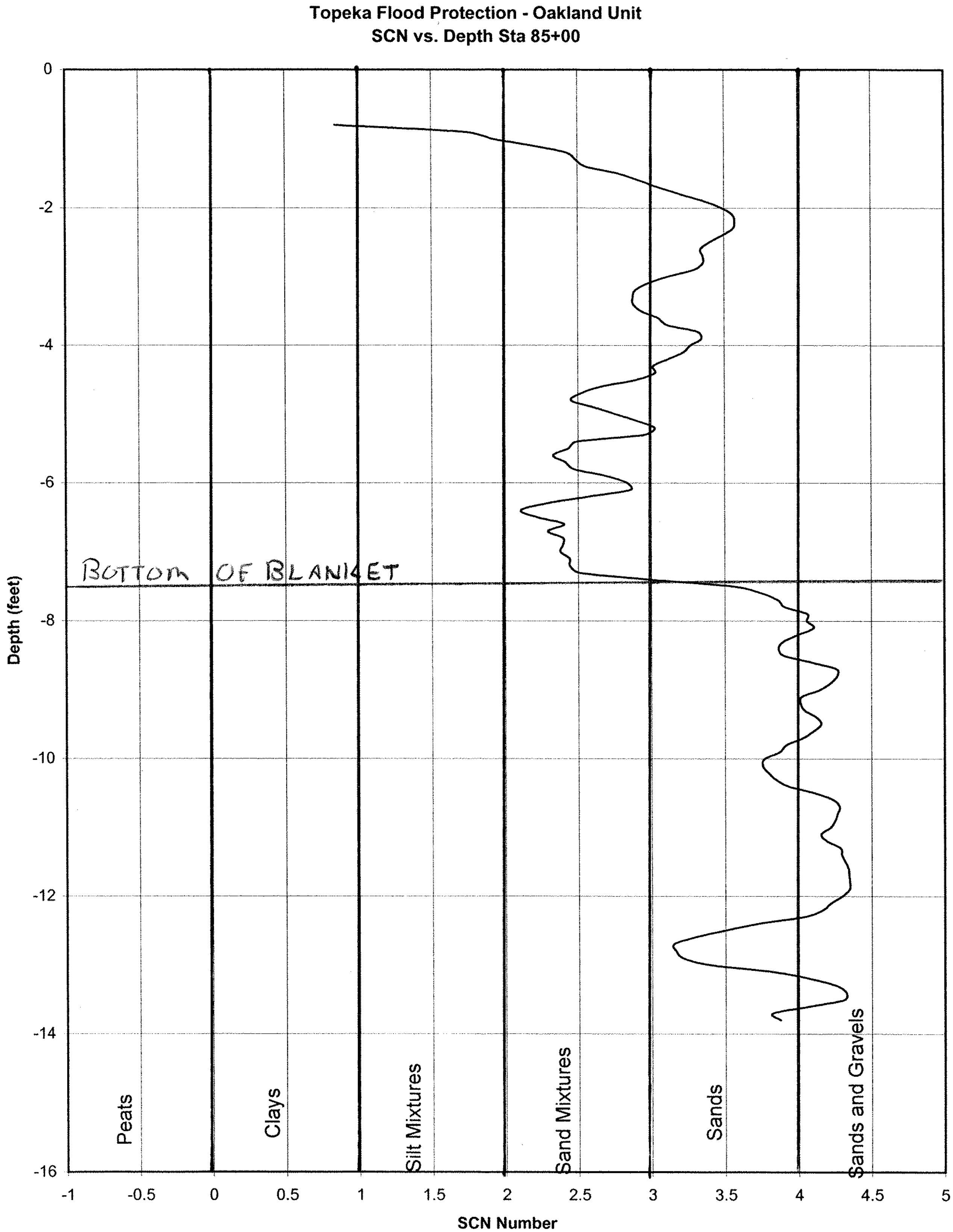
Topeka Flood Protection - Oakland Unit
Cone Resistance vs. Depth Sta 82+00



Topeka Flood Protection - Oakland Unit
Push Force vs. Depth Sta 82+00

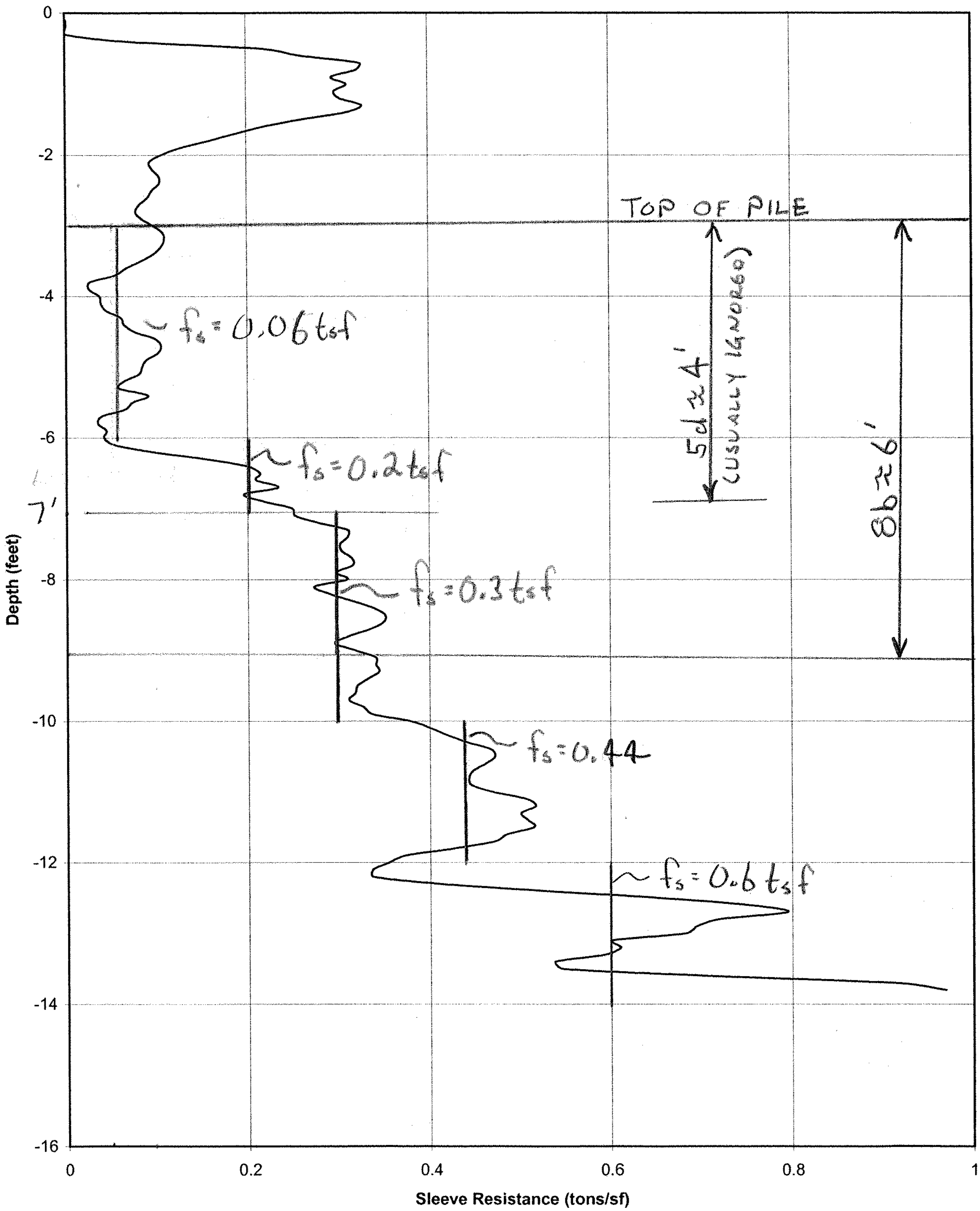


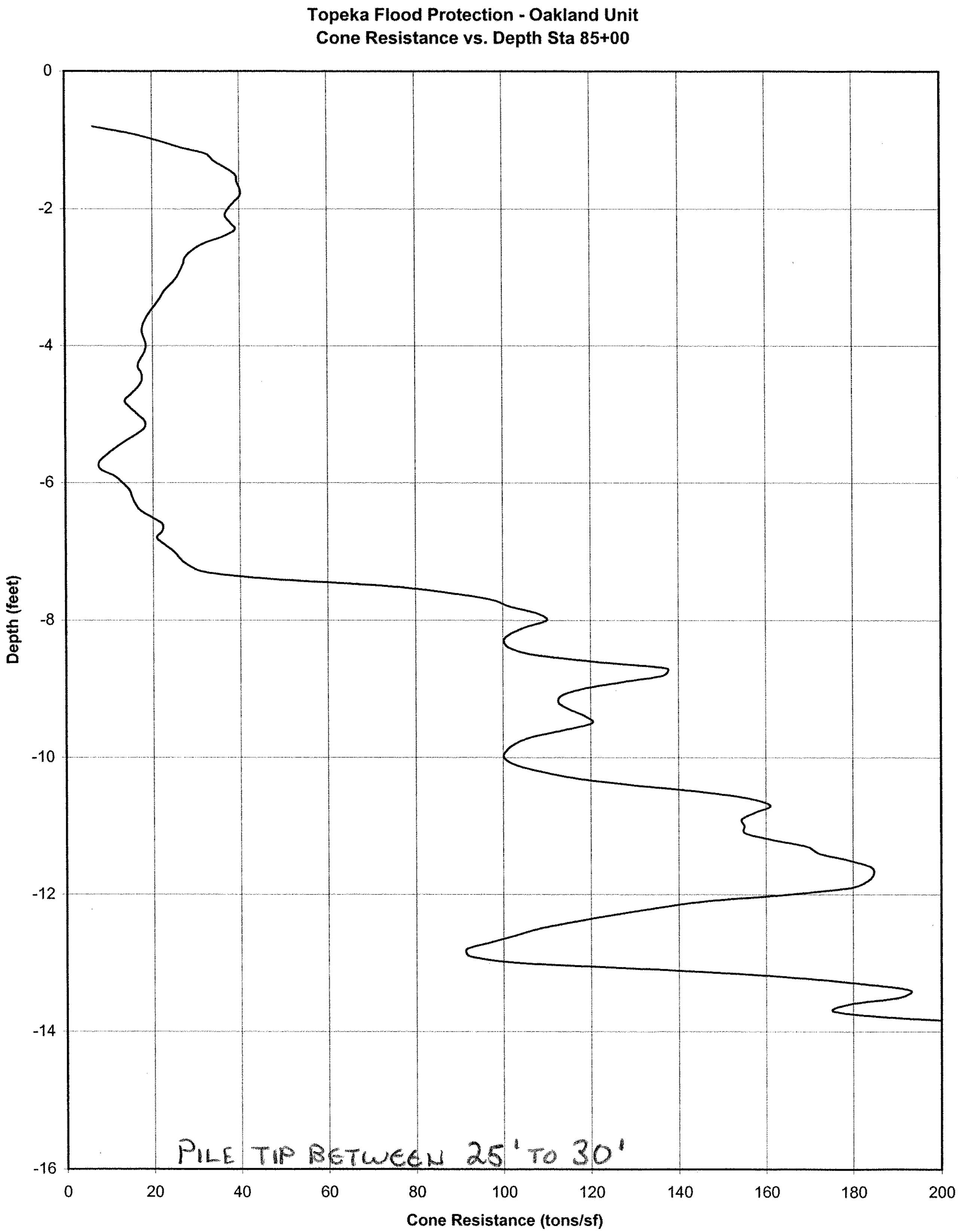
OAK3 CITY



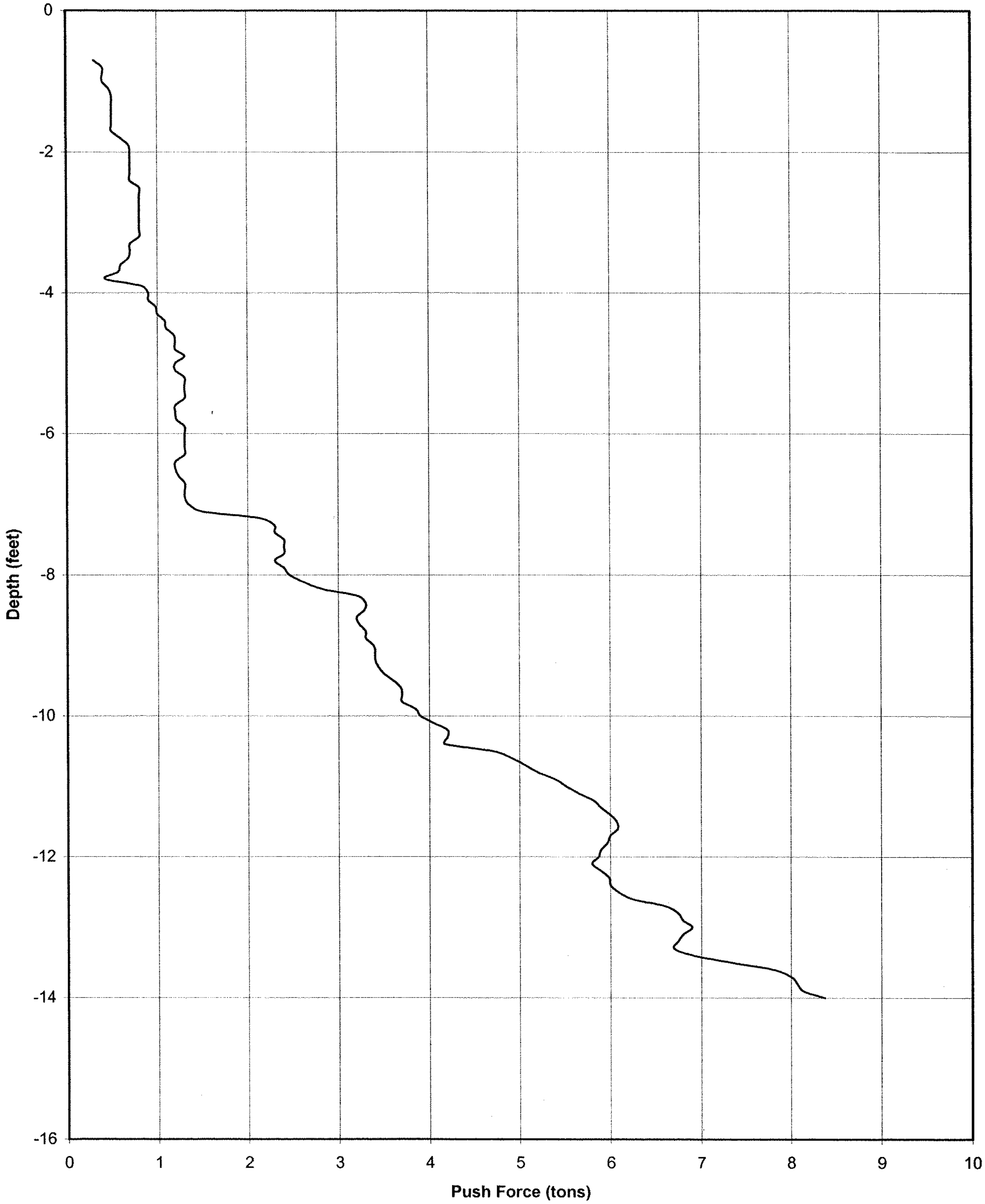
Topeka Flood Protection - Oakland Unit
SCN vs. Depth Sta 85+00
Depth (feet)
SCN Number
BOTTOM OF BLANKET
Peats
Clays
Silt Mixtures
Sand Mixtures
Sands
Sands and Gravels
0
-2
-4
-6
-8
-10
-12
-14
-16
-1
-0.5
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5

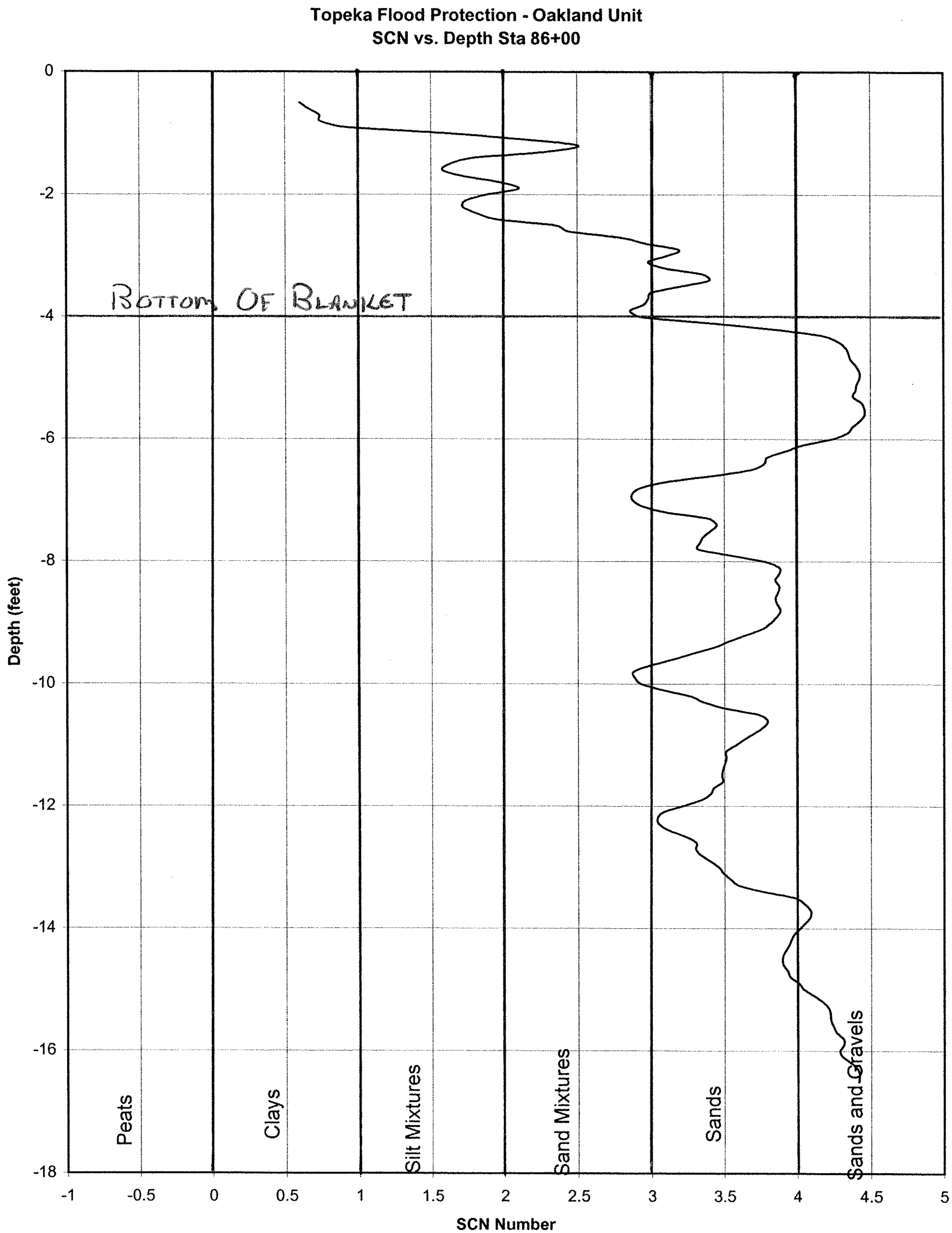
Topeka Flood Protection - Oakland Unit
Sleeve Resistance vs. Depth Sta 85+00



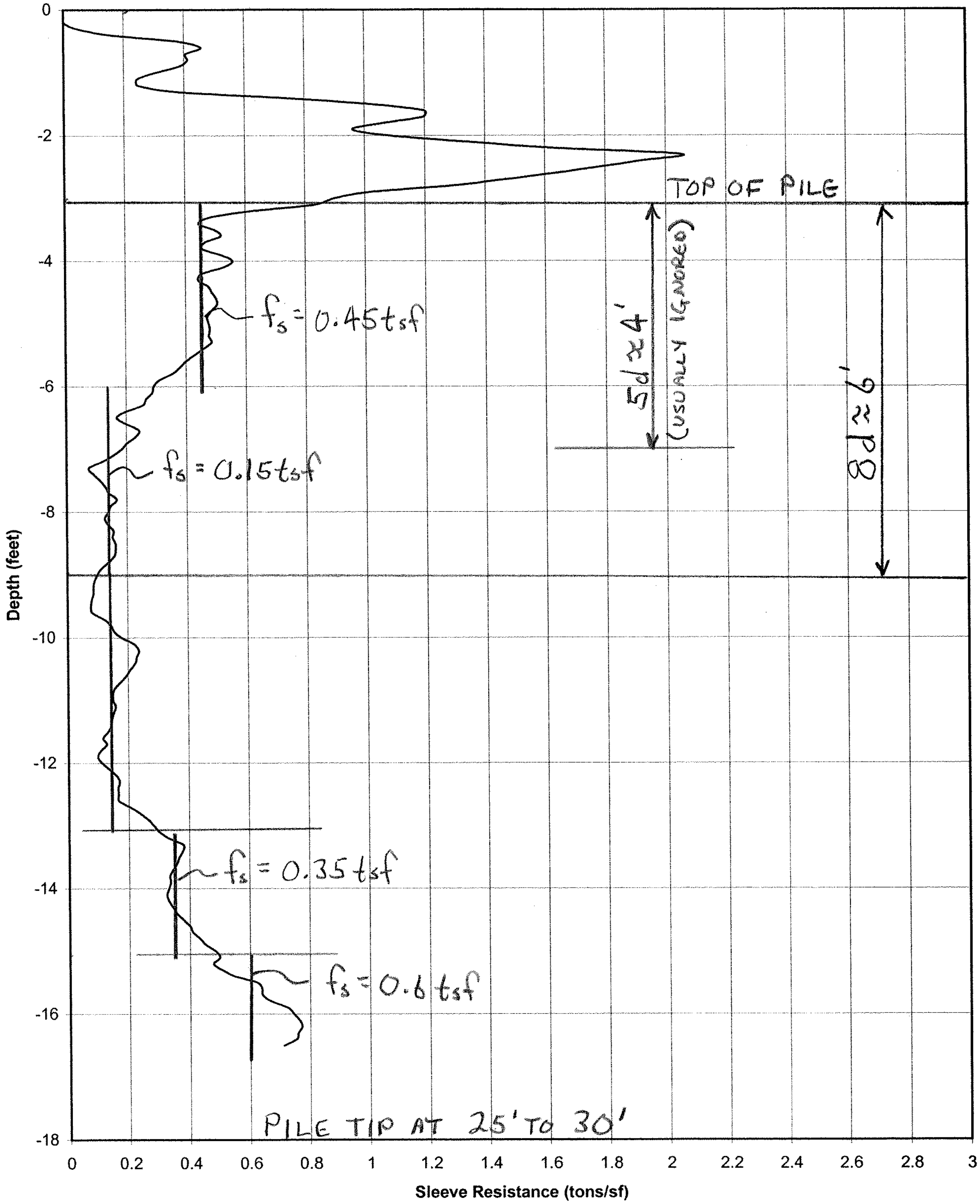

Topeka Flood Protection - Oakland Unit
Cone Resistance vs. Depth Sta 85+00
Depth (feet)
0
-2
-4
-6
-8
-10
-12
-14
-16
0
20
40
60
80
100
120
140
160
180
200
Cone Resistance (tons/sf)
PILE TIP BETWEEN 25' TO 30'

Topeka Flood Protection - Oakland Unit
Push Force vs. Depth Sta 85+00

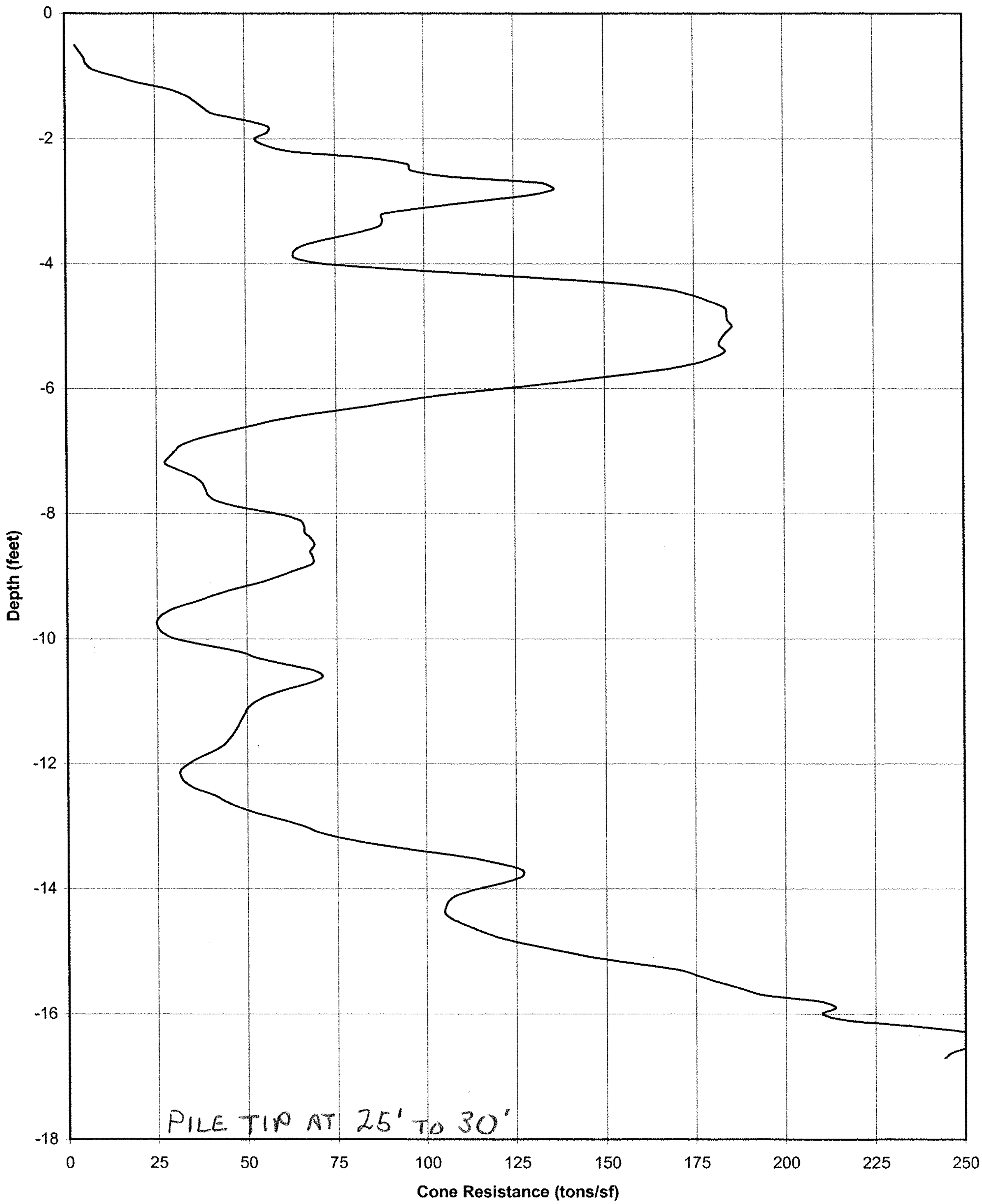


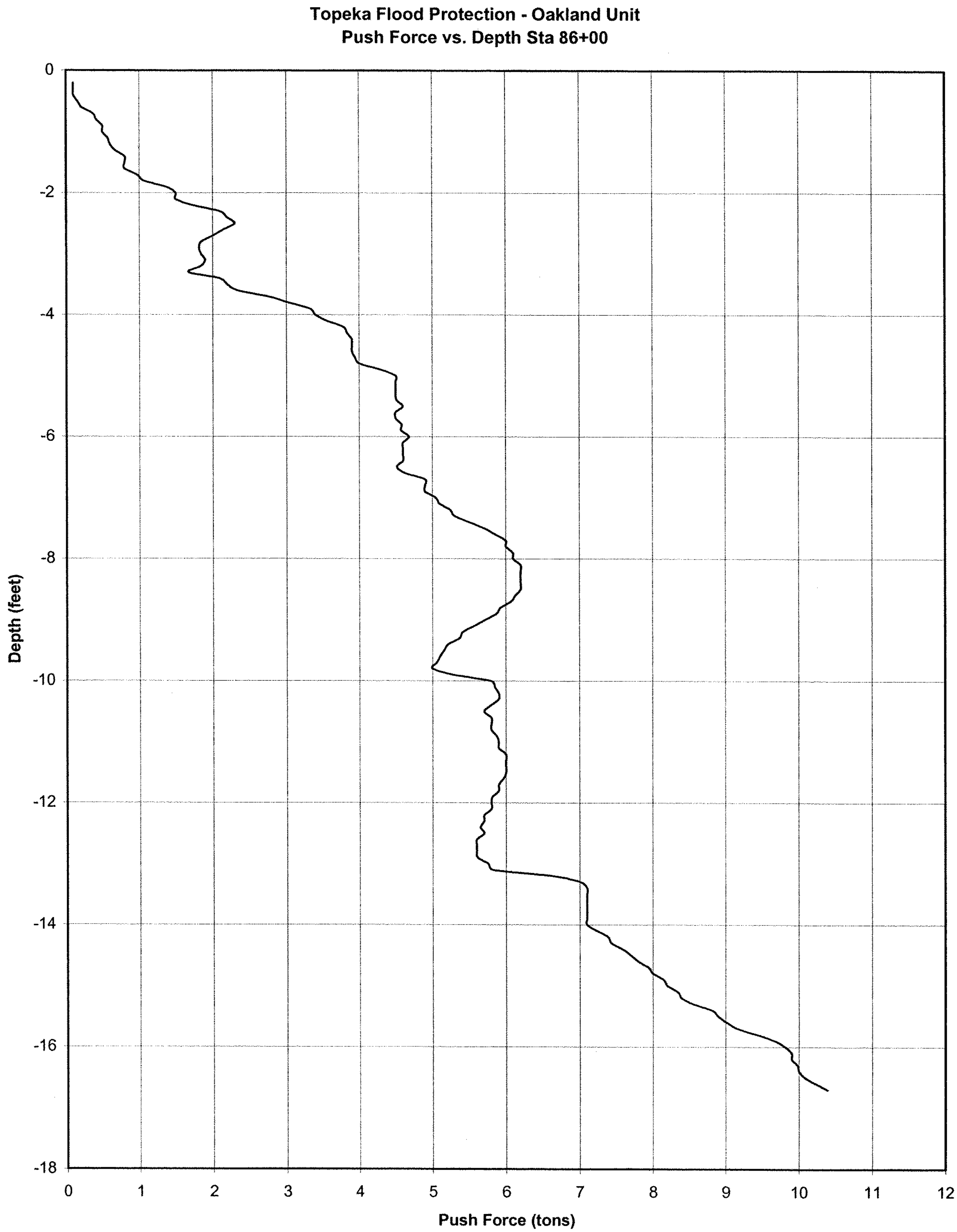

Topeka Flood Protection - Oakland Unit
SCN vs. Depth Sta 86+00
Depth (feet)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
BOTTOM OF BLANKET
Peats
Clays
Silt Mixtures
Sand Mixtures
Sands
Sands and Gravels
-1
-0.5
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
SCN Number

Topeka Flood Protection - Oakland Unit
Sleeve Resistance vs. Depth Sta 86+00



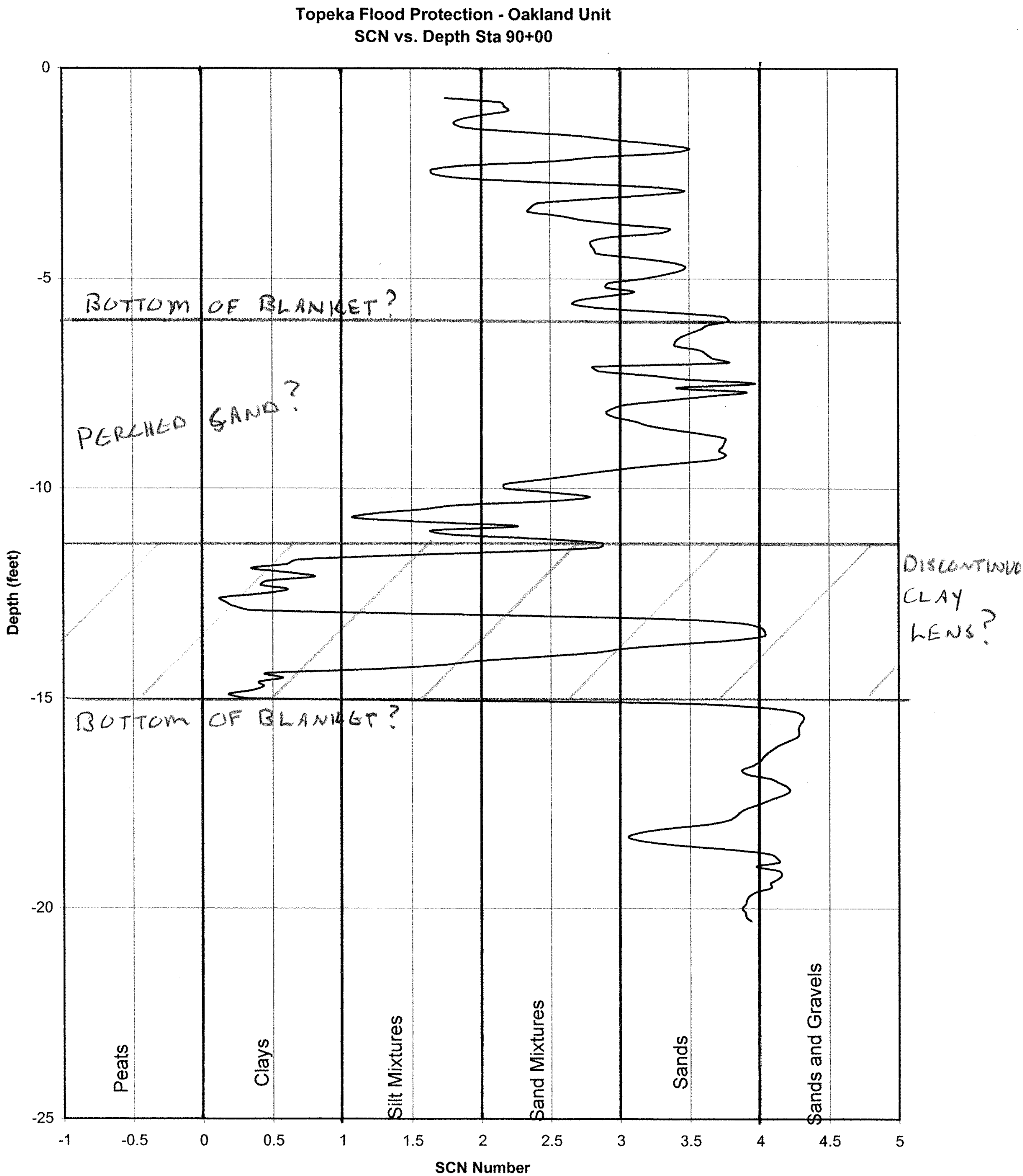
Topeka Flood Protection - Oakland Unit
Cone Resistance vs. Depth Sta 86+00



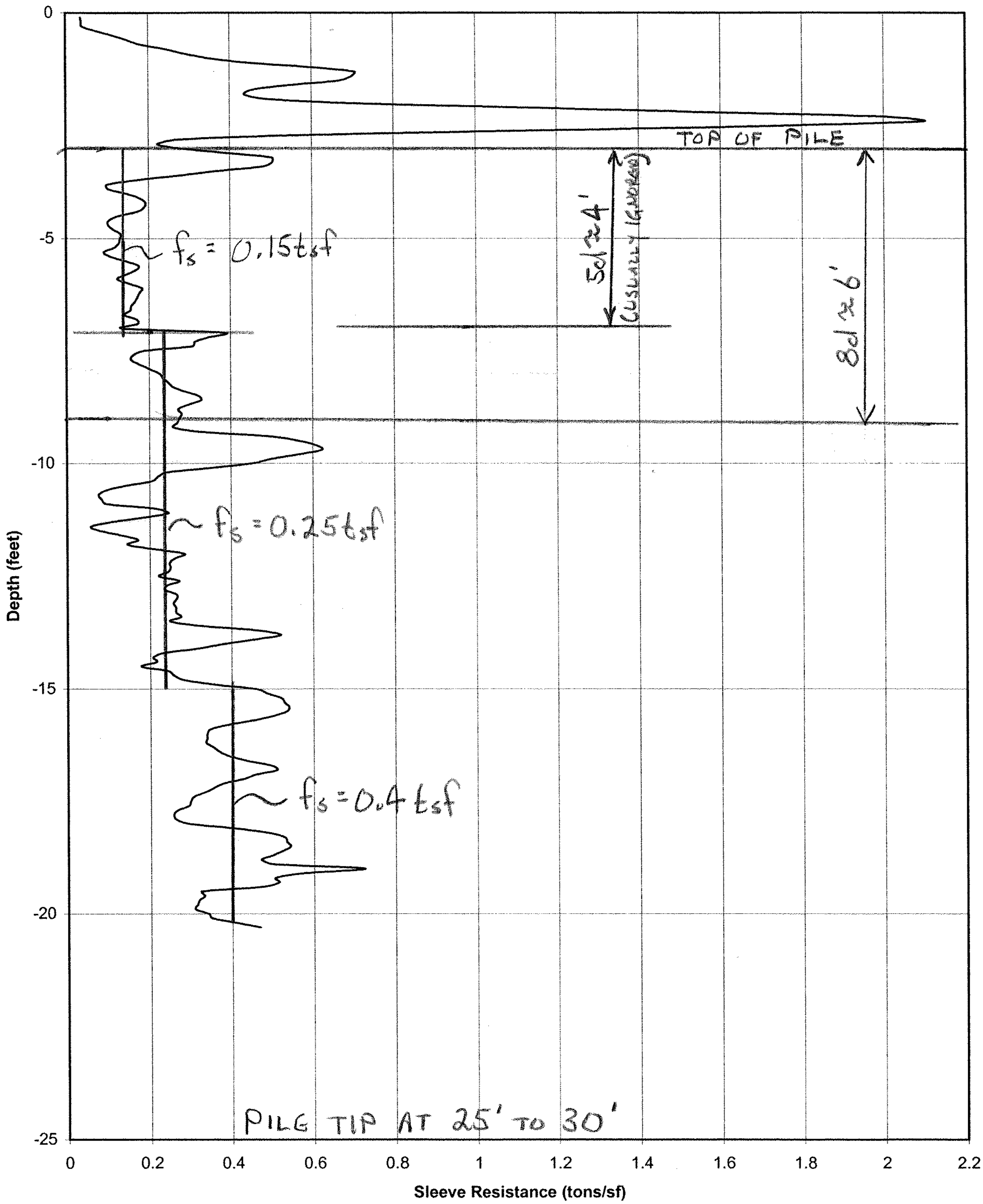


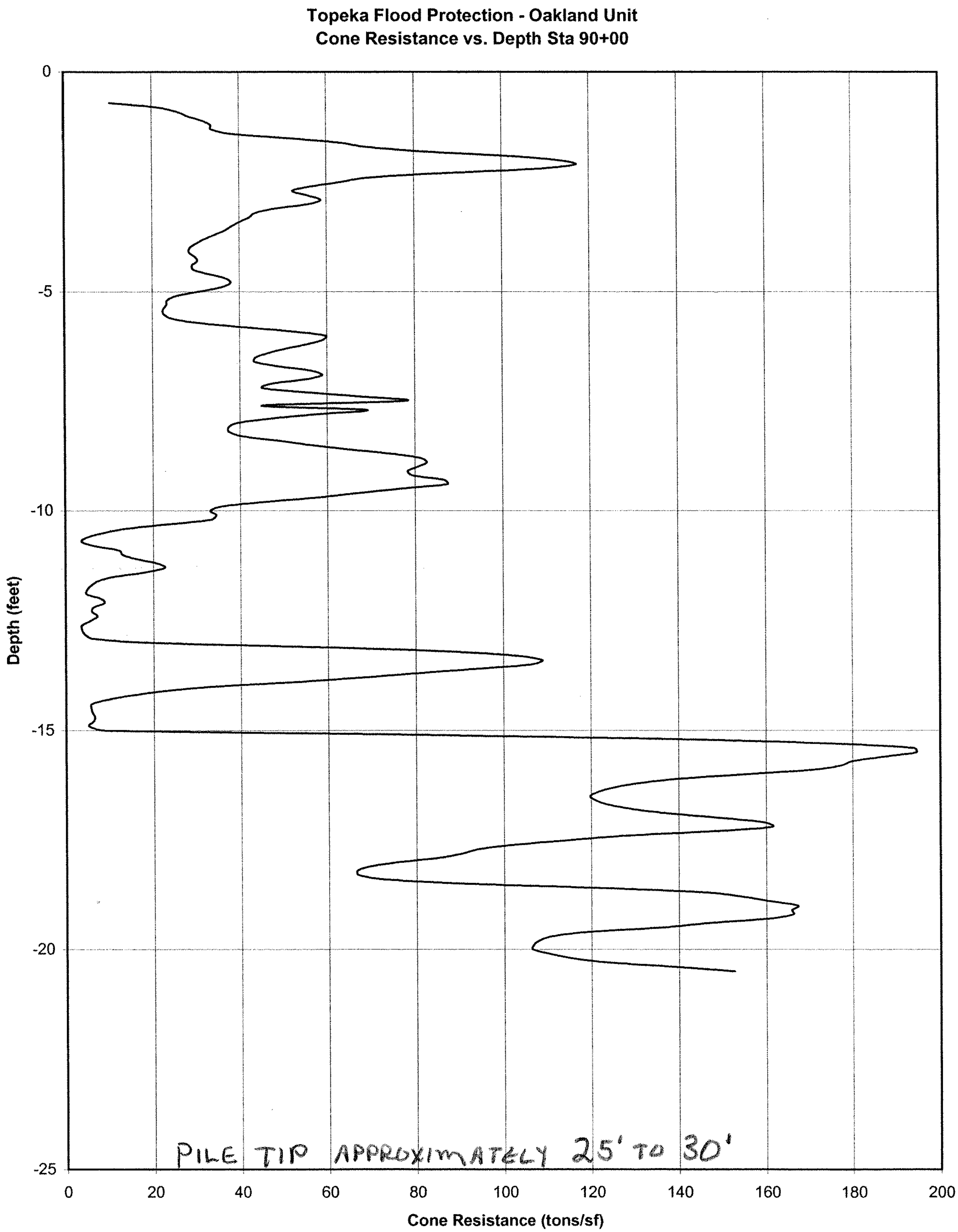
Topeka Flood Protection - Oakland Unit
Push Force vs. Depth Sta 86+00
Depth (feet)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
0
1
2
3
4
5
6
7
8
9
10
11
12
Push Force (tons)

OAKS CITY

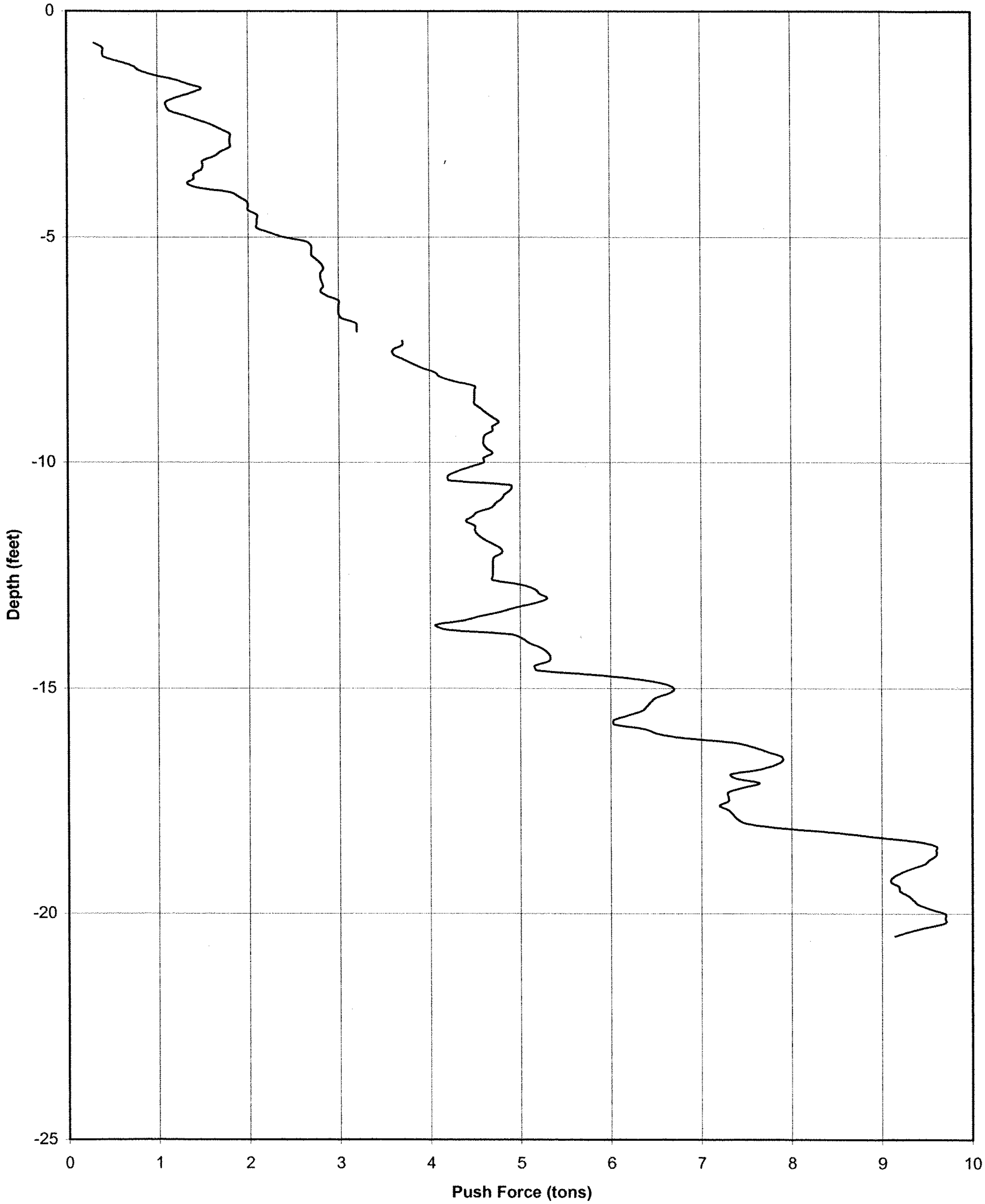

Topeka Flood Protection - Oakland Unit
SCN vs. Depth Sta 90+00
Depth (feet)
SCN Number
0
-5
-10
-15
-20
-25
-1
-0.5
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Peats
Clays
Silt Mixtures
Sand Mixtures
Sands
Sands and Gravels
BOTTOM OF BLANKET?
PERCHED SAND?
DISCONTINUO CLAY LENS?
BOTTOM OF BLANKET?

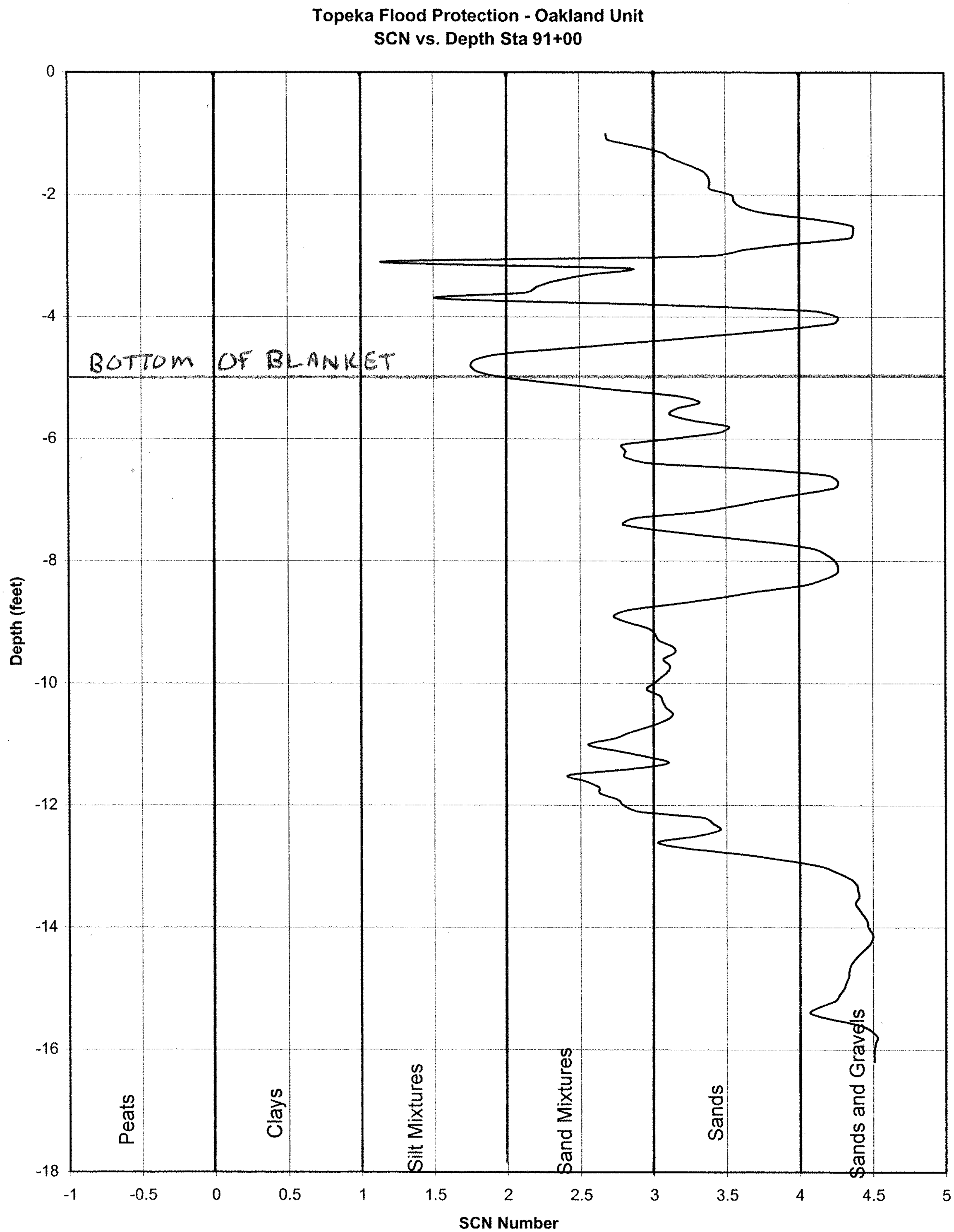
Topeka Flood Protection - Oakland Unit
Sleeve Resistance vs. Depth Sta 90+00

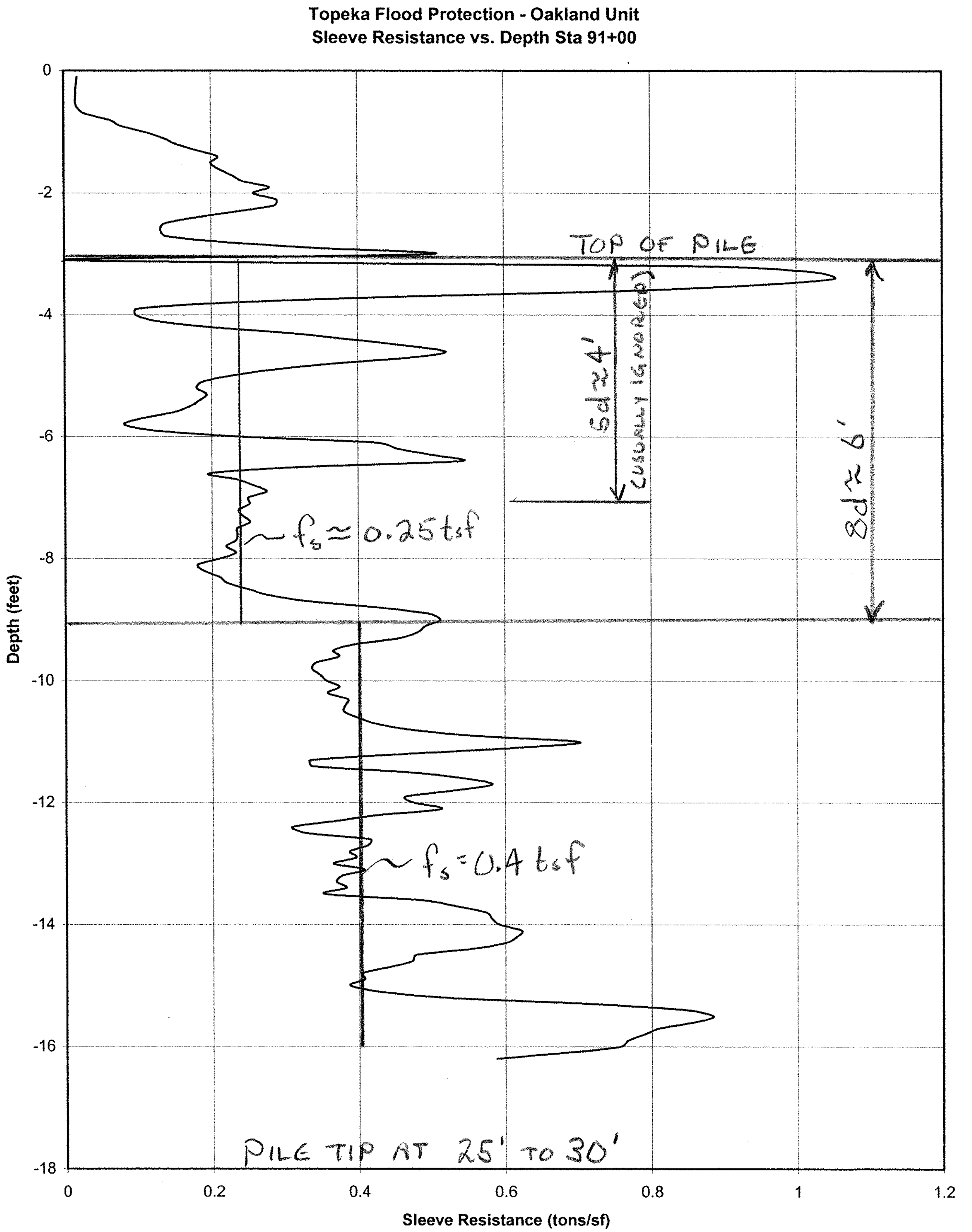



Topeka Flood Protection - Oakland Unit
Cone Resistance vs. Depth Sta 90+00
Depth (feet)
0
-5
-10
-15
-20
-25
0
20
40
60
80
100
120
140
160
180
200
Cone Resistance (tons/sf)
PILE TIP APPROXIMATELY 25' TO 30'

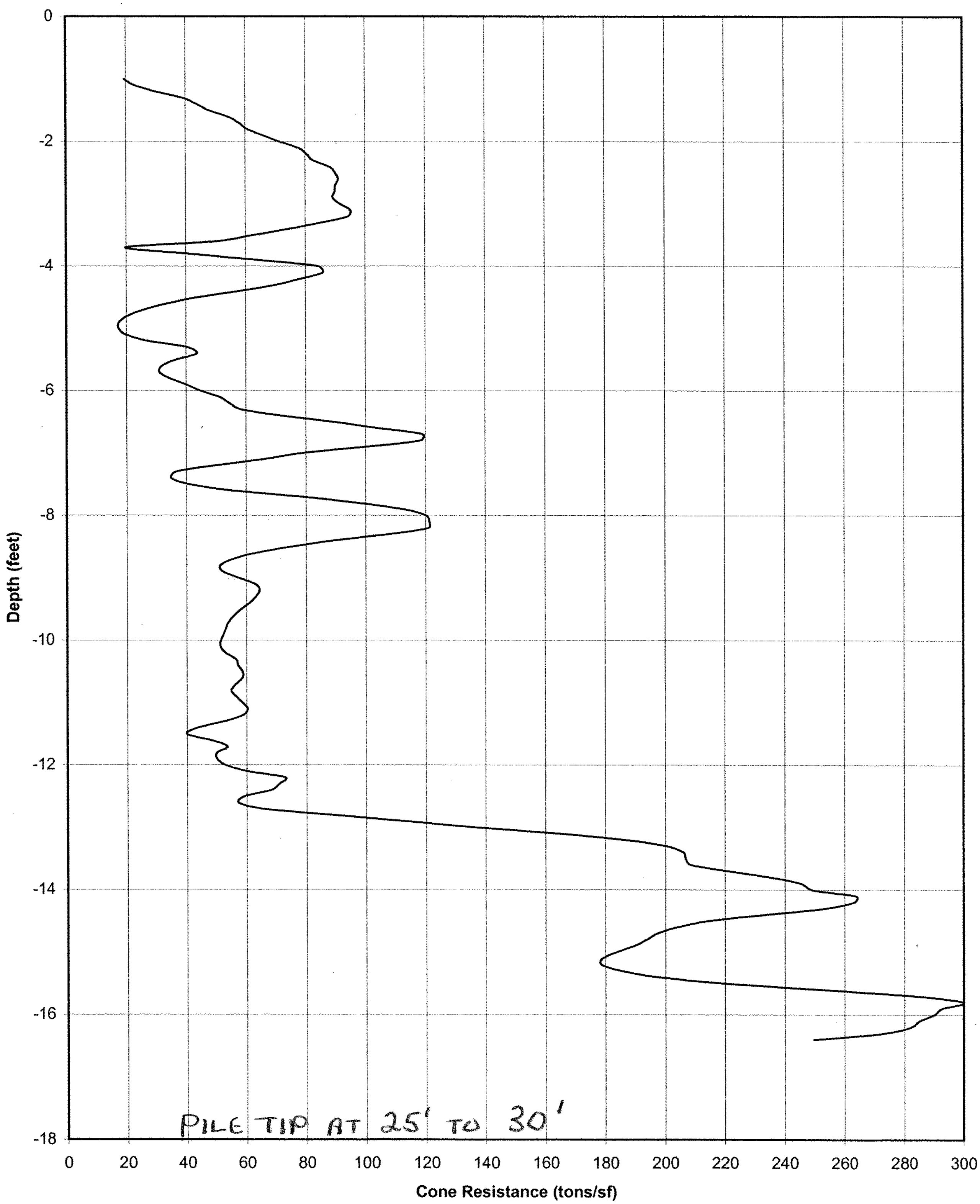
Topeka Flood Protection - Oakland Unit
Push Force vs. Depth Sta 90+00



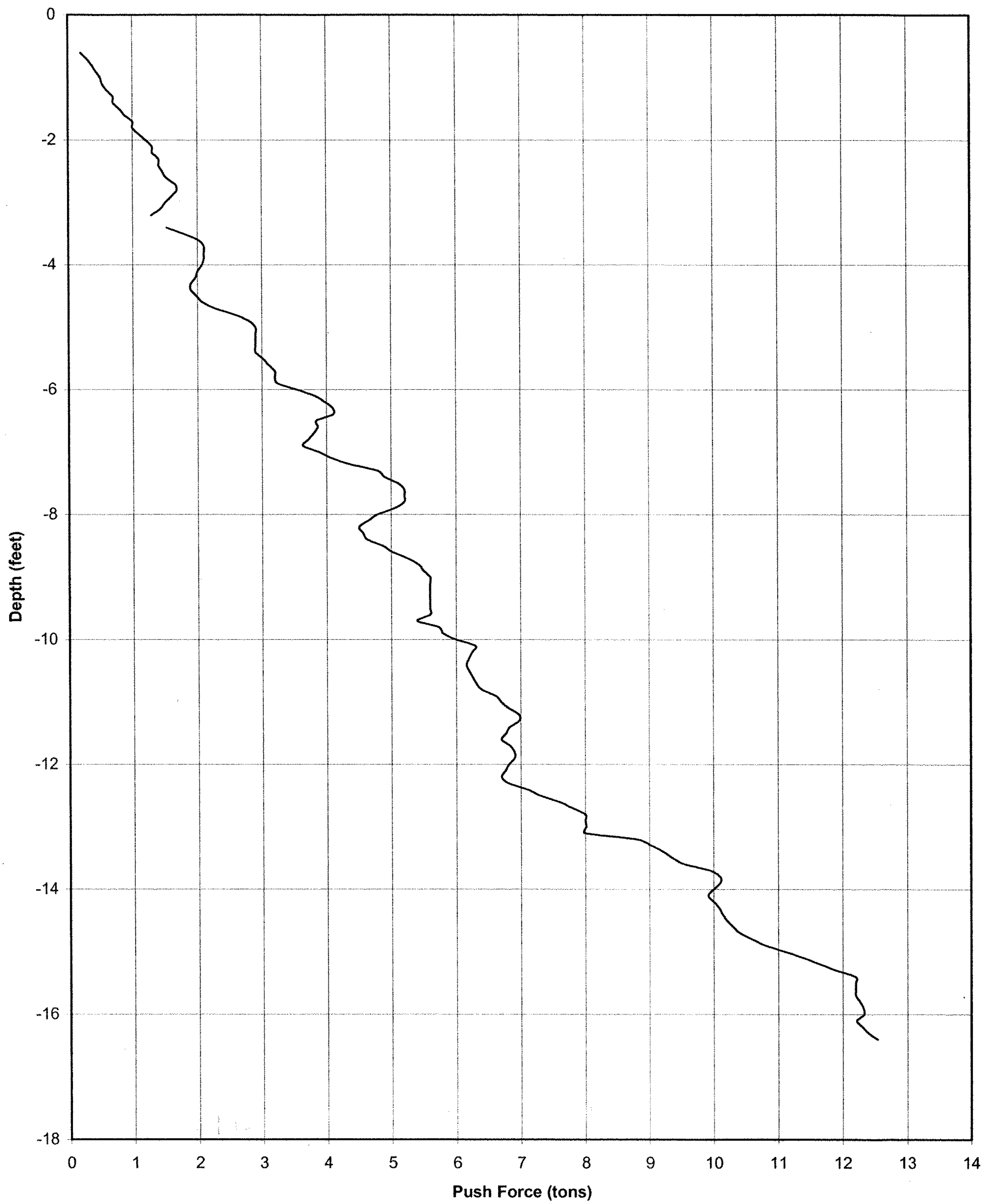

Topeka Flood Protection - Oakland Unit
SCN vs. Depth Sta 91+00
BOTTOM OF BLANKET
Depth (feet)
SCN Number
Peats
Clays
Silt Mixtures
Sand Mixtures
Sands
Sands and Gravels
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-1
-0.5
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5


Topeka Flood Protection - Oakland Unit
Sleeve Resistance vs. Depth Sta 91+00
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
Depth (feet)
0
0.2
0.4
0.6
0.8
1
1.2
Sleeve Resistance (tons/sf)
TOP OF PILE
5d ≈ 4'
(USUALLY IGNORED)
8d ≈ 6'
$f_s \approx 0.25$ tsf
$f_s = 0.4$ tsf
PILE TIP AT 25' TO 30'

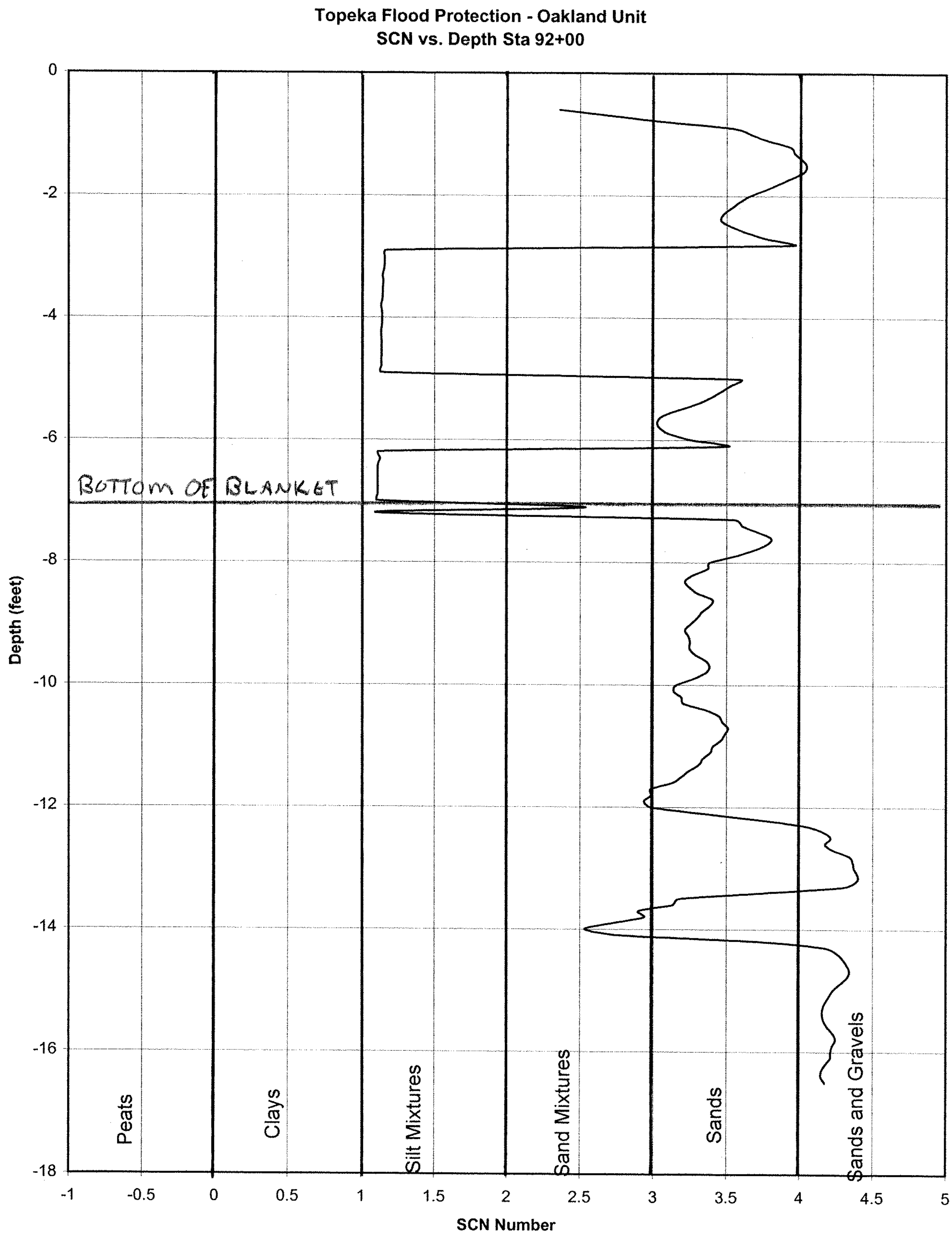
Topeka Flood Protection - Oakland Unit
Cone Resistance vs. Depth Sta 91+00

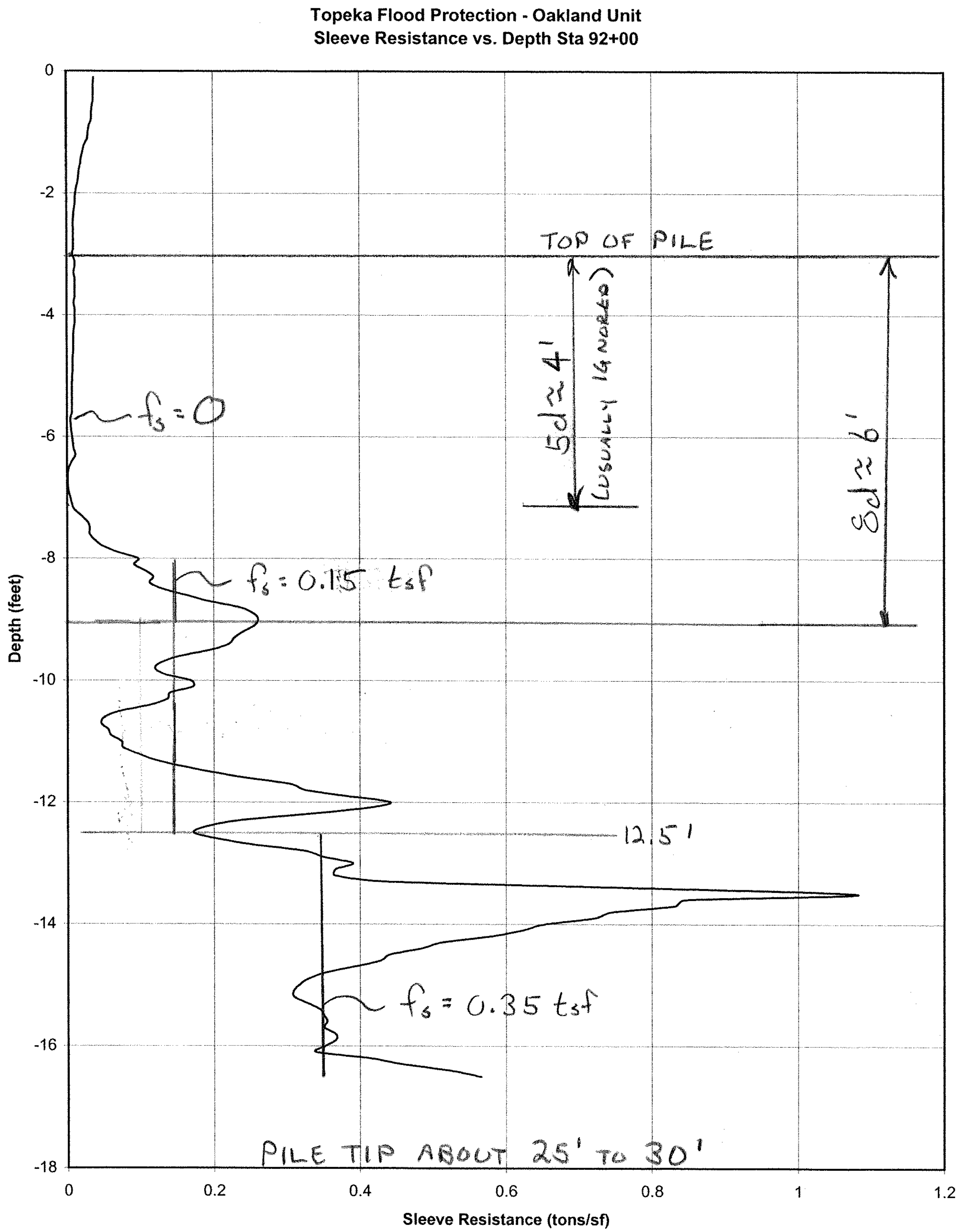


Topeka Flood Protection - Oakland Unit
Push Force vs. Depth Sta 91+00

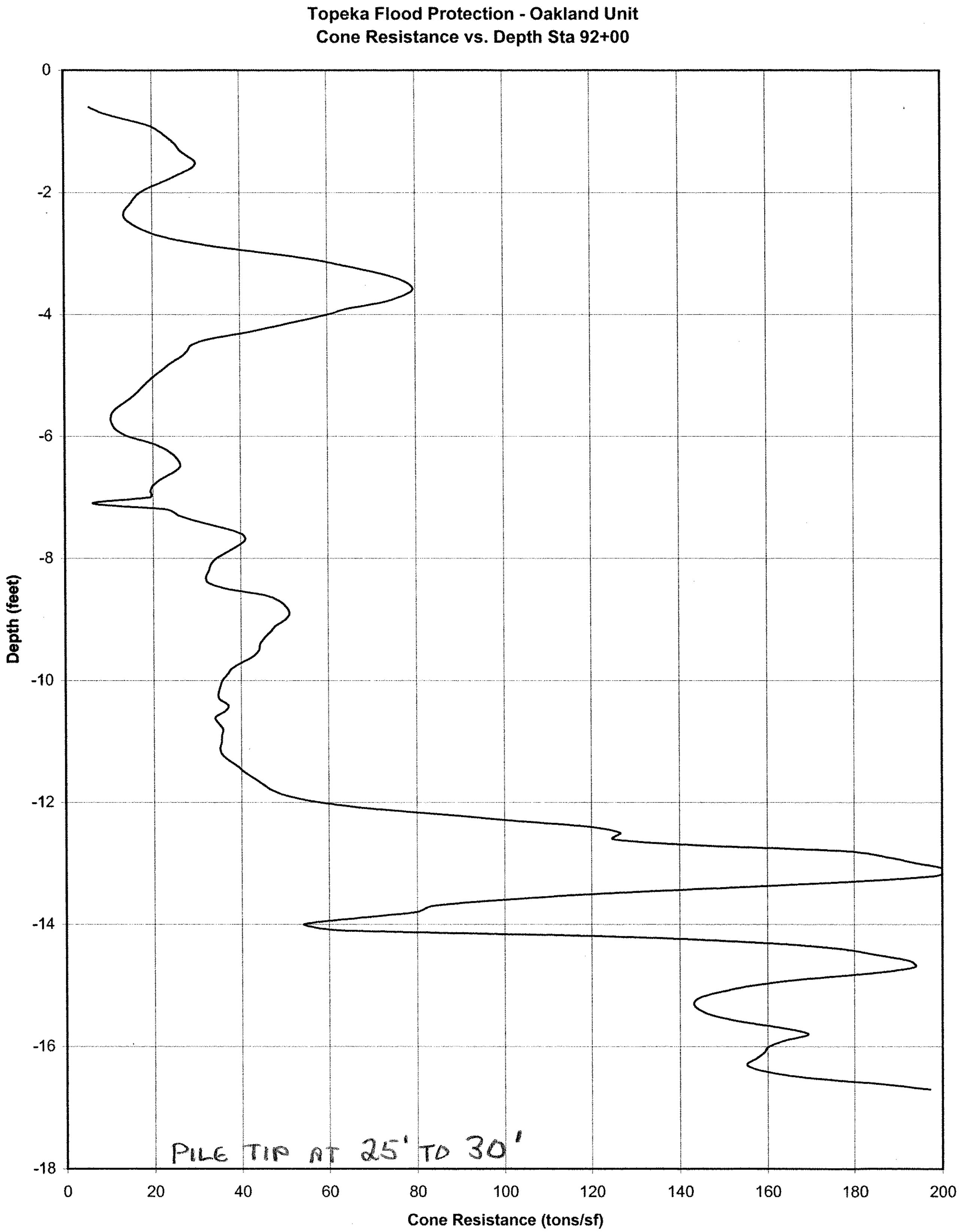


OAK 7 CITY

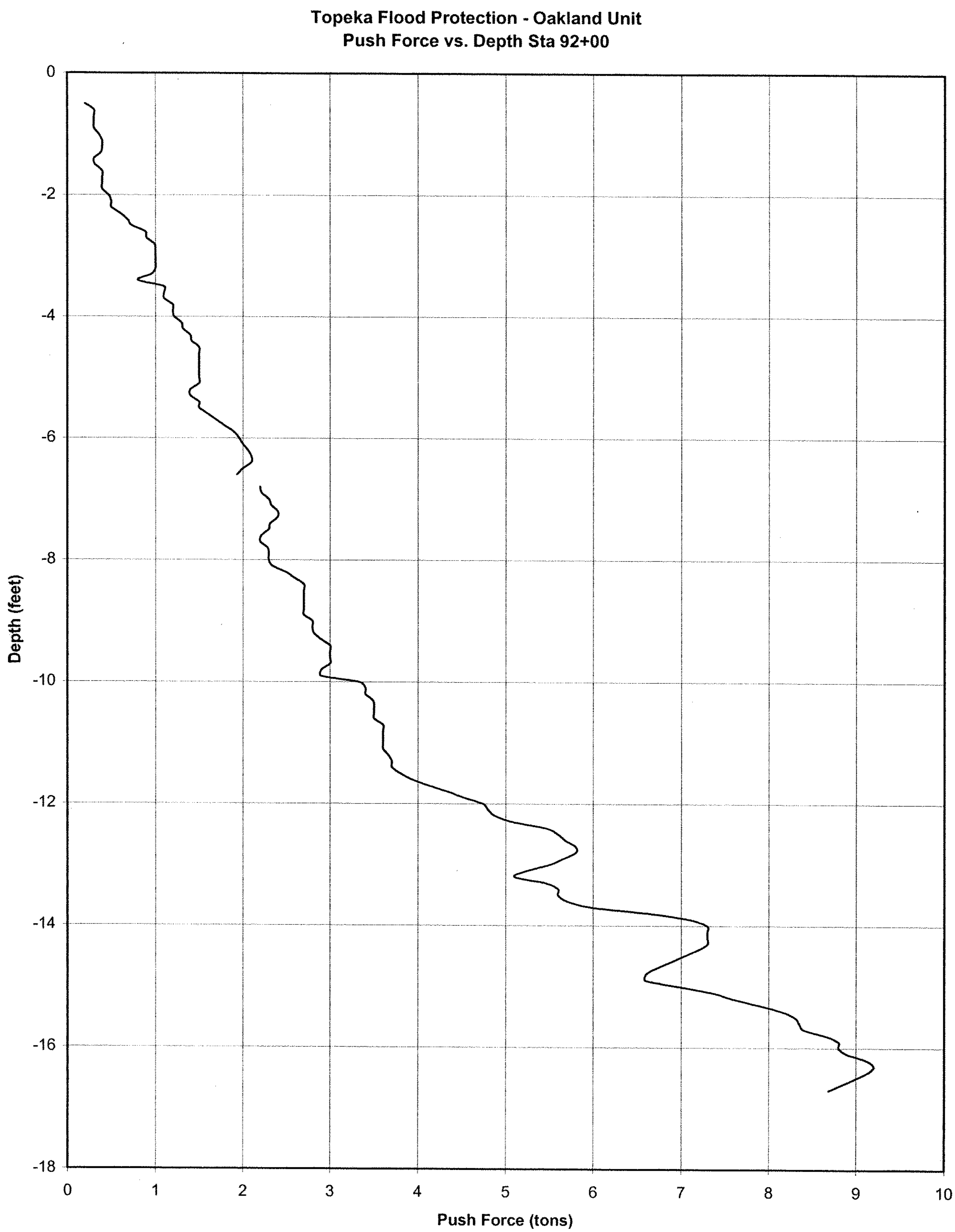

Topeka Flood Protection - Oakland Unit
SCN vs. Depth Sta 92+00
Depth (feet)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
BOTTOM OF BLANKET
Peats
Clays
Silt Mixtures
Sand Mixtures
Sands
Sands and Gravels
-1
-0.5
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
SCN Number



Topeka Flood Protection - Oakland Unit
Sleeve Resistance vs. Depth Sta 92+00
Depth (feet)
Sleeve Resistance (tons/sf)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
0
0.2
0.4
0.6
0.8
1
1.2
TOP OF PILE
5d ≈ 4'
(USUALLY IGNORED)
8d ≈ 6'
$f_s = 0$
$f_s = 0.15$ tsf
12.5'
$f_s = 0.35$ tsf
PILE TIP ABOUT 25' TO 30'

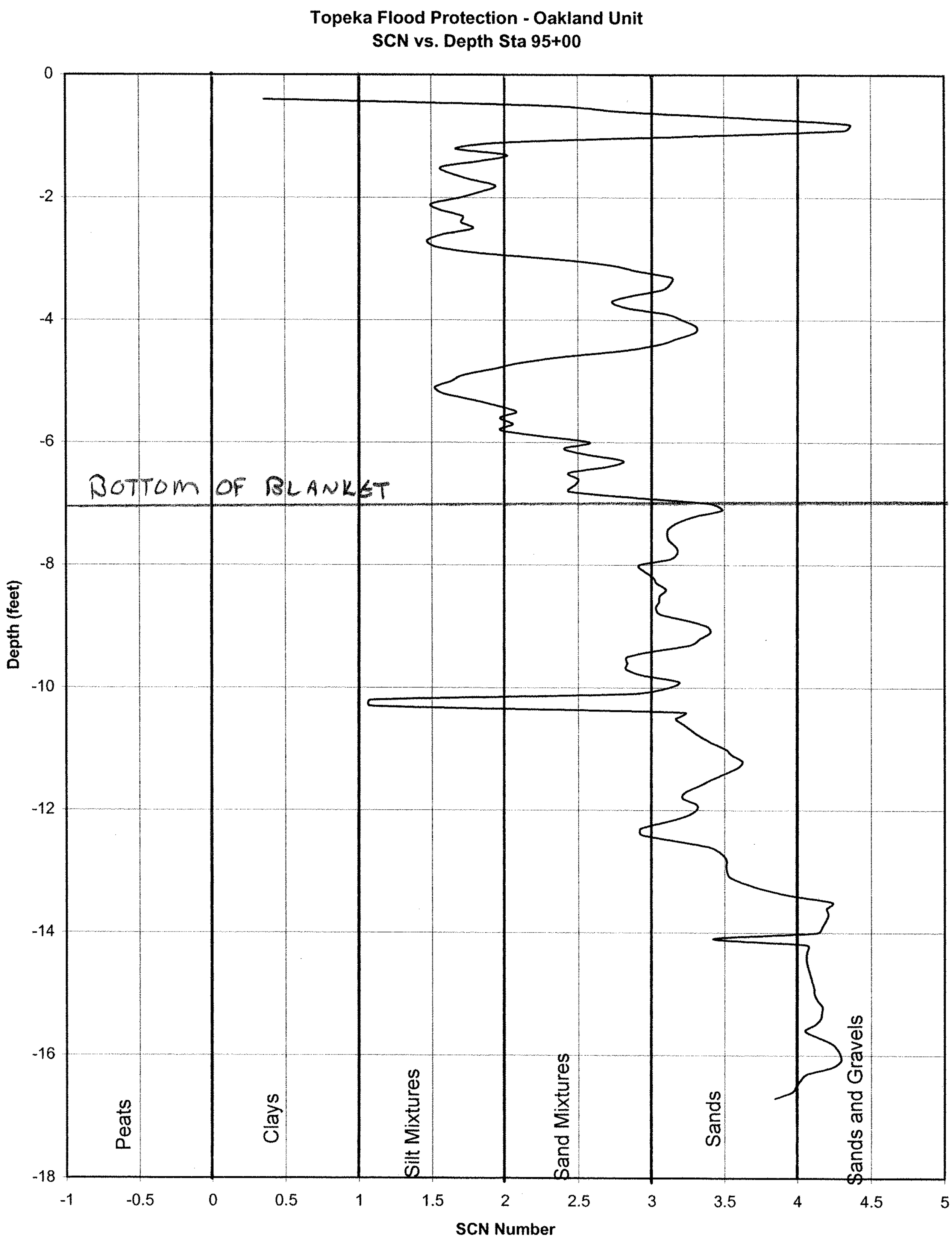


Topeka Flood Protection - Oakland Unit
Cone Resistance vs. Depth Sta 92+00
Depth (feet)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
0
20
40
60
80
100
120
140
160
180
200
Cone Resistance (tons/sf)
PILE TIP AT 25' TO 30'



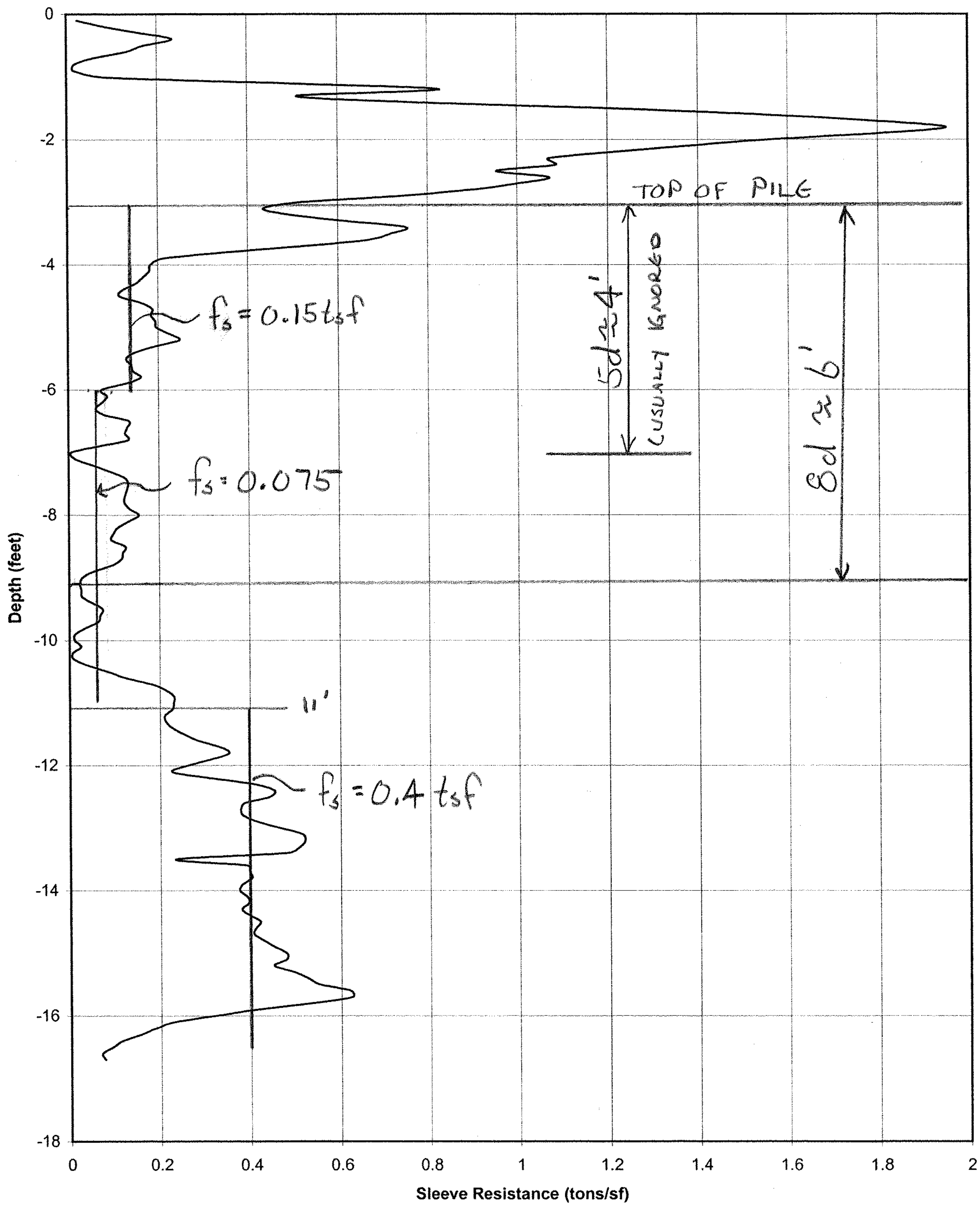
Topeka Flood Protection - Oakland Unit
Push Force vs. Depth Sta 92+00
Depth (feet)
Push Force (tons)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
0
1
2
3
4
5
6
7
8
9
10

OAKIHAL

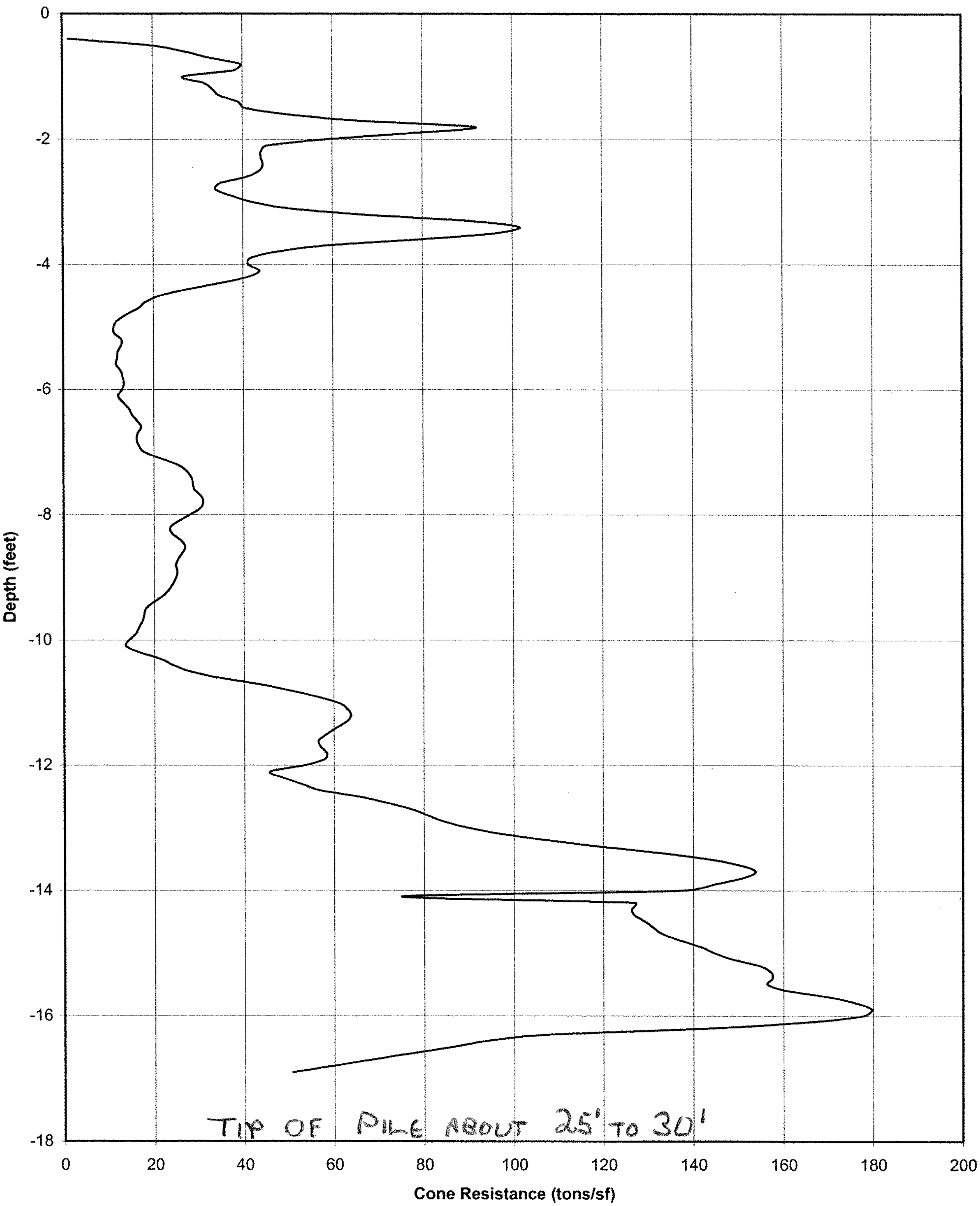


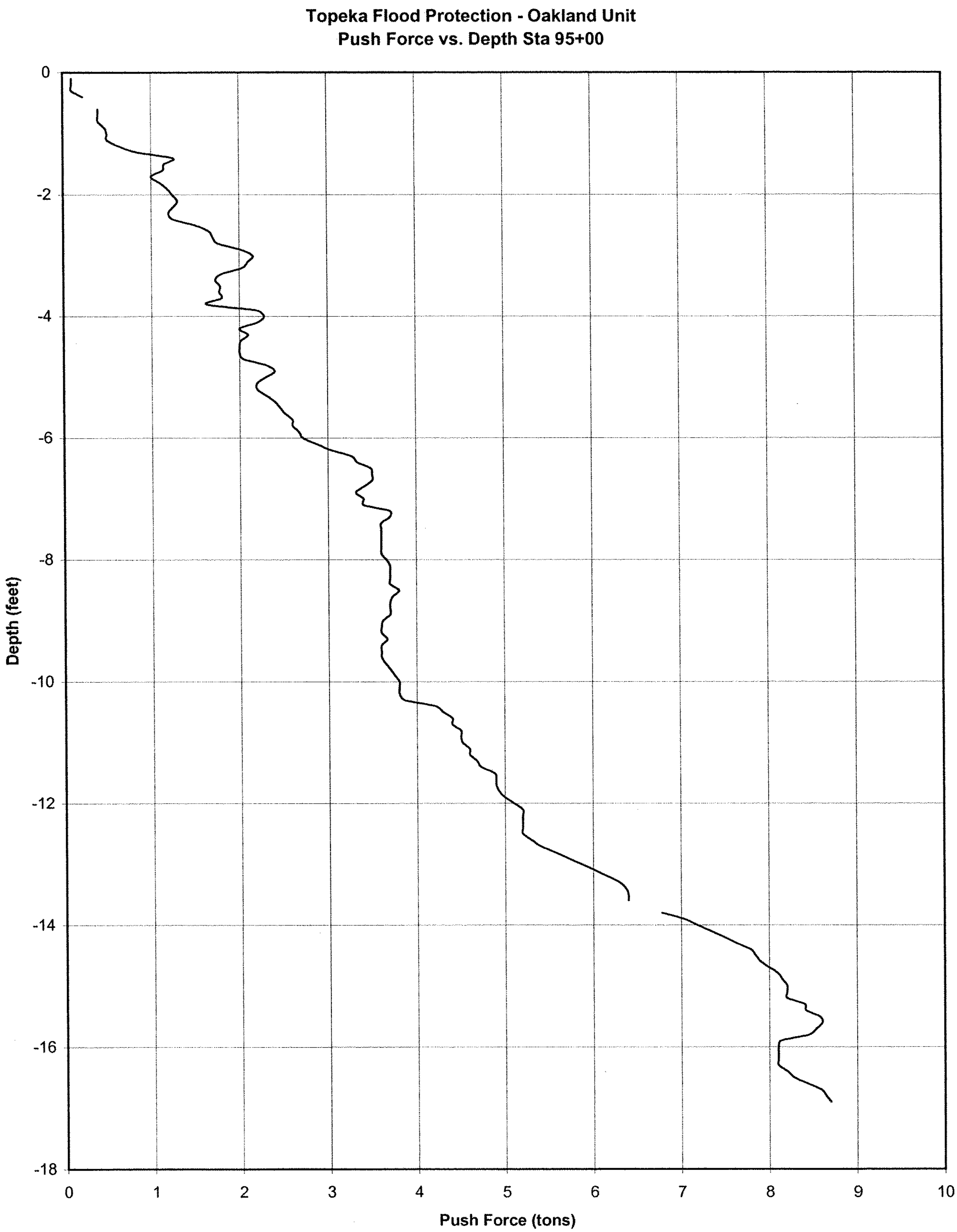
Topeka Flood Protection - Oakland Unit
SCN vs. Depth Sta 95+00
Depth (feet)
SCN Number
BOTTOM OF BLANKET
Peats
Clays
Silt Mixtures
Sand Mixtures
Sands
Sands and Gravels
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-1
-0.5
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5

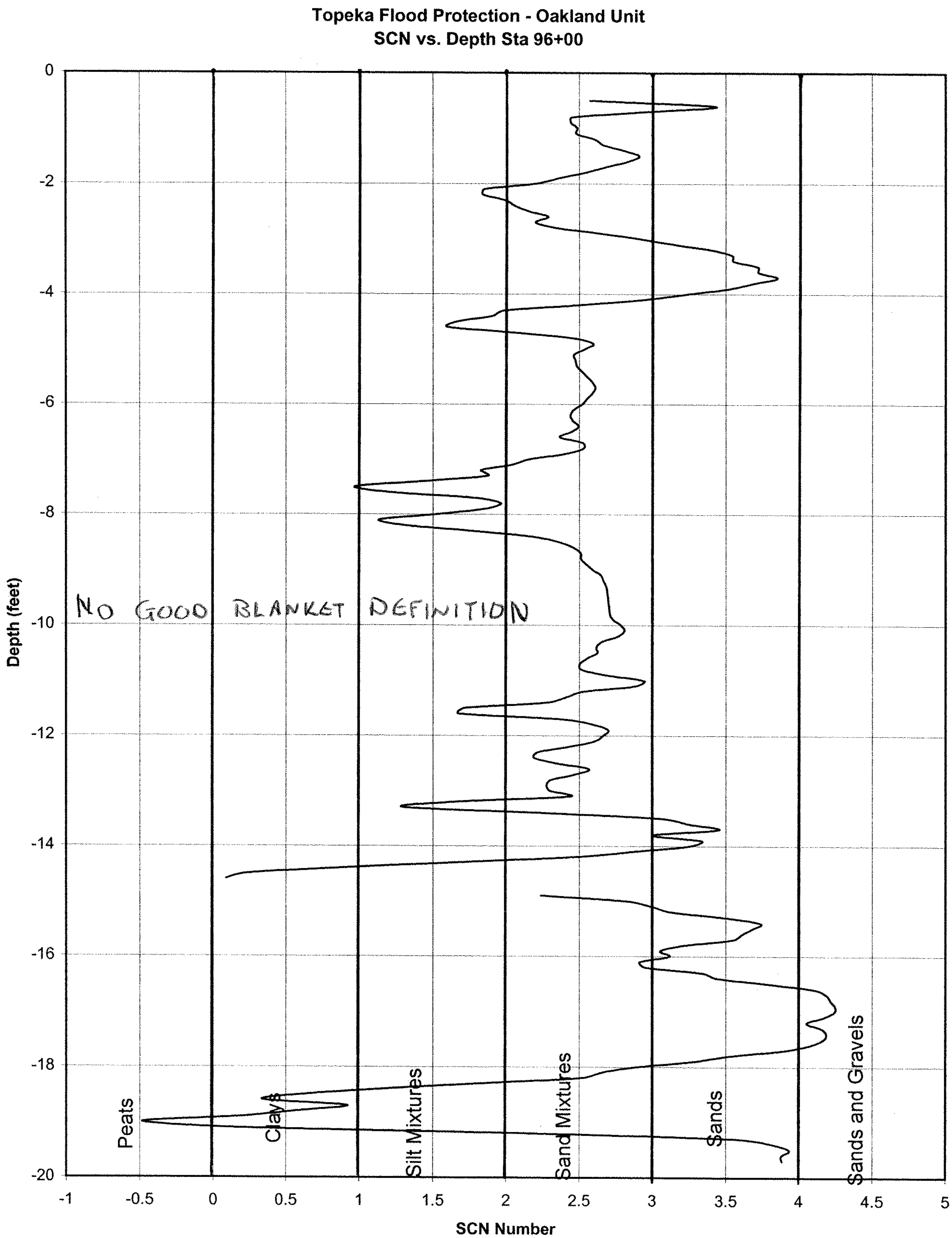
Topeka Flood Protection - Oakland Unit
Sleeve Resistance vs. Depth Sta 95+00

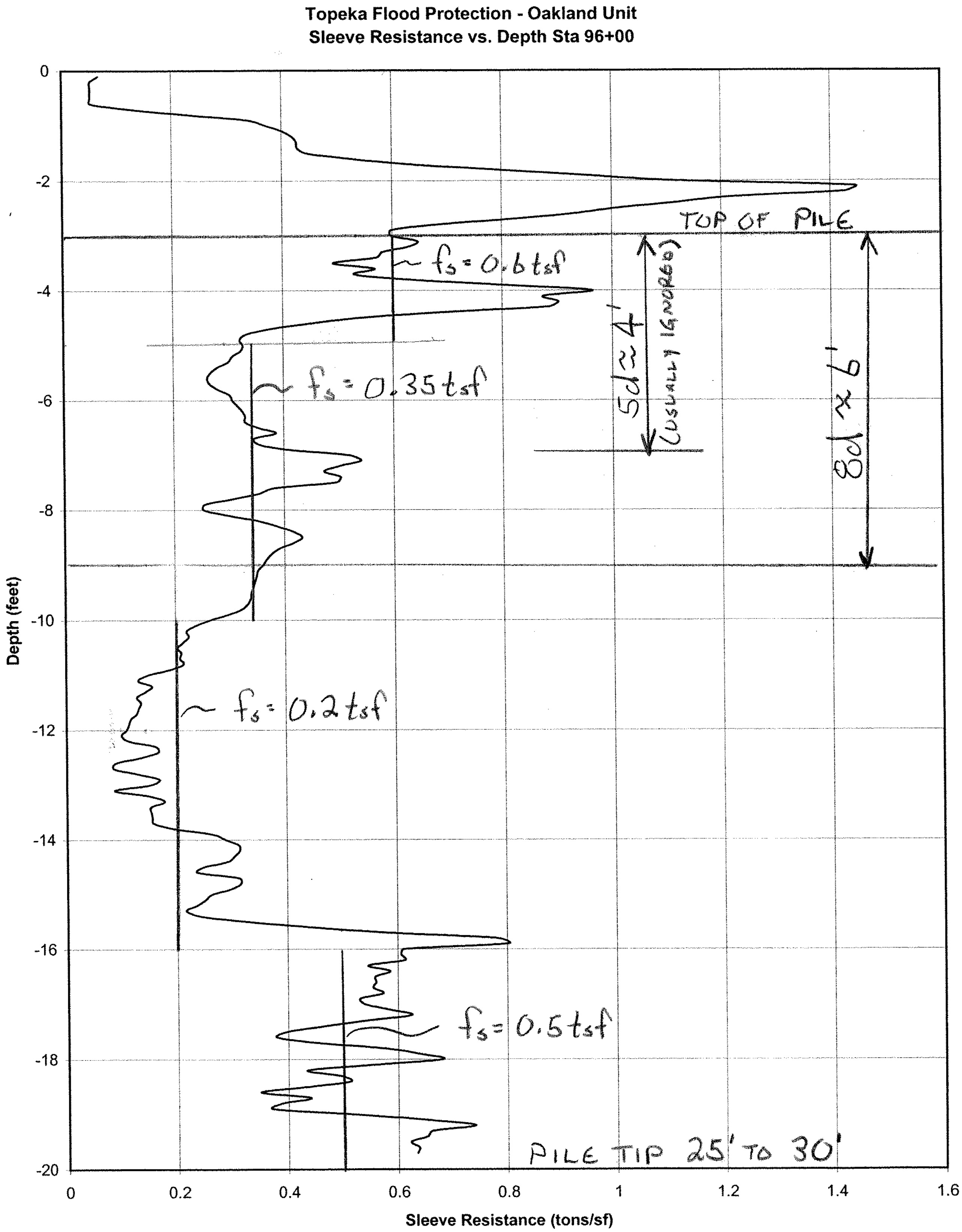


Topeka Flood Protection - Oakland Unit
Cone Resistance vs. Depth Sta 95+00



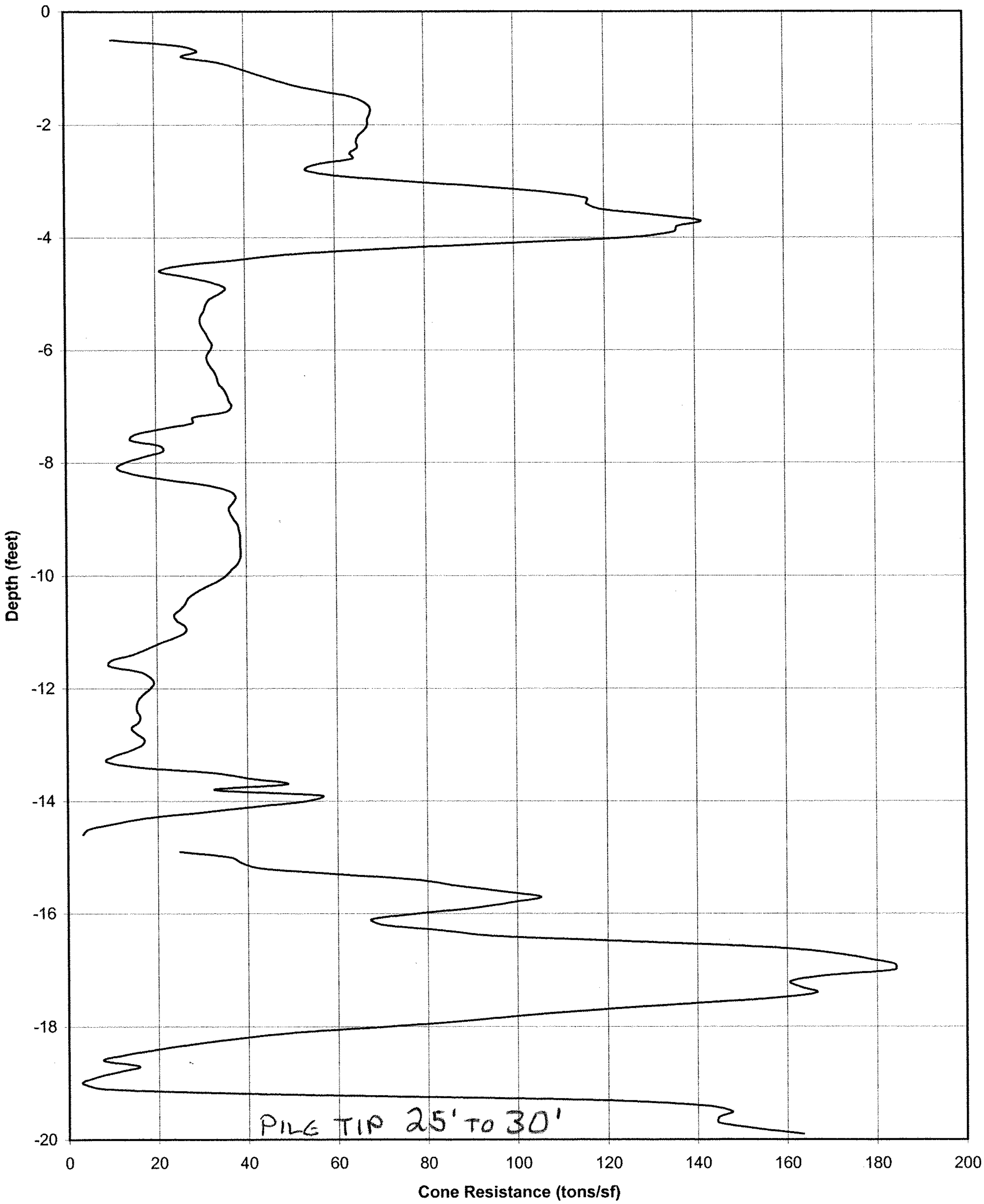

Topeka Flood Protection - Oakland Unit
Push Force vs. Depth Sta 95+00
Depth (feet)
Push Force (tons)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
0
1
2
3
4
5
6
7
8
9
10

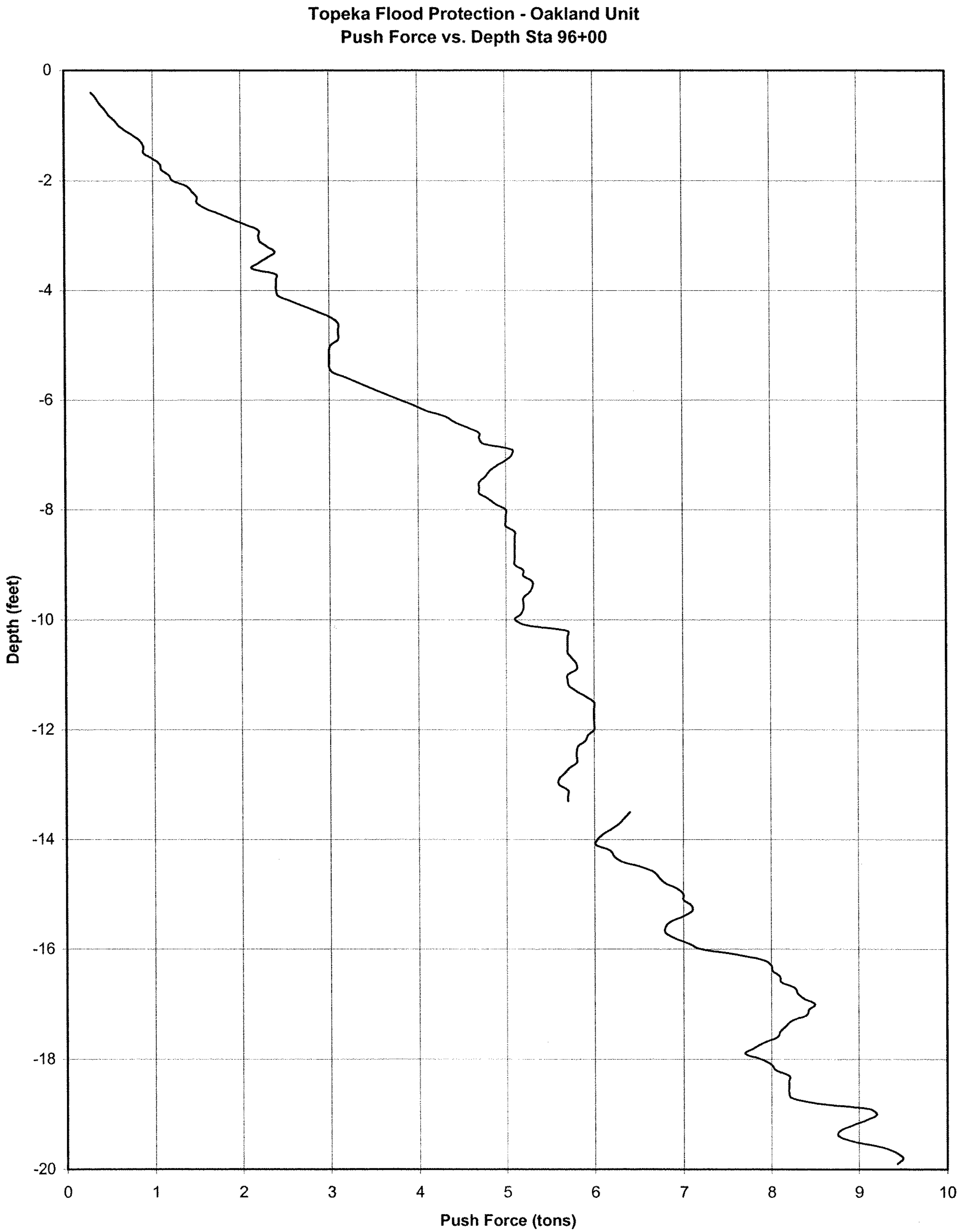

Topeka Flood Protection - Oakland Unit
SCN vs. Depth Sta 96+00
Depth (feet)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
-1
-0.5
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
SCN Number
Peats
Clays
Silt Mixtures
Sand Mixtures
Sands
Sands and Gravels
NO GOOD BLANKET DEFINITION



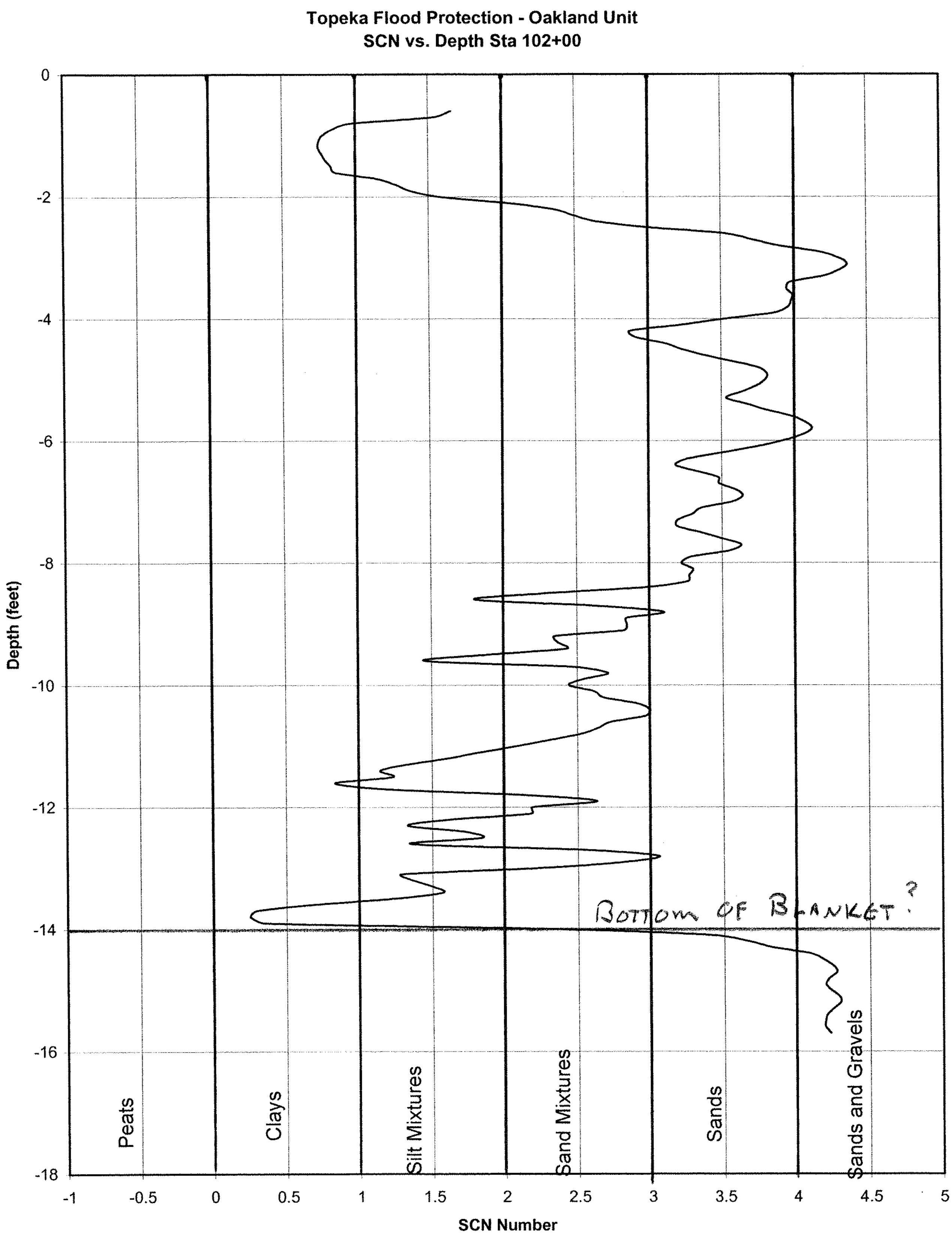
Topeka Flood Protection - Oakland Unit
Sleeve Resistance vs. Depth Sta 96+00
Depth (feet)
Sleeve Resistance (tons/sf)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
0
0.2
0.4
0.6
0.8
1
1.2
1.4
1.6
TOP OF PILE
$f_s = 0.6 tsf$
$f_s = 0.35 tsf$
$f_s = 0.2 tsf$
$f_s = 0.5 tsf$
$5d \approx 4'$
(USUALLY IGNORED)
$8d \approx 6'$
PILE TIP 25' TO 30'

Topeka Flood Protection - Oakland Unit
Cone Resistance vs. Depth Sta 96+00

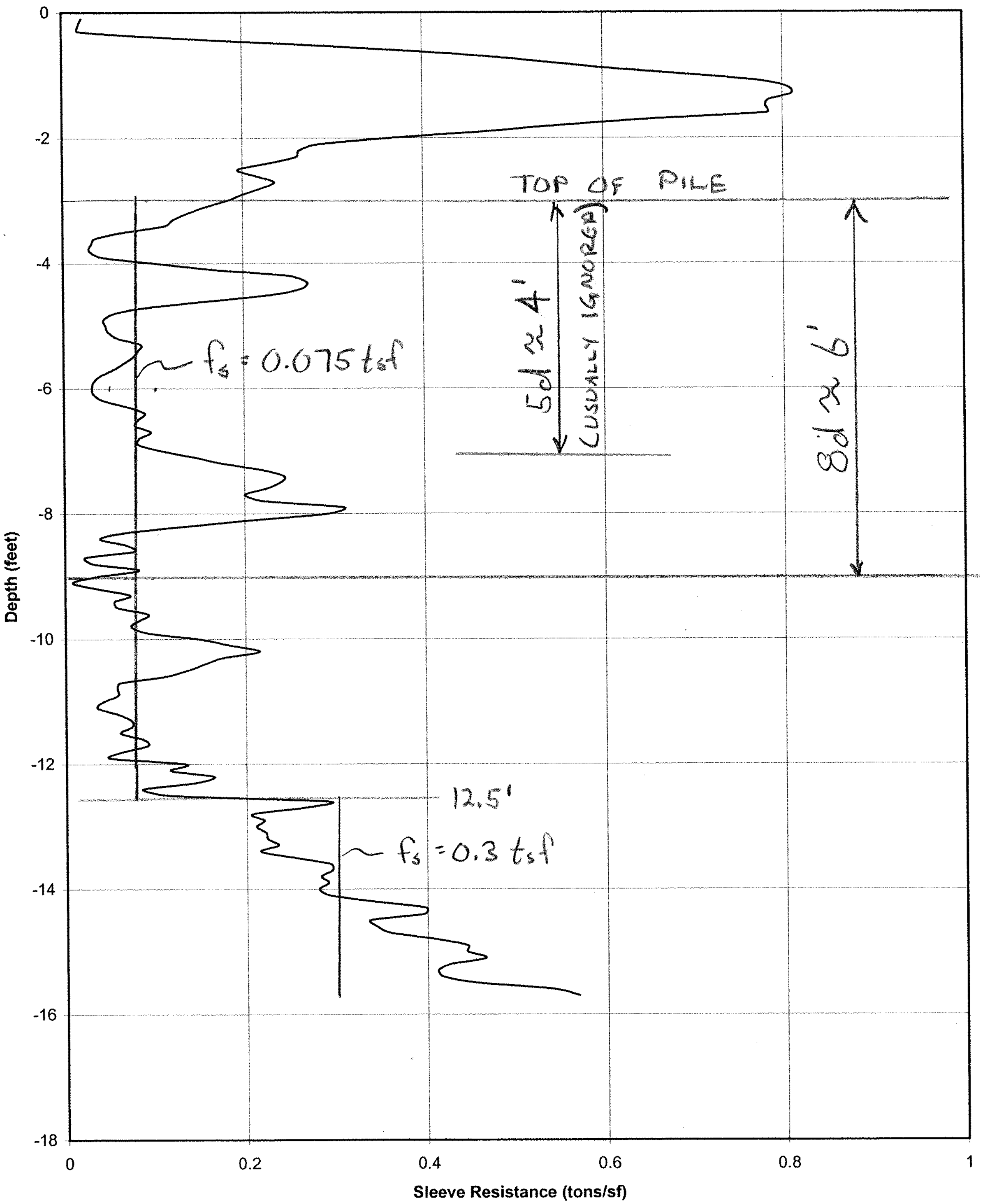


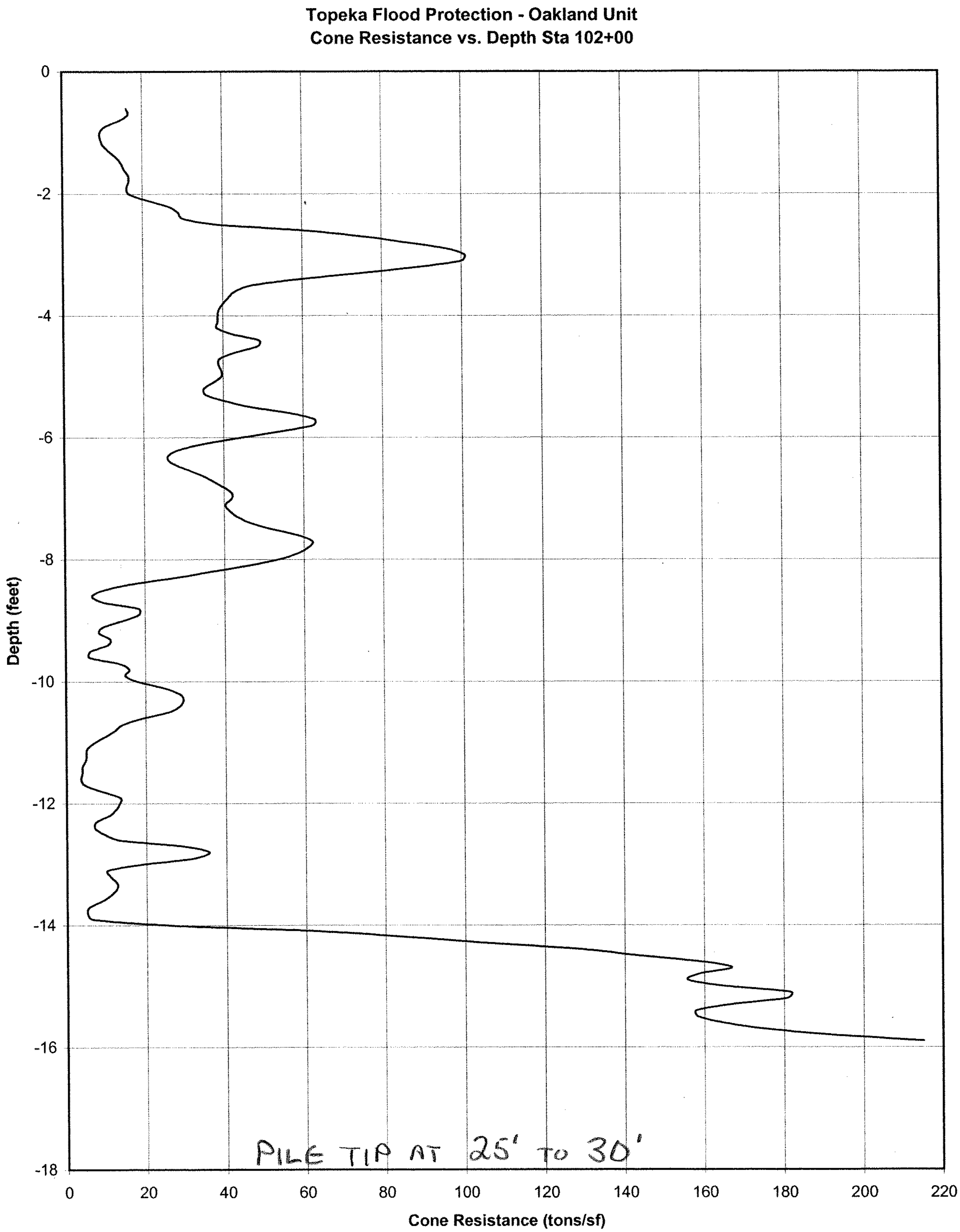

Topeka Flood Protection - Oakland Unit
Push Force vs. Depth Sta 96+00
Depth (feet)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-20
0
1
2
3
4
5
6
7
8
9
10
Push Force (tons)

42

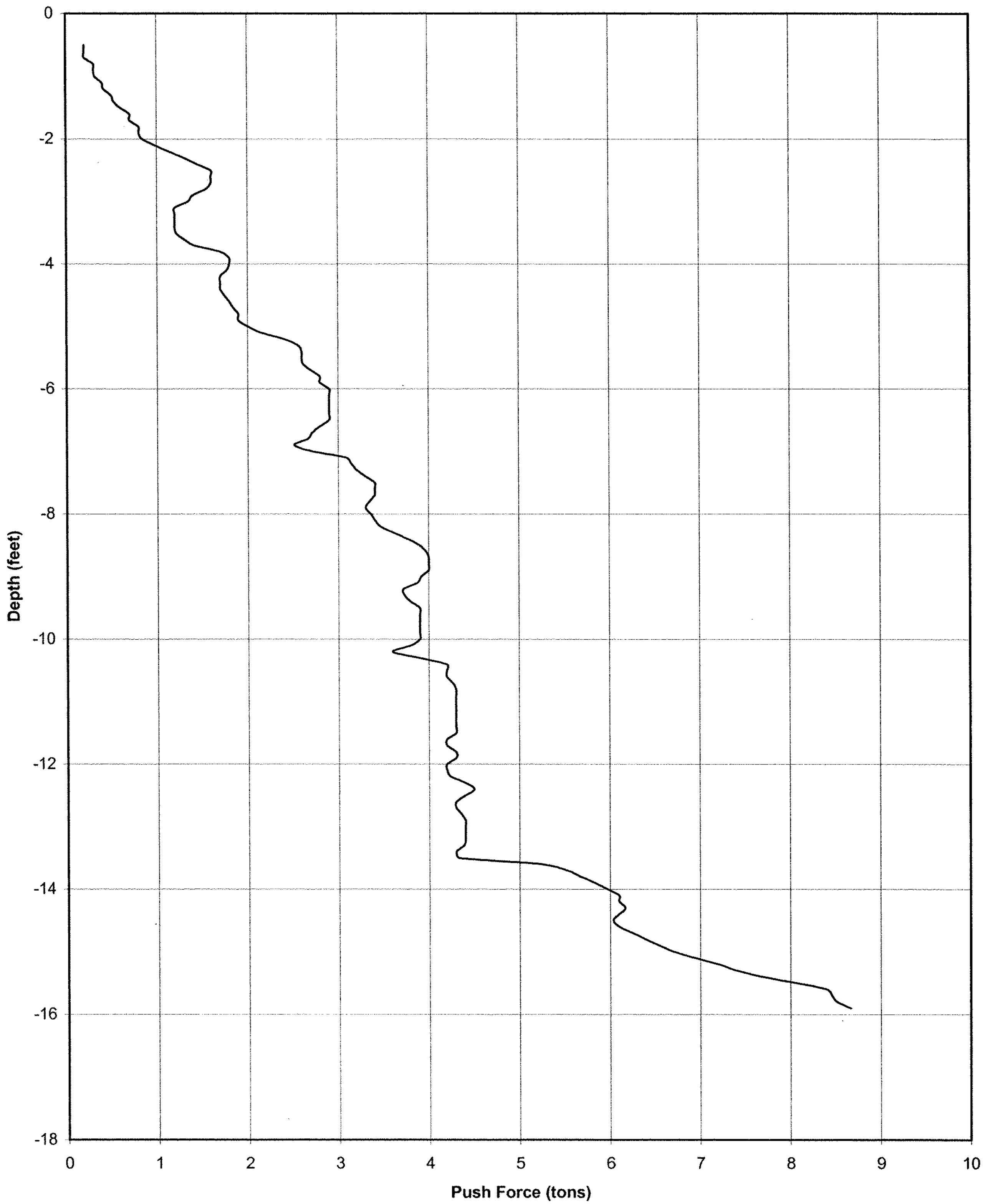

Topeka Flood Protection - Oakland Unit
SCN vs. Depth Sta 102+00
Depth (feet)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
SCN Number
-1
-0.5
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Peats
Clays
Silt Mixtures
Sand Mixtures
Sands
Sands and Gravels
BOTTOM OF BLANKET ?

Topeka Flood Protection - Oakland Unit
Sleeve Resistance vs. Depth Sta 102+00

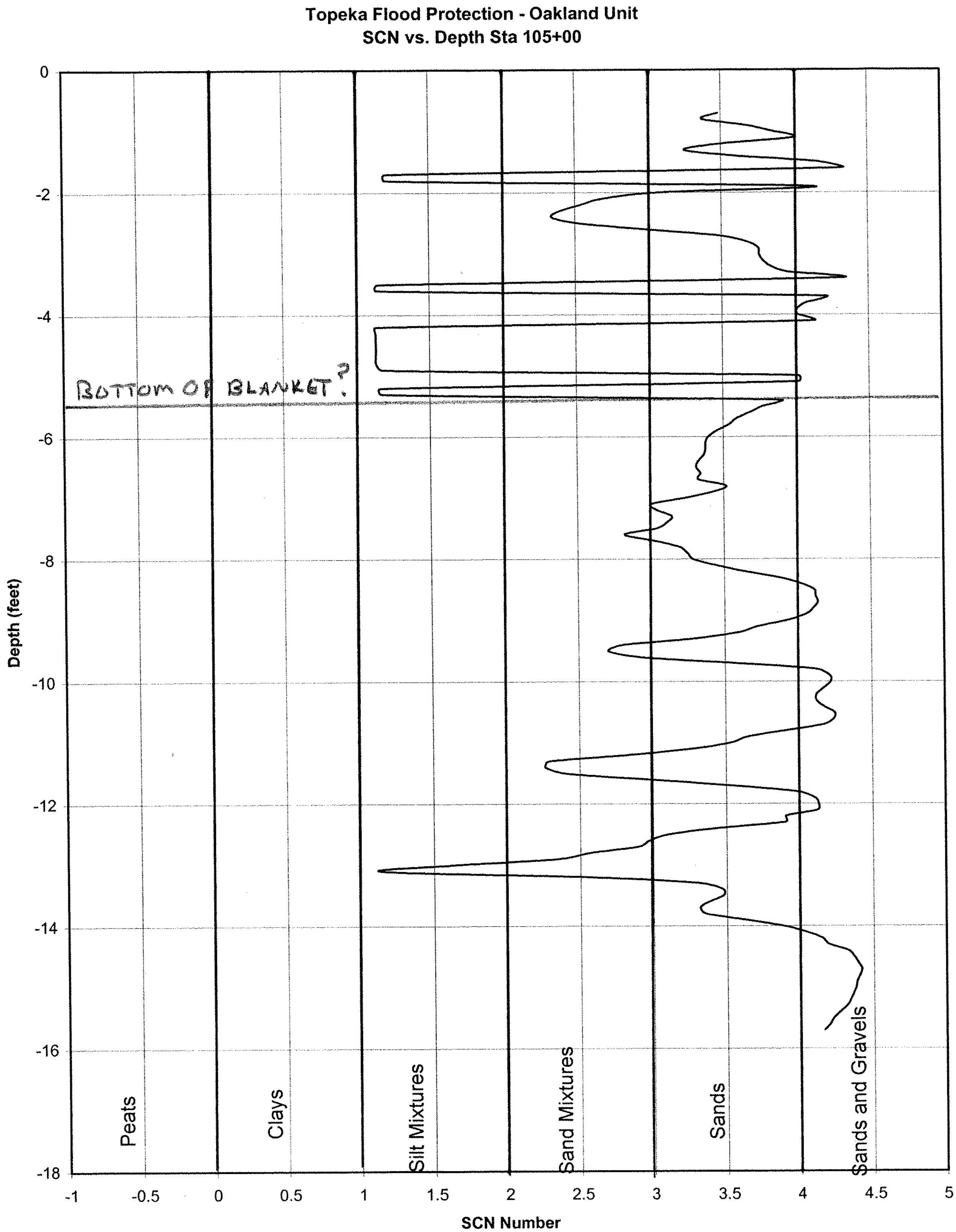



Topeka Flood Protection - Oakland Unit
Cone Resistance vs. Depth Sta 102+00
Depth (feet)
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
0
20
40
60
80
100
120
140
160
180
200
220
Cone Resistance (tons/sf)
PILE TIP AT 25' TO 30'

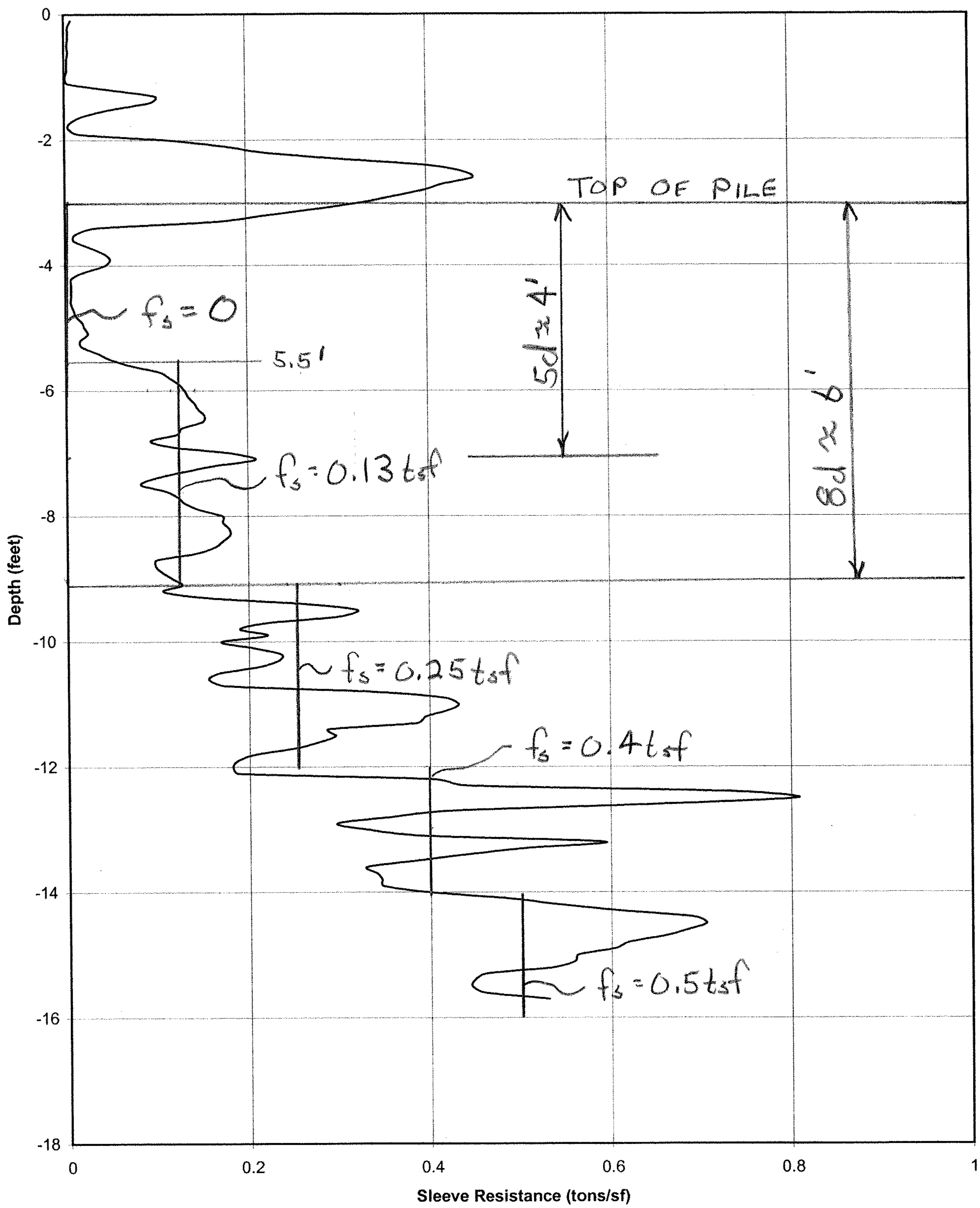
Topeka Flood Protection - Oakland Unit
Push Force vs. Depth Sta 102+00



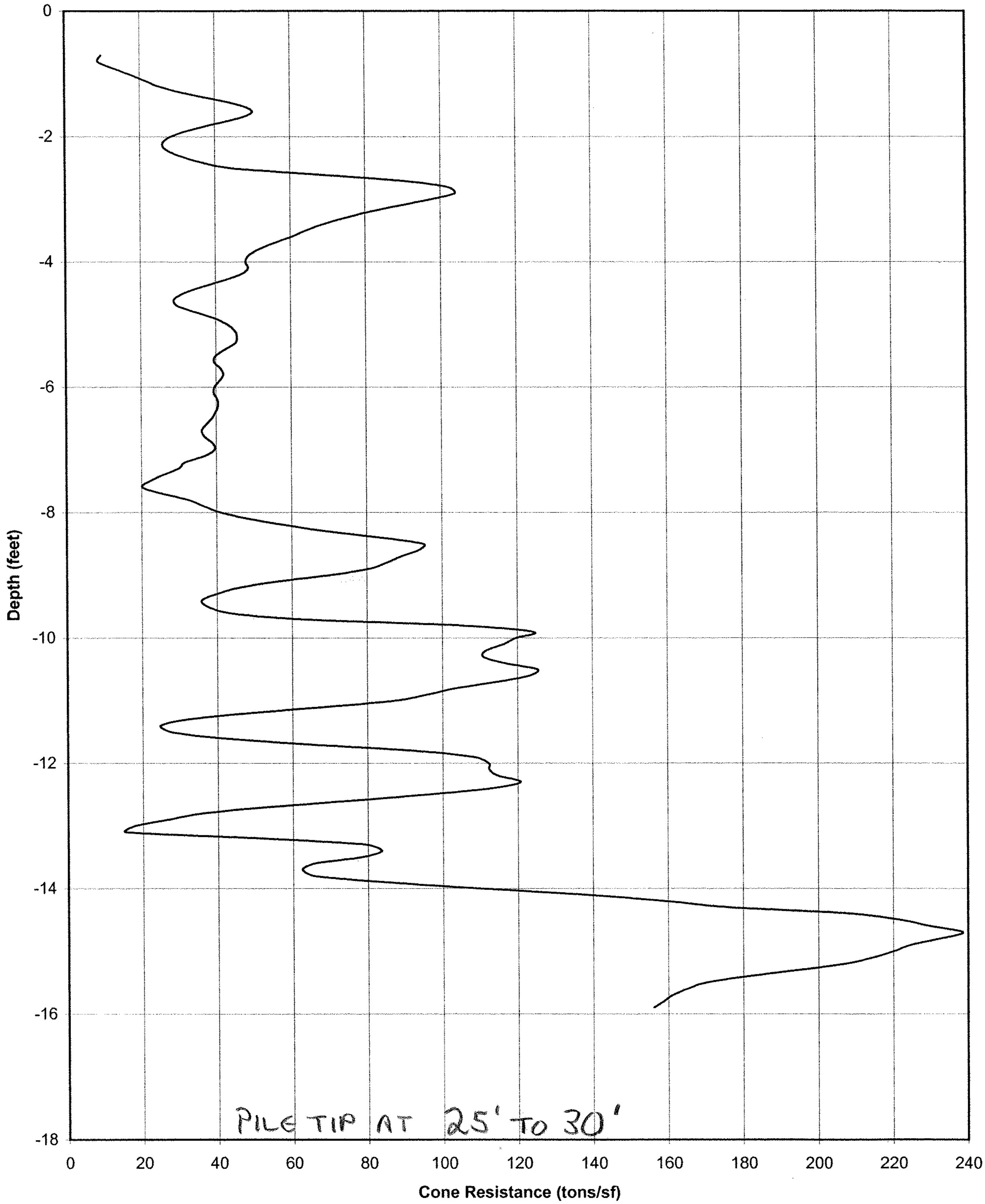
46


Topeka Flood Protection - Oakland Unit
SCN vs. Depth Sta 105+00
Depth (feet)
SCN Number
BOTTOM OF BLANKET?
Peats
Clays
Silt Mixtures
Sand Mixtures
Sands
Sands and Gravels
0
-2
-4
-6
-8
-10
-12
-14
-16
-18
-1
-0.5
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5

Topeka Flood Protection - Oakland Unit
Sleeve Resistance vs. Depth Sta 105+00



Topeka Flood Protection - Oakland Unit
Cone Resistance vs. Depth Sta 105+00



Topeka Flood Protection - Oakland Unit
Push Force vs. Depth Sta 105+00