Appendix I

Geotechnical Analysis of Flood-Risk Management Projects

Appendix A – Chapter 2

Geotechnical Analysis of Flood Risk Management Projects

February 7, 2017 Draft
U.S. Army Corps of Engineers
Kansas City District
Geotechnical Branch
Geotechnical Design and Dam Safety Section

Table of Contents

2. Degradation effects on Flood Damage Risk Reduction Structures

2.1 Introduction

This Appendix presents the results of Missouri River degradation impacts on Flood Damage Risk Reduction Structures (FDRRS), such as levees and floodwalls, in the Kansas City area between Missouri River Miles (RM) 329 and 495. FDRRS within this reach were identified in the 2014 Missouri River Bed Degradation Study as being susceptible to riverbed degradation. The information in this appendix was mostly compiled by the Geotechnical Design and Dam Safety Section (ED-GD).

Prior to performing any analysis, the subsurface conditions and levee and channel geometries were established based on document review. This involved thorough examination of the 2006 Kansas Cities Levee Feasibility Study report, Record Drawings (RD), Design Memorandums (DM), Operation and Maintenance (O&M) manuals, recent inspection reports, Google Earth Maps, and 2013 and 2011 Missouri Riverbed surveys.

2.2 Method of assessing river degradation impacts on FDRRS

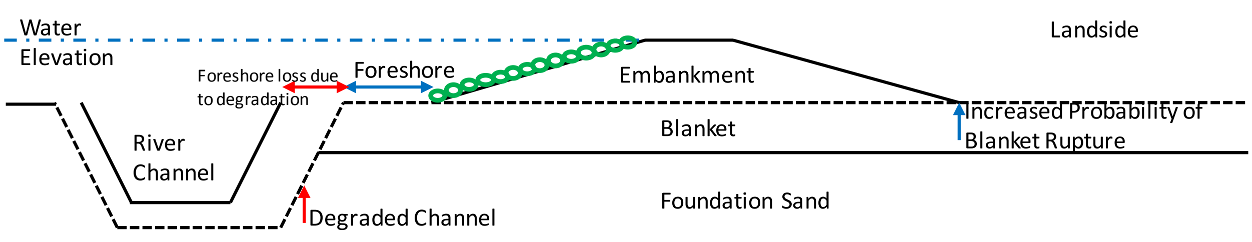
The scope of this portion of the study included quantifying degradation impacts on FDRRS performance during flood events (underseepage, landside slope stability, riverside slope stability) and non-flood events (riverside slope stability). In this study, river degradation impacts on FDRRS were assessed using probabilistic methods.

FDRRS failure during flood events causes direct economic damages and possible loss of life in the leveed area. FDRRS performance during flood events was evaluated probabilistically with respect to underseepage and landside slope stability during the 2006 Kansas Cities Levee feasibility study. Those evaluations were used to the extent possible in this study. Riverside slope stability during flood and non-flood events was fully evaluated for this study.

<u>Underseepage Stability Flood</u>

Underseepage flood information from the 2006 Kansas Cities Levee Feasibility Report was used to quantify changes in expected FDRRS performance during flood events due to decreased channel bed elevations (degradation). Riverbed degradation may cause bank instability and foreshore loss causing shortened seepage paths under the levee as shown in **Figure 2.2-1**. A shortened seepage path will increase pressures landward of the levee and may cause increased probability of having levee failure due to underseepage. The additional risk posed to underseepage by river degradation can be determined by assessing the risk with and without degradation and finding the incremental increase.

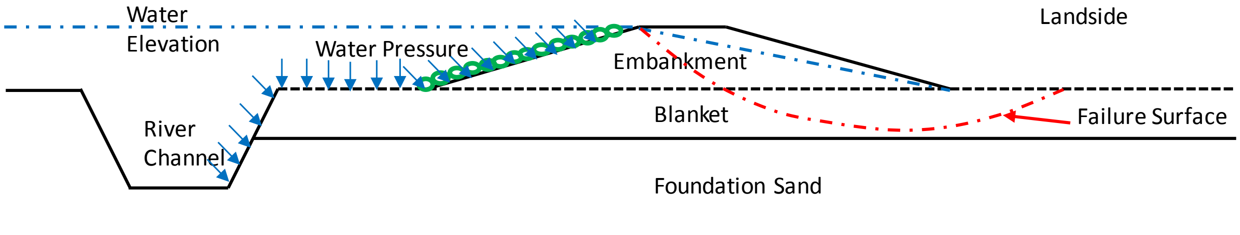
Figure 2.2-1: Schematic of Probabilistic Underseepage Analysis with Water at Top of Levee to Investigate Degradation Impact



Landside Slope Stability, Flood

There are areas where landside slope stability is of concern during flood events identified in the 2006 Kansas Cities Levee Feasibility Study. This failure mode is shown in **Figure 2.2-2**. The foundation seepage pressures at top of levee loading did not heave the landside toe which is critical to slope stability. As a result, landside slope stability during flood events is not considered to be significantly affected by river bed degradation and was not considered in the current study.

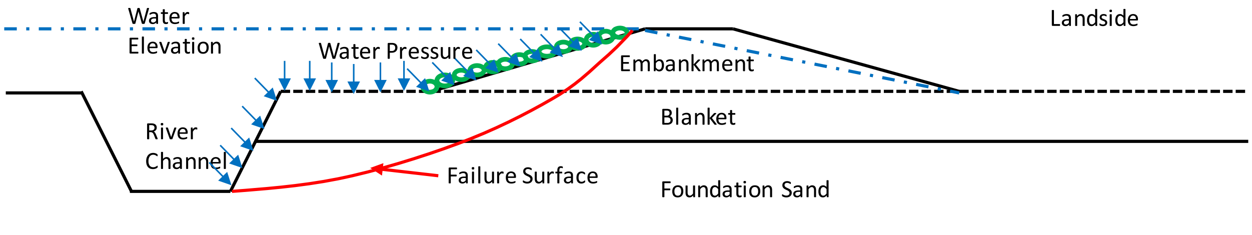
Figure 2.2-2: Schematic of Probabilistic Slope Stability Analysis (landside failure) with Water at Top of Levee



Riverside Slope Stability, Flood

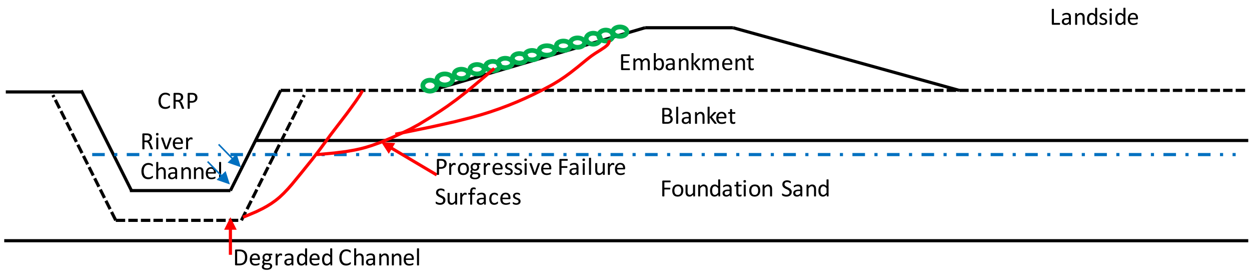
Riverside slope stability was not evaluated during the 2006 Kansas Cities Levee Feasibility Study because it was not considered a significant risk to the protected area. Likewise, channel degradation during flood events is not expected to have a significant effect on channel bank stability that would have an immediately affect on FDRRS while the river elevations are elevated. This is because the elevated river levels provide a resistance and stabilizing force to the river bank and levee slope. However, as the river elevation decreases and the resistant forces are removed, the channel bank becomes more susceptible to failure. A failure after the river has receded does not cause significant economic damages or loss of life to the area protected by the levee. For this reason, riverside slope stability during flood events was not considered during this study. This failure mode is shown in **Figure 2.2-3**.

Figure 2.2-3: Schematic of Probabilistic Slope Stability Analysis (riverside failure) with Water at Top of Levee



FDRRS poor performance during non flood events does not necessarily cause direct economic damages or potential loss of life to the protected area. This is because the Missouri River is generally slow rising and repairs can likely be performed prior to the next flood event. However, riverside bank instability can lead to FDRRS instability that must be repaired prior to the next flood event. Lack of repair could lead to progressive failure of the riverside slope that could impact FDRRS and cause more expensive repairs. This progressive failure mode is shown in **Figure 2.2-4**. During low flows channel degradation is expected to cause more frequent localized bank instability near FDRRS which must be repaired to maintain FDRRS stability. These repairs can cost significant operation and maintenance dollars to maintain FDRRS and bank stability. Decreased river bank slope stability during normal flows was evaluated during this study.

Figure 2.2-4: Schematic of Probabilistic Slope Stability Analysis (progressive failure) with Water at Construction Reference Plane (CRP) to Investigate Degradation Impact



2.3 Probabilistic Analysis Methodology

Probabilistic analysis differs from more traditional deterministic analysis in several ways. Deterministic analysis is performed by selecting a single "design" value for all inputs and parameters to perform an assessment. Typically the ratio of resistant to driving forces must be greater than the required "factor of safety" to ensure stable designs. Probabilistic analysis differs from deterministic analysis by using the "expected" value (statistical mean) and the statistical variability for all inputs and parameters to perform an assessment. Typically a certain "reliability" (or probability of a limit state not being exceeded) is required. Different probabilistic methods were used in this study to assess underseepage and slope stability failure modes. Underseepage analyses were performed using the Taylor Series Method and slope stability analyses were performed using Monte Carlo Simulations. These two methods are discussed in the following subsections.

2.3.1 Underseepage Analysis Methodology

Underseepage probabilistic analysis was performed using Taylor Series Methods discussed in ETL 1110-2-556 *Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies*. The probability of failure can be calculated using the Taylor Series method (ETL 1110-2-556) and the underseepage analysis methods outlined in EM 1110-2-1913 *Design and Construction of Levees*. The average gradient (i) across the blanket was assumed to be log-normally distributed (ETL 1110-2-556) with probabilistic logarithmic moments: expected mean, E[ln i], and standard deviation, $\sigma_{\ln i}$. The limit state for underseepage failure would then be the natural log of the failure gradient (ln [i_f]) with the boundaries for the probability of failure being:

$$P_f = P(\ln i > \ln i_f) \qquad \text{Equation 2-1}$$

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:

$$z = \beta = \frac{\ln i_f - E[\ln i]}{\sigma_{\ln i}} = \frac{\ln\left[\dfrac{i_f * \sqrt{1 + COV[i]^2}}{E[i]}\right]}{\sqrt{\ln(1 + COV[i]^2)}} \qquad \text{Equation 2-2 (ETL 1110-2-556)}$$

Where, *E*[i] is the expected value (mean) of the hydraulic gradient and *COV [i]* is the Coefficient of Variation of the hydraulic gradient.

The underseepage failure limit state, or the actual conditions indicative of an underseepage failure, are highly speculative. The underseepage analysis included in ETL 1110-2-556 uses a threshold value of gradient factor of safety of 1.0 to define failure. However, a gradient factor of safety of 1.0 reflects a condition when blanket blow out and boils can first physically occur and is considered the initiation of an underseepage failure; it is not necessarily a condition indicative of having certain levee failure. Because of this, a different threshold value was selected to represent a failure condition as discussed below.

Observations during the Flood of 1952 on the Missouri River are shown in **Table 2.3.1-1**. The table shows the relation between observed field performance and calculated Factors of Safety (FS). As seen in **Table 2.3.1-1**, undesirable seepage reaches a point where failure could occur without outside intervention (flood fighting) somewhere between FS of 0.55 and 0.80,

In an effort to define a condition more representative of actual levee underseepage failure for this study, a gradient FS of 0.70 at the levee toe was utilized as a threshold value for when levee failure is likely to occur without heroic flood flight efforts. The chosen threshold value of gradient FS of 0.70 falls within the “transition” zone in **Table 2.3.1-1** between tolerable seepage and objectionable seepage. In the probabilistic underseepage analyses a failure gradient (i_f) was calculated as:

$$i_f = \frac{i_c}{FS} = \frac{0.84}{0.70} = 1.23 \qquad \text{Equation 2-3}$$

where i_c is the critical gradient and *FS* is the gradient safety factor.

TABLE 2.3.1-1: Observations of Seepage Conditions during 1952 Flooding on the Missouri River

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

Five random variables were used in the probabilistic analysis: blanket thickness, permeability ratio, aquifer thickness, and critical gradient due to variability in the blanket unit weight. The COV for the five parameters are as shown in **Table 2.3.1-2**.

For each river level analyzed, the underseepage analyses were first performed for the expected values of the random variables. Then analyses are performed by varying one parameter at a time by plus and minus one standard deviation. The results of the analysis determine the probability of failure and also show the variation component for each variable. By performing the analysis at varying river levels, a relation between probability of failure and river elevation can be determined.

Table 2.3.1-2: Underseepage Parameters Coefficients of Variation (COV)

Parameter	COV[4]
Blanket thickness[1]	25
Permeability ratio[2]	40
Aquifer thickness[1]	15
Unit weight[3]	10
Critical gradient[3]	10
River entrance length[2]	50
[1]COV based on Engineering Judgment. [2]COV based on ETL 1110-2-556. [3]COV based on 5% COV for unit weight in ETL 1110-2-556. [4]COV is defined as the ratio of standard deviation and expected value.	

Underseepage analysis methodology used by the Kansas City District is in general accordance with EM 1110-2-1913, Appendix B. However, the Kansas City District uses the following minor variations for underseepage analyses: **1)** permeability ratios are used instead of blanket and aquifer permeability (**see Table 2.3.1-3**), **2)** an infinite landside blanket is assumed (Case 7 in EM 1110-2-1913), and **3)** no blanket transformations are performed and a representative permeability ratio and blanket thickness is used. The Kansas City practice is based on the findings made at a 1962 Missouri River Division Conference held in Omaha, Nebraska. The conference findings are based on experience during the 1952 flood event on the Missouri River.

Table 2.3.1-3: Permeability Ratios for Blanket Material Based on Material Type Used in Underseepage Analysis

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

The analysis was performed for both current river conditions and for conditions assuming 15 feet of river degradation, which exceeds the predicted maximum of 12 feet 50 years from 2013. However, based on observations made by comparing the 2013 riverbed profiles to older surveys, degradation causes relatively minimal reductions of the riverside foreshore.

2.3.2 Slope Stability Analysis Methodology

Probabilistic slope stability analyses were performed using the SLOPE/W package in the Geo-Studio 2007 software. Spencer's method was used because it satisfies both force and moment equilibrium. SLOPE/W has the ability to perform Monte Carlo Simulations for probabilistic analyses. The Monte Carlo technique generates repeated random samples of a given parameter from a given Probability Distribution Function (pdf) and the parameter statistical mean and standard deviation.

The design parameters were sampled using Monte Carlo Simulations based on the COVs shown in **Table 2.3.2-1**. The mean values shown in **Table 2.3.2-1** were selected from the indicated sources while the COVs were based on guidance presented in ETL 1110-2-556. Effective stress soil strength parameters were used in the existing conditions analysis since drained conditions are assumed (no short term or quick load changes exist). Cohesion (c) was set to zero in the probabilistic analysis and kept constant since it was considered negligible. The failure state of a slope is generally considered to be a factor of safety of unity, and corresponds to a probability of failure of 50%. However, for this study the goal is to assess increased actions required to maintain stability.

Table 2.3.2-1: Summary of Geotechnical Parameters used in Slope Stability Probabilistic Analysis

			ETL 1110-2-556 Recommended Range for Coefficient of Variation		Coefficient of Variation used in Probabilistic Analysis	
Material	**Mean Unit Weight (lb/ft^3)[1]**	**Mean Effective Angle of Internal Friction (Degrees)[1]**	**Unit Weight**	**Angle of Internal Friction**	**Unit Weight**	**Angle of Internal Friction**
Levee Fill (Sands)	115	30	3% - 8%	3% - 12%	5%	10%
Levee Fill (Clays/Silts)	115	28	3% - 8%	7% - 10%	5%	9%
Dredged Fill	100	30	3% - 8%	N/A	5%	10%
Sediment Material	90	22	3% - 8%	N/A	5%	10%
Rip Rap	135	38	3% - 8%	N/A	5%	3%
Rock Fill	125	35	3% - 8%	N/A	5%	5%
Foundation Sands	120	34	3% - 8%	3% - 12%	5%	10%

[1]The information provided above was compiled from the following reports:
Kansas City Levees, Kansas and Missouri, Interim Feasibility Report and Environmental Impact Statement, 2006
Jersey Creek Wharf Monitoring Project, Tetra Tech., 2006
Jersey Creek Sheet Pile Wall Reconstruction, CDM, 2011
East Bottoms Analysis of Design, USACE, 1945
North Kansas City Analysis of Design, USACE, 1945
Fairfax-Jersey Creek Analysis of Design, USACE, 1945; Analysis Design for Construction of Jersey Creek Sewer, 1951

2.4 Initial Critical levee Cross Sections

The initial critical levee sections for assessing river degradation effects on FDRRS were chosen after examining historic data, locations of structures crossing the river, areas of localized scour based on Google Earth Maps (2013), prior repairs and maintenance efforts, 2009 channel bank revetment survey, 2013 and 2011 riverbed degradation surveys, the 2006 Kansas Cities Levee Feasibility Study, and recent levee and river inspection results.

The selected initial critical cross sections of Federal levee units are the following: North Kansas City-Lower, Fairfax-Jersey Creek, East Bottoms, Birmingham, CID Missouri, CID Kansas, and L-385. A summary of all the initial critical cross sections are tabulated in **Table 2.4-1**. Pertinent levee record drawings and 2013 riverbed degradation surveys for each initial section, and the 2009 stone revetment and 2011 riverbed surveys for the North Kansas City (Station 45+00) Fairfax-Jersey Creek (Station 21+27) levee units are found in **Enclosure A.1**.

Eleven critical non-Federal levee sections, with limited foreshore, are located within the study reach between River Mile (RM) 334.0 and 350.0 along the Missouri River. These levee units belongs to the drainage districts shown in **Table 2.4-2**. No construction records are available on the non-Federal levees; however, Google Earth maps show average width of 10 ft and height between 5 and 15 ft. Eight critical tributaries confluence on the Missouri River were also identified as potential locations for initiating degradation that matches upstream on the tributaries, **Table 2.4-3**.

Table 2.4-1: Initial Critical Levee Locations used for Evaluating Underseepage and Slope Stability under Existing Conditions

Approximate River Mile	River Bank	Federal Levee	Approximate Federal Levee Station	Failure Mode	River Bank Remarks
[a]RM 370.0	Left	North Kansas City-Lower	Station 45+00	Riverbank Stability	Rock revetment
[a]RM 367.78	Right	Fairfax-Jersey Creek (Wharf)	Station 21+27	Riverbank Stability	Sheet pile revetment
[a]RM 368.6	Right	Fairfax-Jersey Creek	Station 71+00	Riverbank Stability	Rock revetment
[a]RM 364.52	Right	East Bottoms	Station 60+00	Riverbank Stability	Rock revetment
[b]RM 358.9	Left	Birmingham	Station 89+0	Riverbank Stability	Rock revetment
[b]RM 366.5	Right	CID-Missouri	Station 42+00	Riverbank Stability	Rock revetment
[b]RM 367.3	Right	CID-Kansas	Station 1+00	Riverbank Stability	No revetment
[b]RM 374.1	Left	L-385	Station 103+00	Riverbank Stability	Rock revetment
[c]Kansas River	Right	Argentine	Station 37+60	Underseepage/ Riverbank Stability	Rock revetment
[c]Kansas River	Left	Armourdale	Station 89+14	Underseepage/ Riverbank Stability	Rock revetment
[c]RM 355.8	Left	Birmingham	Station 200+00	Underseepage/ Riverbank Stability	Rock revetment
[c]RM 367.1	Right	CID-Missouri	Station 78+00	Underseepage/ Riverbank Stability	Rock revetment
[c]Kansas River	Right	CID-Kansas	Station 70+75	Underseepage/ Riverbank Stability	Rock revetment
[c]Blue River	Right	East Bottoms	Station 389+54	Underseepage/ Riverbank Stability	Rock revetment
[c]RM 373.8	Right	Fairfax-Jersey Creek	Station 310+00	Underseepage/ Riverbank Stability	Rock revetment
[c]RM 365.8	Left	North Kansas City-Lower	Station 226+80	Underseepage/ Riverbank Stability	Rock revetment

[a]Locations with little to no foreshore.
[b]Areas highly susceptible to degradation based on 2013 riverbed survey, see Enclosure A.1.
[c]Critical areas identified in the 2006 Kansas Cities Levee Feasibility Study.

Table 2.4-2: Critical non-Federal Levees within Study Reach

Approximate River Mile	River Bank	Non-Federal Levee	Non-Federal Levee Height (ft)	Foreshore Distance (ft)
RM 434.6 - 435.1	Left	Geary Bend	8	60
RM 426.2 - 426.4	Left	Rushville Sugar Creek	8	85
RM 416.8 - 417.3	Left	Bean Lake	7	20
RM 411.5 - 411.7	Right	Henry Pohl	10	45
RM 410.0 - 410.1	Right	Henry Pohl	5	20
RM 407.6 - 408.9	Left	Struve	7	20
RM 399.2 - 400.2	Right	Sherman Army Airfield	15	60
RM 351.6 - 354.2	Left	City of Independence	4	<10
RM 349.3 - 351.6	Left	Bruening	6	30
RM 348.5 - 349.3	Left	Allen	4	200
RM 345.6 - 349.3	Left	Bell	4	90

Note:
Foreshore and levee height was estimated using Google Earth Maps.

Table 2.4-3: Critical Confluence of Tributaries along the Missouri River.

Approx. River Mile	River Bank	Confluence Location	Tieback Location	Channel Width (ft)
RM 437.3	Left	MRSL 455L	Federal	120
RM 434.2	Right	Jones	Non-Federal	60
RM 433.5	Right	Roundy	Non-Federal	220
RM 418.2	Left	Rushville Sugar Creek	Non-Federal	140
RM 410.5	Left	Iatan Power Plant	Creek Bank	75
RM 406.2	Right	Grape/Bollin/Schwartz	Creek Bank	200
RM 403.3	Left	Kirk	Creek Bank	150
RM 388.1	Right	Kansas City Department of Corrections	Non-Federal	200

2.5 Initial Critical Cross Section Analysis Results

The initial critical sections were evaluated and screened for the failure modes identified in **Table 2.4-1**. The results are discussed in the following subsections.

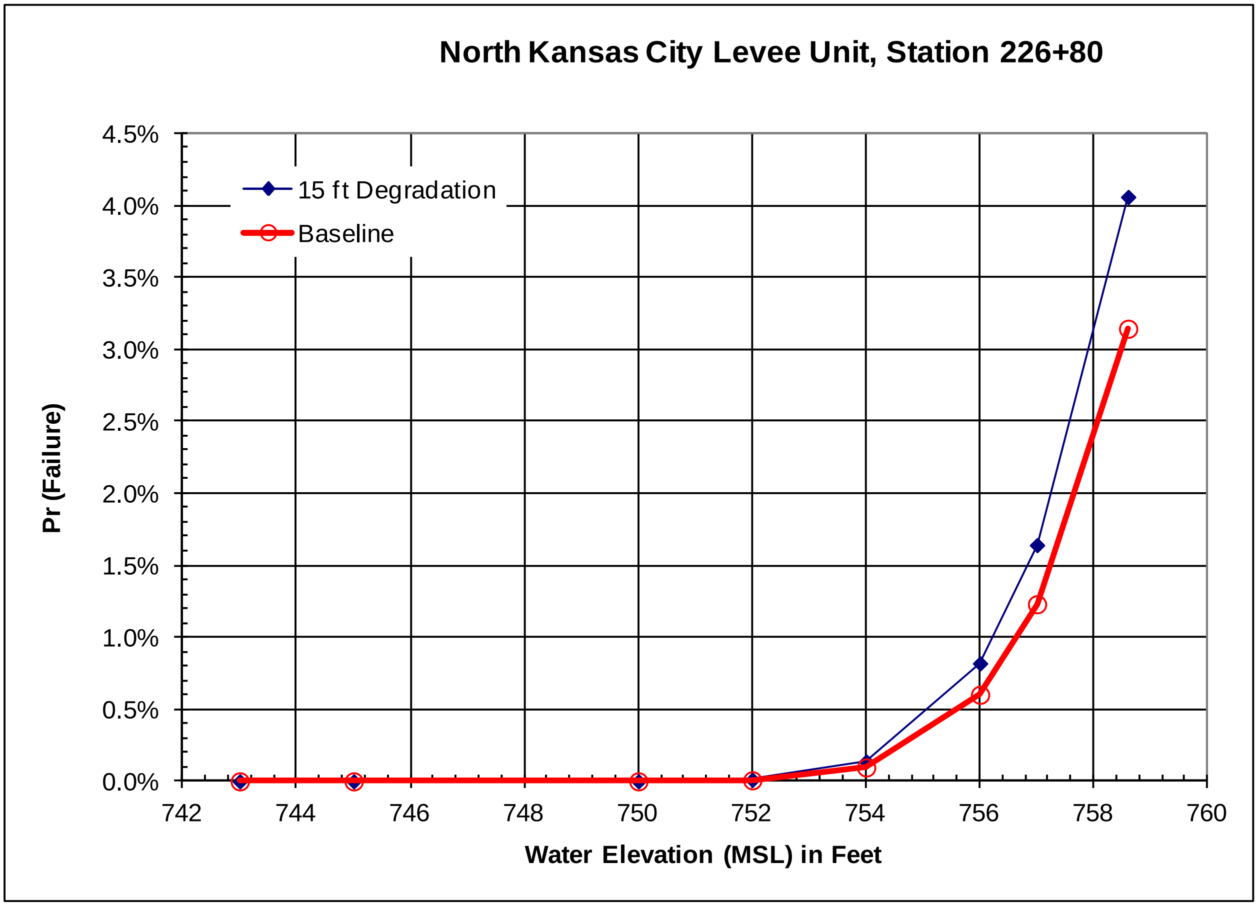
2.5.1 Initial Critical Cross Section Underseepage

Probabilistic underseepage analysis was performed for the North Kansas City levee at Station 226+80 to gain insights into degradation impact on that and other initial critical sections identified for underseepage analysis. The underseepage characteristics of the North Kansas City levee are considered representative of the other levee units in the Kansas City area. The information for this reach contained in the 2006 Kansas Cities Levee Feasibility Study was updated with additional subsurface information obtained from the 2011 Design Documentation Report (DDR) – Underseepage Control Harlem Area, North Kansas City Unit (Lower). The baseline condition analysis used the existing foreshore length of 300 feet between the river and the levee. The 2011 DDR documents relief well design and construction; however, the wells are ignored in this analysis so the results will be more representative of the other initial areas.

The baseline Probability of Failure (PoF) for this area is 3.2% with water at the top of the levee. In this analysis, a worst case scenario of river degradation in this area was assumed to be approximately 15 feet. It is assumed that riverbed degradation leads to foreshore reduction proportional to the existing channel bank side slopes of 3 H: 1V; as a result, 15 feet of degradation leads to approximately 45 feet of foreshore loss. All other inputs in the underseepage analysis remain the same as the base condition to isolate the increased risk due to river degradation. The probability of failure for this area after 15 feet of degradation is 4.1% with water at the top of the levee.

The PoF vs. river elevation curves for the baseline and degradation conditions are shown in **Figure 2.5.1-1**. As seen, the increase in underseepage risk to the levee caused by river degradation is insignificant. Furthermore, the underseepage assessment assumes a worst case scenario that all 15 ft of degradation occurs at once or degradation progressives without repairs. Based on the 2011 flood event, the North Kansas City levee (station 45+00) was repaired immediate after the flood event under PL 84-99. The Fairfax-Jersey Creek levee (station 21+27) was armored with riprap during the 2011 flood event with additional riprap placed in 2014 during stabilization of the Jersey Creek Sheetpile Wall. As a result, all the identified initial critical areas of possible underseepage concern were screened out and not considered in the future without project conditions that assess degradation impacts on underseepage performance. Calculations are included in Enclosure A.2. It was assumed that the non-Federal levees would perform adequately compared to the Federal levees because they are relatively short and have lower hydrologic loads compared to the Federal levees. As a result, no underseepage fragility curve was developed for the non-Federal levees.

Figure 2.5.1-1: Underseepage Fragility Curve for North Kansas City Levee at Station 226+80 to Quantify Degradation Impact



2.5.2 Initial Cross Section Riverbank Slope Stability near FDRRS

The stability performance of the existing riverbanks near the FDRRS initial cross sections identified in **Table 2.4-1** was evaluated probabilistically with water at the 2010 CRP. For a given levee, the CRP was determined based on the 2010 Bank Stabilization and Navigation Project (BSNP). The river channel geometries for all the levee units were determined from the 2013 Missouri River channel survey. The only exception is the geometries for the North Kansas City (Station 45+00) and Fairfax-Jersey Creek (Station 21+27) levee units incorporated the 2009 channel stone revetment and accretion based on the 2011 Missouri River survey.

The levee and foreshore geometries were obtained from the record drawings of the representative levee in conjunction with Google Earth Maps. The record drawings used for the North Kansas City, Fairfax-Jersey Creek, East Bottoms, Birmingham, CID Missouri, CID Kansas, L-385, and R-351 levee units are from 1945, 1939, 1945, 1954, 1962, 1957, 2007, and 1963, respectively. Details of the record drawings and the combined schematics of the levee and channel geometries are found in **Enclosure A.1**.

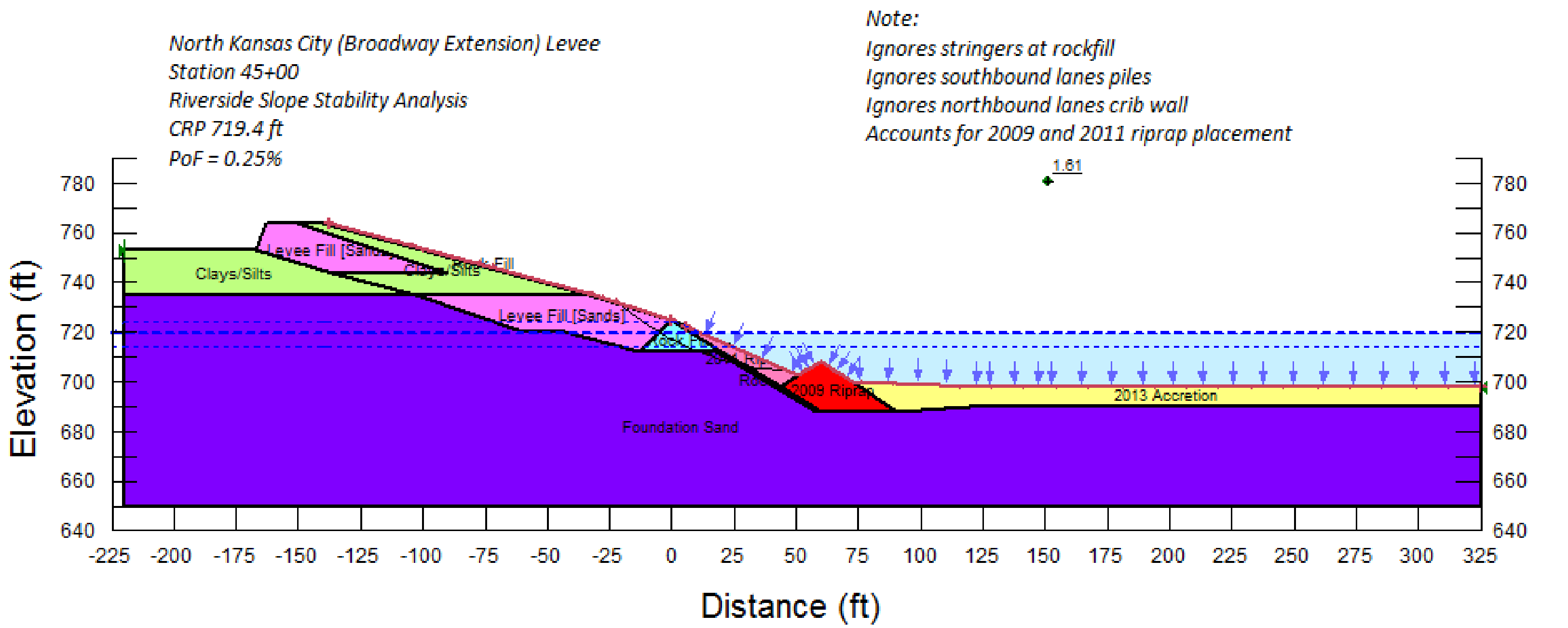
The baseline results for all the Federal levee reaches are tabulated in **Table 2.5.2-1**. Baseline analysis for the non-Federal levees was not performed due to lack of as-built drawings and subsurface information. The non-Federal levees were assumed stable when the foreshore is greater than 50 ft. When the foreshore is less than 50 ft, the non-Federal levees were assumed unstable due to sloughing caused by degradation of the Missouri River channel bank. The slope stability outputs for the Federal levees are shown in **Figures 2.5.1-2** through **2.5.2-9.** Slope stability of the river banks was found to be most critical when the river is low during non-flood events.

Table 2.5.2-1: Riverside Steady State Slope Stability Reliability Results for Critical Federal Levee Reaches Identified in Table 2-1 with Water at Construction Reference Plain (non-Flood Events) – 2013 Riverbed Baseline

Approximate River Mile	Approximate Federal Levee Station	Federal Levee	[c]Construction Reference Plain elevation (CRP)	Average Riverbed Elevation NVD88 (ft)	2013 Baseline Probability of Failure (PoF) – (%)
[a,d]RM 370.00	Station 45+00	North Kansas City	719.4	694.0	0.25
[d]RM 367.78	Station 21+27	Fairfax-Jersey Creek (Wharf)	718.3	688.0	10.10
RM 368.60	Station 71+00	Fairfax-Jersey Creek	718.3	698.0	45.80
[a]RM 364.52	Station 60+00	East Bottoms	715.3	695.0	2.80
[b]RM 358.90	Station 90+00	Birmingham	711.6	692.0	24.30
[a,b]RM 366.50	Station 42+00	CID-Missouri	716.7	685.0	3.80
[a,b]RM 367.30	Station 1+00	CID-Kansas	717.3	680.0	2.50
[a,b]RM 374.10	Station 103+00	L-385	722.6	707.0	9.80

[a]Short foreshore.
[b]Areas of significant degradation based on 2013 riverbed survey, see Enclosure A.1.
[c]Based on 2010 CRP calculation.
[d]Channel profile based on 2011 survey after placement of riprap on riverbed and channel slope, see Enclosure A.1.

Figure 2.5.2-1: Riverside Slope Stability Results for North Kansas City Levee Station 45+00 under Existing Conditions with Water Elevation at CRP



Name: 2013 Accretion Model: Mohr-Coulomb Unit Weight: Multiple Trial: 90 pcf Cohesion: 0 psf Phi: Multiple Trial: 22 ° Piezometric Line: 1
Name: Clays/Silts Model: Mohr-Coulomb Unit Weight: Multiple Trial: 115 pcf Cohesion: 0 psf Phi: Multiple Trial: 28 ° Piezometric Line: 1
Name: Levee Fill [Sands] Model: Mohr-Coulomb Unit Weight: Multiple Trial: 115 pcf Cohesion: 0 psf Phi: Multiple Trial: 30 ° Piezometric Line: 1
Name: Foundation Sand Model: Mohr-Coulomb Unit Weight: Multiple Trial: 120 pcf Cohesion: 0 psf Phi: Multiple Trial: 34 ° Piezometric Line: 1
Name: Rock Fill Model: Mohr-Coulomb Unit Weight: Multiple Trial: 125 pcf Cohesion: 0 psf Phi: Multiple Trial: 35 ° Piezometric Line: 1
Name: 2009 Riprap Model: Mohr-Coulomb Unit Weight: 135 pcf Cohesion: 0 psf Phi: 38 ° Piezometric Line: 1
Name: 2011 Riprap Model: Mohr-Coulomb Unit Weight: Multiple Trial: 135 pcf Cohesion: 0 psf Phi: Multiple Trial: 38 ° Piezometric Line: 1

Figure 2.5.2-2: Riverside Slope Stability Results for Fairfax-Jersey Creek Wharf Levee Station 21+27 under Existing Conditions with Water Elevation at CRP

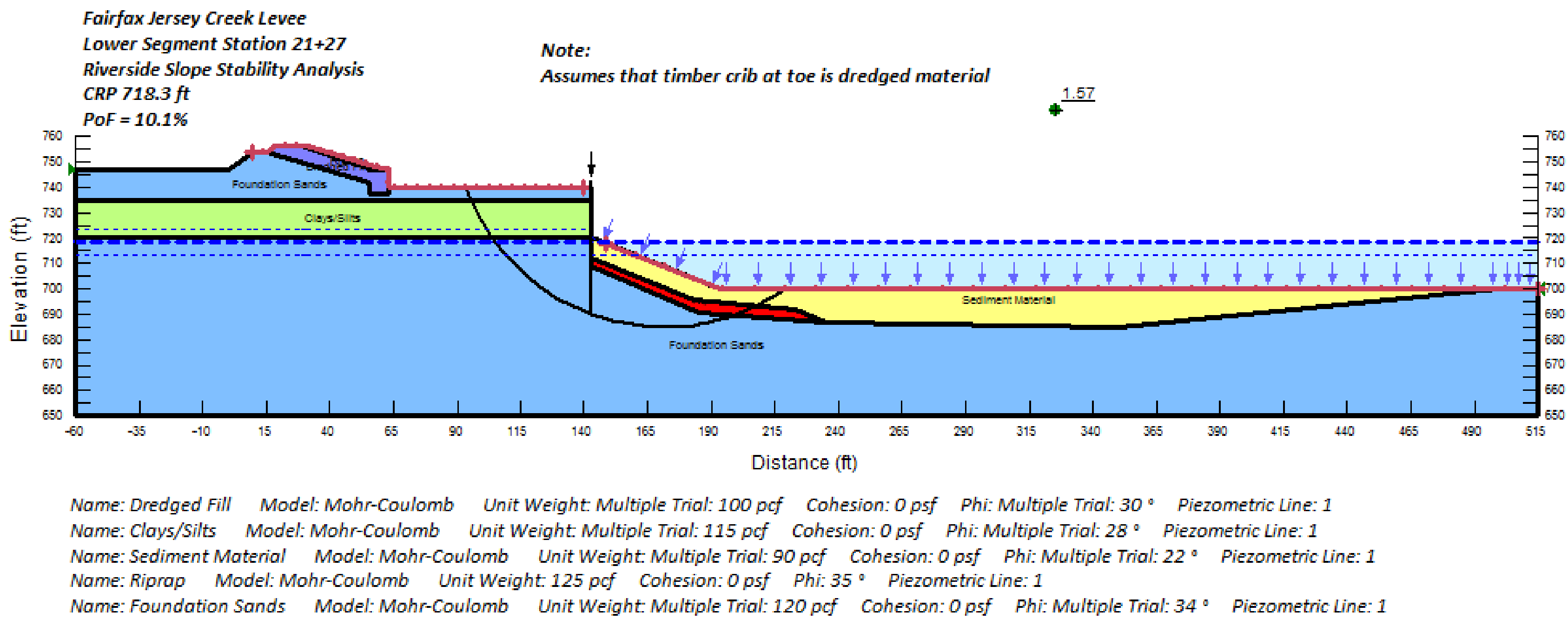
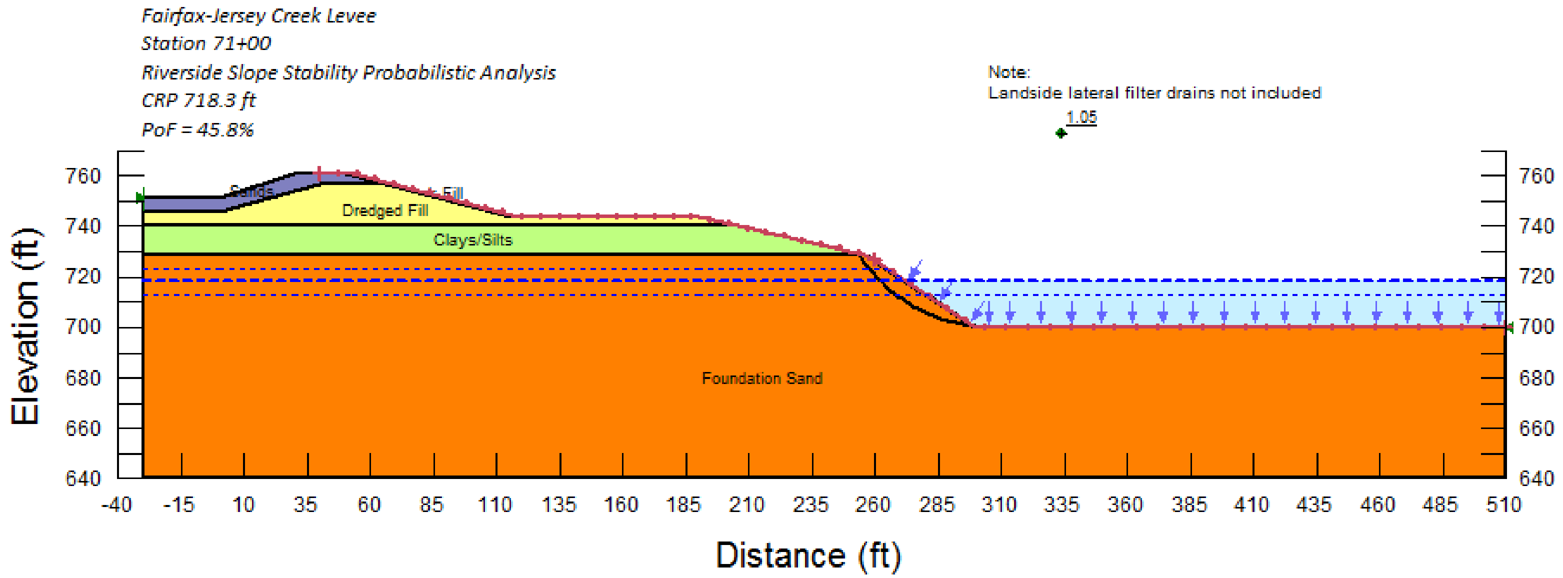


Figure 2.5.2-3: Riverside Slope Stability Results for Fairfax-Jersey Creek Levee Station 71+00 under Existing Conditions with Water Elevation at CRP



Name: Dredged Fill   Model: Mohr-Coulomb   Unit Weight: Multiple Trial: 100 pcf   Cohesion: 0 psf   Phi: Multiple Trial: 26 °   Piezometric Line: 1
Name: Clays/Silts   Model: Mohr-Coulomb   Unit Weight: Multiple Trial: 115 pcf   Cohesion: 0 psf   Phi: Multiple Trial: 28 °   Piezometric Line: 1
Name: Sands   Model: Mohr-Coulomb   Unit Weight: 115 pcf   Cohesion: 0 psf   Phi: 30 °   Piezometric Line: 1
Name: Rock Fill   Model: Mohr-Coulomb   Unit Weight: Multiple Trial: 125 pcf   Cohesion: 0 psf   Phi: Multiple Trial: 35 °   Piezometric Line: 1
Name: Foundation Sand   Model: Mohr-Coulomb   Unit Weight: Multiple Trial: 120 pcf   Cohesion: 0 psf   Phi: Multiple Trial: 34 °   Piezometric Line: 1

Figure 2.5.2-5: Riverside Slope Stability Results for East Bottoms Levee Station 60+00 under Existing Conditions with Water Elevation at CRP

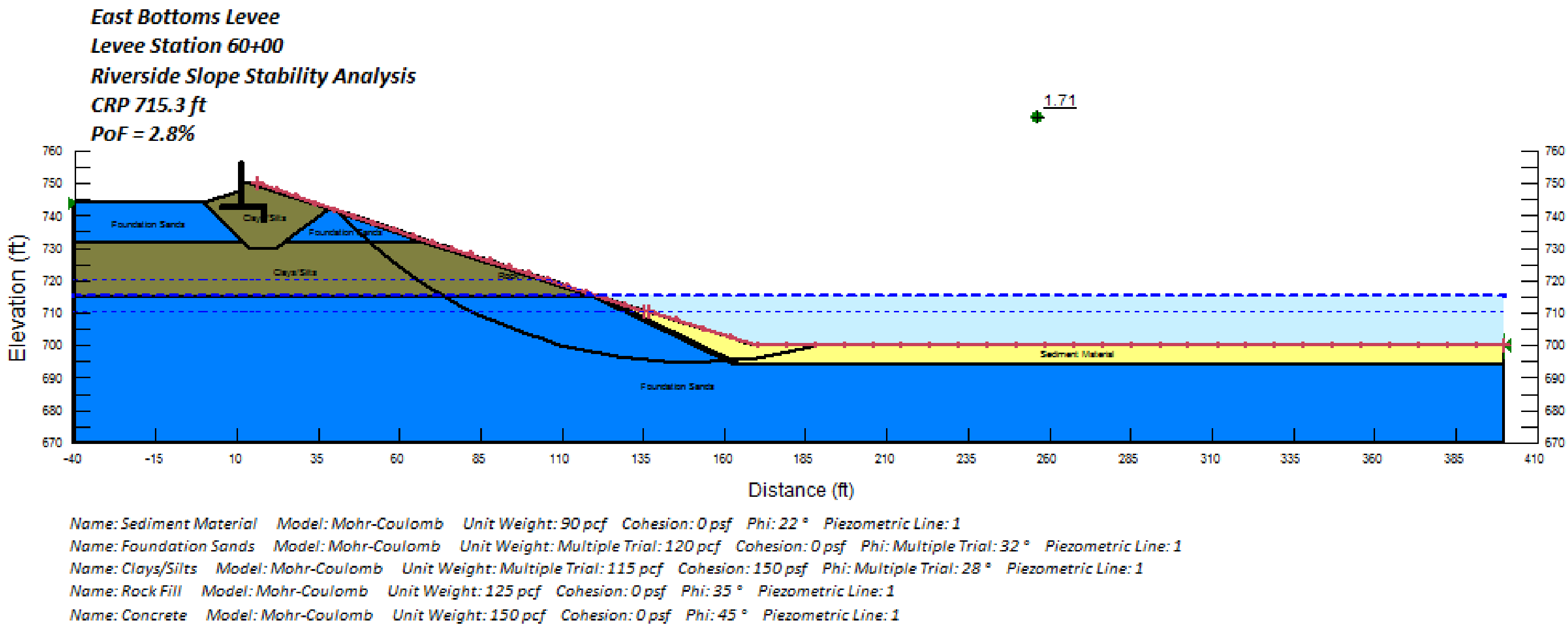


Figure 2.5.2-6: Riverside Slope Stability Results for Birmingham Levee Station 90+00 under Existing Conditions with Water Elevation at CRP

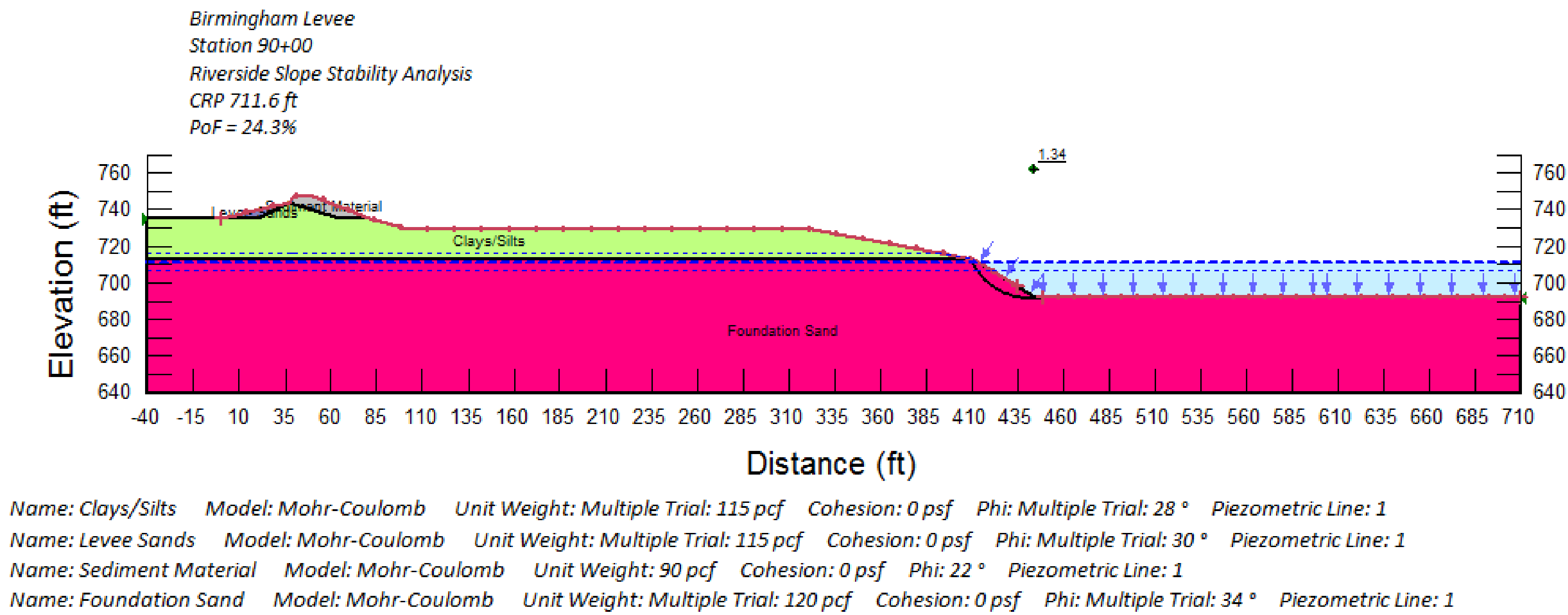


Figure 2.5.2-7: Riverside Slope Stability Results for CID Missouri Levee Station 42+00 under Existing Conditions with Water Elevation near CRP

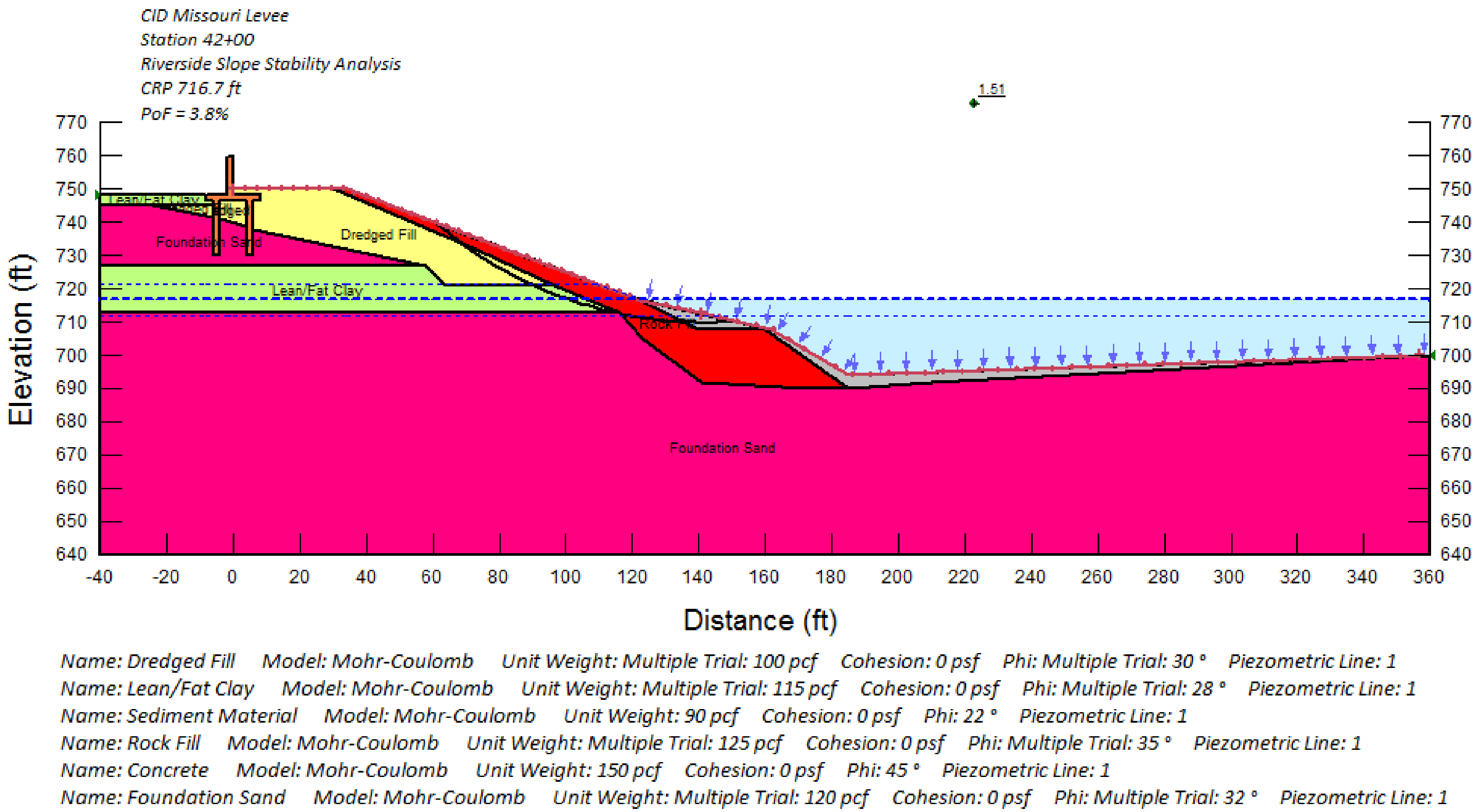


Figure 2.5.2-8: Riverside Slope Stability Results for CID Kansas Levee Station 1+00 under Existing Conditions with Water Elevation near CRP

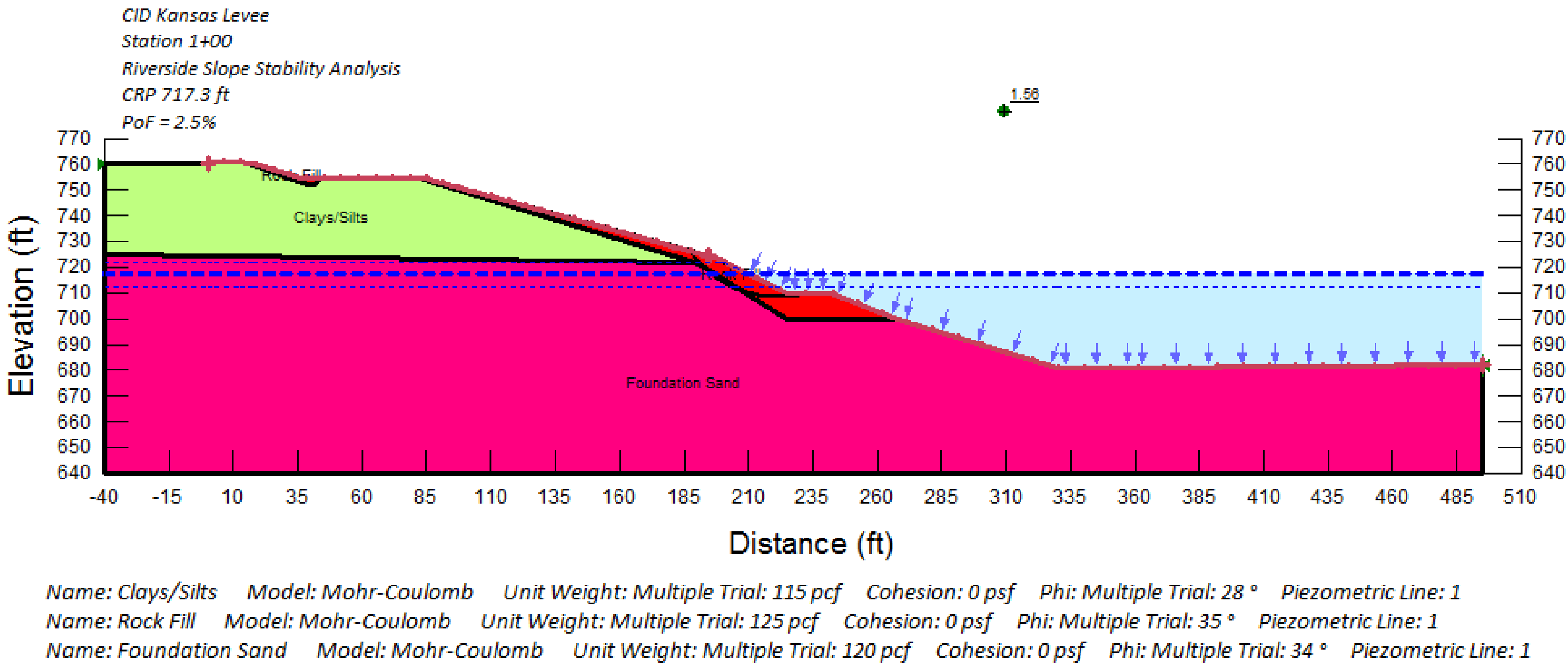
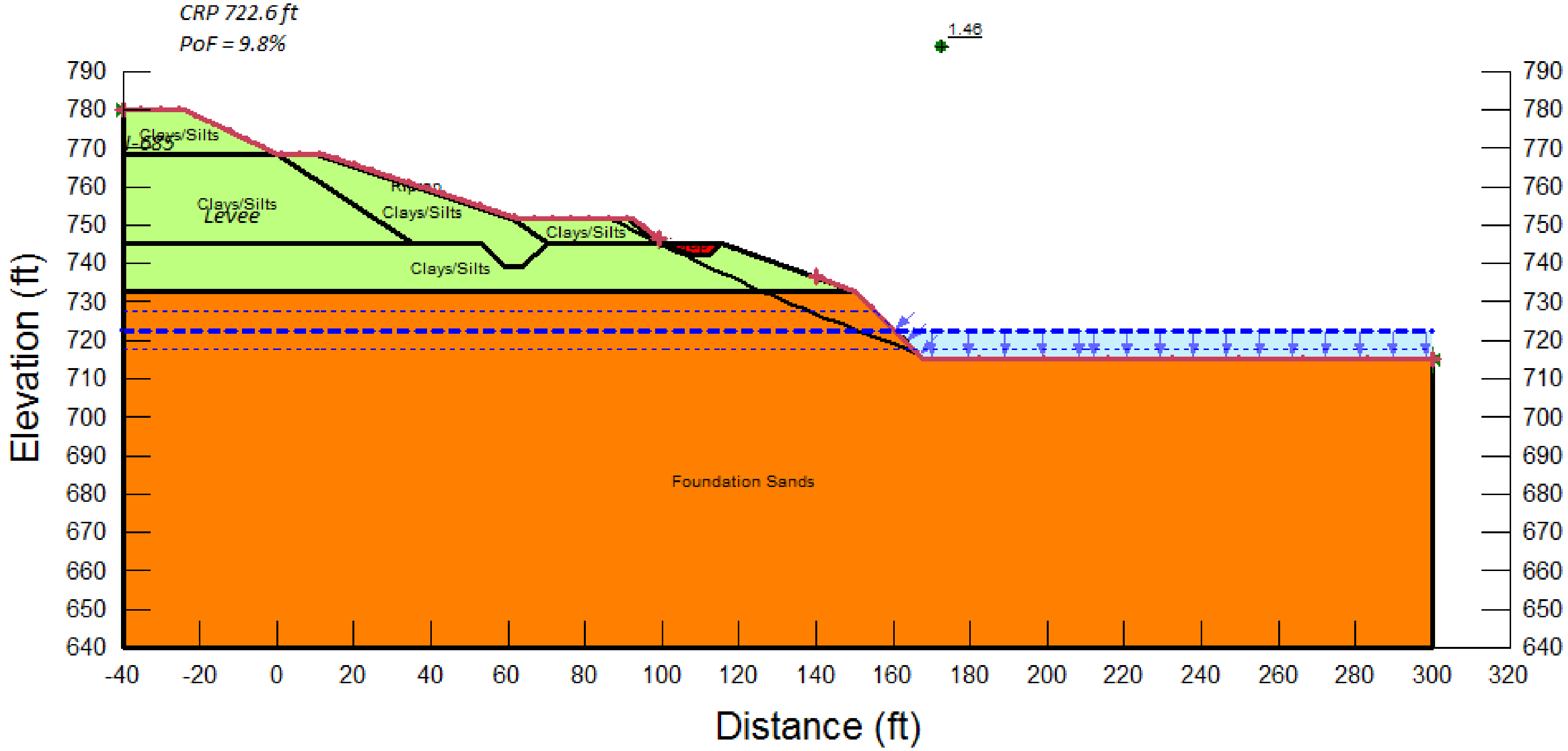


Figure 2.5.2-9: Riverside Slope Stability Results for L-385 Levee Station 103+00 (Harry Darby Memorial Highway – I-685 Hwy) under Existing Conditions with Water Elevation near CRP

Levee L-385
Station 103+00
Riverside Slope Stability Probabilistic Analysis
CRP 722.6 ft
PoF = 9.8%



Name: Clays/Silts Model: Mohr-Coulomb Unit Weight: Multiple Trial: 115 pcf Cohesion: 0 psf Phi: Multiple Trial: 28 ° Piezometric Line: 1
Name: Foundation Sands Model: Mohr-Coulomb Unit Weight: Multiple Trial: 120 pcf Cohesion: 0 psf Phi: Multiple Trial: 34 ° Piezometric Line: 1
Name: Riprap Model: Mohr-Coulomb Unit Weight: Multiple Trial: 135 pcf Cohesion: 0 psf Phi: Multiple Trial: 38 ° Piezometric Line: 1

2.5.3 Final Selection of Critical Cross Sections

Final critical levee sections were selected from those shown in **Table 2.4-1** to analyze impact of future degradation on bank stability that might impact FDRRS if no project degradation control is put in place. The final critical levee reaches were chosen based on proximity of the levee to the riverbank and concerns of foreshore reduction due to degradation. The selection of critical levee locations is remarked in **Table 2.5.3-1**. Degradation impacts on the non-Federal levees along the main channel identified in **Table 2.4-2** were quantified. The repair costs associated with channel slope along the non-Federal levees was assumed to be captured by the Bank Stabilization and Navigation Program (BSNP), see Memorandum for Record for Cost of Degradation. The repair costs related with non-Federal levees with foreshore less than 50 ft was assumed to consist of levee setback.

Table 2.5.3-1: Screening for Final Critical Levee Locations used for Evaluating Slope Stability under Future without Project Conditions

Approximate River Mile	Approximate Federal Levee Station	Station Range		Federal Levee	2013 Baseline Probability of Failure (PoF_b)	Remarks
		From	To			
RM 370.00	Station 45+00	34+27	60+67	North Kansas City	0.25	Critical because of proximity of levee to river.
RM 367.78	Station 21+27	0+00	25+14	Fairfax-Jersey Creek (Wharf)	10.10	Critical because of proximity of levee to river.
RM 368.60	Station 71+00	44+00	84+50	Fairfax-Jersey Creek	45.80	Critical because of proximity of levee to river and high PoF. Sloughing limited to channel bank.
RM 364.52	Station 60+00	57+26	74+00	East Bottoms	2.80	Critical because of proximity of levee to river.
RM 358.90	Station 89+00	89+00	93+00	Birmingham	24.30	Screened out because of long foreshore.
RM 366.50	Station 42+00	21+50	51+46	CID-Missouri	3.80	Critical because of proximity of levee to river.
RM 367.30	Station 1+00	0+00	9+00	CID-Kansas	2.50	Critical because of proximity of levee to river.
RM 374.10	Station 103+00	100+69	113+43	L-385	9.80	Screened out because of minimal degradation predicted by riverbed model.

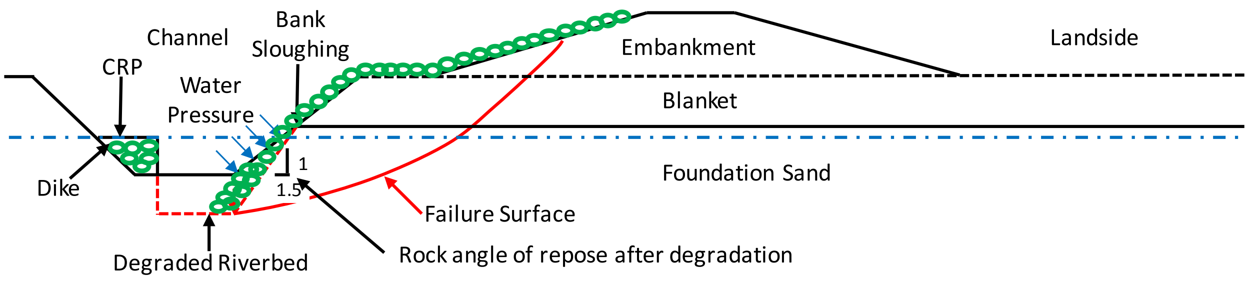
2.5.4 Critical Levee Geometry Description

All the critical levee sections have minimal foreshores and a riverside slope of 1 V: 3 H or 1V: 4 H (Vertical: Horizontal). The existing levee riverside slopes in some of these areas were armored with approximately 1 to 1.5 ft thick riprap layer that was keyed into the channel bottom according to the record drawings.

2.5.5 Riverbed Degradation Model Assumptions

The predicted 50 year riverbed degradation of the Missouri River was induced by a series of flows with a 50% probability of exceedance for the cumulative bed material transport at the end of 5, 10, 25, and 50 years starting in 2013, **Enclosure A.3**. Based on the model predictions, degradation is expected to increase with time and have a maximum degradation of approximately 12 ft over the next 50 years. In the riverbed model, degradation causes vertical lowering of the riverbed. Based on BSNP experience, locations where the riverbank is armored the riprap sloughs at approximately 1 V: 1.5 H (Vertical: Horizontal) slope measured from the toe of the degraded riverbed. As degradation continues, the riprap rolls downstream to the degraded riverbed and accumulates at the toe. The riverbed model also assumes that the water surface elevation decreases proportional to degradation, relative to the baseline CRP. An illustration of the degradation process, riverbed model, and slope stability failure surface is shown in **Figure 2.5.5-1**.

Figure 2.5.5-1: Illustration of Assumptions from Riverbed Degradation Model



2.6.1 Stability Analysis Assumptions and Impact on Stability Performance

In this analysis, lowering of the riverbed results in sloughing of the riverbank riprap around the CRP into the channel. The riprap collects at the toe, buttressing the slope toe. River degradation affects channel bank and levee stability in two ways. Degradation lowers the water surface, which improves effective stress (and in turn shear strength) in the bank. However, lowering the riverbed also reduces water passive resistance at the toe of the slope. Typically the net effect decreases the river bank stability except in the case at Station 45+00 (RM 370.0) where the rock toe configuration increases stability.

2.5.8 Final Critical Cross Section Riverbank Slope Stability Performance near FDRRS

To quantify degradation impacts on bank stability performance during non-flood events, analyses were conducted for various degradation amounts predicted by the future without project riverbed model. Bank stabilization repair was triggered when the riverbank slope stability attained a Probability of Failure (PoF) greater than or equal to 25%. The 25% threshold was selected because it corresponds to the failure surface (with Factor of Safety approximately 1.1) halfway on the river bank for the worst baseline condition at River Mile 368.6 (approximately Levee Station 71+00). Since bank sloughing due to degradation occurs progressively, 25% PoF was selected as the trigger point so that repairs could be implemented to ensure that banks are stable and FDRRS are not impacted.

The analysis was initiated with 1 ft of degradation and increased incrementally to a maximum of 15 ft; the maximum model predicted degradation is 7-8 ft. Stability performance up to 15 ft of degradation was evaluated to gain insights into performance beyond the 50 year prediction and for completeness. During the analysis, when a repair was triggered at a given degradation level, riprap revetment was assumed to be constructed to restore the channel bank to achieve PoF of 5%. The repair PoF of 5% was selected based on a parametric study of PoF after degradation repair versus cost of repair, see **Figure 2.5.7-1**. As seen in **Figure 2.5.7-1**, the repair threshold was selected based on the optimum repair frequency and associated repair cost.

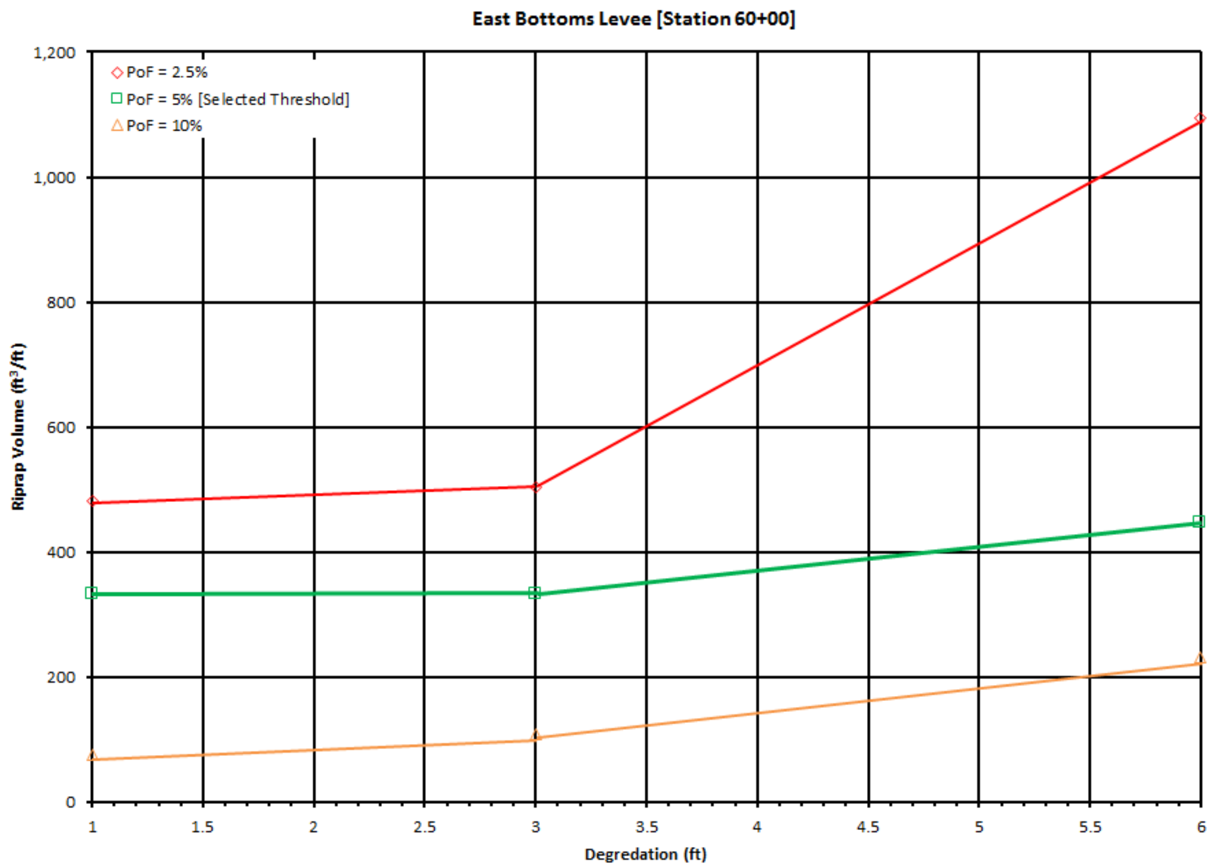


Figure 2.5.7-1: Illustration of Selection of Channel Bank Repair Threshold Probability of Failure.

During the analysis, the minimum riprap revetment required to obtain the required stability of the channel slope was determined iteratively. After revetment, degradation was assumed to continue eroding the riverbed and undermining the new riprap. Since degradation causes riprap displacement, the analyses assumed a constant riprap volume balance before and after degradation at each cross section. After degradation, the riprap at the toe launches into the degraded riverbed triggering rolling of the riprap below until the movement stabilizes at the angle of repose (1V:1.5H). When another repair is triggered, the revetment process was repeated. Riverbed channel rebound does not trigger repair unless the mass is undermined by degradation causing stability performance to fall beyond the threshold PoF of 25%.

The goal of this study was to capture O&M costs associated with revetment of river bank/channel slopes caused by degradation that would impact FDRRS. The summary of slope stability results for the analysis of the critical levees is tabulated in **Table 2.5.7-1** with output details **in Enclosure A.3**. Also shown in **Table 2.5.7-1**, in parentheses, is repair volume/foot of riprap required to restore stability to criteria (PoF of 5%). As seen, all the levee units are sensitive to degradation and require repair at some point except for the North Kansas City, CID-Missouri, and CID-Kansas levee units. The riprap volume required to repair the levee units was increased by 20% to account for riprap waste attributed to consolidation in underconsolidated riverbed sediment or random displacement by the swift Missouri River current during placement. This is based on BSNP experience with placement of riprap on the Fairfax-Jersey

Creek wharf and North Kansas City levees in 2011. Typical outputs of levee performance after degradation for a given levee unit are shown in **Figures 2.5.7-2** through **2.5.7-7**.

Typical schematics showing slope stability performance during the degradation process leading to O&M repair is shown in **Figure 2.2.7-8**. As shown, degradation lowers the riverbed which undermines the levee toe leading to rolling of riprap onto the degraded riverbed until a new equilibrium is attained. In this analysis, initial riprap repair was limited to 50 ft beyond the existing riverside levee toe. This set-back is based on BSNP requirements to avoid encroaching into the navigation channel.

Table 2.5.7-1: Summary of Slope Stability Results for Final Critical Levee Locations for Future without Project Conditions before Channel Slope Revetment

Degradation (ft)	**North Kansas City (Broadway Extension) [Station 45+00]**	**Fairfax-Jersey Creek (Wharf) [Station 21+27]**	**Fairfax-Jersey Creek (Levee) [Station 71+00]**	**East Bottoms [Station 60+00]**	**CID-Missouri [Station 42+00]**	**CID-Kansas [Station 1+00]**	**[a]Predicted Degradation Year**
Calculated Percent Probability of Failure (Riprap Repair Volume (ft^3/ft))							
1	0.30	9.6	47.3 (612)[b]	3.36	3.5	2.6	2014
3	0.55	12.1	3.5	7.36	2.15	2.5	2020
6	0.85	13.6	5	15.73	14.8	0.3	2038
7	1.20	13.6	4.3	15.73	18.5	0.2	2042
8	1.45	13.7	7.2	18.88	24.8	9.2	2045
9	1.70	13.8	7.8	19.33	29.7 (245)[b]	10.0	2047
12	11.22	24.2	26.1 (538)[b]	26.9 (1,010)[b]	5.9	12.7	2064
15	16.94	14.7	15	5.0	8.9	10.3	>2064

[a]Year of predicted degradation estimated from a series of flows with a 50% probability of exceedance for the cumulative bed material transport at the end of 5, 10, 25, and 50 years shown in **Enclosure A.3**.
[b]Repair volume includes 20% loss during placement as documented in previous BSNP projects.

The repair areas shown in **Table 2.5.7-1** were used for determining material repair volume for stabilizing levee/channel bank slope. This is the material needed to restore the river slope to meet PoF of 5%. The riprap quantity for the levee units was calculated from the repaired section by assuming that repair at the critical section was needed over the entire reach represented by the critical section, plus a 200 ft transition zone on each end, see **Table 2.5.3-2**. Based on the cost of buying and placing the riprap, a relationship was developed between river degradation and increased O&M repair cost to maintain channel bank bank and levee stability.

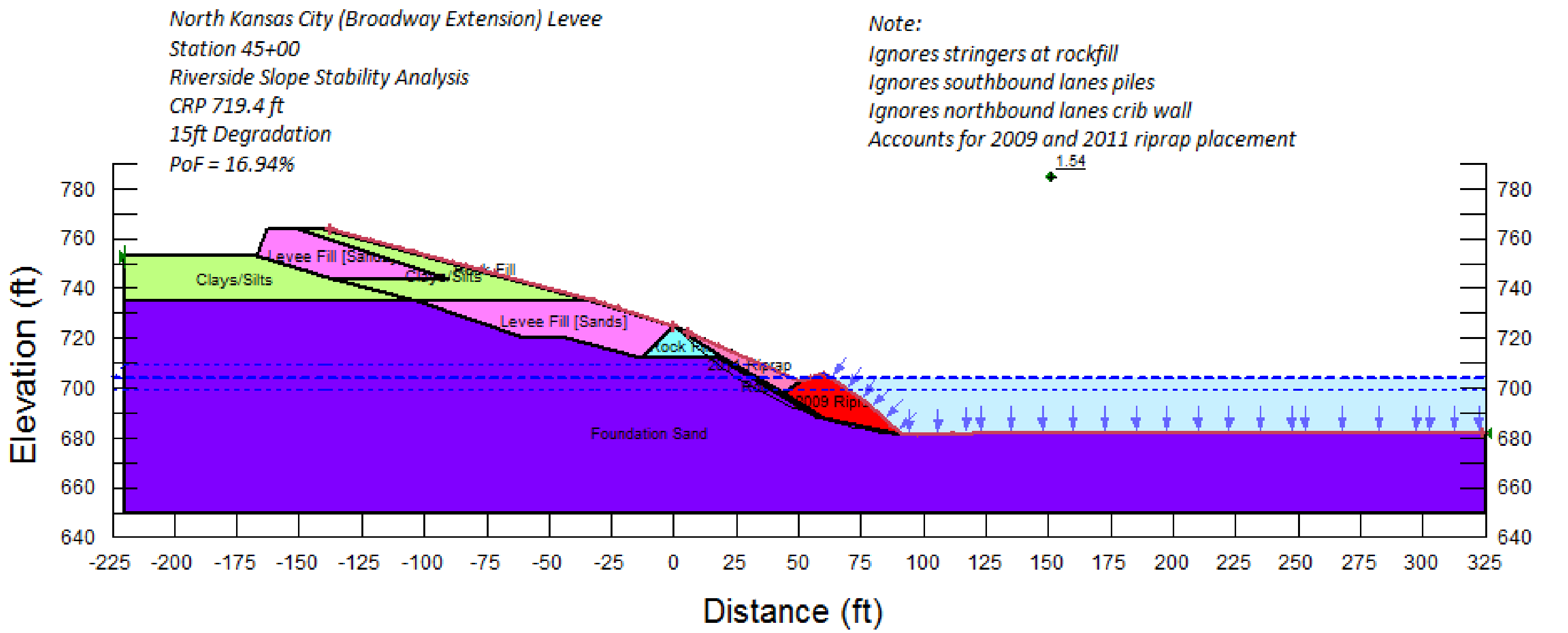
The cost of buying and placing the riprap was estimated by multiplying the riprap volume with a rate of \$49/ton or \$72/cubic yard (2014 dollars). The cost of repairing the levee units shown in **Table 2.5.7-1** is summarized in **Figure 2.5.7-9** which shows an average cost of approximately **\$6.4 million dollars** to repair the channel bank for the Fairfax-Jersey Creek levee unit after 1 ft and 12 ft of degradation. The cost of repairing the East Bottoms levee unit is roughly **\$5.7 million dollars** after 12 ft of degradation. And the cost of repairing the CID Missouri levee after 9 ft of degradation is approximately **\$2.2 million dollars**.

Performance of the East Bottoms levee unit was analyzed under a worst case scenario where channel bank repairs are neglected until 12 ft of riverbed degradation (maximum predicted in 50 years) has occurred for comparison. Under the scenario (see **Figure 2.2-4**) the East Bottoms levee unit has PoF of approximately 31.4%. The outputs from the analysis before and after repair are shown in **Figures 2.5.7-**

10 and **2.5.7-11**, respectively. However, the critical surface does not include the FDRRS. Based on this analysis, the repair in the worst case scenario will cost around **$6.9 million dollars**.

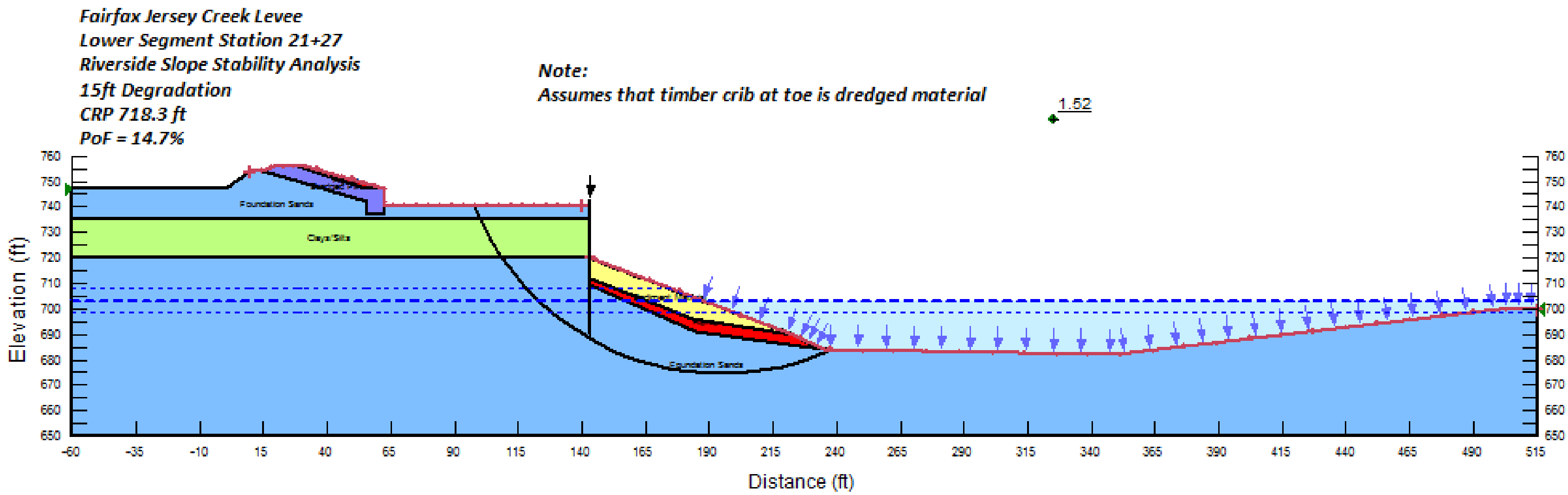
To determine the increased O&M repair cost with time, which is attributed to degradation, the relationship between O&M repair cost versus riverbed degradation and the relationship between riverbed degradation and time should be combined with the annual chance of exceedance of degradation with time.

Figure 2.5.7-2: No Channel Bank Repair Required for the North Kansas City (Broadway Extension) Station 45+00 after 15 ft of Degradation under Future without Project Condition.



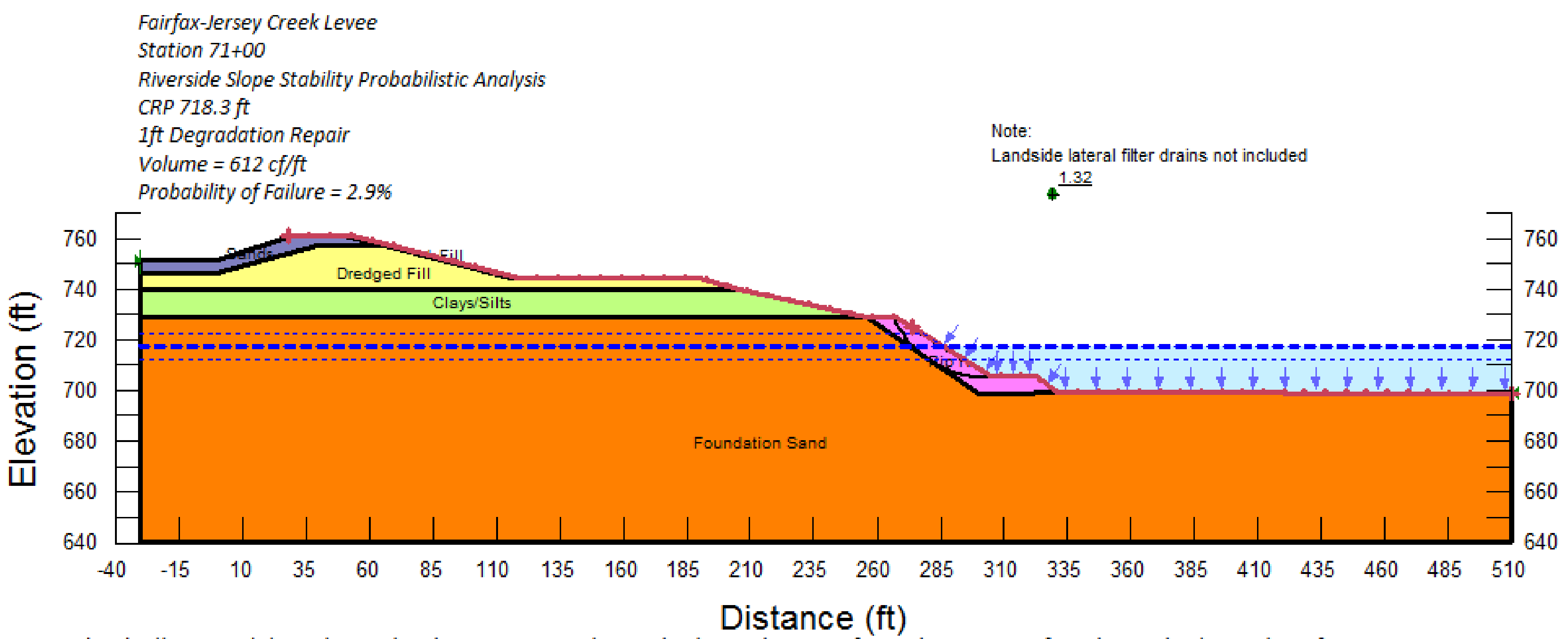
Name: Clays/Silts Model: Mohr-Coulomb Unit Weight: Multiple Trial: 115 pcf Cohesion: 0 psf Phi: Multiple Trial: 28 ° Piezometric Line: 1
Name: Levee Fill [Sands] Model: Mohr-Coulomb Unit Weight: Multiple Trial: 115 pcf Cohesion: 0 psf Phi: Multiple Trial: 30 ° Piezometric Line: 1
Name: Foundation Sand Model: Mohr-Coulomb Unit Weight: Multiple Trial: 120 pcf Cohesion: 0 psf Phi: Multiple Trial: 34 ° Piezometric Line: 1
Name: Rock Fill Model: Mohr-Coulomb Unit Weight: Multiple Trial: 125 pcf Cohesion: 0 psf Phi: Multiple Trial: 35 ° Piezometric Line: 1
Name: 2009 Riprap Model: Mohr-Coulomb Unit Weight: 135 pcf Cohesion: 0 psf Phi: 38 ° Piezometric Line: 1
Name: 2011 Riprap Model: Mohr-Coulomb Unit Weight: Multiple Trial: 135 pcf Cohesion: 0 psf Phi: Multiple Trial: 38 ° Piezometric Line: 1

Figure 2.5.7-3: Repair of Riverside Slope of Fairfax-Jersey Creek Floodwall Wharf Station 21+27 after 15 ft of Degradation under Future without Project Condition.



Name: Dredged Fill Model: Mohr-Coulomb Unit Weight: Multiple Trial: 100 pcf Cohesion: 0 psf Phi: Multiple Trial: 30 ° Piezometric Line: 1
Name: Clays/Silts Model: Mohr-Coulomb Unit Weight: Multiple Trial: 115 pcf Cohesion: 0 psf Phi: Multiple Trial: 28 ° Piezometric Line: 1
Name: Sediment Material Model: Mohr-Coulomb Unit Weight: Multiple Trial: 90 pcf Cohesion: 0 psf Phi: Multiple Trial: 22 ° Piezometric Line: 1
Name: Riprap Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 0 psf Phi: 35 ° Piezometric Line: 1
Name: Foundation Sands Model: Mohr-Coulomb Unit Weight: Multiple Trial: 120 pcf Cohesion: 0 psf Phi: Multiple Trial: 34 ° Piezometric Line: 1

Figure 2.5.7-4: Repair of Riverside Slope of Fairfax-Jersey Creek Floodwall Wharf Station 21+27 after 1 ft of Degradation under Future without Project Condition. The Levee Unit also requires Repair after 12 ft of Riverbed Degradation.



Name: Dredged Fill   Model: Mohr-Coulomb   Unit Weight: Multiple Trial: 100 pcf   Cohesion: 0 psf   Phi: Multiple Trial: 26 °   Piezometric Line: 1
Name: Clays/Silts   Model: Mohr-Coulomb   Unit Weight: Multiple Trial: 115 pcf   Cohesion: 0 psf   Phi: Multiple Trial: 28 °   Piezometric Line: 1
Name: Sands   Model: Mohr-Coulomb   Unit Weight: 115 pcf   Cohesion: 0 psf   Phi: 30 °   Piezometric Line: 1
Name: Rock Fill   Model: Mohr-Coulomb   Unit Weight: Multiple Trial: 125 pcf   Cohesion: 0 psf   Phi: Multiple Trial: 35 °   Piezometric Line: 1
Name: Foundation Sand   Model: Mohr-Coulomb   Unit Weight: Multiple Trial: 120 pcf   Cohesion: 0 psf   Phi: Multiple Trial: 34 °   Piezometric Line
Name: Rip Rap   Model: Mohr-Coulomb   Unit Weight: Multiple Trial: 135 pcf   Cohesion: 0 psf   Phi: Multiple Trial: 38 °   Piezometric Line: 1

Figure 2.5.7-5: Typical Repair of Riverside Slope of East Bottoms Levee Station 60+00 after 12 ft of Degradation under Future without Project Condition at different Levels of Degradation.

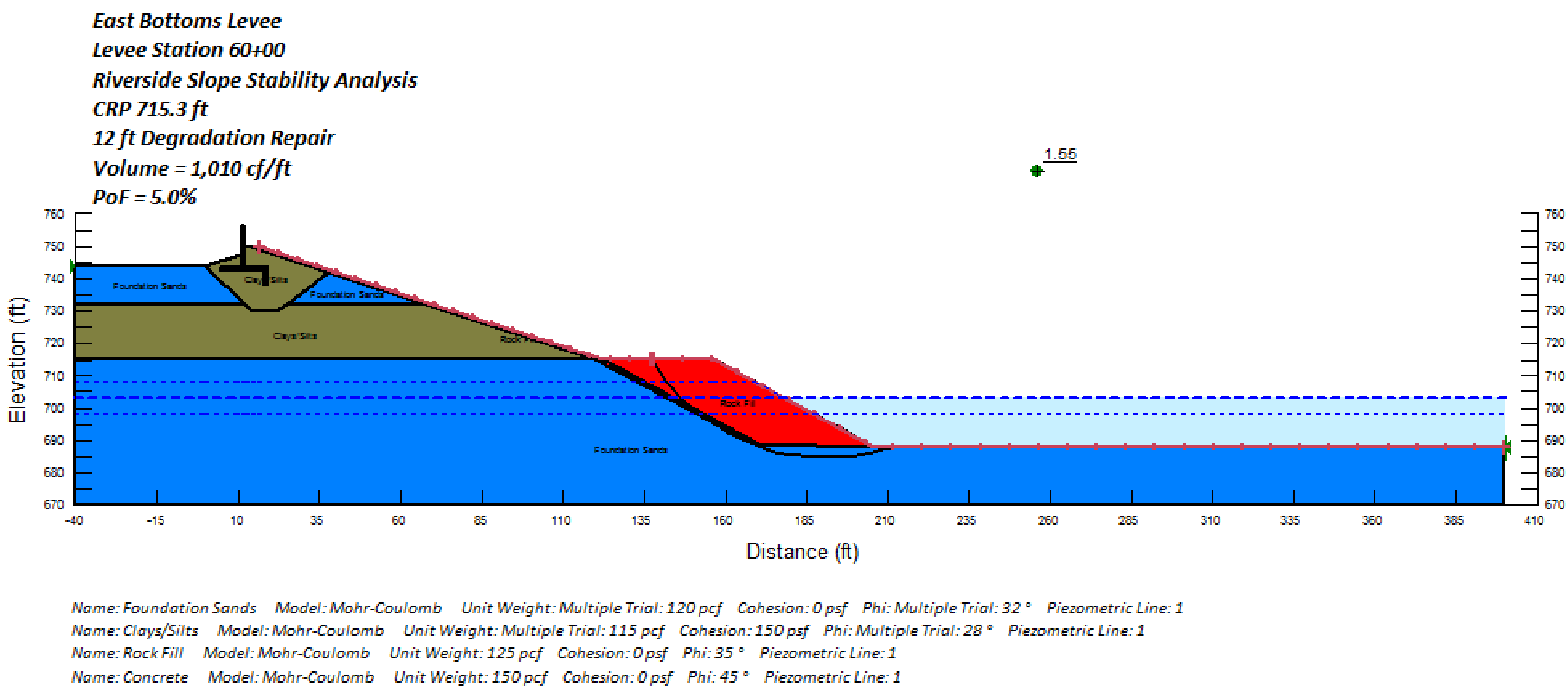


Figure 2.5.7-6: Typical Repair of Riverside Slope of CID Missouri Levee Station 42+00 after 9 ft of Degradation under Future without Project Condition at different Levels of Degradation.

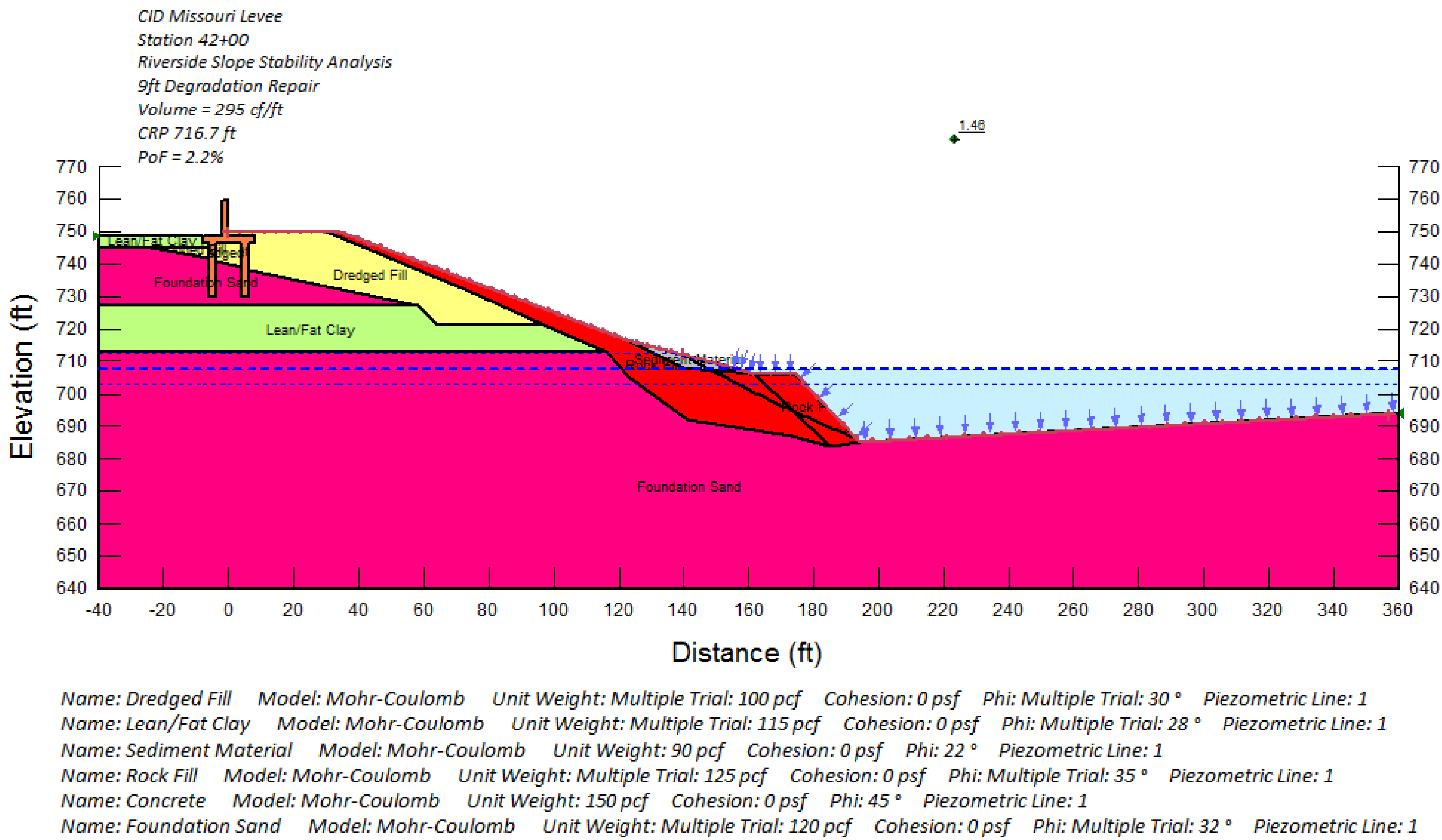
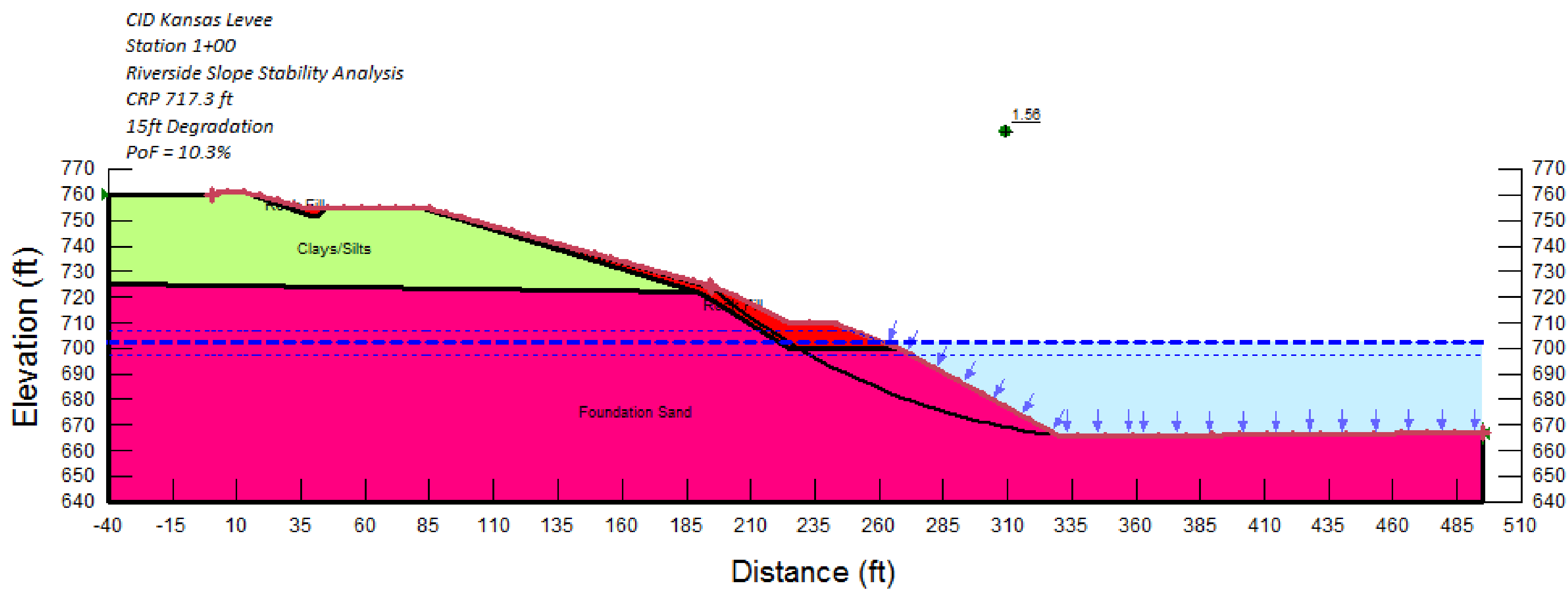


Figure 2.5.7-7: Typical Repair of Riverside Slope of CID Kansas Levee Station 1+00 after 15 ft of Degradation under Future without Project Condition at different Levels of Degradation.



Name: Clays/Silts Model: Mohr-Coulomb Unit Weight: Multiple Trial: 115 pcf Cohesion: 0 psf Phi: Multiple Trial: 28 ° Piezometric Line: 1
Name: Rock Fill Model: Mohr-Coulomb Unit Weight: Multiple Trial: 125 pcf Cohesion: 0 psf Phi: Multiple Trial: 35 ° Piezometric Line: 1
Name: Foundation Sand Model: Mohr-Coulomb Unit Weight: Multiple Trial: 120 pcf Cohesion: 0 psf Phi: Multiple Trial: 34 ° Piezometric Line: 1

Figure 2.5.7-8: Typical Slope Stability Performance Schematic Triggering Operation and Maintenance Bank Repair

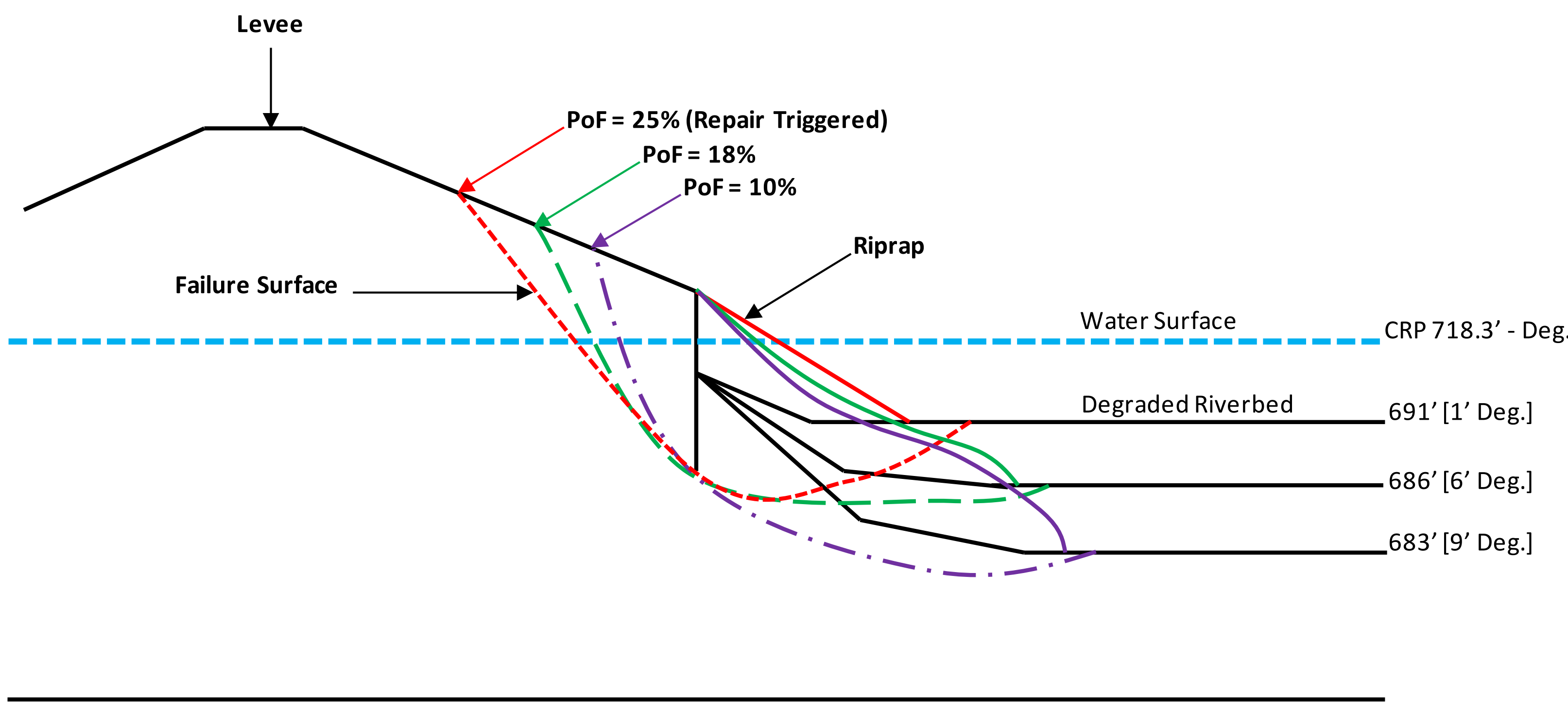


Figure 2.5.7-9: Summary of O&M Channel Bank Repair Cost of Resulting from Riverbed Degradation

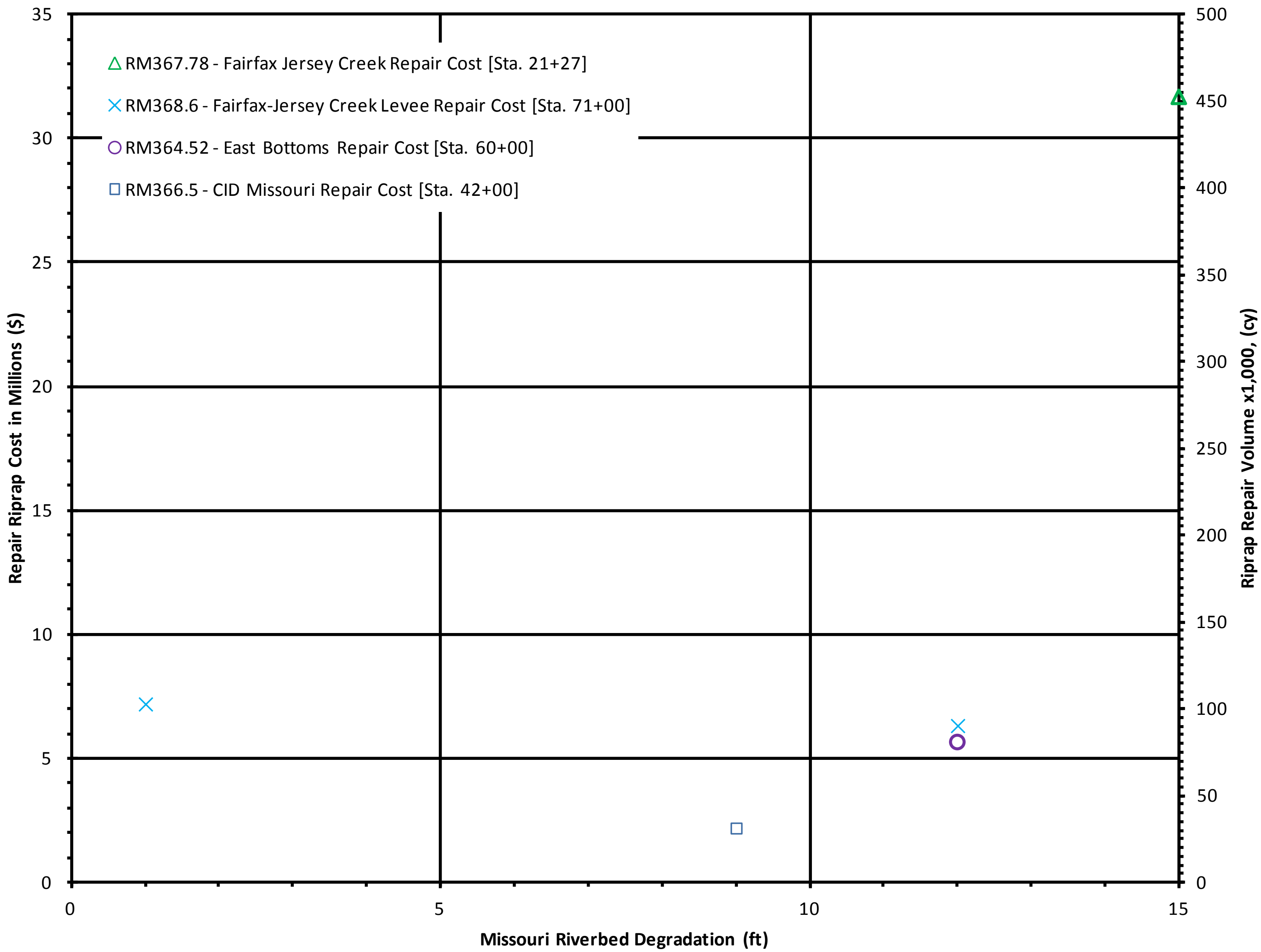


Figure 2.5.7-10: Worst Case Scenario of Neglecting Channel Bank Repairs until after 12 ft of Riverbed Degradation.

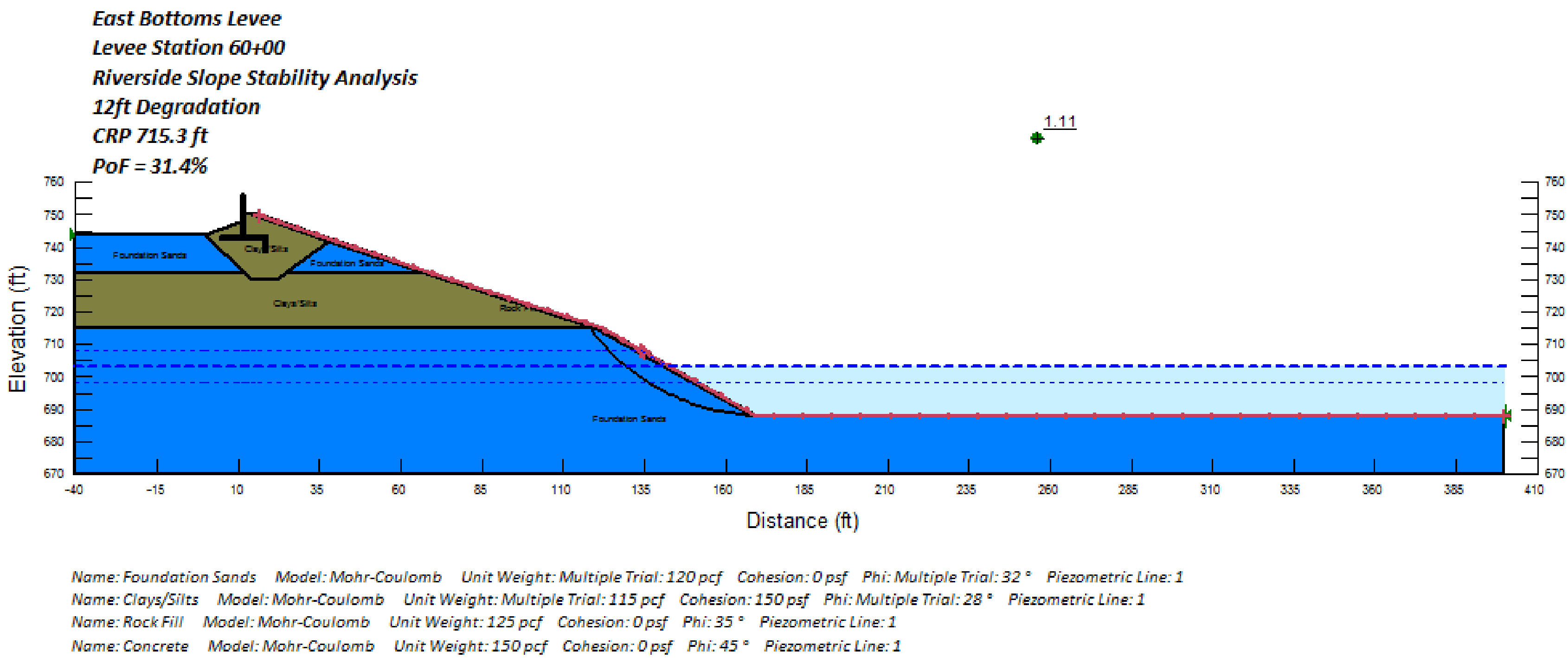
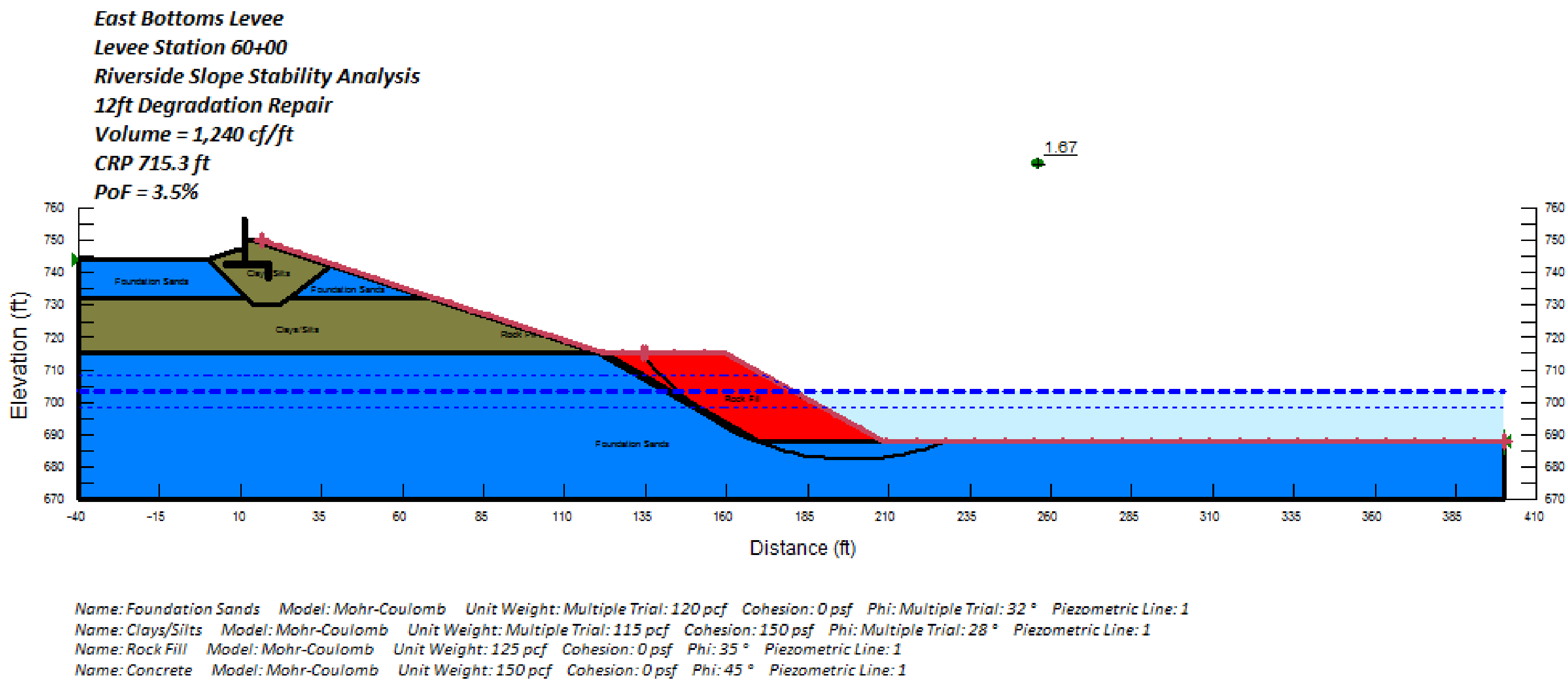


Figure 2.5.7-11: Channel Bank Repair Associated with Worst Case Scenario of Neglecting Channel Bank Repairs until after 12 ft of Riverbed Degradation.


East Bottoms Levee
Levee Station 60+00
Riverside Slope Stability Analysis
12ft Degradation Repair
Volume = 1,240 cf/ft
CRP 715.3 ft
PoF = 3.5%
1.67
Foundation Sands
Clays/Silts
Foundation Sands
Clays/Silts
Rock Fill
Rock Fill
Foundation Sands
Elevation (ft)
Distance (ft)
760
750
740
730
720
710
700
690
680
670
-40
-15
10
35
60
85
110
135
160
185
210
235
260
285
310
335
360
385
410
Name: Foundation Sands Model: Mohr-Coulomb Unit Weight: Multiple Trial: 120 pcf Cohesion: 0 psf Phi: Multiple Trial: 32 ° Piezometric Line: 1
Name: Clays/Silts Model: Mohr-Coulomb Unit Weight: Multiple Trial: 115 pcf Cohesion: 150 psf Phi: Multiple Trial: 28 ° Piezometric Line: 1
Name: Rock Fill Model: Mohr-Coulomb Unit Weight: 125 pcf Cohesion: 0 psf Phi: 35 ° Piezometric Line: 1
Name: Concrete Model: Mohr-Coulomb Unit Weight: 150 pcf Cohesion: 0 psf Phi: 45 ° Piezometric Line: 1

2.5.8 Estimated Cost of Repairing Channel Bank and non-Federal Levees along the Missouri River

The cost of repairing the non-Federal levees along the Missouri River was quantified as shown in **Table 2.5.8-1**.

Table 2.5.8-1: Estimated Cost of Repairing non-Federal Levees along the Missouri River

Approx. River Mile	River Bank	Non-Federal Levee	Non-Federal Levee Height (ft)	Foreshore Distance (ft)	Levee Setback Distance (ft)	Levee Material Vol. (cy)	Cost of Levee Material ($)
RM 434.6 - 435.1	Left	Geary Bend	8	60	N/A	N/A	N/A
RM 426.2 - 426.4	Left	Rushville Sugar Creek	8	85			
RM 416.8 - 417.3	Left	Bean Lake	7	20	3,500	28,133	361,466
RM 411.5 - 411.7	Right	Henry Pohl	10	45	3,600	53,333	685,333
RM 410 - 410.1	Right	Henry Pohl	5	20	2,000	9,259	118,981
RM 407.6 - 408.9	Left	Struve	7	20	5,500	44,204	568,018
RM 399.2 - 400.2	Right	Sherman Army Airfield	15	60	N/A	N/A	N/A
RM 351.6 - 354.2	Left	City of Independence	4	<10	7,200	23,467	301,547
RM 349.3 - 351.6	Left	Bruening	6	30	4,000	24,889	319,822
RM 348.5 - 349.3	Left	Allen	4	200	N/A	N/A	N/A
RM 345.6 - 349.3	Left	Bell	4	90			

Note:
N/A = Not Applicable because foreshore distance is greater than 50 feet.
Levee setback cost assumed $12.85/cy of impervious material (material + placement).
[a]Foreshore greater than 50 ft so repair cost is only associated with BSNP.
[b]Foreshore is less than 50 ft so repair cost consists of BSNP and non-federal levee setback.

2.5.9 Estimated Cost of Constructing Grade Control Structures at the Confluence of Tributaries on the Missouri River

The cost of limiting degradation using a grade control structure to the channel of the Missouri River at the confluence of tributaries was quantified as shown in **Table 2.5.9-1**.

Table 2.5.9-1: Estimated Cost of Constructing Grade Structures at the Confluence of Tributaries.

Approx. River Mile	River Bank	Confluence Location	Tieback Location	Channel Width (ft)	Grade Control Cost ($)
RM 437.3	Left	MRSL 455L	Federal	120	201
RM 433.5	Right	Roundy	Non-Federal	220	270
RM 418.2	Left	Rushville Sugar Creek	Non-Federal	140	171
RM 406.2	Right	Grape/Bollin/Schwartz	Creek Bank	200	116

Note: The following confluence locations were not analyzed because they have revetment and it was assumed that armoring will arrest degradation from the Missouri River Channel.
RM 434.1, Right Bank, Jones Non-Federal Levee
RM 406.2, Right Bank, Grape/Bollin/Schwartz Non-Federal Levee
RM 403.3, Left Bank, Kirk Non-Federal Levee
RM 388.1, Right Bank, Kansas City Department of Corrections Non-Federal Levee

2.6 Future Conditions

2.6.1 Alternatives Array

To address riverbed degradation, an array of three alternatives was developed with the first being do nothing option. Summary of the alternatives is shown in **Table 2.6.1-1** with detailed discussion found in the alternatives array section. As seen in **Table 2.6.1-1**, alternative 4 involves modifying the dikes and sills on the inside bend of the Missouri River while alternative 5 which consist of only grade control. The 200 ft of channel widening in alternative 4 may affect underseepage and stability performance of the levee units because it reduces foreshore. Schematics of alternatives 4 and 5 are shown in **Figures 2.6.1-1** and **2.6.1-2**, respectively. As shown in **Figure 2.6.1-2** (alternative 5), the grade control across the river channel increases resistance at the levee toe improving stability. Alternative 4 was considered critical in this study because it lacks grade control. As a result, underseepage and stability analysis was performed on alternative 4 to quantify the impact of reduced foreshore due to the 200 ft channel widening.

Table 2.6.1-1: Viable Array of Alternatives

Alternative Number	Alternative Description
1	No Structural Action – Future Without Project Condition
4	Lower the dike portion of the dike structure and excavate land from RM 291 to 449 to -5 feet of CRP for 200 feet landward of the rectified channel line.
[a]5	Six bed grade control structures (crest at -11 feet of CRP) from RM 393.63 to 443.39

[a]Grade control along River Mile (RM) 447.9, 442.2, 429.1, 426.1, 399.5, 387.7, 386.8, 385.4, 383.1, 379, 375.5, 371.6, 369.2, 366.2, 362.4, 359, 356, 351.4, 349.1, and 347.

Figure 2.6.1-1: Schematic showing Alternative 4 with both dike and sill lowered to the same elevation and 200 ft landward channel widening

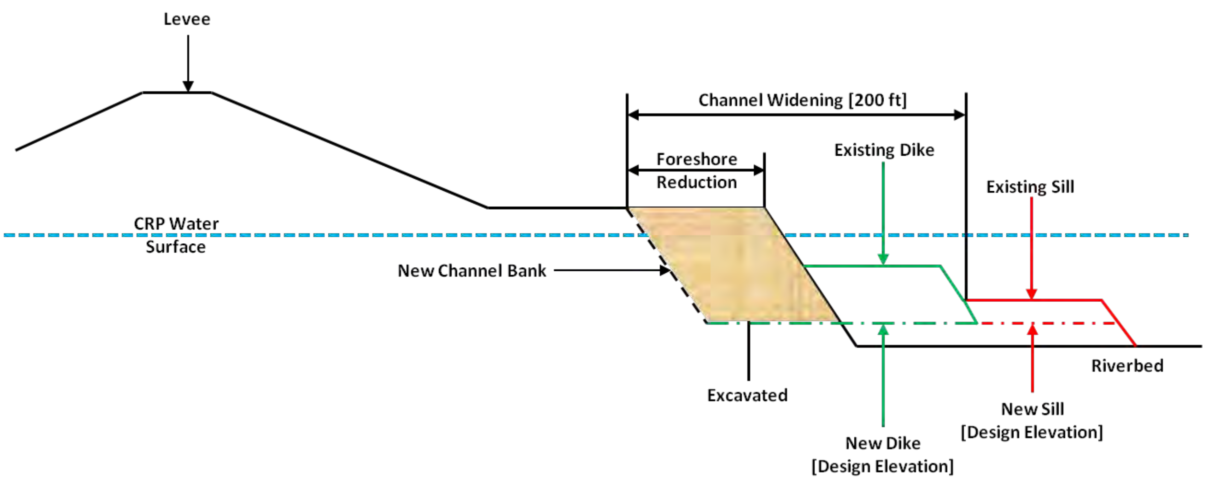
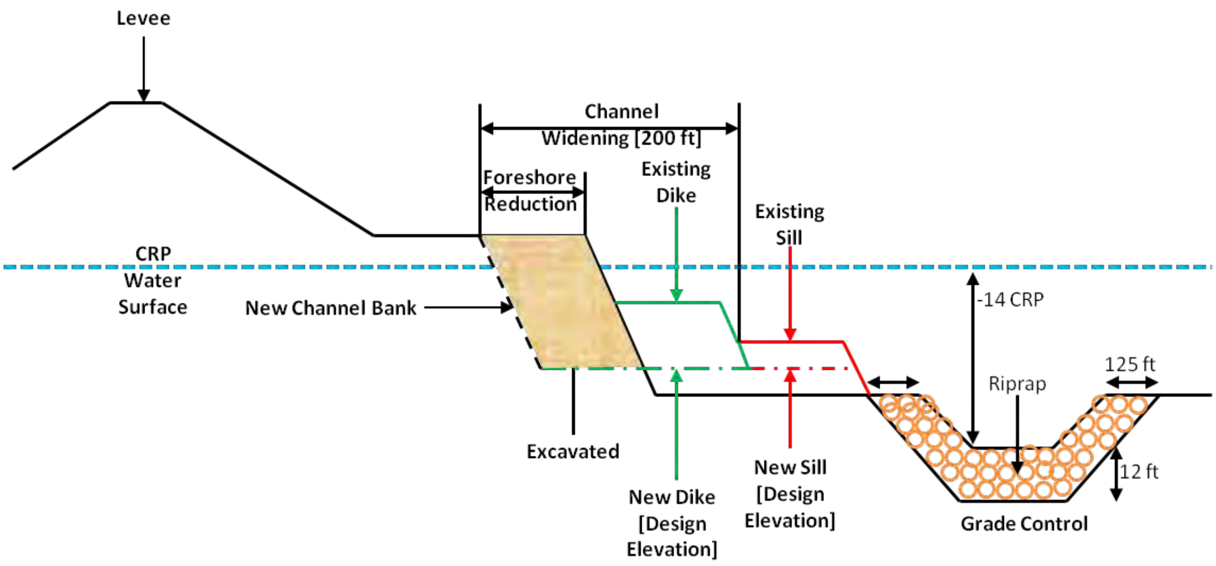


Figure 2.6.1-2: Schematic showing Alternative 5 with both dike and sill lowered to same elevation, 200 ft landward channel widening, and channel grade control



2.6.2 Impacted Levee Units

Alternative 4 impacts the inside bends of 9 levee units along the levee reaches shown in **Table 2.6.2-1** which are different than those analyzed in the future without project conditions, see **Section 2.5.7**. At each levee reach, the critical stations in **Table 2.6.2-1** were selected based on shortest foreshore after the 200 ft landward channel widening. Layout of the levee units with the proposed channel widening and reduced foreshores are shown in **Enclosure A.4**. The map in **Enclosure A.4** show the limits of the widened channel. The levee sections affected by Alternative 4 are not necessarily the same critical sections under existing conditions.

Table 2.6.2-1: Levee Units Impacted by 200 ft Channel Widening: Alternative 4

Approximate River Mile	Station		Critical Station	Levee	CRP ELEV. [FT]	Design Dike/Sill ELEV. [FT]	Original Foreshore [FT]	Reduced Foreshore [FT]
	From	To						
447.9[a]	423+00	425+00	424+00	MRLS471-461R	796.65	NA	515	[b]515
442.2[a]	274+00	276+00	275+00	MRLS455L	792.11	NA	1,800	1,600
429.1[a]	192+00	194+00	193+00	MRLS448-443L	780.05	NA	1,600	1,460
424.8[a]	248+00	250+00	249+00	MRLS440R	775+84	NA	1,880	1,680
399.5[a]	143+00	145+00	144+00	MRLS408L	750.41	NA	560	[b]560
385.6[a]	279+00	281+00	280+00	MRLS400L	734.84	NA	1,900	1,700
366.6	163+00	190+00	[c]190+00	North Kansas City	715.0	713.0	540	225
358.6	134+00	298+00	[c]134+00	East Bottoms	714.4	709.6	1,150	690
358.6	104+00	110+00	[c]104+00	Birmingham	711.0	709.0	620	180
[d]353.2	-	-	-	City of Independence	707.7	707.1	610	110
[d]350.5	-	-	-	Bruening Set-back	706.0	705.2	345	0
[d]349.8	-	-	-	Bruening	705.6	705.2	520	220
[d]349.1	-	-	-	Allen	704.9	703.6	1,030	700
[d]348.0	-	-	-	Bell	703.6	708.6	590	290

[a]No underseepage or slope stability impacts.
[b]No foreshore reduction.
[c]Shortest foreshore locations for Federal levees.
[d]Shortest foreshore locations for non-Federal levees.
NA = Not Applicable.

2.6.3 Underseepage Analysis

The summary of the underseepage analysis for the Federal levees impacted by alternative 4 are shown in **Table 2.6.3-1** with details found in **Enclosure A.5**. For a given levee reach, the underseepage analysis was performed at the critical station (reduced foreshore: shortest foreshore). The impact of degradation on the underseepage performance of the levee units between RM 385.6 and 448.0 was also evaluated and was found to be insignificant. The underseepage factor of safety (FS) for the North Kansas City Airport levee reach was less that the allowable FS of 1.6. Preliminary design showed that due to landside constraints seventeen additional relief wells are required to be installed between the existing relief wells. The relief wells will be spaced approximately 100 ft apart fully penetrating (approximately 50 ft deep). Installing the new relief wells will also involve modifying the existing relief well collector system.

As seen in **Table 2.6.3-1**, the FS for the East Bottoms levee segment without relief wells between Station 130+00 and 298+00 is greater than the allowable FS of 1.6. However, the levee reach with existing relief wells between Station 298+00 and to 330+00 needs a total of 4 additional relief wells. Like the North Kansas City Airport levee, the new relief wells will be spaced approximately 100 ft and fully penetrating (approximately 50 ft deep). Installing the relief wells will also involve modifying the relief well collector system. As shown in **Table 2.6.3-1**, the performance of the Birmingham levee is deficient at Station 104+00. To address the underseepage deficiency, a 300 ft wide underseepage berm roughly 1,000 ft long is recommended to be constructed on the landside.

Underseepage analysis for the non-Federal levee units was not performed due to lack of as-built drawings and subsurface information. However, based on underseepage performance experience with non-Federal levees of similar geometry a 100 ft underseepage berm width was recommended to be constructed along the levee lengths shown in **Table 2.6.3-2**. In areas with landside constraints like the City of Independence levee (Water Treatment Plant) construction of 14 relief wells spaced 250 ft apart is recommended. A relief well collector system is also recommended to be constructed for the full length of the relief wells, **Table 2.6.3-2**. A set-back is recommended for the Bruening levee unit between RM 350.1 and 350.5 due to foreshore reduced to less than 10 ft.

It is recommended that the preliminary design of the relief wells be refined in the Preconstruction Engineering and Design (PED) phase of this study. The refinement should also account for modification of the existing relief well collector systems at the North Kansas City Airport and East Bottoms levees unit and the collector system at the City of Independence (Water Treatment Plant) levee unit.

Table 2.6.3-1: Underseepage Analysis Results and Proposed Solutions to Address Deficiency for Future with Project Conditions for Federal Levees

Approximate River Mile	Station		[c]Critical Station	Federal Levee	Factor of Safety [FS]	Number of Relief Wells	Underseepage Berm Width (ft)	Length (ft)
	From	To						
367.8[a]	163+00	176+00	165+00	North Kansas City Airport	1.07	8	N/A	N/A
367.2[a]	176+00	188+50	176+00	North Kansas City Airport	1.08	2	N/A	N/A
366.6[a]	188+00	193+00	190+00	North Kansas City	0.92	7	N/A	N/A

				Airport				
363.2[b]	130+00	160+00	130+00	East Bottoms	2.05	N/A	N/A	N/A
362.2[b]	160+00	298+00	170+00	East Bottoms	1.62	N/A	N/A	N/A
359.3[a]	298+00	318+00	298+00	East Bottoms	1.14	1	N/A	N/A
358.6[a]	298+00	330+00	330+00	East Bottoms	1.08	3	N/A	N/A
358.6	94+00	104+00	104+00	Birmingham	1.52	N/A	300	1,000

[a]Levee reaches with existing relief wells. Additional relief wells evaluated.
[b]Levee reach without relief wells.
[c]Location of shortest foreshore considered representative of levee reach analyzed.
N/A: Not Applicable.

Table 2.6.3-2: Proposed Solutions to Address Underseepage Deficiency for Future with Project Conditions for Non-Federal Levees

Approximate River Mile		[c]Critical River Mile	Non-Federal Levee/Berm	Channel Bank	Number of Relief Wells	Underseepage Berm Width (ft)[c]	Length (ft)
From	To						
354.2	353.5	353.5	City of Independence	Right	N/A	N/A	3,400
[a]353.5	352.7	353.2	City of Independence (Water Plant)	Right	14[d]	100	3,500
353.2	351.9	352.6	City of Independence	Right	N/A	100	4,200
[b]350.8	350.1	350.5	Bruening (Set-back)	Left	N/A	100	3,800
350.5	349.3	349.8	Bruening	Left	N/A	100	5,700
349.3	348.0	349.1	Allen	Left	N/A	100	6,300
348.0	347.4	348.0	Bell	Left	N/A	100	2,500

[a]Landside constrained by Water Treatment Plant.
[b]Set-back needed due to foreshore less than 10 ft.
[c]Underseepage berm width assumed for length of non-Federal levee/berm.
[d]Assumed 250 ft spacing for relief wells in City of Independence Water Treatment Plant.
N/A: Not Applicable.

2.6.4 Slope Stability Analysis

Steady state performance of the riverside slope stability during low-flow (water at CRP) of the critical levee units was evaluated using the design values shown in **Table 2.6.4-1**. The analysis accounted for reduced foreshore, the presence of dike/sill on the riverside toe, and the absence of dike/sill at the critical reaches. The channel cross sections together with the embankment and dike/sill geometries are shown in **Enclosure A.4**. It was assumed that the area between the dikes/sills will erode with time and stabilize when it reaches the bank alignment at the dike/sill zones. The results of the stability analysis for alternative 4 at the dike/sill locations and areas without the dikes/sills are shown in **Table 2.6.4-2** and **Table 2.6.4-3**, respectively.

The outputs for the areas without the dikes/sills are shown in **Figures 2.6.4-1** through **2.6.4-4**. Outputs at the dike/sill locations are found in **Enclosure A.5**. As seen in **Tables 2.6.4-1** and **2.6.4-2**, the impact of the 200 ft channel widening associated with alternative 4 is insignificant for areas with and without dikes/sills for all the levee units except for the North Kansas City Airport levee. The channel slope along the North Kansas City Airport levee is less than the allowable FS of 1.4 but is equal to FS of 1.0 suggesting that the channel bank may be susceptible to shallow slides. Revetment of the channel bank along the North Kansas City Airport is recommended to address negative underseepage performance impact and future slope stability issues. The recommended revetment will require approximately 72 cubic yard/ft of riprap. Based on a unit price of $49/ton/ft (2014 dollars), the revetment cost between Station 163+00 and 193+00 is roughly $18,522,000.00.

Table 2.6.4-1: Summary of Design Geotechnical Parameters used in Slope Stability Analysis for Future with Project

			ETL 1110-2-556 Recommended Range for Coefficient of Variation		Coefficient of Variation used in Estimating Design Values[1]	
Material	**Design Unit Weight (lb/ft^3)**	**Design Effective Angle of Internal Friction (Degrees)**	**Unit Weight**	**Angle of Internal Friction**	**Unit Weight**	**Angle of Internal Friction**
Levee Fill (Sands)	112	28	3% - 8%	3% - 12%	5%	10%
Levee Fill (Clays/Silts)	112	26	3% - 8%	7% - 10%	5%	9%
Dredged Fill	97	28	3% - 8%	N/A	5%	10%
Rip Rap	131	37	3% - 8%	N/A	5%	3%
Foundation Sands	117	32	3% - 8%	3% - 12%	5%	10%

[1]Coefficient of Variation (COV) used for estimating design value (Mean – ½ Standard Deviation)

Table 2.6.4-2: Slope Stability Analysis Results for Future with Project Conditions at Dike/Sill Locations

Approximate River Mile	Station		Critical Station	Federal Levee	Factor of Safety [FS]
	From	To			
366.6	163+00	193+00	190+00	North Kansas City Airport	1.13
358.6	130+00	330+00	330+00	East Bottoms	1.85
358.6	94+00	104+00	104+00	Birmingham	1.75

Table 2.6.4-3: Slope Stability Analysis Results for Future with Project Conditions between Dike/Sill Locations

Approximate River Mile	Station		Critical Station	Federal Levee	Factor of Safety [FS]
	From	To			
366.6	163+00	193+00	190+00	North Kansas City Airport	1.00
358.6	130+00	330+00	330+00	East Bottoms	1.83
358.6	94+00	104+00	104+00	Birmingham	1.65

Figure 2.6.4-1: North Kansas City Levee Station 190+00 Stability under Future with Project Conditions at Location between Dikes/Sills

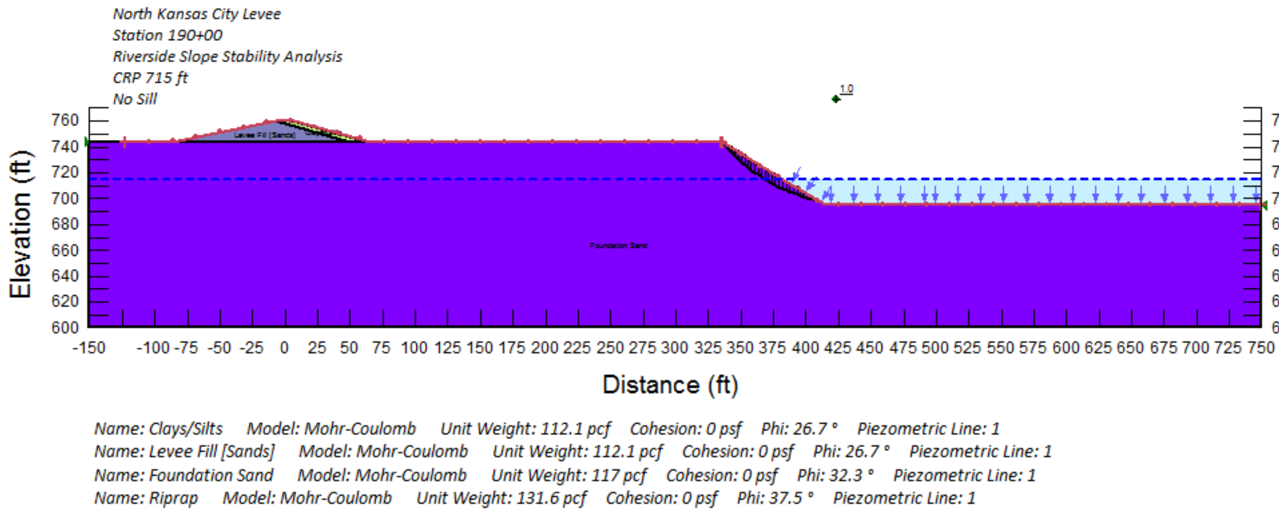


Figure 2.6.4-2: North Kansas City Levee Station 190+00 Channel Bank Repair under Future with Project Conditions at Location between Dikes/Sills

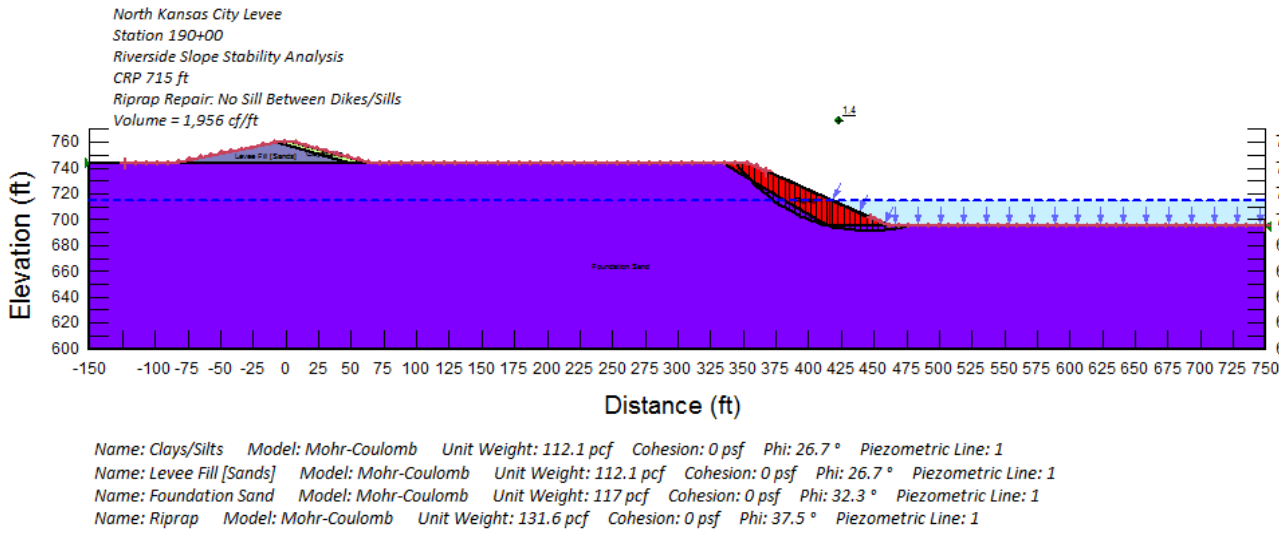


Figure 2.6.4-3: East Bottoms Levee Station 330+00 Stability under Future with Project Conditions at Location between Dikes/Sills

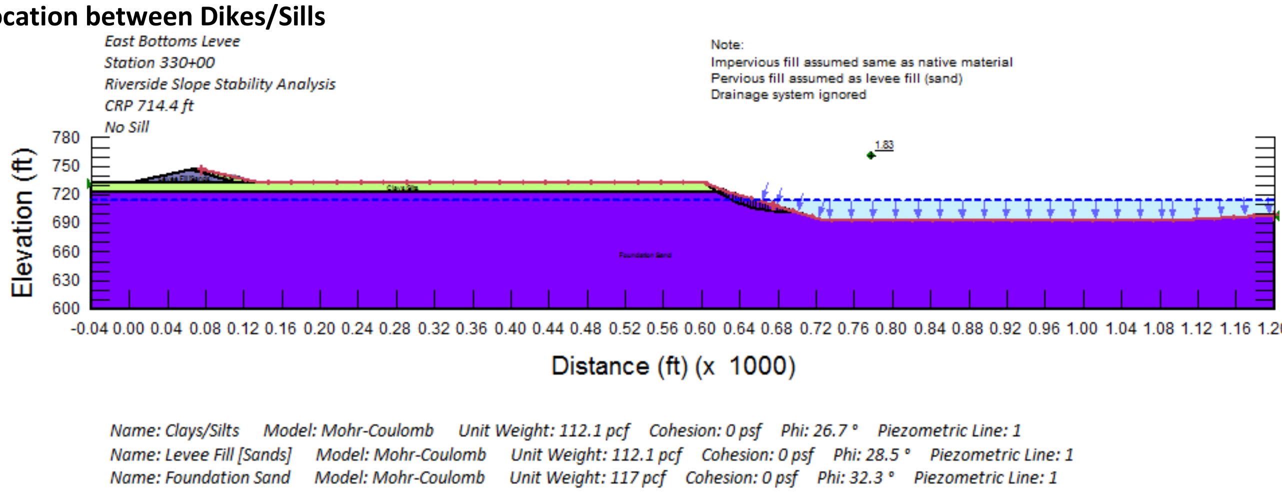
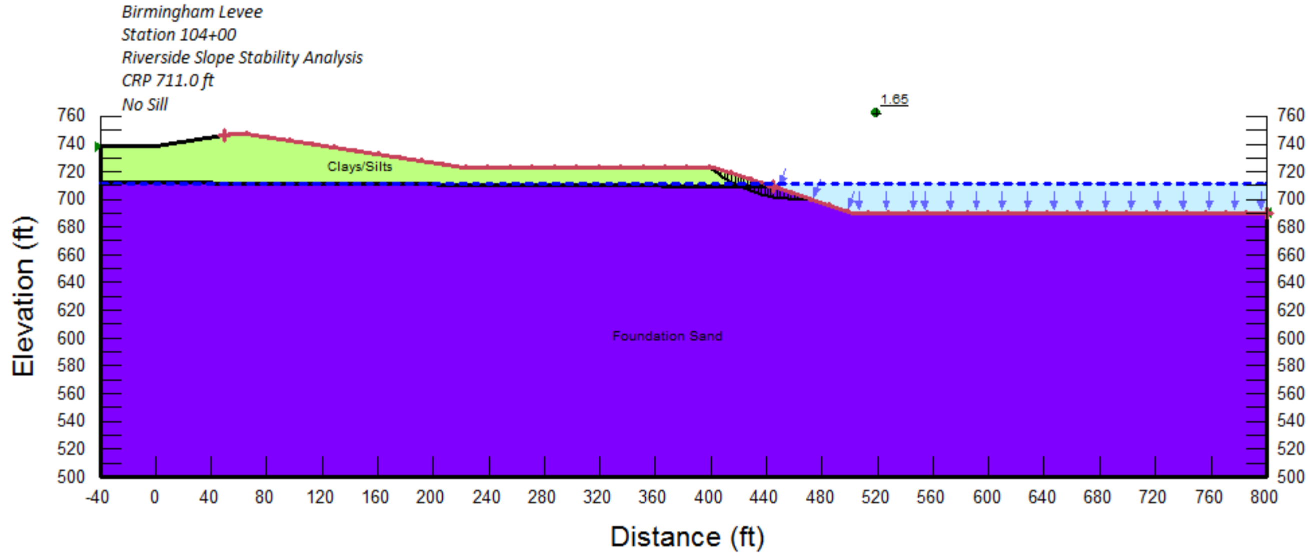


Figure 2.6.4-4: Birmingham Levee Station 104+00 Stability under Future with Project Conditions at Location between Dikes/Sills



Name: Clays/Silts Model: Mohr-Coulomb Unit Weight: 112.1 pcf Cohesion: 0 psf Phi: 26.7 ° Piezometric Line: 1
Name: Foundation Sand Model: Mohr-Coulomb Unit Weight: 117 pcf Cohesion: 0 psf Phi: 32.3 ° Piezometric Line: 1