GENERAL RE-EVALUATION REPORT (GRR)
MIDDLE RIO GRANDE – BERNALILLO TO BELEN LEVEES

APPENDIX F – GEOTECHNICAL ENGINEERING

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1 INTRODUCTION

This appendix presents the results of geotechnical analyses and feasibility level geotechnical recommendations to address levee height, geometry, erosion, access, vegetation, seepage, and slope stability within the project area. This geotechnical engineering evaluation included the following tasks:

- review available geology, geomorphology, and geotechnical information;
- review of construction history and past performance of existing spoil-bank;
- assess proposed levee design;
- seepage analysis of proposed levee design;
- stability analysis of proposed levee design;
- develop geotechnical conclusions and recommendations.

Enclosure E-1 at the end of this geotechnical engineering appendix is a calculation package containing details of the analyses, including a preliminary assessment of seismic load. Numerical modeling was done using a commercially available software package (GeoStudio 2012) containing the programs SEEP/W for seepage and SLOPE/W for slope stability.

1.1 Project Description

Current feasibility-level planning included preliminary designs for approximately 48 miles of levee, extending along both banks of the Rio Grande, as described below. An existing spoil bank is present for almost the entire length of the project. The proposed levee would replace the existing spoil-bank within its current alignment.

1.2 Reach Identification

The project has been divided into four specific reaches, two on the east bank and two on the west bank (Figure F-1).

East side of Rio Grande:

Mountain View Unit -- extends on the east side of the river from the south outlet of the Albuquerque South Diversion Channel to 3,000 feet downstream of the I-25 river crossing.

Belen East Unit -- extends from high ground upstream of NM 147 bridge crossing, to 3,700 feet downstream from the railroad bridge at Belen.

West side of Rio Grande:

Isleta West Unit -- extends from the Interstate 25 bridge and Atchison, Topeka and Santa Fe railroad bridge, to the NM 147 bridge approach at Isleta.

Belen West Unit -- extends from the railroad track immediately downstream of Isleta Marsh to approximately 7,000 feet downstream of the railroad bridge at Belen.

Mountain View Unit TSP Plan	
East bank 4.4 miles (red)	
Isleta West Unit TSP Plan	
West Bank 3.2 miles (yellow)	
Belen East Unit TSP Plan	
East bank 18.1 miles (purple)	
Belen West Unit TSP Plan	
West Bank 22.1 miles (pink)	

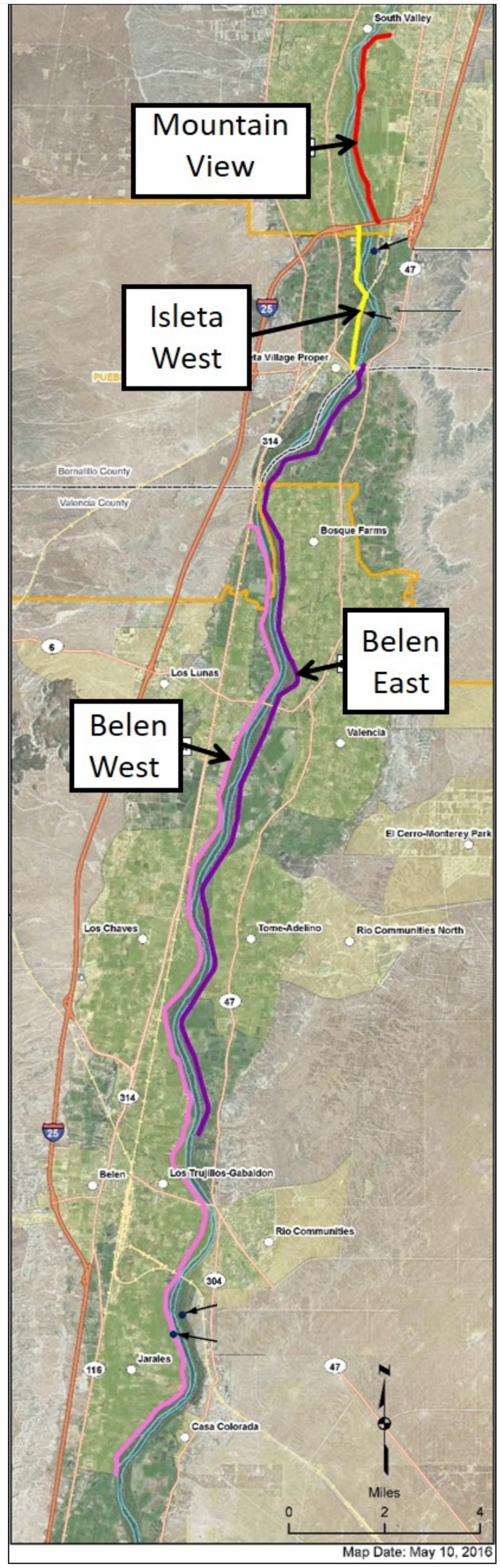

Figure F-1. Map showing approximate levee reach locations (see colors, above). Green area is agricultural land and riparian habitat which corresponds to the inner Rio Grande valley (Section 2.2).

2 GEOLOGY

2.1 Geologic Setting

The project area is located in the Rio Grande rift, a broad physiographic and structural depression composed of a series of north-trending, elongate topographic and structural basins extending from southern Colorado to northern Mexico. The structural basins are arranged in a right stepping, en echelon pattern, and are characterized by abundant late Quaternary faults, Quaternary volcanism, and thick accumulations of basin sediment fill (Morgan et al. 1986).

The project area lies within the Albuquerque Basin, the largest of the Rio Grande rift basins. The eastern margin of the Albuquerque Basin is bordered by active and potentially active faults adjacent to the Sandia, Manzanita, and Manzano mountain uplifts. These normal faults (e.g., Sandia/Rincon, Manzano, Hubbell Spring) have more than 10,000 ft of down-to-the-west vertical separation, and have exposed Proterozoic rocks in the footwall uplifts (Hitchcock and Kelson 2007). The Albuquerque Basin is bordered to the west by the Albuquerque Volcanoes and on the north and northeast by the west-tilted Española Basin and late rift–stage volcanic fields. Basins with generally west-tilted half-graben segments lie immediately to the south, such as the Socorro Basin (Grauch and Connell 2013).

Syn-rift sediments, known as the Santa Fe Group (Spiegel and Baldwin 1963), were deposited in the Albuquerque Basin during latest Oligocene through early Pleistocene time. Sediment thickness in boreholes varies from ~3280–6560 ft along basin margins to more than 14,000 ft in basin centers (Lozinsky 1994; May and Russell 1994). Sediments of the Santa Fe Group include alluvial, eolian, fluvio-lacustrine, and volcaniclastic detritus that were deposited within internally drained basins (Chapin and Cather 1994).

After local tilting and erosion, the ancestral Rio Grande became organized as a through-going drainage system, depositing fluvial sediments starting by early Pliocene time (Connell 2004). Late rift–stage volcanic activity occurred in several isolated centers within the basin, mostly along linear vent alignments (Maldonado et al. 2007).

Contemporaneous sediment accumulation was less than during the Miocene phase of basin subsidence. Consequently, the Plio-Pleistocene section represents only a fraction of the total volume of rift fill within this basin. This younger basin fill buries much of the older syn-rift deposits, making interpretation of the older history of rifting ambiguous in many places (Grauch and Connell 2013).

2.2 Site Geology

Within the Rio Grande rift, the Rio Grande River flows from north to south transporting sediments from northern New Mexico and southern Colorado. Geologic mapping by Connell (1997, 1998a, 1998b; Connell et al. 1995) shows that this alluvium is inset into Pleistocene alluvium, alluvial-fan deposits, and Tertiary bedrock that comprise the adjacent piedmont slopes. Holocene and alluvial-fan deposits derived from arroyos draining the piedmonts west and east of the inner valley interfinger with Rio Grande fluvial deposits (Hitchcock and Kelson 2007). The inset trough of Holocene (Recent) alluvium is known as the inner Rio Grande valley.

The project site is located within the inner Rio Grande valley (Figure F-1). The inner Rio Grande valley is underlain primarily by saturated, unconsolidated sandy alluvium deposited by the Rio Grande and tributary arroyos. This alluvium consists predominantly of sand and gravel with discontinuous interbeds of silt and clay (Kelson et al. 1999). Groundwater in the inner valley is very shallow, with depths beneath most of the valley of less than 40 ft (Hitchcock and Kelson 2007).

Typical alluvial deposits in the project site are variable and discontinuous. Foundation materials along the proposed levee alignment are generally sands, silty sands, and sandy clays. These soils are generally suitable as foundation material provided locations where low-density materials have been identified receive adequate preparation. Weak clay layers composed of high-plasticity clay are also locally present.

2.3 Faulting and Seismicity

The Rio Grande rift region in north-central New Mexico contains numerous late Quaternary faults, demonstrating that there is a real potential for significant strong ground motion in the Albuquerque region (Wong et al. 2004). Paleoseismic studies of major faults in the region suggest that, although infrequent, several major faults in the Albuquerque area have experienced large earthquakes in the late Holocene (Machette et al. 1998; Personius et al. 1999, 2001). These data provide direct evidence for the occurrence of large earthquakes of magnitude (M) 7 or greater in the Albuquerque area, despite the scarcity of moderate and large historical earthquakes (Hitchcock and Kelson 2007). A map of known, suspected, and inferred potentially seismic faults is provided in the figure below.

Geologic evidence of large earthquakes near Albuquerque suggest that peak ground accelerations (PGA) in the middle Rio Grande Valley were sufficient to trigger liquefaction in highly susceptible sediments, although no instances of liquefaction or paleo-liquefaction have been reported in the literature (Hitchcock and Kelson 2007).

Liquefaction is the liquefying of certain sediments during seismic ground-shaking, resulting in temporary loss of support to overlying sediments and structures. Poorly consolidated, water saturated fine sands located within 30 to 50 feet of the surface typically are considered the most susceptible to liquefaction. Dry soils and sediments consisting of finer grained materials are generally not susceptible to liquefaction.

The consequences of a large earthquake in the vicinity of the project area would be significant because of the high likelihood of liquefaction-related ground failures in the inner Rio Grande valley, where the project area is located. The inner Rio Grande valley is underlain by sediments with high or very high susceptibility to liquefaction (Kelson et al. 1999) and it is reasonable to assume that liquefaction-related damage could result from a moderate to large earthquake on any of several nearby late Quaternary faults. Along with damage to buildings, vital bridges, and other infrastructure, liquefaction-related failure of river levees in the inner Rio Grande valley may cause localized flooding (Hitchcock and Kelson 2007).

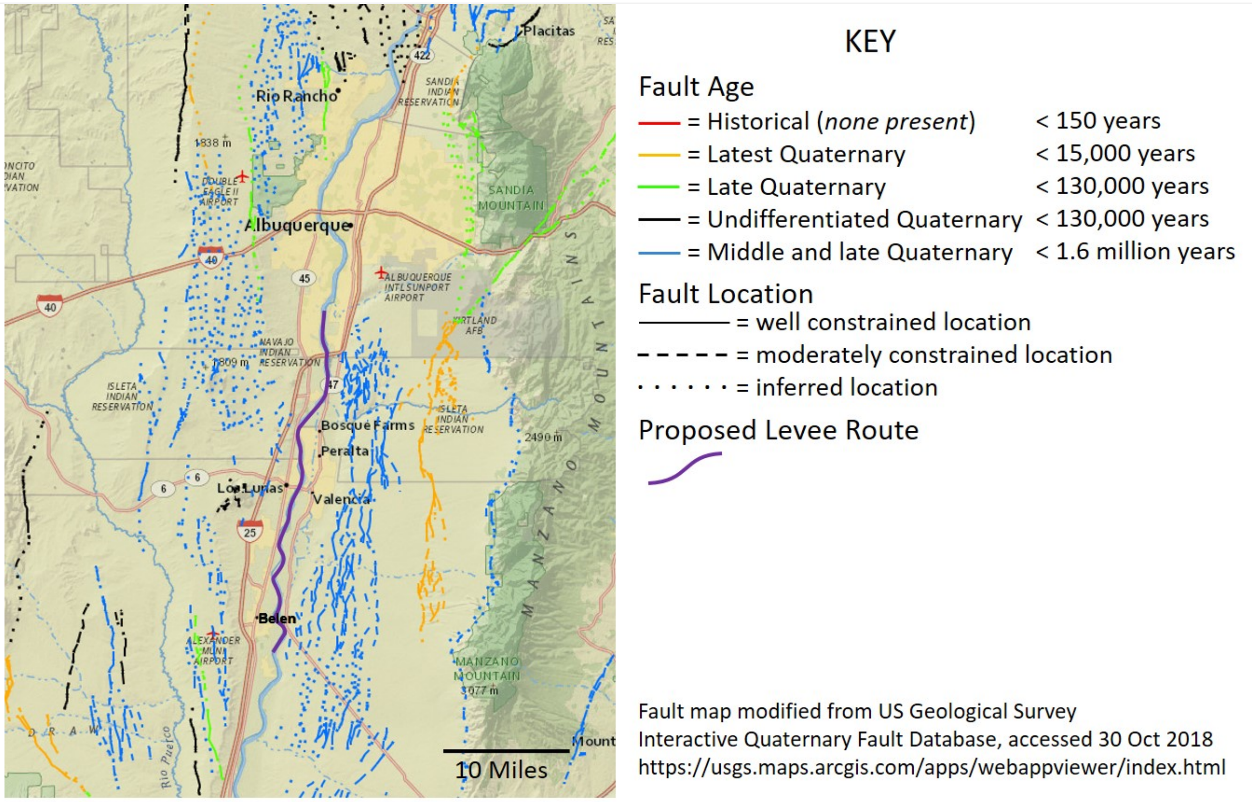

Figure F-2. Fault map of the Albuquerque basin.

2.3.1 Liquefaction and Ground Deformation

Liquefaction is the tendency for a loosely packed saturated sand or silt to behave as a liquid due in response to increased pore water pressure. This increase in pore pressure reduces the shear strength of the soil and is typically caused by earthquake ground motion. As the pore water pressure increases the soil particles lose contact with each other and the soil structure loses strength resulting in a range of foundation issues including loss of bearing strength and differential settlement.

Three factors are required for liquefaction to occur: loose granular soils, saturation of the soil, and strong shaking of the soil mass. In general, saturated poorly graded soils composed of uniformly sized particles are more susceptible to greater settlement subsequent to liquefaction than well graded soils because of the gap volume between soil particles within the soil structure. After ground motion has ceased the particles in a poorly graded soil tend to have been rearranged into a denser configuration which is observed as settlement within the soil mass.

The existing spoil-bank in the project area is constructed over alluvial deposits consisting of both well graded and poorly graded sands and silts that may be susceptible to liquefaction during a seismic event. The proposed levees will be infrequently saturated and are not expected to liquefy during a seismic event. However, liquefaction in foundation soils may lead to excessive settlement or failure of the spoil-bank slopes.

Differential settlement occurs when the layers that liquefy are not of uniform thickness, a common problem when the liquefaction occurs in discontinuous fluvial sediments or artificial fills. Liquefaction-induced failure of the spoil-bank slope may occur due to development of excess pore pressures at the toe of the spoil-bank slope and loss of foundation support due to lateral spreading toward sides of natural or man-made channels adjacent to the spoil-bank. Localized ground settlement likely will be controlled by the subsurface distribution of river sands within abandoned river channels adjacent to the current Rio Grande.

Earthquake-induced liquefaction can occur over widespread areas during long-duration, strong ground shaking with a PGA equal to or greater than 0.15 g (Tinsley et al. 1985), which may be produced by large-magnitude earthquakes (M 6.5). Within central New Mexico, there are several potential seismic sources that are capable of producing a PGA greater than 0.15 g in the greater Albuquerque area (Hitchcock and Kelson 2007). These include the Sandia-Rincon fault along the western margin of the Sandia Mountains, the West Mesa fault zone northwest of the project site, and the Hubbell Spring fault bordering the Manzanita and Manzano Mountains east of the project site (Connell et al. 1995; Machette et al. 1998; Wong et al. 2004). The greatest liquefaction hazard is localized within the narrow, inner Rio Grande valley, adjacent to the river (Hitchcock and Kelson 2007).

2.3.2 Ground Shaking

Earthquake ground shaking varies from place to place and hazard maps are available to estimate the potential ground shaking. The mapped hazard refers to an estimate of the probability of exceeding a certain amount of ground shaking, or ground motion, in 50 years. The hazard depends on the magnitudes and locations of likely earthquakes, how often they occur, and the properties of the rocks and sediments that earthquake waves travel through (USGS 2017).

Most of New Mexico (including the project area) is subject to a Moderate Seismic Hazard (USACE 2016, Appendix C), and a standard seismic study must be performed to properly assess the response of the foundation and structures to the earthquake events possible at the project site (USACE 2016).

A standard seismic study is based on existing, generic seismological studies, available site data, and simplified methods of evaluation including published sources and on-line United States Geological Survey (USGS) probabilistic ground motion maps. A preliminary structural analysis and simplified assessment of soil liquefaction and deformation is required to assess how seismic loads impact the design. The standard seismic study is also used to set the scope of possible site-specific seismic studies (USACE 2016). An analysis of the effect of the estimated potential ground shaking on the proposed levee design is included in this engineering appendix as Enclosure 1.

2.3.3 Peak Horizontal Ground Acceleration

Peak Ground Accelerations at critical project locations over a range of annual exceedance probabilities were estimated using the USGS Unified Hazard Tool (UHT) along with response spectrums using threshold spectral periods of 0.2s and 1.0s. Three specific locations near the proposed levee alignments were chosen for analysis: Isleta Village Proper, Main St. near central Los Lunas, and south of East River Rd. in Belen. The Operating Basis Earthquake (OBE) and Maximum Design Earthquake (MDE) as events with a 50% probability of exceedance in 100 years (average return period of 144 years) and 10% probability of exceedance in 100 years (average return period of 950 years) in accordance with ER 1110-2-1806. The PGA at each location for three separate return periods including the MDE are tabularized below.

The UHT utilizes a class B seismic site classification by default which is valid for soil profiles consisting primarily of rock. Available soil boring data is not adequate to accurately categorize the seismic site class of each alignment however enough data is available to provide a basis for assumption. Available Standard Penetration Test (SPT) blowcount data suggests that each site can be classified as either Class D (stiff soil) or Class E (soft soil). Due to the uncertainty and lack of data Site Class E was selected for the entirety of each project. In order to correlate the Class B PGA calculated using the UHT to Class E PGA a site classification transform coefficient was used in accordance with Table 11.8-1 from ASCE Standard 10-7. The Class B PGA and correlated Class E PGA are presented in the tables below.

Table F-1. Peak Horizontal Ground Acceleration Coordinates

PGA Site	Coordinates (lat./long.)
Isleta Village	34.911 / -106.687
Los Lunas	34.806 / -106.723
Belen	34.646 / -106.743

Table F-2. USGS 2014 Peak Horizontal Ground Accelerations – Isleta Village.

Probability of Exceedance	PGA (g) Site Class B	Site Class E Coefficient	PGA (g) Site Class E
50% in 100 years (OBE) (return period of 144 years)	**0.026**	**2.5**	**0.065**
10% in 50 years (return period of 475 years)	0.06	2.5	0.15
10% in 100 years (MDE) (return period of 950 years)	**0.10**	**2.5**	**0.25**
2% in 50 years (return period of 2,475 years)	0.18	1.7	0.31

Table F-3. USGS 2014 Peak Horizontal Ground Accelerations – Los Lunas.

Probability of Exceedance	PGA (g) Site Class B	Site Class E Coefficient	PGA (g) Site Class E
50% in 100 years (OBE) (return period of 144 years)	**0.027**	**2.5**	**0.068**
10% in 50 years (return period of 475 years)	0.07	2.5	0.15
10% in 100 years (MDE) (return period of 950 years)	**0.10**	**2.5**	**0.25**
2% in 50 years (return period of 2,475 years)	0.18	1.7	0.31

Table F-4. USGS 2014 Peak Horizontal Ground Accelerations – Belen.

Probability of Exceedance	PGA (g) Site Class B	Site Class E Coefficient	PGA (g) Site Class E
50% in 100 years (OBE) (return period of 144 years)	**0.028**	**2.5**	**0.07**
10% in 50 years (return period of 475 years)	0.07	2.5	0.15
10% in 100 years (MDE) (return period of 950 years)	**0.11**	**2.5**	**0.28**
2% in 50 years (return period of 2,475 years)	0.19	1.7	0.32

In addition to the MDE and generic PGA values available from the USGS (2014), ground accelerations for the project area were estimated as part of a larger effort to prepare ground-shaking hazard maps for the Belen to Santa Fe corridor (Wong et al. 2004). These maps suggest higher peak horizontal ground accelerations than the generic USGS ground motion maps. In particular, the map for the 2% probability of exceedance in 50 years (Wong el al. 2004, Figure 13) shows the northern one-quarter of the project area as adjacent to a region of very high potential ground acceleration (0.4 – 0.5 g). Wong et al. (2004) explain that in their model, wave attenuation by loose sediments in the inner Rio Grande valley result in a lower ground acceleration along the river.

Wong et al. (2004) also analyze the effect of a M 7.0 earthquake on the Rincon-Sandia fault east and northeast of Albuquerque and estimate ground accelerations in the northern half of the project area to range from 0.3 g to 0.6 g. The most recent movement on the Rincon-Sandia fault is Holocene to latest-Pleistocene in age (i.e. <15,000 years ago) (Connell et al. 1995).

2.3.4 Results of Preliminary Seismic Evaluation

The GeoStudio numerical modeling software to evaluate the levee slope stability under an earthquake load (Enclosure E-1). Based on PGA values developed for the project vicinity (Table F-3 to F-5), horizontal, pseudo-static earthquake loadings of 0.28g, correlating to the MDE, and 0.07g, corresponding to the OBE were used to simulate earthquake conditions.

In accordance with ER 1110-2-1806 the MDE is defined as the maximum level of ground motion for which the levee sections are to be evaluated. The associated performance requirement is that the proposed levee cross sections withstand the MDE ground motions without loss of life or catastrophic failure. The OBE is an earthquake that can reasonably be expected to occur within the service life of the levee embankment and the associated performance requirement is that when subjected to the OBE ground motion the proposed levee cross sections function with little or no damage and without interruption of function. To determine the performance of the proposed levee systems under seismic loading pseudo-static analyses for both the MDE and the OBE ground motions were performed for both the 12 foot and 15 foot levee cross sections.

When subjected to the MDE ground motion, results show that under a 0.28g pseudo-static load the levee would experience moderate to significant sloughing as reflected in the calculated slope stability factor of safety for the embankment. This analysis was performed with no hydraulic load on the levee as it is highly improbable that a flood event would occur coincident with the MDE. Because it is extremely unlikely for the levee to be significantly loaded coincident to the MDE it can be determined that that subjecting the levee to MDE ground motions, although causing significant damage to the levee itself, would not lead to loss of life or catastrophic failure of the system.

For the ground motion associated with the OBE the model that was developed assumed that the toe of the levee was loaded with three feet of water, correlating to a bank-full river stage, and allowed to develop a steady-state seepage condition prior to being subjected to ground motion. Although it is improbable that any steady-state seepage condition could develop and the bank-full river stage and the OBE would occur coincident with each other, this model was designed to conservatively determine the performance of the levee embankment in response to the OBE. Results show that under a 0.7g pseudo-static load the levee embankment would remain intact as indicated by the factor of safety for slope failure remaining greater than one.

The analysis for the 12-foot-high and 15-foot-high levee cross sections show both meet required performances levels in accordance with the seismic design requirements for the MDE and the OBE provided in ER 1110-2-1806. The results of this analysis are provided at the end of this report in Enclosure E1.

A liquefaction analyses could not be performed on the foundation beneath the proposed levees due to a lack of boring data to a sufficient depth. To compensate, both a comparative analysis and an economic analysis were performed to aid in informing the risk associated with potential earthquake loadings.

The comparative analysis utilized information available from an Urban Levee program study developed to aid in assessing levee seismic vulnerability in areas throughout California (Shewbridge, et. al.). The model foundation was composed of silty sand which correlates well with the sandy alluvial deposits beneath the proposed Bernalillo to Belen levee system. For a specified PGA of 0.17g the levee model was determined to be compromised and unable to provide post-earthquake flood protection. The MDE PGA for the Bernalillo to Belen levee project is 0.28g, 65% greater than the PGA used in the Urban Levee program analysis, strongly suggesting that the proposed levee systems would also be compromised and unable to provide flood protection in the event that the MDE were to occur.

With the expectation that the project will not provide flood protection until repaired after the occurrence of the MDE, an economic evaluation was performed to determine appropriate mitigation measures. This evaluation considered two scenarios: the probability that the design flood and MDE were to occur simultaneously and the conditional probability of the design flood occurring subsequent to the MDE. The assumption was made for both cases that in the event the MDE occurred, damage to each levee would introduce significant discontinuities in each alignment to the point that it was conservatively assumed that the protected areas would no longer be protected and would return to the "without-project" condition. The probability and consequence of flooding is captured in the EAD computation, which for the study area is $105,369,800 (May, 2016 prices). The likelihood of a seismic event could be independent of the likelihood of flooding, which makes them (statistically) independent events. Therefore, in any given year:

Prob(Earthquake) AND Prob(Flood) = Prob(Earthquake) x Prob(Flood)

Prob(Flood) = $105,369,800 (Table D-20, Economic Appendix, Equivalent Annual Damages)

Prob(Earthquake) = 0.0010526 = 1/950 (MDE)

Prob(Flood) x Prob(Earthquake) = $105,369,800 x 0.0010526 = $110,900

The $110,900 is the probability-adjusted damages on an equivalent annual basis that the flood regime occurs AND an earthquake occurs. That figure would justify roughly $3 million in seismic protection at the current discount rate of 2.75% or $1.5 million at 7%. Spending more than the indicated amounts on mitigation features on the proposed levee would be infeasible. The proposed levees extend for 40 miles. Spending $3 million on seismic protection will not address the unlikely event.

The evaluation of “earthquake then flood” impacts to the proposed levee can be addressed qualitatively, given the first evaluation. The computation of two independent events represents an upper limit on justifiable construction costs. The more likely value of earthquake mitigation features is much smaller for a couple reasons. First, the earthquake MUST happen before the flood to provide the damages indicated, which is a vanishingly small chance of occurrence. Any earthquake which damages the proposed levee would most assuredly damage the properties protected by that levee, meaning the subsequent flood would have less available property subject to flood damages. Next, the flood must occur within a period of time that the proposed levee has already been damaged by the MDE, but is not yet completely repaired. There are also reasonable assumptions that earthquake damaged properties in the study area are not replaced in the same time frame the levee is repaired, if at all, which would reduce the floodplain inventory subject to the subsequent flood.

2.3.5 Nonphysical Nonstructural Seismic Mitigation

In the event that an earthquake were to damage the proposed levee system and reduce its ability to mitigate flood risk for protected areas the most economical course of action would be to implement a system of nonphysical nonstructural mitigation measures. Nonphysical nonstructural mitigation measures are contingent measures that focus on reducing the consequences of flooding instead of focusing on reducing the probability of flooding. Such measures would include flood warning systems, floodplain mapping, flood emergency preparedness plans, evacuation plans, and risk communication.

3 SITE EXPLORATION

3.1 Previous Work

Subsurface investigations were conducted on the existing spoil banks to determine the condition and composition of the spoil-bank, and suitability of the foundation soils for construction of the new levees. Subsurface investigations were conducted in 1984 and 1985 as part of the General Design Memorandum (GDM) (USACE 1986).

A total of 362 borings were drilled using an eight-inch diameter hollow stem auger. The borings were spaced between 1250-ft and 1500-ft apart along the length of the existing spoil-bank and were advanced to a maximum depth of 40 feet below ground surface. Standard Penetration Tests (SPT) were taken at 5-foot intervals and disturbed samples obtained for sieve analysis, Atterberg limits, and moisture content. SPT soil samples were classified and logged in the field prior to lab testing.

Trenches were excavated in native soils outside of the footprint of the spoil-bank to examine foundation soils and to look for potential sources of borrow material for future levee construction. Materials were visually classified and samples obtained for sieve analysis, Atterberg limits, and moisture content. Excavation was performed using a backhoe.

The elevation of groundwater in the borings and trenches was generally coincident with the elevation of the river. Results of this investigation were presented in Volume II of the GDM (USACE 1986).

Additional subsurface investigations were conducted in 1990 and 1991 for the Feature Design

Memorandum (FDM). The FDM investigations were intended to support the design of tie-back levees and low-flow channel control structures located at the upper and lower end of the project, and also to help locate potential off-site borrow sources. Drilling logs and lab test data for the FDM exploration are included in Appendix C of Volume I of the FDM.

Finally, supplementary drilling was performed in 2009, 2010, and 2011 to comply with Technical Letter 111O-2-569 "Design Guidance for Levee Underseepage" (USACE 2005).

Borings were generally spaced at intervals of approximately 1000-ft to a depth of 30-ft along the crest of existing spoil bank. Where access allowed, two additional borings to 15 feet depth were drilled at the landside and riverside toe of the spoil bank. Borings were typically sampled at 2.5-foot intervals starting from the ground surface by use of a SPT split barrel sampler. SPT soil samples were described and classified in the field.

A total of 1359 samples were taken of the spoil bank within a depth of 0 to 10 feet for the five reaches combined. Index soil property tests including laboratory classification of soils, sieve analysis, Atterberg limits, and moisture content were performed on each sample.

Boring logs, provided electronically, and the lab test results are not included as part of this appendix but are available upon request from the USACE Albuquerque District.

3.2 Previous Investigations by Reach

3.2.1 Mountain View Unit

The Mountain View Unit is located at the north end of the project area on the east side of the Rio Grande.

In December 2008 and October-November 2009, sixty-eight (68) additional borings were drilled along the existing spoil bank alignment to characterize the existing spoil bank and foundation materials. As stated above, the borings were generally spaced at 1000-foot intervals and where access allowed, one boring was drilled at or near the riverside toe, the landside toe, and on the crest of spoil bank at each boring location. At each location, borings were sampled with a Standard Penetration Test (SPT) split barrel sampler at 2.5-foot intervals to depths of 15 feet at the landside and riverside toes of the spoil bank and to depths of 30 feet on the crest of the spoil bank.

Materials sampled were visually classified and logged. Soils were classified according to the Unified Soil Classification System (USCS). Spoil bank materials were generally classified as silty sands (SM) and silt (ML). Foundation materials were generally classified as silt (SM), sandy clay (CL), fine sand (SP), and poorly graded sand with silt (SP-SM). Visual inspection of this area was performed to supplement the borings.

The inspection, which looked at the existing low-flow channel and river bank, indicates that foundation materials consist of alluvial materials, which in this area are predominantly fine silty sands (SM) and poorly graded sand (SP) with traces of silts, clays, and gravels. Spoil bank and foundation soils are typically very loose to medium dense with field (uncorrected) blow counts ranging from a low of 2 to a high of 29 per foot of penetration. The majority of the materials are very loose to loose. Low SPT blow counts recorded below the water table are suspect for indicating the relative density due to the heaving nature of the sands below the water table.

Overall, laboratory classification tests indicate that about one-third of the spoil-bank soils have less than the design target of 20 percent fine-grained materials. The average percent passing the No. 200 sieve (based on 119 samples) was 33 percent, suggesting that some mixing of spoil-bank soils may be necessary before excavated material is re-used as engineered fill for the proposed new levee.

3.2.2 Isleta West Unit

The Isleta West Unit is located at the north end of the project area on the west side of the Rio Grande.

As part of the supplemental drilling performed in March 2010, 31 additional borings were drilled along the spoil bank alignment for Isleta West Unit 1, and 15 borings were drilled for Isleta West Unit 2.

The existing spoil banks are comprised of silty sands (SM), silt (ML), poorly graded sands with silt (SP-SM), and poorly graded sands (SP). A few layers of clayey sands (SC) and clay with silt (CL-ML) were found. Spoil bank materials range from very loose to medium dense with SPT blow counts ranging from 3 to 11 blows per foot of penetration. However, the majority of the spoil bank materials are very loose to loose.

Overall, laboratory classification tests indicate that about one-third to one-half of the spoil-bank soils have less than the design target of 20 percent fine-grained materials (passing the No. 200 sieve). These results indicate that some mixing of spoil-bank soils may be necessary before excavated material is re-used as engineered fill for the proposed new levee.

Foundation materials are generally poorly-graded sands (SP) and silty sands (SM). Based on SPT results, foundation soils range from very loose to dense with SPT blow counts ranging from 2 to 35 blows per foot of penetration. Low SPT blow counts recorded below the water table are suspect for indicating the relative density due to the heaving nature of the sands below the water table.

Soft layers of low plasticity clay (CL) are present at foundation depth in some areas. In general, clay layers are randomly located, relatively thin, and have sand layers above and below which will allow any excess pore pressures to dissipate. This will lead to consolidation and increased strength upon construction of the levee. The existing layers have also been pre-consolidated by the existing levee and therefore only the weight required to provide any additional height of levee will contribute to settlement. If soft clays are encountered at foundation depth during construction of the proposed levee the clay will need be excavated and removed.

3.2.3 Belen East Unit

The Belen East Unit extends from the central project area to south end on the east side of the Rio Grande.

As part of the supplemental drilling performed in January and February 2010, two hundred fourteen (214) additional borings were drilled along the spoil bank alignment. The existing spoil banks are constructed of silty sand (SM), poorly graded sand with silt (SP-SM), poorly graded sand (SP), sandy silt (ML), and sandy clay (CL).

Existing spoil banks range from approximately 2 to 22 feet in height, have a crest width that varies from approximately 12 to 25 feet, and side slopes which are generally 2H:1V (horizontal to vertical) or flatter on the landslide and somewhat steeper on the riverside. SPT results show the density of spoil bank materials range from very loose to medium dense with SPT field blow counts ranging from 1 to 20.

Overall, laboratory classification tests indicate that about one-quarter of the spoil-bank soils have less than the design target of 20 percent fine-grained materials. The average percent passing the No. 200 sieve (based on 598 samples) was 37 percent, suggesting that some mixing of spoil-bank soils may be necessary before excavated material is re-used as engineered fill for the proposed new levee.

Foundation materials are generally poorly graded sands (SP), silty sands (SM), poorly graded sands with silt (SP-SM), low to medium plasticity clays (CL), poorly graded sands with gravel (SP), and poorly graded gravels (GP, GP-GM). Foundation soils are very loose to dense with SPT field blow counts ranging from 2 to 45 blows per foot of penetration. The low SPT blow counts recorded for the sands and silty sands below the water table are suspect for indicating the relative density due to the heaving nature of the sands below the water table. Higher SPT blow counts greater than 30 blows per foot of penetration were recorded at depths of 25 feet to 30 feet for the poorly graded gravels.

3.2.4 Belen West Unit

The Belen West Unit extends from the central project area to south end on the West side of the Rio Grande.

As part of the supplemental drilling performed between December 2009 and March 2010, two hundred seven (207) additional borings were drilled along the spoil bank alignment. The existing spoil banks were constructed of sandy silt (ML), silty sand (SM, SP-SM), poorly graded sand (SP), sandy clay (CL), and silty, sandy clay (CL-ML).

Existing spoil banks vary from approximately 3 to 17 feet in height, have a variable crest width of approximately 12 to 20 feet, with side slopes which are generally 2H:1V or flatter on the landslide and somewhat steeper on the riverside. The majority of the existing spoil bank materials ranged from very loose to medium dense with SPT blow counts ranging from 1 to 20.

Overall, laboratory classification tests indicate that about one-quarter of the spoil-bank soils have less than the design target of 20 percent fine-grained materials. The average percent passing the No. 200 sieve (based on 601 samples) was 41 percent, suggesting that some mixing of spoil-bank soils may be necessary before excavated material is re-used as engineered fill for the proposed new levee.

Foundation materials are generally poorly graded sands (SP), silty sands (SM), poorly graded sands with silt (SP-SM), sandy clays (CL), clayey sands (SC), poorly graded sands with gravel (SP), and poorly graded gravels (GP, GP-GM). Foundation soils are very loose to very dense with SPT blow counts ranging from 2 blows per foot of penetration to refusal (i.e. 50 blows per 5 in. or less of penetration). The low SPT blow counts recorded for the sands and silty sands below the water table are suspect for indicating the relative density due to the heaving nature of the sands below the water table. Higher SPT blow counts greater than 30 blows per foot of penetration were recorded at depths of 25 ft to 30 ft for the poorly graded sands with gravels (SP) and well graded gravels with silt and sand (GW-GM).

3.3 Future Exploration Work

Additional subsurface investigations, including field and laboratory tests, will be required during the Preliminary Engineering Design (PED) stage. The development of levee plans and specifications for each reach of this project is needed to verify estimated shear strengths and permeability values used in the models for slope stability and seepage analyses. Modeling results should be revised accordingly.

In addition, new subsurface investigation will be needed during PED to support detailed seismic hazard analysis. A detailed seismic hazard analysis may be required although the results of the preliminary seismic evaluation do not indicate that earthquake loads govern levee stability. The study could be used to potentially inform the risk for areas where seismically induced liquefaction and subsequent damage to the levee might occur.

4 EXISITING SPOIL-BANK

Existing spoil-banks in the project area are not engineered structures. Material for the construction of the spoil-banks was obtained from excavation of the “Riverside Channel”, an agricultural canal used to collect irrigation runoff from farmlands on the landward side of the spoil-bank. Thus, the Riverside Channel flows on the landward side of the spoil-bank.

Soils used in the construction of the spoil-bank are grossly representative of the local foundation soils and suitable materials from the existing spoil bank can be excavated and reused for construction of the proposed levee.

Spoil-banks are typically in close proximity to the Riverside Channel. Channel embankment materials are generally poorly-graded sands, silty sands, silts, and (less commonly), low plasticity clays. Soil densities are typically moderately low.

Slopes along the sides of the channel are commonly in a state of sloughing, and many spoil-banks are known to be in poor condition regarding vegetation and erosion.

For much of the year water surface elevation of the Rio Grande is below the toe of most spoil-banks, however it does encroach on some spoil-banks during seasonal high flows.

4.1 Seepage and Stability Analysis

A seepage and stability analysis of the existing spoil-banks was conducted in 2010 as part of an environmental study (USACE 2010). In an effort to increase habitat restoration along the Middle Rio Grande valley a proposal was considered to increase the area inundated by the Rio Grande below Cochiti Dam. The seepage and stability analysis was conducted to estimate the response of the spoil-bank system to the proposed increases in flow.

Modeling analysis of the spoil-banks was conducted using assumed values for soil properties rather than actual values determined from field exploration. Also, the study considered controlled water releases from Cochiti Dam rather than flooding conditions within the Middle Rio Grande valley. Nonetheless, the modeling provides important insight into the potential behavior of spoil bank system during high-flow events similar to flood conditions.

The stability analysis concluded that "the global [slope stability] failure mode shows high probability of undesirable performance for flows above 6,000 cfs" (USACE 2010, pg. 19). The seepage and stability study recommended that, to limit the risk of global stability failure of the existing spoil bank system to 1%, "the combined river discharge not exceed 6,000 cfs north (upstream) of Albuquerque" (USACE 2010, pg. 25).

Observations of the performance of the existing spoil-bank system, as well as the results of numerical modeling of spoil-bank seepage and stability, indicate that the existing spoil-bank system may provide protection to the floodplain on both sides of the river from flood events of a magnitude of less than the 20% ACE for short duration flow events. Contrast this with the proposed replacement levee system which is expected to provide protection from flood events up to 0.5-0.2% ACE.

5 PROPOSED LEVEE DESIGN

The area available for the construction of the proposed levees is restricted because of environmental and economic considerations. There are areas adjacent to the Rio Grande which are environmentally sensitive and disturbance of these areas must be kept to a minimum. In other areas, the cost of relocation of the Riverside Channel and other structures would increase the cost of the project substantially. The proposed levee design was therefore limited to the minimum requirements for flood control plus the following assumptions:

- Levee construction materials will be obtained from the existing spoil-bank to the maximum degree possible;
- A filter/drainage blanket will be incorporated for approximately 30 ft of the landside toe footprint;
- A sub-drain system will run parallel to the levee along the landside toe (i.e. a toe drain);
- The planned 5-ft deep (minimum), 8-ft wide inspection trench with 1H:1V sides will be backfilled with engineered fill to facilitate a longer seepage path; and
- A minimum 20-ft wide maintenance/access road will be located between the levee toe and the top edge of the Riverside Channel.

Subsurface investigations indicate that foundation underseepage will be a greater concern than through embankment seepage. Seepage was observed daylighting near the waterline into the bank of the Riverside Channel closest to the spoil-bank. Water levels were low in the Rio Grande during this site visit. The invert of the channel is several feet below elevation of the proposed toe drain for the new levee suggesting the landside blanket drain and toe drain will not capture all of the foundation seepage.

Seepage control measures will be required for the proposed levees, and several control measures were considered during design. Impervious fill for the partial cut-off trench, sheet piles, or a slurry cut-off wall are not considered practical for this project because of the depth to which cut-off features would have to extend to provide effective seepage control.

Impervious riverside blankets were also considered as a means of lengthening the seepage path and reducing exit gradients at the landside toe of the new levee. However, the subsurface investigations suggest that sufficient quantities of fine-grained impervious or semi-impervious materials are not available within the project boundaries. The cost of importing large quantities of suitable material would likely make the cost of construction prohibitive. In addition, many large trees would have to be removed from the riverside of the levees in order to place the impervious riverside blanket, and the removal of large number of trees is not environmentally acceptable.

Impervious, semi-impervious, and free-draining landslide seepage berms were also considered. Because of the close proximity of the conveyance channel, a landslide seepage berm would not effectively control the anticipated seepage, and would possibly concentrate seepage flows to the conveyance channel slope, causing additional sloughing and instability problems. A pervious landside drainage blanket and a foundation toe drain were therefore considered the best method of controlling shallow under-seepage and protecting the area in the immediate vicinity of the landside levee toe.

An inspection trench, 5 ft deep, 8 ft wide with 1H:1V side slopes shall be provided along the centerline of the new levee for inspection of foundation materials during construction. When backfilled with engineered fill, the inspection trench will act as a partial cut off wall for foundation seepage, creating a longer seepage path and interrupting the lateral continuity of shallow, highly pervious layers.

Note that the inspection trench can be widened or deepened, as necessary, to excavate unsatisfactory foundation materials. The landside filter/drainage blanket, sub-drain system, and inspection trench are provided for embankment and foundation seepage control. The subdrain system should extend 5 feet into the foundation and include a sloped to drain 8-in. diameter slotted PVC pipe and discharge system embedded in a granular filter drain material, tied into the landside drainage blanket.

The new levee will be constructed of compacted random fill materials using satisfactory material obtained from the required excavation of the existing spoil bank. Spoil bank materials consist of generally silty sands (SM) and poorly graded sands (SP, SP-SM). Soft clay, oversized material, organic debris, and other unsuitable materials will be removed during excavation of the spoil-bank. During demolition and construction material will need to be excavated from the spoil-bank, then temporarily stockpiled while the new levee foundation is prepared, then replaced in the new levee in 8-in. lifts for compaction as engineered fill. It is anticipated that the handling of the spoil-bank material during spoil-bank excavation and construction of the new levee will be sufficient to mix coarser- and finer-fraction spoil-bank soils and yield a fill containing at least 20 percent fines content.

The new levee sections will required 15-ft wide vegetation free zone (VFZ) extending from the toe of the levee on both the landward and riverward sides. On the landside the 15-ft wide VFZ is part of the 20-ft wide (minimum) Maintenance Road.

Scour protection is required in specific reaches where the levee slope is exposed to possible erosion from the Rio Grande. Riprap to be placed by end-dump is proposed for this slope protection. In addition, articulated concrete block (ACB) and filter fabric will be required along areas of the Riverside Channel where the bank height is greater than 5 feet to protect the bank from sloughing due to foundation seepage.

5.1 Width of Levee Crest and Maintenance Road

A crest width of 15 ft was selected as an acceptable width for access to the top of the levee for construction and maintenance; this width also meets the minimum requirement of the Middle Rio Grande Conservancy District (MRGCD) who will be responsible for levee maintenance.

A minimum distance of 20 ft is required between the levee landside toe and the top edge of the Riverside Channel for slope stability and adequate length of seepage path. A maintenance road will be constructed between the levee landside toe and the edge of the channel.

Recommended levee slopes are 2.5H:1V for a levee height of 12 feet or less, and 3H:1V for a levee height of greater than 12 feet.

Due to the presence of pervious foundation soils, a 5-ft deep inspection trench is required for all levee reaches to expose or intercept undesirable subsurface features prior to construction of the new levee. The inspection trench will be backfilled with compacted random fill having greater than 20% passing the No. 200 sieve. The foundation excavation assures that weak materials are not present in the foundation. Backfilling with compacted random fill adds stability to the levee section and provides a potentially longer seepage path for foundation seepage control.

5.2 Embankment and Foundation Seepage

Embankment and foundation seepage is a major design concern. Proposed control measures include a 5-ft deep inspection trench, a landside drainage blanket, and a 2-ft wide by 5-ft deep toe drain with perforated PVC collector pipe.

A preliminary design of the pervious fill required for the landside drainage blanket and toe drain are provided herein. A filter cloth wrap for the drainage collector pipe will be required. A minimum 8-in. diameter pipe is specified because it can be easily cleaned and flushed, unlike smaller diameter pipes. The perforated toe drain pipe will collect seepage and discharge it to the Riverside Channel at 400-ft intervals to minimize disruption of the maintenance road and the proposed channel slope protection. The perforated pipe and the solid drainage pipe have been sized and spaced based on the flux (volume/day) of seepage collected at the pipe location, calculated in the Steady-State SEEP/W modeling results. The toe trench is located 5 ft inside the levee toe to provide approximately 2 ft of embankment fill above the trench. Embankment fill over the toe trench will allow some excess pressures to develop without endangering the toe. Locating the toe drain within the levee allows the use of risers to discharge the seepage should the drain pipe become blocked. The 400-ft interval between risers allows cleaning of the toe drain system, if required.

6 SEEPAGE ANALYSIS

The site stratigraphy and existing spoil bank soils vary along the various reaches of this project. To reduce the complexity of the model, soils for the different levee units that have similar soil properties at similar depths were combined under general classifications of poorly graded sands with silt (SP-SM) for the foundation and silty sand (SM) for the embankment fill. Two dimensional seepage under and through the proposed levee was analyzed. The seepage analysis assumed a steady-state flood corresponding to a water level at the top of levee.

Seepage analysis was performed on two variations of the typical levee replacement section:

1) a 12-ft-high option with levee slopes of 1V:2.5H, and
2) a 15-ft-high option with levee slopes of 1V:3H.

Both options have an approximately 30-ft-wide drainage blanket plus toe drain at the landside toe of the levee. The toe drain includes an 8-in. diameter, slotted PVC pipe wrapped in filter fabric, and discharge system embedded in a granular filter drain material and tied into the landside drainage blanket.

The results of modeling were used to establish the phreatic surface through the levee from the riverside to the landside, including the adjacent Riverside Channel, a ditch about 20 ft to the landside of the toe of the levee used to drain nearby fields. The model was also used to evaluate the potential for internal erosion/piping, sloughing or sand boils associated with seepage gradients.

6.1 Results and Discussion

The model shows the total pressure head and velocity vectors generated under and across the levee, as foundation (underseepage) and through-embankment seepage, respectively. The pressure head defines the simulated phreatic surface, shown as a pressure at or above zero (i.e. the unsaturated interface for unconfined or confined conditions). The velocity vectors are used to determine the hydraulic and exit gradients. Gradients are determined by taking the velocity vector and dividing it by the hydraulic conductivity of the soil. Velocity vector magnitudes show where excessive exit gradients may arise, causing sloughing or forming sand boils. Under both of the flooding scenarios analyzed the Riverside Channel is empty, which is the worst-case scenario for the drain-side slope failure for the steady-state seepage and sudden drawdown conditions. (Actual water levels in the channel during both scenarios are unknown, but it is likely that the channel would contain water).

The model calculated a maximum exit Y-gradient of approximately 0.6 at the base of the Riverside Channel slope. Draft ECB, Design Guidance for Levee Underseepage (July 2012), intended to supplement EM 1110-2-1913 which is in the process of being updated, indicates that a minimum Factor of Safety of 1.6 for the Riverside Channel is desired. However, this calculation is for the condition where a relatively impervious "top stratum" overlies the pervious foundation materials. Use of the formula provided indicates a factor of safety of 2.3 against sand boils when modeling the foundation materials as the top stratum along with the gradient of 0.6 generated by SEEP/W software. However, if a confining top stratum is assumed, exit gradients will increase, lowering the factor of safety to below 1, indicating that sand boils occur where relatively thin top stratums exist. The ECB later addresses this case accordingly (Item 7.), stating that in cases where the presence of a less pervious "top stratum" is questionable or another unfiltered seepage may be present (e.g., riverside channel), additional evaluation should be performed. This will be done during the design phase when more accurate information is available.

The proposed levee drainage blanket and toe drain discharge to the Riverside Channel via solid PVC pipe with a 1% gradient. Thus, if the water level in the channel rises above the invert of the discharge pipe, the drainage system could become ineffective due to reverse flow from the channel. This common condition should also be evaluated in greater detail during the design phase.

6.2 Blanket and Toe Drain Design Issues

6.2.1 Filter Material

To prevent clogging of the blanket and toe drain by fine-grained material (silt, clay, and fine sand), the drain filter material must be designed based on the characteristics of the surrounding soil. Laboratory analysis indicates the existing spoil bank has in excess of 20 percent fines. Based on the lab gradation results, a filter fabric with a nominal size equivalent to the No. 70 to No. 100 sieve placed as a sleeve around the drainage pipe will restrict fines from entering the pipe.

Preliminary design of the filter material is based on EM 1110-2-1913, Design and Construction of Levees (USACE 2000). The table below shows the selected gradation.

Table F-5. Blanket and Toe Drain Filter Material.

Standard Sieve Size	Percent Passing by Weight
1-1/2 inches	100
1-inch	90-100
3/8 inch	25-60
No. 4	5-40
No. 8	0-20

6.2.2 Quantity of Seepage Flow

The perforated and solid drainage pipe should be sized for the volume of seepage as determined in the SEEP/W model results.

The quantity of flow was calculated for both the 12-ft-high levee option and the 15-ft-high levee option. In both cases seepage flow is based on full saturation of the levee and a conservative value for the foundation permeability.

The 12-ft-high levee option yielded a maximum seepage rate of:

- 48 cubic feet/day/foot of levee at the toe drain; and

The 15-ft-high levee option yielded a maximum seepage rate of:

- 79 cubic feet/day/foot of levee at the toe drain.

7 SLOPE STABILITY ANALYSIS

The site stratigraphy and existing spoil bank soils vary along the various reaches of this project. To reduce the complexity of the model, soils for the different levee units that have similar soil properties at similar depths were combined under general classifications of poorly graded sands with silt (SP-SM) for the foundation and silty sand (SM) for the embankment fill.

The slope stability analysis uses the same two variations of the typical levee replacement section as the seepage analysis:

1) a 12-ft-high option with levee slopes of 1V:2.5H, and
2) a 15-ft-high option with levee slopes of 1V:3H.

Three load cases (corresponding to two flooding scenarios) were analyzed:

- Load Case I - End of Construction
- Load Case II - Steady-State flood corresponding to the top of levee.
- Load Case III - Transient flood (Sudden Drawdown) duration of 0.5 day, with model results protracted up to 14 days after drawdown.

A preliminary seismic stability analysis was also performed on the levee sections with two additional seismic load cases:

- Seismic Load Case I - End of Construction load case subjected to MDE pseudo-static loading.
- Seismic Load Case II - Steady-State loading of levee toe and subjected to OBE pseudo-static loading.

The type of analysis was basic in nature to provide a quantitative assessment of the potential impact of an earthquake on the levee designs. A non-staged pseudo-static analysis was performed using a horizontal ground accelerations associated with the MDE and the OBE. The lowest factor of safety generated was 0.86 and 0.93 for Seismic Load Case I for the 12-ft-high levee and 15-ft-high levee, respectively.. More detailed stability analyses, including liquefaction (not yet performed), will be performed as necessary prior to final design and during construction as necessary, depending on design and field variations from these preliminary models.

Additional details associated with load cases are provided in the Calculation Package (Enclosure E-1).

7.1 In Situ Soil Conditions

The geotechnical investigation of the existing spoil banks included drilling and soil sampling in the foundation soils on the riversides and landsides of the spoil banks to depths of 15-21 feet. Laboratory testing included grain size analyses, Atterberg Limits, moisture content, and classification of soils according to the USCS. Spoil bank foundation materials are in-situ native soils, generally classified as silty sand (SM), poorly graded sand (SP), poorly graded sand with silt (SP-SM), low plasticity clay (CL), and silt (ML).

Material from the spoil bank will be used to construct the engineered, replacement levee, and the replacement levee fill will be a composite of spoil bank material and excavated foundation material.

In situ soils are mostly stream channel or overbank deposits laid down by the Rio Grande during channel migration and overbank flooding, and typically consist of very loose to medium dense, low cohesion sands and silty sands.

Foundation soils beneath the existing spoil banks are assumed to be slightly over-consolidated. Based on the D_{20} grain size, foundation soils have high hydraulic conductivity (permeability). Relative densities of the spoil bank and foundation materials determined from Standard Penetration Tests (SPT) indicate poor compaction and low shear strength. Organic matter in the form of weeds, brush, stumps, branches, roots and matted grasses were encountered in the spoil banks during exploration.

7.2 Soil Material Properties

The stability of the proposed levees will be affected by the material properties of the engineered levee fill and the in situ foundation soils.

The strength of the in situ soils encountered during the subsurface investigations was estimated from the SPT blow counts.

Blow counts varied along the spoil banks. Low SPT values for foundation were recorded in silty sand (SM) and poorly graded sand (SP, SP-SM) layers between 2.5 to 20 feet below ground surface (bgs). The minimum value recorded was 1 blow per foot of penetration and the maximum value was 54 blows per foot. However, most blow counts ranged from 2 to 12 blows per foot which corresponds to a relative density of between 10 and 50 percent at a vertical effective stress of 1 kip per square foot.

The SPT blow counts were correlated with drained shear strengths using the method developed by Peck et al. (1974).

Relative densities were correlated to the SPT blow counts using the relationships developed by Gibbs and Holtz (1982) and used to estimate the unit weight of foundation soils. The relative density of cohesionless soils was used to estimate the angle of internal friction using the correlation in the US Navy Facilities Design Manual (NAVFAC 1982).

7.3 Stability Load Cases

7.3.1 Load Case I - End of Construction

For the End of Construction Condition, the embankment was modeled assuming that failure would occur through the riverside slope opposite the Riverside Channel. There would be no water pressure acting on the typical levee cross section other than any ground water within the foundation (underseepage). SLOPE/W forced failure to occur through the riverside slope of the levee. Failure was limited to the zone between the overbank area and the riverside levee slope. These initiation and termination zones are shown by a thick red line on the cross sections. This model also assumed an empty channel.

7.3.2 Load Case II - Steady-State Seepage

Steady seepage was assumed to occur with the design water surface elevation at the top of the levee. Saturation of the compacted levee embankment materials and development of a phreatic water surface were conservatively assumed. For the steady-state seepage condition, the embankment was modeled assuming that failure would occur through the landside slope opposite the river, since the water pressures acting on the submerged, riverside slope act as a stabilizing hydraulic buttress. The external water pressure helps to stabilize the riverside levee embankment slope against slope failure and counteracts the effects of water pressure with the embankment induced by through seepage.

The external water pressure resisting forces are not present on the landside slope opposite the river, thereby prompting slope failure in this location. The failure surface was forced through the landside slope of the levee. Failure was limited to the area just upstream of the levee crest and the invert of the Riverside Channel. This model also assumed an empty channel.

7.3.3 Load Case III - Sudden Drawdown

The duration of the flood stage was assumed to occur long enough to saturate the major part of the riverside embankment portion of the levee, and then fall faster than the embankment materials can drain. For the Sudden Drawdown (Transient) Condition, the embankment was modeled assuming that failure would occur through the riverside slope opposite the Riverside Channel, consistent with observed failure modes for levees during sudden drawdown. A discharge hydrograph having a duration of approximately 10 -12 hours from a peak of greater than 16,000 cfs to less than 2,000 cfs was used for the sudden drawdown analysis.

A transient interval was used for the 12-foot levee of 0.5 days, and the interval used for the 15-foot levee was 0.5 days, both allowed to run for 14 days to evaluate longer-term conditions. The maximum phreatic water surface used for the sudden drawdown condition was the same as used for the water level placed at the levee crest elevation. However, due to sudden drawdown, the external water pressures no longer exist and do not act riverside slope as a stabilizing hydraulic buttress. The nonexistent external water pressure does not stabilize the riverside levee embankment slope against slope failure, thereby prompting slope failure in this location. The failure surface was forced through the riverside slope of the levee. Failure was limited to the area just upstream of the levee crest and through the overbank area on the riverside. This model also assumed an empty channel.

7.3.4 Seismic Load Cases I and II

The results for Seismic Load Cases I and II are discussed in section 2.3.4 “Results of Preliminary Seismic Evaluation” and the factors-of-safety for these load cases are provided in the table below. Factors-of-safety for both the 12-foot-high and 15-foot-high levees dropped below one for Seismic Load Case I (subjected to pseudo static loads associated with the MDE) and factors of safety for both remained well above one for Seismic Load Case II (toe of levee hydraulically loaded to steady state condition and subjected to pseudo static loads associated with the OBE).

7.4 Factor of Safety Results

The factor of safety describes the stability of the levee slopes against failure. The factor of safety is the ratio of forces resisting slope movement to the driving forces causing slope movement. A factor of safety of 1.0 indicates a state of limit equilibrium and a factor of safety value less than 1.0 indicates slope failure. USACE recommends that levee slopes be no steeper than 1V:2H for ease of construction and ease of maintenance and requires a minimum factor of safety of 1.4 for long term stability (USACE 2000). Calculated factors-of-safety for the load cases discussed above are provided in the table below. **Table F-6. Calculated Factors-of-Safety**

Factors-of-Safety				Factors-of-Safety (Seismic)	
Levee Height (ft.)	**End of Construction**	**Steady State Seepage**	**Sudden Drawdown**	**End of Construction with Seismic (MDE)**	**Steady State Seepage with Seismic (OBE)**
12	1.65	1.66	1.58	0.86*	1.39
15	1.98	1.64	1.83	0.93*	1.43

* Levee meets performance requirements for the MDE in accordance with ER 1110-2-1806

7.5 Discussion and Recommendations

None of the calculated Factors of Safety are less than the minimum required value of 1.4 (USACE 2000) for any of the three load conditions (i.e. End of Construction, Steady-State Seepage, and Sudden Drawdown). Analysis was performed using assumed values for soil properties, and seepage and slope stability analyses may be adjusted during detailed design if soils at specific levee reaches are significantly different from the soil parameters used here.

Preliminary design cross sections investigated minimum setback distance requirements between the landside levee toe and top of the Riverside Channel slope. A minimum setback distance prevents the channel slope instability from impacting the landside slope and toe of the levee. A minimum distance of 20 feet is recommended for construction, maintenance, inspection, flood fighting, and traffic considerations, as reported in the Albuquerque West Levee Project, August 2008. The Albuquerque West Levee Project, August 2008 also modeled channel cross sections for slope stability with both an empty and full channel. Based on the slope stability results, a channel slope of 1V:3H with a minimum setback distance of 20 feet was recommended. This same analysis found that progressive failure of the channel would eventually produce a stable slope of 1V:3H and this would further flatten due to erosion.

Considering the channel slope of 1V:2.5H currently shown and modeled for this project, progressive channel slope failure could eventually impact the landside toe and slope of the newly constructed levee. This potential will be further evaluated and mitigated as necessary during the design phase.

8 DEWATERING and FOUNDATION CONSOLIDATION

Design and construction of this project will proceed in various phases and will be determined by the availability of project federal funds. Construction of this project is expected to begin with the Mountain View Unit and proceed downstream with construction of the Isleta West Unit, and then proceeding with Belen East Unit and Belen West Unit. Portions of the levee can be excavated and material placed on previously excavated sections on an on-going basis.

Dewatering issues are expected to be a problem in all units where construction of interior drainage structures, box culverts, and diversion structures are required. In Isleta West Unit, a "swamp area", located north of the BNSF Railroad could pose a problem with dewatering during construction. Another dewatering concern is an area of the Belen West Unit, north of NM Highway 309, where the town of Belen has constructed a fish pond adjacent to the spoil bank. In the area of the Isleta Diversion Dam, located at the upper end of the Belen East Unit just south of NM Highway 147, dewatering is expected during construction of the diversion works. This area is constricted downstream of diversion works, and includes the dam, low-flow channel and irrigation control structures.

Other local areas, including several areas of the Belen East Unit where the Riverside Channel flows have eroded the landside access/maintenance road back to the crest of the spoil bank, are also expected to require dewatering. There are designated wildlife areas within the Mountain View, Belen East, and Belen West Units which may be restricted to construction during some periods. (See the Environmental Considerations section of this report for details.)

8.1 Consolidation

The levees will be founded on silty sands and sand for the majority of all the units and in most areas, the levees will be built on the same alignment as the existing spoil bank. Consolidation of the foundation has already occurred or expected to be rapid. In some localized areas, the levees may be founded on clays but these appear to have been pre-loaded by the weight of the existing spoil bank. Clay layers interlayered with sands would expedite the consolidation process. No over-build is required for the new levees. Any areas of long-term localized settlement could be built up at a future date; however, no such areas are anticipated.

8.2 Special Foundations

There are several areas along all the units of this project that require special foundation preparation for structures, including dewatering.

8.2.1 Mountain View Unit

The first structure is a 5-ft high x 10-ft wide ungated concrete box culvert (CBC), which is located in the Riverside Channel where the levee crosses the channel just north of I-25.

8.2.2 Isleta West Unit

The first structure is a 5-ft high x 10-ft wide ungated CBC, which is located in the low flow conveyance channel where the levee crosses the Riverside Channel just south of I-25. The second structure is a 5-ft high x 10-ft wide x 50-ft long gated CBC, which is located in the low flow conveyance channel where the levee crosses the channel north of the BNSF Railroad bridge.

8.2.3 Belen East Unit

The first structure is a 5-ft high x 10-ft wide gated CBC, which is located in the low flow conveyance channel where the levee crosses the Riverside Channel just north of I-25. The second structure is a 5-ft high x 10-ft wide gated CBC, which is located in the Barr Chicla Diversion channel where the levee crosses this diversion channel, south of Highway 147. The third structure is a 5-ft high x 10-ft wide gated CBC, which is located in an unnamed diversion channel where the levee crosses this diversion channel, south of Highway 147. The fourth structure is a 48-inch diameter gated reinforced concrete pipe (RCP), which is located in an unnamed diversion channel, south of Highway 147. The fifth structure is a 36-inch diameter gated RCP, which is located in an unnamed irrigation channel, south of Highway 147. The sixth structure is an 8-ft high x 10-ft wide gated CBC, which is located in the LFCC where the levee crosses the LFCC, south of NM Highway 147.

Structure numbers 2 through 6 (above), are part of the diversion works and irrigation works, located at the Isleta Diversion Dam. The seventh structure is a 5-ft high x 10-ft wide gated CBC, which is located in the Upper Peralta Riverside Channel where the levee crosses this drain, south of the Otero Drain. The eighth structure is a 5-ft high x 10-ft wide gated CBC, which is located in the Peralta Main Canal.

8.2.4 Belen West Unit

The first structure is a 48-inch diameter x 150-ft long RCP, where the levee crosses the Upper Belen Riverside Channel, east of the Los Luna Lateral. The second structure is a 5-ft high x 10-ft wide x 100-ft long ungated CBC, which is located in the channel where the levee crosses the 240 Wasteway, east of the Los Luna Lateral. The third structure is a 60-inch diameter x 125-ft long RCP, where the levee crosses the Los Chavez Wasteway. The fourth structure is a 5-ft high x 10-ft wide x 57-ft long ungated CBC, which is located in the Lower Belen channel. The fifth structure is an 8-ft high x 10-ft wide x 5464-ft long gated CBC, which is located in the Lower Belen channel where the levee crosses this drain, south of the BNSF bridge and Rio Grande crossing and re-enters the Lower Belen channel. The sixth structure is a 5-ft high x 10-ft wide x

100-ft long ungated CBC, which is located in an unnamed wasteway.

The above structures will be founded at approximately the existing invert elevations of their respective channel, drain, diversion, or wasteway invert elevations, thus the requirement for care and diversion of water during construction. Based on the drilling logs, dewatering will be required for construction of the above structures. As discussed above, SPT data presented indicates that the majority of the foundation materials consist of silty sands (SM) and poorly graded sands (SP, SP-SM), having SPT field blow counts ranging from 2 to 12 blows per foot. An allowable bearing capacity of 2 ksf is recommended for preliminary design of the above structures. Additional subsurface investigations, including field and laboratory tests, will be required for each structure during detailed design.

9 EMBANKMENT QUANTITIES

Distribution and material usage for a project of this length is the primary cost consideration. The material usage may also be dictated by available Federal funds and phasing of the contract. An estimated 3.7 million cubic yards of compacted random fill will be required to construct the levees. An estimated 420,000 cubic yards of pervious fill will be required to construct the landside drainage blanket and toe drain. An estimated 2.95 million cubic yards of required excavation of in-place spoil bank material has been calculated.

The existing spoil bank will be the main source of material for construction of the proposed replacement levee. The volume of the existing spoil bank is based on numerous measured sections along the length of the spoil bank. A volume loss of about 10% is anticipated due to the presence of deleterious or otherwise unsuitable material in the spoil bank; unsuitable material must be removed before soils are placed as engineered fill in the proposed replacement levee.

The volume of material available from the existing spoil bank after removal of unsuitable material was adjusted to account for a gain in volume as the spoil bank is disturbed during demolition (bulking factor = 1.18), and a loss in volume as the soil is subsequently compacted during construction of the proposed replacement levee (bulking factor = 0.79).

The resulting spoil bank volume is compared to the volume of the proposed replacement levee to estimate the amount of additional material needed (borrow), or the amount of excess material that will need to be removed (waste).

Waste fill may be placed along and within the 15-ft vegetation free zone (VFZ) on the riverside of the levee. Noted that there are currently no identified borrow areas. The above total quantities of required spoil bank excavation, embankment random fill materials, pervious fill for the landside blanket drain and toe drain, waste fill, and borrow materials are shown in Table F-7.

It appears that adequate quantities of suitable random fill material are available for construction of Mountain View and Isleta West Unit reaches of the project from required excavation of the spoil banks. Suitable borrow materials (with fines equal to or greater than 20% passing the #200 sieve) are required for construction of the Belen East and Belen West reaches of the project.

Currently, there are no borrow areas identified for this project. However, if suitable borrow materials with fines having equal to or greater than 20% passing the #200 sieve cannot economically be attained from nearby proposed commercial sources and borrow is required to construct the new levees, a design and construction alternative may include a soil-bentonite slurry cutoff trench and wall incorporated within the levee to provide seepage control for through embankment seepage. In addition, the only areas considered as possible sources of suitable borrow materials are those areas along the overbank areas of the Rio Grande. However, these areas were not considered as being feasible, based on the environmental impacts and required mitigation. Pervious fill materials will be obtained from local commercial sources.

9.1 New Levee Construction Materials

9.1.1 Random Fill

Random fill required for levee construction will consist of sands, silty sands, sandy silts with some clayey sands, and sandy clays which will be obtained from excavation of existing spoil banks. Subsurface investigation in the areas of required excavation indicate that suitable materials obtained from required spoil bank excavations are available for random fill, after clearing and grubbing and stripping of foundation materials. Thickness of random layers before compaction will not be more than 9 inches for tamping rollers or more than 12 inches for rubber-tired rollers.

Table F-7. Fill Quantities for Existing Spoil-Bank and Proposed New Levee.

Column	A	B	C	D	E	F	G	H
Levee Unit	**Existing Spoil Bank Volume**	**(A) less 10% unsuitable material**	**(B) times 1.18 Bulking Factor**	**(C) times 0.79 Compaction Factor**	**New Levee Volume**	**(E) minus (D) Fill Shortage (Borrow)**	**(D) minus (E) Fill Excess (Waste)**	**Pervious Fill Needed (Drains, etc.)**
Mountain View 100-year-flood elevation plus 4 ft	309,273 cu yds	278,345 cu yds	328,447 cu yds	259,473 cu yds	244,259 cu yds	None	15,214 cu yds	54,268 cu yds
Isleta West Unit 1 100-year-flood elevation plus 4 ft	139,563 cu yds	125,607 cu yds	148,216 cu yds	117,091 cu yds	125,402 cu yds	8,311 cu yds	None	17,333 cu yds
Belen East Unit 100-year-flood elevation plus 5 ft	1,237,529 cu yds	1,113,776 cu yds	1,038,262 cu yds	820,227 cu yds	1,284,955 cu yds	464,728 cu yds	None	119,992 cu yds
Belen West Unit 100-year-flood elevation plus 5 ft	1,262,433 cu yds	1,136,190 cu yds	1,340,704 cu yds	1,059,156 cu yds	1,284,955 cu yds	225,799 cu yds	None	228,269 cu yds

Moisture content of random fill during compaction will be between minus 2 percent and plus 2 percent of optimum moisture content. If more than a single lane of random fill is being placed simultaneously, the leading edge of any lane will be at least 100 feet from the leading edge of adjacent lanes. Eight complete passes of tamping rollers or 4 complete passes of rubber-tired rollers will be required for each lift of random fill. A minimum of 95 percent maximum dry density, as determined by the standard compaction test ASTM D698, will be required in the random fill sections. The more impervious random fill materials will be placed in the center of the levee.

9.1.2 Drain Material

Drain material will be required for the landside drainage blanket and toe drain and shall be obtained from commercial sources. Local sources were not investigated for this report. Approximately 420,000 cubic yards of drain material will be required. The contractor will not be required to use any specific source, provided that the material used meets all contract requirements. Prior to use in the embankment, the contractor will be required to furnish test data proving the quality of the source he intends to use, and proof that the required gradation can be produced at that source. The grading of the drain material will be in accordance with Appendix D, Filter Design, of EM 1110-2-1913. The material will meet requirements specified for concrete aggregate as to quality and be reasonably well-graded within the proposed limits shown in Table F-6.

Table F-8. Drain Material Gradation.

Standard Sieve Size	Percent Passing by Weight
1-1/2 inches	100
1-inch	90-100
3/8 inch	25-60
No. 4	5-40
No. 8	0-20

Drain material will be placed in layers with a lift thickness not to exceed the thickness required to achieve a 12-inch compacted lift after compaction and compacted by 4 passes of a rubber-tired roller or vibratory steel wheel roller. A minimum relative density of 85 percent will be required at optimum moisture content for compaction.

9.1.3 Concrete and Concrete Aggregates

Maximum size coarse aggregate will be 1-1/2 inches. Concrete mix design will require 4,000 psi, 28-day compressive strength. Adequate quantities of concrete and concrete aggregates are available from commercial sources in the area.

9.1.4 Water for Construction

The contractor will be required to provide water for construction.

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APPENDIX F – GEOTECHNICAL ENGINEERING

ENCLOSURE E1
CALCULATION PACKAGE

NUMERICAL MODELING

Model Set-up

Seepage and stability analysis were performed on two variations of the typical levee replacement section: a 12-ft-high option with levee slopes of 1V:2.5H, and a 15-ft-high option with levee slopes of 1V:3H. Both options have a 30-ft-wide drainage blanket plus toe drain with an 8-inch diameter perforated PVC pipe at the landside toe of the levee.

Figure E-1 shows the mesh layout and the distribution of materials used for modeling of the 12-ft-high levee option, and Figure E-2 shows the 15-ft-high levee option.

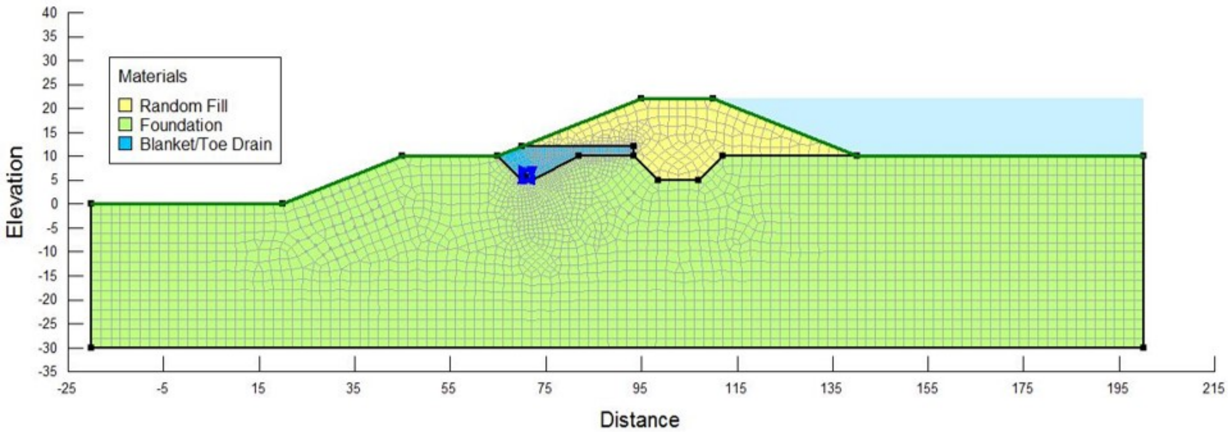

Figure E-1. Numerical modeling mesh for 12-ft-high levee option.

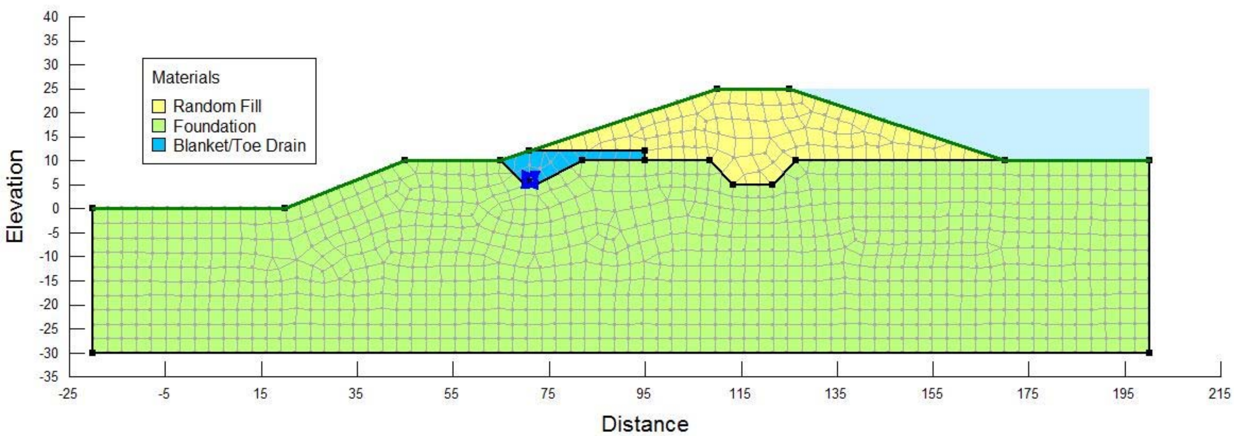

Figure E-2. Numerical modeling mesh for 15-ft-high levee option.

The levee profiles in Figures E-1 and E-2 extend 20 feet from the landside toe of the levee to the centerline of the Riverside Channel (i.e. half the width of the channel). The Riverside Channel was assumed to be empty, which is the worst case scenario with the channel slope and invert modeled as possible seepage faces. The cross sections were analyzed to demonstrate the effect of an empty channel on the phreatic water surface, as well as the exit gradients on the channel slope and invert.

SEEPAGE MODEL

Seepage modeling for both through-embankment and foundation (underseepage) conditions was performed using SEEP/W, a finite-difference computer model for simulating water and material movement under saturated and unsaturated conditions (GeoStudio 2012). Model output includes graphic plots of pressure head, pore water pressure, or total head.

Model boundary conditions were set to correspond to a constant-head river height (steady-state) to simulate conditions of maximum seepage (worst-case condition).

The embankment and foundation materials are neither homogeneous nor isotropic, so embankment and foundations soil properties vary both vertically and horizontally. Horizontal permeability is assumed to be larger than the vertical permeability because of natural layering in the foundation soils, and the proposed construction method of placing and compacting soil in lifts in the embankment. We assumed a vertical to horizontal ratio or permeability of 1:2 for the embankment fill, and 1:4 for the foundation native soils.

The permeability of the foundation and embankment material was estimated from the soil grain-size distribution. Laboratory grain size analysis was performed on soil samples collected from various reaches of the project. A correlation between the D_{20} size and permeability (Justin et al. 1961) was used to estimate permeability for preliminary design.

A D_{20} size for the foundation materials (sand; USCS class of "SP") of 0.20 mm and a D_{20} size of 0.07 mm for the embankment random fill (silty sand and sand; USCS class of "SM" and "ML") were chosen. (A D_{20} of 0.20 mm relates to a sieve size of between 70 and 100. A D_{20} of 0.07 mm relates to a sieve size of between 140 and 200).

The permeability using the correlation with grain-size from Justin et al. (1961) is similar to permeability values from other water control projects in the area. These results were combined with engineering judgment to assign the permeability values shown in table E-1.

Table E-1. Soil Properties used for seepage modeling.

Material Property	Foundation	Random Fill	Blanket
Ky (ft/day)	6.25	1	100
Kx (ft/day)	25	2	100
Phi (degrees)	30	32	35
Unit weight (pcf)	110	115	110

Seepage Model Results

Figures E-3 and E-4 show the seepage model results for the 12-ft-high and 15-ft-high levee options, respectively. The figures show the total head and velocity vectors generated through the levee, as foundation seepage (below the heavy black line) and through-embankment seepage (above the heavy black line). The pressure head defines the phreatic surface as a pressure at or above zero (i.e. the unsaturated interface; shown by the dashed blue line). Flux labels were added to the flux section around the drain pipe to show the flow of approximately 50 cubic feet per day into the pipe for both levee sections.

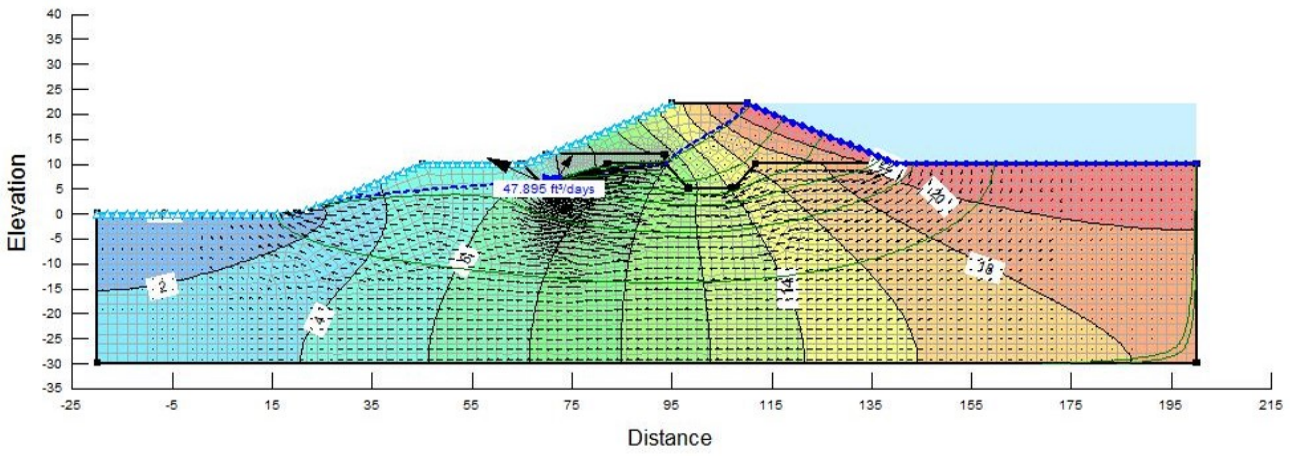

Figure E-3. Flow net diagram of steady-state seepage for 12-ft-high levee option.

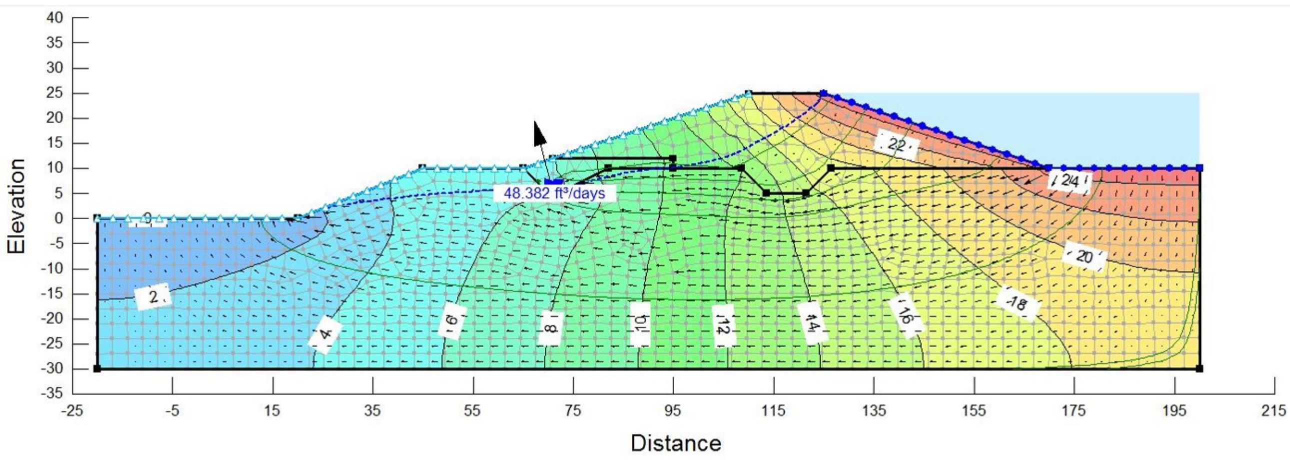

Figure E-4. Flow net diagram showing steady-state seepage for 15-ft-high levee option.

The velocity vectors are used to determine the hydraulic and exit gradients. Gradients are determined by taking the velocity vector and dividing it by the hydraulic conductivity of the soil. Velocity vector magnitudes show where excessive exit gradients may arise, potentially causing sloughing, sand boils, or internal erosion and piping. Modeling assumes the Riverside Channel is empty, which is the worst-case scenario for the landside slope failure of the levee for both steady-state seepage and sudden drawdown conditions. (During flood conditions it is likely that the channel would contain water).

The model shows that the maximum exit Y-gradient occurs at the base of the Riverside Channel slope and is 0.4 to 0.6 for the 12-ft-high levee (Figure E-5), and 0.4 to 0.5 for the 15-ft-high levee (Figure E-6).

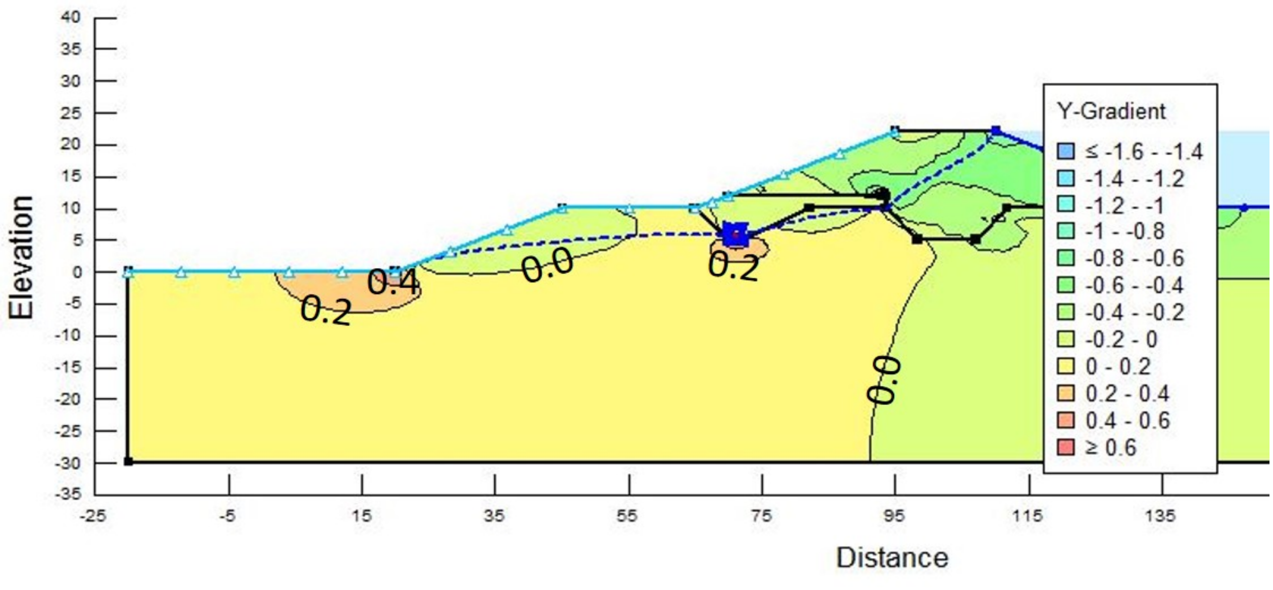

Figure E-5. Vertical (Y) seepage gradients for 12-ft-high levee option.

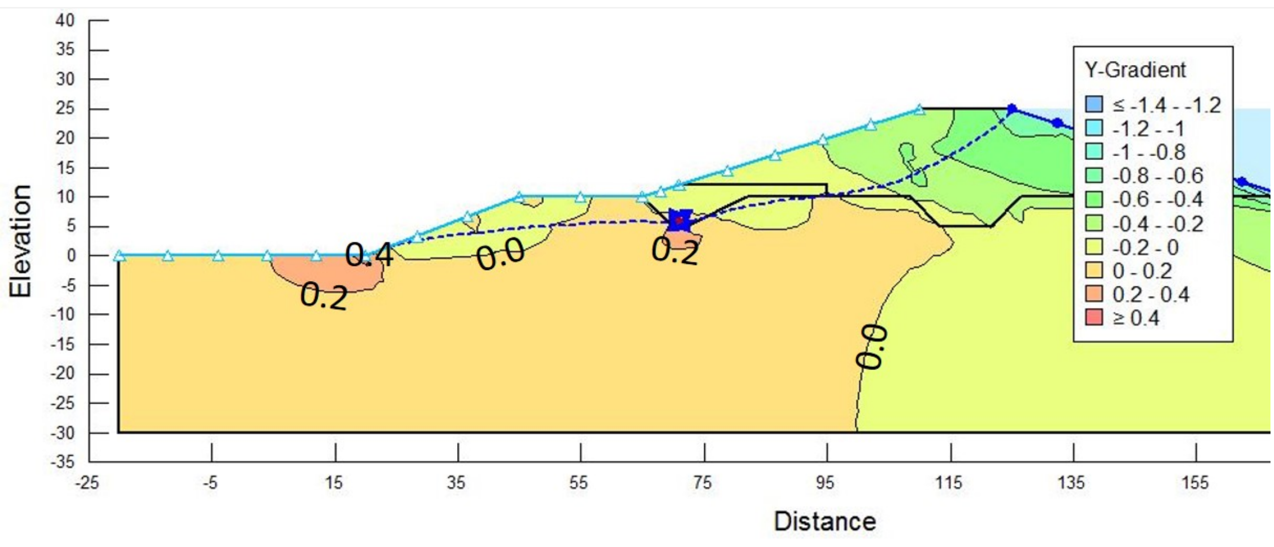

Figure E-6. Vertical (Y) seepage gradients for 15-ft-high levee option.

Floodwater was introduced into the model as a constant head along the riverside levee boundary. Floodwater for the steady-state, high flood scenario was added as a constant head equal to the elevation of the levee crest and extending from the riverside edge of the model domain, across the ground surface to the riverside crest of the levee.

Floodwater for the transient scenario was added as a falling head equal to the elevation of the rain fall estimated at the cross section location and extending from the riverside edge of the model domain, across the ground surface to the riverside crest of the levee. The transient flood duration was set to 10-12 hours using the hydrograph for the Mountain View Unit.

The floodwater for the transient, high flood scenario was set to the crest of the levee and allowed to develop a steady state seepage condition. Then the floodwater elevation was dropped to the riverside levee toe over 0.5 day to model the Mountain View Unit hydrograph. The model was allowed to run for up to 14 days to check predicted longer term conditions.

SLOPE STABILITY MODEL

Slope stability modeling was performed using SLOPE/W computer software (GeoStudio 2012). The existing spoil banks were not analyzed for slope stability since they are to be replaced by a new engineered levee. Stability modeling assumed no water in the Riverside Channel.

The foundation depth of the proposed levee is based on a 6-inch minimum stripping below existing grade, plus a 5-foot deep inspection trench with 1V:1H side slopes, and the toe drain location.

Available construction soils are predominantly silty sand (SM), poorly graded sand (SP), silt (ML) and poorly graded sand with silt (SP-SM). The SLOPE/W model requires values for cohesion, angle of internal friction, and in-situ unit weight of soils, along with groundwater and surface water elevations.

Table E-2 summarizes the soil material properties used in the slope stability analyses. Soil properties were estimated using SPT blow counts and USCS soil classifications from previous explorations at the project site, together with engineering experience with similar projects and soil types.

Table E-2. Soil material properties used in slope stability analysis.

Location	USCS Soil Class	Dry Unit Weight (pcf)	Cohesion C	Phi angle Ø
Foundation	SP-SM	110	0	30
Random Fill	SM	115	0	32
Blanket/Toe Drain	GW	110	0	35

The SLOPE/W software program uses the Mohr-Coulomb strength criteria to determine the soil shear strength, according to the equation:

$\tau = C + \sigma_n \tan \phi$ where

τ = shear strength at failure

C = cohesion

σ_n = normal stress acting on shear surface

ϕ = angle of internal friction

The Morgenstern-Price method was used in the SLOPE/W analyses to calculate the factor of safety. The Morgenstern-Price method has the advantage of satisfying moment and force equilibrium, and accounting for shear and normal interslice forces.

Modeling was originally performed for three loading cases: end-of-construction; steady-state-seepage; and sudden-drawdown (transient) over a 12-hour period.

Preliminary results of the SLOPE/W analysis suggested that the riverside of the Riverside Channel (i.e. the channel bank adjacent to the levee) was susceptible to slope failure, independent of the risk of failure to the landside of the levee during steady-state seepage conditions. To highlight the risk of failure of the Riverside Channel slope, an additional failure case was modeled that involved only the Riverside Channel. Consequently, four loading cases were ultimately modeled:

1) End of Construction;
2) Steady-State (Channel);
3) Steady-State (Global/Landside); and
4) Sudden Drawdown.

Slope stability was also evaluated with consideration for seismic ground motions using a pseudo-static analysis. The “End of Construction” load case was evaluated subject to the MDE seismic loading and the “Steady-State” load condition was modified and evaluated subject to the OBE seismic loading. The modified “Steady State” analysis included a three-foot hydraulic loading on the riverside toe of the levee.

Each of the eight load cases was run against both the 12-ft-high levee option and the 15-ft-high option, for a total of 16 model runs (Table E-3).

Stability Model Results

<u>Load Case I - End of Construction</u>

Figures E-7 and E-8 show the end-of-construction model results for the 12-ft-high and 15-ft-high levee options, respectively. The embankment was modeled assuming there would be no water pressure acting on the levee cross section other than groundwater within the foundation, and under this condition the SLOPE/W model forces slope failure to occur on the riverside of the levee. The potential failure zone was limited to the zone between the crest of the levee and the riverside toe area (shown in the figures as a thick red line). The green hatched semi-circle is the area of the potential slope failure.

Table E-3. Model iterations using SLOPE/W.

Figure No.	Levee Height Option	Static vs. Seismic Load Case	Flooding Load Case	Slope Analyzed
E-7	12-ft	Static	End of Construction	Levee Embankment
E-8	15-ft	Static	End of Construction	Levee Embankment
E-9	12-ft	Seismic (MDE)	End of Construction	Levee Embankment
E-10	15-ft	Seismic (MDE)	End of Construction	Levee Embankment
E-11	12-ft	Static	Top of Levee, Steady-State	Riverside Channel
E-12	15-ft	Static	Top of Levee, Steady-State	Riverside Channel
E-13	12-ft	Static	Top of Levee, Steady-State	Levee Embankment
E-14	15-ft	Static	Top of Levee, Steady-State	Levee Embankment
E-15	12-ft	Seismic	Toe Loaded to Steady-State	Levee Embankment
E-16	15-ft	Seismic	Toe Loaded to Steady-State	Levee Embankment
E-17	12-ft	Static	Sudden Drawdown	Levee Embankment
E-18	15-ft	Static	Sudden Drawdown	Levee Embankment

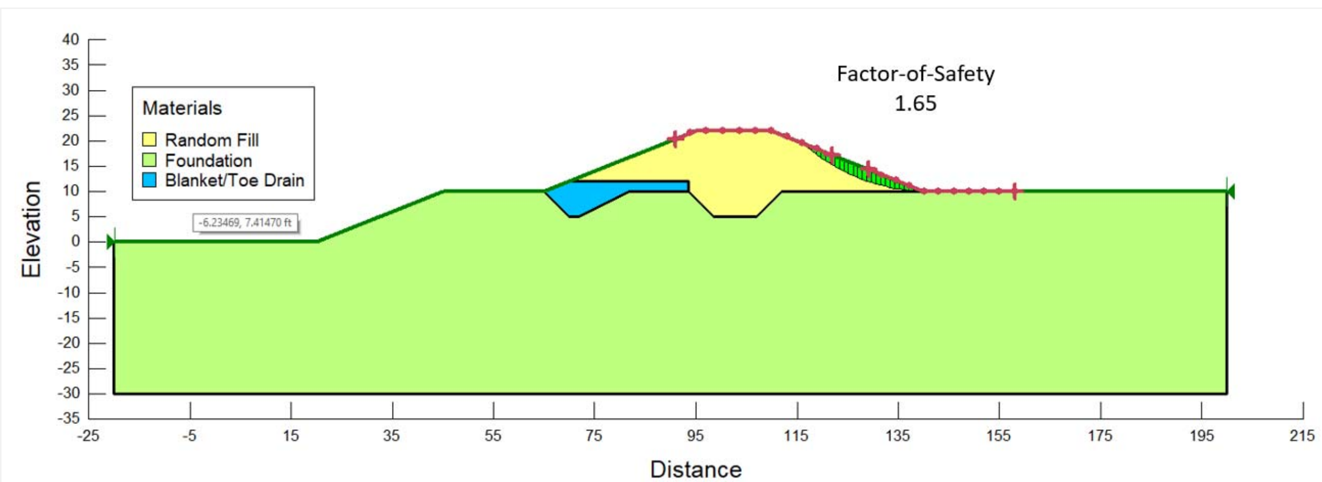

Figure E-7. End of construction slope stability for 12-ft-high levee option.

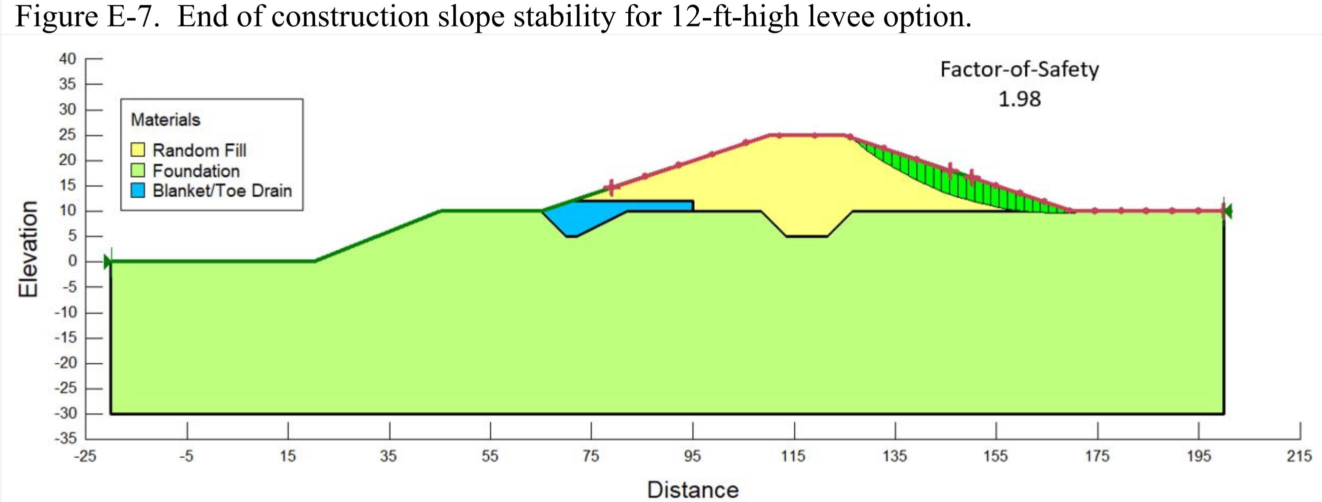

Figure E-8. End of construction slope stability for 15-ft-high levee option.

The end-of-construction model results are shown with an additional horizontal seismic load of 0.28gin Figures E-9 and E-10 for the 12-ft-high and 15-ft-high levee options, respectively. This loading is representative of the Maximum Design Earthquake (MDE) as determines using the USGS Unified Hazard Tool for an earthquake event having an average return period of 950 years in accordance with ER 1110-2-1806. All failure planes are encompassed by a red envelope and the slope-failure plane with the minimum factor of safety is highlighted.

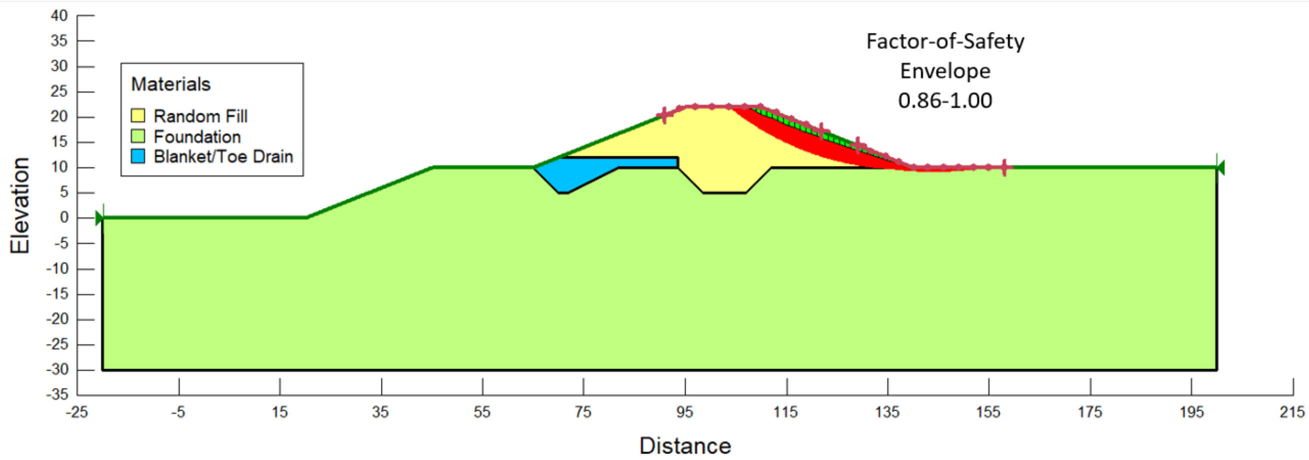

Figure E-9. End of construction slope stability for 12-ft-high levee option subject to MDE (0.28g).

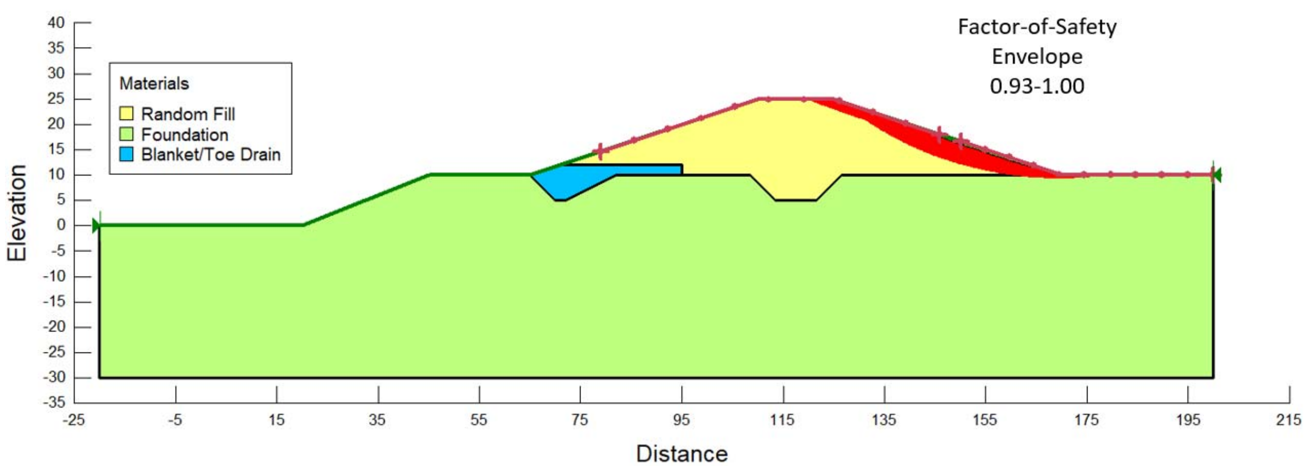

Figure E-10. End of construction slope stability for 15-ft-high levee option subject to MDE (0.28g).

Load Case II - Steady-State

Figures E-11 and E-12 show the steady-state model results for the 12-ft-high and 15-ft-high levee options, respectively. The steady-state seepage condition was assumed to occur with the water elevation at the top of the levee. This model provides the worst-case slope stability scenario with regard to embankment saturation and maximizes the potential development of the phreatic surface through the levee cross section. The embankment was modeled assuming that failure would occur through the landside slope opposite the river; this failure mode is characteristic for levees that are not overtopped.

Since the water level was placed at the levee crest elevation, the water pressures acting on the submerged, riverside slope acts as a hydraulic buttress. The external water pressure helps to stabilize the riverside embankment slope and counteracts the effect of water pressure within the embankment induced by through-seepage, as defined by the phreatic water surface. The landside slope lacks a comparable external support, thereby promoting slope failure on the landside slope.

The minimum factor of safety has been determined in two locations for both the 12-foot-high and 15-foot-high levee options: the Riverside Channel side-slope nearest the levee toe and the landside levee embankment. The results are presented in Figures E11 - E14.

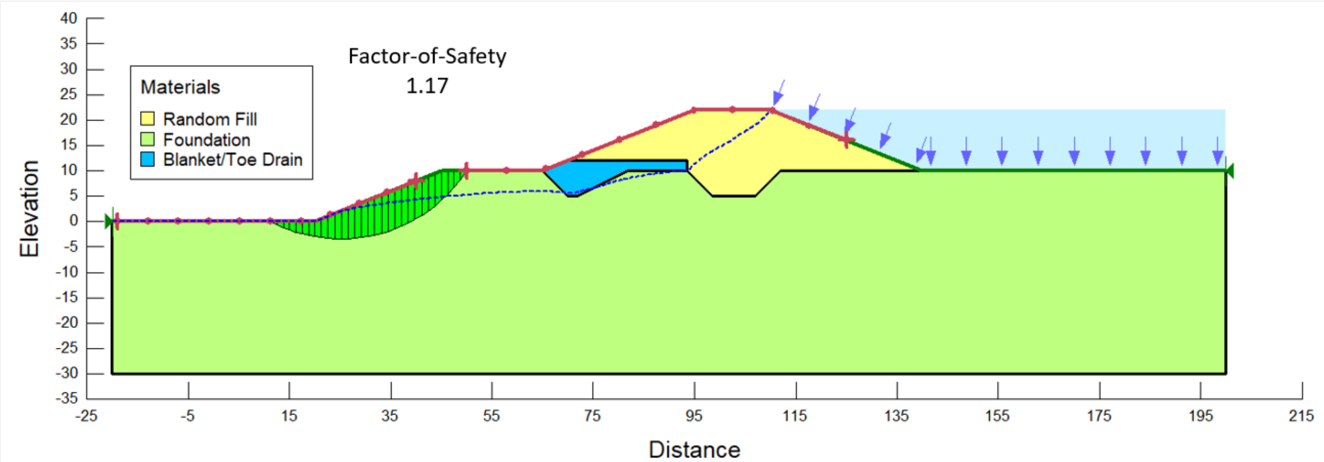

Figure E-11. Riverside Channel steady-state slope stability for 12-ft-high levee option.

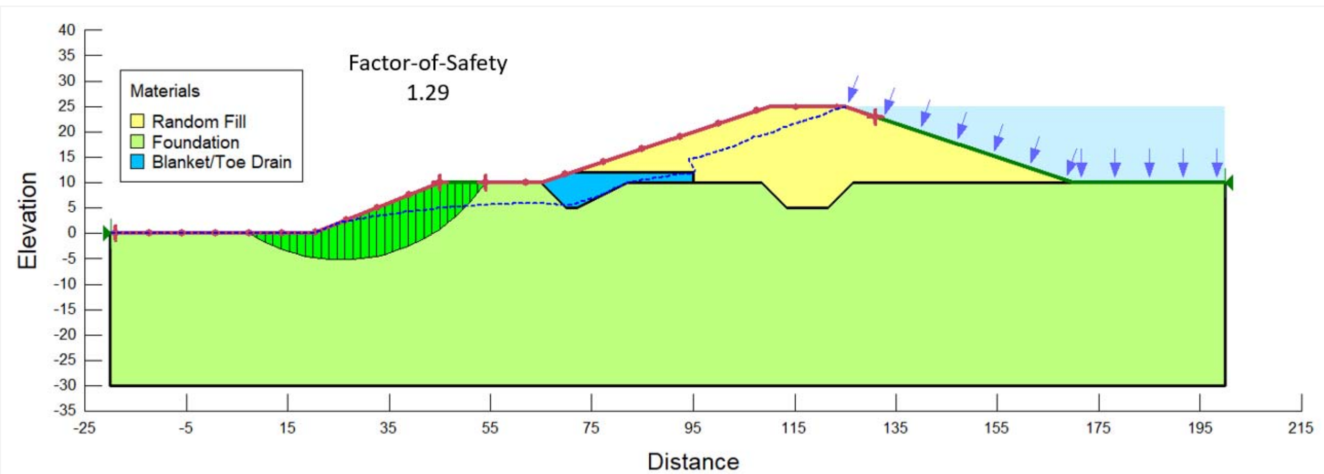

Figure E-12. Riverside Channel steady-state slope stability for 15-ft-high levee option.

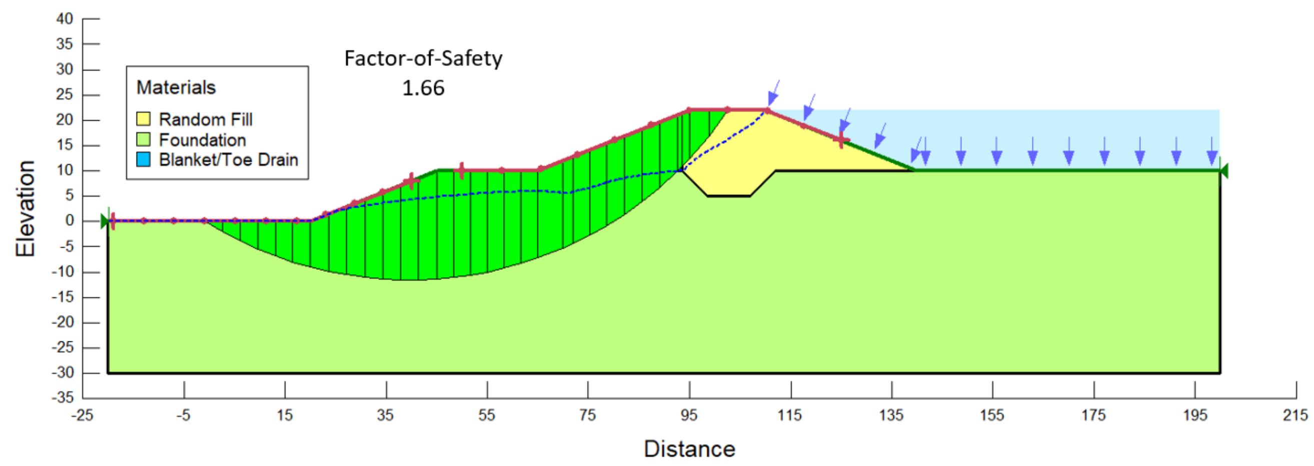

Figure E-13. Levee embankment steady-state slope stability for 12-ft-high levee option.

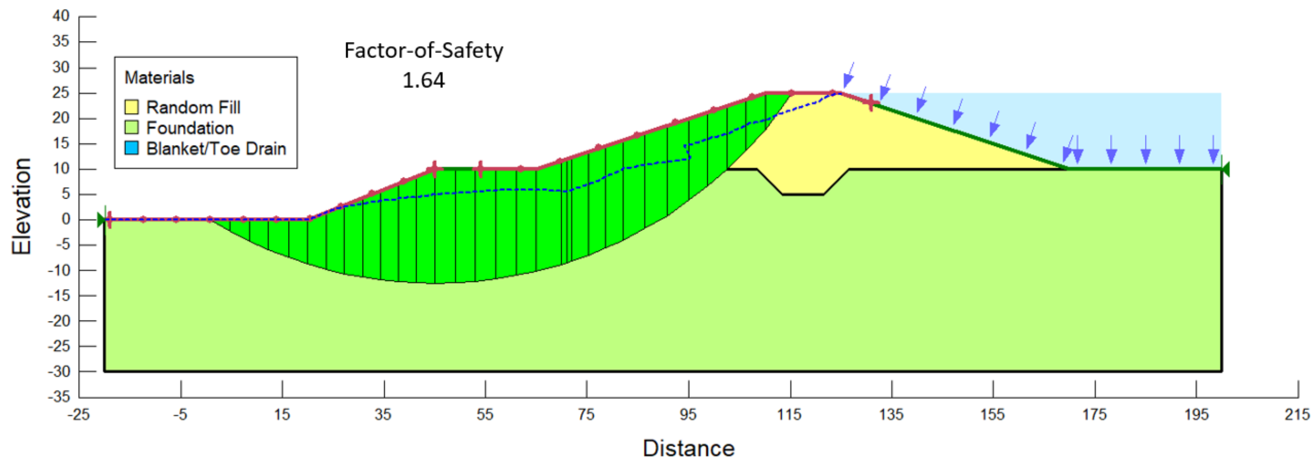

Figure E-14. Levee embankment steady-state slope stability for 15-ft-high levee option..

A combined steady-state and seismic slope stability model was also considered wherein the lower three feet of embankment toe were loaded to a steady-state seepage condition and the embankment subsequently subjected to an earthquake loading equal to 0.07g which corresponds to the OBE ground motion. The performance requirement for the OBE is that the levee functions with little or no damage and without interruption of function as required by ER 1110-2-1806. The results of this analysis are provided in Figures E15 and E16 and show the OBE performance requirement is met for both the 12-ft-high and 15-ft-high levee options.

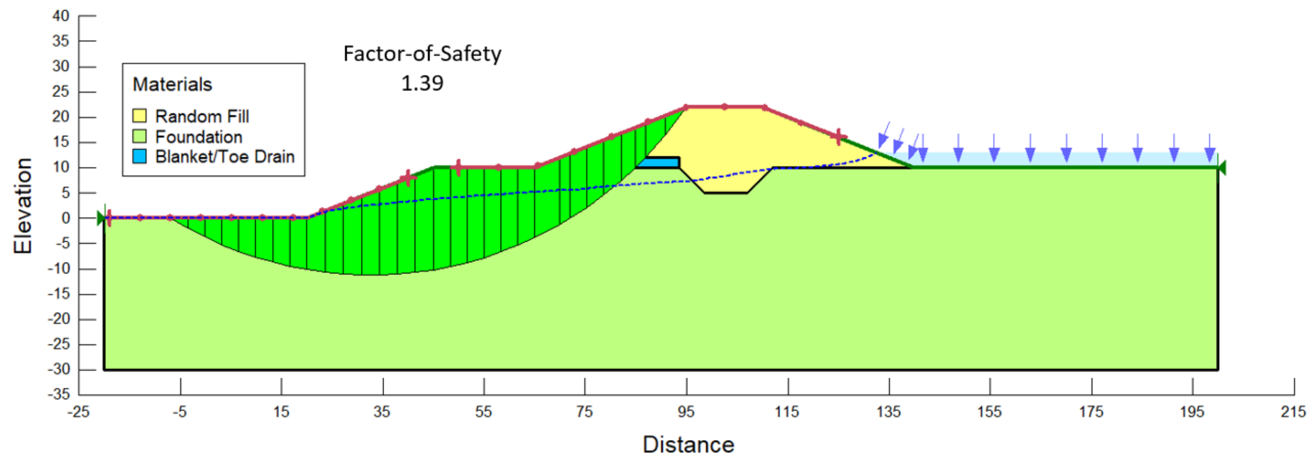

Figure E-15. Levee embankment slope stability for 12-ft-high option with toe loaded to steady-state condition and OBE (0.07g) seismic load applied.

Figure E-16. Levee embankment slope stability for 15-ft-high option with toe loaded to steady-state condition and OBE (0.07g) seismic load applied.

<u>Load Case III - Sudden Drawdown</u>

For this load case, duration of the flood stage was assumed to occur long enough to saturate the major part of the riverside embankment portion of the levee and then falls faster than the embankment materials can drain. For the Sudden Drawdown (Transient) Condition, the embankment was modeled assuming that failure would occur through the riverside slope opposite the Riverside Channel. This slope stability failure is consistent with failure modes for levees during sudden drawdown. Based on USACE Hydraulic & Hydrologic studies, a discharge hydrograph having a duration of approximately 10 -12 hours from a peak of greater than 16,000 cfs to less than 2,000 cfs was used for the sudden drawdown analysis.

The GeoStudio SLOPE/W allows the user to define this transient condition in time increments. The increment used was 0.03 days, increasing exponentially, for a total duration of 14 days. The maximum phreatic water surface used for the sudden drawdown condition was the same as used for the water level placed at the levee crest elevation. However, due to sudden drawdown, the external water pressures no longer exist and do not act on the riverside slope as a stabilizing hydraulic buttress. The nonexistent external water pressure does not stabilize the riverside levee embankment slope against slope failure, thereby prompting slope failure in this location. SLOPE/W forced failure to occur through the riverside slope of the levee. Failure was limited to the area just upstream of the levee crest and through the overbank area on the riverside. These initiation and termination zones are shown by a thick red line on the cross sections. This model also assumed an empty channel. The channel is on the left side of each typical levee replacement section with the Rio Grande on the right side. The blue dashed line on the cross section represents the phreatic surface elevation and the slope failures are shown as green, with arc-shaped vertical bars near the levee embankment slope surface. The minimum four-digit factor of safety is shown above the family of red arcs.

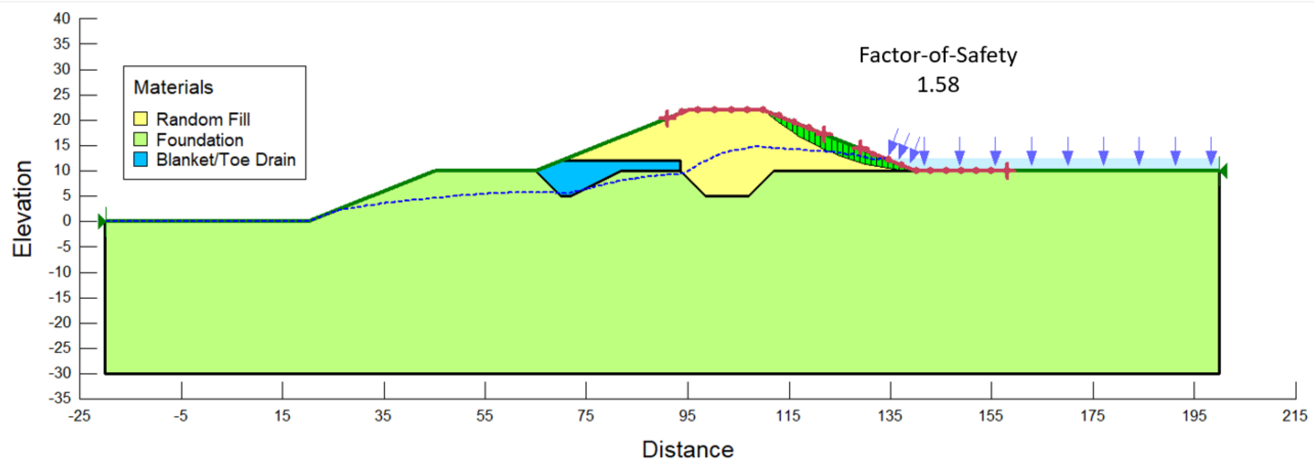

Figure E-17. Sudden drawdown slope stability for 12-ft-high levee option.

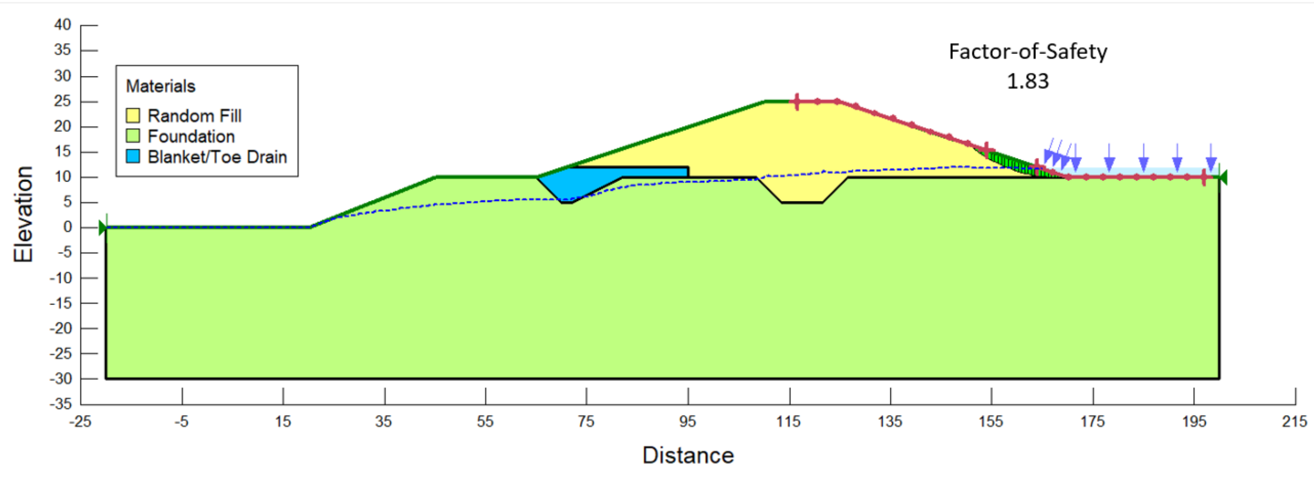

Figure E-18. Sudden drawdown slope stability for 15-ft-high levee option.

The expected duration of the maximum flood would be a short duration based on the discharge hydrograph, and the phreatic surface shown would not necessarily have the time to develop through the levee embankment. The phreatic surface shown for the Steady-State conditions is probably higher than what would develop under a Rio Grande short duration flood event. Seepage values would correspondingly be lower because the phreatic surface has limited time to develop. A lower phreatic surface would equate to a higher stability since less levee embankment would be saturated and the driving forces for slope failure would be lower. Also it is highly unlikely that the Riverside Channel will be empty, and the additional external water weight would resist the sliding forces. Therefore, the calculated GeoStudio SLOPE/W factors of safety are conservative. Also, a fully developed phreatic water surface through the levee embankment during the Sudden Drawdown condition was assumed for design. In reality, as soon as the water recedes, the levee embankment will also start to drain through the silty sand (random fill) and foundation material and excess pore pressures will dissipate.