Kansas Citys, Missouri and Kansas
Flood Risk Management Feasibility Study

Engineering Appendix to the Final Feasibility Report

Chapter A-4

GEOTECHNICAL ANALYSIS

CHAPTER A-4
GEOTECHNICAL ANALYSIS

A-4.1 INTRODUCTION

This Appendix presents the results of the geotechnical evaluation performed for the Armourdale Unit in the Kaw Valley Drainage District. The evaluation started with a thorough review of existing project documentation, defining existing subsurface conditions along the entire unit based upon existing subsurface information, and estimation of soil parameters for the existing levees, the natural blanket and the aquifer materials. The estimated soil parameters are based on geotechnical laboratory testing data from Design Memorandum No. 3. All elevations used in the geotechnical portion of the feasibility study are NGVD 29.

Geotechnical analysis of the unit consisted mainly of underseepage and stability calculations for the following loading conditions:

- Existing Conditions, to identify the most critical areas with respect to risk of failure for use in the HEC-FDA economic model.
- Proposed Design Conditions, all of which included raising the current level of protection as follows:
    - Nominal 500 year flood event, i.e. a 0.2% chance of occurrence in any one year
    - Nominal 500 year flood event plus 3 feet
    - Nominal 500 year flood event plus 5 feet

The majority of the design work focused upon the nominal 500 year flood event plus 3 feet, referred to hereafter as the n500+3 flood event. The raise above the current level of protection varied from 3.8 feet to 5.2 feet, except for a short section near the confluence with the Missouri where the raise varied approximately about 1.2 to 1.8 feet.

Underseepage was addressed along the entire Armourdale unit. Where calculations showed hydraulic gradients in the natural blanket did not meet current criteria, for the N500+3 design condition seepage control measures were designed to reduce the gradient to meet criteria.

Slope stability analyses were performed for three different levee sections: 1) An earthen levee raise with fill on the river side of the protection, 2) An earthen levee raise with fill on the land side of the protection, and 3) A cantilever retaining wall raise on top of an existing levee. An attempt was made to identify the most critical sections for each type of raise for the analyses. The sections were selected based on the initial height of the protection, the amount of raise proposed, and the pore pressures in the natural blanket based upon the underseepage calculations. Each of the sections analyzed were modified as necessary to obtain the required factor of safety against sliding for both the end of construction case and the steady seepage case. Rapid drawdown was not considered due to the lack of existing strength data required for this analysis.

The results of the analyses are discussed in additional detail in later sections. For the purposes of economic modeling of proposed levee raises, all features which meet current Corps of Engineers criteria are arbitrarily assigned a reliability of 99.8%.

A-4.2 DESCRIPTION OF EXISTING LEVEE UNIT

A-4.2.1 Levee Description

The Armourdale Unit is located in Wyandotte County, Kansas on the left bank of the Kansas River between approximate Kansas River miles 7.0 and 0.25. Mile 0.0 would be at the confluence with the Missouri river. The levee begins at Station 0+00 UE (Upper End) where it ties into high ground across the Kansas City Southern Railroad tracks just downstream of the I-635 bridge, and extends downstream to Station 61+00 LE (Lower End). The unit protects numerous commercial and light industrial properties as well as a significant residential population. The total length of the unit is 34,853.45 feet or 6.6 miles.

The line of protection alignment stationing has been separated into three designations due to several changes made to the alignment throughout the history of the unit. The upper end stationing begins at 0+00 U.E. and ends at 20+08.89 U.E BK (back). A station equation is inserted at this point to change the station to 9+71.16 AH (ahead). The stationing continues to 206+12.43 BK. This location has a station equation that changes to 212+00 AH. The stationing then remains consistent to 257+66.26 BK, where another station equation is inserted to change the station to 257+64.97 AH. The stationing continues to 322+85 BK. At this point a final station equation is inserted which defines the lower end and changes the stationing to 39+71.83 LE. The unit finally ends at station 61+00 LE.

There are many bridges, structures, and utilities within the critical area of the line of protection. For the purposes of the feasibility study, it was assumed that all bridge foundation elements, structures, and utilities within the levee embankment and critical area of the foundation blanket material meet all pertinent Corps of Engineers criteria.

A-4.2.1.1 History

The Kaw Valley Drainage District initially began work on the Armourdale Unit prior to any involvement by the Federal Government. Previous works included the construction of earthen levee sections, drainage structures and even pump plants. The Flood Control Act of 1936 authorized the Corps of Engineers to provide assistance. Work began to improve parts of the project in 1949. The flood of 1951 caused extensive damage to the original levee, and the Corps of Engineers designed and constructed the restoration of the protection. The Corps of Engineers again became involved in the 1960's to raise the level of protection along the Armourdale Unit, reference Design Memorandum No. 3, dated May 1971. The raise was constructed in the early 1970's. The following discussion describes the unit in additional detail by major features.

Station 0+00 UE to 2+90 UE. This is a stoplog gap across several sets of railroad tracks, and starts the upper end of the project. Two sets of openings exist across this significant span. This

work was part of an upper end extension that was a modification to the original 1962 Modification as outlined in Design Memorandum No. 3.

Station 2+90 UE to 60+40. This section of the line of protection is an earth fill levee section.

Station 60+40 to 77+77.5. This section is a floodwall that extends across the West Kansas Avenue Bridge abutment. The bridge has since been rebuilt and the original stoplog section at station 62+30 has been replaced with floodwall.

Station 77+77.5 to 226+01. This section of the line of protection is an earth fill levee section. A short retaining wall exists along the landside toe starting at approximately station 212+00 and continuing to 226+10. The retaining wall was constructed to avoid fill placement on an existing railroad track that parallels the levee. The railroad subsequently has been abandoned.

Station 226+01 to 227+46. This is a floodwall and stoplog gap section for the Kansas City Terminal bridge.

Station 227+46 to 246+88. This section of the line of protection is an earth fill levee section. This reach of the line of protection also has a landside retaining wall and parallel railroad track.

Station 246+88 to 250+52. This is a floodwall and stoplog gap section for the East Kansas Avenue bridge.

Station 250+52 to 257+66.26 BK. This section of the line of protection is an earth fill levee section. This reach of the line of protection also has a landside retaining wall and parallel railroad track.

Station 257+64.97 AH to 302+57.65. This section of the line of protection is a floodwall. The wall ties into high ground at a railroad embankment that follows the bluff of the left river bank. The floodwall has stoplog or sandbag gaps for the Missouri Pacific and Union Pacific railroad bridges, as well as for the Central Avenue bridge.

Station 302+57.65 to 61+00 LE. This section is a railroad embankment located at the toe of the bluff of the left river bank.

Based upon the record drawings, the existing levee sections have a thick impervious riverside section and a random fill section on the land side. For the upper reaches of the project from approximately Station 205+00 to 257+00, there is a pervious fill section protected with riprap on the river side slope.

A-4.2.1.2 General Geology of the Region (Kansas River)

The Kansas River Valley, near its mouth, is cut into Pennsylvanian bedrock of the Missourian Series. The oldest bedrock exposed is the Bethany Falls Limestone member of the Swope Limestone formation, Kansas City Group. Bedrock of the Missourian Series is characterized by numerous limestone beds separated by clayey to somewhat sandy shale. The

bedrock is generally overlain by much younger unconsolidated materials consisting of glacial drift, loess of the Pleistocene age, alluvium deposits and isolated remnants of till of Kansas stage ice sheet occurring on the hilltops. The Kansas River is near the southern edge of Kansas glaciation. Wind blown deposits of silt (loess) form an irregular deposit covering much of the eastern part of Wyandotte County. Alluvium, ranging from clay and silt to sand and gravel, occurs in the Kansas River Valley. Much of this alluvium is probably of glacial origin, having been deposited as glacial outwash from the melting ice sheets.

A-4.2.1.3 Subsurface Conditions for Armourdale

Assessments of the subsurface conditions for the Armordale project were derived from the Record Drawings, Design Memorandums and borings made at selected sites during the feasibility study. Typical subsurface blanket conditions for Station 0+00 UE to Station 190+00 generally consist of silts, sandy clays and lean clays of average thickness ranging from 13 feet to 40 feet. Beyond Station 190+00 to the Lower End of Armourdale, the foundation blanket has multiple layers of sand intermixed with clays and silts. The aquifer thickness ranges from 25 feet to 77 feet. Groundwater levels are dependent on the seasonal changes and rises in the river. The subsurface investigation measured the water levels in the borings after allowing for disturbances due to drilling to stabilize. The water levels are shown on drill logs and recorded on the strip log summary. In general the water levels measured adjacent to the existing level of protection were on average at least 15 feet below the landside ground surface for normal river levels.

A-4.2.2 Existing Underseepage Control Features

Throughout the existence of the Armourdale Levee Unit, many underseepage control measures have been constructed to aid in the prevention of developing an underseepage condition that could cause a levee failure. Underseepage control measures were designed and constructed during the restoration of the levee unit after the 1951 Flood, and during the 1962 Modification of the unit.

The underseepage control feature designed and constructed during the restoration after the 1951 Flood was an extensive impervious fill on the river side of the levee to prevent seepage through sand lenses in the stratified natural blanket. The impervious fill “cutoffs” are extensions of the impervious fill section in the levee embankment. The impervious fill cutoff is a minimum of 5 feet in thickness measured normal to the slope. The impervious fill is protected from erosion and scour by stone riprap protection. The impervious fill was extended to varying elevations sufficient to ensure full cut off of sand lenses in the natural blanket. Typical cross sections, revised for the “As-Built” conditions can be found in Armourdale Unit Record Drawings Volume 1, O&M Plate No. 75-78, 126-128, 149-150, and 101-102. The extents of the riverside impervious fills are summarized in Table A-4.1:

Table A-4.1: Extents of the Riverside Impervious Fills

Beginning Station	Ending Station	Low Elevation of Riverside Impervious CutOff
65+00	69+00	740
69+00	85+00	735
85+00	90+75	732
90+75	131+00	735
131+00	193+50	740
193+50	195+00	735
195+00	199+00	740

Underseepage control features designed and constructed during the 1962 Modification are detailed below. Additional details can be found in Design Memorandum No. 3 – Armourdale Unit.

Station 78+50 to 94+00 – An aerial fill was constructed in a low lying area landward of the levee. The fill was designed and constructed as an underseepage berm. The aerial fill was constructed to elevation 760.0 and extends up to 200 feet landward of the levee centerline. The downstream limit of the aerial fill tied into the 18th Street roadway embankment. The aerial fill was designed to provide a factor of safety with respect to hydraulic gradient of 1.5 at the landside levee toe, and 1.1 at the berm toe, with the water at approximately 3 feet below the levee crest. Details on the aerial fill can be found in the Armourdale Unit Record Drawings Volume 2, O&M Plate No. 169-170.

Station 190+00 to 248+00 – A relief well system, consisting of 24 fully penetrating artesian relief wells, was installed to remediate a series of underseepage concerns mostly related to existing building foundations. The relief well system was designed to provide a factor of safety with respect to hydraulic gradient of 1.5 at all check points, and 1.0 in basements or pits, with the water at the top of the levee. The wells are variably spaced and connected by a gravity header system which discharges into the Shawnee Avenue Pump Station. The relief well header system and pump station were designed to handle a maximum flow from the relief well system of 32 cfs, with the flow from each well assumed to be 1.33 cfs. An aerial fill was constructed in a low lying area between Stations 220+00 and 226+50 to supplement the relief well system. The aerial fill was constructed to elevation 749.0 and extends up to 300 feet landward of the levee centerline. Details on the relief well system can be found in the Armourdale Unit Record Drawings Volume 2, O&M Plate No. 175-179, 197-200, 201, and 204.

Station 274+00 to 283+00 – A relief well system, consisting of 8 fully penetrating artesian relief wells, was installed to protect a low lying railroad bed (which is now abandoned) directly adjacent to the landside of the existing floodwall. This area is commonly referred to as the "slot" area, and has an elevation up to approximately 15 feet below the surrounding ground. The system originally consisted of 14 wells, but portions of the old railroad bed have been filled and 6 of the wells have since been abandoned. The relief wells discharge into manholes that discharge through lateral pipes directly into the "slot" area. The relief wells serve two purposes: 1) pressure relief at the base of the blanket, and 2) the discharge ponds in the slot and further

reduces the gradient through the blanket. The relief well system was designed to provide a factor of safety of 1.0 with 5 feet of water ponded in the slot. The well flows were assumed to be between 1.75 and 2.0 cfs. Details on the relief well system can be found in the Armourdale Unit Record Drawings Volume 2, O&M Plate No. 180-181, 197-200, 205.

Station 295+00 to 305+00 – A relief well system, consisting of 7 fully penetrating artesian relief wells, was installed to protect a large low lying area which contained a packing plant (which is no longer present) approximately 100 feet from the landside toe of the floodwall. The relief well system was designed to provide a factor of safety with respect to hydraulic gradient of 1.5 at all check points, and 1.0 in the basement of the packing plant, with the water at the top of the levee. The wells are connected by a gravity header system which discharges into the Central Avenue Pump Station. The relief well header system and pump station were designed to handle a maximum flow from the relief well system of 10.5 cfs, with the flow from each well assumed to be 1.5 cfs. Details on the relief well system can be found in the Armourdale Unit Record Drawings Volume 2, O&M Plate No. 182-183, 197-200, 205.

A-4.2.2.1 Overall Underseepage Along Armourdale Unit

For the underseepage analysis, the entire Armourdale Unit was divided into reaches of similar protection height, blanket thickness, blanket composition, aquifier thickness, and seepage entrance conditions. The factor of safety with respect to hydraulic gradient through the natural blanket was calculated for each of these reaches at the landside toe of the levee section or floodwall. **Exhibit 1** shows the calculated factor of safety with respect to hydraulic gradient for the entire Armourdale Levee Unit (without the effects of existing relief wells or cutoff walls), as well as the parameters used to calculate the factor of safety with respect to hydraulic gradient.

A-4.3 SOIL STRENGTH PARAMETERS

The required parameters for soils in the Armourdale reach were estimated mainly from the significant amount of geotechnical laboratory testing performed for the 1962 Modification and provided in Design Memorandum No. 3. A summary of the soil parameters is provided in Table A-4.2 below and discusses in the following paragraphs.

Table A-4.2 - Geotechnical Design Parameters

Material	Unit Weight		Shear Strength			
	Moist	Saturated	Undrained		Drained	
	γ (pcf)	γ (pcf)	c (psf)	ϕ (deg)	c' (psf)	ϕ' (deg)
Levee Fill	115	120	1000	0	0	29
Foundation Blanket	110	115	500	0	0	26
Foundation Sands	115	120	N/A	N/A	0	32

The existing levee sections consist of a riverward impervious zone and landward random fill zone, and toward the lower end of the unit there is also a pervious fill section on the river side. To simplify the analyses, one set of parameters was used for the entire levee section and was called "levee fill".

The blanket materials consist mostly of ML and CL materials, with some discontinuous layers of CH and SM material. Design Memorandum No. 3 presented the laboratory test results sorted by soil classification. To simplify the analysis for this study, the blanket was modeled as a single material with only one set of strength parameters used. The soil strength applied to the blanket was a weighted average of the strength parameters for CL, ML and CH from the Design Memorandum.

Undrained shear strength data was not readily available for most of the materials, so undrained strengths were estimated from the limited 1962 Modification test data and typical values for these types of soils. Foundation blanket strength data was increased slightly from the existing test data to account for an increase in material strength under the footprint of the existing levee due to consolidation from the weight of the levee. It is recommended that additional sampling and testing be performed during PED to verify the undrained strength of the blanket materials.

A-4.4 EXISTING CONDITIONS RELIABILITY ANALYSIS

A-4.4.1 Introduction

The purpose of this portion of the study was to determine the probability of failure of the Armourdale Levee Unit for the existing condition of the unit. The analysis considered both underseepage piping failures and landward slope failures under steady state seepage conditions. The evaluations were performed in general accordance with the USACE Engineering Technical Letter (ETL) 1110-2-556 "Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies." The results of the analyses were used to determine the economic benefits attributed to proposed levee raises.

A-4.4.2 Probabilistic Theory

A-4.4.2.1 Probabilistic Parameters

Several parameters are commonly used to describe probability distributions such as the normal distribution shown in **Exhibit 2**. Probably the most common of these is the mean or expected value. The expected value of a continuous random variable X (a variable that can take on any value within some continuous range) with some distribution $f(x)$ is defined as:

$$\mu_X = \int_{-\infty}^{\infty} x_i f_X(x) \; dx \qquad \text{Equation A-4.1}$$

where μ_X is the mean value of the random variable X, x_i is a particular value of the random variable X and $f_X(x)$ is the frequency of occurrence of the random variable X. The expected value, or mean, of a random variable is the weighted average of the values of the random variable with the weighting being the frequency of occurrence of the value. For a set of discrete measurements of a random variable, the mean value is computed as:

$$\mu_X = \frac{\sum_{i=1}^{N} x_i}{N} \qquad \text{Equation A-4.2}$$

The variance of the random variable X, Var[X], is a measure of the spread, or variability of the random variable about the mean. The variance is computed as:

$$\text{Var[X]} = \int_{-\infty}^{\infty} (x_i - \mu_X)^2 f_X(x)\, dx \qquad \text{Equation A-4.3}$$

For a set of discrete measurements of a random variable X, the variance is computed as:

$$\text{Var}[X] = \frac{\sum_{i=1}^{N} (x_i - \mu_X)^2}{N} \qquad \text{Equation A-4.4}$$

If the number of observations N is a relatively small set of an entire population, an unbiased estimate of the variance can be given as:

$$\text{Var}[X] = \sigma_X^2 = \frac{\sum_{i=1}^{N} (x_i - \mu_X)^2}{N-1} \qquad \text{Equation A-4.5}$$

The standard deviation, σ_x, is also a measure of the distribution of the random variable about the expected value and is the square root of the variance:

$$\sigma_X = \sqrt{Var[X]} \qquad \text{Equation A-4.6}$$

The coefficient of variation, COV, is a convenient dimensionless parameter used to express the uncertainty or variability of a random variable and is computed as:

$$\text{COV} = \frac{\sigma_X}{\mu_X} \qquad \text{Equation A-4.7}$$

The coefficient of variation is useful because it expresses the variability of a random variable normalized with respect to the mean of the random variable. The expected value,

standard deviation and coefficient of variation are interrelated; therefore, the third can be determined by knowing any two of the parameters.

A-4.4.2.2 Probability Distributions

Many forms of probability distribution are available that can be used to represent the variability and uncertainty. However, based on previous work (Kitch, 1994) the normal and log-normal distributions are by far the most commonly used for risk based analyses.

The normal distribution is the most widely used distribution in the description of statistical phenomenon. The probability density function for a normally distributed random variable is expressed as:

$$f_X(x) = \frac{1}{\sigma_X \sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{x - \mu_X}{\sigma_X} \right)^2 \right] dx \qquad \text{Equation A-4.8}$$

where $f_X(x)$ is the relative frequency of the random variable X and is not a probability, but a representation of the distribution of probability that a particular random variable may lie within some stated interval. As shown in **Exhibit 2** the normal distribution has a bell shape with upper and lower limits of positive and negative infinity.

Another distribution that has been proven useful for reliability-based analysis in geotechnical engineering is the log-normal distribution shown in **Exhibit 3**. In the log-normal distribution, it is assumed that the natural logarithm of a random variable X is normally distributed. As shown in **Exhibit 3**, the log-normal distribution is positively skewed towards the lower values. However, it has the distinct advantage that the probability of the random variable cannot be less than zero. The log-normal distribution is therefore useful for representing parameters that cannot take on negative values (e.g. factors of safety and hydraulic gradient).

If a random variable X is log-normally distributed, the *ln X* is normally distributed. The probability density function can therefore be expressed as:

$$f_X(x) = \frac{1}{x\sigma_{\ln X} \sqrt{2\pi}} \exp\left[-\frac{1}{2}\left(\frac{\ln x - E[\ln X]}{\sigma_{\ln X}} \right)^2 \right] dx \qquad \text{Equation A-4.9}$$

where $\sigma_{\ln X} = \sqrt{Var[\ln X]}$, and $E[\ln X]$ is the expected value(mean) of the natural logarithim of X.

A-4.4.2.3 Probabilistic Measure of Slope Stability

In reliability-based analysis of slope stability, the input parameters that are not well defined are considered to vary according to some form of distribution as described in the previous section. These variable parameters are then used as input into a series of stability analyses to obtain the overall distribution of the performance function. The performance

function is used to report the stability of the slope. The performance function used throughout this study for slope stability is the factor of safety.

A hypothetical distribution of the factor of safety that could result from analyses using probabilistic parameters is shown in **Exhibit 4**. As shown in the figure, the distribution indicates that the actual factor of safety may take on a range of possible values, ranging from well below the limiting value of FS = 1.0 to well above the limiting value. While knowledge of the complete distribution of the factor of safety is useful, it is the relative frequency of factors of safety less than the limiting value that are of primary importance (FS ≤ 1.0 => Failure). Three different probabilistic parameters are typically used to represent this relative frequency.

The probability of failure of a system is the area under the probability density function shown as the shaded area in **Exhibit 4**. For the log-normal function, this would be from the boundaries (0 ≤ FS ≤ 1). In mathematical terms it can be expressed as:

$$P_f = \int_0^1 f_X(x)\,dx \qquad \text{Equation A-4.10}$$

where $f_X(x)$ is the probability density function expressed in Equation A-4.8.

The reliability of a system is conversely the area under the probability density function bounded by the limiting value and positive infinity. In **Exhibit 4**, it is represented by the non-shaded area under the curve. For a log-normal distribution, the boundaries would be (1< FS ≤ +∞). Since the total probability for all possible values of the random variable is 1.0, the probability of failure, P_f, and the reliability, denoted as R, are related by:

$$P_f = 1\text{-}R \qquad \text{Equation A-4.11}$$

Based on the assumption that the factor of safety is log-normally distributed, the natural log of the factor of safety will be normally distributed. In this case, the boundaries for the probability of failure would be (-∞< lnFS ≤ 0). Under this assumption, the probability curve and its probabilistic parameters would be represented in **Exhibit 5** with the probability of failure in the shaded area.

The reliability index, β, is a gage of the reliability of a system that takes into account technicalities of the procedure and the uncertainties introduced by random input variables. The reliability index gives a measure of comparative reliability for a system, thereby making it unnecessary to calculate or determine the actual probability distribution. It is defined using the probabilistic terms of standard deviation and the expected value (mean) of the performance function. Graphically, the reliability index multiplied by the standard deviation is equal to the distance from the expected value (mean) to the limiting state as shown in **Exhibit 4**. For a log-normal distribution, the reliability index is computed as:

$$\beta = \frac{\ln\left[E[FS] \Big/ \sqrt{1 + COV[FS]^2} \right]}{\sqrt{\ln\left(1 + COV[FS]^2\right)}} \qquad \text{Equation A-4.12}$$

where β is the reliability index, *E[FS]* is the expected value (mean) of the factor of safety, and *COV[FS]* is the coefficient of variation of the factor of safety.

A-4.4.2.4 Probabilistic Measure of Stability for Underseepage

When the excess head at the ground surface on the landward side of the levee toe is greater than zero and the blanket material is thicker than one-fourth the levee height, the probability of failure can be calculated using the method described in ETL 1110-2-556.

Using this method, the exit gradient (i) is assumed to be a log-normally distributed random variable with probabilistic moments E[i] and σ_i. Based on this assumption, the equivalent normally distributed random variable has moments E[ln i] and $\sigma_{\ln i}$. The limit state for the underseepage would then be the natural log of the failure gradient (i_f) with the boundaries for the probability of failure being:

$$P_f = P(\ln i > \ln i_f) \qquad \text{Equation A-4.13}$$

The probability of the ln i being greater than the ln i_f is determined by using the standard normalized variate (z), which is also analogous to the reliability index β. The standard normalized variate is calculated as:

$$z = \beta = \frac{\ln i_f - E[\ln i]}{\sigma_{\ln i}} = \frac{\ln\left[\dfrac{i_f * \sqrt{1 + COV[i]^2}}{E[i]} \right]}{\sqrt{\ln(1 + COV[i]^2)}} \qquad \text{Equation A-4.14}$$

where, *E[i]* is the expected value (mean) of the hydraulic gradient and *COV [i]* is the coefficient of variation of the hydraulic gradient. **Exhibit 6** shows a graphical representation of the probabilistic parameters for the underseepage analysis with the probability of failure in the shaded area.

A-4.4.2.5 Taylor Series Approximation Method for Determining Risk and Uncertainty Analysis

As described in the previous sections, the probability of failure can be computed if the expected value (mean) and variance of the distribution are known. Numerous methods are available for computing the probability of failure for reliability-based analyses, including first order second moment methods (FOSM), the point estimate method, the Hasofer-Lind method, and Monte Carlo simulations (Baecher & Christian 2000). While all of these methods can be used, the most commonly used method to date in geotechnical applications is the Taylor Series

Approximation of the FOSM method (USACE, 1999). The basis of the Taylor series method is that it uses the first two linear terms on the Taylor series expansion of the performance function to determine the probabilistic measures of performance. As such, the method is exact for linear performance functions and is approximated for higher order functions. While this method is approximate from a strictly probabilistic point of view, it has the significant advantage of being relatively simple to implement.

For a function (Y) of random independent variables (X_1, X_2, . . .X_n) of the form

$$Y = g(X_1, X_2, \ldots X_n) \qquad \text{Equation A-4.15}$$

the expected value (mean) of Y can be found by evaluating the function at the expected values (mean) of the random variables. In the slope stability analysis application, the function Y is chosen to be the factor of safety and the random variables are the input parameters that are chosen as probabilistic. The expected value of the factor of safety is therefore computed directly from the expected values (mean) of the random variables.

Stated in mathematical form, this is:

$$E[FS] = FS(E[\bar{\phi}_{\text{foundation}}], E[\bar{\phi}_{\text{blanket}}], E[\bar{\phi}_{\text{embankment}}]) \qquad \text{Equation A-4.16}$$

where E[FS] is the expected value (mean) of the factor of safety and $E[\bar{\phi}_{\text{foundation}}]$, $E[\bar{\phi}_{\text{blanket}}]$, and $E[\bar{\phi}_{\text{embankment}}]$ are the expected values (mean) of the random variables.

The Taylor Series approximation for the variance of the factor of safety can be expressed as:

$$\text{Var[FS]} = \sum\left[\left(\frac{\partial FS}{\partial X_i}\right)^2 Var[X_i]\right] \qquad \text{Equation A-4.17}$$

where X_i represents a value of the ith random variable for the stability analysis, $Var[X_i]$ is the variance of that random variable, and $\frac{\partial FS}{\partial X_i}$ is the partial derivative of the distribution of the factor of safety evaluated at the expansion point. Noting that the $Var[X] = \sigma_X^2$ and approximating the partial derivative with a difference form, Equation A-4.17 becomes:

$$\text{Var[FS]} = = \sum\left(\left[\frac{\Delta FS}{\Delta X_i}\right]^2 \sigma_i^{\,2}\right) \qquad \text{Equation A-4.18}$$

where σ_i is the standard deviation of the ith random variable and $\frac{\Delta FS}{\Delta X_i}$ is the approximated partial derivative. It has become common to evaluate the partial derivative $\frac{\Delta FS}{\Delta X_i}$ at the expected value (mean) plus one standard deviation and at the expected value (mean) minus one standard deviation as shown in **Exhibit 7** so that $\Delta X_i = 2\sigma_i$. Making this simplification, the expression for the variance becomes:

$$\text{Var[FS]} = \sum\left(\frac{FS(E[FS]+\sigma_{FS}) - FS(E[FS]-\sigma_{FS})}{2}\right)^2 \qquad \text{Equation A-4.19}$$

where $FS(E[FS] + \sigma_{FS})$ is the factor of safety calculated at the expected value plus one standard deviation and $FS(E[FS] - \sigma_{FS})$ is the factor of safety calculated at the expected value minus one standard deviation. Noting that the $\sqrt{Var} = \sigma$, the equation for the standard deviation for the factor of safety will become:

$$\sigma_{FS} = \sqrt{\left(\frac{\Delta FS_1}{2}\right)^2 + \left(\frac{\Delta FS_2}{2}\right)^2 + \ldots\ldots + \left(\frac{\Delta FS_n}{2}\right)^2} \qquad \text{Equation A-4.20}$$

where σ_{FS} is the standard deviation of the factor of safety and ΔFS is the difference between the factors of safety calculated at the expected value plus and minus one standard deviation for each of the random variables.

The discussion above describes how the factor of safety was evaluated as the limit state function. The exact same procedure can also be used with the critical hydraulic gradient as the limit state with different input parameters applicable to the underseepage analysis.

Once the standard deviation and expected value for the factor of safety are known, the coefficient of variation COV for the factor of safety may be calculated and then used in Equation A-4.12 to compute the reliability index. Given the reliability index, β, the probability of failure is calculated using the built-in function NORMSDIST in Microsoft Excel. This function uses the reliability index as the argument allowing for the probability of failure to be computed as:

$$P_f = 1 - \text{NORMSDIST}(\beta) \qquad \text{Equation A-4.21}$$

A-4.4.3 Uncertainty Analyses

A-4.4.3.1 General

Risk-based analyses for the Armourdale Levee Unit were performed for the existing conditions. In these reliability analyses, geotechnical uncertainties were assessed by determining probability distributions for the blanket thickness and soil material properties for typical levee sections representative of the Armourdale Levee Unit.

Two types of geotechnical failures were analyzed: 1.) a slope failure, defined as failure of the landside embankment slope resulting in water from the river flowing to the landside areas of the levee resulting in economic damages to the interior and 2.) an underseepage failure, defined by excessive seepage initiating a levee failure and resulting in economic damages to the interior. Geotechnical failures may occur when river stages reach elevations at or below the top of levee.

The probability of failure of the levee is also conditional on the uncertainties associated with the hydrologic and hydraulic aspects of determining the water surface profile during a flood. These uncertainties can be combined with the geotechnical uncertainties and used in the HEC-FDA program. This is performed for economic purposes through the development of a relationship between the probability of failure of the levee and the height of water on the levees.

A-4.4.3.2 Probabilistic Underseepage Analysis

The actual conditions indicative of an underseepage failure are highly speculative. The underseepage analysis included in ETL 1110-2-556 - Appendix B uses a threshold value of gradient factor of safety of 1.0 to define failure. A gradient factor of safety of 1.0 reflects a condition where floatation of particles theoretically begins and seepage and boils can first physically occur, however it is not necessarily a condition indicative of having certain levee failure. Observations during the Flood of 1952 on the Missouri River are shown in Table A-4.3. The table shows the relation between observed field performance and calculated factors of safety. From the observations it can be seen that somewhere between a factor of safety of 0.55 and 0.80, undesirable seepage reaches a point where a failure could occur without outside intervention in the form of flood fighting. In an effort to define a condition more representative of actual levee failure due to underseepage for this study, a gradient safety factor of 0.70 was utilized as a threshold value for when certain levee failure is likely to occur. The chosen threshold value of gradient factor of safety of 0.70 falls within the "transition" zone in Table A-4.3 between tolerable seepage and objectionable seepage. In the probabilistic underseepage analyses a failure gradient (i_f) was calculated as:

$$i_f = \frac{i_c}{FS} = \frac{0.86}{0.70} = 1.23 \qquad \text{Equation A-4.22}$$

where i_c is the critical gradient and FS is the gradient safety factor. The factor of safety that defines failure was used to define the failure gradient in Equation A-4.22 and the limit state in Equation A-4.13.

TABLE A-4.3
Observations of seepage conditions during 1952 flooding on the Missouri River at the Kansas Citys flood control project

Computed Safety Factor at Flood Crest	Seepage conditions during flood Crest
Less than 0.55	Objectionable seepage: major flood fight; boils requiring sandbagging
0.55 to 0.80	Transition zone
Greater than 0.80	Tolerable seepage: distributed seepage, pin boils

The Kansas City District method of estimating the hydraulic gradients due to underseepage is slightly different than the method described in the EM 1110-2-1913. It is based on the findings made at the Missouri River Division Conference held by the Corps of Engineers in 1962 in Omaha. The underseepage analysis was based on experience during the flood event in 1952 along the Missouri River. The main differences in the Kansas City District method are:

1. The Kansas City District Method uses permeability ratios (See Table A-4.4.) related to differing material types of the blanket material instead of using actual horizontal and vertical permeabilities.

2. The Kansas City District Method assumes an infinite landside blanket in the analysis.

3. The Kansas City District Method does not use a transformed thickness for the soil stratum considered as EM 1110-2-1913 allows, instead, a representative permeability ratio is applied to the overall blanket thickness.

TABLE A-4.4
Permeability Ratios for Blanket Material Based on Material Type

Blanket Material	Assumed Permibility Ratio
SM	100
ML	200-400
ML-CL	400
CL	400-600
CH	800-1000

Additional information concerning the underseepage analysis for the Kansas City procedure can be found on the District's website at http://www.nwk.usace.army.mil/local_protection/guidance.html.

The critical section for an underseepage failure along the Armourdale Levee Unit was chosen by calculating the expected value of the factor of safety with respect to hydraulic gradient at the toe of the levee for the entire unit. The reach with the lowest expected factor of safety was chosen for a risk analysis.

In the probabilistic analyses of underseepage using the Kansas City District method, three random variables were considered: blanket thickness, the permeability ratio and thickness of the aquifer.

Using existing subsurface information, it was assumed that the COV of the blanket thickness and thickness of the aquifer was 20 percent and 15 percent, respectively. These values for COV are deemed appropriate for the level of information available.

Using the published value given in ETL 1110-2-556, it was assumed that the COV of the permeability ratio was 40 percent. The permeability ratios used in the analyses followed the Kansas City District Guidance based on the type of material making up the blanket layer. In the existing conditions phase of the study the permeability ratios used in the underseepage analyses were based on material descriptions obtained from historical borings information from the Armourdale unit. Table A-4.4 lists the permeability ratios.

The underseepage analyses are then performed using the expected values of the random variables and plus and minus one standard deviations at different river levels. Using the log normal distributions and the limit state function for underseepage, a probability of failure can be developed for each river level at the critical locations.

A-4.4.3.3 Probabilistic Slope Stability Analysis

The conditions leading to a stability failure are less uncertain than those of an underseepage failure. A threshold value of stability factor of safety of 1.0 to define a slope failure is nearly universally accepted. The assumptions made for the slope stability component of the risk-based analysis allowed the evaluation to be more specific as to the magnitude of the failure and the actual consequences associated with that type of failure. The slope stability analyses assumed that the failure surface should be of significant magnitude to remove a major portion of the levee allowing the interior of the levee unit to flood.

The critical section for the stability analysis was chosen based on levee height and side slope steepness. The section with the tallest levee height and steepest side slopes was chosen for the probabilistic analysis.

Each zone of material making up the critical cross section of the levee was considered homogenous. The zones were comprised of three areas: the foundation sands, the blanket materials, and the embankment material. The foundation sand strengths were considered

constant in the anlaysis. The piezometric surface through the levee cross section was simplified and considered to be in a steady state condition. The model that was used assumed that the water surface entered the slope at the point on the riverside where the river intersected the upstream slope face. The piezometric surface then continued in a linear path to the landside levee toe.

The soil strength parameters considered in the existing conditions analysis were modeled with drained strengths because steady seepages conditions were considered. The mean values and coefficients of variations were computed from raw data located in Design Memorandum No 3 – Armourdale Unit, Dated May 1971. The raw data used in this study was taken from consolidated drained direct shear tests performed for the 1962 Modification of the Armourdale Unit. The effective stress failure envelopes for normal effective stresses less than 2000 psf were used to characterize the strengths of the soils. This was done because the "working load" effective stresses in the embankment and foundation materials are generally near, or less than, this value during flood conditions.

The materials evaluated were designated as either foundation blanket material or embankment fill material. Based upon available laboratory test data, with the results shown in Table A-4.5, it was determined that the blanket had an expected value (E[$\bar{\phi}$]) of 33˚ with a coefficient of variation (COV$_{\bar{\phi}}$) of 16 percent, and the embankment had an expected value (E[$\bar{\phi}$]) of 32˚ with a coefficient of variation (COV$_{\bar{\phi}}$) of 14 percent. Cohesion (c) was assumed to be zero with no variation for both materials.

TABLE A-4.5
Effective Strength Data Used Embankment and Foundation Materials

Boring	Sample	Soil	Material	τ (tsf)	σ (tsf)	ϕ (degrees)
U-549	3	Sandy Clay	Foundation	0.67	1.0	33.8
U-549	Wax-9a	Silt	Foundation	0.73	1.0	36.1
U-549	Wax-10	Silt	Foundation	0.63	1.0	32.2
U-549	Wax-8a	Silt	Foundation	0.41	1.0	22.3
U-549	Wax-8b	Silt	Foundation	0.67	1.0	33.8
U-549	Wax-9b	Silt	Foundation	0.56	1.0	29.2
U-549	Wax-10	Silt	Foundation	0.71	1.0	35.4
U-550	Wax-1a	Lean Clay	Foundation	0.67	1.0	33.8
U-550	Wax-1b	Lean Clay	Foundation	0.77	1.0	37.6
U-550	Wax-2	Lean Clay	Foundation	0.60	1.0	31.0
U-550	Wax-8a	Lean Clay	Foundation	0.67	1.0	33.8
U-550	Wax-8b	Sandy Silt	Foundation	0.79	1.0	38.3
U-550	Wax-4	Lean Clay	Foundation	0.71	1.0	35.4
U-550	Wax-9	Lean Clay	Foundation	0.66	1.0	33.4
U-550A	Wax-2b	Sandy Clay	Foundation	0.71	1.0	35.4
U-550A	Wax-2a	Lean Clay	Foundation	0.67	1.0	33.8
U-551	Wax-2	Fat Sandy Clay	Foundation	0.63	1.0	32.2
U-551	Wax-3	Fat Clay	Foundation	0.59	1.0	30.5
U-551	Wax-4	Sandy Silt	Foundation	0.66	1.0	33.4
U-551	Wax-5	Fat Sandy Clay	Foundation	0.55	1.0	28.8
U-551	Wax-6	Fat Sandy Clay	Foundation	0.41	1.0	22.3
U-551	Wax-6	Fat Clay	Foundation	0.41	1.0	22.3
U-552	Wax-4a	Fat Sandy Clay	Foundation	1.00	1.0	45.0
U-552	Wax-4b	Fat Sandy Clay	Foundation	0.99	1.0	44.7
U-552	Wax-5b	Fat Organic Clay	Foundation	0.71	1.0	35.4
U-552	Wax-5a	Fat Clay	Foundation	0.59	1.0	30.5
U-552	Wax-6	Fat Clay	Foundation	0.59	1.0	30.5
D-530	sk-2	Lean Clay	Embankment	0.49	1.0	26.1
A-534	sk-2	Lean Clay	Embankment	0.45	1.0	24.2
A-537	sk-2	Lean Clay	Embankment	0.67	1.0	33.8
A-537	sk-3	Fat Clay	Embankment	0.59	1.0	30.5
D-539	2	Lean Clay	Embankment	0.71	1.0	35.4
D-539	3	Silt	Embankment	0.71	1.0	35.4
HA-539	5	Fat Clay	Embankment	0.62	1.0	31.8
543	7	Lean Clay	Embankment	0.73	1.0	36.1

The pore pressures developed in the blanket material were determined from the hydraulic gradient calculated at the base of the blanket material due to underseepage. The hydraulic gradient line was based on the output from the underseepage analysis using the Kansas City District Method. Assuming that the elevation head datum is at the same elevation as the base of the blanket material, the pore pressure (u) at a point along the base of the blanket material would

be equal to the distance from the hydraulic gradient line (h_p) to the base of the blanket multiplied by the unit weight of water (γ_w). The mathematical relation can be stated as follows:

$$u = h_p * \gamma_w \qquad \text{Equation A-4.23}$$

For points within the slope, the pore pressure at the top of the blanket was calculated as the distance from the phreatic surface to the top of the blanket (h_p) multiplied by the unit weight of water (γ_w) (as in Equation A-4.23). The pore pressure at the base of the blanket was calculated using the distance from the hydraulic gradient line as the pressure head (h_p) in Equation A-4.23. A linear interpolation between these two pore pressures would give the pressure distribution through the blanket material used in the slope stability analysis.

The embankment was assumed to be homegenous and impervious, even though it is comprised of impervious and random zones. This was done to simplify the analysis and due to the fact the random material is mostly comprised of impervious material.

The slope stability analyses were carried out in the same manner prescribed in ETL 1110-2-556. Utilizing the slope stability program UTEXAS 4 (using Spencer's Method), an initial circular search was performed using the expected values (means) for the random variables considered in the analysis. In order to determine a surface that would mobilize a large portion of the embankment that would lead to a catastrophic failure, a series of single surface searches were performed to locate the critical surface. The failure surface was forced through the intersection of the water surface and the slope face to model a catastrophic failure that would cause interior flooding. Using this boundary condition, the failure would be of significant magnitude to inundate the levee interior instead of assuming a progressive slope failure from the landward levee toe.

An initial run in the UTEXAS 4 program was made using the expected values E[$\bar{\phi}$] for each of the different material types. The factor of safety (FS) obtained from this analysis gave the expected value for the factor of safety E [FS]. The failure surface obtained from this initial run was then considered the critical surface. The remaining series of runs were made at plus and minus one standard deviation of the expected values for strength along the critical surface defined in the initial run. As each material property was changed, a resulting factor of safety was computed. The variation resulting in each change for that particular material type can then be used in the Taylor Series Approximation. Using the probabilistic methods described previously, a probability of failure could be determined for a specific river elevation. The procedure was then repeated for various river levels and a probability curve was computed based on slope stability relationships with river levels.

A-4.4.4 Results for the Reliability-Based Analyses of The Kansas Citys – Missouri and Kansas Flood Risk Management Project

A-4.4.4.1 Underseepage Results

The critical section for the Armourdale Levee Unit with respect to an underseepage failure was computed to be the "slot" area at approximately Station 276+00 under the railroad

bridges. This section was chosen as the critical section because it had the lowest expected value for factor of safety (0.90) for the entire unit. The "slot" area is comprised of an old railroad bed, up to 15 feet lower than the surrounding ground, directly adjacent to the landside of the existing floodwall. There is a system of fully penetrating artesian relief wells in the "slot" area that discharge directly into the slot. The purpose of the wells is to reduce the pressure at the base of the blanket and to fill the slot with water to further reduce the gradient through the blanket. The original design condition for the slot area is for the slot to be filled with a minimum of 5 feet of water during the maximum flood water level. The slot is to be maintained inundated with a minimum of 5 feet of water until the river levels recede. It is highly unlikely that a flood of any magnitude would occur and a minimum of 5 feet of water would not be ponded in the slot area. The operational restriction and required ponding levels are detailed in the Operation and Maintenance Manual and Design Manual No. 3.

The typical section used in the analysis consisted of 18.5 ft of driving head, and an expected value for blanket thickness in the slot of 14 feet. The expected value for permeability ratio and foundation sand depth is 300 and 47 feet, respectively.

The calculation necessary to determine the probability of failure for the slot area, which would include the uncertainties in the well flows, would be computationally intense. So the probability of a levee failure due to piping in the "slot" area was calculated with some deviations from the method described in the probabilistic underseepage analysis discussion due to the relief wells in the "slot" area. First, the expected value of the factor of safety was calculated, including the effects of the relief wells and interior ponding, for varying river stages for the slot area. The information used to calculate the expected value of the factor of safety is provided in Table A-4.6. To approximate the probability of failure from the calculated expected value of the factor of safety considering the relief wells, a relation between expected value of the factor of safety and probability of failure was developed using the methods described in the probabilistic underseepage analysis discussion that does not consider the relief wells. The statistical parameters described above for the coefficient of variances and threshold values were used in the determination of the relation. The relation, shown in **Exhibit 8**, was then used to determine the probability of failure using the calculated expected value of the factor of safety.

The probability of a levee failure due to piping in the "slot" area for the existing condition is shown in **Exhibit 9**. At the maximum river level, during steady state seepage conditions, the probability of failure is 8%. It should be noted that the probability of failure due to a piping failure in the "slot" area greatly increases if the slot is not allowed to fill with a minimum of 5 feet of water as it was designed to operate. Naturally, allowing the slot to fill with more than 5 feet of water decreases the probability of failure, as additional water on top of the blanket decreases the gradient through the blanket. Field visits indicate that the slot would be able to hold approximately 10 feet of water with no adverse affect to the protected area.

TABLE A-4.6
Calculation of Expected Value of Factor of Safety for Piping – Station 276+00

River Elevation (ft)	HGL @ Base of Blanket (ft)	h_0 w/o Ponding (ft)	Flow into the Slot (cfs)	Likely Depth of Ponded Water (ft)	h_0 w/ Ponded Water	i	$FS_{iexpected}$
753	751.6	12.6	2.6	5.0	7.6	0.54	1.55
755	752.7	13.7	3.3	5.0	8.7	0.62	1.35
757	753.8	14.8	4.0	5.0	9.8	0.70	1.20
759	754.9	15.9	4.7	5.0	10.9	0.78	1.08
761	756.3	17.3	5.4	5.0	12.3	0.88	0.96
762.5	757.0	18.0	6.0	5.0	13.0	0.93	0.90

i_c calculated with a blanket unit weight of 115 pcf
Top of Blanket in Slot = 739
Bottom of Blanket in slot = 725
Volume of Slot = 195,000 ft^3
Probability of Failure taken from Probability of Failure vs Expected Factor of Safety Relation

A-4.4.4.2 Stability Results

The critical section for the Armourdale Levee Unit with respect to slope stability was located at approximately Station 222+00. This section was chosen as the critical section due to the levee height and levee side slopes.

The levee at Station 222+00 has a typical cross section of a 17.5-ft high levee with a side slope of 2.5:1 (horizontal to vertical) on the riverside, a crest width of 10-ft, and a net side slope of 2.5:1 (horizontal to vertical) on the landward side. The net landside side slope is comprised of a series of slopes and retaining walls that result in a net slope of approximately 2.5:1.

The probability of failure due to slope stability is shown in **Exhibit 10**. At the maximum river level, during steady state seepage conditions, the probability of failure was calculated to be 24%.

A-4.4.5 Summary

The geotechnical existing conditions analysis was performed to identify the critical sections from a geotechnical perspective and determine their probability of failure. The probabilistic analyses performed for this study were modeled with guidance given in ETL 1110-2-556 "Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies" (28 May 1999).

Two modes of unsatisfactory performance were considered at various river stages-underseepage and landside slope stability under a steady state seepage condition. Where enough information was present, the probabilistic parameters needed for each of the variables as

calculated. If little or no raw data was available, assumptions were made based on work done by others in the field of geotechnical risk-based analysis.

A-4.5 DETERMINISTIC AND ANALYSIS METHODS FOR DESIGN

A-4.5.1 Slope Stability Criteria

For the Kansas City Levees Phase 2 Feasibility Study, three proposed methods for raising the line of protection were analyzed for stability. These included an earth fill landside raise, an earth fill riverside raise, and a cantilever retaining wall raise on top of an existing levee. The criteria used for the slope stability analysis was from Engineering Manual 1110-2-1913, Design and Construction of Levees, dated April 2000. The engineering manual lists the following minimum requirements with respect to a deterministic slope stability analysis:

Table A-4.7 – Minimum Factors of Safety

Loading Condition	Minimum Factor of Safety
End of Construction	1.3
Steady Seepage	1.4
Rapid Drawdown	1.0 to1.2*
*Lower factors of safety may be appropriate when the consequences of failure in terms of safety, environmental damage and economic losses are small.	

The end of construction and steady seepage cases were analyzed for this Feasibility Study. The rapid drawdown stability analysis was not performed due to the lack of required shear strength parameters for the two stage analysis. Additional drilling and testing will be required as part of PED to determine the shear strength parameters for this analysis.

For levees in an urban area, rapid drawdown failure could be significant in terms of economics, not only for the temporary loss of protection but also for repairs to the levee. It is recommended that a factor of safety of 1.2 be used for this failure condition. The engineering manual does not specify the water levels for the loading condition, so the following was assumed:

Table A-4.8 – Water Loading Conditions

Loading Condition	Water Level for Stability Analysis
End of Construction	Water at Top of Natural Blanket
Steady Seepage	Water at Top of Protection, River Side
Rapid Drawdown	Water at Top of Protection, Stage 1 Water at Landside Toe of Levee, Stage 2*

*Or landside ground elevation, whichever is lower

Guidance was published by the HQUSACE in April 2007 with respect to Hurricane Protection System slope stability design criteria guidance. The document was published in the form of a Memorandum to the Commander, Mississippi Valley Division, and intended for use during levee rehabilitation in Southeast Louisiana. The revised design criteria was based on criteria presented in EM 1110-2-1902 Slope Stability, dated Oct 2003, for new embankment dams. The original criteria are consistent with that presented in the Table A-4.7. The new guidance suggests a factor of safety of 1.5 (if the site conditions are "well defined") for what is called the "extreme hurricane" condition, when steady state conditions are expected to develop with water at the top of protection. This is an increase in factor of safety from what was used for this feasibility study. Though the published criteria are currently only related to hurricane loadings, it could easily be transferred to all levees in the future. It is suggested that slope stability criteria be reviewed and revised as necessary during PED. If increased factors of safety are required for the Armourdale unit, the implications would likely consist of additional required real estate for expansion of stability berms.

A-4.5.2 Underseepage Criteria and Analysis

The current Corps of Engineers guidance on underseepage is contained in Engineering Technical Letter (ETL) 1110-2-569. The ETL recommends using all definitions, design equations, and procedures in Engineering Manual (EM) 1110-2-1913 except as noted within. The greatest deviation from the EM is the requirement for a maximum hydraulic gradient through the landside blanket at all points landward of the levee of 0.5, which provides for a factor of safety with respect to hydraulic gradient (FS_i) of approximately 1.6. For the design of future conditions alternatives of the Armourdale Unit, the criteria shown below was used to determine whether underseepage control measures are necessary.

With water at the top of line of protection:

*FS_i equal to, or greater than, 1.6 – No underseepage control measures are necessary.

*FS_i less than 1.6 – Design underseepage control measures to achieve a $FS_i = 1.6$.

Procedure for calculating hydraulic grade lines landward of the line of protection:

The general procedure outlined in EM 1110-2-1913 Design and Construction of Levees was used to calculate the factor of safety with respect to hydraulic gradient for the natural blanket, and to calculate the excess head at the landside toe (assumed to be acting at the bottom of the blanket) of the line of protection. The variations from EM 1110-2-1913 used in the analysis are as follows and discussed previously in this chapter:

1. The use of permeability ratios relating to different material types for the natural blanket, as opposed to actual horizontal and vertical permeabilities.

2. The assumption of an infinite landside blanket.

3. No blanket thickness transformation is performed.

Procedure for analyzing and designing relief well systems:

The general procedure outlined in EM 1110-2-1914 Design, Construction, and Maintenance of Relief Wells, Figure 5-3 was used to analyze and design all relief well systems. The variations from EM 1110-2-1914 used in the analysis and design are:

1. The excess head computed at the landside toe was used as the net head on the system of wells instead of full driving head. This was done because the procedure outlined in Figure 5-3 assumes an impervious blanket. However, a semi-pervious blanket was assumed for the underseepage calculations.

2. An efficiency reduction factor of 0.8 was applied to the expected well flows. This was done to account for the reduction in efficiency with time of the relief wells. An efficiency factor of 0.8 was chosen as EM 1110-2-1914 requires remedial action once a loss of 20% in specific capacity of a well is observed from pumping test.

A-4.6 N500+3 STABILITY ANALYSES

A-4.6.1 Sections Analyzed

For the Armourdale levee unit, the design team developed 4 line of protection raise configurations to raise the level of protection to an n500+3 event on the Kansas River. The method for computing the river stage for this event is discussed in the hydrology and hydraulics chapter. The top of the proposed raise was set approximately equal to the river stage for the n500+3 event. The four raise configurations are as follows:

1. Land side earth fill raise
2. River side earth fill raise
3. Cantilever floodwall on top of the existing levee
4. Floodwall

All levee sections will maintain a 10-foot crest width to maintain the current level of vehicle access. Floodwall stability is addressed in the structural chapter of this Feasibility Report.

To begin the evaluation process, a site visit was made to Armourdale to make an initial attempt to identify the most appropriate raise configuration along the entire unit on a reach by reach basis. Based upon the initial assignments the next step was to identify the most critical cross sections to analyze. The critical sections were selected by considering the existing levee height, height of proposed raise, thickness of the natural blanket, and the results of the underseepage analyses. Two cross sections were analyzed for each of the proposed raise configurations. Calculations are provided in **Exhibit 11.**

The computer program UTEXAS4, developed by Stephen G. Wright of the University of Texas at Austin, was used to perform the analyses. The selected analysis method was Spencer's method, which is a limit-equilibrium approach that satisfies both force and moment equilibrium. The program has the ability to "search" for the critical failure surface with the lowest factor of safety for the given input parameters. As stated previously, only the end of construction and steady seepage loading conditions were analyzed. Steady seepage conditions controlled the final section dimensions for all sections analyzed. Potential rapid drawdown failure of the river side slope will also have to be evaluated after additional geotechnical laboratory testing can be performed to determine the necessary strength parameters required for this analysis.

A-4.6.2 Land side earth fill raise

A land side earth fill raise is the preferred raise configuration due to the low cost and ease of construction, and was proposed wherever possible. There are several reaches of the unit where this raise configuration was proposed. Two sections were selected to be analyzed. One section was selected due to the high piezometric levels in the foundation, and the other section was selected because it was the tallest levee section. The levee section was raised by maintaining the riverside slope and shifting the levee centerline landward.

The first land side fill section analyzed was at station 90+00. This section has the highest piezometric levels in the foundation of the reaches where the land side fill was proposed. The proposed height of the levee for the n500+3 raise is 16.5 feet. The stability analysis indicated an acceptable cross section requires a 1V on 4H land side slope. The existing levee cross section has a 1V on 3H land side slope. A typical cross section for this location is shown in **Fig C1 of the map book**, and is labeled Section 1**.** A drainage layer was added under the new land side fill to improve the internal seepage conditions in the embankment.

The second land side fill section analyzed was at station 245+50. This section was the tallest proposed section for the land side raise configuration. The proposed height of the levee is 20 feet. The general approach for analyzing this section was the same as for Section 1. For this section to meet the minimum factor of safety two stability berms were required to be added to the basic levee section developed for station 90+00; a 20-foot wide, 8-foot tall berm with a 1V on 4H slope and a shorter 10-foot wide, 4-foot tall berm with a 1V on 3.5H slope. A typical cross section for this location is shown in **Fig C1 of the map book**, and is labeled Section 2**.** Again, a drainage layer was added under the new land side fill to control the internal seepage conditions in the embankment.

There are several reaches of the levee with a proposed land side raise configuration that have new levee heights between 16.5 feet and 20 feet. For the feasibility levee design it was assumed that the section for station 245+50 would apply to these reaches. It is suggested that for future work that at least one intermediate section be analyzed to optimize the cross sections in these reaches.

A-4.6.3 River side earth fill raise

A river side earth fill raise is proposed at one location; at the upper end of the project between station 3+25 UE and 10+00 UE. At this location there are real estate constraints on the land side of the current levee and a wide foreshore area on the river side. The river side fill levee section configuration is raised by maintaining the land side slope, so the levee centerline shifts riverward.

The river side fill section evaluated was at Station 7+00 UE. The analysis indicates a small stability berm is required on the landside due to high piezometric levels in the foundation from the steady seepage loading condition. A typical cross section for this location is shown in **Fig C1 of the map book**, and is labeled Section 3**.** A drainage layer was added in the land side berm to control the steady seepage water surface through the embankment.

A-4.6.4 Cantilever floodwall on top of existing levee

Using a cantilever wall to raise the level of the existing levee section is proposed where minimal real estate impacts on the landward side of the current protection are preferred. The floodwall is placed in the levee section so the river side face of the wall is at the riverward edge of the crest. The local wall stability (i.e. sliding, bearing capacity and overturning) is discussed in the structures chapter in this Feasibility Study Report. This discussion focuses on the overall global stability of the section. Critical failure surfaces were searched both riverward and landward of the wall. Again, two sections were analyzed for this raise configuration. The end of construction loading condition was not analyzed due to the limited amount of additional fill that would be placed in this section.

Utilizing a cantilever floodwall to raise a levee section presented a significant design challenge due to the piezometric water levels developed in the foundation and the embankment for the steady seepage condition. The piezometric water levels increase significantly over the existing condition levels, however due to the wall raise there is minimal additional levee section being added to balance the increase in seepage pressures. Because of this situation, additional landside fill was required to improve the stability of this section.

The first cantilever wall section analyzed was at Station 100+00. This section was selected because it is the largest height increase for this configuration, 5.2 feet. The existing height of levee (measured from the land side) is 10.5 feet. To meet stability criteria, the existing levee section was modified until the minimum acceptable factor of safety was obtained. The final proposed section consists of a 1-foot crest raise, a 10-foot wide crest width, and a 1V on 4H land side slope. A typical section for the cantilever wall section is shown in **Fig C1 in the map book**, and is labeled Section 5 thru 9. The section at station 100+00 is section number 5. A drainage layer in the existing levee section was added at the elevation of the wall footing to control the steady seepage water surface through the embankment and minimize the amount of additional fill required.

The second cantilever wall raise section analyzed was at Station 122+50. This section was initially thought to have the tallest overall proposed section height, (This sentence is confusing as to its importance, maybe reword to clarify) a 14-foot existing levee height and a 4.8-foot raise. The section was analyzed using the section developed for Station 100+00 (section

5). However, a small stability berm is required for this taller section to meet stability criteria. A typical section for the cantilever wall section is shown in **Fig C1 of the map book**, and is labeled Section 5 thru 9. The section at station 122+50 is shown in **Fig c1 of the map book** and is labeled Section 7. A drainage layer in the existing levee section was added at the elevation of the wall footing to control the steady seepage water surface through the embankment and minimize the amount of additional fill required. Due to the additional land side fill required to meet the stability requirements of this section, the proposed raise configuration through this reach was changed to a floodwall. The results of this analysis is still valid, however, and were used for assigning sections to other similar reaches.

For the cantilever wall on existing levee configuration, the two sections evaluated were "interpolated" between to estimate the requirements of sections with existing levee heights intermediate between the two analyzed sections. The existing levee heights were broken into 2-foot intervals starting at < 10 feet. A table showing the section requirements for each interval is provided on **Fig C1 of the map book**. After the initial round of analyses a reach of the existing levee unit was found to be up to 18 feet in height. This is 4 feet taller than the tallest section previously analyzed. For feasibility study level design, the required retaining wall section for two additional intervals of existing levee heights were "projected" based upon the existing analysis. It is recommended that during the next phase of the project that these taller sections be analyzed to verify an acceptable section. The reach in question is between Station 228+00 and Station 245+00.

A-4.7 N500+3 UNDERSEEPAGE ANALYSES AND CONTROL FEATURES

A-4.7.1 Subsurface Information

The natural blanket was characterized using subsurface information obtained from Design Manual No 3 – Armourdale Unit (DM3). DM3 contains the results of an extensive subsurface investigation that was performed for the design of the 1962 Modification of the Armourdale Unit. The subsurface investigation was used to identify the soils present in the foundation of the line of protection, and establish their geotechnical parameters. This information was used to determine blanket thickness and composition which was used in the underseepage analysis. The Armourdale Unit has a foundation blanket varying between 12 feet and 40 feet thick, consisting of silts, clays, and discontinuous sand lenses. Underlying the foundation blanket is between 50 feet and 70 feet of clean sand before bedrock is encountered.

A-4.7.2 N500+3 Underseepage Analysis

Raising the Armourdale Levee Unit to a N500+3 level of protection increases the water pressures in the foundation sands, which in turn increases the hydraulic gradient through the natural blanket material. For the underseepage analysis, the entire Armourdale Levee Unit was divided into reaches of similar protection height, blanket thickness, blanket composition, aquifer thickness, and seepage entrance conditions. The factor of safety with respect to hydraulic gradient through the blanket was calculated for each of these reaches at the toe of the line of protection, and other critical areas such as building foundations and low areas as necessary. If the calculated factor of safety with respect to hydraulic gradient was calculated to be greater than

1.6 at all locations landward of the line of protection, no remedial measures were proposed. If the calculated factor of safety with respect to hydraulic gradient was calculated to be less than 1.6, remedial measures were proposed that would achieve a factor of safety with respect to hydraulic gradient of 1.6 at all locations landward of the line of protection. The design condition is to have a factor of safety with respect to hydraulic gradient equal to, or greater than, 1.6 at all locations landward of the line of protection toe. **Exhibit 13** shows the calculated factor of safety with respect to hydraulic gradient for the entire Armourdale Levee Unit (without the effects of relief wells or cutoff walls), as well as the parameters used to calculate the factor of safety.

A-4.7.3 Special Features Analyzed for Underseepage

Several existing structures and special features were analyzed to ensure a factor of safety with respect to hydraulic gradient of 1.6 is available in the thinner blanket under the feature. **Exhibit 14** provides details on all special features that were analyzed for the Armourdale Levee Unit. Building basement and feature elevations were assumed to be 10 feet below the ground surface if their elevations were not provided in DM3. In the calculation of the allowable excess head in the reduced blanket thickness under specified features, the differential head across the blanket was assumed to be the excess head above the surrounding ground surface. Essentially, the structures were analyzed as a void in the blanket filled with water to the elevation of the surrounding ground. This analysis method is valid as long as the basements are water tight, completely flooded, or as long as seepage into the basements is controlled or stopped. This approach ensures an acceptable gradient through the foundation blanket under the structure for the conditions noted above, however does not prevent potential seepage related problems into basements or structural problems due to large water pressures under basement slabs.

The increase in water pressure under basement slabs due to the N500+3 raise was calculated to determine the possible implications on the integrity of the structures. First, the existing conditions were analyzed to establish the baseline for any increase in pressure. The conditions analyzed for the 1962 Modification are the same as the current existing conditions, as the Armourdale Unit has not undergone significant modification since the 1962 Modification was analyzed. The existing conditions were modeled utilizing the same methods as the N500+3 analyses and compared to the calculations performed for the 1962 Modification at individual checkpoints. This comparison is shown in **Exhibit 15**. The initial hydraulic grade line (not considering well effects) calculated for the 1962 Modification and the Feasibility Study are nearly identical. This is because the same blanket theory equations used in 1962 are still in use today. However, the drawdown calculated for the 1962 Modification is an average of 2.2 feet greater than that calculated for the Feasibility Study at all checkpoints. A portion of the difference in calculated drawdown is attributed to the difference in well flow rates used in each calculation. The 1962 Modification calculations assumed a flow from each well of 1.33 cfs. The well flows for the Feasibility Study were calculated, and averaged approximately 1.18 cfs. This difference in well flows amounts to approximately a 0.5 difference in drawdown. The remaining difference of approximately 1.7 feet is likely attributed to differences in the drawdown calculation methods. Due to the difference of approximately 2.2 feet in the hydraulic grade lines at all checkpoints, the existing conditions at building locations was approximated by subtracting 2.2 feet from the that calculated for the Feasibility Study to capture the intent of the original designers in 1962.

The water pressure acting under basement slabs was calculated assuming a linear distribution of water pressure between the base of the blanket and the top of the blanket. The hydraulic grade line calculated in the underseepage analysis acting at the base of the blanket is used to calculate the water pressure acting at the base of the blanket. The water pressure is assumed to be zero at the ground surface.

The water pressure acting on the basement slabs is shown for the existing condition (1962 Modification and Feasibility Study Calculations), the N500+3 condition, and for a hydrostatic condition in **Exhibit 14**. The water pressure acting on the bottom of the feature increases by 1-foot of hydraulic head or less over the existing condition (1962 Modification Calculation) as a result of the N500+3 raise considering all underseepage control measures for both conditions assuming a linear pressure drop across the blanket. The effects of the increase in pressure caused by the N500+3 raise over the existing conditions are not considered to be significant. However, no analysis was performed to determine the structural integrity of the basement slabs under either existing conditions or the N500+3 raise. This should be further analyzed during final design, as the pressures developed during a flood event due to underseepage may be significantly greater than hydrostatic pressure.

A-4.7.4 Underseepage Control Requirements

The reaches outlined in detail in this section are either at the minimally acceptable factor of safety with respect to hydraulic gradient without remedial measures, or require remedial measures to increase the factor of safety to the minimally acceptable factor of safety with respect to hydraulic gradient. All existing underseepage control features on the Armourdale Unit outlined previously are assumed to remain in place as functional features, except as noted in the following discussion.

Station 66+00 to 79+00 – This reach is characterized by an area with a locally thin natural blanket between 12 and 15 feet in thickness. The calculated factor of safety with respect to hydraulic gradient in this reach ranges from 0.8 to 0.9 with water at the N500+3 elevation. A slurry cutoff wall is proposed to remediate this underseepage concern. The slurry cut off wall should extend to bedrock (approximately 90 feet below the landside ground surface) and should hydraulically connect to an impervious section of the line of protection riverward of the centerline. The slurry cutoff wall needs to extend beyond the critical reach of Station 66+00 to 79+00 to negate the seepage that will occur around the ends of the cut off wall. To determine the extension of the wall beyond the critical reach necessary to achieve the minimally acceptable factor of safety, hydraulic grade lines calculated using the Kansas City District method for the reaches between Station 60+00 to 66+00 and Station 73+00 to 79+00 were used to determine the shortest allowable seepage path around the slurry cutoff wall that will result in the maximum allowable head at the critical points. The maximum allowable excess head to have the minimally acceptable factor of safety with respect to hydraulic gradient at Station 66+00 and 79+00 (the ends of the critical reach) is 6.5 feet and 7.8 feet, respectively. The hydraulic grade lines indicate that the slurry cutoff wall must extend beyond the critical reach approximately 400 feet beyond Station 66+00 and approximately 300 feet beyond Station 79+00. The slurry cutoff wall should be constructed between Stations 62+00 and 82+00. **Exhibit 16** shows the hydraulic grade lines

that were used to determine the cutoff wall extensions beyond the critical area. It should be noted that there are some significant utilities that will have to be abandoned or modified to accommodate the slurry wall near Station 62+00, between Stations 75+00 and 77+00, and near Station 79+50. Additionally, the installation of a cutoff wall may change the overall groundwater flow in the area. The effects of the cutoff wall on the local ground water table were not considered. Should the effects on the local ground water table become an issue, surface discharging relief wells would become an acceptable alternative.

Station 86+00 to 93+00 – This reach is comprised of an area with the minimally acceptable factor of safety with respect to hydraulic gradient of 1.6 at the landside toe with water at the N500+3 elevation. No remedial measures are proposed at this time.

Station 100+100 to 130+00 – This reach is comprised of an area with the minimally acceptable factor of safety with respect to hydraulic gradient of 1.6 at the landside toe with water at the N500+3 elevation. No remedial measures are proposed at this time.

Station 157+00 – There is a localized rectangular "ditch" landward of the Mill Street Pump Station situated perpendicular to the line of protection. The "ditch" is approximately 100 feet from the line of protection toe, and is approximately 13 feet lower than the adjacent ground. A property line transverses the ditch centerline, indicating that the "ditch" may be the remnants of separate fills placed on two adjacent parcels of property which were each sloped to the property line. In its current configuration, the calculated factor of safety with respect to hydraulic gradient is approximately 1.1 in the "ditch" with water at the N500+3 elevation. It is recommended that the ditch be filled with impervious material to the elevation of the surrounding impervious blanket (elevation 760) to ensure the minimally acceptable factor of safety with respect to hydraulic gradient of 1.6 is achieved at all points landward of the line of protection.

Station 190+00 to 254+00 – This long reach is comprised of a marginally thin blanket with zones of sand and fill in the upper portions of the blanket. This reach is also characterized by the many existing structures in close proximity to the line of protection. The existing relief well system between Stations 190+75 and 246+35 and aerial fill between Stations 220+00 and 226+50 was constructed during the 1962 Modification to alleviate minor underseepage concerns through the blanket and major underseepage concerns related to the existing building foundations. The existing relief well system is a series of 24 fully penetrating artesian relief wells connected by a header pipe which directs well flows to the Shawnee Pump Station. To provide a calculated factor of safety with respect to hydraulic gradient of 1.6 at all locations landward of the line of protection for the N500+3 water level, minor modifications to the existing system and additional fully penetrating artesian relief wells are proposed.

The proposed modifications to the existing system of 24 fully penetrating wells are very minor. The modifications will be limited to removing the extensions of the riser pipe above the lateral header pipe, and relocating the discharge elevation to the elevation of the header pipe at each relief well location.

A total of 39 new relief wells are proposed to be added to the existing relief well system between Stations 190+00 and 254+00 at the landside toe of the line of protection. All new wells were assumed to discharge at the ground surface to avoid additional pump station requirements. **Exhibit 17** shows the locations of existing and proposed wells, along with the proposed discharge elevations and computed flow rates. Of the new wells, 25 wells with discharge elevations at the design landside ground elevation at each well location are required between Stations 190+00 and 246+00, mostly to protect the existing building foundations. The remaining 14 new wells are required between Stations 246+00 and 254+00 to ensure the minimally acceptable factor of safety with respect to hydraulic gradient of 1.6 through a regionally thin blanket. The wells between Stations 246+00 and 254+00 were designed using a conservative blanket thickness based on the available limited subsurface information on the riverside of the line of protection. Additional subsurface information should be obtained on the landside of the line of protection during final design to confirm that the wells between Stations 246+00 and 254+00 are required. **Exhibits 18** shows the computed excess head at the landside toe of the line of protection between Stations 190+00 and 254+00 for the N500+3 water elevation with the existing and proposed relief wells. **Exhibit 19** shows the computed excess head at the special features located in this same reach shown in **Exhibit 14**.

The total flow from the existing 24 wells, after the proposed modifications, was computed to be approximately 47 cfs for the N500+3 water level. The Shawnee Pump Station, which services the wells, was originally designed for a well flow of 32 cfs. The pump station will have to be modified to continue servicing the wells. The capacity of the header pipe, however, is sufficient for the increased flows from the existing system. The total flow from the new 39 wells was computed to be approximately 40 cfs. The wells were designed to discharge at the landside ground elevation at the well location. All flow from the new wells will discharge at the ground surface and flow into existing interior drainage features.

All building basement elevations should be verified during future design. The relief well system should be refined at that time to incorporate the actual presence of basements and their elevations. The existing relief wells should be pump tested during final design to determine if they still perform adequately. If they do not, they should be replaced. If the existing wells must be replaced, it would be prudent to rework the well layout to economize the design.

During refinements of the well system during future design, it may be possible to economize and/or add reliability to the relief well system. Some required redundancies in the Shawnee Avenue Pump Station (due to the below grade discharge of the existing well system) may be removed if the existing relief well system can discharge at the elevation of the top of the manhole. Additional surface discharge relief wells could be added to provide the required pressure relief due to the higher discharge elevation of the existing wells. However, due to the current assumptions and unknowns on building elevations, additional refinements to the system beyond what is being proposed here are better performed when the required information is available during future design.

Station 254+00 to 275+00 – This reach is comprised of a portion of the old “slot” area that appears to have been filled to the elevation of the surrounding ground. The minimum elevation required of the landside blanket is approximately 752 to achieve the minimally

acceptable factor of safety with respect to hydraulic gradient of 1.6. Existing topographical information and a field visit have indicated that the landside elevation is currently at or above the minimum required. However, the elevation of the landside blanket should be verified during final design. If the landside elevation is below the minimum required, the area will need to be brought to proper grade to maintain the required factor of safety with respect to hydraulic gradient.

Station 275+00 to 282+00 – This reach is comprised of a portion of the "slot" area that remains in its original configuration. Currently the "slot" is landward of the floodwall in this reach. However, portions of the floodwall may be relocated to be landward of the slot in the vicinity of the railroad bridges. In areas where the "slot" will remain landward of the floodwall, the minimum elevation required of the landside blanket is approximately 750 to achieve the minimally acceptable factor of safety with respect to hydraulic gradient of 1.6. This will require filling of the "slot" with impervious material to elevation 750. In locations where the floodwall will be moved landward of the "slot" the slot should be filled with impervious material to the elevation of the riverside fill on the existing floodwall (approximately elevation 748). The existing relief well system should be abandoned in place following all pertinent criteria. The abandonment procedures of the relief wells that have been previously abandoned (southern most 6 wells) should also be verified.

Station 296+00 to 313+00 – This reach is characterized by a regionally thin blanket combined with a large low lying area landward of the line of protection between Stations 296+00 and 303+00. The existing relief well system located near the landside toe between Stations 295+00 and 305+00 was designed and constructed to protect the low lying area which contained a large packing plant. The packing plant is no longer present, but some structures remain in use in the low lying area. The calculated factor of safety with respect to hydraulic gradient (with the N500+3 loading) for the area in its current configuration (taking into account the existing relief well system) is approximately 1.0 between Stations 296+00 and 303+00 and approximately 1.2 between Stations 303+00 and 313+00. The existing relief well system would require such significant modification that is not feasible to retain it. To protect the low lying area between Stations 296+00 and 303+00, the existing relief well system should be abandoned in place, and a new system of fully penetrating artesian relief wells should be constructed. A total of 35 fully penetrating artesian relief wells are required between Stations 296+00 and 313+00. **Exhibit 20** shows the proposed locations of the new relief wells, their discharge elevations, and computed individual well flows. A total of 26 fully penetrating artesian relief wells, variably spaced between 25 and 50 feet apart, which discharge at elevation 745 are required to achieve the minimally acceptable factor of safety with respect to hydraulic gradient in the low lying area between Stations 296+00 and 303+00. The total flow from the 26 relief wells was calculated to be approximately 14.5 cfs (or an average of 0.6 cfs per well). The new wells should be placed along the riverward edge of the low lying area, and should discharge directly into the low lying area which will be used as a temporary ponding area. The well system has been designed to achieve the minimally acceptable factor of safety with regards to hydraulic gradient in the low lying area without the consideration of any water being stored in the area. As a result, there are no ponding requirements in the low lying area, i.e. the ponding area can be pumped dry during flood events. **Exhibit 21** shows the computed excess head along the riverward edge of the low lying area, which is the critical area in the reach between Stations 296+00 and 303+00. To

protect the area between Stations 303+00 and 313+00, an additional 9 new fully penetrating artesian relief wells should be constructed. The 9 relief wells, variably spaced between 50 and 140 feet apart, should discharge at the landside ground elevation. The total flow from the 9 relief wells was calculated to be approximately 7.5 cfs (or an average of 0.9 cfs per well). The well flow from the 9 relief wells should also be directed to the low lying area which will be used as a temporary ponding area. **Exhibit 22** shows the computed excess head along the landside toe of the line of protection, which is the critical area in the reach between Stations 303+00 and 313+00. The total flow into the ponding area from all relief wells will be approximately 22 cfs.

A-4.8 EXPECTED SETTLEMENT OF DESIGN FEATURES

No calculations were performed to determine the expected settlement of the proposed line of protection raise for the n500+3 condition. This is because no consolidation test data was found to determine the appropriate parameters required for settlement calculations. For feasibility level design the following estimations were made for the proposed raise configurations:

- Floodwall. No settlement
- Cantilever wall on existing levee. Minimal settlement
- Earth fill raise on existing levee. 3 inches maximum settlement.
- Earth fill in place of existing floodwall. 6 inches maximum settlement.

These estimates will be used to determine the overbuild required for the different sections and for quantity estimation. It is recommended that during PED that additional soil sampling and testing be performed so the consolidation characteristics of the foundation materials can be quantitatively determined and settlement analysis performed.

A-4.9 RECOMMENDATIONS FOR PED PHASE

1. Slope Stability.

a. Two reaches, sta 228+00 to 245+00 and sta 250+60 to 257+65. There is a potential that a T-wall on existing levee section will be used through these reaches. If this section is used through these reaches, the slope stability of the tallest section needs to be analyzed. Currently the tallest existing levee section with T-wall analyzed is 14 feet (section 7). The maximum height in the above listed reaches approaches 18 feet.

b. Landside levee raise sections. The two levee raise sections analyzed for a landside earth raise were an intermediate height raise and the maximum height raise. The two proposed sections are significantly different, as the shorter section required no stability berms and the taller section required two large stability berms. It is recommended that at least one additional section with a height between the two sections be analyzed in order to minimize the amount of fill and real estate required.

c. It is recommended that the criteria used for slope stability be evaluated. The existing criteria were used for the Feasibility level design, however due to Hurricane Katrina there was a

lot of discussion about increasing factors of safety during this time, and some interim guidance had been published. By PED the criteria for flood risk management projects may be revised.

2.. Underseepage

a. It is recommended that all building elevations be confirmed, and proposed layouts of remedial measures are refined accordingly.

b. It is recommended that the levee unit be revisited for additional features, such as pits and low spots which need special attention with respect to the underseepage analysis.

c. It is recommended that all existing relief wells to remain in use be pump tested and inspected to ensure required well flows can be achieved and adequate condition of the wells.

d. It is recommended that any changes in Corps of Engineers (or local district) guidance which governs underseepage analysis methods or criteria be captured during final design.

3. It is recommended that a drilling and testing program be implemented to verify gaps in the existing data and to meet all criteria regarding sub-surface investigation intensity. Those include (at a minimum):

a. Landside blanket thickness between 246+00 and 254+00.

b. Soil Strength Testing.

i. Undrained strengths for the fill material and the blanket materials both under the existing levee sections and in the natural blanket outside the levee footprint.

ii. R-bar triaxial testing on the fill section and the natural blanket materials to develop strength parameters needed for rapid drawdown analysis.

iii. Consolidaiton testing in reaches to receive fill for purposes of settlement estimation.

4. Recommend a full topographic survey in the critical zone of the line of protection, including all the way to the riverbank.

5. Attempt to provide unrestricted vehicle access along the entire length of the line of protection for inspection. Currently only small reaches of the protection can be inspected at one time, and access to adjacent reaches requires navigating around the industries being protected.

6. Recommend evaluating the impact of ground discharging relief wells on the interior drainage. The quantity of expected discharge from proposed wells for the n500+3 conditions would indicate that interior flooding could be a significant problem.

7. A ground water study should take place in the area of the proposed cutoff wall to ensure local water interests will not be affected.

A-4.10 REFERENCES

1. Operations and Maintenance Manual, Kansas Citys Flood Control Project, Missouri and Kansas River, Argentine Unit, Volume I, Dated 1979.

2. Operations and Maintenance Manual, Record Drawings, Kansas Citys Flood Control Project, Missouri and Kansas River, Argentine Unit, Volume I, Appendix II, Dated 1951 - 1974.

3. Operations and Maintenance Manual, Kansas Citys Flood Control Project, Missouri and Kansas River, Armourdale Unit, Volume I, Dated 1979.

4. Operations and Maintenance Manual, Record Drawings, Kansas Citys Flood Control Project, Missouri and Kansas River, Armourdale Unit, Volume I, Appendix I, Dated 1951 - 1954.

5. Operations and Maintenance Manual, Record Drawings, Kansas Citys Flood Control Project, Missouri and Kansas River, Armourdale Unit, Volume II, Appendix I, Dated 1954 - 1976.

6. Operations and Maintenance Manual, Kansas Citys Flood Control Project, Missouri and Kansas River, Central Industrial Unit Kansas Section, Volume I, Dated 1980.

7. Operations and Maintenance Manual, Record Drawings, Kansas Citys Flood Control Project, Missouri and Kansas River, Central Industrial Unit Kansas Section, Volume I, Appendix I, Dated 1950 - 1955.

8. Operations and Maintenance Manual, Record Drawings, Kansas Citys Flood Control Project, Missouri and Kansas River, Central Industrial Unit Kansas Section, Volume II, Appendix I, Dated 1979.

9. Operations and Maintenance Manual, Kansas Citys Flood Control Project, Missouri and Kansas River, Central Industrial Unit Missouri Section, Volume I, Dated 1981.

10. Operations and Maintenance Manual, Record Drawings, Kansas Citys Flood Control Project, Missouri and Kansas River, Central Industrial Unit Missouri Section, Appendix I, Dated 1948 - 1955.

11. Operations and Maintenance Manual, Kansas Citys Flood Control Project, Missouri and Kansas River, East Bottoms Unit, Volume I, Dated 1978.

12. Operations and Maintenance Manual, Record Drawings, Kansas Citys Flood Control Project, Missouri and Kansas River, East Bottoms Unit, Volume I, Appendix II, Dated 1950 - 1974.

13. Operations and Maintenance Manual, Kansas Citys Flood Control Project, Missouri and Kansas River, Fairfax-Jersey Creek Unit, Volume I, Dated 1979.

14. Operations and Maintenance Manual, Record Drawings, Kansas Citys Flood Control Project, Missouri and Kansas River, Fairfax-Jersey Creek Unit, Volume I, Appendix I, Dated 1944 - 1955.

15. Operations and Maintenance Manual, Record Drawings, Kansas Citys Flood Control Project, Missouri and Kansas River, Fairfax-Jersey Creek Unit, Volume II, Appendix I, Dated 1954 - 1955.

16. Kansas Citys Flood Control Project, Definite Project Report on Fairfax-Jersey Creek Unit, Supplement on Interior Drainage, Dated 1952.

17. Operations and Maintenance Manual, Kansas Citys Flood Control Project, Missouri and Kansas River, North Kansas City Unit Lower Section, Volume I, Dated 1978.

18. Operations and Maintenance Manual, Record Drawings, Kansas Citys Flood Control Project, Missouri and Kansas River, North Kansas City Unit Lower Section, Volume I, Appendix I, Dated 1948 - 1979.

19. Kansas Citys Flood Control Project, Definite Project Report on North Kansas City Unit Unit, Supplement on Interior Drainage, Dated 1951.

20. Ang, A., and Tang, W., (1975), *Probability Concepts in Engineering Planning and Design* (Vol. I). New York: John Wiley & Sons, Inc.

21. Baecher, G. B., & J. J. Christian (2000), "Uncertainty, Probability, and Geotechnical Data," paper presented at Performance Confirmation of Constructed Geotechnical Facilities, ASCE, Amherst, MA; April 9-12.

22. Hunt, R. E., (1986), *Geotechnical Engineering Analysis and Evaluation,* New York: McGraw-Hill Book Company.

23. Reese, L. C., Wang, S. T., & Arrellaga, J., (1998), "Computer Program APILE Plus – A Program for the Analysis of the Axial Capacity of Driven Piles" ENSOFT, INC., Austin, TX.

24. US Army Corps of Engineers (1999), *Reconnaissance Report – Kansas Citys, Missouri and Kansas Flood Damage Reduction Project*, Kansas City District.

25. Wolff, T. F., (1985), "Analysis and Design of Embankment Dam Slopes: A Probabilistic Approach", Doctoral Dissertation presented to Purdue University, West Lafayette, IND.

26. Wright, S. G., (1999), "UTEXAS 4 – A Computer Program for Slope Stability Calculations", prepared for the Department of the Army, U. S. Army Corps of Engineers, Washington D. C.

A-4.11 SUPPLEMENTAL EXHIBITS AND TABLES

EXHIBIT 1

Armourdale Levee Kansas City Phase 2 Feasibility
Thursday, December 28, 2006
Levee Foundation Information - EXISTING CONDITIONS, Water to Top of Levee

Blanket Unit Weight = 115.0 pcf (saturated)

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_{bl}	Permeability Ratio Land Side k_f/k_{bl}	Impervious Blanket Thickness (ft) River Side z_{bl}	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
5+00 UE to 30+00	774.0	746	770	735	735	671	4.0	300	300	11	23	35	64	260	100	1000	0.002176	0.001220	235	820	2.84	0.08	0.84	10.39	OK
30+00 to 47+00	773.5	746	760	730	730	671	13.5	300	300	16	23	30	59	200	100	1000	0.001879	0.001372	191	729	9.65	0.32	0.84	2.62	OK
47+00 to 57+00	773.3	750	762	740	740	670	11.3	300	300	10	16	22	70	200	100	1000	0.002182	0.001471	188	680	7.94	0.36	0.84	2.34	OK
57+00 to 62+00	773.0	750	762	735	735	670	11.0	300	300	15	21	27	65	350	100	1000	0.001849	0.001378	308	726	7.04	0.26	0.84	3.23	OK
62+00 to 66+00	773.3	750	762	740	740	671	11.3	400	400	10	16	22	69	300	50	1000	0.001903	0.001283	271	779	8.00	0.36	0.84	2.32	OK
66+00 to 73+00	773.0	755	760	748	748	671	13.0	300	300	7	9.5	12	77	175	50	1000	0.002487	0.001899	165	526	9.23	0.77	0.84	1.10	FS below current criteria
73+00 to 79+00	772.5	753	760	732	745	671	12.5	300	300	21	18	15	61	75	50	1000	0.001613	0.001909	75	524	10.10	0.67	0.84	1.25	FS below current criteria
79+00 to 86+00	772.0	750	760	730	730	667	12.0	400	400	20	25	30	63	100	75	1000	0.001409	0.001150	99	869	10.00	0.33	0.84	2.53	OK
86+00 to 93+00	772.0	750	760	735	735	667	12.0	400	400	15	20	25	68	75	100	1000	0.001566	0.001213	75	825	9.90	0.40	0.84	2.13	OK
93+00 to 100+00	771.0	745	760	730	730	667	11.0	250	250	15	22.5	30	63	75	125	1000	0.002057	0.001455	74	687	8.53	0.28	0.84	2.97	OK
100+00 to 116+00	770.5	742	758	732	732	667	12.5	350	350	10	18	26	65	150	60	1000	0.002097	0.001300	145	769	9.87	0.38	0.84	2.22	sand layer between ground surface and 5 feet below ground surface not considered in analysis. Restoration drawings indicate 5' minimum impervious layer on riverside to el 735 to cut off seepage through sand layer. OK
116+00 to 130+00	770.0	744	756	733	733	667	14.0	350	350	11	17	23	66	250	125	1000	0.001984	0.001372	231	729	9.40	0.41	0.84	2.06	OK
130+00 to 175+00	769.0	742	760	730	730	672	9.0	300	300	12	21	30	58	225	100	1000	0.002188	0.001384	208	722	6.31	0.21	0.84	4.01	sand lenses between ground surface and 755ish not considered as seepage path. Restoration drawings indicate 5' minimum impervious layer on riverside to el 740 to cut off seepage through sand lenses. OK
175+00 to 185+00	768.0	742	762	725	725	655	6.0	300	300	17	27	37	70	130	100	1000	0.001674	0.001134	128	881	4.77	0.13	0.84	6.54	OK
185+00 to 195+00	767.5	740	752	728	728	668	8.5	300	300	12	18	24	60	80	100	1000	0.002152	0.001521	79	657	6.68	0.28	0.84	3.03	existing relief wells and buildings not considered in analysis. there is apparent sand pockets on the landside, but were ignored in analysis as rehab drawings indicate impervious fill on riverside to elevation 740ish. additional borings may be needed.
195+00 to 205+00	767.0	740	752	728	728	668	10.0	300	300	12	18	24	60	80	100	1000	0.002152	0.001521	79	657	7.86	0.33	0.84	2.57	existing relief wells and buildings not considered in analysis. there is apparent sand pockets on the landside below the extent of the impervious fill on the river side el 745, for the sake of this analysis, it is assumed they are not continuous under the levee. additional borings may be needed.
205+00 to 215+00	766.5	740	752	728	728	668	14.5	300	300	12	18	24	60	80	100	1000	0.002152	0.001521	79	657	11.39	0.47	0.84	1.78	existing relief wells and buildings not considered in analysis. there is apparent sand pockets on the landside, but were ignored in analysis as rehab drawings indicate impervious fill on riverside to elevation 742ish. additional borings may be needed
215+00 to 225+00	766.0	740	748	722	722	668	18.0	300	300	18	22	26	54	80	100	1000	0.001852	0.001541	79	649	14.10	0.54	0.84	1.55	existing relief wells and buildings not considered in analysis.
225+00 to 235+00	765.5	740	748	725	730	668	17.5	300	300	15	16.5	18	57	80	100	1000	0.001974	0.001802	79	555	13.22	0.73	0.84	1.15	existing relief wells and buildings not considered in analysis.
235+00 to 245+00	765.0	740	749	730	730	668	16.0	300	300	10	14.5	19	62	80	100	1000	0.002319	0.001682	79	594	12.30	0.65	0.84	1.30	existing relief wells and buildings not considered in analysis.
245+00 to 251+00	764.5	738	751	730	730	653	13.5	200	200	8	14.5	21	77	75	95	1000	0.002849	0.001758	74	569	10.41	0.50	0.84	1.70	existing relief wells and buildings not considered in analysis
251+00 to 254+00	763.5	741	752	740	740	663	11.5	350	350	1	6.5	12	77	100	125	1000	0.006091	0.001758	89	569	8.35	0.70	0.84	1.21	blanket appears to thin out here based on riverside borings. additional borings on the land side may indicate a thicker blanket on the land side. if this is the case, underseepage might not be a problem. FS below current criteria.
254+00 to 265+00	763.2	744	752	728	728	671	11.2	400	400	16	20	24	57	125	50	1000	0.001656	0.001352	123	740	9.07	0.38	0.84	2.23	analysis does not include shallow sand lenses, as they have been cut off as indicated by rehab drawings showing impervious fill down to 740 on river side. field investigation indicated slot area is filled to 752
265+00 to 282+00	762.5	742	750	725	725	678	12.5	300	300	17	21	25	47	100	50	1000	0.002043	0.001684	99	594	10.00	0.40	0.84	2.11	existing wells not considered in analysis . Slot not considered in blanket theory calcs. HGL = 760. Slot El = 739. h_h in slot = 760-739-5 (per DM 3) = 16. $z_{bl\ slot}$ = 739-725 = 14. i_{slot} = 1.14. **FS_{slot} = 0.74 w/o wells. with wells - FS = 0.9**
282+00 to 286+00	762.0	738	746	717	717	678	16.0	400	400	21	25	29	39	75	75	1000	0.001747	0.001487	75	673	13.09	0.45	0.84	1.87	OK
286+00 to 296+00	761.7	738	750	725	725	678	11.7	400	400	13	19	25	47	100	50	1000	0.002023	0.001459	99	686	9.62	0.38	0.84	2.19	OK
296+00 to 303+00	761.5	740	742	728	728	678	19.5	400	400	12	13	14	50	75	75	1000	0.002041	0.001890	74	529	15.21	1.09	0.84	0.78	existing wells not considered in analysis. **FS = 1.0 - 1.4 with wells.** below current criteria
303+00 to 313+00	760.5	740	747	730	730	678	13.5	400	400	10	13.5	17	52	50	100	1000	0.002193	0.001682	50	595	10.78	0.63	0.84	1.33	FS below current criteria
313+00 to 40+00LE	760.5	745	750	725	725	700	10.5	400	400	20	22.5	25	25	110	100	1000	0.002236	0.002000	108	500	7.42	0.30	0.84	2.84	sand layers are located on riverside and landside between elevations 738ish and ground surface. Restoration drawings indicate 5' minimum impervious layer on riverside to el 740 to cut off seepage through sand layer. performance during 93 in this area indicates the layers do not pose a problem. OK
40+00 LE to 59+60LE	760.0	745	758	725	725	700	2.0	400	400	20	26.5	33	25	110	100	1000	0.002236	0.001741	108	574	1.47	0.04	0.84	18.94	sand layers are located on riverside and landside between elevations 740ish and ground surface. Restoration drawings indicate 5' minimum impervious layer on riverside to el 740 to cut off seepage through sand layer. performance during 93 in this area indicates the layers do not pose a problem. OK

EXHIBIT 2
Typical shape of the normal probability distribution function showing the expected value or mean, E[X]

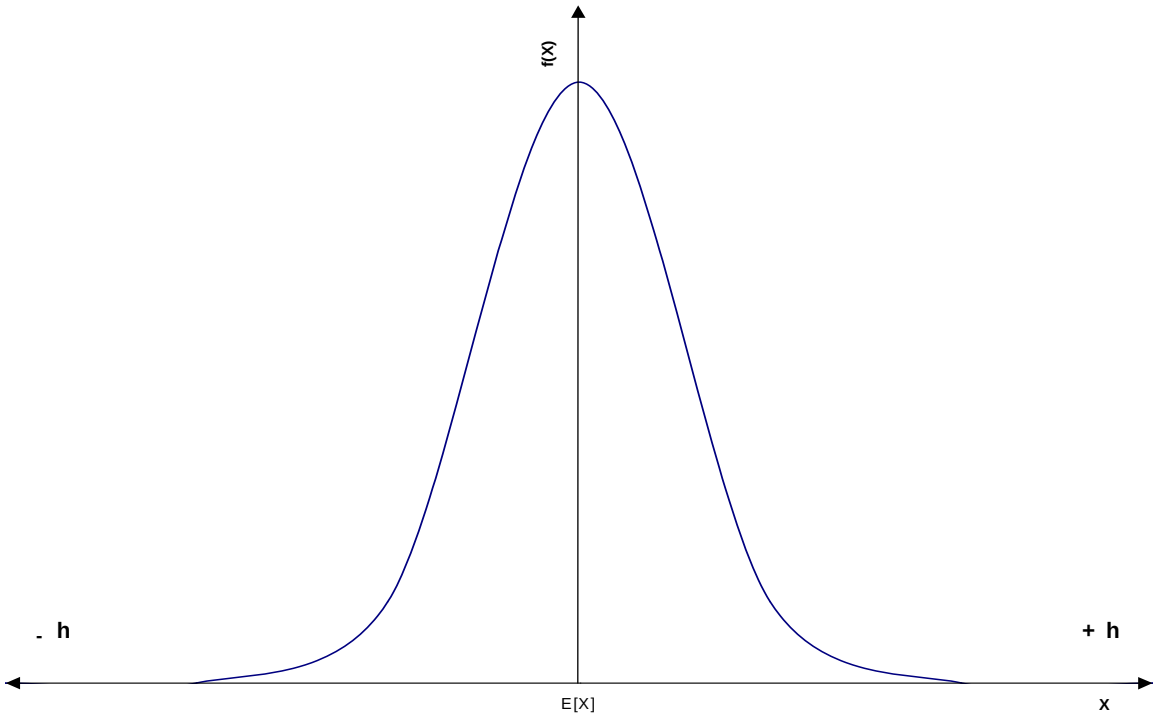


EXHIBIT 3
Typical shape of the log-normal distribution function showing the expected value, E[X]

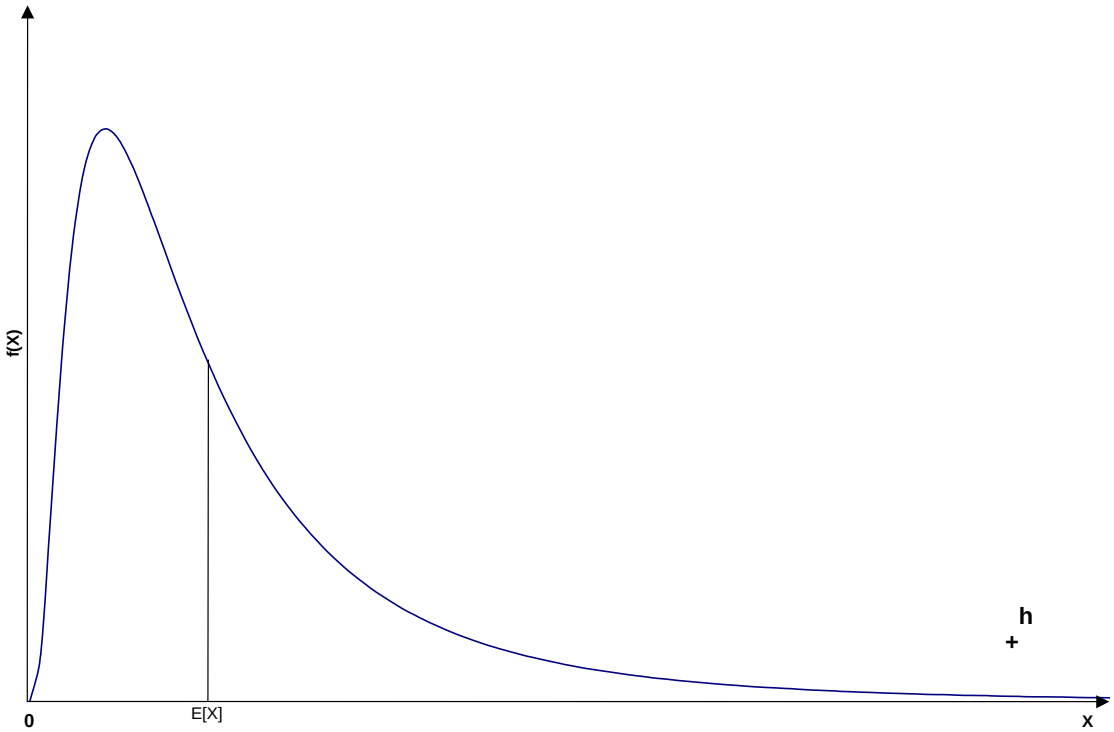


EXHIBIT 4
Hypothetical normal probability distribution showing the probabilistic parameters

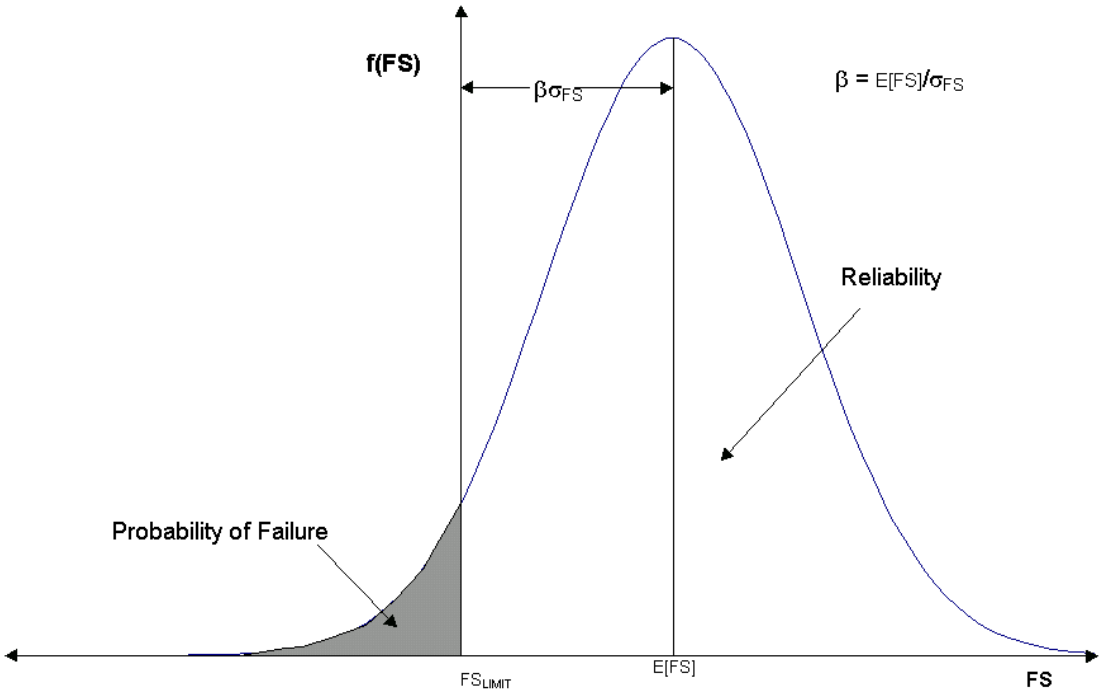


EXHIBIT 5
Normal probability distribution for the natural log of the factor of safety, assuming that the factor of safety is log-normally distributed

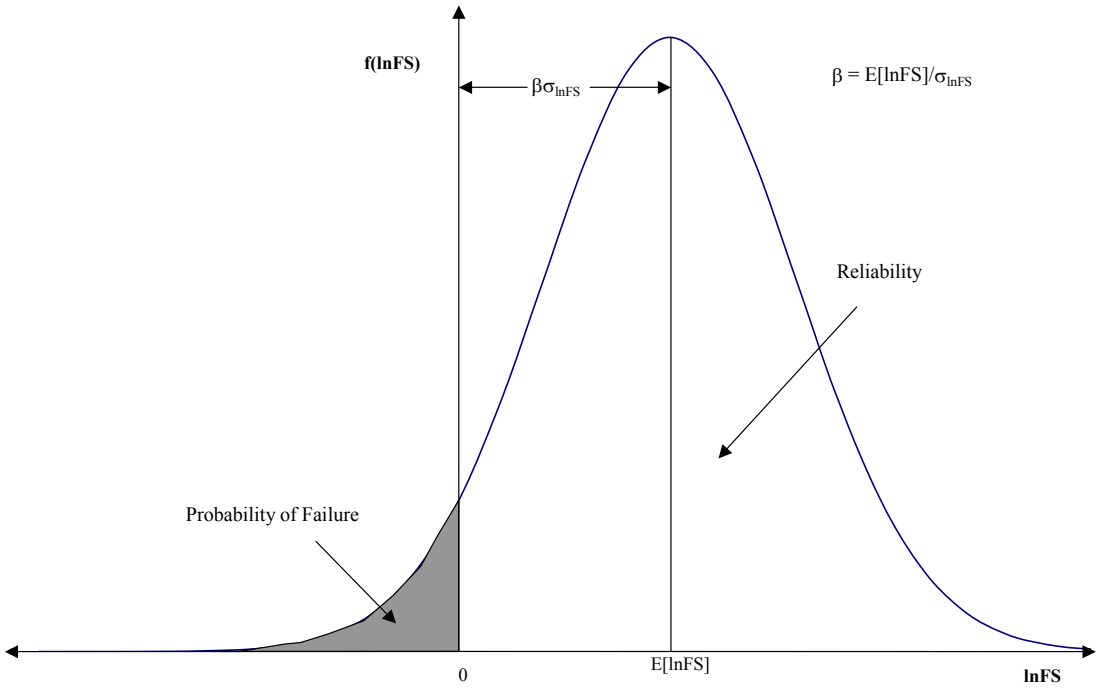


EXHIBIT 6
Normal probability distribution for the natural log of the hydraulic gradient, assuming that the hydraulic gradient is log-normally distributed where the failure gradient is defining the limit state

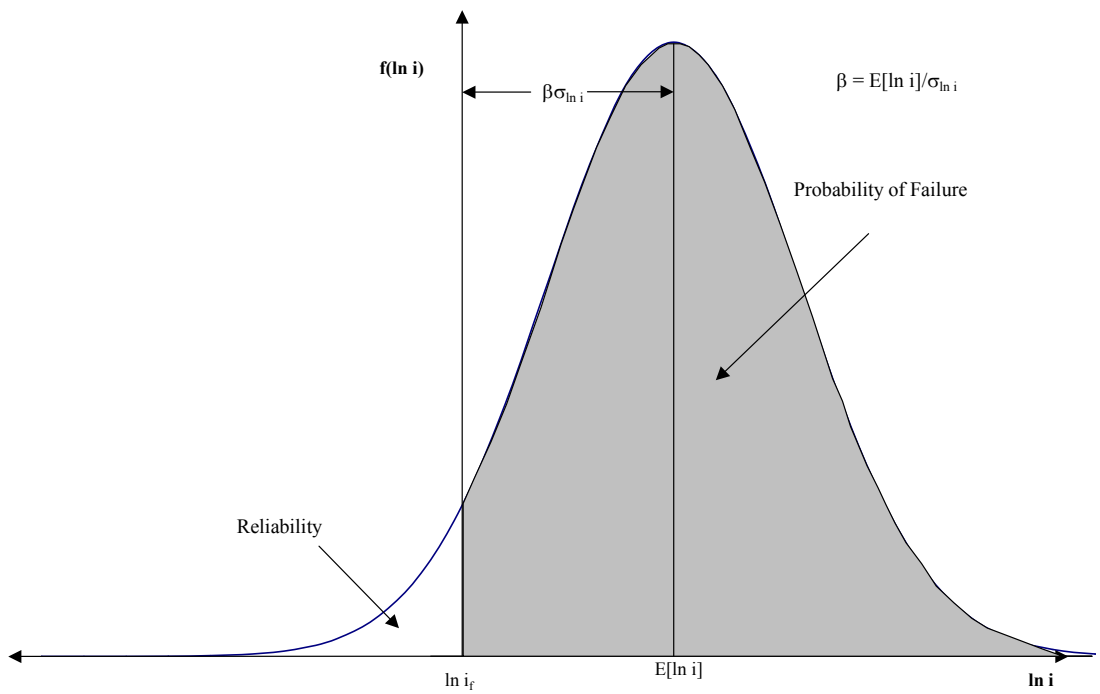


EXHIBIT 7
The probability distribution curve illustrating the assumptions used in developing the Taylor Series Approximation

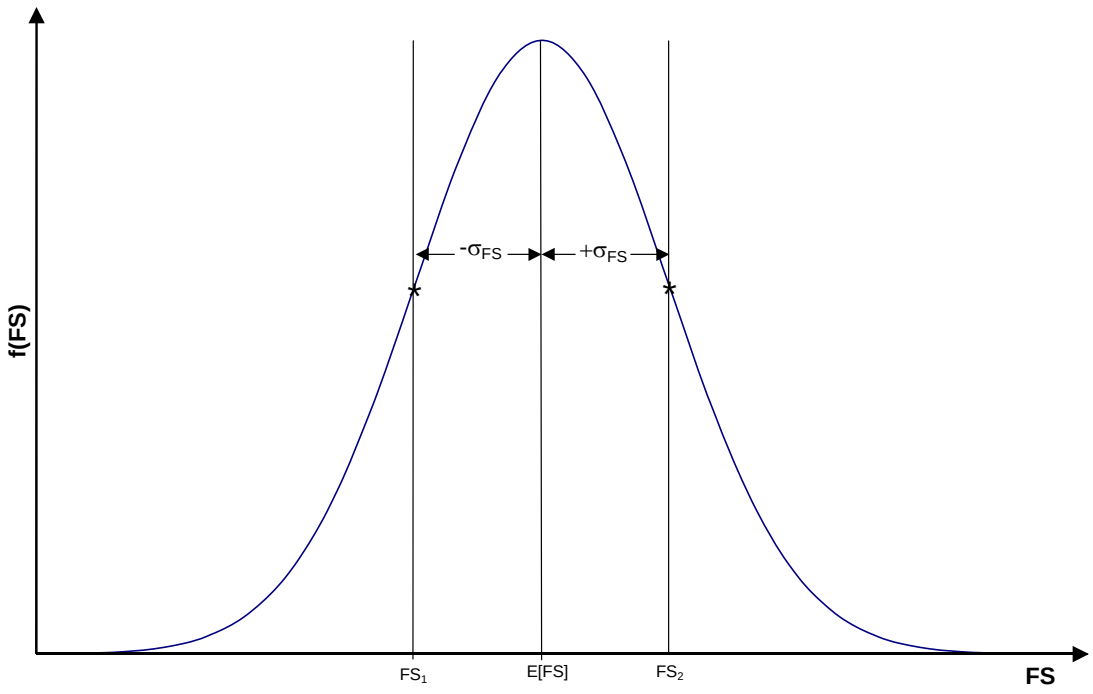


EXHIBIT 8
Expected Value of Factor of Safety and Probability of Failure Relation

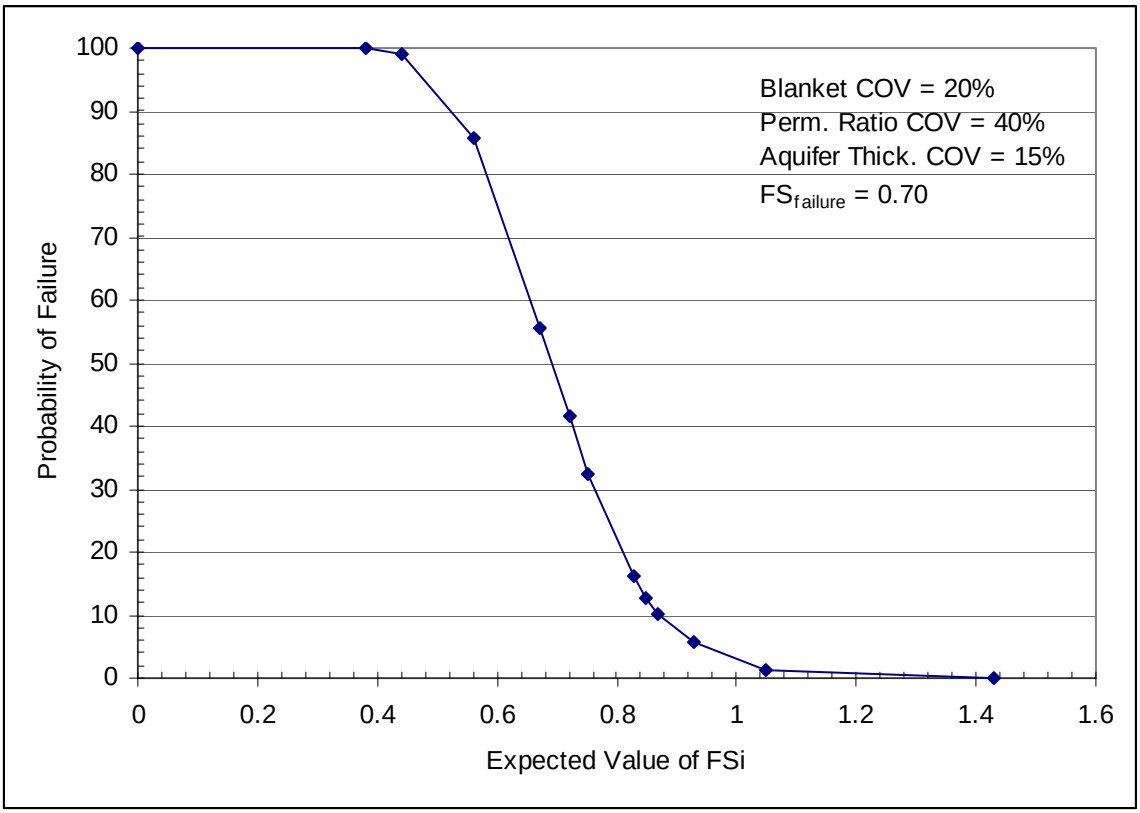


EXHIBIT 9
Probability of Failure Due to Underseepage Failure at Station 276+00

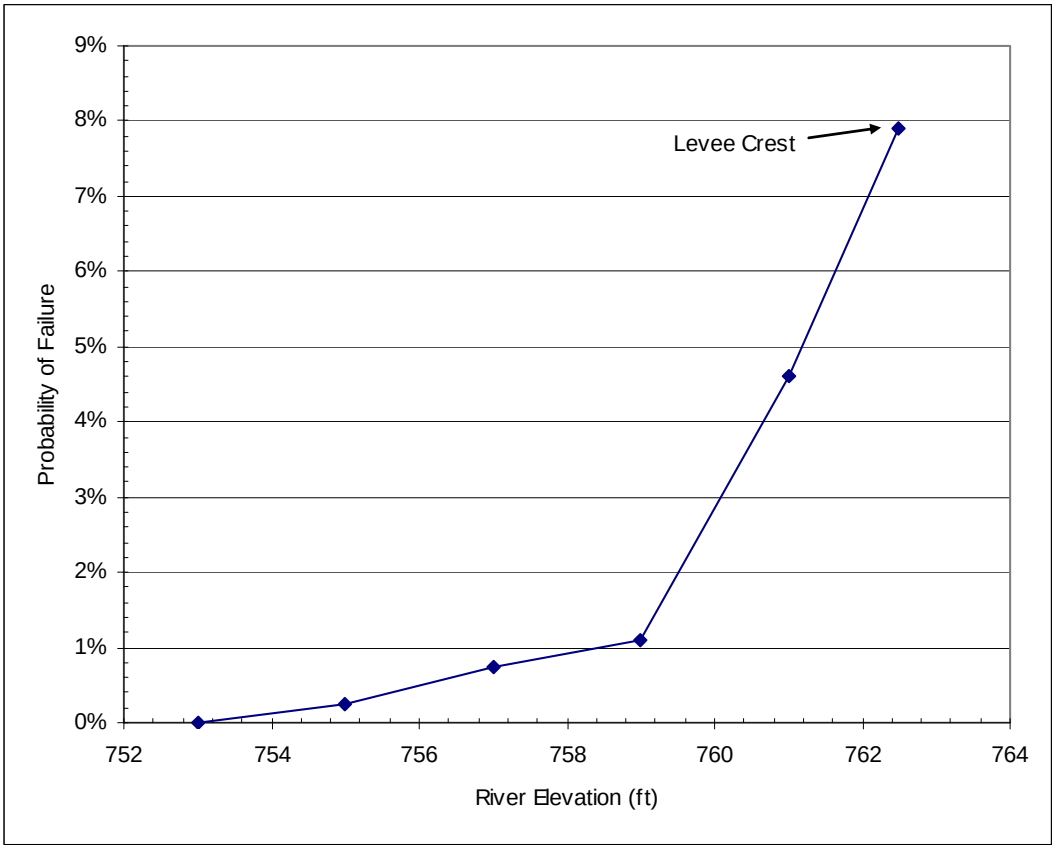


EXHIBIT 10
Probability of Failure Due to Stability Failure at Station 222+0

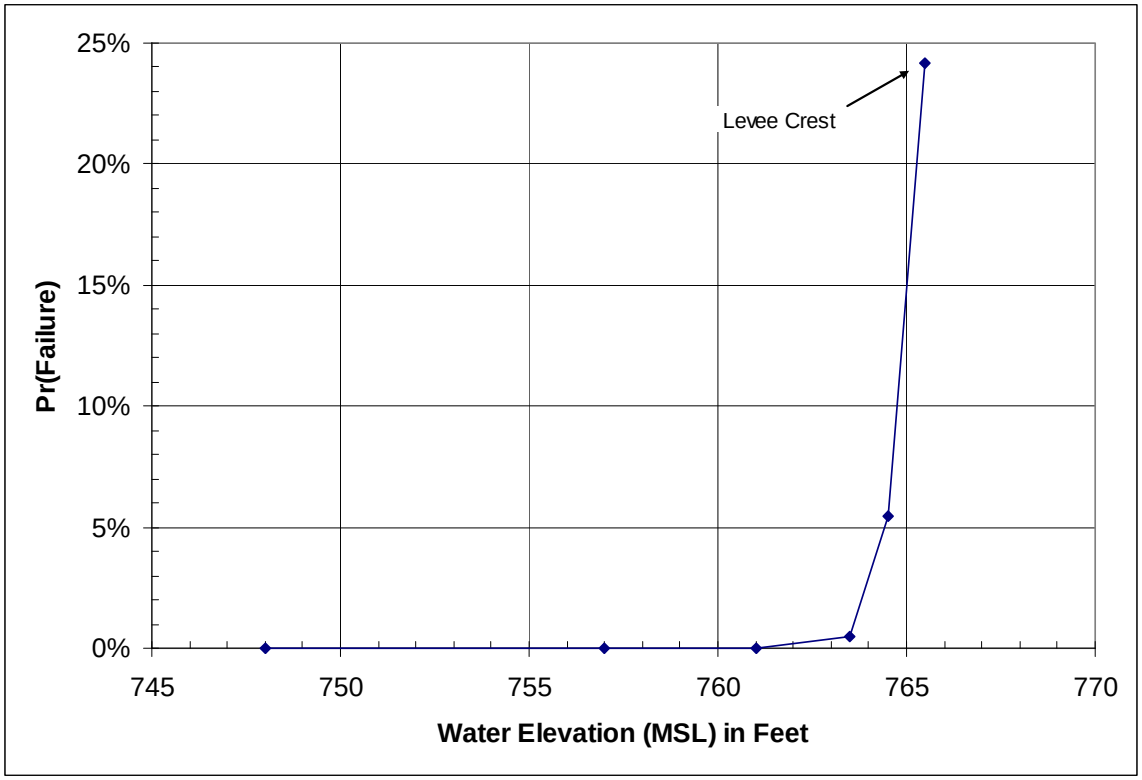


EXHIBIT 11

```
HEADING
    Kansas City Levees - Armourdale Unit
    Station 7+00UE Stability Analysis
    End of Construction Analysis - Riverside Fill
    March 2007

PROFILE LINES
         3     1 Levee, moist
               256.08     774.00
               263.00     774.00
               290.00     765.00

         2     2 Levee, Raise, saturated
               110.00     750.00
               241.00     778.00
               256.08     774.00

         4     2 Levee saturated
               199.00     756.00
               253.00     774.00
               256.08     774.00
               290.00     765.00

         5     4 Foundation, Silt, saturated
                  .00     750.00
               110.00     750.00
               199.00     756.00
               290.00     765.00
               500.00     765.00

         6     5 Foundation, Sand, saturated
                  .00     735.00
               500.00     735.00

         7     6 Bedrock
                  .00     670.00
               500.00     670.00

         1     1 Levee Raise, moist
               241.00     778.00
               251.00     778.00
               263.00     774.00

         8     1 Berm
               272.00     771.00
               284.00     771.00
               304.50     765.00


MATERIAL PROPERTIES
     1 Levee, moist
          115.00 Unit Weight
          Conventional Shear
             1000.00        .00
          No Pore Pressure
     2 Levee, saturated
          120.00 Unit Weight
          Conventional Shear
             1000.00        .00
          No Pore Pressure
     4 Foundation, Silt, saturated
```

```
          115.00 Unit Weight
          Conventional Shear
              500.00        .00
          No Pore Pressure
     5 Foundation, Sand, saturated
          120.00 Unit Weight
          Conventional Shear
                 .00      32.00
          Piezometric Line
          2
     6 Bedrock
          150.00 Unit Weight
          Very Strong

PIEZOMETRIC LINES
         2      62.40 Foundation Sand Pressures
                  .00     750.00
               143.00     750.00
               199.00     756.00
               290.00     765.00
               500.00     765.00


INTERPOLATION DATA
Pore Water Pressure
            .00     750.00   1747.20          4
            .00     735.00   2683.20          4
          40.00     750.00   1747.20          4
          40.00     735.00   2652.00          4
          80.00     750.00   1747.20          4
          80.00     735.00   2620.80          4
         110.00     750.00   1747.20          4
         110.00     735.00   2597.10          4
         143.00     750.00   1747.20          4
         143.00     735.00   2570.90          4
         170.00     752.89   1538.70          4
         170.00     744.00   2040.70          4
         170.00     735.00   2549.70          4
         199.00     756.00   1322.50          4
         199.00     746.00   1996.20          4
         199.00     735.00   2527.20          4
         220.00     758.08   1117.80          4
         220.00     735.00   2510.30          4
         220.00     746.00   1875.20          4
         241.00     760.15   1035.10          4
         241.00     748.00   1741.40          4
         241.00     735.00   2496.00          4
         266.00     762.65    594.60          4
         266.00     748.00   1589.80          4
         266.00     735.00   2474.20          4
         290.00     765.00       .00          4
         290.00     750.00   1227.70          4
         290.00     735.00   2455.40          4
         330.00     765.00       .00          4
         330.00     750.00   1212.10          4
         330.00     735.00   2424.20          4
         370.00     765.00       .00          4
         370.00     750.00   1198.10          4
         370.00     735.00   2396.20          4
         410.00     765.00       .00          4
         410.00     750.00   1184.10          4
```

NOT USED FOR THIS ANALYSIS

```
        410.00    735.00   2368.10          4
        450.00    765.00       .00          4
        450.00    750.00   1171.60          4
        450.00    735.00   2343.10          4
        500.00    765.00       .00          4
        500.00    750.00   1161.90          4
        500.00    735.00   2323.80          4


ANALYSIS/COMPUTATION
     Circular Search 1
        180.00    820.00      1.00    670.00
     Radius
         80.00
SINgle-stage Computations
TRIal max
 100
SAVe n most
 150
LONg-form output
CRAck
         12.00         D
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```

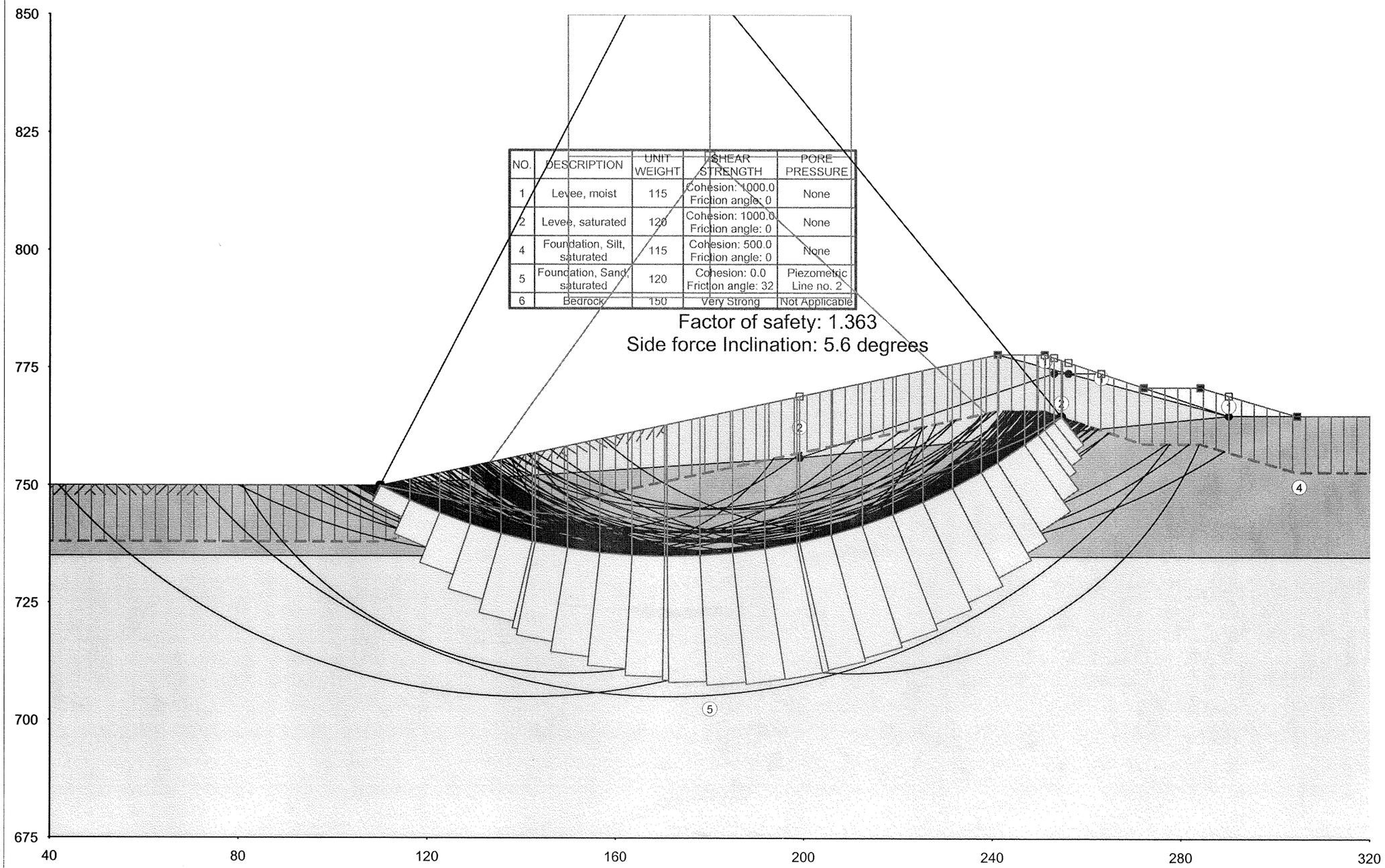
29 MAR 2007 (1)

END OF CONSTRUCTION

Kansas City Levees - Armourdale Unit

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Levee, moist	115	Cohesion: 1000.0 Friction angle: 0	None
2	Levee, saturated	120	Cohesion: 1000.0 Friction angle: 0	None
4	Foundation, Silt, saturated	115	Cohesion: 500.0 Friction angle: 0	None
5	Foundation, Sand, saturated	120	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 2
6	Bedrock	150	Very Strong	Not Applicable

Factor of safety: 1.363
Side force Inclination: 5.6 degrees



Date: Thu Mar 29 2007 Filename: K:\MissionProjects\civ\kansas_citys\Geotech\Armourdale FC\Slope Stability\3 Ft Raise\Kuz ARM Sta 7+00 UE WW\7 construction-riverside Final Design.UT4 Time: 12:20:55

```
HEADING
    Kansas City Levees - Armourdale Unit
    Station 7+00UE Stability Analysis
    Steady Seepage Analysis With Landside Berm - Riverside Fill
    March 2007

PROFILE LINES
         3     1 Levee, moist
               256.08    774.00
               263.00    774.00
               290.00    765.00

         2     2 Levee, Raise, saturated
               110.00    750.00
               241.00    778.00
               256.08    774.00

         4     2 Levee saturated
               199.00    756.00
               253.00    774.00
               256.08    774.00
               290.00    765.00

         5     4 Foundation, Silt, saturated
                  .00    750.00
               110.00    750.00
               199.00    756.00
               290.00    765.00
               500.00    765.00

         6     5 Foundation, Sand, saturated
                  .00    735.00
               500.00    735.00

         7     6 Bedrock
                  .00    670.00
               500.00    670.00

         1     1 Levee Raise, moist
               241.00    778.00
               251.00    778.00
               263.00    774.00

         8     1 Berm
               272.00    771.00
               284.00    771.00
               304.50    765.00


MATERIAL PROPERTIES
     1 Levee, moist
          115.00 Unit Weight
          Conventional Shear
                 .00     29.00
          No Pore Pressure
     2 Levee, saturated
          120.00 Unit Weight
          Conventional Shear
                 .00     29.00
          Piezometric Line
          1
```

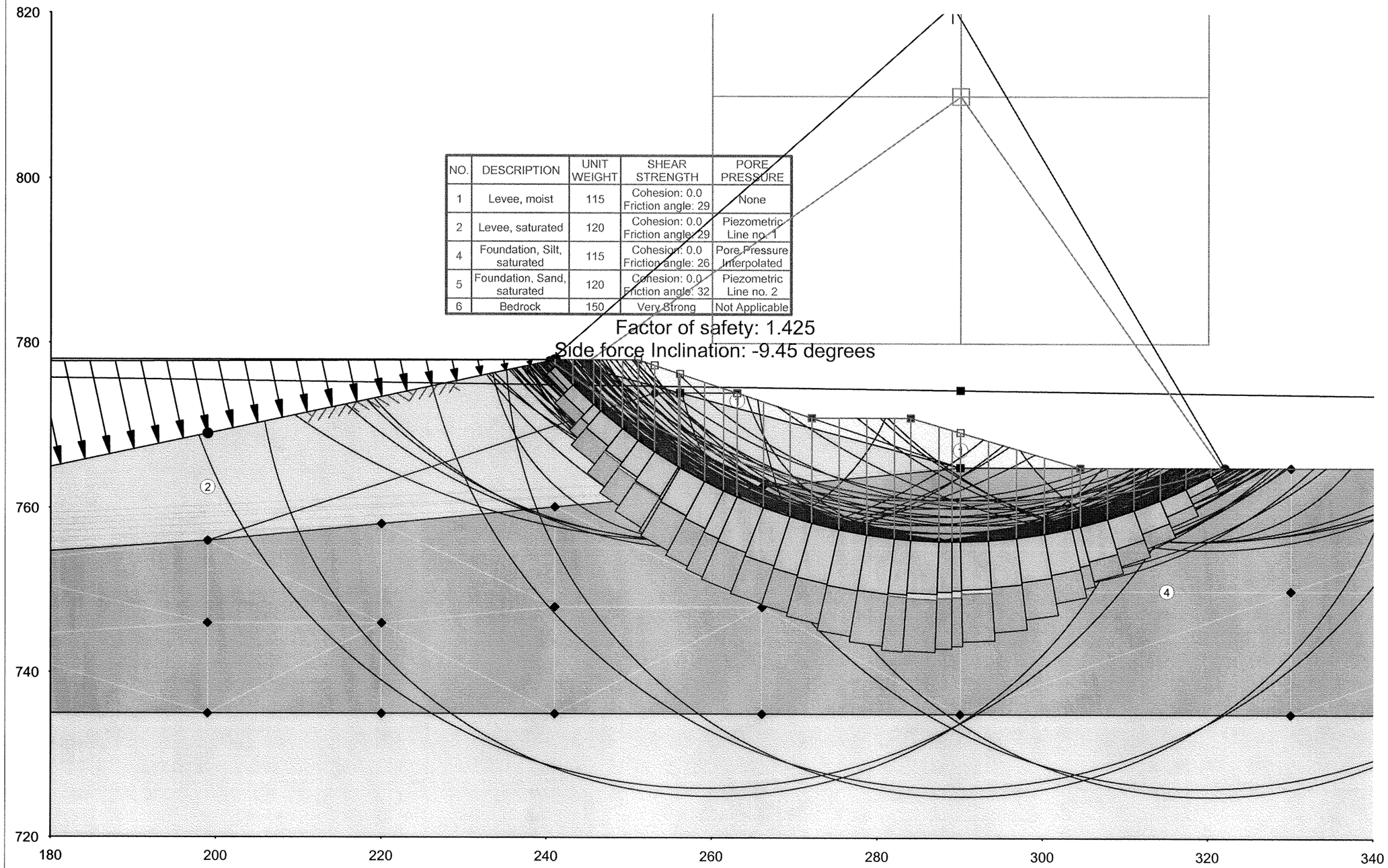
```
     4 Foundation, Silt, saturated
          115.00 Unit Weight
          Conventional Shear
                 .00      26.00
          Interpolate Pore Water Pressure
     5 Foundation, Sand, saturated
          120.00 Unit Weight
          Conventional Shear
                 .00      32.00
          Piezometric Line
          2
     6 Bedrock
          150.00 Unit Weight
          Very Strong

PIEZOMETRIC LINES
         1      62.40 1 - Line of Seepage
                  .00     778.00
               241.00     778.00
               256.08     774.00
               290.00     765.00
               500.00     765.00

         2      62.40 2 - Foundation Sand Pressures
                  .00     778.00
               290.00     774.35
               370.00     773.40
               500.00     772.04


DISTRIBUTED LOADS
      1
INTERPOLATION DATA
Pore Water Pressure
            .00     750.00   1747.20          4
            .00     735.00   2683.20          4
          40.00     750.00   1747.20          4
          40.00     735.00   2652.00          4
          80.00     750.00   1747.20          4
          80.00     735.00   2620.80          4
         110.00     750.00   1747.20          4
         110.00     735.00   2597.10          4
         143.00     750.00   1747.20          4
         143.00     735.00   2570.90          4
         170.00     752.89   1538.70          4
         170.00     744.00   2040.70          4
         170.00     735.00   2549.70          4
         199.00     756.00   1322.50          4
         199.00     746.00   1896.20          4
         199.00     735.00   2527.20          4
         220.00     758.08   1177.80          4
         220.00     735.00   2510.30          4
         220.00     746.00   1875.20          4
         241.00     760.15   1036.10          4
         241.00     748.00   1741.40          4
         241.00     735.00   2496.00          4
         266.00     762.63    594.60          4
         266.00     748.00   1589.80          4
         266.00     735.00   2474.20          4
         290.00     765.00       .00          4
         290.00     750.00   1227.70          4
```

```
        290.00    735.00   2455.40          4
        330.00    765.00       .00          4
        330.00    750.00   1212.10          4
        330.00    735.00   2424.20          4
        370.00    765.00       .00          4
        370.00    750.00   1198.10          4
        370.00    735.00   2396.20          4
        410.00    765.00       .00          4
        410.00    750.00   1184.10          4
        410.00    735.00   2368.10          4
        450.00    765.00       .00          4
        450.00    750.00   1171.60          4
        450.00    735.00   2343.10          4
        500.00    765.00       .00          4
        500.00    750.00   1161.90          4
        500.00    735.00   2323.80          4


ANALYSIS/COMPUTATION
     Circular Search 1
         290.00    810.00      1.00    670.00
     Radius
          55.00
SINgle-stage Computations
RIGht Face of Slope
SAVe n most
 150
LONg-form output
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```

29 MAR 2007

STEADY SEEPAGE

Kansas City Levees - Armourdale Unit



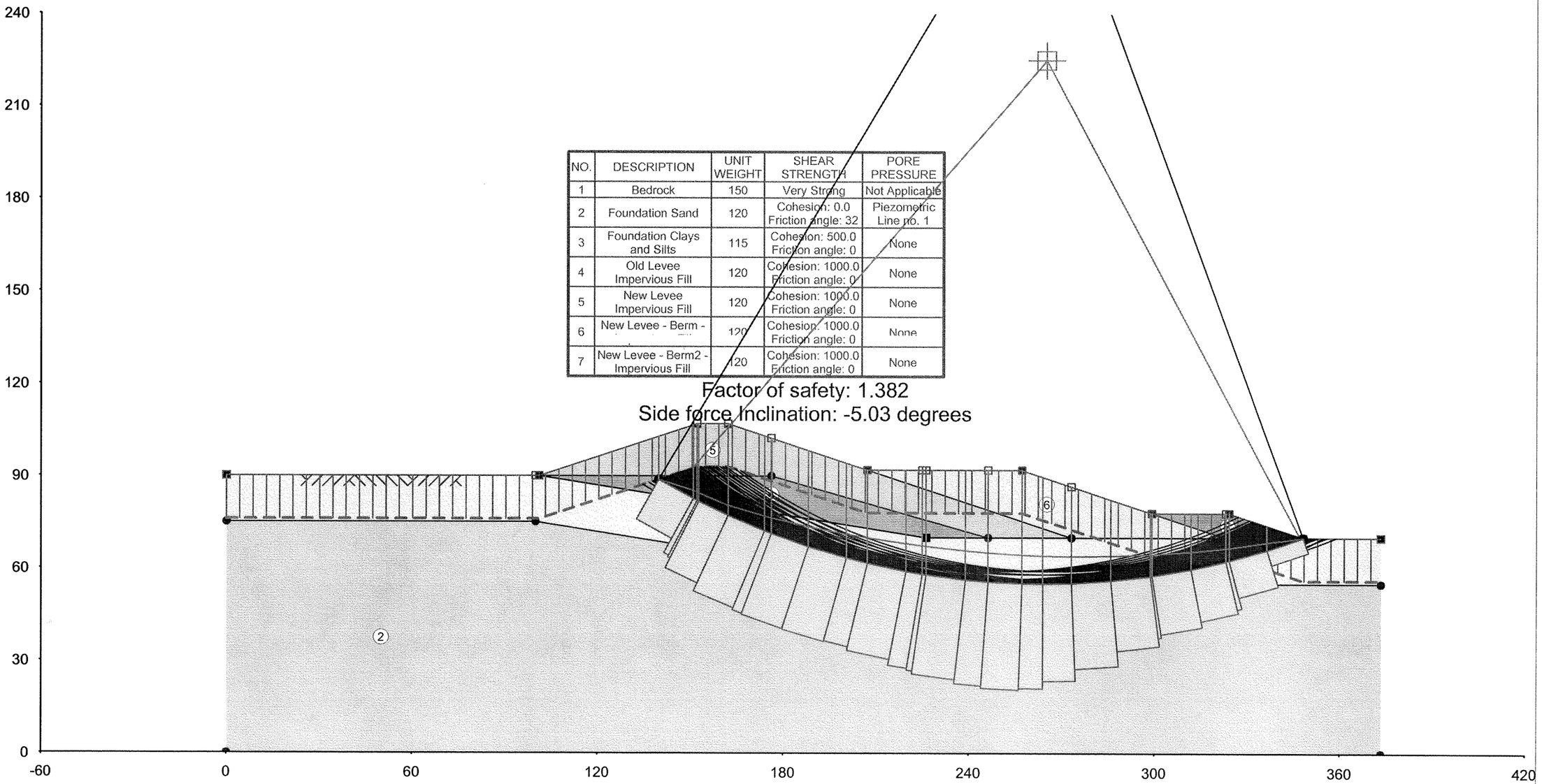
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Levee, moist	115	Cohesion: 0.0 Friction angle: 29	None
2	Levee, saturated	120	Cohesion: 0.0 Friction angle: 29	Piezometric Line no. 1
4	Foundation, Silt, saturated	115	Cohesion: 0.0 Friction angle: 26	Pore Pressure Interpolated
5	Foundation, Sand, saturated	120	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 2
6	Bedrock	150	Very Strong	Not Applicable

Date: Wed May 09 2007 Filename: K:\MissionProjects\civ\kansas_citys\Geotech\Armourdale FC\Slope Stability\3 Ft Raise\Kuz ARM Sta 7+00 UE WW\7 steady berm Final Design.UT4 Time: 07:06:55

```
HEADING
    KC Levees Phase 2
    Armourdale Levee Unit
    Station 60+00 to 75+00
    Levee Raise to Replace Existing Floodwall
    End of Construction Case

PROFILE LINES
         1     1 Bedrock
                  .00        .00
               373.20        .00

         2     2 Foundation Sand
                  .00      75.00
               100.00      75.00
               225.00      55.00
               373.20      55.00

         3     3 Foundation Clays and Silts
                  .00      90.00
               101.20      90.00
               226.20      70.00
               246.20      70.00
               373.20      70.00

         4     4 Old Levee Impervious Fill
               101.20      90.00
               176.20      90.00
               246.20      70.00

         5     5 New Levee Impervious Fill
               101.20      90.00
               152.20     107.00
               162.20     107.00
               207.20      92.00
               273.20      70.00

         6     6 New Levee - Berm - Impervious Fill
               207.20      92.00
               257.20      92.00
               299.20      78.00
               323.20      70.00

         7     7 New Levee - Berm2 - Impervious Fill
               299.20      78.00
               324.20      78.00
               348.20      70.00


MATERIAL PROPERTIES
     1 Bedrock
          150.00 Unit Weight
          Very Strong
     2 Foundation Sand
          120.00 Unit Weight
          Conventional Shear
                 .00      32.00
          Piezometric Line
          1
     3 Foundation Clays and Silts
          115.00 Unit Weight
```

```
          Conventional Shear
              500.00       .00
          No Pore Pressure
     4 Old Levee Impervious Fill
          120.00 Unit Weight
          Conventional Shear
             1000.00       .00
          No Pore Pressure
     5 New Levee Impervious Fill
          120.00 Unit Weight
          Conventional Shear
             1000.00       .00
          No Pore Pressure
     6 New Levee - Berm - Impervious Fill
          120.00 Unit Weight
          Conventional Shear
             1000.00       .00
          No Pore Pressure
     7 New Levee - Berm2 - Impervious Fill
          120.00 Unit Weight
          Conventional Shear
             1000.00       .00
          No Pore Pressure

PIEZOMETRIC LINES
         1     62.40 Groundwater
                 .00     90.00
               101.20     90.00
               226.20     70.00
               373.20     70.00


SLOPE GEOMETRY

ANALYSIS/COMPUTATION
     Circular Search 1
         265.00    225.00       .10       .00       .00
     Radius
         175.00
SINgle-stage Computations
RIGht Face of Slope
FACtor of safety
 1.5
ITEration limit
 1000
SAVe n most
 5000
LONg-form output
CRAck
          14.00         D
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```

KC Levees Phase 2



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Bedrock	150	Very Strong	Not Applicable
2	Foundation Sand	120	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 1
3	Foundation Clays and Silts	115	Cohesion: 500.0 Friction angle: 0	None
4	Old Levee Impervious Fill	120	Cohesion: 1000.0 Friction angle: 0	None
5	New Levee Impervious Fill	120	Cohesion: 1000.0 Friction angle: 0	None
6	New Levee - Berm -	120	Cohesion: 1000.0 Friction angle: 0	None
7	New Levee - Berm2 - Impervious Fill	120	Cohesion: 1000.0 Friction angle: 0	None

Factor of safety: 1.382
Side force inclination: -5.03 degrees

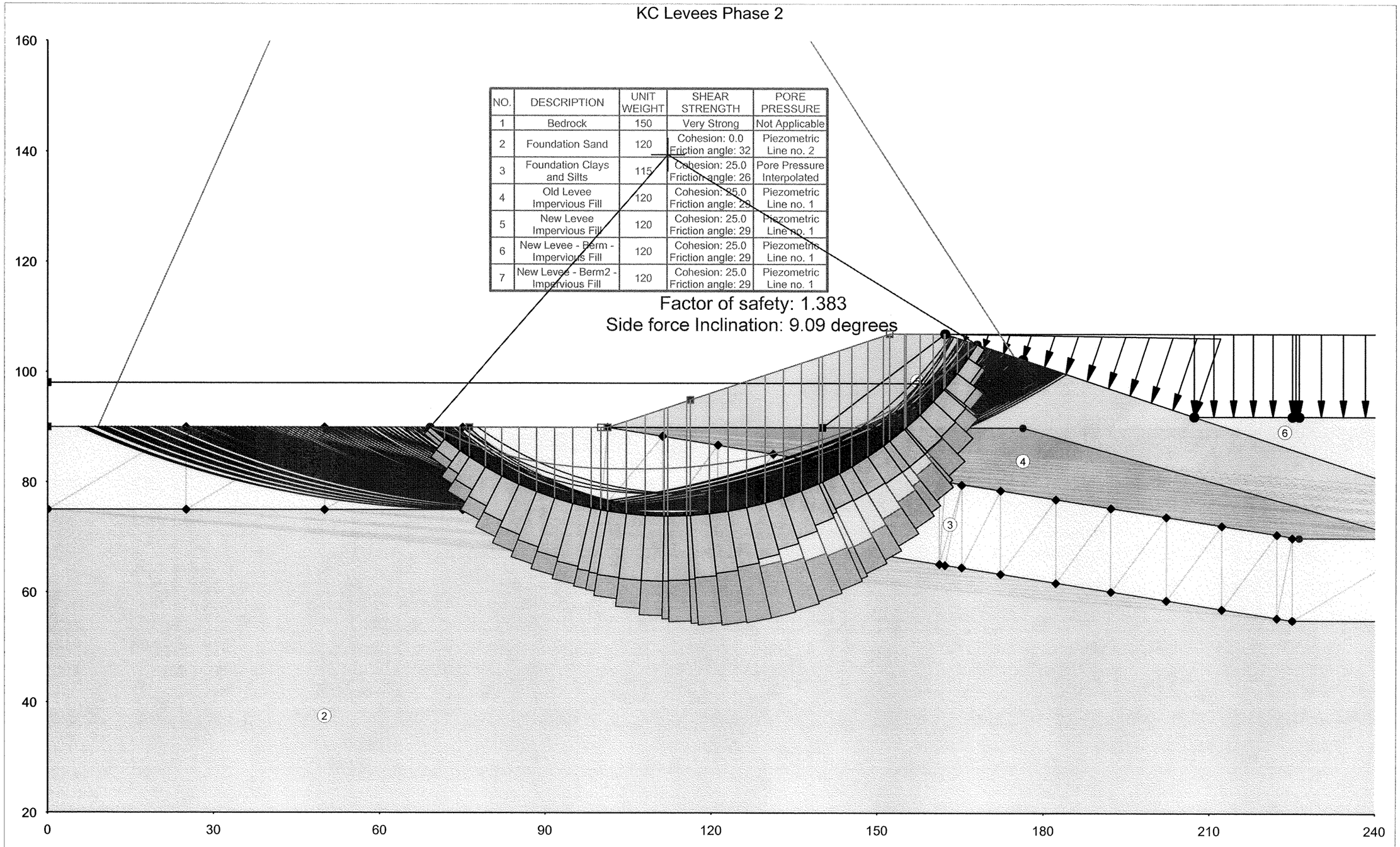
Filename: K:\MissionProjects\civ\kansas_citys\Geotech\Armourdale FC\Sta 60+00 to 79+00 Work Bellew\Armourdale 60+00 to 79+00\Armourdale 60+00 to 79+00 Stability N500+3\Levee Raise Analysis\Con Time: 10:32:57 sk.UT4

```
HEADING
    KC Levees Phase 2
    Armourdale Levee Unit
    Station 60+00 to 75+00
    Levee Raise to Replace Existing Floodwall
    Steady State Seepage Case - Full Head

PROFILE LINES
         1    1 Bedrock
                 .00        .00
              373.20        .00

         2    2 Foundation Sand
                 .00      75.00
              100.00      75.00
              225.00      55.00
              373.20      55.00

         3    3 Foundation Clays and Silts
                 .00      90.00
               76.20      90.00
              101.20      90.00
              226.20      70.00
              246.20      70.00
              373.20      70.00

         4    4 Old Levee Impervious Fill
              101.20      90.00
              176.20      90.00
              246.20      70.00

         5    5 New Levee Impervious Fill
              101.20      90.00
              116.20      95.00
              152.20     107.00
              162.20     107.00
              207.20      92.00
              273.20      70.00

         6    6 New Levee - Berm - Impervious Fill
              207.20      92.00
              257.20      92.00
              299.20      78.00
              323.20      70.00

         7    7 New Levee - Berm2 - Impervious Fill
              299.20      78.00
              324.20      78.00
              348.20      70.00


MATERIAL PROPERTIES
     1 Bedrock
          150.00 Unit Weight
          Very Strong
     2 Foundation Sand
          120.00 Unit Weight
          Conventional Shear
                 .00      32.00
          Piezometric Line
          2
```

```
     3 Foundation Clays and Silts
          115.00 Unit Weight
          Conventional Shear
               25.00     26.00
          Interpolate Pore Water Pressure
     4 Old Levee Impervious Fill
          120.00 Unit Weight
          Conventional Shear
               25.00     29.00
          Piezometric Line
          1
     5 New Levee Impervious Fill
          120.00 Unit Weight
          Conventional Shear
               25.00     29.00
          Piezometric Line
          1
     6 New Levee - Berm - Impervious Fill
          120.00 Unit Weight
          Conventional Shear
               25.00     29.00
          Piezometric Line
          1
     7 New Levee - Berm2 - Impervious Fill
          120.00 Unit Weight
          Conventional Shear
               25.00     29.00
          Piezometric Line
          1

PIEZOMETRIC LINES
         1      62.40 Conservative Line of Seepage
                  .00     90.00
               76.20     90.00
              101.20     90.00
              140.00     90.00
              162.20    107.00
              373.20    107.00

         2      62.40 Foundation Sand Pressures
                  .00     98.00
              162.00     98.00
              162.20    107.00
              373.20    107.00


SLOPE GEOMETRY

DISTRIBUTED LOADS
      1
INTERPOLATION DATA
Pore Water Pressure
             .00     90.00        .00          3
             .00     75.00    1465.00          3
           25.00     90.00        .00          3
           25.00     75.00    1435.00          3
           50.00     90.00        .00          3
           50.00     75.00    1465.00          3
           75.00     90.00        .00          3
           75.00     75.00    1435.00          3
          100.00     75.00    1435.00          3
```

```
101.20     90.00       .00        3
111.20     73.20   1548.00        3
111.20     88.40    100.00        3
121.20     71.60   1647.00        3
121.20     86.80    200.00        3
131.20     70.00   1747.00        3
131.20     85.20    300.00        3
141.20     68.40   1847.00        3
141.20     83.60    399.00        3
151.20     66.80   2053.00        3
151.20     82.00   1104.00        3
161.20     65.20   2565.00        3
161.20     80.40   1616.00        3
162.20     65.00   2650.00        3
162.20     80.00   1714.00        3
165.20     64.60   2650.00        3
165.20     79.60   1714.00        3
172.20     63.40   2720.00        3
172.20     78.60   1772.00        3
182.20     61.80   2820.00        3
182.20     77.00   1872.00        3
192.20     60.20   2920.00        3
192.20     75.40   1972.00        3
202.20     58.60   3020.00        3
202.20     73.80   2072.00        3
212.20     57.00   3120.00        3
212.20     72.20   2172.00        3
222.20     55.40   3220.00        3
222.20     70.60   2271.00        3
225.00     55.00   3245.00        3
225.00     70.00   2310.00        3
250.00     55.00   3245.00        3
250.00     70.00   2310.00        3
275.00     55.00   3245.00        3
275.00     70.00   2310.00        3
300.00     55.00   3245.00        3
300.00     70.00   2310.00        3
325.00     55.00   3245.00        3
325.00     70.00   2310.00        3
350.00     55.00   3245.00        3
350.00     70.00   2310.00        3
373.20     55.00   3245.00        3
373.20     70.00   2310.00        3

ANALYSIS/COMPUTATION
     Circular Search 1
          80.00    250.00       .10       .00       .00
     Tangent
          75.00
SINgle-stage Computations
LEFt Face of Slope
FACtor of safety
 1.5
ITEration limit
 2500
SAVe n most
 5000
CHAnge initial trial factor of safety
LONg-form output
SORt radii
```

```
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Bedrock	150	Very Strong	Not Applicable
2	Foundation Sand	120	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 2
3	Foundation Clays and Silts	115	Cohesion: 25.0 Friction angle: 26	Pore Pressure Interpolated
4	Old Levee Impervious Fill	120	Cohesion: 25.0 Friction angle: 29	Piezometric Line no. 1
5	New Levee Impervious Fill	120	Cohesion: 25.0 Friction angle: 29	Piezometric Line no. 1
6	New Levee - Berm - Impervious Fill	120	Cohesion: 25.0 Friction angle: 29	Piezometric Line no. 1
7	New Levee - Berm2 - Impervious Fill	120	Cohesion: 25.0 Friction angle: 29	Piezometric Line no. 1

rojects\civ\kansas_citys\Geotech\Armourdale FC\Sta 60+00 to 79+00 Work Bellew\Armourdale 60+00 to 79+00\Armourdale 60+00 to 79+00 Stability N500+3\Levee Raise Analysis\Steady State Seepage Case - Lsures.UT4Time: 10:34:27ed Pore Pressur

29 MAR 2007
RSK

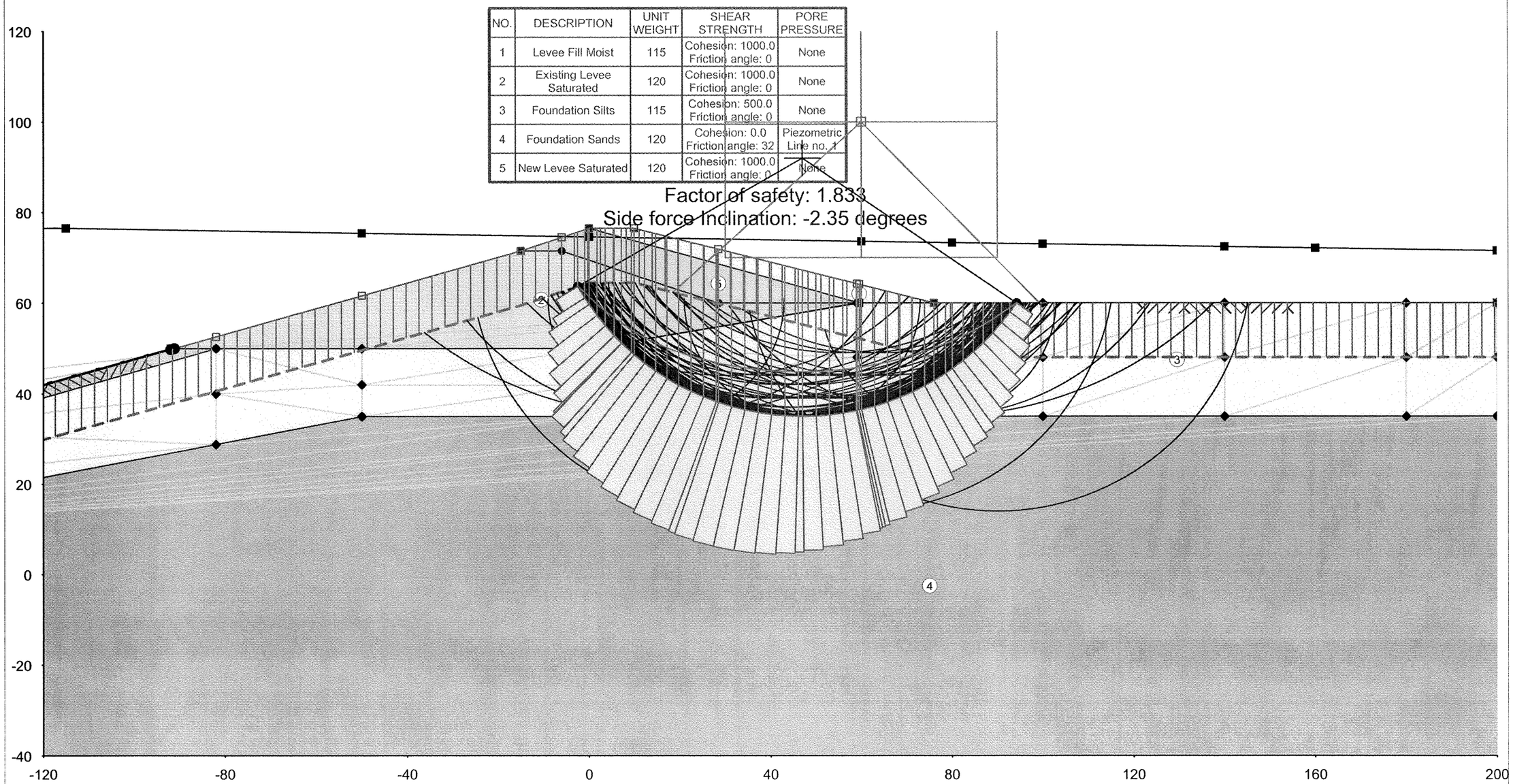
```
HEADING
    Kansas City Levees Feasibility Study Phase II
    Armourdale Station 90+00
    End of Construction - n500 + 3 raise

PROFILE LINES
         1    1 New Levee Fill Moist
                 .00      76.50
               10.00      76.50
               76.00      60.00

         2    5 New Levee Fill Saturated
              -15.00      71.50
                 .00      76.50
               59.50      60.00

         3    2 Existing Levee Saturated
             -155.00      32.00
              -15.00      71.50
               -6.00      71.50
               28.50      60.00
               59.00      60.00

         4    3 Foundation Silt
             -300.00      25.00
             -180.00      25.00
             -155.00      32.00
             -145.00      32.00
              -82.00      50.00
                 .00      50.00
               59.00      60.00
              200.00      60.00

         5    4 Sand
             -300.00      10.00
             -180.00      10.00
              -50.00      35.00
                 .00      35.00
              200.00      35.00


MATERIAL PROPERTIES
     1 Levee Fill Moist
          115.00 Unit Weight
          Conventional Shear
             1000.00        .00
          No Pore Pressure
     2 Existing Levee Saturated
          120.00 Unit Weight
          Conventional Shear
             1000.00        .00
          No Pore Pressure
     3 Foundation Silts
          115.00 Unit Weight
          Conventional Shear
              500.00        .00
          No Pore Pressure
     4 Foundation Sands
          120.00 Unit Weight
          Conventional Shear
                 .00      32.00
```

```
          Piezometric Line
          1
     5 New Levee Saturated
          120.00 Unit Weight
          Conventional Shear
             1000.00        .00
          No Pore Pressure

PIEZOMETRIC LINES
         1      62.40 Groundwater Surface Elevation
             -300.00      25.00
             -180.00      25.00
              -92.00      50.00
                 .00      50.00
               59.50      60.00
              200.00      60.00


DISTRIBUTED LOADS
      1
INTERPOLATION DATA
Pore Water Pressure
        -300.00      25.00   3213.00          3
        -300.00      15.00   3837.00          3
        -300.00      10.00   4149.60          3
        -222.00      25.00   3213.00          3
        -222.00      15.00   3837.00          3
        -222.00      10.00   4149.60          3
        -180.00      25.00   3213.00          3
        -180.00      10.00   4149.60          3
        -180.00      15.00   3837.60          3
        -155.00      32.00   2764.30          3
        -155.00      21.00   3463.00          3
        -155.00      14.80   3851.00          3
         -82.00      50.00   2553.00          3
         -82.00      40.00   2777.00          3
         -82.00      28.80   2942.00          3
         -50.00      50.00   1654.00          3
         -50.00      42.00   2151.00          3
         -50.00      35.00   2216.00          3
            .00      50.00   1654.90          3
            .00      40.00   2153.00          3
            .00      35.00   2470.40          3
          59.00      60.00       .00          3
          59.00      48.00    749.00          3
          59.00      35.00   2353.00          3
         100.00      60.00       .00          3
         100.00      48.00    749.00          3
         100.00      35.00   2371.00          3
         140.00      60.00       .00          3
         140.00      48.00    749.00          3
         140.00      35.00   2334.00          3
         180.00      60.00       .00          3
         180.00      48.00    749.00          3
         180.00      35.00   2296.00          3
         200.00      60.00       .00          3
         200.00      48.00    749.00          3
         200.00      35.00   2277.60          3


ANALYSIS/COMPUTATION
```

NOT USED FOR THIS ANALYSIS

```
     Circular Search 1
          60.00    100.00      1.00       .00
     Radius
          56.00
SINgle-stage Computations
ITEration limit
 50
TRIal max
 70
SAVe n most
 150
LONg-form output
CRAck
          12.00         D
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```

Kansas City Levees Feasibility Study Phase II



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Levee Fill Moist	115	Cohesion: 1000.0 Friction angle: 0	None
2	Existing Levee Saturated	120	Cohesion: 1000.0 Friction angle: 0	None
3	Foundation Silts	115	Cohesion: 500.0 Friction angle: 0	None
4	Foundation Sands	120	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 1
5	New Levee Saturated	120	Cohesion: 1000.0 Friction angle: 0	None

Date: Fri Mar 30 2007 Filename: K:\MissionProjects\civ\kansas_citys\Geotech\Armourdale FC\Slope Stability\3 Ft Raise\Kuz ARM Sta 90+00 WW\arm90 End Of Construction Final Design.UT4 Time: 15:24:06

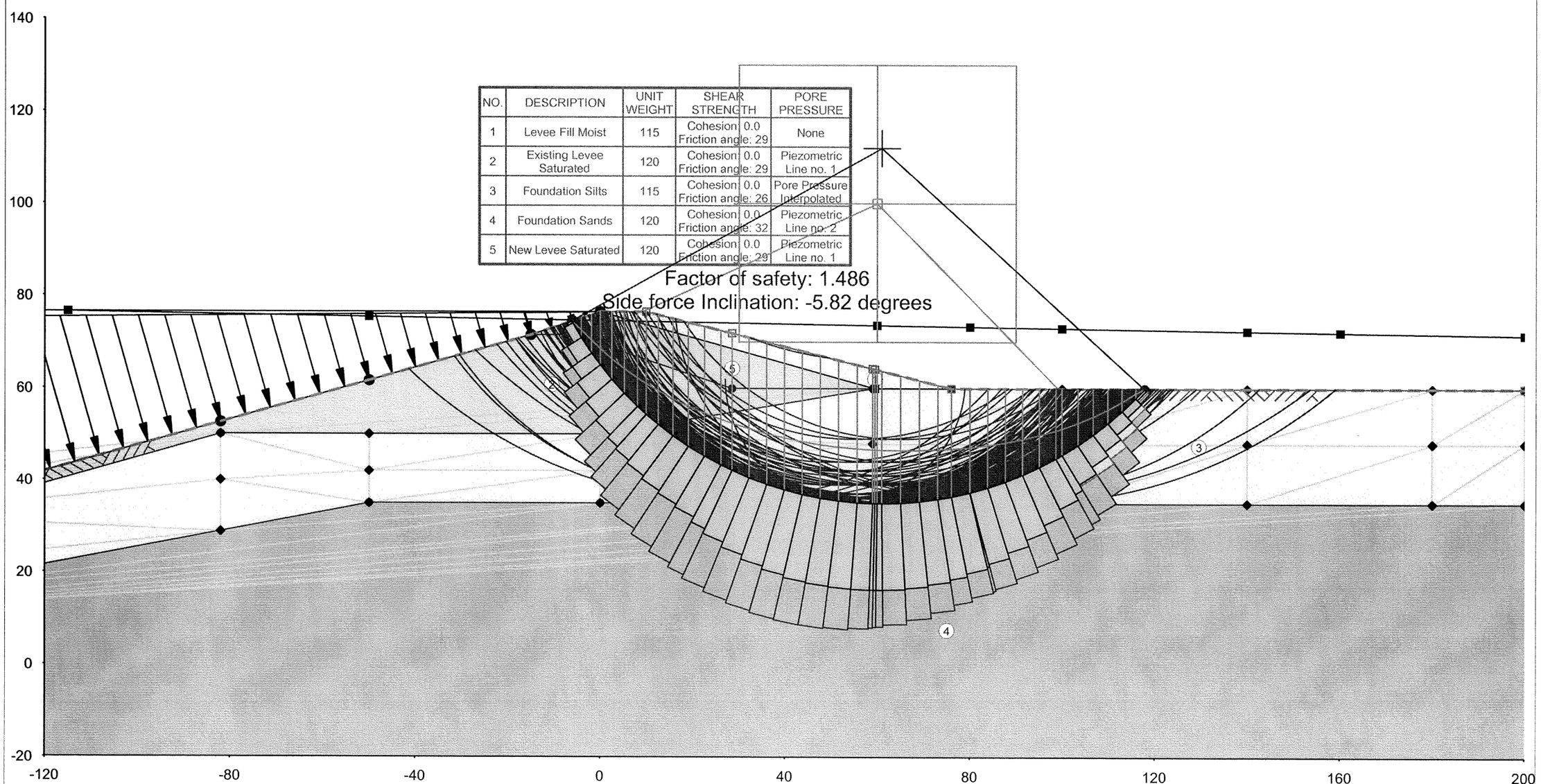
29 Mar 2007
RSK

```
HEADING
    Kansas City Levees Feasibility Study Phase II
    Armourdale Station 90+00
    Steady state Seepage - n500 + 3 raise

PROFILE LINES
         1     1 New Levee Fill Moist
                  .00      76.50
                10.00      76.50
                76.00      60.00

         2     5 New Levee Fill Saturated
               -15.00      71.50
                  .00      76.50
                59.50      60.00

         3     2 Existing Levee Saturated
             -155.00      32.00
              -15.00      71.50
               -6.00      71.50
               28.50      60.00
               59.00      60.00

         4     3 Foundation Silt
             -300.00      25.00
             -180.00      25.00
             -155.00      32.00
             -145.00      32.00
              -82.00      50.00
                 .00      50.00
               59.00      60.00
              200.00      60.00

         5     4 Sand
             -300.00      10.00
             -180.00      10.00
              -50.00      35.00
                 .00      35.00
              200.00      35.00


MATERIAL PROPERTIES
     1 Levee Fill Moist
          115.00 Unit Weight
          Conventional Shear
                 .00      29.00
          No Pore Pressure
     2 Existing Levee Saturated
          120.00 Unit Weight
          Conventional Shear
                 .00      29.00
          Piezometric Line
          1
     3 Foundation Silts
          115.00 Unit Weight
          Conventional Shear
                 .00      26.00
          Interpolate Pore Water Pressure
     4 Foundation Sands
          120.00 Unit Weight
          Conventional Shear
```

```
                .00      32.00
        Piezometric Line
        2
    5 New Levee Saturated
        120.00 Unit Weight
        Conventional Shear
                .00      29.00
        Piezometric Line
        1

PIEZOMETRIC LINES
        1      62.40 Water Surface Elev.
            -300.00      76.50
                .00      76.50
              59.50      60.00
             200.00      60.00

        2      62.40 Foundation Sand Hydraulic Grade Line
            -300.00      76.50
            -115.00      76.50
             -50.00      75.40
                .00      74.60
              60.00      73.60
              80.00      73.30
             100.00      73.00
             140.00      72.40
             160.00      72.10
             200.00      71.50


DISTRIBUTED LOADS
      1
INTERPOLATION DATA
Pore Water Pressure
        -300.00      25.00   3213.00          3
        -300.00      15.00   3837.00          3
        -300.00      10.00   4149.60          3
        -222.00      25.00   3213.00          3
        -222.00      15.00   3837.00          3
        -222.00      10.00   4149.60          3
        -180.00      25.00   3213.00          3
        -180.00      10.00   4149.60          3
        -180.00      15.00   3837.60          3
        -155.00      32.00   2764.30          3
        -155.00      21.00   3463.00          3
        -155.00      14.80   3850.00          3
         -82.00      50.00   1653.00          3
         -82.00      40.00   2777.00          3
         -82.00      28.80   2942.00          3
         -50.00      50.00   1654.00          3
         -50.00      42.00   2153.00          3
         -50.00      35.00   2556.00          3
            .00      50.00   1654.90          3
            .00      42.00   2153.00          3
            .00      35.00   2470.40          3
          59.00      60.00       .00          3
          59.00      48.00    749.00          3
          59.00      35.00   2353.00          3
         100.00      60.00       .00          3
         100.00      48.00    749.00          3
         100.00      35.00   2371.00          3
```

```
        140.00      60.00        .00          3
        140.00      48.00     749.00          3
        140.00      35.00    2334.00          3
        180.00      60.00        .00          3
        180.00      48.00     749.00          3
        180.00      35.00    2296.00          3
        200.00      60.00        .00          3
        200.00      48.00     749.00          3
        200.00      35.00    2277.60          3


ANALYSIS/COMPUTATION
     Circular Search 1
          60.00    100.00      1.00       .00
     Radius
          56.00
SINgle-stage Computations
ITEration limit
 50
TRIal max
 70
SAVe n most
 150
LONg-form output
CRAck
            .00         D
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```

Kansas City Levees Feasibility Study Phase II



NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Levee Fill Moist	115	Cohesion: 0.0 Friction angle: 29	None
2	Existing Levee Saturated	120	Cohesion: 0.0 Friction angle: 29	Piezometric Line no. 1
3	Foundation Silts	115	Cohesion: 0.0 Friction angle: 26	Pore Pressure Interpolated
4	Foundation Sands	120	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 2
5	New Levee Saturated	120	Cohesion: 0.0 Friction angle: 29	Piezometric Line no. 1

Date: Thu Mar 29 2007 Filename: K:\MissionProjects\civ\kansas_citys\Geotech\Armourdale FC\Slope Stability\3 Ft Raise\Kuz ARM Sta 90+00 WW\arm90 Steady Seepage Final Design.UT4 Time: 15:38:41

31 MAR 2007
RSK

```
HEADING
    Armourdale Station 245+50 n500+3
    Station 245+50
    End of Construction conditions

PROFILE LINES
         1     1 New Levee Fill Moist
                  .00      68.10
                10.00      68.10
                90.00      48.00

         2     5 New Levee Fill Saturated
               -10.00      64.30
                  .00      68.10
                65.00      48.00

         3     2 Existing Levee Saturated
               -94.00      38.00
               -33.00      55.00
               -10.00      64.30
                  .00      64.30
                35.00      50.00

         4     3 Foundation Silt
              -172.00      30.00
              -144.00      38.00
               -94.00      38.00
                35.00      50.00
                55.00      50.00
                65.00      48.00
               200.00      48.00

         5     4 Sand
              -300.00      20.00
              -207.00      20.00
              -172.00      30.00
               200.00      30.00

         6     1 Berm 1
                58.16      56.00
                78.00      56.00
               110.00      48.00

         7     1 Berm 2
                94.00      52.00
               104.00      52.00
               118.00      48.00


MATERIAL PROPERTIES
     1 Levee Fill Moist
          115.00 Unit Weight
          Conventional Shear
             1000.00        .00
          No Pore Pressure
     2 Existing Levee Saturated
          120.00 Unit Weight
          Conventional Shear
             1000.00        .00
          No Pore Pressure
     3 Foundation Silts
```

```
        115.00 Unit Weight
        Conventional Shear
            500.00        .00
        No Pore Pressure
    4 Foundation Sands
        120.00 Unit Weight
        Conventional Shear
                .00     32.00
        Piezometric Line
        1
    5 New Levee Saturated
        120.00 Unit Weight
        Conventional Shear
           1000.00        .00
        No Pore Pressure

PIEZOMETRIC LINES
        1      62.40 Water Surface Elev.
            -207.00      20.00
            -172.00      30.00
            -144.00      38.00
             -94.00      38.00
              35.00      50.00
              55.00      50.00
              65.00      48.00
             200.00      48.00


INTERPOLATION DATA
Pore Water Pressure
        -172.00      30.00   2377.40          3
        -144.00      38.00   1878.20          3
        -144.00      30.00   2377.40          3
        -120.00      38.00   1878.20          3
        -120.00      30.00   2340.00          3
         -94.00      38.00   1878.20          3
         -94.00      30.00   2282.80          3
         -64.00      40.79   1646.80          3
         -64.00      30.00   2209.00          3
         -33.00      [illegible]   1405.70          3
         -33.00      30.00   2115.40          3
             .00     46.74   1133.50          3
             .00     30.00   2021.80          3
          35.00      50.00    512.90          3
          35.00      30.00   1921.90          3
          65.00      48.00       .00          3
          65.00      30.00   1872.00          3
          90.00      48.00       .00          3
          90.00      30.00   1847.00          3
         135.00      30.00   1809.60          3
         135.00      48.00       .00          3
         185.00      48.00       .00          3
         185.00      30.00   1778.40          3
         235.00      48.00       .00          3
         235.00      30.00   1747.20          3
         285.00      48.00       .00          3
         285.00      30.00   1722.20          3


ANALYSIS/COMPUTATION
     Circular Search 1
```

NOT USED FOR THIS ANALYSIS

```
          65.00    100.00      1.00       .00
     Radius
          70.00
SINgle-stage Computations
ITEration limit
 50
SAVe n most
 100
LONg-form output
CRAck
          12.00         D
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```

31 MAR 2007

END OF CONST. FINAL DESIGN

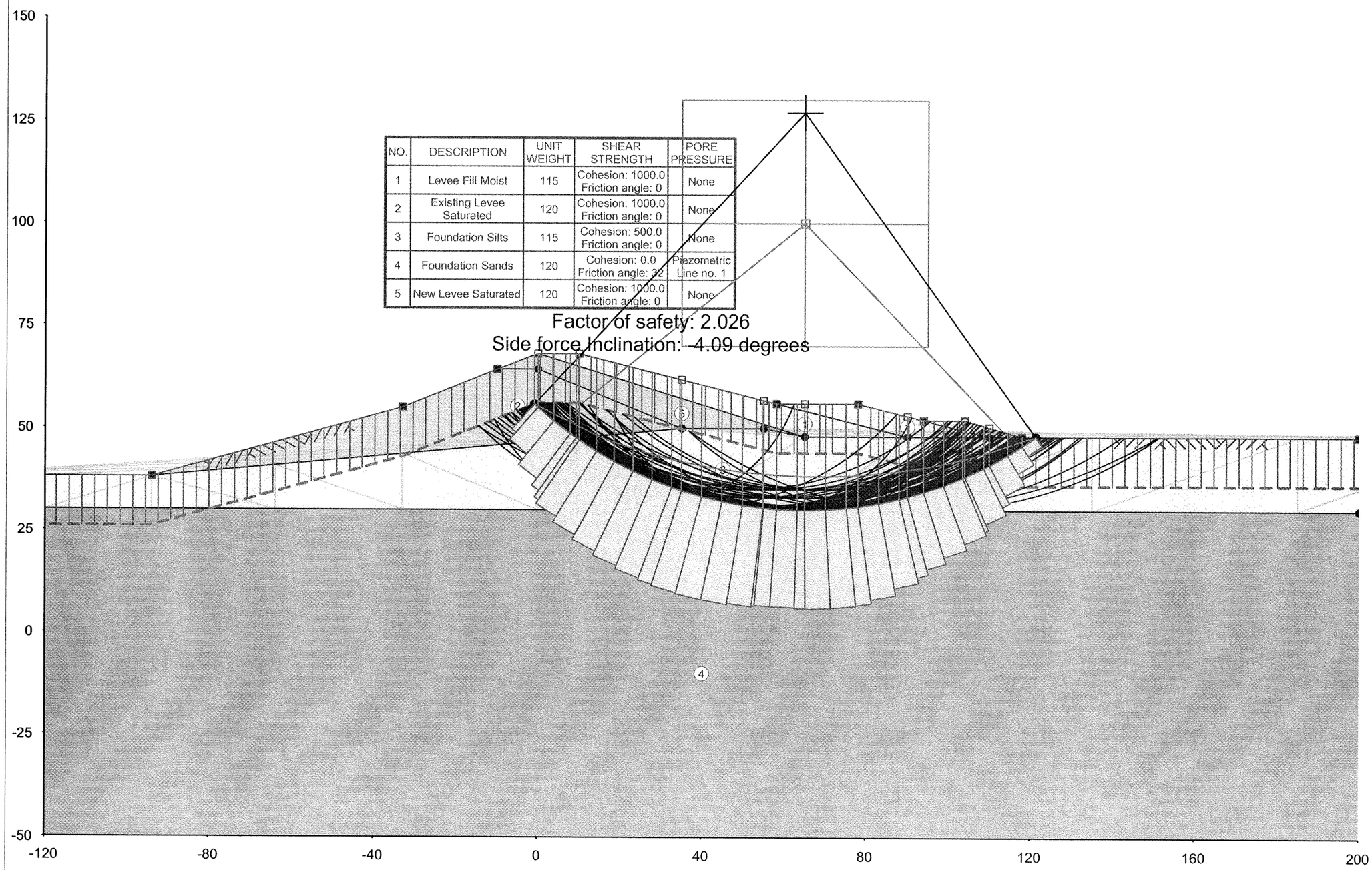
1

Armourdale Station 245+50 n500+3

NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Levee Fill Moist	115	Cohesion: 1000.0 Friction angle: 0	None
2	Existing Levee Saturated	120	Cohesion: 1000.0 Friction angle: 0	None
3	Foundation Silts	115	Cohesion: 500.0 Friction angle: 0	None
4	Foundation Sands	120	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 1
5	New Levee Saturated	120	Cohesion: 1000.0 Friction angle: 0	None

Factor of safety: 2.026

Side force Inclination: -4.09 degrees



Date: Sat Mar 31 2007 Filename: K:\MissionProjects\civ\kansas_citys\Geotech\Armourdale FC\Slope Stability\3 Ft Raise\Kuz ARM Sta 245+50 WW\arm245 End of Construction Final Design.UT4 Time: 15:21:56

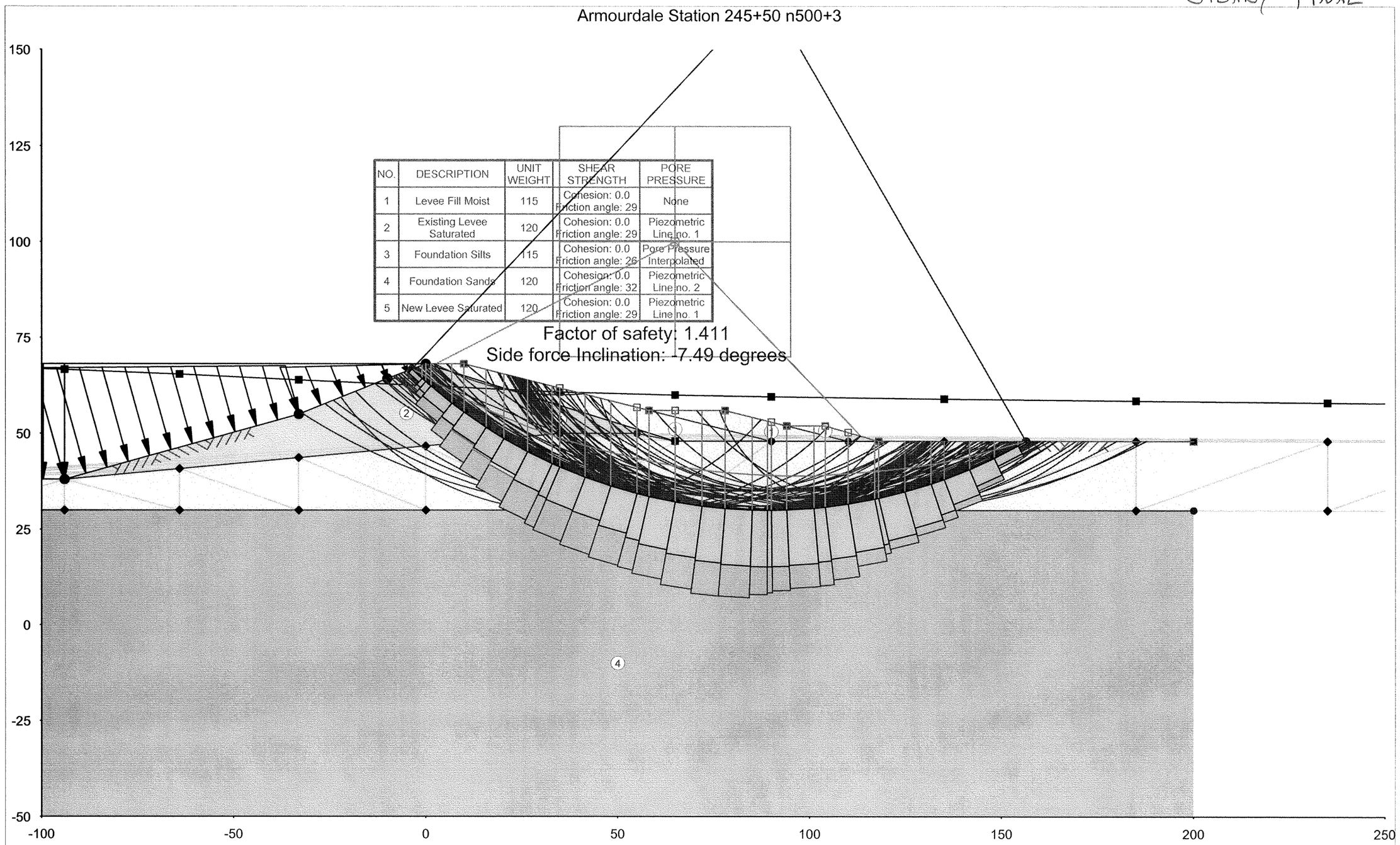
31 MARCH 2007
RSK

```
HEADING
    Armourdale Station 245+50 n500+3
    Station 245+50
    Steady state Seepage conditions

PROFILE LINES
         1     1 New Levee Fill Moist
                  .00      68.10
                10.00      68.10
                90.00      48.00

         2     5 New Levee Fill Saturated
               -10.00      64.30
                  .00      68.10
                65.00      48.00

         3     2 Existing Levee Saturated
               -94.00      38.00
               -33.00      55.00
               -10.00      64.30
                  .00      64.30
                35.00      50.00

         4     3 Foundation Silt
              -172.00      30.00
              -144.00      38.00
               -94.00      38.00
                35.00      50.00
                55.00      50.00
                65.00      48.00
               200.00      48.00

         5     4 Sand
              -300.00      20.00
              -207.00      20.00
              -172.00      30.00
               200.00      30.00

         6     1 Berm 1
                58.16      56.00
                78.00      56.00
               110.00      48.00

         7     1 Berm 2
                94.00      52.00
               104.00      52.00
               118.00      48.00


MATERIAL PROPERTIES
     1 Levee Fill Moist
          115.00 Unit Weight
          Conventional Shear
                  .00      29.00
          No Pore Pressure
     2 Existing Levee Saturated
          120.00 Unit Weight
          Conventional Shear
                  .00      29.00
          Piezometric Line
          1
```

```
     3 Foundation Silts
          115.00 Unit Weight
          Conventional Shear
                 .00      26.00
          Interpolate Pore Water Pressure
     4 Foundation Sands
          120.00 Unit Weight
          Conventional Shear
                 .00      32.00
          Piezometric Line
          2
     5 New Levee Saturated
          120.00 Unit Weight
          Conventional Shear
                 .00      29.00
          Piezometric Line
          1

PIEZOMETRIC LINES
         1      62.40 Water Surface Elev.
             -300.00      68.10
                 .00      68.10
               65.00      48.00
              200.00      48.00

         2      62.40 Foundation Sand Hydraulic Grade Line
             -300.00      68.10
             -144.00      68.10
             -120.00      67.50
              -94.00      66.60
              -64.00      65.40
              -33.00      63.90
                 .00      62.40
               35.00      60.80
               65.00      60.00
               90.00      59.60
              135.00      59.00
              185.00      58.50
              235.00      58.00
              285.00      57.60


DISTRIBUTED LOADS
      1
INTERPOLATION DATA
Pore Water Pressure
        -172.00      30.00   2377.40          3
        -144.00      38.00   1878.20          3
        -144.00      30.00   2377.40          3
        -120.00      38.00   1878.20          3
        -120.00      30.00   2340.00          3
         -94.00      38.00   1878.20          3
         -94.00      30.00   2283.80          3
         -64.00      40.79   1646.80          3
         -64.00      30.00   2209.00          3
         -33.00      43.67   1405.70          3
         -33.00      30.00   2115.40          3
            .00      46.74   1133.50          3
            .00      30.00   2021.80          3
          35.00      50.00    512.90          3
          35.00      30.00   1921.90          3
```

```
          65.00      48.00        .00          3
          65.00      30.00    1872.00          3
          90.00      48.00        .00          3
          90.00      30.00    1847.00          3
         135.00      30.00    1809.60          3
         135.00      48.00        .00          3
         185.00      48.00        .00          3
         185.00      30.00    1778.40          3
         235.00      48.00        .00          3
         235.00      30.00    1747.20          3
         285.00      48.00        .00          3
         285.00      30.00    1722.20          3


ANALYSIS/COMPUTATION
     Circular Search 1
          65.00     100.00       1.00        .00
     Radius
          70.00
SINgle-stage Computations
ITEration limit
 50
SAVe n most
 100
LONg-form output
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```

STEADY FINAL



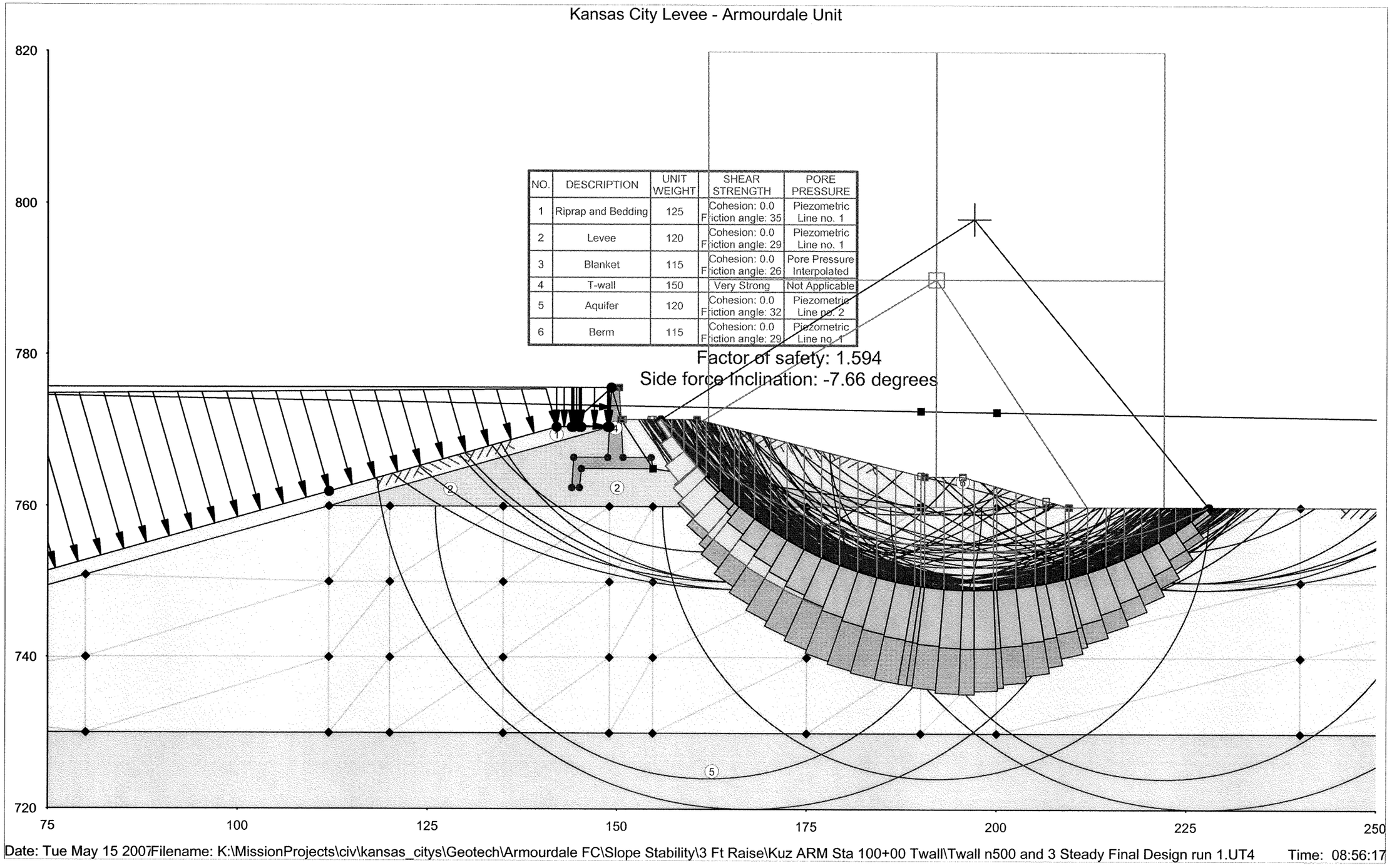
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Levee Fill Moist	115	Cohesion: 0.0 Friction angle: 29	None
2	Existing Levee Saturated	120	Cohesion: 0.0 Friction angle: 29	Piezometric Line no. 1
3	Foundation Silts	115	Cohesion: 0.0 Friction angle: 26	Pore Pressure Interpolated
4	Foundation Sands	120	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 2
5	New Levee Saturated	120	Cohesion: 0.0 Friction angle: 29	Piezometric Line no. 1

Date: Sat Mar 31 2007 Filename: K:\MissionProjects\civ\kansas_citys\Geotech\Armourdale FC\Slope Stability\3 Ft Raise\Kuz ARM Sta 245+50 WW\arm245 Steady Final Design.UT4 Time: 15:03:08

```
HEADING
    Kansas City Levee - Armourdale Unit
    Feasibility Study
    Phase II
    Global Stability for T-wall on Levee Raise
    N500 + 3 Design Condition - STA 100+00

PROFILE LINES
         1     1 Riprap and Bedding
                  .00     730.00
               142.00     770.50
               149.00     770.50

         2     2 Levee 1
               112.00     760.00
               149.00     770.50

         3     6 Levee 2
               150.50     771.50
               160.50     771.50
               206.50     760.00

         4     2 Levee 3
               144.00     762.50
               145.00     762.50
               145.25     765.00
               154.75     765.00
               190.00     760.00

         5     3 Blanket
                 7.00     730.00
               112.00     760.00
               300.00     760.00

         6     4 T-wall
               144.00     762.50
               144.25     766.50
               148.75     766.50
               149.00     770.50
               149.25     775.70
               150.25     775.70
               150.50     771.50
               150.75     766.50
               154.50     766.50
               154.75     765.00

         8     6 Berm
               190.50     764.00
               195.50     764.00
               209.50     760.00

         7     5 Aquifer
                  .00     730.00
               300.00     730.00


MATERIAL PROPERTIES
     1 Riprap and Bedding
          125.00 Unit Weight
          Conventional Shear
                 .00      35.00
```

```
            Piezometric Line
            1
      2 Levee
            120.00 Unit Weight
            Conventional Shear
                   .00     29.00
            Piezometric Line
            1
      3 Blanket
            115.00 Unit Weight
            Conventional Shear
                   .00     26.00
            Interpolate Pore Water Pressure
      4 T-wall
            150.00 Unit Weight
            Very Strong
      5 Aquifer
            120.00 Unit Weight
            Conventional Shear
                   .00     32.00
            Piezometric Line
            2
      6 Berm
            115.00 Unit Weight
            Conventional Shear
                   .00     29.00
            Piezometric Line
            1

PIEZOMETRIC LINES
         1      62.40 Piezometric Line 1
                  .00    775.70
               149.25    775.70
               150.50    771.50
               154.75    765.00
               190.00    760.00
               300.00    760.00

         2      62.40 Piezometric Line 2
                  .00    775.70
                 7.00    775.70
               190.00    772.65
               200.00    772.50
               300.00    771.00


DISTRIBUTED LOADS
      1
INTERPOLATION DATA
Pore Water Pressure
           7.00    730.00   2851.70          3
          40.00    739.43   2263.25          3
          40.00    735.00   2520.60          3
          40.00    730.00   2811.10          3
          80.00    750.86   1550.00          3
          80.00    740.00   2185.50          3
          80.00    730.00   2770.60          3
         112.00    760.00    979.70          3
         112.00    750.00   1566.30          3
         112.00    740.00   2152.80          3
         112.00    730.00   2739.40          3
```

```
120.00    760.00    979.70        3
120.00    750.00   1563.10        3
120.00    740.00   2146.60        3
120.00    730.00   2730.00        3
135.00    760.00    979.70        3
135.00    750.00   1557.90        3
135.00    740.00   2136.20        3
135.00    730.00   2714.40        3
149.00    760.00    979.70        3
149.00    750.00   1553.80        3
149.00    740.00   2127.80        3
149.00    730.00   2701.90        3
154.75    760.00    385.10        3
154.75    750.00   1155.30        3
154.75    740.00   1925.50        3
154.75    730.00   2695.70        3
175.00    760.00    175.10        3
175.00    750.00   1009.10        3
175.00    730.00   2677.00        3
175.00    740.00   1843.00        3
190.00    760.00       .00        3
190.00    750.00    887.10        3
190.00    740.00   1774.30        3
190.00    730.00   2661.40        3
200.00    760.00       .00        3
200.00    750.00    884.00        3
200.00    740.00   1768.00        3
200.00    730.00   2652.00        3
240.00    760.00       .00        3
240.00    750.00    871.50        3
240.00    730.00   2614.60        3
240.00    740.00   1743.10        3
280.00    760.00       .00        3
280.00    750.00    859.00        3
280.00    740.00   1718.00        3
280.00    730.00   2577.10        3
300.00    760.00       .00        3
300.00    750.00    852.80        3
300.00    740.00   1705.60        3
300.00    730.00   2558.40        3


ANALYSIS/COMPUTATION
     Circular Search 1
         192.00    790.00      1.00    670.00
     Radius
          36.00
SINgle-stage Computations
SAVe n most
 150
LONg-form output
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```



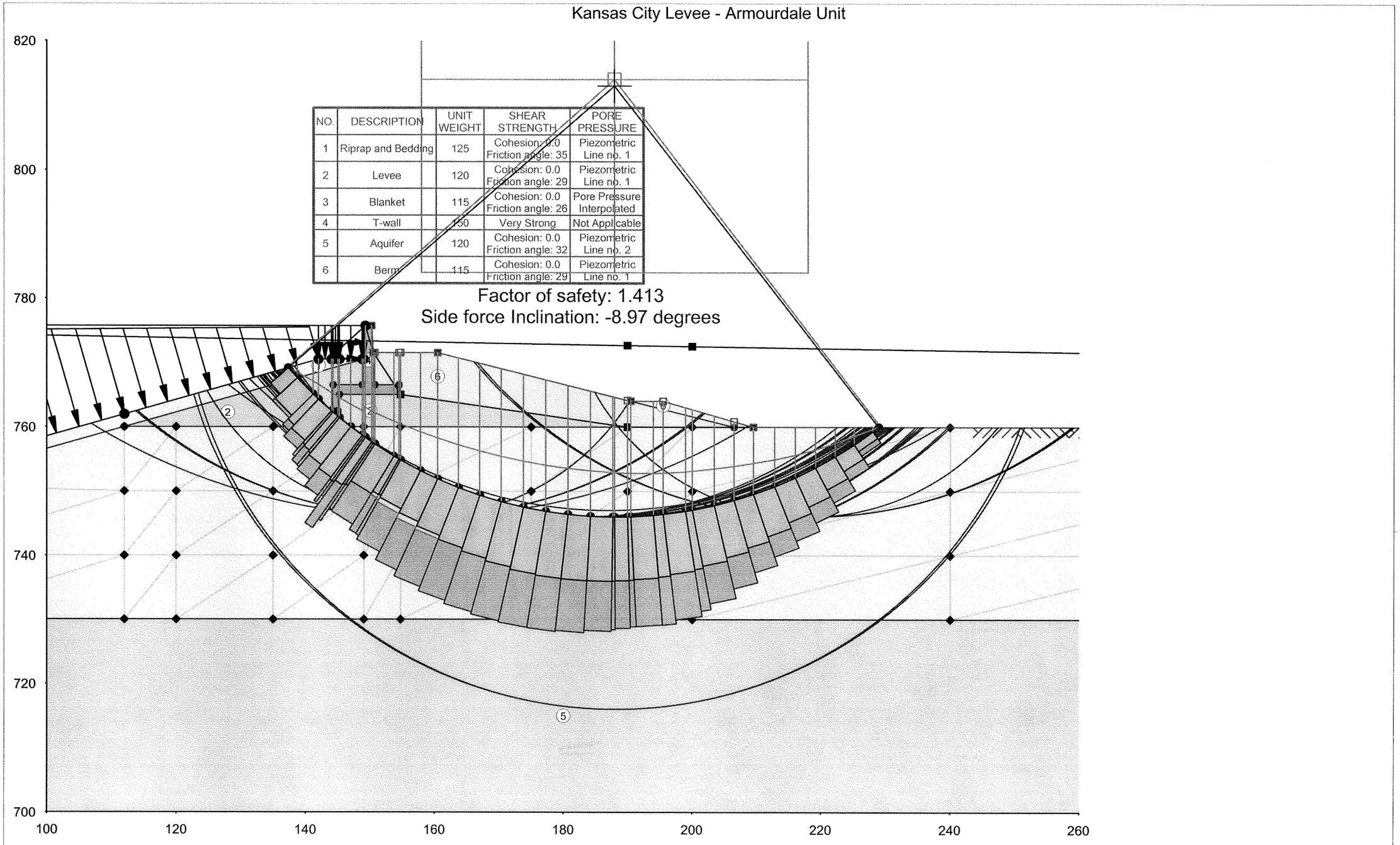
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Riprap and Bedding	125	Cohesion: 0.0 Friction angle: 35	Piezometric Line no. 1
2	Levee	120	Cohesion: 0.0 Friction angle: 29	Piezometric Line no. 1
3	Blanket	115	Cohesion: 0.0 Friction angle: 26	Pore Pressure Interpolated
4	T-wall	150	Very Strong	Not Applicable
5	Aquifer	120	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 2
6	Berm	115	Cohesion: 0.0 Friction angle: 29	Piezometric Line no. 1

```
HEADING
    Kansas City Levee - Armourdale Unit
    Feasibility Study
    Phase II
    Global Stability for T-wall on Levee Raise
    N500 + 3 Design Condition  STA 100+00

PROFILE LINES
        1    1 Riprap and Bedding
                 .00     730.00
             142.00     770.50
             149.00     770.50

        2    2 Levee 1
             112.00     760.00
             149.00     770.50

        3    6 Levee 2
             150.50     771.50
             160.50     771.50
             206.50     760.00

        4    2 Levee 3
             144.00     762.50
             145.00     762.50
             145.25     765.00
             154.75     765.00
             190.00     760.00

        5    3 Blanket
               7.00     730.00
             112.00     760.00
             300.00     760.00

        6    4 T-wall
             144.00     762.50
             144.25     766.50
             148.75     766.50
             149.00     770.50
             149.25     775.70
             150.25     775.70
             150.50     771.50
             150.75     766.50
             154.50     766.50
             154.75     765.00

        8    6 Berm
             190.50     764.00
             195.50     764.00
             209.50     760.00

        7    5 Aquifer
                 .00     730.00
             300.00     730.00


MATERIAL PROPERTIES
     1 Riprap and Bedding
          125.00 Unit Weight
          Conventional Shear
                 .00      35.00
```

```
            Piezometric Line
            1
      2 Levee
            120.00 Unit Weight
            Conventional Shear
                   .00     29.00
            Piezometric Line
            1
      3 Blanket
            115.00 Unit Weight
            Conventional Shear
                   .00     26.00
            Interpolate Pore Water Pressure
      4 T-wall
            150.00 Unit Weight
            Very Strong
      5 Aquifer
            120.00 Unit Weight
            Conventional Shear
                   .00     32.00
            Piezometric Line
            2
      6 Berm
            115.00 Unit Weight
            Conventional Shear
                   .00     29.00
            Piezometric Line
            1

PIEZOMETRIC LINES
         1     62.40 Piezometric Line 1
                 .00    775.70
              149.25    775.70
              150.50    771.50
              154.75    765.00
              190.00    760.00
              300.00    760.00

         2     62.40 Piezometric Line 2
                 .00    775.70
                7.00    775.70
              190.00    772.65
              200.00    772.50
              300.00    771.00


DISTRIBUTED LOADS
      1
INTERPOLATION DATA
Pore Water Pressure
           7.00    730.00   2851.70         3
          40.00    739.43   2263.25         3
          40.00    735.00   2520.60         3
          40.00    730.00   2811.10         3
          80.00    750.86   1550.00         3
          80.00    740.00   2185.50         3
          80.00    730.00   2770.60         3
         112.00    760.00    979.70         3
         112.00    750.00   1566.30         3
         112.00    740.00   2152.80         3
         112.00    730.00   2739.40         3
```

120.00	760.00	979.70	3
120.00	750.00	1563.10	3
120.00	740.00	2146.60	3
120.00	730.00	2730.00	3
135.00	760.00	979.70	3
135.00	750.00	1557.90	3
135.00	740.00	2136.20	3
135.00	730.00	2714.40	3
149.00	760.00	979.70	3
149.00	750.00	1553.80	3
149.00	740.00	2127.80	3
149.00	730.00	2701.90	3
154.75	760.00	385.10	3
154.75	750.00	1155.30	3
154.75	740.00	1925.50	3
154.75	730.00	2695.70	3
175.00	760.00	175.10	3
175.00	750.00	1009.10	3
175.00	730.00	2677.00	3
175.00	740.00	1843.00	3
190.00	760.00	.00	3
190.00	750.00	887.10	3
190.00	740.00	1774.30	3
190.00	730.00	2661.40	3
200.00	760.00	.00	3
200.00	750.00	884.00	3
200.00	740.00	1768.00	3
200.00	730.00	2652.00	3
240.00	760.00	.00	3
240.00	750.00	871.50	3
240.00	730.00	2614.60	3
240.00	740.00	1743.10	3
280.00	760.00	.00	3
280.00	750.00	859.00	3
280.00	740.00	1718.00	3
280.00	730.00	2577.10	3
300.00	760.00	.00	3
300.00	750.00	852.80	3
300.00	740.00	1705.60	3
300.00	730.00	2558.40	3

```
ANALYSIS/COMPUTATION
     Circular Search 1
         188.00     814.00       1.00     670.00
     Radius
          68.00
SINgle-stage Computations
SAVe n most
 150
LONg-form output
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```


Kansas City Levee - Armourdale Unit
NO. | DESCRIPTION | UNIT WEIGHT | SHEAR STRENGTH | PORE PRESSURE
1 | Riprap and Bedding | 125 | Cohesion: 0.0 Friction angle: 35 | Piezometric Line no. 1
2 | Levee | 120 | Cohesion: 0.0 Friction angle: 29 | Piezometric Line no. 1
3 | Blanket | 115 | Cohesion: 0.0 Friction angle: 26 | Pore Pressure Interpolated
4 | T-wall | 150 | Very Strong | Not Applicable
5 | Aquifer | 120 | Cohesion: 0.0 Friction angle: 32 | Piezometric Line no. 2
6 | Berm | 115 | Cohesion: 0.0 Friction angle: 29 | Piezometric Line no. 1
Factor of safety: 1.413
Side force Inclination: -8.97 degrees
820
800
780
760
740
720
700
100
120
140
160
180
200
220
240
260
Date: Tue May 15 2007Filename: K:\MissionProjects\civ\kansas_citys\Geotech\Armourdale FC\Slope Stability\3 Ft Raise\Kuz ARM Sta 100+00 Twall\Twall n500 and 3 Steady Final Design run 2.UT4
Time: 09:00:50

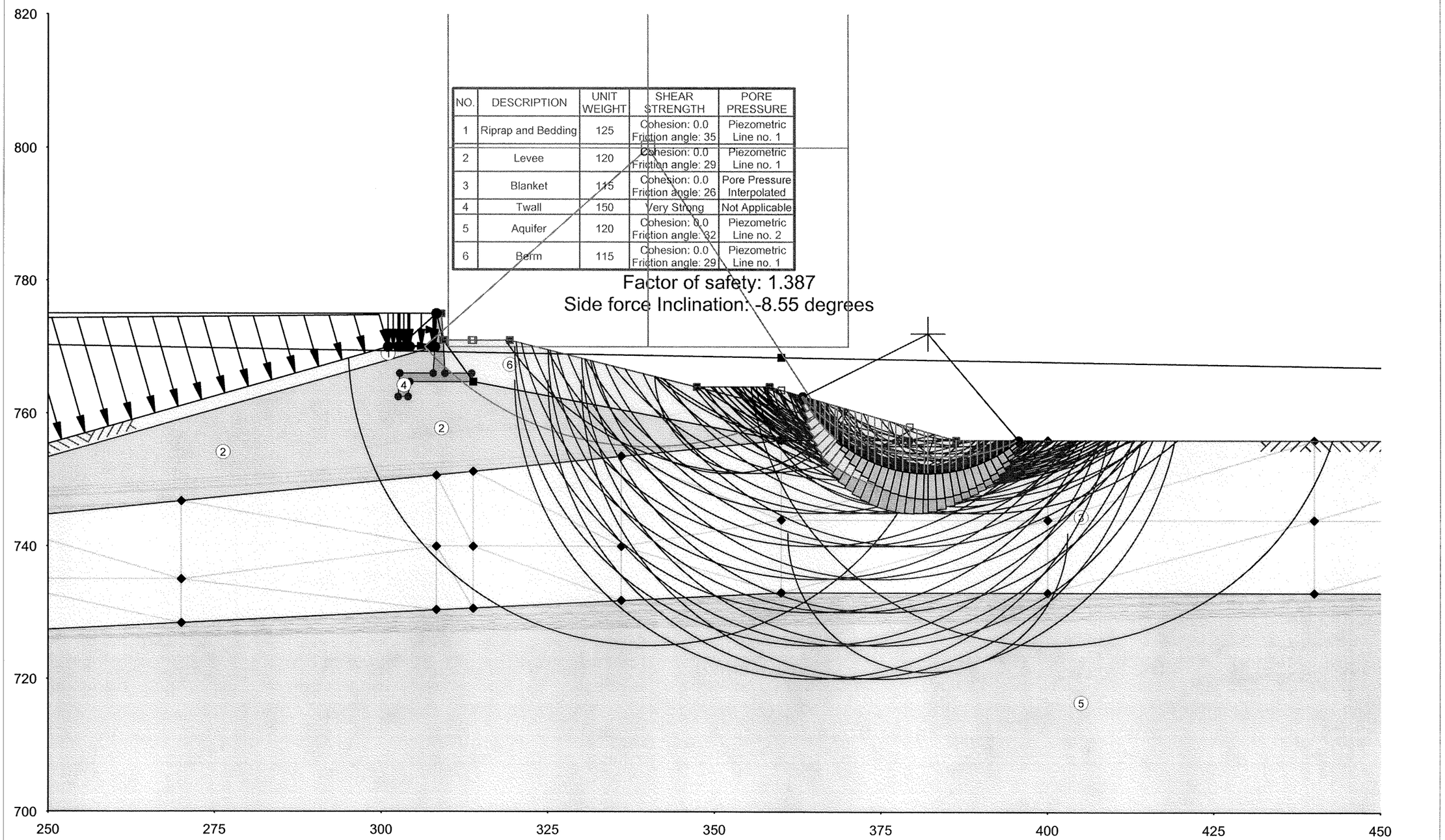
```
HEADING
    Kansas City Levees Phase 2 Feasibility Study
    Armourdale Unit
    T-wall on Levee Station 122+50
    Global Stability Steady Seepage Condition

PROFILE LINES
        8    6 Berm
             347.25    764.00
             358.25    764.00
             386.25    756.00

        2    2 Levee 1
             203.00    740.00
             308.00    770.00

        3    6 Levee 2
             309.25    771.00
             319.25    771.00
             379.25    756.00

        4    2 Levee 3
             302.50    762.50
             304.00    762.50
             304.25    764.75
             313.75    764.75
             360.00    756.00

        5    3 Blanket
             100.00    740.00
             203.00    740.00
             360.00    756.00
             500.00    756.00

        6    4 Twall
             302.50    762.50
             302.75    766.00
             307.75    766.00
             308.00    770.00
             308.25    775.00
             309.00    775.00
             309.25    771.00
             309.50    766.00
             313.50    766.00
             313.75    764.75

        7    5 Aquifer
             100.00    725.00
             203.00    725.00
             360.00    733.00
             500.00    733.00

        1    1 Riprap and Bedding
             196.00    740.00
             301.00    770.00
             308.00    770.00


MATERIAL PROPERTIES
     1 Riprap and Bedding
          125.00 Unit Weight
```

```
            Conventional Shear
                   .00     35.00
            Piezometric Line
            1
      2 Levee
            120.00 Unit Weight
            Conventional Shear
                   .00     29.00
            Piezometric Line
            1
      3 Blanket
            115.00 Unit Weight
            Conventional Shear
                   .00     26.00
            Interpolate Pore Water Pressure
      4 Twall
            150.00 Unit Weight
            Very Strong
      5 Aquifer
            120.00 Unit Weight
            Conventional Shear
                   .00     32.00
            Piezometric Line
            2
      6 Berm
            115.00 Unit Weight
            Conventional Shear
                   .00     29.00
            Piezometric Line
            1

PIEZOMETRIC LINES
         1      62.40 Piezometric Line 1
               100.00    775.00
               308.25    775.00
               309.25    771.00
               313.75    764.75
               360.00    756.00
               600.00    756.00

         2      62.40 Piezometric Line 2
                85.00    773.00
               360.00    768.40
               460.00    766.80
               560.00    765.40
               600.00    764.70


DISTRIBUTED LOADS
      1
INTERPOLATION DATA
Pore Water Pressure
         120.00    740.00   2184.00          3
         120.00    733.00   2545.10          3
         120.00    725.00   2957.80          3
         160.00    740.00   2184.00          3
         160.00    733.00   2524.70          3
         160.00    725.00   2914.00          3
         203.00    740.00   2184.00          3
         203.00    733.00   2650.80          3
         203.00    725.00   2870.40          3
```

240.00	743.80	1864.80	3
240.00	735.00	2307.20	3
240.00	726.90	2714.40	3
270.00	746.80	1634.40	3
270.00	735.00	2251.00	3
270.00	728.40	2595.80	3
308.25	750.70	1516.30	3
308.25	740.00	1996.50	3
308.25	730.40	2427.40	3
313.75	751.30	839.30	3
313.75	740.00	1696.00	3
313.75	730.60	2408.60	3
336.00	753.60	433.70	3
336.00	740.00	1603.50	3
336.00	731.80	2308.80	3
360.00	756.00	.00	3
360.00	744.00	1152.50	3
360.00	733.00	2209.00	3
400.00	756.00	.00	3
400.00	744.00	1038.50	3
400.00	733.00	2171.50	3
440.00	756.00	.00	3
440.00	744.00	1110.20	3
440.00	733.00	2127.80	3
480.00	756.00	.00	3
480.00	744.00	1090.60	3
480.00	733.00	2090.40	3
520.00	756.00	.00	3
520.00	744.00	1074.40	3
520.00	733.00	2059.20	3
560.00	756.00	.00	3
560.00	744.00	1054.90	3
560.00	733.00	2021.80	3
600.00	756.00	.00	3
600.00	744.00	1032.10	3
600.00	733.00	1978.10	3

```
ANALYSIS/COMPUTATION
     Circular Search 1
          340.00     800.00       1.00     670.00
     Radius
           45.00
SINgle-stage Computations
SAVe n most
 150
LONg-form output
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```

Kansas City Levees Phase 2 Feasibility Study



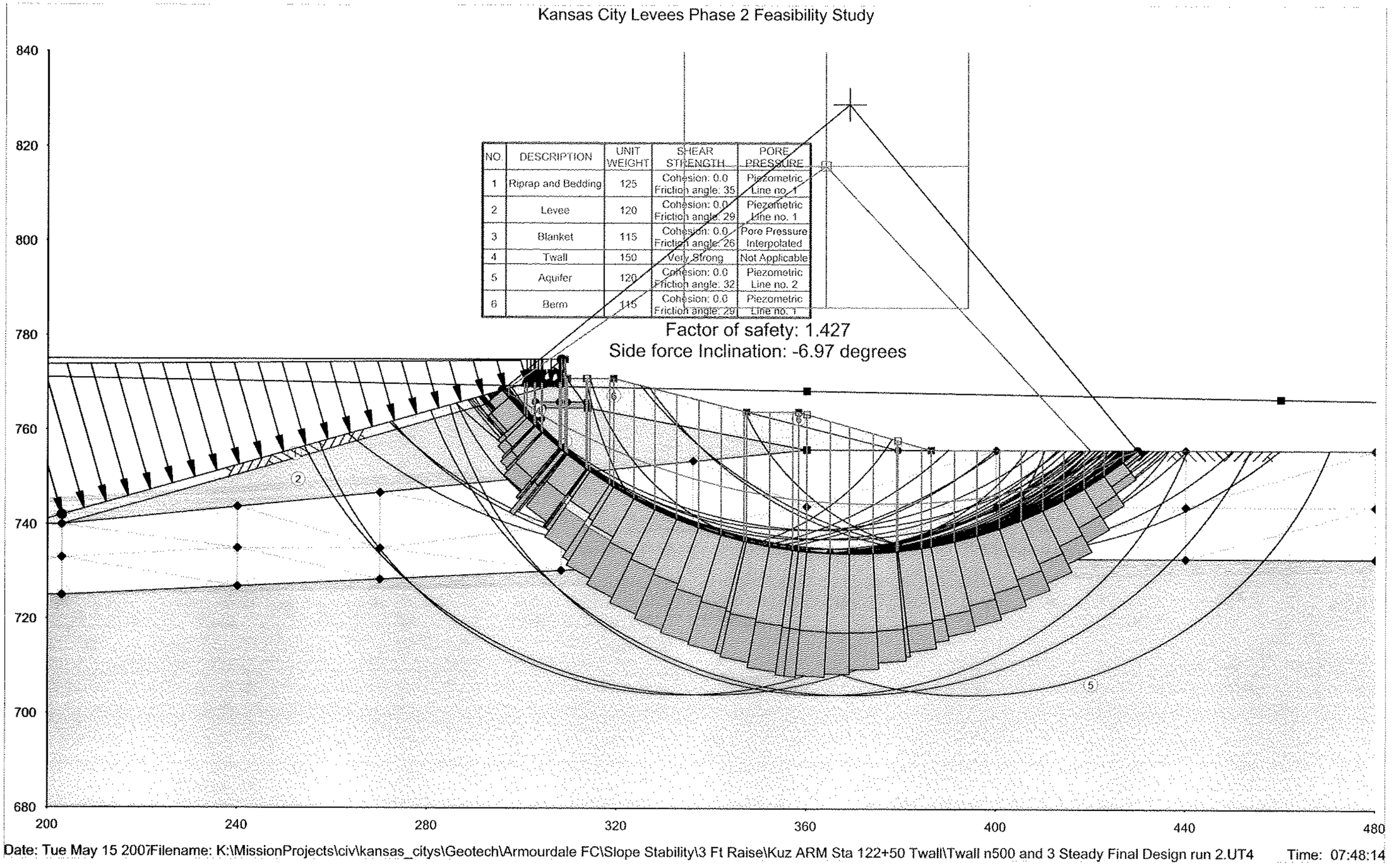
NO.	DESCRIPTION	UNIT WEIGHT	SHEAR STRENGTH	PORE PRESSURE
1	Riprap and Bedding	125	Cohesion: 0.0 Friction angle: 35	Piezometric Line no. 1
2	Levee	120	Cohesion: 0.0 Friction angle: 29	Piezometric Line no. 1
3	Blanket	115	Cohesion: 0.0 Friction angle: 26	Pore Pressure Interpolated
4	Twall	150	Very Strong	Not Applicable
5	Aquifer	120	Cohesion: 0.0 Friction angle: 32	Piezometric Line no. 2
6	Berm	115	Cohesion: 0.0 Friction angle: 29	Piezometric Line no. 1

Date: Tue May 15 2007 Filename: K:\MissionProjects\civ\kansas_citys\Geotech\Armourdale FC\Slope Stability\3 Ft Raise\Kuz ARM Sta 122+50 Twall\Twall n500 and 3 Steady Final Design run 1.UT4 Time: 07:43:05

```
HEADING
    Kansas City Levees Phase 2 Feasibility Study
    Armourdale Unit
    T-wall on Levee Station 122+50
    Global Stability Steady Seepage Condition

PROFILE LINES
        8     6 Berm
              347.25     764.00
              358.25     764.00
              386.25     756.00

        2     2 Levee 1
              203.00     740.00
              308.00     770.00

        3     6 Levee 2
              309.25     771.00
              319.25     771.00
              379.25     756.00

        4     2 Levee 3
              302.50     762.50
              304.00     762.50
              304.25     764.75
              313.75     764.75
              360.00     756.00

        5     3 Blanket
              100.00     740.00
              203.00     740.00
              360.00     756.00
              500.00     756.00

        6     4 Twall
              302.50     762.50
              302.75     766.00
              307.75     766.00
              308.00     770.00
              308.25     775.00
              309.00     775.00
              309.25     771.00
              309.50     766.00
              313.50     766.00
              313.75     764.75

        7     5 Aquifer
              100.00     725.00
              203.00     725.00
              360.00     733.00
              500.00     733.00

        1     1 Riprap and Bedding
              196.00     740.00
              301.00     770.00
              308.00     770.00


MATERIAL PROPERTIES
     1 Riprap and Bedding
          125.00 Unit Weight
```

```
          Conventional Shear
                 .00     35.00
          Piezometric Line
          1
     2 Levee
          120.00 Unit Weight
          Conventional Shear
                 .00     29.00
          Piezometric Line
          1
     3 Blanket
          115.00 Unit Weight
          Conventional Shear
                 .00     26.00
          Interpolate Pore Water Pressure
     4 Twall
          150.00 Unit Weight
          Very Strong
     5 Aquifer
          120.00 Unit Weight
          Conventional Shear
                 .00     32.00
          Piezometric Line
          2
     6 Berm
          115.00 Unit Weight
          Conventional Shear
                 .00     29.00
          Piezometric Line
          1

PIEZOMETRIC LINES
         1      62.40 Piezometric Line 1
               100.00    775.00
               308.25    775.00
               309.25    771.00
               313.75    764.75
               360.00    756.00
               600.00    756.00

         2      62.40 Piezometric Line 2
                85.00    773.00
               360.00    768.40
               460.00    766.80
               560.00    765.40
               600.00    764.70


DISTRIBUTED LOADS
      1
INTERPOLATION DATA
Pore Water Pressure
         120.00     740.00    2184.00          3
         120.00     733.00    2545.10          3
         120.00     725.00    2957.80          3
         160.00     740.00    2184.00          3
         160.00     733.00    2524.70          3
         160.00     725.00    2914.00          3
         203.00     740.00    2184.00          3
         203.00     733.00    2650.80          3
         203.00     725.00    2870.40          3
```

```
        240.00    743.80   1864.80          3
        240.00    735.00   2307.20          3
        240.00    726.90   2714.40          3
        270.00    746.80   1634.40          3
        270.00    735.00   2251.00          3
        270.00    728.40   2595.80          3
        308.25    750.70   1516.30          3
        308.25    740.00   1996.50          3
        308.25    730.40   2427.40          3
        313.75    751.30    839.30          3
        313.75    740.00   1696.00          3
        313.75    730.60   2408.60          3
        336.00    753.60    433.70          3
        336.00    740.00   1603.50          3
        336.00    731.80   2308.80          3
        360.00    756.00       .00          3
        360.00    744.00   1152.50          3
        360.00    733.00   2209.00          3
        400.00    756.00       .00          3
        400.00    744.00   1038.50          3
        400.00    733.00   2171.50          3
        440.00    756.00       .00          3
        440.00    744.00   1110.20          3
        440.00    733.00   2127.80          3
        480.00    756.00       .00          3
        480.00    744.00   1090.60          3
        480.00    733.00   2090.40          3
        520.00    756.00       .00          3
        520.00    744.00   1074.40          3
        520.00    733.00   2059.20          3
        560.00    756.00       .00          3
        560.00    744.00   1054.90          3
        560.00    733.00   2021.80          3
        600.00    756.00       .00          3
        600.00    744.00   1032.10          3
        600.00    733.00   1978.10          3


ANALYSIS/COMPUTATION
     Circular Search 1
         364.00    816.00      1.00    670.00
     Radius
          82.00
SINgle-stage Computations
SAVe n most
 150
LONg-form output
SORt radii
CRItical
PROcedure for computation of Factor of Safety
SPENCER

GRAPH
COMPUTE
```



Date: Tue May 15 2007Filename: K:\MissionProjects\civ\kansas_citys\Geotech\Armourdale FC\Slope Stability\3 Ft Raise\Kuz ARM Sta 122+50 Twall\Twall n500 and 3 Steady Final Design run 2.UT4 Time: 07:48:14

EXHIBIT 12

Armourdale Levee Unit
Phase 2 Feasibility Study
4/26/2007 rev 2

Station	Existing Levee Crest Elevation (ft)	Existing Landside Elevation (ft)	Existing Levee Landside Height (ft)	n500+3 Raise (ft)	First Berm Berm Height (ft)	First Berm Berm Width (ft)	Second Berm Berm Height	Second Berm Berm Width	Proposed Landside Height (ft)	Distance from Exist. Centerline to Exist. Toe (ft)	Proposed Section Number	Landside T-Wall Distance from Exist. Centerline to Proposed Toe (ft)	Landside Levee Distance from Exist. Centerline to Proposed Toe (ft)
3+25 UE to 10+00 UE	774.10	765.0	9.10	4.10	6	12			13.20	32.3	3		**47.3**
10+00 UE to 16+48 UE	774.10	765.0	9.10	4.00					13.10	32.3	Floodwall		
16+48 UE To 15+00	773.95		high ground	4.05						5.0	1		**19.2**
15+00 To 20+00	773.95	770.0	3.95	4.05					8.00	16.9	1		**51.2**
20+00 to 25+00	773.90	768.0	5.90	4.05					9.95	22.7	1		**59.0**
25+00 to 30+00	773.80	768.0	5.80	4.10					9.90	22.4	1		**58.9**
30+00 to 35+00	773.75	768.0	5.75	4.10					9.85	22.3	1		**58.8**
35+00 to 40+00	773.65	760.0	13.65	4.15	8	20	4	10	17.80	45.9	2		**118.7**
40+00 to 42+50	773.60	760.0	13.60	4.20	8	20	4	10	17.80	45.8	2		**118.9**
42+50 to 45+00	773.50	760.0	13.50	4.30	8	10			17.80	45.5	7	**70.5**	**101.3**
45+00 to 50+00	773.45	760.0	13.45	4.30	8	10			17.75	45.4	7	**70.3**	**101.1**
50+00 to 55+00	773.40	762.0	11.40	4.30	6	8			15.70	39.2	6	**61.1**	**82.8**
55+00 to 58+00	773.35	762.0	11.35	4.30	6	8			15.65	39.1	6	**60.9**	**82.7**
58+00 to 60+40	773.25	762.0	11.25	4.30					15.55	38.8	?		
60+40 to 65+00	773.25	762.0	11.25	4.15					15.40	38.8	?		**existing toe**
65+00 to 70+00	773.15	760.0	13.15	4.20					17.35	44.4	?		**existing toe**
70+00 to 74+00	772.85	758.0	14.85	4.35					19.20	49.6	?		**existing toe**
74+00 to 77+78											Floodwall		
77+78 to 81+00	772.25	760.0	12.25	4.80	8	10			17.05	41.8	7	**65.5**	**98.0**
81+00 to 86+00	772.10	760.0	12.10	4.80					16.90	41.3	1		**89.4**
86+00 to 90+00	771.75	760.0	11.75	4.80					16.55	40.3	1		**88.0**
90+00 to 95+00	771.50	760.0	11.50	4.80					16.30	39.5	1		**87.0**
95+00 to 96+00	771.15	760.0	11.15	4.85					16.00	38.4	1		**86.0**
96+00 to 100+00	771.00	760.0	11.00	4.90	6	8			15.90	38.0	6	**59.5**	**85.8**
100+00 to 105+00	770.60	760.0	10.60	5.20	6	8			15.80	36.8	6	**57.9**	**86.4**
105+00 to 110+00	770.45	762.0	8.45	5.10					13.55	30.4	1		**77.1**
110+00 to 115+00	770.25	760.0	10.25	5.05					15.30	35.8	1		**83.9**
115+00 to 120+00	770.10	756.0	14.10	4.95	8	20	4	10	19.05	47.3	2		**126.5**
120+00 to 125+00	769.90	756.0	13.90	4.90	8	20	4	10	18.80	46.7	2		**125.4**
125+00 to 129+00	769.70	756.0	13.70	4.85	8	20	4	10	18.55	46.1	2		**124.2**
129+00 to 130+00	769.60	758.0	11.60	4.75					16.35	39.8	1		**87.0**
130+00 to 135+00	769.55	764.0	5.55	4.75					10.30	21.6	5	**32.7**	**62.8**
135+00 to 140+00	769.40	764.0	5.40	4.75					10.15	21.2	5	**32.1**	**62.2**
140+00 to 145+00	769.20	764.0	5.20	4.75					9.95	20.6	5	**31.3**	**61.4**
145+00 to 150+00	769.00	764.0	5.00	4.75					9.75	20.0	5	**30.5**	**60.6**
150+00 to 156+50	768.85	764.0	4.85	4.75					9.60	19.6	5	**29.9**	**60.0**
156+50 to 160+00	768.65	764.0	4.65	4.70					9.35	18.9	1		**58.8**
160+00 to 165+00	768.55	764.0	4.55	4.65					9.20	18.6	1		**58.1**
165+00 to 170+00	768.40	764.0	4.40	4.65					9.05	18.2	1		**57.5**
170+00 to 175+00	768.25	764.0	4.25	4.60					8.85	17.8	1		**56.5**
175+00 to 180+00	768.05	764.0	4.05	4.55					8.60	17.1	1		**55.3**
180+00 to 185+00	767.85	764.0	3.85	4.40					8.25	16.6	1		**53.4**
185+00 to 189+00	767.65	752.0	15.65	4.30	8	20	4	10	19.95	51.9	2		**127.9**
189+00 to 197+00											Floodwall		
197+00 to 200+00	767.05	754.0	13.05	4.30					17.35	44.1	1		**89.4**
200+00 to 205+00	767.00	754.0	13.00	4.00					17.00	44.0	1		**87.0**
205+00 to 206+12.43	766.50	756.0	10.50	4.15					14.65	36.5	1		**78.1**
212+00 to 225+44											Floodwall		
225+44 to 228+00											Stoplog Gap		
228+00 to 230+00	765.40	750.0	15.40	4.20	10	15			19.60	51.2	?	**82.1**	**113.1**
230+00 to 235+00	765.30	750.0	15.30	4.10	10	15			19.40	50.9	?	**81.7**	**112.0**
235+00 to 240+00	765.00	748.0	17.00	4.00	10	15	5	15	21.00	56.0	?	**103.5**	**130.5**
240+00 to 245+00	764.66	748.0	16.66	3.95	10	15	5	15	20.61	55.0	?	**102.1**	**128.8**
245+00 to 246+50	764.35	748.0	16.35	3.80	8	20	4	10	20.15	54.1	2		**126.9**
246+50 to 248+50											Floodwall		
248+50 to 249+30	763.90	756.0	7.90	3.95					11.85		1		**66.2**
249+30 to 250+60											Floodwall		
250+60 to 255+00	763.55	750.0	13.55	4.15	8	10			17.70	45.6	?	**70.7**	**100.3**
255+00 to 257+65	763.45	750.0	13.45	4.15	8	10			17.60	45.4	?	**70.3**	**99.9**
257+65 to 295+50											Floodwall		
295+50 to 300+00	761.65	744.0	17.65	1.80	8	20	4	10	19.45	57.9	2		**117.1**
300+00 to 302+58	761.65	744.0	17.65	1.35	8	20	4	10	19.00	57.9	2		**113.7**
302+58 to 305+00	761.65	746.0	15.65	1.10					16.75	51.9	1		**75.8**
305+00 to 310+00	760.65	748.0	12.65	1.80					14.45	42.9	1		**69.1**
310+00 to 315+00	760.65	748.0	12.65	1.40					14.05	42.9	1		**66.1**
315+00 to 320+00	760.65	752.0	8.65	1.30					9.95	30.9	1		**49.3**
320+00 to 322+85.41	760.65	756.0	4.65	1.20					5.85	18.9	1		**32.6**

Notes:
1. Distance from existing centerline to the existing toe assumes a 1V on 3H landside slope. This is not always the case.
2. Station 185+00 to 189+00, assumed a berm for this work. Should be analyzed.
3. Computed levee proposed toe for the proposed T-wall reaches assume no stability berms required for proposed raises under 17 feet, and may be incorrect in some cases. Numbers shown is the minimum required real estate.
4. Computed levee proposed toe for the proposed T-wall reaches assume stability berms equal to those required for the T-wall for proposed raises over 17 feet, and may be incorrect in some cases.

Numbers shown indicate minimum real estate needs.

EXHIBIT 13: ARMORDALE UNIT INDERSEEPAGE SUMMARY

Armourdale Levee Kansas City Phase 2 Feasibility
Thursday, December 28, 2006
Levee Foundation Information - N500+3, Water to Top of Levee

Blanket Unit Weight = 115.0 pcf (saturated)

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_{bv}	Permeability Ratio Land Side k_f/k_{bh}	Impervious Blanket Thickness (ft) River Side z_{iv}	Impervious Blanket Thickness (ft) Levee z_{ih}	Impervious Blanket Thickness (ft) Land Side z_{il}	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
5+00 UE to 30+00	777.9	746	770	735	735	671	7.9	300	300	11	23	35	64	260	100	1000	0.002176	0.001220	235	820	5.61	0.16	0.84	5.26	
30+00 to 47+00	777.5	746	760	730	730	671	17.5	300	300	16	23	30	59	200	100	1000	0.001879	0.001372	191	729	12.50	0.42	0.84	2.02	OK
47+00 to 57+00	777.4	750	762	740	740	670	15.4	300	300	10	16	22	70	200	100	1000	0.002182	0.001471	188	680	10.81	0.49	0.84	1.71	OK
57+00 to 62+00	777.2	750	762	735	735	670	15.2	300	300	15	21	27	65	350	100	1000	0.001849	0.001378	308	726	9.73	0.36	0.84	2.34	OK
62+00 to 66+00	777.1	750	762	740	740	671	15.1	400	400	10	16	22	69	300	50	1000	0.001903	0.001283	271	779	10.69	0.49	0.84	1.73	OK
66+00 to 73+00	777.0	755	760	748	748	671	17.0	300	300	7	9.5	12	77	175	50	1000	0.002487	0.001899	165	526	12.08	1.01	0.84	0.84	Slurry Cut off Wall to bring to FS to 1.6 from 62+00 to 83+00
73+00 to 79+00	777.0	753	760	732	745	671	17.0	300	300	21	18	15	61	75	50	1000	0.001613	0.001909	75	524	13.73	0.92	0.84	0.92	Slurry Cut off Wall to bring to FS to 1.6 from 62+00 to 83+00
79+00 to 86+00	776.7	750	760	730	730	667	16.7	400	400	20	25	30	63	100	75	1000	0.001409	0.001150	99	869	13.91	0.46	0.84	1.82	OK
86+00 to 93+00	776.5	750	760	735	735	667	16.5	400	400	15	20	25	68	75	100	1000	0.001566	0.001213	75	825	13.62	0.54	0.84	1.55	Minimally Acceptablc
93+00 to 100+00	776.0	745	760	730	730	667	16.0	250	250	15	22.5	30	63	75	125	1000	0.002057	0.001455	74	687	12.40	0.41	0.84	2.04	OK
100+00 to 116+00	775.5	742	758	732	732	667	17.5	350	350	10	18	26	65	150	60	1000	0.002097	0.001300	145	769	13.81	0.53	0.84	1.59	sand layer between ground surface and 5 feet below ground surface not considered a seepage path. Restoration drawings indicate 5' minimum impervious layer on riverside to el 735 to cut off seepage through sand layer. Minimally Acceptable.
116+00 to 130+00	774.5	744	756	733	733	667	18.5	350	350	11	17	23	66	250	125	1000	0.001984	0.001372	231	729	12.43	0.54	0.84	1.56	Minimally Acceptablc
130+00 to 175+00	773.5	742	760	730	730	672	13.5	300	300	12	21	30	58	225	100	1000	0.002188	0.001384	208	722	9.46	0.32	0.84	2.67	sand lenses between ground surface and 755ish no considered a seepage path. Restoration drawings indicate 5' minimum impervious layer on riverside to el 740 to cut off seepage through sand lenses. OK except for "ditch" at Sta 157+00 behind Mill St where FS = 1.14.
175+00 to 190+00	772.0	742	762	725	725	655	10.0	300	300	17	27	37	70	130	100	1000	0.001674	0.001134	128	881	7.95	0.21	0.84	3.93	OK
190+00 to 195+00	771.5	740	752	728	728	668	19.5	300	300	12	18	24	60	80	100	1000	0.002152	0.001521	79	657	15.32	0.64	0.84	1.32	existing relief wells not considered in analysis. there is apparent sand pockets on the landside, but were ignored in analysis as rehab drawings indicate impervious fill on riverside to elevation 740ish. add new wells to get FS = 1.6 under buildings
195+00 to 205+00	771.0	740	752	728	728	668	19.0	300	300	12	18	24	60	80	100	1000	0.002152	0.001521	79	657	14.93	0.62	0.84	1.36	existing relief wells not considered in analysis. there is apparent sand pockets on the landside below the extent of the impervious fill on the river side el 745, for the sake of this analysis, it is assumed they are not continuous under the levee add new wells to get FS = 1.6 under buildings
205+00 to 215+00	770.5	740	752	728	728	668	18.5	300	300	12	18	24	60	80	100	1000	0.002152	0.001521	79	657	14.54	0.61	0.84	1.39	existing relief wells not considered in analysis. there is apparent sand pockets on the landside, but were ignored in analysis as rehab drawings indicate impervious fill on riverside to elevation 742ish. add new wells to get FS = 1.6 under buildings
215+00 to 225+00	770.0	740	748	722	722	668	22.0	300	300	18	22	26	54	80	100	1000	0.001852	0.001541	79	649	17.24	0.66	0.84	1.27	existing relief wells not considered in analysis. add new well to get FS = 1.6 under buildings
225+00 to 235+00	769.5	740	748	725	725	668	21.5	300	300	15	19	23	57	80	100	1000	0.001974	0.001595	79	627	16.72	0.73	0.84	1.16	existing relief wells not considered in analysis. add new well to get FS = 1.6 under buildings
235+00 to 245+00	769.0	740	749	730	730	668	20.0	300	300	10	14.5	19	62	80	100	1000	0.002319	0.001682	79	594	15.37	0.81	0.84	1.04	existing relief wells not considered in analysis. add new well to get FS = 1.6 under buildings
245+00 to 251+00	768.0	738	751	730	730	653	17.0	300	300	8	14.5	21	77	75	95	1000	0.002326	0.001436	74	696	13.68	0.65	0.84	1.29	existing relief wells not considered in analysis. add new well to get FS = 1.6 under buildings
251+00 to 254+00	767.5	741	752	740	740	663	15.5	400	400	1	6.5	12	77	100	125	1000	0.005698	0.001645	90	608	11.44	0.95	0.84	0.88	blanket appears to thin out here based on riverside borings. additional borings on the land side may indicate a thicker blanket on the land side. if this is the case, underseepage might not be a problem. add new wells to get FS = 1.6
254+00 to 265+00	767.0	744	752	728	728	671	15.0	400	400	16	20	24	57	125	50	1000	0.001656	0.001352	123	740	12.15	0.51	0.84	1.66	analysis does not include shallow sand lenses, as they have been cut off as indicated by rehab drawings showing impervious fill down to 740 on river side. Analysis assumes "slot" is filled to min EL 752 with impervious material landward of levee. need up to date topo and mapping to verify existing ground elevations. a rough field investigation indicates the area is already filled.
265+00 to 275+00	766.2	742	751	728	728	678	15.2	300	300	14	18.5	23	50	100	50	1000	0.002182	0.001703	98	587	12.13	0.53	0.84	1.60	Analysis assumes "slot" is filled to min EL 751 with impervious material landward of levee . need up to date topo and mapping to verify existing ground elevations. a rough field investigation indicates this area is already filled, or can be filled more if needed.
275+00 to 282+00	765.5	742	750	725	725	678	15.5	300	300	17	21	25	47	100	50	1000	0.002043	0.001684	99	594	12.40	0.50	0.84	1.70	Analysis assumes "slot" is filled to min EL 750 with impervious material landward of levee. need up to date topo and mapping to verify existing ground elevations. a rough field investigation indicates this area can be filled.
282+00 to 286+00	764.7	738	746	717	717	678	18.7	400	400	21	25	29	39	75	75	1000	0.001747	0.001487	75	673	15.30	0.53	0.84	1.60	Minimally Acceptablc
286+00 to 296+00	764.5	738	750	725	725	678	14.5	400	400	13	19	25	47	100	50	1000	0.002023	0.001459	99	686	11.92	0.48	0.84	1.77	OK
296+00 to 303+00	763.5	740	742	728	728	678	21.5	400	400	12	13	14	50	75	75	1000	0.002041	0.001890	74	529	16.77	1.20	0.84	0.70	existing wells not considered in analysis. add additional well and use APAC site as temporary ponding
303+00 to 313+00	762.5	740	747	730	730	678	15.5	400	400	10	13.5	17	52	50	100	1000	0.002193	0.001682	50	595	12.38	0.73	0.84	1.16	additional relief wells will be added to bring FS = 1
313+00 to 40+00LE	762.0	745	750	725	725	700	12.0	400	400	20	22.5	25	25	110	100	1000	0.002236	0.002000	108	500	8.48	0.34	0.84	2.49	sand layers are located on riverside and landside between elevations 738ish and ground surface. Restoration drawings indicate 5' minimum impervious layer on riverside to el 740 to cut off seepage through sand layer. performance during 93 in this area indicates the layers do not pose a problem. OK

EXHIBIT 14: SPECIAL FEATURES ANALYZED FOR UNDERSEEPAGE

								Pressures on Base of Blanket			Pressures on Base of Feature					
Building or Feature	Station Start	Station Stop	Distance From Seepage Entrance	Top of Ground	Bottom of Blanket	Design Feature Elevation	Design Feature Elevation Source	$h_{o\text{-}allow}$ to protect blanket (ft)	Existing Conditions - '62-MOD Calculations h_p (ft)	Existing Conditions - Feasibility Calculations h_p (ft)	N500+3 h_p (ft)	Hydrostatic Pressure Head at Feature Elevation (ft)	Existing Pressure Head at Feature Elevation (ft)	N500+3 Pressure Head at Feature Elevation (ft)	Increase in Pressure at Feature Elevation between Existing and N500+3 (psf)	Remedial Measure for Blanket Underseepage
Proctor and Gamble	5700	6200	500	762	735	755	DM3	10.5	7.0	7.0	9.7	7.0	8.8	9.5	0.7	None Required
Inland Container	6800	6800	850 (around wall)	762	743	754	DM3	5.8	9.2	9.2	5.5	8.0	11.9	10.3	-1.6	Cut-Off
Mill Street Pump Station Ditch	15700	15700	400	760	730	752	Topo	-	-	-	-	-	-	-	-	Fill with impervious
Midwest Cold Storage	19200	19700	200	752	728	742	Assumed	7.4	7.4	9.6	7.5	10.0	13.1	13.1	0.0	Wells
Kansas Fish and Oyster	19700	20000	400	752	728	745	DM3	9.0	3.7	5.9	5.8	7.0	8.1	8.7	0.6	Wells
Schock Truck and Leasing	21400	21400	350	752	728	742	Assumed	7.4	4.9	7.1	7.4	10.0	12.0	13.1	1.0	Wells
Jones Store Distributing	21700	21700	300	752	722	742	Assumed	10.5	6.3	8.5	9.0	10.0	12.1	13.0	0.9	Wells
Glass Co - Prime Investments	21800	22000	250	750	722	740	Assumed	9.5	8.3	10.5	8.9	10.0	13.0	13.2	0.2	Wells
Overland Park Bank	22500	22500	270	748	722	738	Assumed	8.4	8.0	10.2	6.8	10.0	13.1	12.6	-0.5	Wells
Sambol Meat Packing	22900	23050	250	748	725	742	DM3	9.0	7.4	9.6	8.0	6.0	7.9	8.1	0.2	Wells
Selco	23100	23500	250	749	730	748	DM3	9.5	8.4	10.6	8.1	1.0	1.4	1.4	0.0	Wells
Selco	23500	23700	350	749	730	748	DM3	9.5	7.8	10.0	7.5	1.0	1.4	1.4	0.0	Wells
PBI Gordon Co	28300	28600	200	746	717	743	DM3	13.7	12.2	12.2	14.0	3.0	4.3	4.4	0.2	None Required

*Assuming buildings built since 2000, including Pensky, Pitney Bowes, and USF Distribution do not have basements

*All building basement elevations were assumed to be 10 feet below grade if not found in DM-3

*Any building not specificially called out in table was not analyzed

*$h_{o\text{-}allow}$ is calculated using a FS_i = 1.60

*h_p is excess head measured from top of surrounding ground"

*Assumed hydrostatic water pressures act on top of blanket at design feature elevation with water elevation at the top of the surrounding ground

*All Feature Elevations should be verified during final design.

*Existing Conditions 1962-MOD calculations were estimated by subtracting 2.2 feet (AVG Difference in calculations at checkpoints) from the Existing Condtions Feasibility Calculations

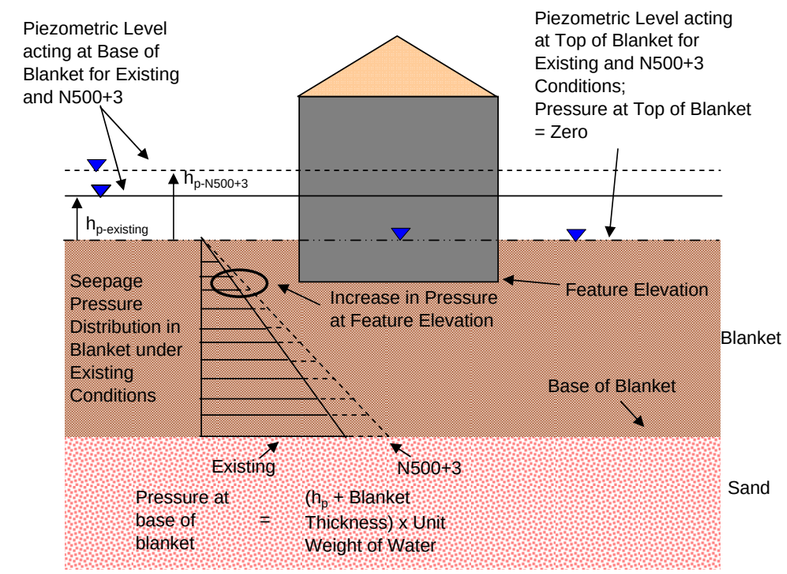
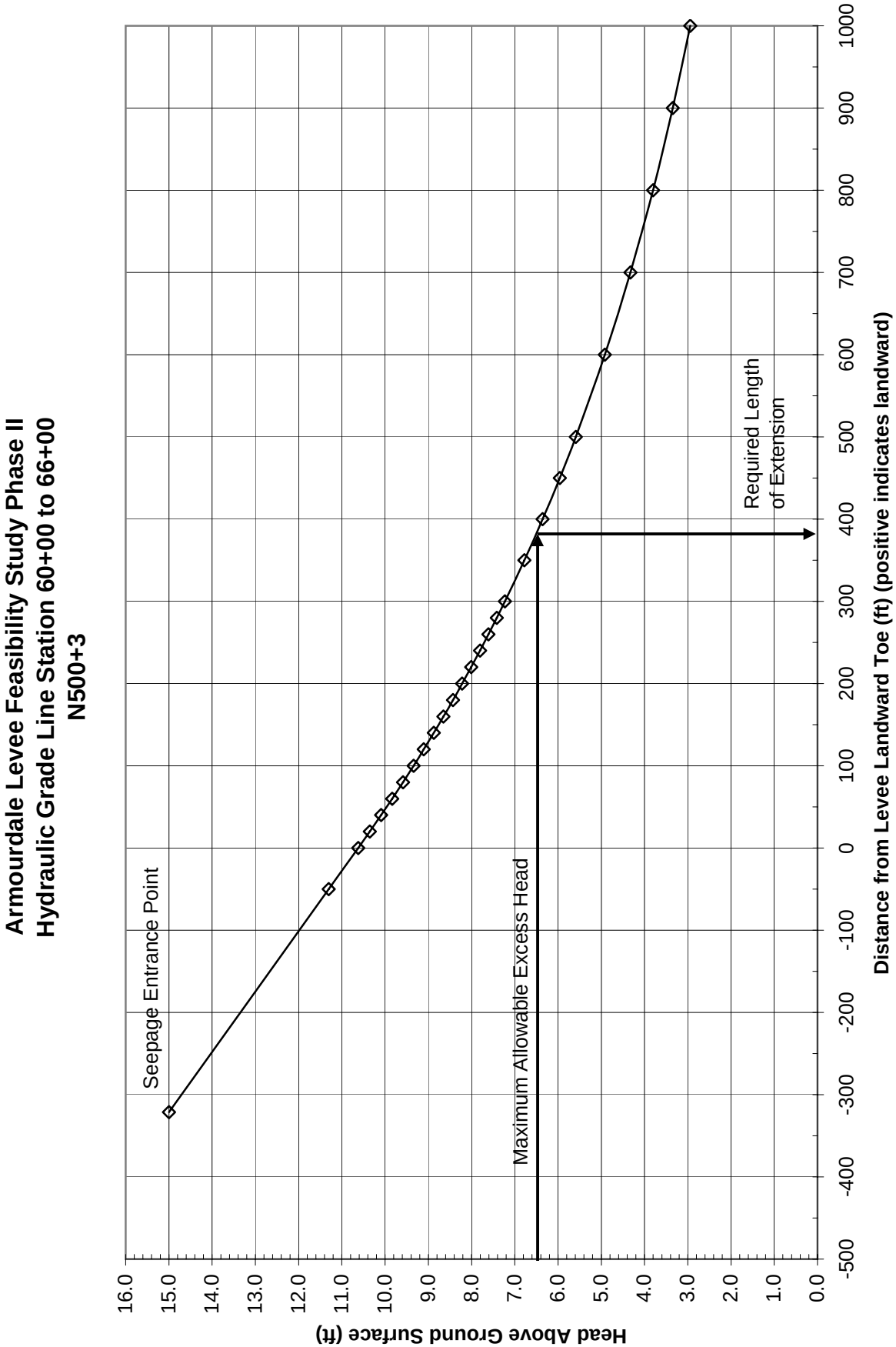


EXHIBIT 15: COMPARISON OF EXISTING CALCULATIONS PERFORMED FOR 1962 MODIFICATION AND FEASIBILITY STUDY

			1962 Modification Calculations			Feasibility Study Calculations			1962 minus Feasibility		
Checkpoint	Station	Seepage Distance	Initial HGL	Drawdown (ft)	HGL	Initial HGL	Drawdown (ft)	HGL	Initial HGL	Drawdown (ft)	HGL
C-C	194+50	235	762.5	4.3	758.3	762.8	2.3	760.5	-0.3	2.0	-2.3
C-D	193+75	400	760.5	4.3	756.2	760.4	2.3	758.1	0.1	2.0	-1.9
C-E	197+00	250	762.5	4.5	758.0	762.6	2.5	760.1	-0.1	2.0	-2.1
C-I	199+30	440	760.0	4.7	755.3	759.9	2.4	757.5	0.1	2.3	-2.2
C-J	204+30	350	760.5	4.5	756.0	761.1	2.2	758.9	-0.6	2.3	-2.9
C-K	212+70	270	762.5	4.6	757.9	761.9	1.8	760.1	0.6	2.8	-2.2
C-L	215+50	270	762.0	3.9	758.1	761.9	1.2	760.7	0.1	2.7	-2.6
C-M	215+50	400	758.5	4.3	754.2	760.2	2.6	757.6	-1.7	1.7	-3.4
C-O	220+20	325	758.0	5.9	752.1	759.3	3.8	755.5	-1.3	2.1	-3.4
C-P	220+70	350	758.0	5.6	752.4	758.9	3.4	755.5	-0.9	2.2	-3.1
C-Q	221+50	240	761.0	5.8	755.3	760.9	5.0	755.9	0.1	0.8	-0.6
C-R	222+00	430	757.9	5.3	752.6	757.6	4.0	753.6	0.3	1.3	-1.0
C-S	222+50	230	761.5	5.6	755.9	761.1	4.6	756.5	0.4	1.0	-0.6
C-T	225+20	240	761.0	4.2	756.8	760.9	2.1	758.8	0.1	2.1	-2.0
C-U	225+00	365	759.5	4.6	754.9	758.1	1.6	756.5	1.4	3.0	-1.6
C-V	225+00	500	757.5	4.6	752.9	756.2	1.3	754.9	1.3	3.3	-2.0
C-W	229+40	280	759.0	4.7	754.3	759.6	2.6	757.0	-0.6	2.1	-2.7
C-X	231+50	200	760.5	4.1	756.4	761.2	2.2	759.0	-0.7	1.9	-2.6
C-Y	231+80	330	758.5	4.4	754.2	758.2	1.1	757.1	0.3	3.3	-3.0
C-A'	234+70	420	759.0	4.2	754.8	756.9	1.4	755.5	2.1	2.8	-0.7
C-C'	236+70	280	759.0	4.4	754.6	759	1.9	757.1	0.0	2.5	-2.5
C-D'	240+80	230	760.0	7.8	752.2	759.9	5.6	754.3	0.1	2.2	-2.1
C-E'	241+00	400	757.5	6.6	750.9	757.2	4.4	752.8	0.3	2.2	-1.9
C-H'	242+75	250	759.0	7.8	751.2	759.6	5.5	754.1	-0.6	2.3	-2.9
C-I'	243+50	350	758.0	6.9	751.1	757.9	4.6	753.3	0.1	2.3	-2.2
C-J'	245+20	210	760.0	7.2	752.8	760.3	5.1	755.2	-0.3	2.1	-2.4
C-K'	247+20	250	760.0	5.1	754.9	759.6	2.8	756.8	0.4	2.3	-1.9
C-M'	249+20	320	759.0	4.2	754.8	758.4	1.6	756.8	0.6	2.6	-2.0

EXHIBIT 16: HYDRAULIC GRADE LINE USED TO DETERMINE CUTOFF WALL EXTENSION, STATIONS 66+00 AND 79+00



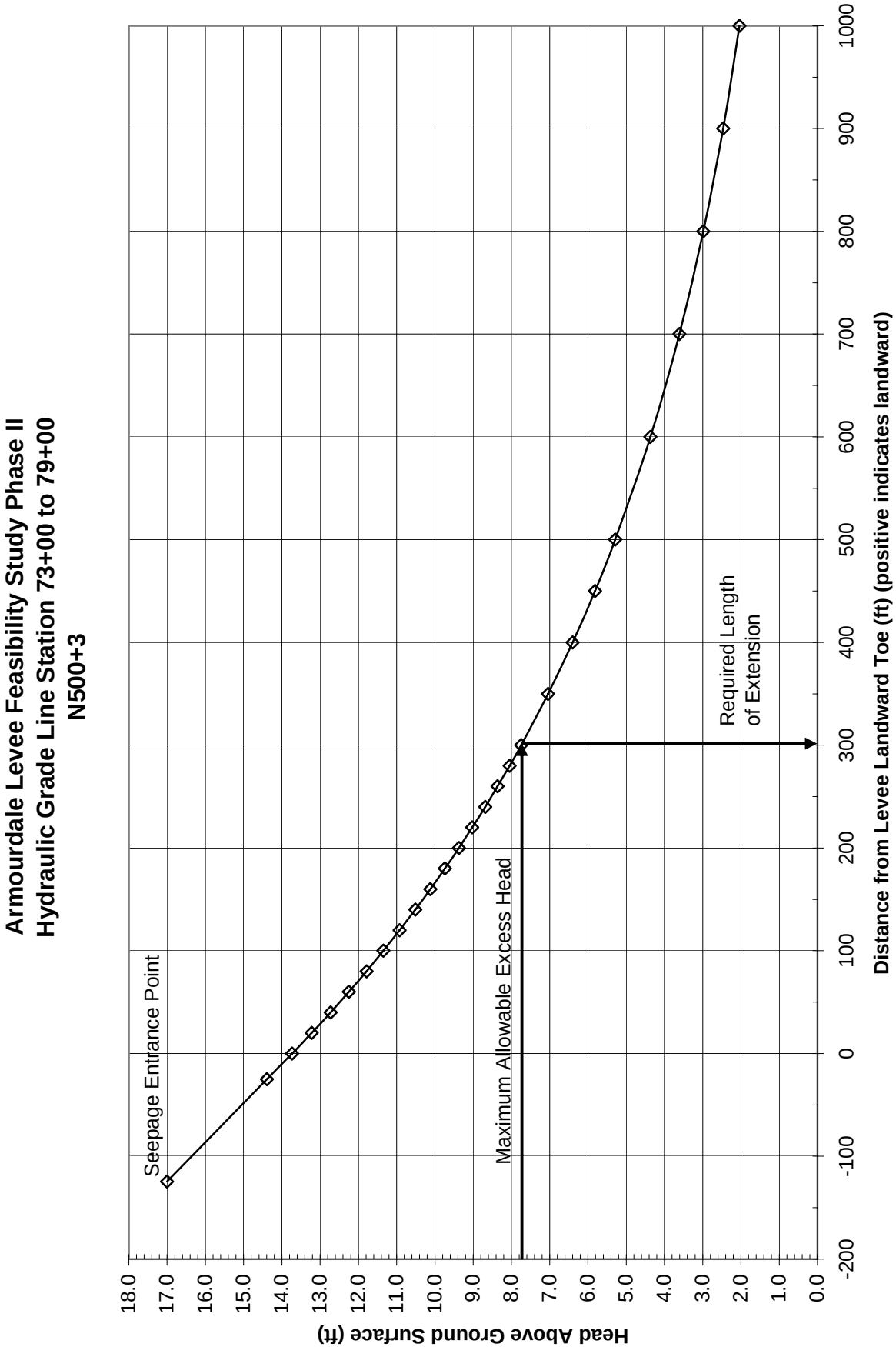

Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 73+00 to 79+00
N500+3
Seepage Entrance Point
Maximum Allowable Excess Head
Required Length
of Extension
Head Above Ground Surface (ft)
18.0
17.0
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
-200
-100
0
100
200
300
400
500
600
700
800
900
1000
Distance from Levee Landward Toe (ft) (positive indicates landward)

EXHIBIT 17: WELL SYSTEM SUMMARY - STATION 190+00 TO 254+00

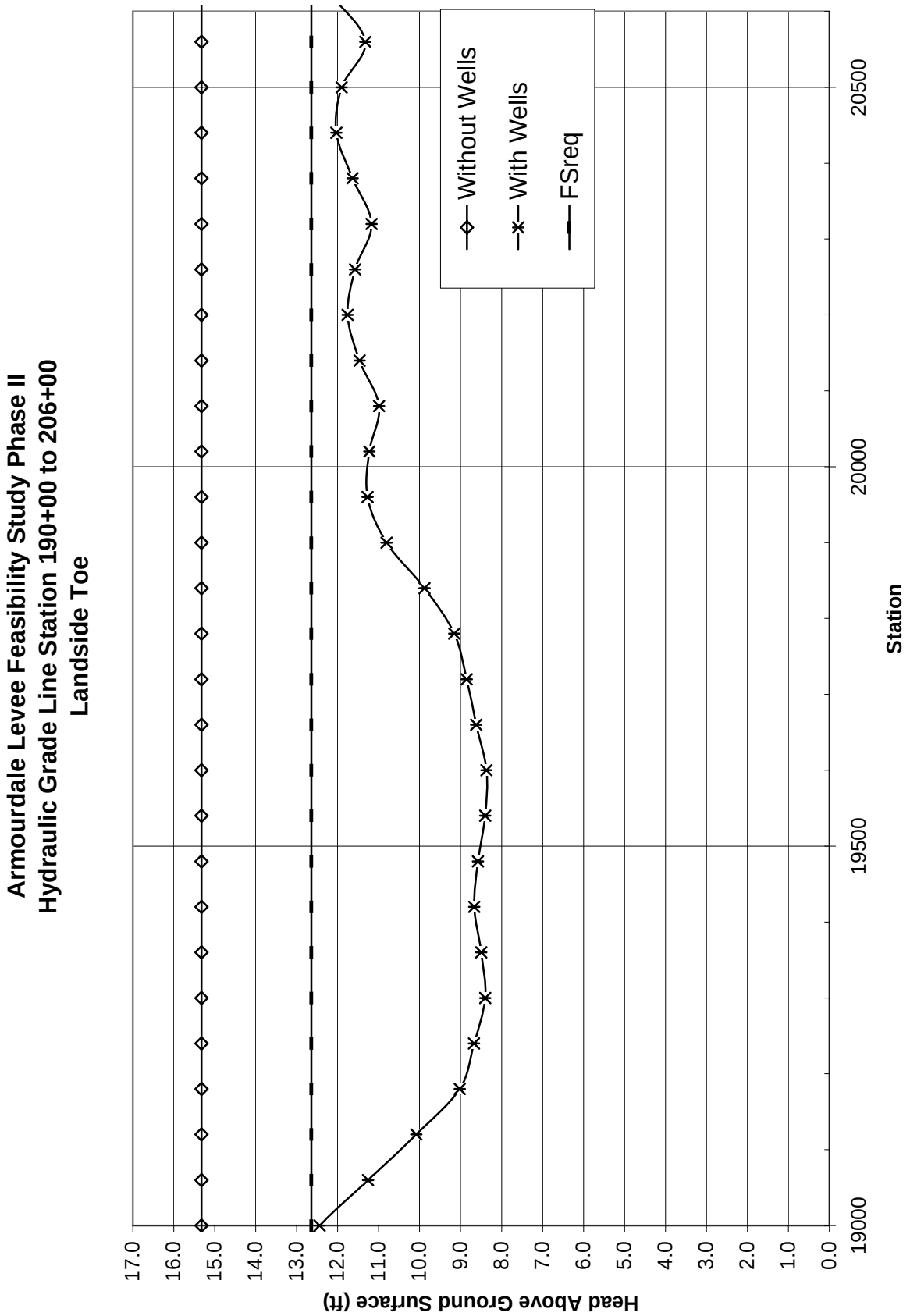
Proposed Relief Well System, Station 190+00 to 254+00

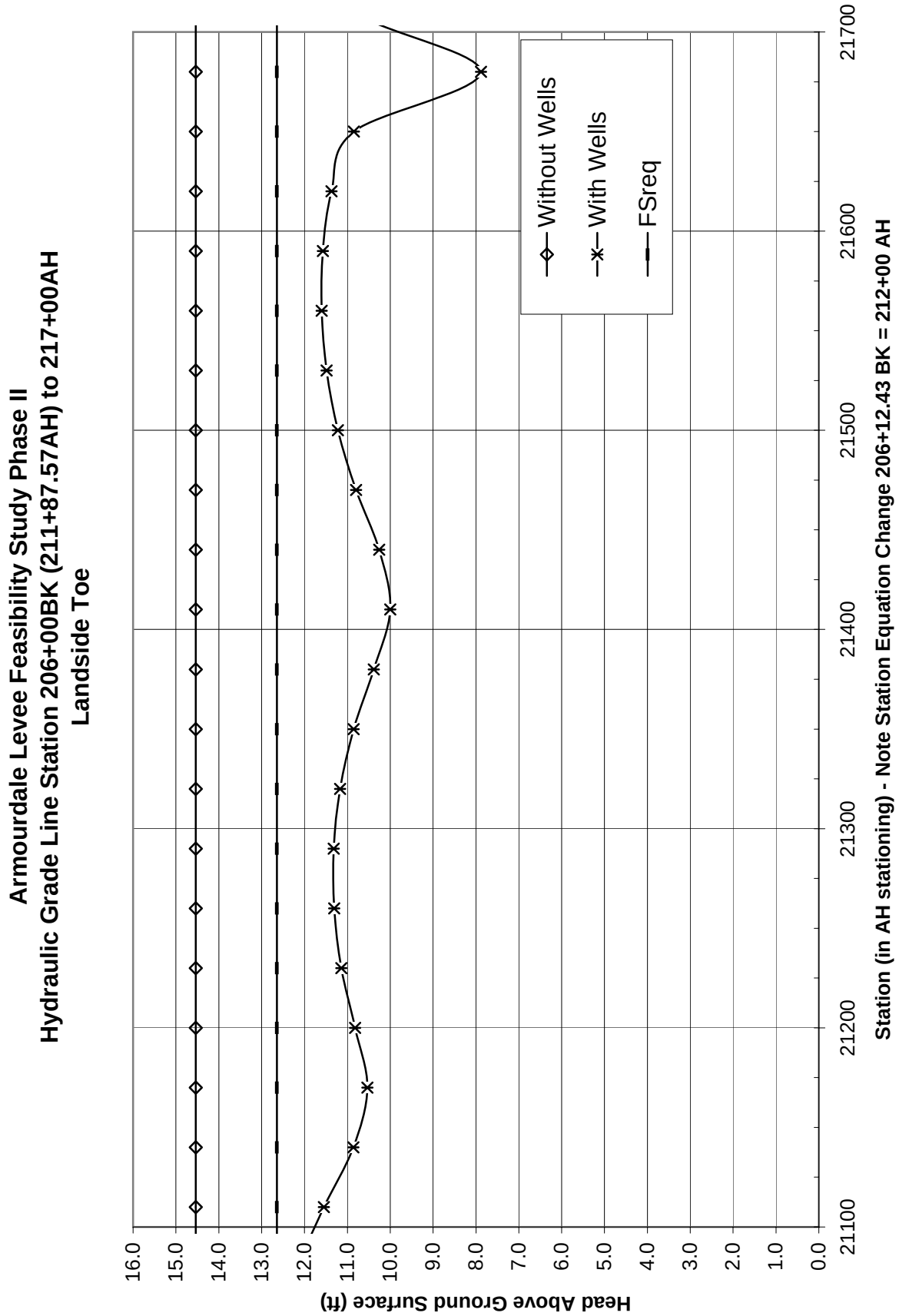
Status	Well	Distance From Seepage Entrance	Station	Station Designation	Discharge Elevation (ft)	Flow (cfs)
Existing	1	230	19075	BK	751.0	0.97
Proposed	25	220	19150	BK	752.0	0.77
Proposed	26	220	19175	BK	752.0	0.74
Proposed	27	220	19200	BK	752.0	0.73
Proposed	28	220	19250	BK	752.0	0.72
Proposed	29	220	19300	BK	752.0	0.70
Existing	2	230	19325	BK	749.0	0.82
Proposed	30	220	19350	BK	752.0	0.70
Proposed	31	220	19430	BK	752.0	0.74
Proposed	32	220	19490	BK	752.0	0.73
Proposed	33	220	19550	BK	752.0	0.72
Existing	3	240	19580	BK	747.5	0.91
Proposed	34	220	19610	BK	752.0	0.73
Proposed	35	220	19670	BK	752.0	0.77
Proposed	36	220	19730	BK	752.0	0.80
Proposed	37	220	19780	BK	752.0	0.84
Existing	4	230	19830	BK	746.0	1.24
Existing	5	230	20080	BK	743.5	1.81
Existing	6	230	20325	BK	742.5	2.09
Existing	7	220	20574	BK	740.5	2.67
Existing	8	220	21415	AH	739.0	2.72
Proposed	38	180	21680	AH	752.0	1.49
Proposed	39	180	21800	AH	748.0	1.29
Existing	9	220	21916	AH	736.5	3.07
Existing	10	230	22016	AH	736.0	2.02
Existing	11	240	22209	AH	734.5	2.12
Existing	12	240	22407	AH	733.5	2.66
Proposed	40	180	22450	AH	748.0	1.43
Proposed	41	180	22500	AH	748.0	1.41
Proposed	42	180	22600	AH	748.0	1.45
Proposed	43	180	22700	AH	748.0	1.50
Existing	13	230	22838	AH	731.0	2.97
Existing	14	240	23053	AH	730.0	3.08
Existing	15	200	23238	AH	739.5	2.25
Existing	16	200	23396	AH	741.5	1.87
Proposed	44	180	23460	AH	748.0	1.31
Proposed	45	180	23500	AH	748.0	1.30
Proposed	46	180	23560	AH	749.0	1.29
Proposed	47	180	23680	AH	749.0	1.43
Existing	17	220	23795	AH	743.5	2.02
Proposed	48	180	23900	AH	749.0	1.17
Existing	18	180	23995	AH	744.5	1.51
Existing	19	180	24095	AH	745.0	1.47
Existing	20	180	24189	AH	745.5	1.43
Existing	21	180	24290	AH	746.5	2.02
Existing	22	180	24396	AH	747.0	1.75
Proposed	49	180	24460	AH	749.0	1.49
Existing	23	180	24515	AH	747.5	1.65
Existing	24	190	24635	AH	749.0	1.58
Proposed	50	180	24800	AH	751.0	1.44
Proposed	51	180	24950	AH	751.0	1.32
Proposed	52	180	25075	AH	751.0	1.13
Proposed	53	180	25100	AH	752.0	1.01
Proposed	54	180	25130	AH	752.0	1.01
Proposed	55	180	25170	AH	752.0	1.06
Proposed	56	180	25210	AH	752.0	1.15
Proposed	57	180	25240	AH	752.0	0.78
Proposed	58	180	25280	AH	752.0	0.79
Proposed	59	180	25320	AH	752.0	0.80
Proposed	60	180	25360	AH	752.0	0.81
Proposed	61	180	25390	AH	752.0	0.82
Proposed	62	180	25420	AH	752.0	0.85
Proposed	63	180	25440	AH	752.0	0.90

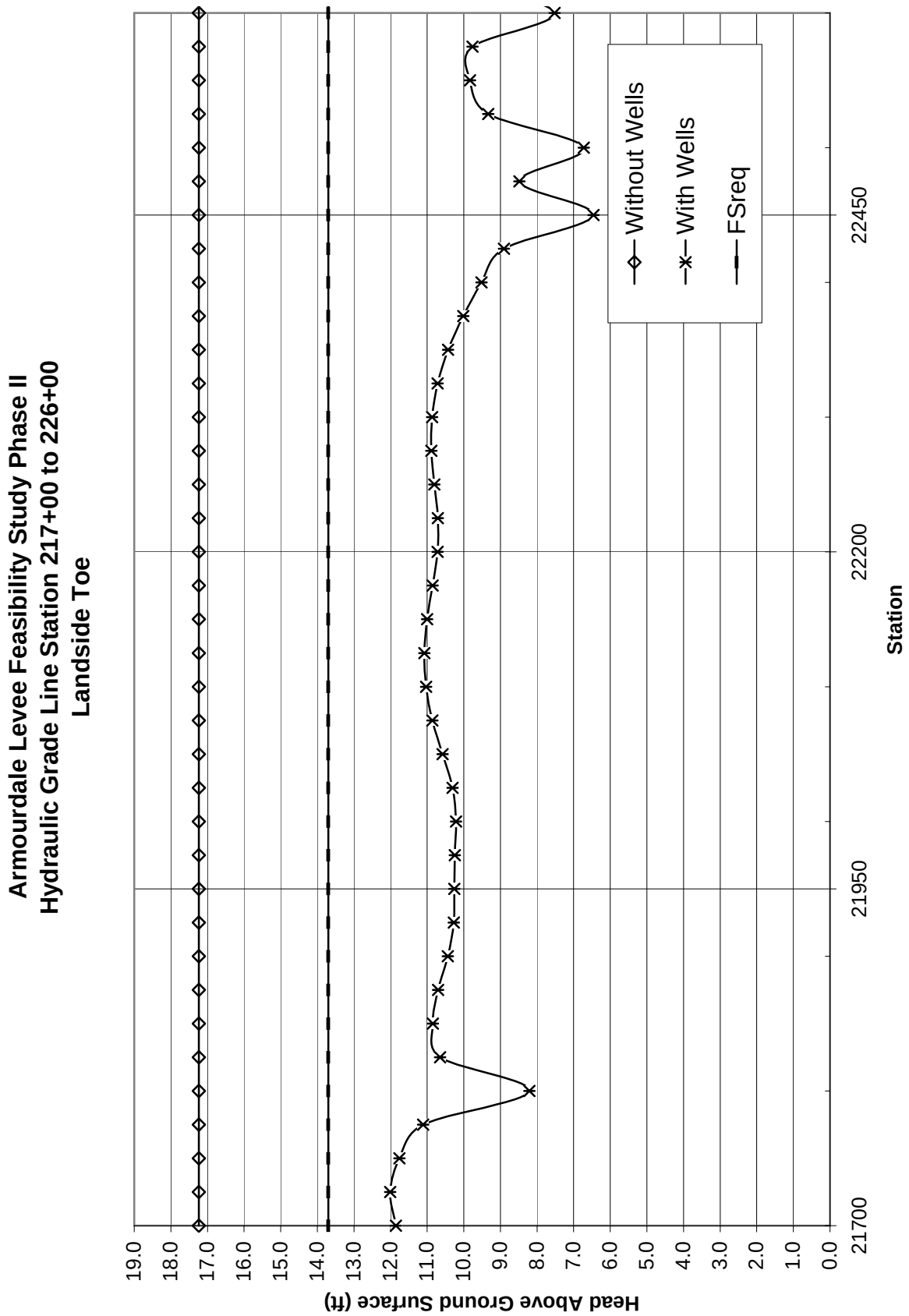
*Relief well flows shown are expected flows. The flows have been reduced by 80% in the calculations to account for future well efficiency reduction

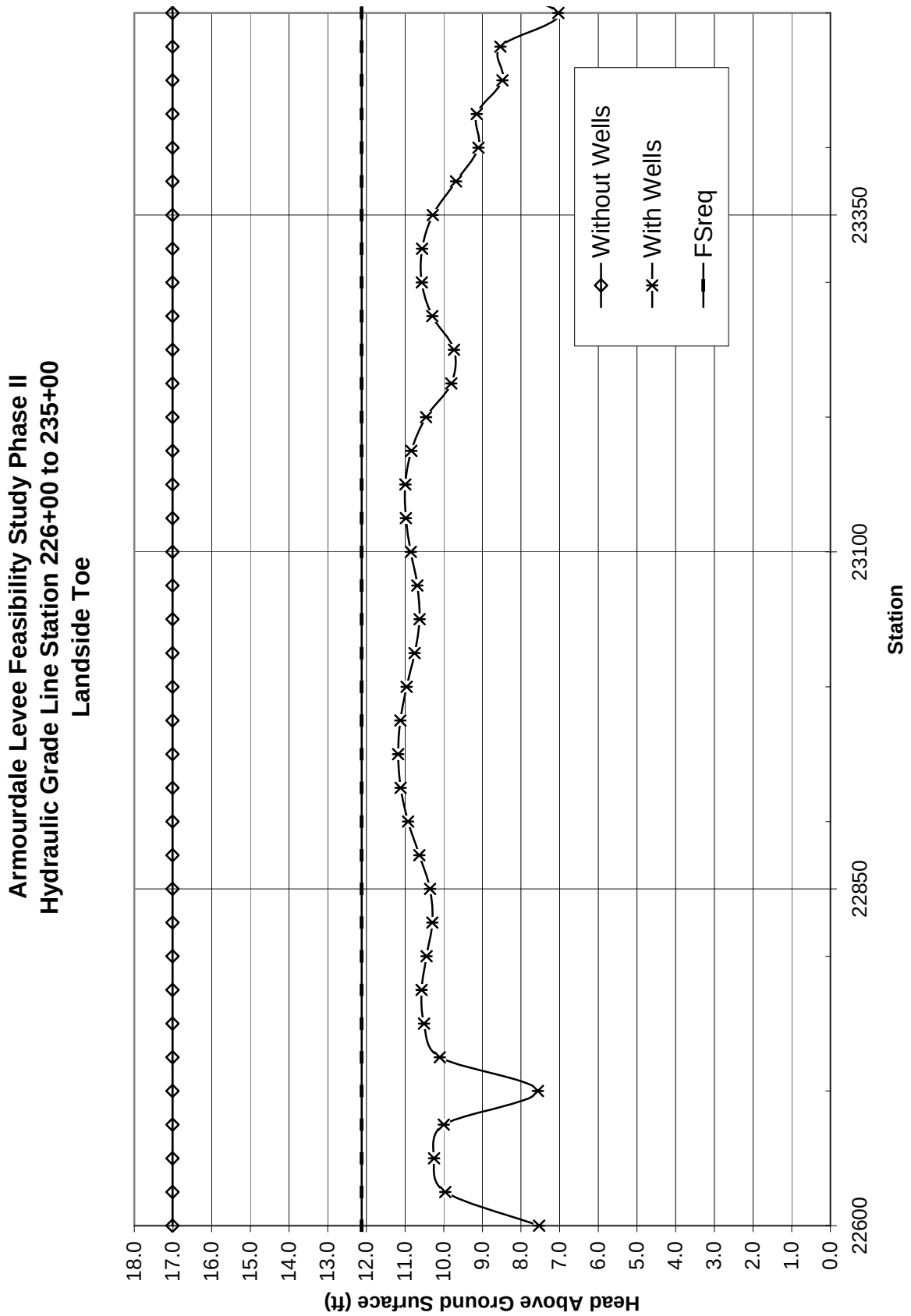
*Seepage engrance is assumed to be at the elevation of the bottom of the riverside blanket

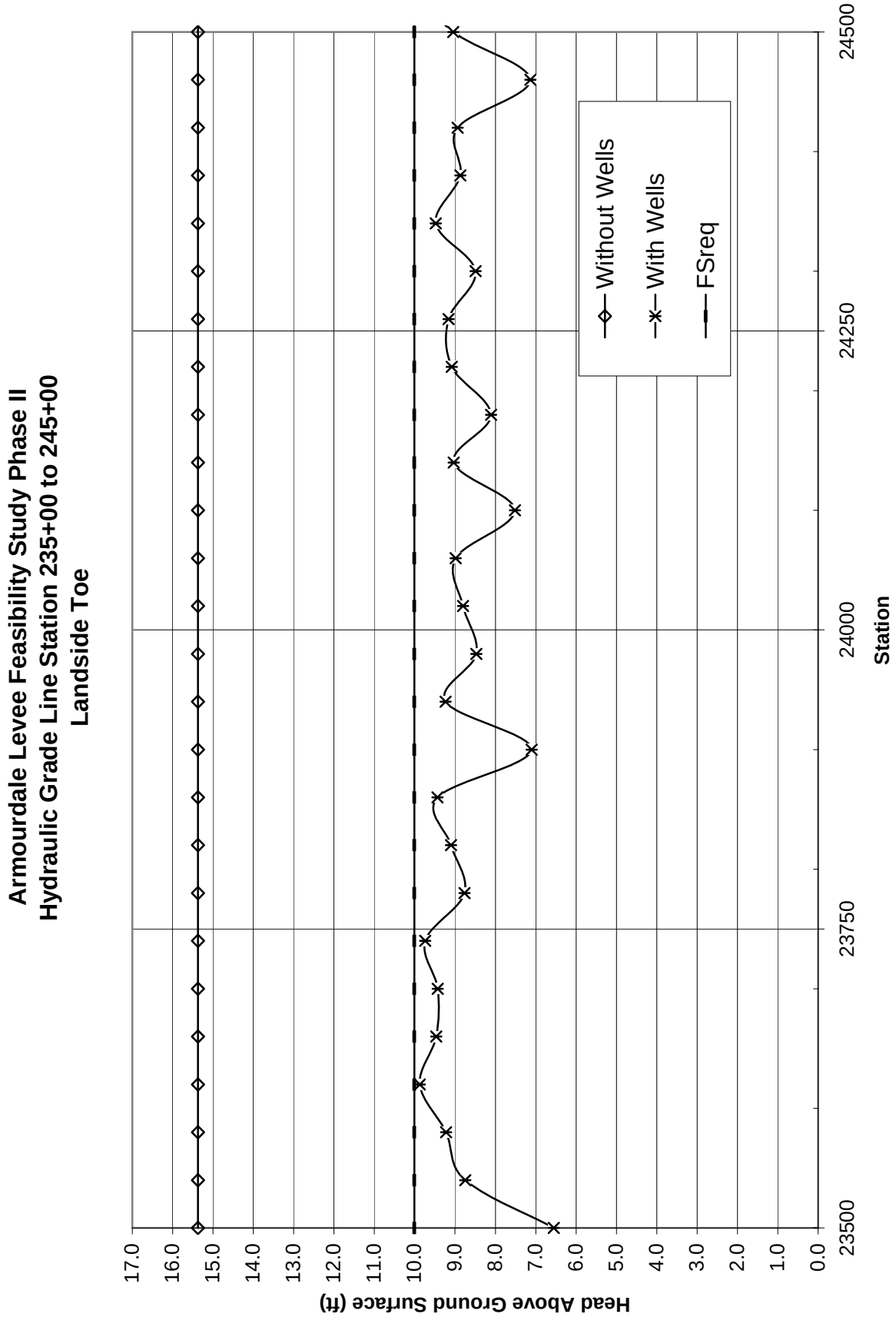
EXHIBIT 18: COMPUTED EXCESS HEAD AT LANDSIDE TOE

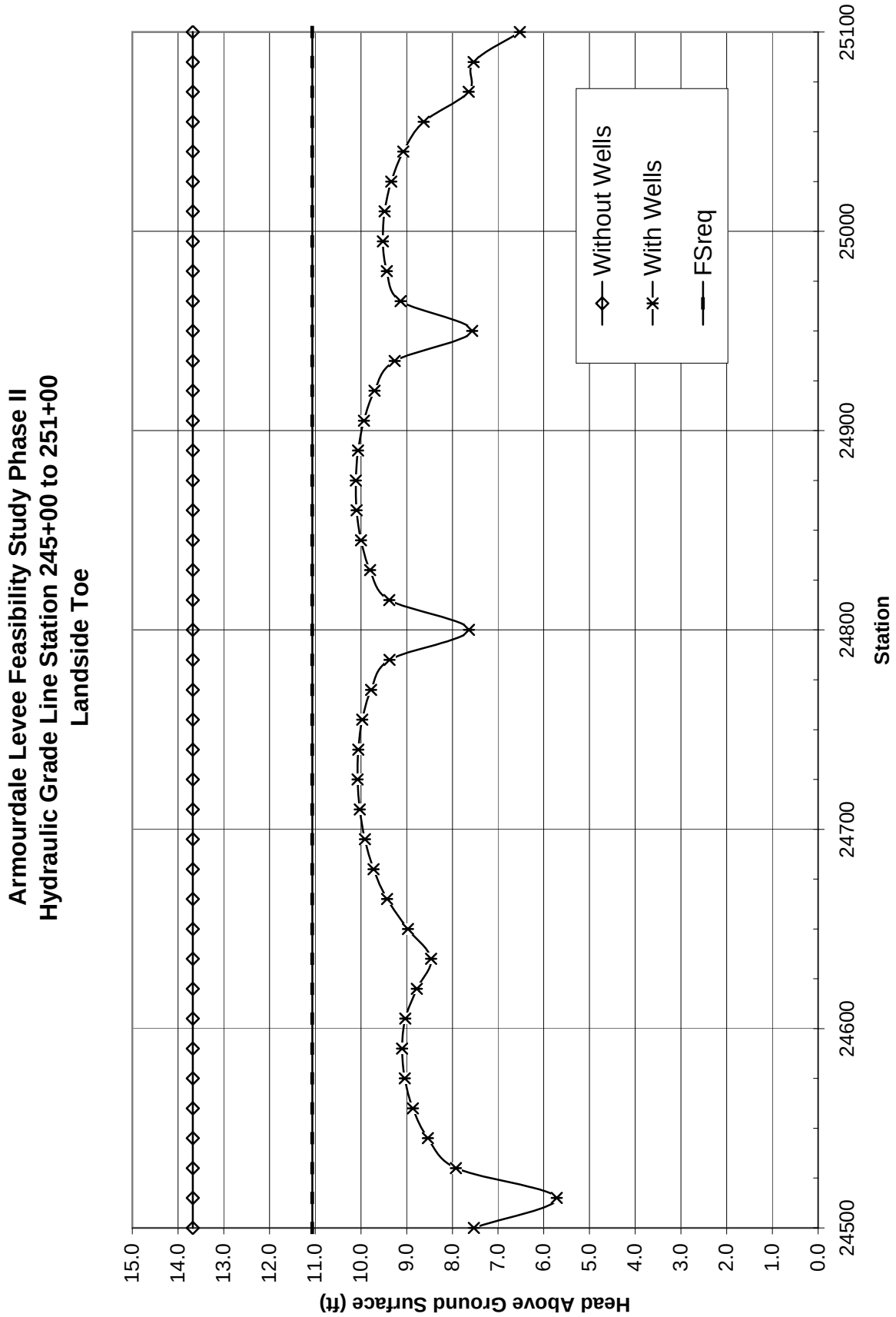



Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 206+00BK (211+87.57AH) to 217+00AH
Landside Toe
Head Above Ground Surface (ft)
16.0
15.0
14.0
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
21100
21200
21300
21400
21500
21600
21700
Station (in AH stationing) - Note Station Equation Change 206+12.43 BK = 212+00 AH
Without Wells
With Wells
FSreq


Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 217+00 to 226+00
Landside Toe
Head Above Ground Surface (ft)
19.0
18.0
17.0
16.0
15.0
14.0
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
21700
21950
22200
22450
Station
Without Wells
With Wells
FSreq


Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 226+00 to 235+00
Landside Toe
Head Above Ground Surface (ft)
18.0
17.0
16.0
15.0
14.0
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
22600
22850
23100
23350
Station
Without Wells
With Wells
FSreq


Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 235+00 to 245+00
Landside Toe
Head Above Ground Surface (ft)
17.0
16.0
15.0
14.0
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
23500
23750
24000
24250
24500
Station
Without Wells
With Wells
FSreq


Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 245+00 to 251+00
Landside Toe
Head Above Ground Surface (ft)
15.0
14.0
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
24500
24600
24700
24800
24900
25000
25100
Station
Without Wells
With Wells
FSreq

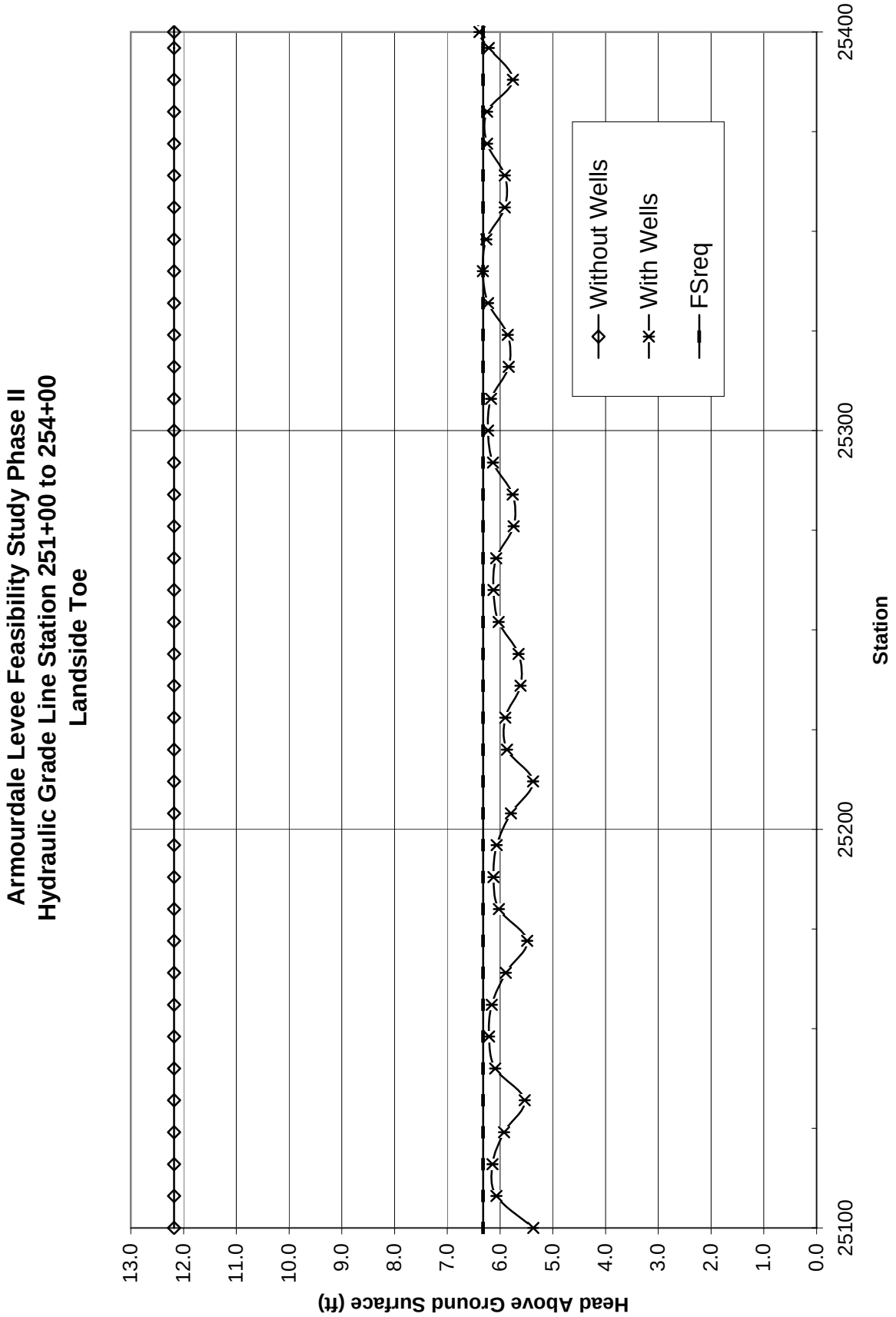
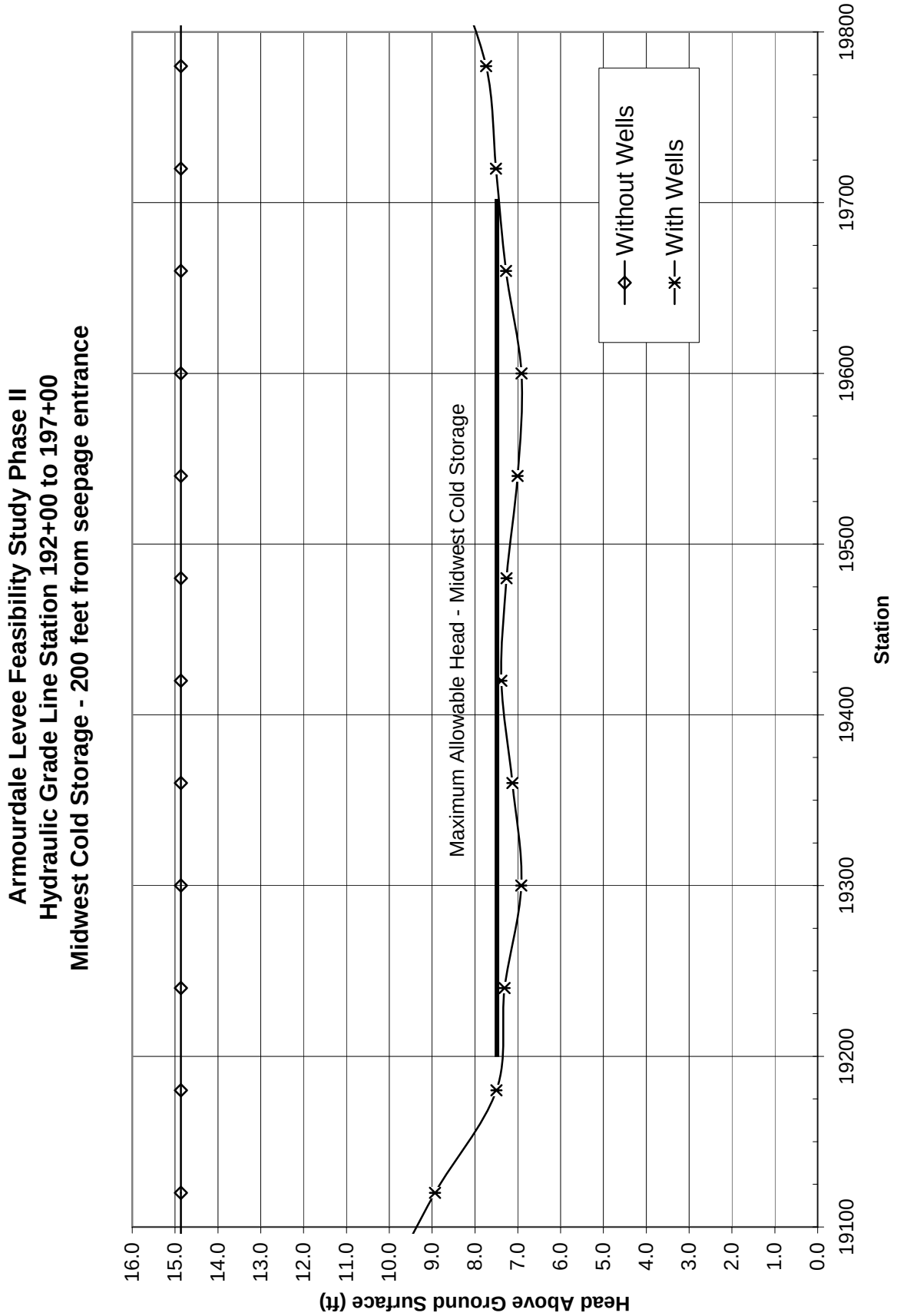
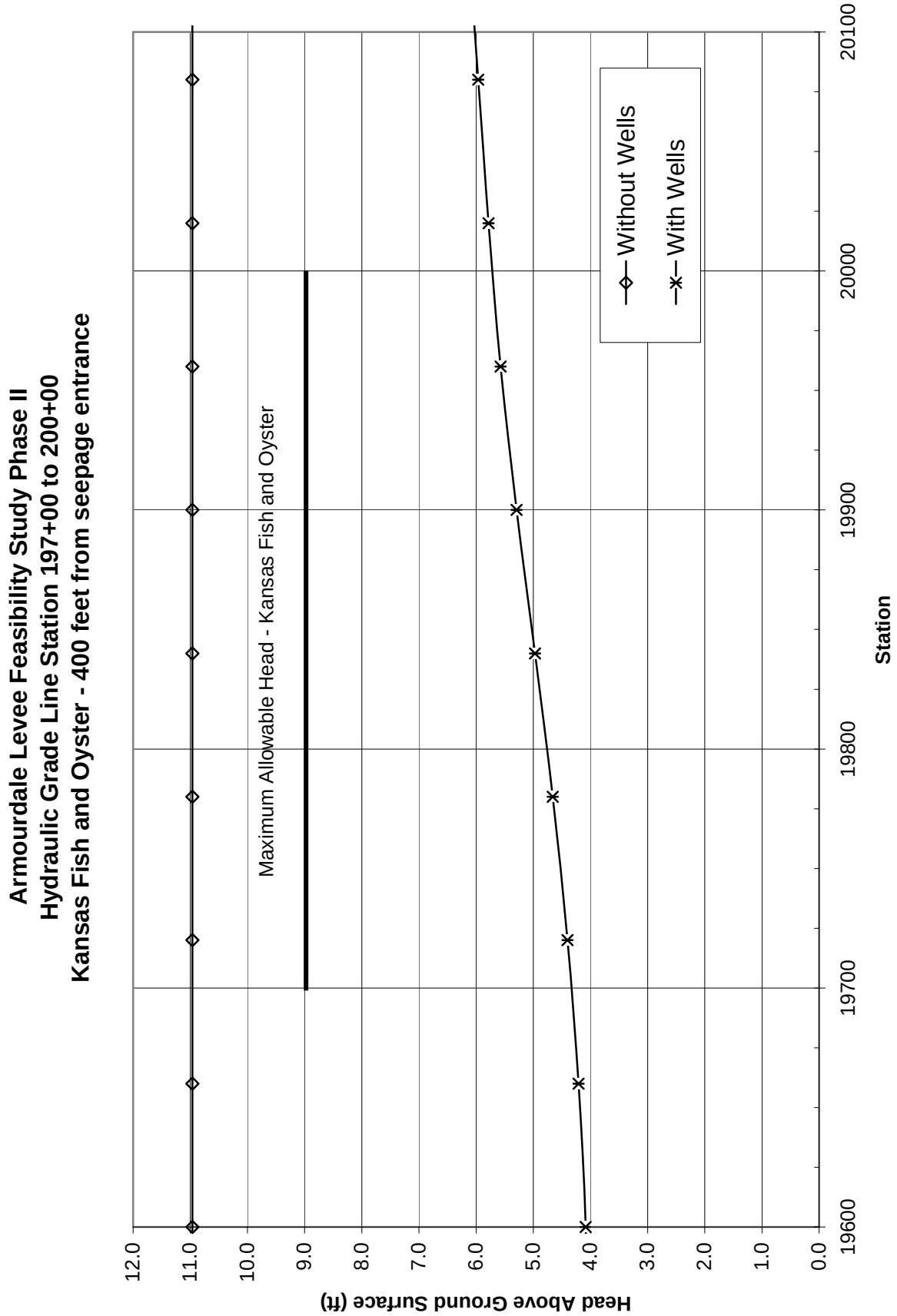
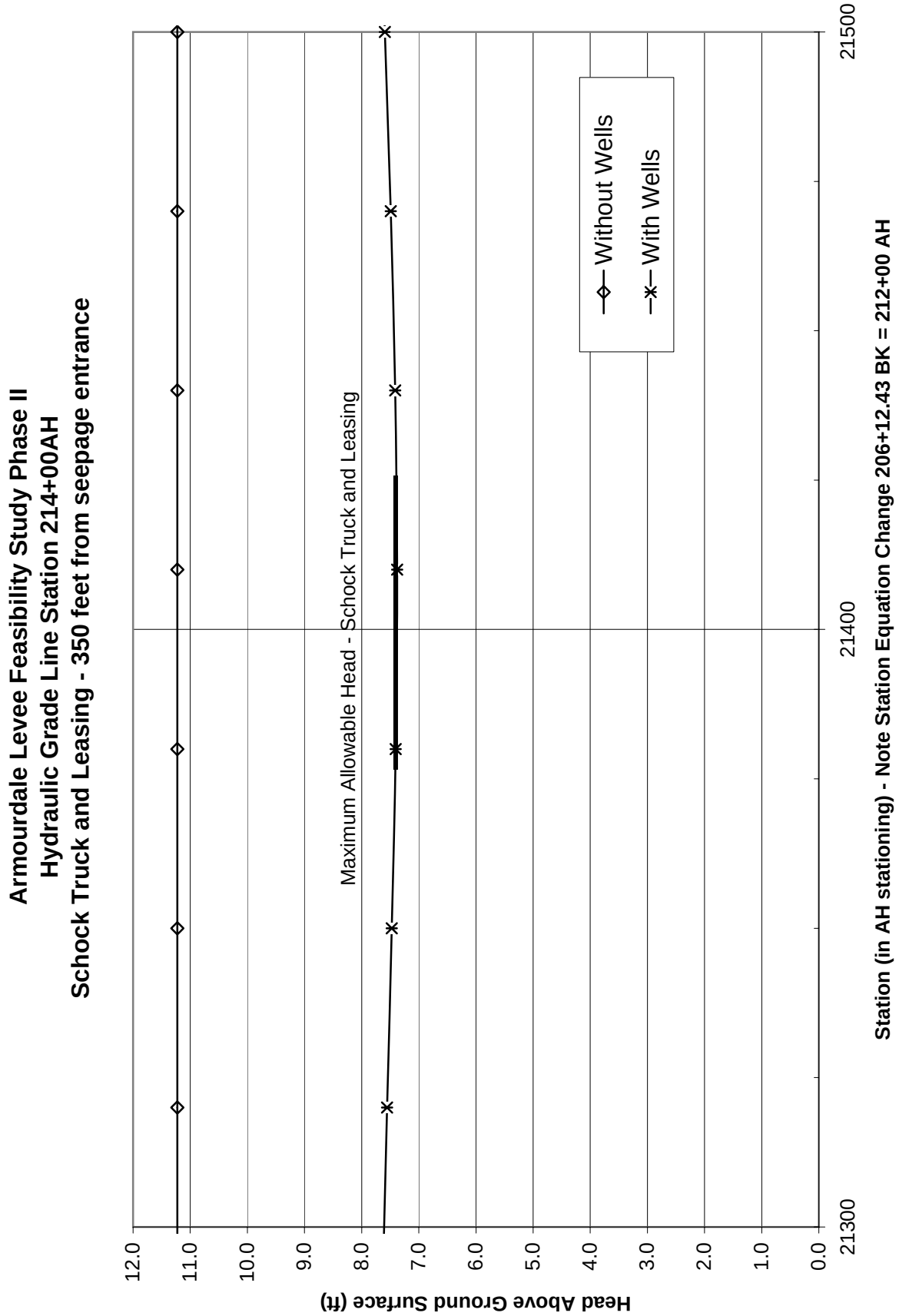
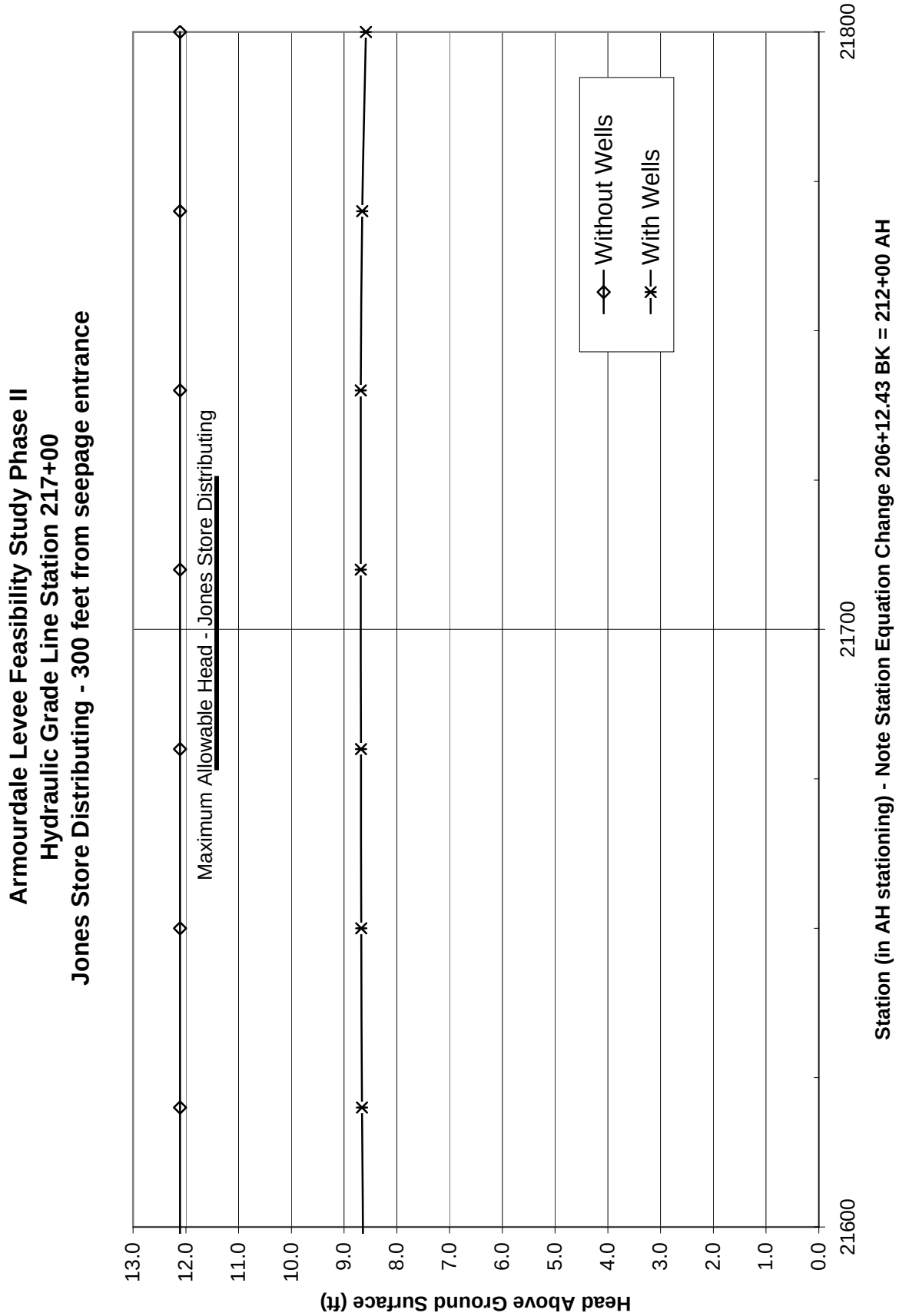

Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 251+00 to 254+00
Landside Toe
Head Above Ground Surface (ft)
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
25100
25200
25300
25400
Station
Without Wells
With Wells
FSreq

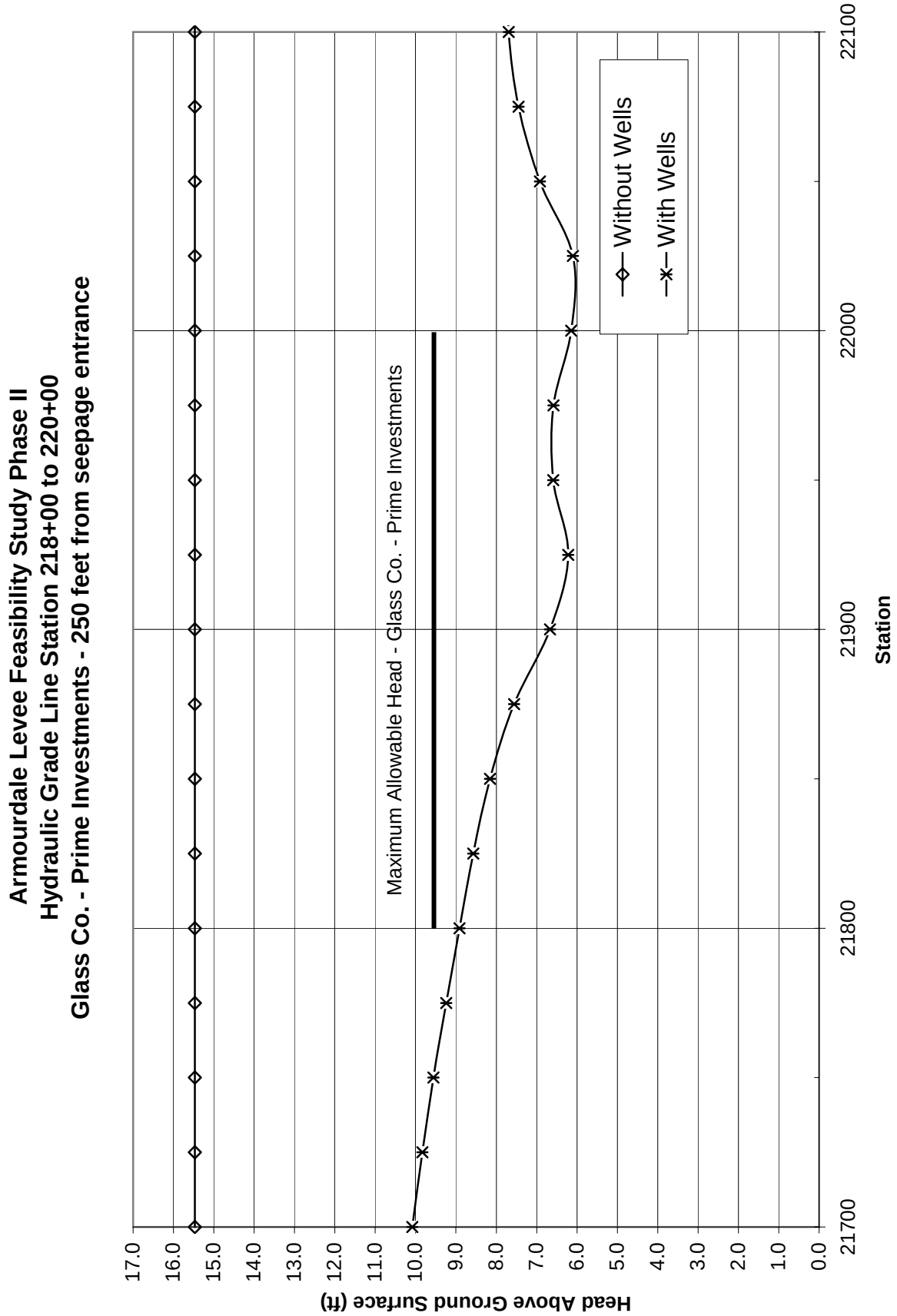
EXHIBIT 19: COMPUTED EXCESS HEAD AT SPECIAL FEATURES ANALYZED

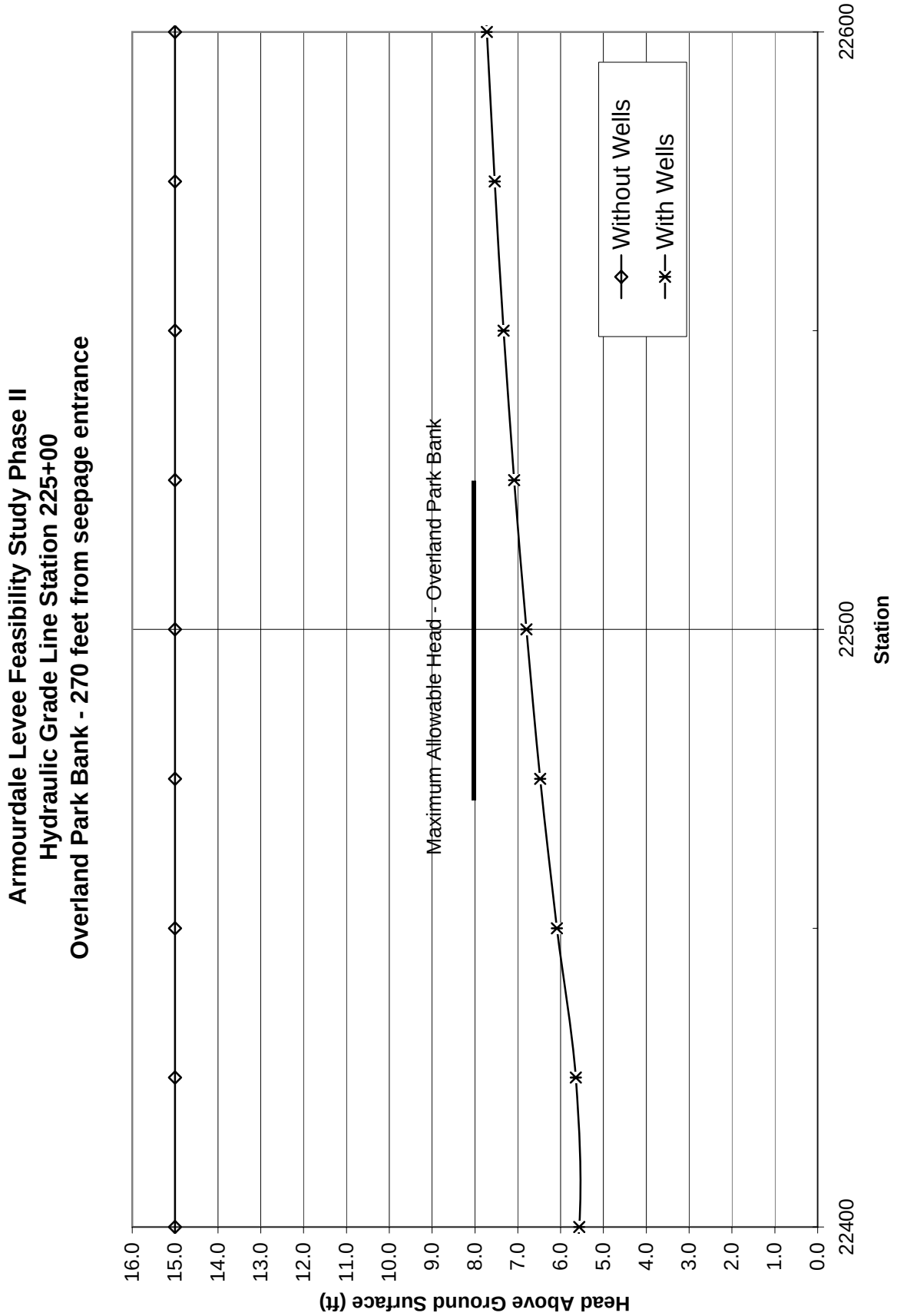


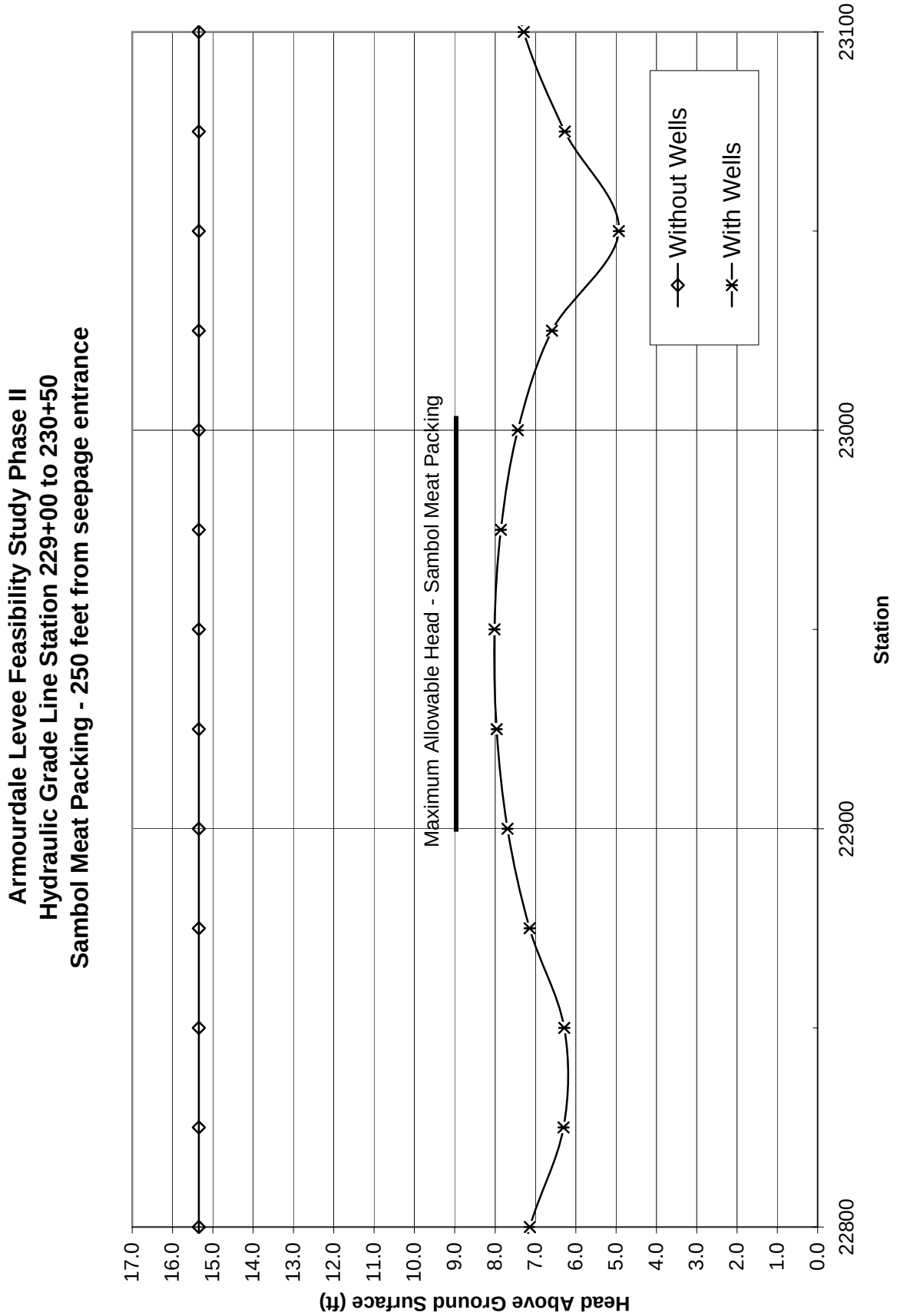

Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 197+00 to 200+00
Kansas Fish and Oyster - 400 feet from seepage entrance
Maximum Allowable Head - Kansas Fish and Oyster
Without Wells
With Wells
Head Above Ground Surface (ft)
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
19600
19700
19800
19900
20000
20100
Station

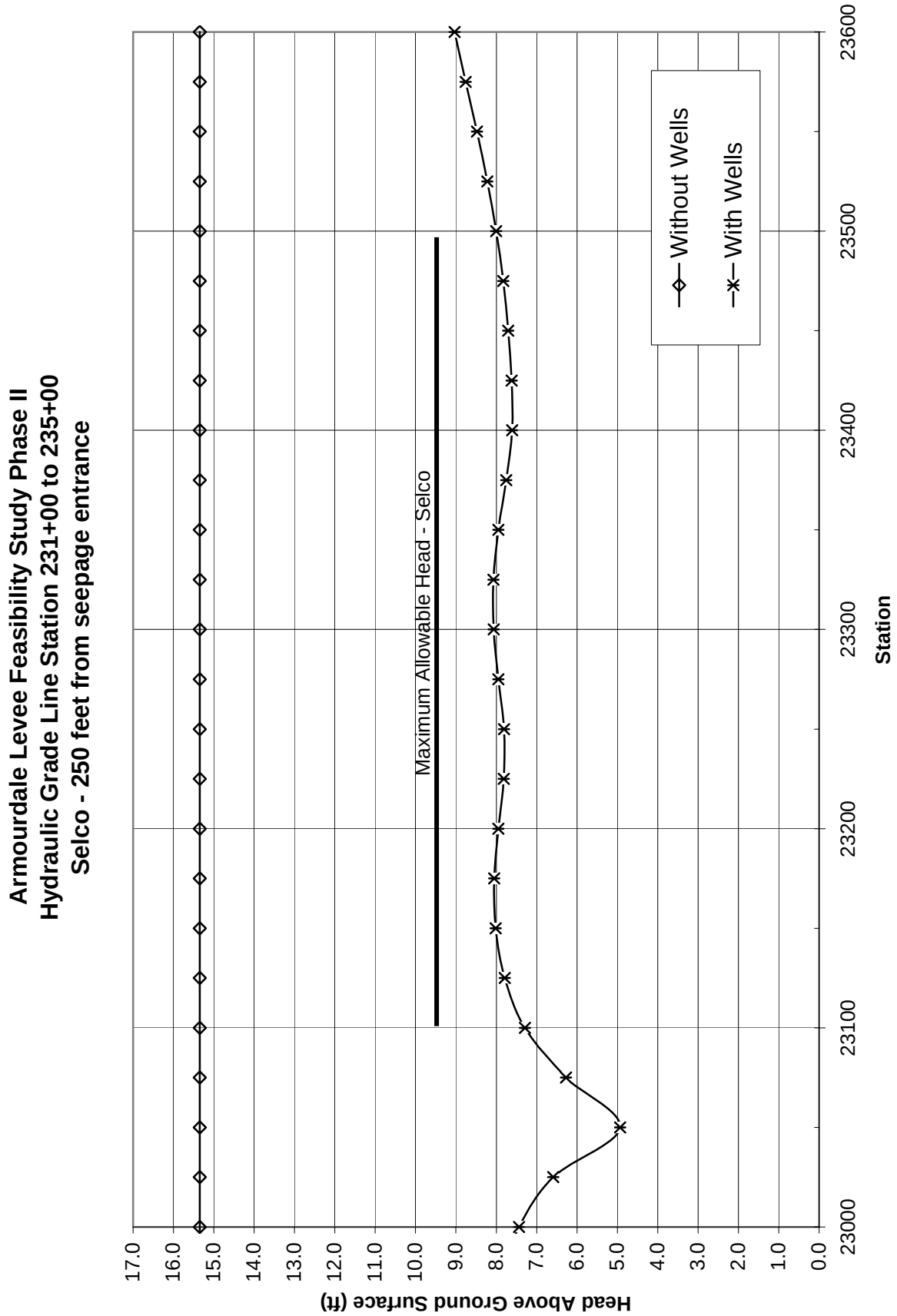

Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 214+00AH
Schock Truck and Leasing - 350 feet from seepage entrance
Maximum Allowable Head - Schock Truck and Leasing
Without Wells
With Wells
Head Above Ground Surface (ft)
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
21300
21400
21500
Station (in AH stationing) - Note Station Equation Change 206+12.43 BK = 212+00 AH


Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 217+00
Jones Store Distributing - 300 feet from seepage entrance
Maximum Allowable Head - Jones Store Distributing
Without Wells
With Wells
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
21600
21700
21800
Station (in AH stationing) - Note Station Equation Change 206+12.43 BK = 212+00 AH


Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 218+00 to 220+00
Glass Co. - Prime Investments - 250 feet from seepage entrance
Head Above Ground Surface (ft)
17.0
16.0
15.0
14.0
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
21700
21800
21900
22000
22100
Station
Maximum Allowable Head - Glass Co. - Prime Investments
Without Wells
With Wells


Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 225+00
Overland Park Bank - 270 feet from seepage entrance
Head Above Ground Surface (ft)
16.0
15.0
14.0
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
22400
22500
22600
Station
Maximum Allowable Head - Overland Park Bank
Without Wells
With Wells


Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 229+00 to 230+50
Sambol Meat Packing - 250 feet from seepage entrance
Head Above Ground Surface (ft)
17.0
16.0
15.0
14.0
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
22800
22900
23000
23100
Station
Maximum Allowable Head - Sambol Meat Packing
Without Wells
With Wells


Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 231+00 to 235+00
Selco - 250 feet from seepage entrance
Head Above Ground Surface (ft)
17.0
16.0
15.0
14.0
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
23000
23100
23200
23300
23400
23500
23600
Station
Maximum Allowable Head - Selco
Without Wells
With Wells

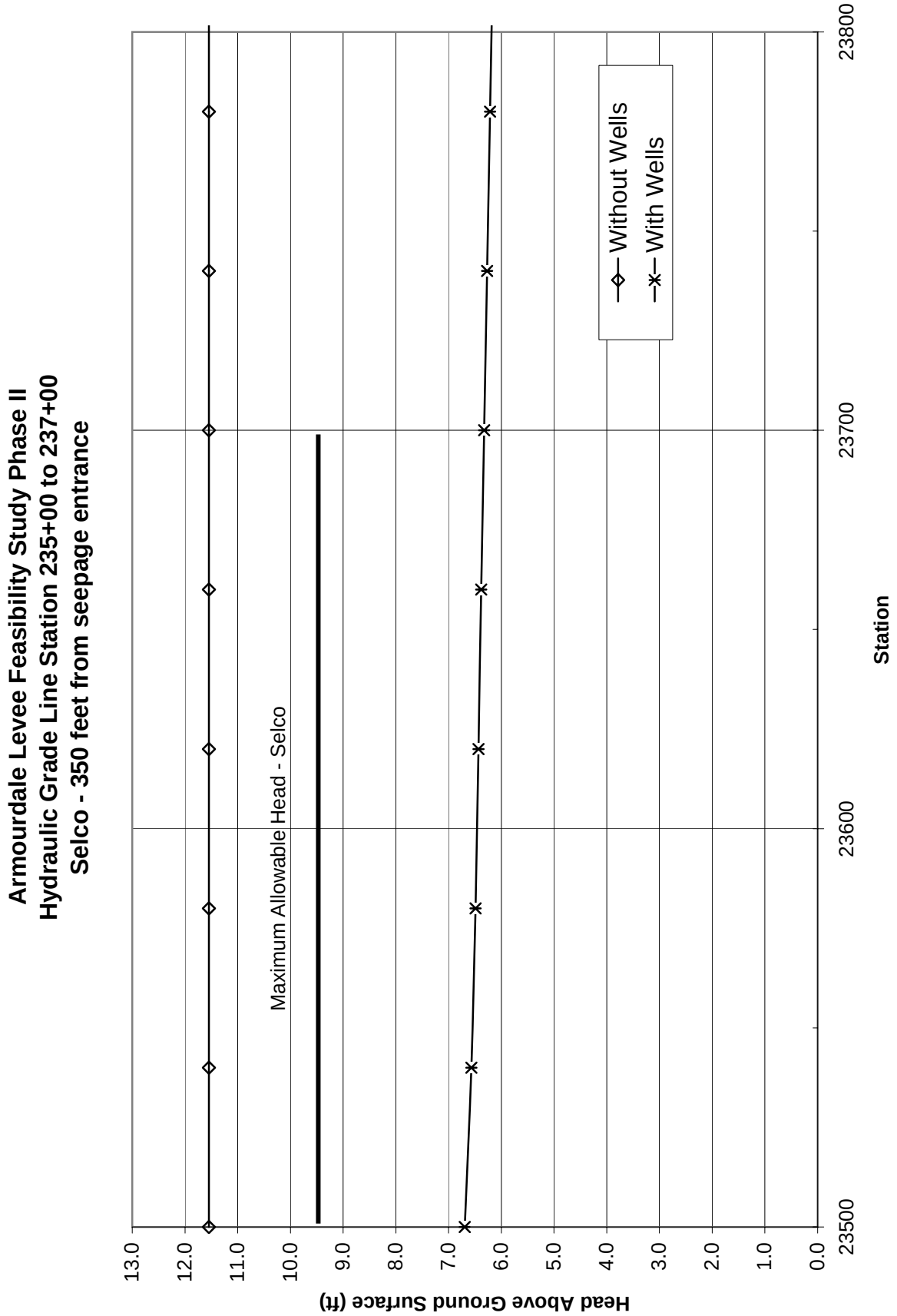

Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 235+00 to 237+00
Selco - 350 feet from seepage entrance
Head Above Ground Surface (ft)
13.0
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
23500
23600
23700
23800
Station
Maximum Allowable Head - Selco
Without Wells
With Wells

EXHIBIT 20: WELL SYSTEM SUMMARY - STATION 296+00 TO 313+00

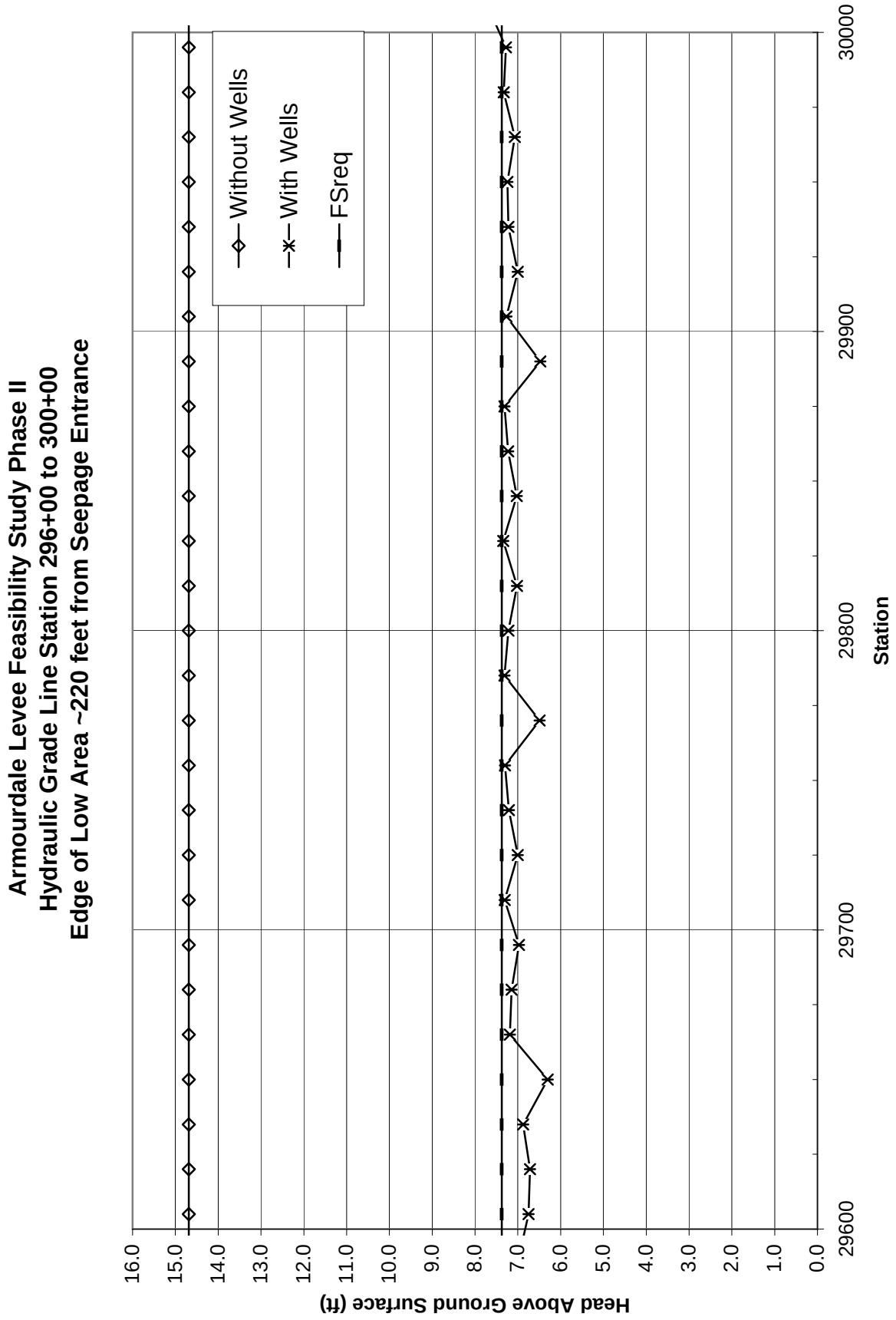
Proposed Relief Well System, Station 296+00 to 313+00

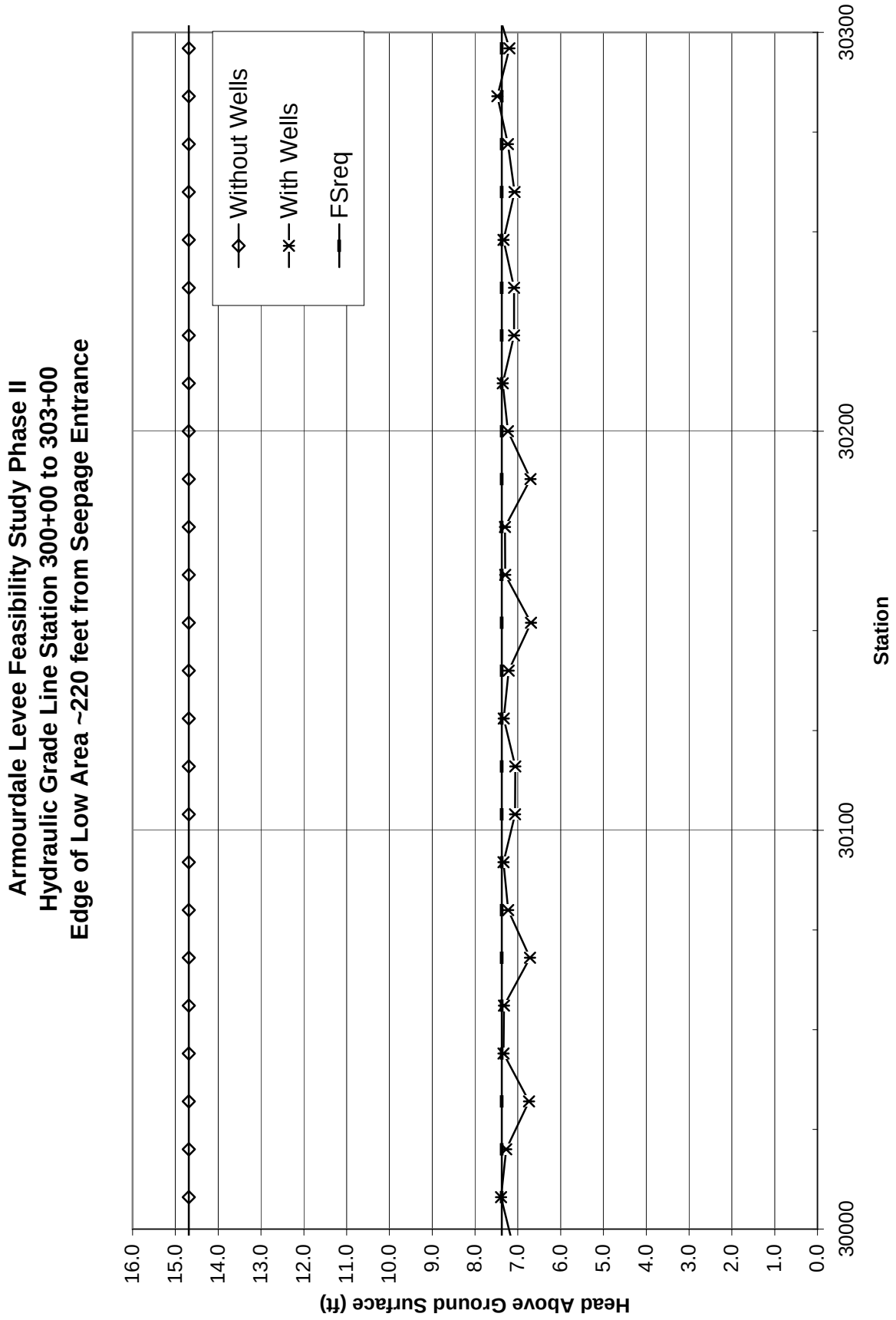
Status	Well	Distance From Seepage Entrance (ft)	Station (AH)	Discharge Elevation (ft)	Well Flow (cfs)
Proposed	1	220	29500	745	0.54
Proposed	2	220	29525	745	0.50
Proposed	3	220	29550	745	0.48
Proposed	4	220	29575	745	0.46
Proposed	5	220	29600	745	0.46
Proposed	6	220	29625	745	0.45
Proposed	7	220	29650	745	0.46
Proposed	8	220	29690	745	0.46
Proposed	9	220	29730	745	0.47
Proposed	10	220	29770	745	0.47
Proposed	11	220	29810	745	0.63
Proposed	12	220	29850	745	0.57
Proposed	13	220	29890	745	0.54
Proposed	14	220	29925	745	0.51
Proposed	15	220	29960	745	0.50
Proposed	16	220	29990	745	0.49
Proposed	17	220	30030	745	0.51
Proposed	18	220	30070	745	0.54
Proposed	19	220	30110	745	0.57
Proposed	20	220	30150	745	0.62
Proposed	21	220	30190	745	0.49
Proposed	22	220	30230	745	0.86
Proposed	23	220	30265	745	0.80
Proposed	24	220	30300	745	0.78
Proposed	25	220	30350	745	0.77
Proposed	26	220	30390	745	0.77
Proposed	27	200	30450	747	0.69
Proposed	28	200	30500	747	0.70
Proposed	29	200	30550	747	0.73
Proposed	30	200	30660	747	0.81
Proposed	31	200	30800	747	0.88
Proposed	32	200	30940	747	0.91
Proposed	33	200	31080	747	0.94
Proposed	34	200	31210	747	0.96
Proposed	35	200	31300	747	1.02

*Relief well flows shown are expected flows. The flows have been reduced by 80% in the calculations to account for future well efficiency reduction.

*Seepage entrance is assumed to be at the elevation of the bottom of the riverside blanket

EXHIBIT 21: COMPUTED EXCESS HEAD AT LOW LYING AREA




Armourdale Levee Feasibility Study Phase II
Hydraulic Grade Line Station 300+00 to 303+00
Edge of Low Area ~220 feet from Seepage Entrance
Without Wells
With Wells
FSreq
Head Above Ground Surface (ft)
16.0
15.0
14.0
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
30000
30100
30200
30300
Station

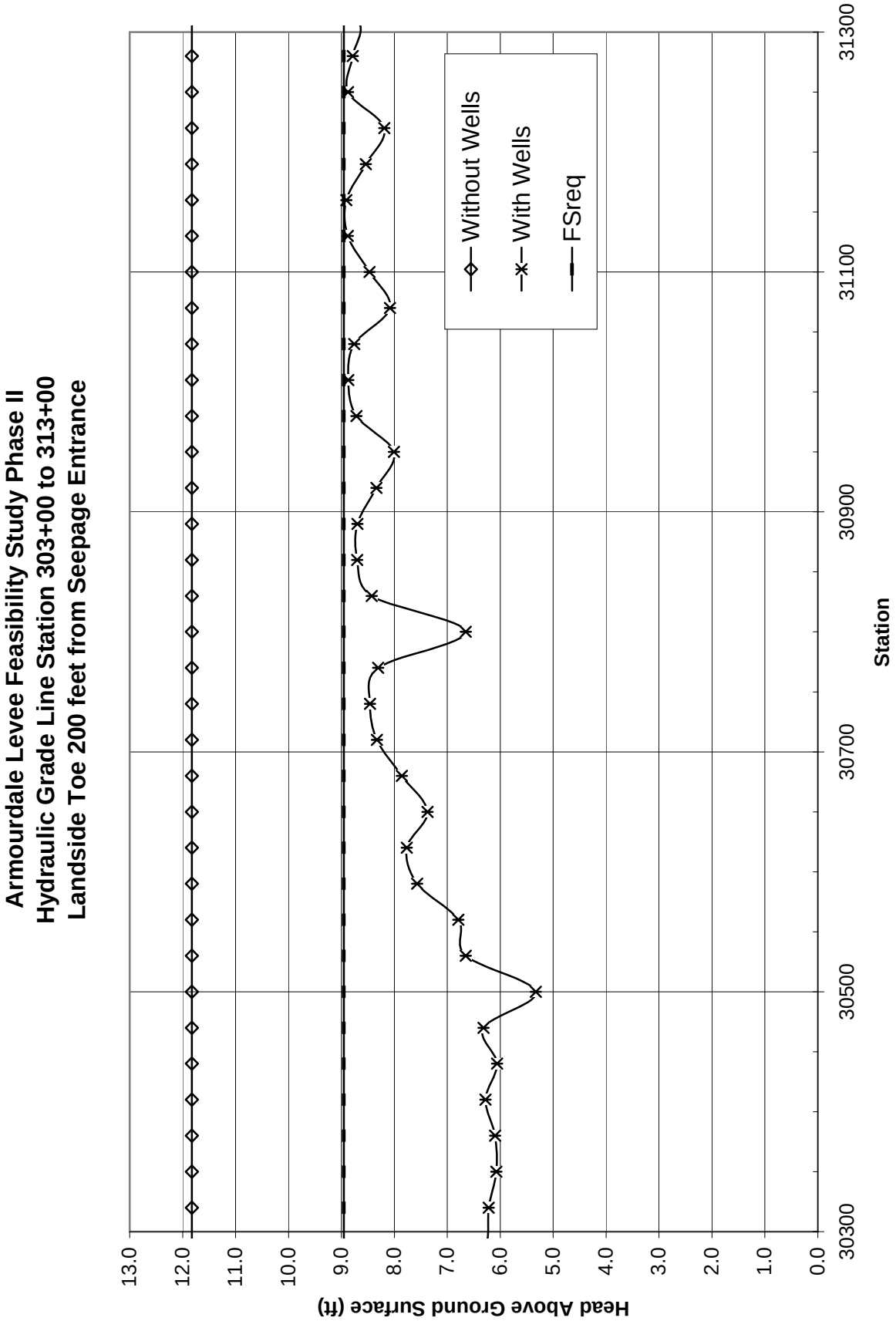



EXHIBIT 22: COMPUTED EXCESS HEAD AT LANDSIDE TOE STATION 303+00 TO 313+00