US Army Corps of Engineers®

Papillion Creek and Tributaries Lakes, Nebraska

General Reevaluation Report

Appendix C
Geotechnical Analysis

June 2021

Omaha District
Northwestern Division

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1. INTRODUCTION

This technical appendix describes and summarizes the geotechnical and civil site plan aspects and design considerations of the feasibility report.

All elevation data in this report references the North American Vertical Datum of 1988 (NAVD88) unless otherwise specified.

The horizontal coordinate system used for civil modeling was NAD 1983 (2011) State Plane Nebraska FIPS 2600 (US Feet).

2. THE PAPILLION CREEK BASIN

2.1. GEOGRAPHY

The Papillion Creek basin is in three different counties in Nebraska: Washington, Douglas, and Sarpy. It is on the order of 41 miles long with a maximum width of approximately 17 miles. The basin drains slightly more than 400 square miles with the Omaha metropolitan area accounting for much of its middle third. The region is a loess-mantled upland till surface dissected by deeply incised stream valleys with moderately to steeply sloping banks. The three major streams in the watershed are shown in Figure 1: the Little Papillion Creek (red), the Big Papillion Creek (yellow), and the West Papillion Creek (fuchsia).

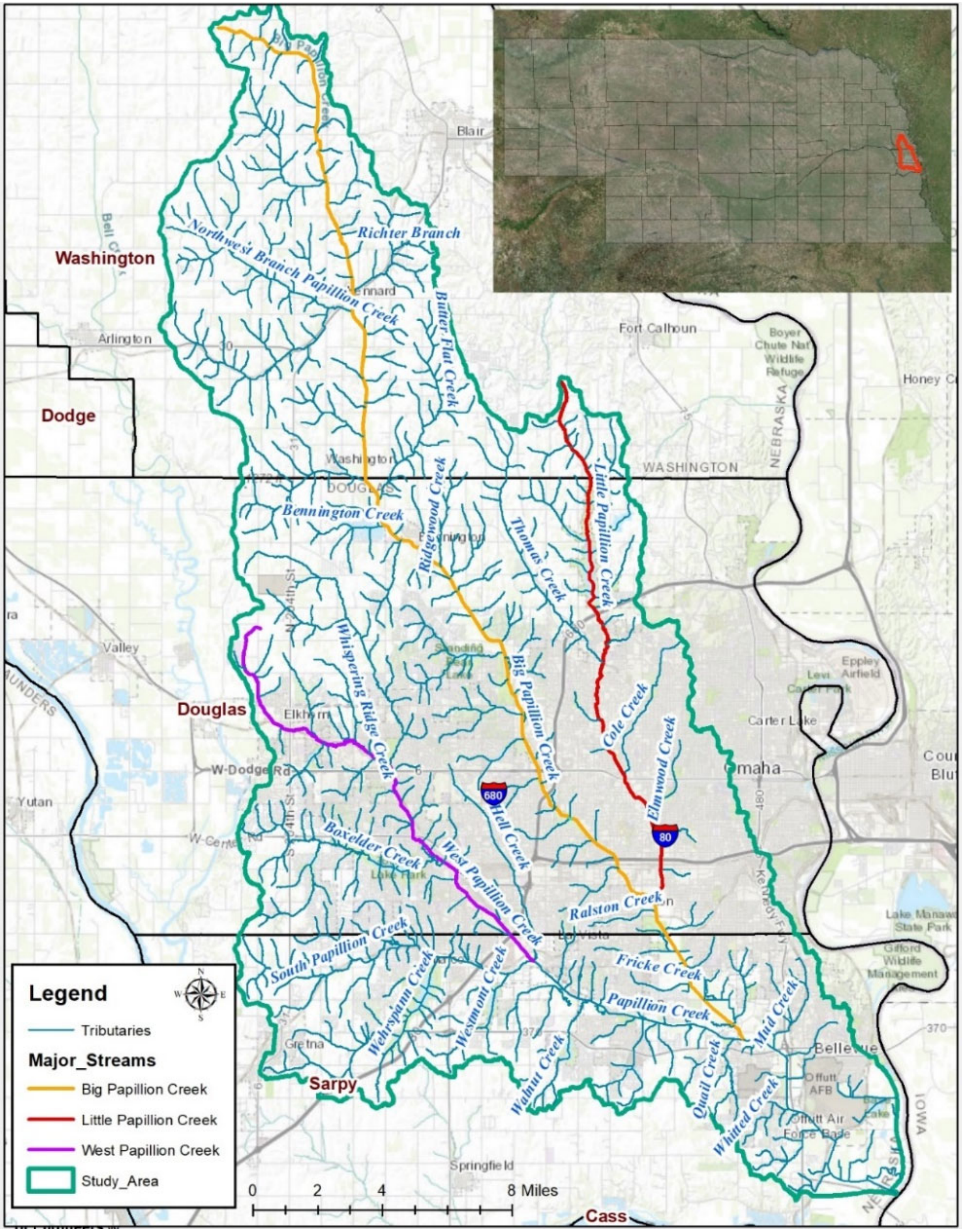

Figure 1: The three major streams in the Papillion Creek Basin.

2.2. TRIBUTARY CREEKS

Major tributaries on both sides of the Missouri River flow throughout the year, including Papillion Creek. Over time, channel modifications, levee construction, and other modifications have been made to reduce risk during flood events. Numerous small streams and secondary tributaries drain the surrounding area on both sides of the creek. Third and fourth order tributaries of these streams finger outward into the bluffs. These minor tributaries are dry most of the year but carry considerable water during seasonal rainfall.

The floodplain of Papillion Creek is generally flat with gentle to moderately sloping hills bordering both sides. The stream channels are well-entrenched, moderately sloped, and for the most part, manmade.

2.2.1. Little Papillion Creek

Little Papillion Creek originates approximately 5 miles north of Irvington, on the edge of the Omaha metropolitan area. The land north of Irvington is primarily agricultural ground. The Creek flows south, through Omaha (highly urbanized land) for nearly 10 miles to its confluence with Big Papillion Creek.

2.2.2. Big Papillion Creek

The Big Papillion Creek headwaters are nearly 20 miles north of Omaha, west of Blair, Nebraska. The upstream reach of the Creek extends through agricultural land except short segments along Kennard and Bennington, which are relatively small towns. South of Bennington, the land transitions to progressively more urbanized uses with more buildings and pavement. Except for the last 3 miles, the remaining 25 miles of the Creek extends through highly urbanized areas of the Omaha metropolitan area. The last three miles before the confluence with the Platte River are rural.

2.2.3. West Papillion Creek

The West Papillion Creek drainage is also a mix of agricultural and urban land. West Papillion Creek originates in the vicinity of Elkhorn, Nebraska and flows southeast to its confluence with Big Papillion Creek just downstream of 36th Street.

2.3. PAPILLION CREEK SITE DESIGNATORS

Hydrologic and hydraulic analyses provided the basis for identifying extents of stream reaches for consideration and evaluation. The streams in the watershed were divided into damage reaches for the purpose of H&H analysis. The reaches were indexed 1, 2, 3, etc., upstream to downstream for each watercourse. The first letter of the creek name was used to construct unique segment identifiers throughout: BP1 = Big Papillion Creek, reach 1. The complete list of damage reaches can be found in Appendix B, Hydraulics.

The two dam sites are designated by DS10 and DS19 based on their numbering in the 1971 USACE report, General Design Memorandum No. MPC-10, Papillion Creek and Tributaries, Nebraska. The site designators for the various reaches considered for further evaluation for the

Tentatively Selected Plan (TSP) are indicated in Figure 2. The sites that will be included in the Recommended Plan after optimization are indicated in Figure 3.

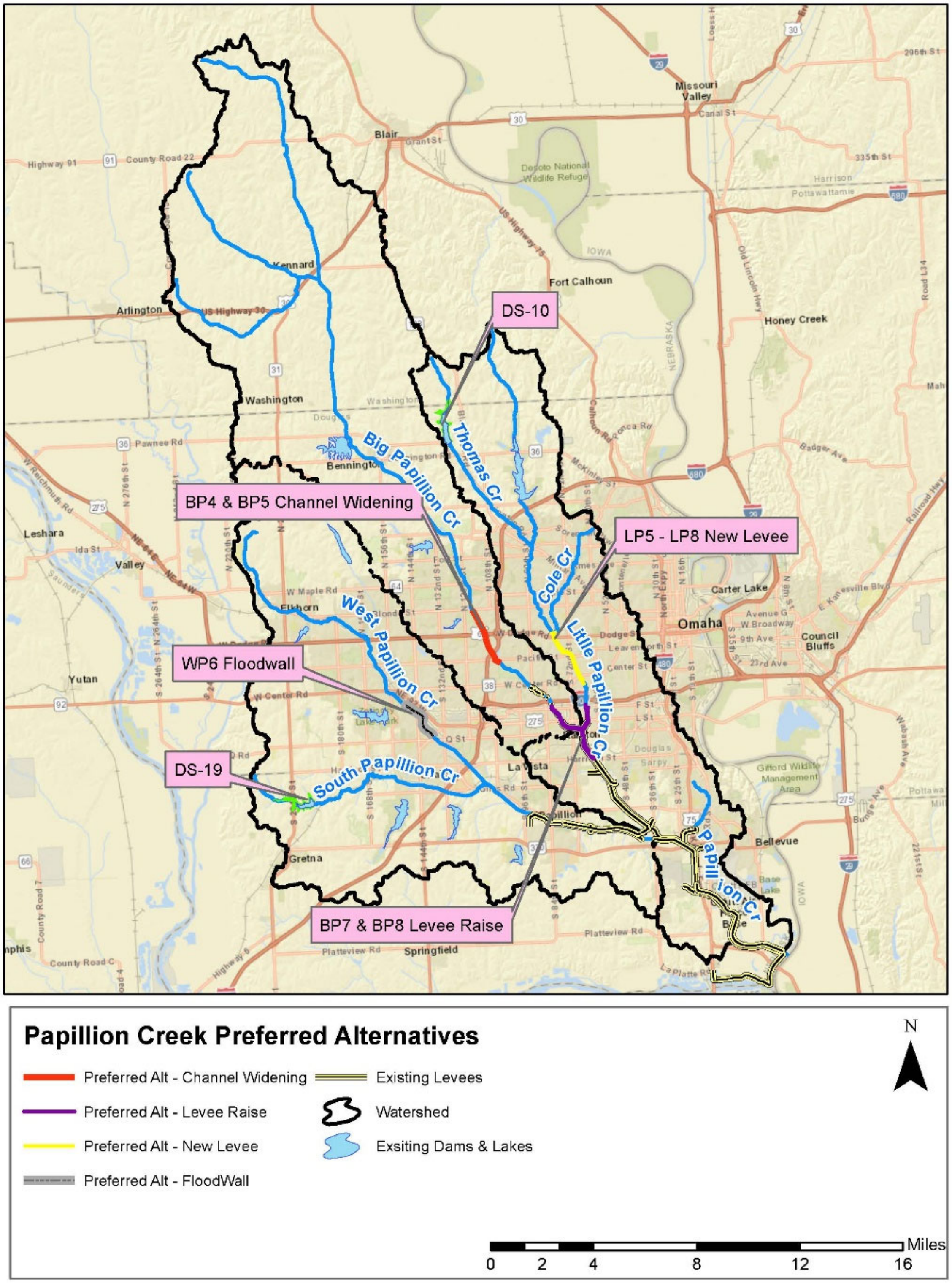

Figure 2: General location of the preferred alternatives in the TSP.

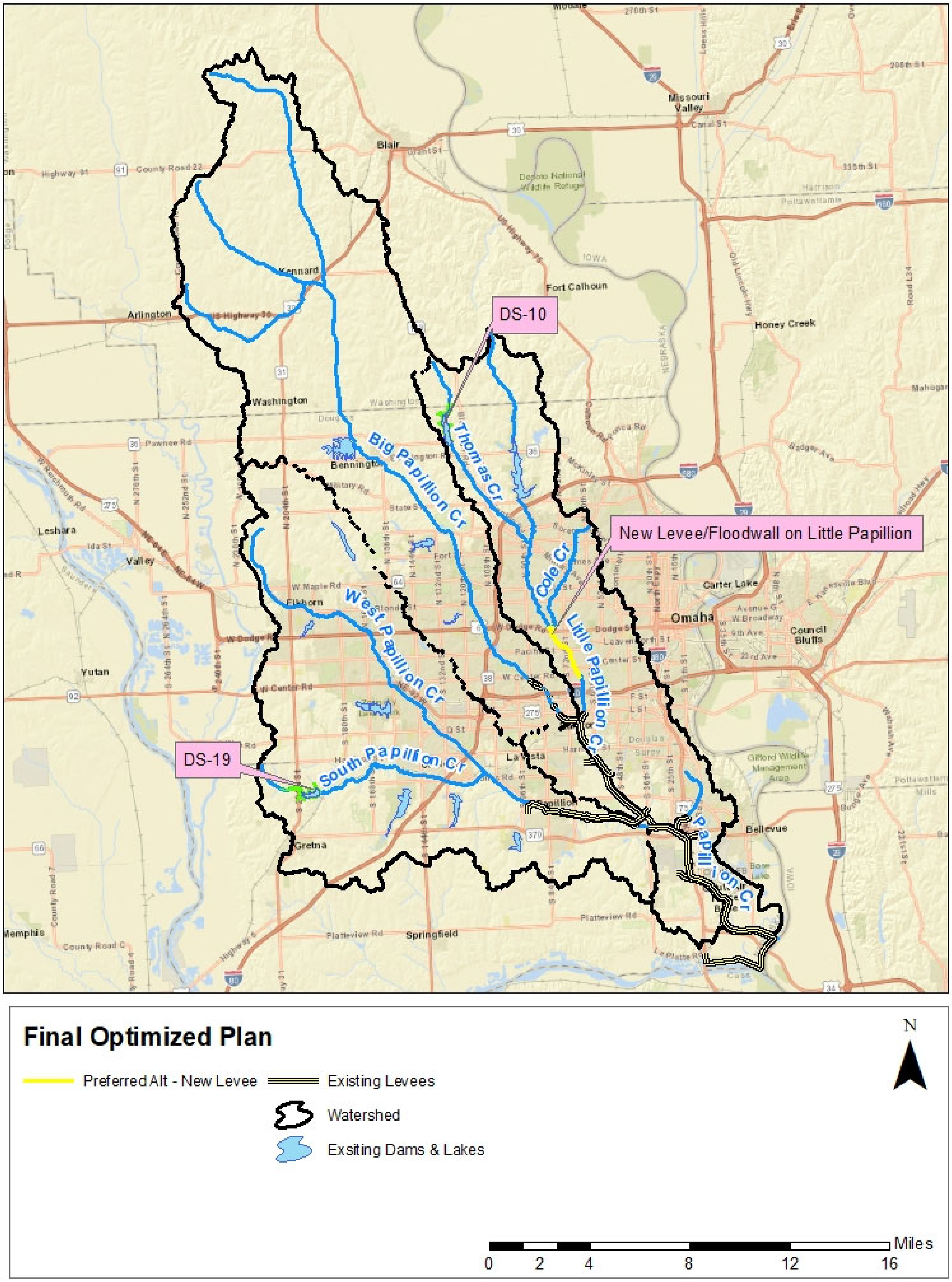

Figure 3: General location of the alternatives in the Recommended Plan after optimization.

2.4. GEOLOGIC CONDITIONS

2.4.1. Bedrock Geology

The Omaha-Council Bluffs area is in the Dissected Till Plains section of the Central Lowlands Province. Approximately 2,000 feet of sedimentary rock of Cambrian to Pennsylvanian age cover Pre-Cambrian igneous and metamorphic rock.

Structurally the area is at the eastern boundary of the Table Rock Arch, a north-south trending extension of the Nemaha anticline of northeastern Kansas. The Richfield Arch is a relatively local east-west trending structure to the south and the inactive La Platte Fault borders the area to the southwest. This fault is steeply dipping to the south and up thrown to the north.

Exposed bedrock in the area consists of interbedded limestone, siltstone, and claystone of the Kansas City Group, Missouri Series recognized as late Pennsylvanian age. Deposition is interpreted to have been in shallow open seas in near-shore waters or in lagoons and swamps. Cyclotherm sequences of limestone repeatedly overlain by siltstone and claystone are related to cycles of sea level fluctuations.

2.4.2. Regional Setting

The Omaha-Council Bluffs area consists of a broad loess-mantled upland till surface (bluffs) bisected by the Missouri River valley. The valley floor ranges in width from 3 to 8 miles. In most cases it extends from bluff to bluff as a plain that slopes gradually toward the river channel. Except for minor relief caused by meanders and oxbow lakes (i.e., Crater Lake) the plain rises in altitude about 5 to 10 feet from channel to bluff. Terraces on the margins of the valley range in height from 25 to 80 feet above the floodplain. The loess-mantled uplands lie approximately 250 feet above the valley floor.

2.4.3. Surface Geology

The surface geology in the Papillion Creek basin is dominated by pleistocene formations: Nebraskan to Wisconsin age unconsolidated deposits of glacial till with additional sand, silt, and clay of fluvial, lacustrine, colluvial, and eolian origins. Recent age deposits mapped in the area include alluvium in terraces and on flood plains, alluvial fans, and slope wash.

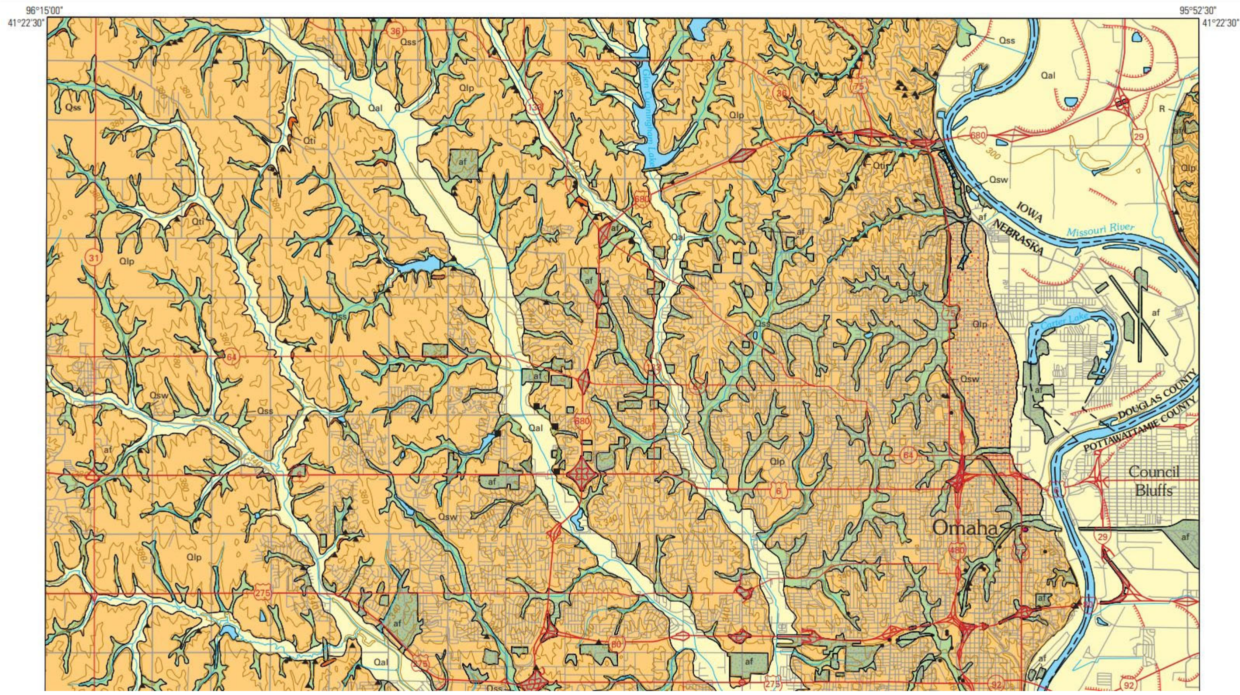

Figure 4: Surficial Geologic Map of the Greater Omaha Area (Image from 2001)

The preponderance of surface materials mapped in the basin include flood-plain and stream-channel alluvium: Qal (yellow), and Peoria Loess: Qlp (orange).

2.4.3.1. *Qal – Stream-channel Alluvium*

In the valleys of Papillion and Big Papillion Creeks and their major tributaries, Qal commonly consists of dark-brown to black clayey silt in the upper 3 to 15 feet, light brown, gray, and greenish and bluish clayey silt in the underlying 15 to 60 feet, and silty, fine to coarse sand or sand and gravel in the basal 0 to 3 feet. Clasts are angular to well rounded. They reflect the composition of the bedrock and older coarse-grained surficial deposits in the respective drainage basins. The deposits are poorly to well sorted and poorly to well stratified.

Unit Qal along minor streams tributary to the Platte River and the Missouri River north of Omaha is similar in composition to alluvium along Papillion and Big Papillion Creeks and their major tributaries. Much of the alluvium in the upper 3 to 15 feet or more in the Papillion and Big Papillion Creeks and their major tributaries is probably Holocene in age. Some of the underlying alluvium probably is Wisconsin in age, and some of it may be pre-Wisconsin in age.

2.4.3.2. *Qlp – Peoria Loess*

The Peoria Loess is massive, calcareous or non-calcareous, pale-yellow to light-yellowish brown, wind-deposited clayey silt (silt loam). Peoria Loess locally stands nearly vertically in road cuts and stream cuts, and locally it has columnar joints. The grain-size distribution for 14 samples of Peoria Loess in and near Omaha average 7 percent sand (0.063–2 mm), 74 percent silt (0.004–0.063 mm) and 19 percent clay (<0.004 mm) (Miller, 1964). Peoria Loess mantles extensive areas of older loess and pre-Illinoian till (Qti) on valley sides and uplands, and it mantles late Wisconsin terrace alluvium in and near Omaha, Bellevue, and Springfield, Nebraska (Miller, 1964). Locally in eastern Nebraska and western Iowa, the lower part of the Peoria Loess has structures that may have been produced by solifluction, indicating the possible former presence of permafrost. The Peoria Loess overlies thin (generally 3 to 5 feet) loess of the Gilman Canyon Formation in Nebraska.

Peoria Loess is prone to slumping on steep slopes, and disturbed and sparsely vegetated areas are prone to gullying and sheet erosion. The thick, dark deposits of clayey silt beneath flood plains adjacent to Papillion and Big Papillion Creeks (Qal) and beneath flood plains and in terraces along their tributaries (Qal and Qss) may have been derived in part from soil eroded from Peoria Loess surfaces.

Deposition of Peoria Loess began approximately 24,000 yr. B.P. and ended approximately 11,000 yr. B.P. Deposition in the Omaha area began approximately 25,000–24,000 yr. B.P. and ended approximately 12,000 yr. B.P. Unit Qlp at upland sites north of the Platte River is thickest in bluffs adjacent to the flood plain of the Missouri River (generally 55 to 95 feet). It is slightly thinner on adjacent late young alluvium

2.4.3.3. Kansan Glacial Drift

The older Pleistocene deposits of the Kansan and Nebraskan glacial stages underlie the entire basin but are not exposed at the surface. In general, most of the glacial material encountered is Kansan in age; however, this does not preclude the possibility that some Nebraskan age deposits may be encountered at the lower elevations. As the Nebraskan age deposits are very similar to those of the Kansan, with no differences in the foundation properties from an engineering viewpoint, all glacial deposits will be considered Kansan for simplicity. Cretaceous sandstones or Pennsylvanian limestones and shales form the bedrock surface underlying most of the Papillion Creek drainage basin; however, they occur at a depth that was not a factor in design or expected to be encountered during construction of the projects.

2.4.4. Surface Soil Mapping

The geologic conditions in the Papillion Creek watersheds are complex. The stratified glacial, glacio-fluvial, and alluvial materials beneath the levees and exposed in the Creek banks will vary in unit weight, permeability, and soil strength. Site specific explorations and information is needed to characterize site soils and geotechnical conditions at the various sites included in the Tentatively Selected Plan (TSP).

While the geologic conditions are complex, they also span the width and breadth of the basin. Surface mapping by the Natural Resources Conservation Service (NRCS) on the US Department of Agriculture Web Soil Survey website confirms that conditions are relatively consistent across the Papillion Creek basin: https://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm

Map data for the basin indicates:

- The predominant surface soils along the creek channels include silt loam, silty clay loam.

- These materials classify as a low plasticity clay (CL) with more than 97% passing the #40 sieve and more than 95% passing the US No. 200 sieve.
- The Plasticity Index (PI) varies from 15 to 23.
- It appears the predominant soils mapped along the creek channels are generally classified as resistant to erosion – with a grass cover. (Note that almost all the materials more than tens of feet away from the channel are typically erodible or highly erodible, even with grass cover.)

These and similar materials will be common throughout the basin, especially along the levees and surfaces at the top of the banks of the creek channels. However, the mapping is limited to near-surface materials and is assumed to be valid for a vertical distance of 5 feet from the tops of the creek banks.

2.4.5. Seismic Hazards

The 2014 State of Nebraska Hazard Mitigation Plan describes the seismic hazard in the state and shows that between 1866 and 1990, 51 earthquakes occurred in Nebraska. The strongest of which occurred in the southeast half of the state. Most of the damages occurred in Columbus, which is in Platte County. Historically the only damages from earthquakes have been minor; cracked plaster, broken windows, and damage to chimneys. No casualties or fatalities have been recorded.

Faulting in Nebraska is concentrated near the southeast corner of the state. Fault map information from 1990 is included in the Hazard Mitigation Plan:

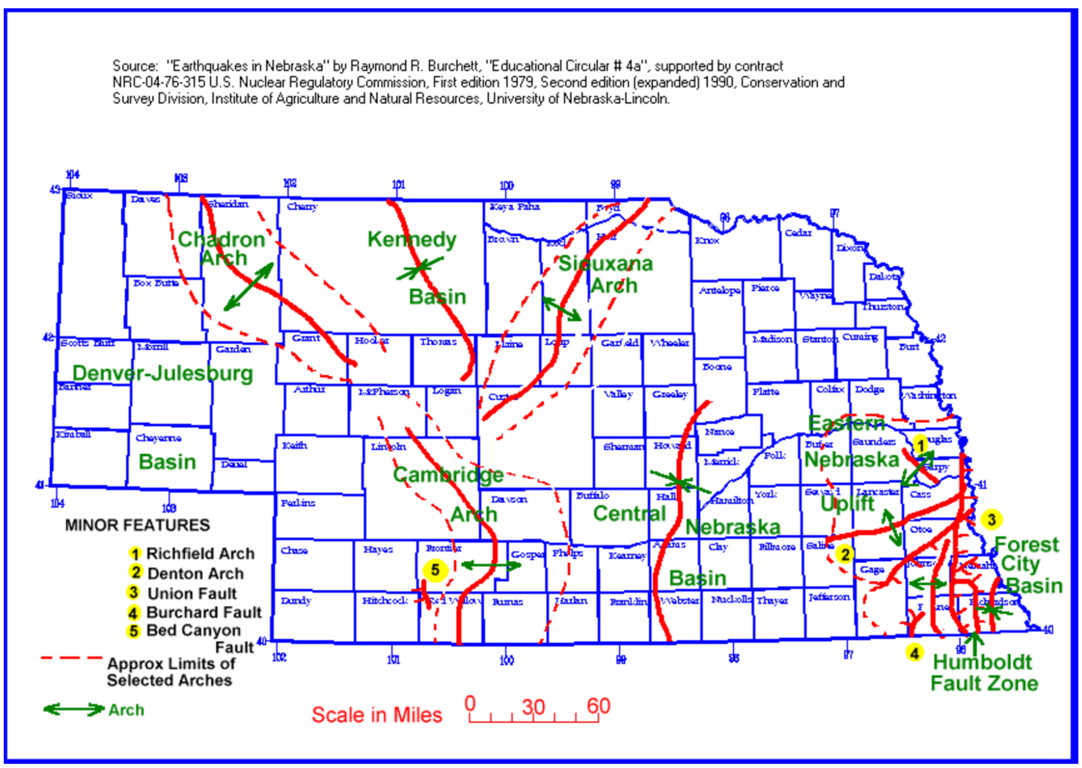

Figure 5: Earthquake fault map for Nebraska

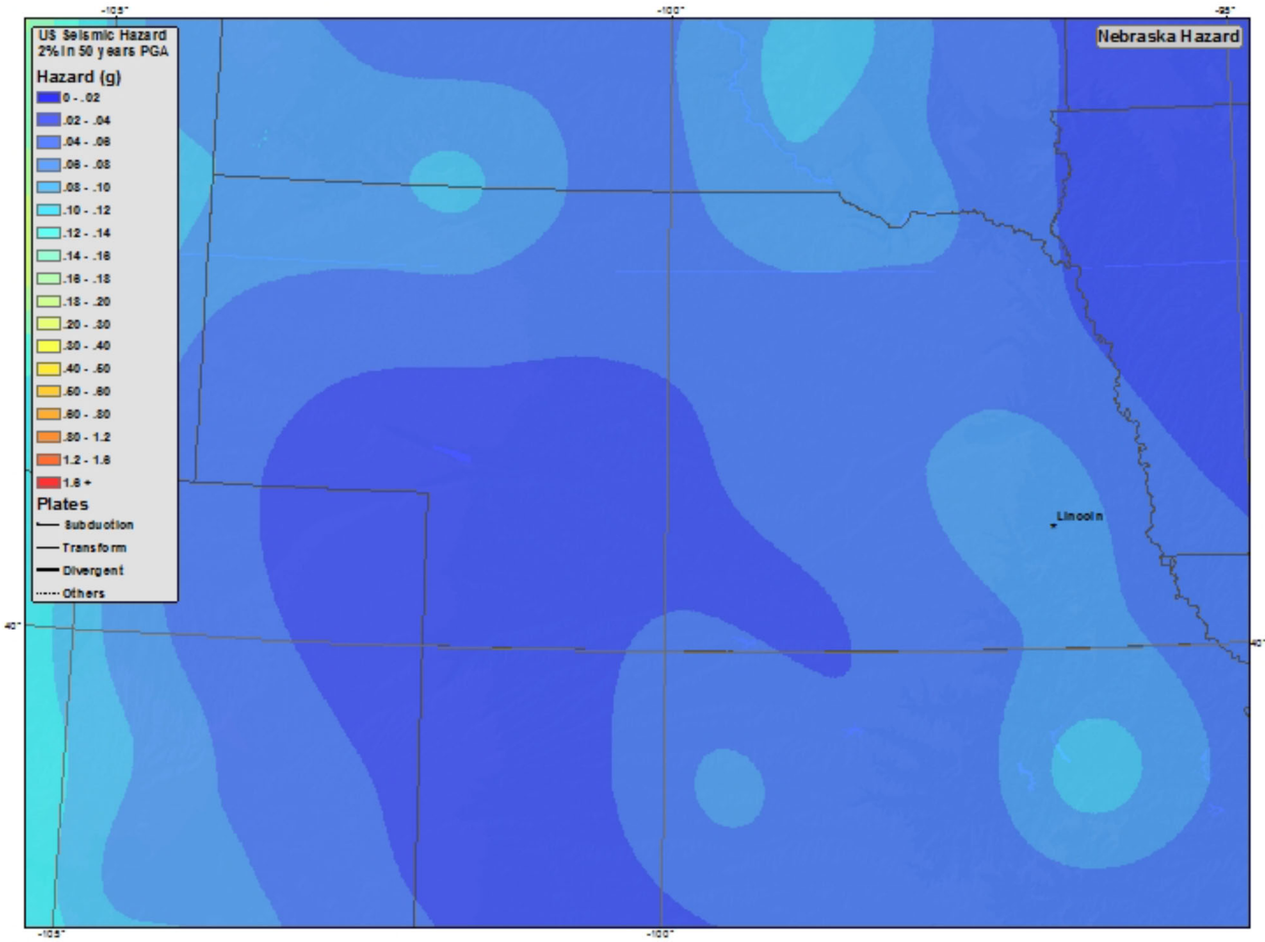

Figure 6: Seismic Hazard Map for Nebraska (from 2014 Nebraska Hazard Mitigation Plan)

Earthquakes in Nebraska are expected to cause only minor earth shaking with minimal damage to infrastructure and buildings. The United States Geographical Service (USGS) has rated earthquakes and earth-shaking events in Nebraska to be of only "a moderate concern". Unless an earthquake of greater magnitude than has historically occurred or is predicted to occur happens, there should be no impact on continuity of operations for critical services or emergency responders.

According to the USGS the first recorded earthquake occurred on November 15, 1877 near Norfolk, Nebraska. This is also the largest earthquake in Nebraska history, at a magnitude of 5.6. A 5.1 magnitude earthquake occurred on the Nebraska-South Dakota border, northwest of Merriman, Nebraska on March 28, 1964.

The USGS earthquake mapper shows 127 earthquakes within or near the Nebraska state border, as shown in Figure 7 below. The most recent recorded earthquake was a 2.8 magnitude on January 13, 2021 near Esbon, Kansas. In 2018 a cluster of 27 earthquakes occurred southeast of Arnold, Nebraska. These earthquakes ranged in magnitude from 2.0 to 4.1.

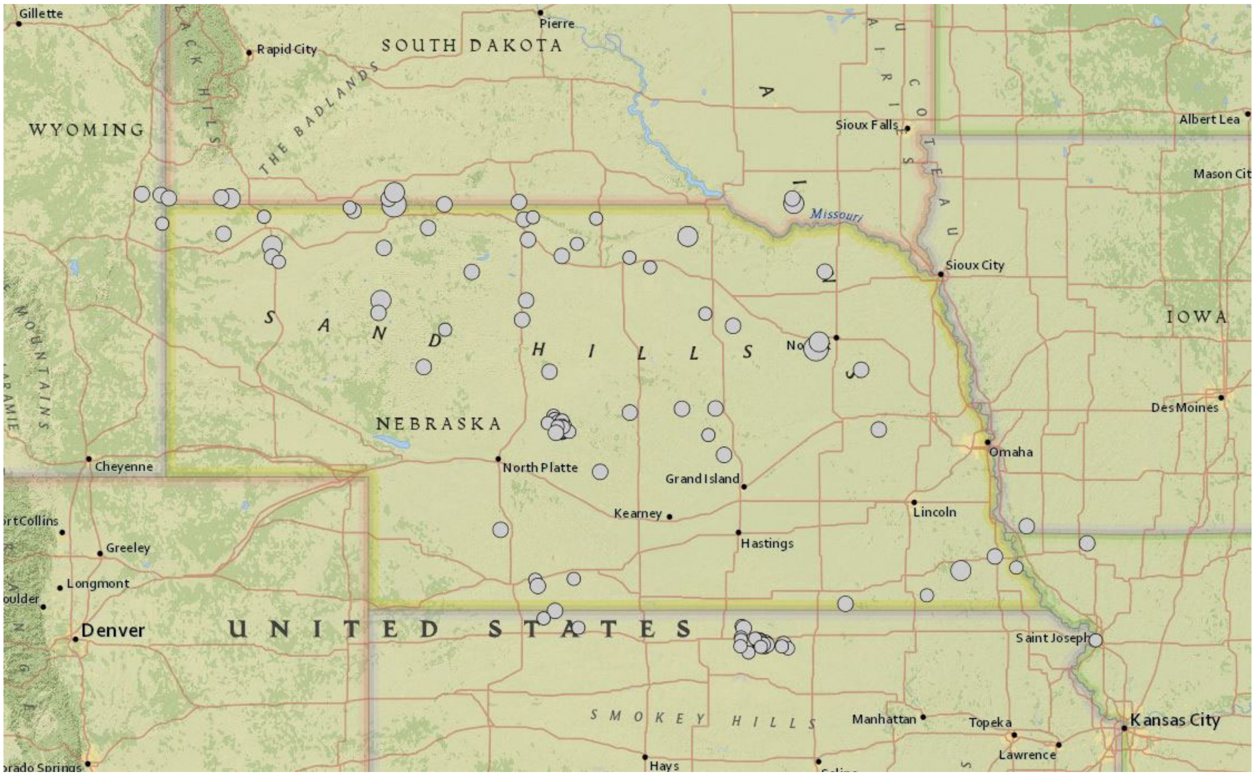

Figure 7: Earthquake Locations in Nebraska (from USGS Interactive Earthquake Mapper)

Mapping information indicates the Papillion Creek basin is not subject to significant earthquake risk. Earthquakes that do occur in Nebraska are often minor. In the rare occasions of significant ground movements, only minimal damages to roads, buildings, and other structures occurs.

Site specific seismic loading information was not developed for DS10 and DS19. However, the seismic loading for DS10 is expected to be comparable to the loading developed for existing Papillion Creek Dam Site 11 (DS11) located approximately 3.5 miles southeast of DS10. Additionally, the seismic loading for DS19 is expected to be comparable to the loading developed for existing Papillion Creek Dam Site 20 (DS20) located approximately 3.5 miles east of DS19. Table 1 provides seismic loading information developed for DS11 and DS20.

Table 1: Seismic loading information

USACE Design Earthquake	Return Period (years)	DS11 (near DS10)	DS20 (near DS19)
		PGA (g)	PGA (g)
Long return period earthquake	10,000	0.08	0.13
Intermediate earthquake	4,950	0.05	0.08
IBC “maximum considered earthquake”	2,475	0.04	0.06
Maximum design earthquake (MDE)	950	0.02	0.03
Operating basis earthquake (OBE)	145	0.01	0.01

3. GEOTECHNICAL CONDITIONS

3.1. SOURCES OF GEOTECHNICAL INFORMATION

There is not enough geotechnical exploration and testing information on the Little Papillion Creek to allow preliminary design of the proposed levee and floodwall sections to be completed based on site specific explorations and testing. Levee fill will rely on large quantities of imported materials, which could potentially be excavated from the reservoir area of a dam. Soil mapping by the NRCS provides information about surficial materials and confirms that conditions throughout the basin are generally consistent. This, in turn, is consistent with detailed information from two primary sources:

- General Design Memorandum No. MPC-52, Big Papillion Channel Improvement, Papillion Creek and Tributaries Lakes, Nebraska, USACE dated January 1989, and
- Feature Design Memorandum No. MPC-53, Volume 1, Channel Control Structure, Big Papillion Channel Improvements, Papillion Creek and Tributaries Lakes, Nebraska, USACE, dated March 1991.

Note that the Channel Control Structure is the concrete chute beneath the railroad crossing of Big Papillion Creek.

Preliminary designs for DS10, completed by USACE in 1975, and DS19, completed by engineering firm HDR in 2018, were based off geotechnical investigations including subsurface investigations and laboratory testing. No further geotechnical exploration and testing information was completed for DS10 and DS19 design updates for this feasibility report. The following preliminary design documents for DS10 and DS19 were the primary sources of geotechnical information:

- Specific Design Memorandum No. MPC-33, Site 10, Papillion Creek and Tributaries Lakes, Nebraska, USACE dated May 1975, and
- Engineering Preliminary Design Report, Dam Site 19 and Associated Improvements, West Papillion Creek Subwatershed, HDR Engineering, Inc., dated April 2018.

3.2. DAM SITE 19

A total of 36 borings were completed during the 2018 design completed by HDR Engineering and documented in the following report:

- Engineering Preliminary Design Report, Dam Site 19 and Associated Improvements, West Papillion Creek Subwatershed, HDR Engineering, Inc., dated April 2018.

These borings ranged from 10 to 110 feet in depth, penetrating four types of material: alluvium, loess, red cloud formation, and glacial drift. Disturbed soil samples from the borings were obtained using push and drive sampling, and undisturbed samples were obtained with thin-walled tube samplers: 3-inch outside diameter, hydraulically pushed in general accordance with ASTM D 1587, "Standard Practice for Thin Walled Tube Sampling of Soils for Geotechnical Purposes.” at prescribed intervals.

Split-barrel samples, designated "S" samples, were obtained while performing Standard Penetration Tests (SPT) with a thick walled sampler, 1.5-inch inside diameter, driven in general accordance with ASTM D 1586, "Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils." The N-value, reported in blows per foot, represents the number of blows required to drive the sampler over the last 12 inches of the 18-inch sample interval.

Laboratory tests were performed on disturbed and undisturbed samples in accordance with current ASTM or state-of-the-practice test procedures. The foundation soils were tested to determine moisture content, dry density, plasticity, grain size distribution, shear strength (UU triaxial and unconfined compression tests) and consolidation properties. Soils that will be excavated for potential use as borrow material for the Main Dam were tested to determine moisture content, dry density, plasticity, grain-size, and moisture-density relationship.

The foundation materials encountered during the explorations are summarized in Table 2. Detailed boring records and laboratory test results for DS19 can be found in Appendix A, "Main Dam and Water Quality Basins" of the Engineering Preliminary Design Report for Dam Site 19 prepared by HDR Engineering.

For geological cross-sections, refer to Attachment 1 – DS19 Background Information in Section 8.1.

Table 2: Summary of DS19 Foundation Materials

Unit	USCS	Consistency Description
Alluvium	Lean clay (CL) and Fat clay (CH)	Soft to Very Hard
Loess	Mostly lean clay (CL)	Very Soft to Very Hard
Glacial Drift Till	Lean clay (CL) and Fat clay (CH)	Firm to Very Hard
Red Cloud Formation	Poorly graded sand (SP) and Silty sand (SM)	Loose to Very Dense

3.3. DAM SITE 10

A total of 45 borings were completed during the 1975 design completed by USACE and documented in the following report:

- Specific Design Memorandum No. MPC-33, Site 10, Papillion Creek and Tributaries Lakes, Nebraska, USACE dated May 1975

These borings ranged from 15 to 129 feet in depth, penetrating three types of material: alluvium, loess, and glacial drift. Disturbed jar and moisture samples were taken in each boring at 5-foot intervals or at change of material, whichever occurred first. Standard penetration blow counts, using the rope and drum method, were made in representative borings within the floodplain and outlet works area. Undisturbed Shelby tube samples were taken at prescribed intervals in representative holes in the alluvium and loess.

Laboratory tests were performed on disturbed and undisturbed samples in accordance with procedures in the Laboratory Soils Testing Manual EM 1110-2-1906 dated November 1970. This testing consisted of classification by mechanical analyses and Atterberg limits of jar and undisturbed samples. Moisture was determined on materials from moisture tins. Undisturbed testing consisted of unconsolidated-undrained "Q" tests, consolidated-undrained "R" tests, direct shear “S” tests, and consolidation tests. Remolded testing consisted of compaction, classification, and "Q", "R" and direct shear tests on material to be used for embankment fill. Loess testing consisted of collapse consolidation, and density tests. The foundation materials encountered during the explorations are summarized in Table 3. Detailed boring records and laboratory test results for DS10 can be found in the 1975 design report.

Table 3: Summary of DS10 Foundation Materials

Unit	USCS	Consistency Description
Alluvium	Mostly lean clay (CL)	Soft to Stiff
Loess	Mostly lean clay (CL)	Medium Stiff to Stiff
Glacial Drift Till	Sandy clay (CL-CH)	Very Stiff

For geological cross-sections and testing results, refer to Attachment 2 – DS10 Background Information in Section 8.2.

3.3.1. Alluvium Foundation

The alluvium foundation material consists of three layers: a top stratum of medium stiff to stiff, a middle stratum of soft to medium stiff, and a lower stratum of medium stiff to stiff alluvium materials. One “Q” test was performed on the top stratum, on six samples of the middle stratum and on nine samples in the lower stratum. “R” and “S” tests were also conducted on some samples of the alluvium materials which were classified as either lean clay (CL) or fat clay (CH).

3.3.2. Loess

Dry densities of the loess at site 10 ranged from 88 pcf to 102 pcf, moisture contents were consistently above the plastic limits and generally at or about 20 percent, per the test results conclusion included in Specific Design Memorandum No. MPC-33. “Q” and “R” tests were conducted on undisturbed loess samples and the strengths were found to be much higher than the foundation alluvium material.

3.3.3. Glacial Drift

“Q”, “R”, and unconfined compression tests were performed on the glacial drift samples. As the test results indicated much higher strengths than the alluvium material, no further analysis was completed for the 1975 report.

3.3.4. Remolded Embankment Material

Sack samples taken from investigations in the proposed spillway area were tested with compactions tests, triaxial compression and direct shear tests. Two types of loess were encountered and separated into composite “A” and composite “B” based upon Atterberg limits. The two composites showed similar results when under similar moisture and density conditions.

3.4. CONDITIONS AT UPRR CROSSING

The Union Pacific Railroad (UPRR) crossing with the concrete channel for Big Papillion Creek is located near the middle of the Papillion Creek watershed. Surface geologic mapping indicates conditions are generally consistent across the basin. Based on this, it is reasonable to consider the information collected for the channel control structure to be sufficiently applicable for preliminary geotechnical analyses of levee raises, and new levees or floodwalls.

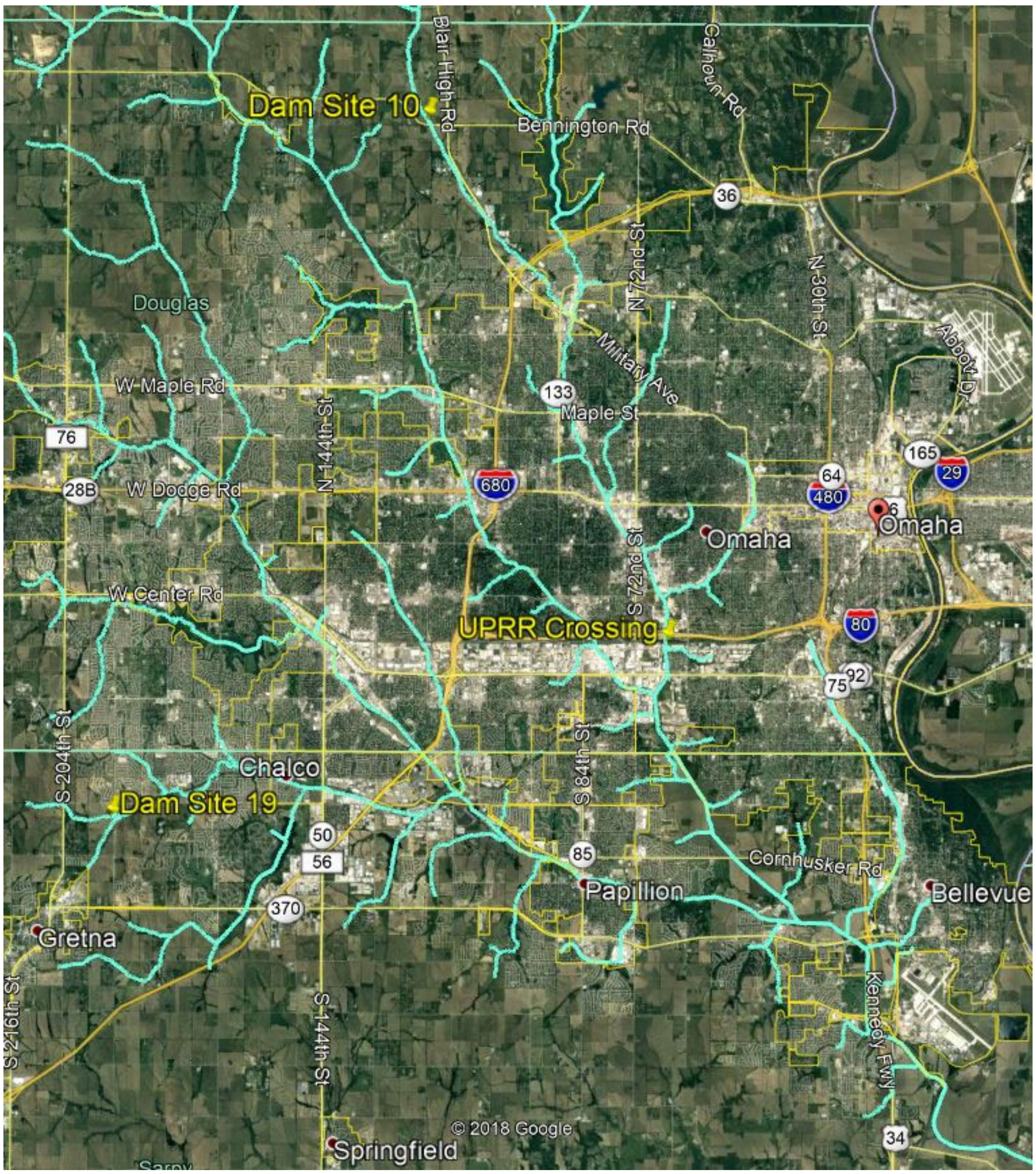

Figure 8: Vicinity Map showing location of the UPRR crossing of Big Papillion Creek

3.4.1. Borings at the Top-of-Slope

Geotechnical explorations were completed for construction of a concrete-lined channel extending beneath the Union Pacific Railroad (UPRR) bridge near 84th Street and I-80. The explorations are documented in the Feature Design Memorandum, which is dated March 1991.

Boring logs indicate subsurface conditions consist of loosely compacted fill underlain by clays, with horizons of sands and gravels covering limestone, shale, and claystone bedrock. Near the bridge, fill depths are approximately 30 to 40 feet at the farthest points from the channel to less than 10 feet at the channel edge.

Underlying native clay soils are approximately 40 to 45 feet thick and consist of soft to medium stiff, moist to saturated, dark brown to black lean clay (CL), fat clay (CH), and silty clay (CL).

Sand and gravel horizons vary from 0 to 5 feet thick and occur immediately above bedrock, usually interbedded with the overlying clays. The sand (SP-SW) is loose to medium dense, saturated, poor to well sorted and tan to gray in color. Minor constituents include silt, clay, and gravel. Gravels (GP) generally occur below the sand unit but are occasionally are interbedded.

Bedrock consists of alternating layers of limestone, siltstone (shale), and claystone. The limestone is light to dark gray to black, massive bedded, hard, and occasionally argillaceous, crystalline, and fossiliferous. Limestone beds range from 1 to 10 feet thick. Thin to thick fossil horizons contain crinoids and brachiopods. Occasional oolite beds are noted. Shale and claystone horizons were 1 to 3 feet thick, tan to black, thin to medium bedded, laminated, soft to hard, occasionally friable, argillaceous, and non-fossiliferous. Thin to thick bedded lignite or carbonaceous shale seams occur intermittently throughout the bedrock column. A large seam, 1 to 2 feet thick, was encountered at a depth of 62 feet.

3.4.2. Borings Along the Bottom-of-Channel

Borings were drilled at approximate 500-foot intervals along the drainage over 2.25 miles extending upstream and downstream of the railroad crossing. Depths of termination varied from 26.5 feet to 36.5 feet. Surface material at each boring varied based on location and ranged from tilled farm soils, slope fill, rubble, junk fill, concrete, and 4-inch-thick asphalt.

The logs indicate conditions remain generally similar along the reach. Site soils consist of alluvial lean clays (CL), fat clays (CH), and silty clays (CL) with minor clayey silts (ML) and silty fine sands (SM-SW). Soil colors ranged from light brown and gray to dark brown and gray, grayish brown, brownish gray and greenish gray. Clays were soft to very stiff, with low to high plasticity and slightly moist to saturated moisture content. Occasional thin, loose to medium dense sand horizons were present.

Silty fine sands with traces of medium to coarse sand and fine gravel were found at approximately 30 feet in some borings. These sands were loose to medium dense, saturated, and greenish gray in color. Most alluvium samples contained minor constituents of wood fragments, roots, organics, and carbon. Iron staining was observed in most samples.

3.5. GEOTECHNICAL PARAMETERS FOR PLANNING STUDY ANALYSES

3.5.1. Geotechnical Parameters for Clay Fill

The materials used for the dam embankment fill are expected to be excavated from the spillway area for each site. Most of the materials used as levee embankment fill are expected to be low plasticity clay (CL) soils from local sources. While it may be possible to obtain laboratory test results from explorations of other local levee projects it is recommended that laboratory testing be performed on the selected borrow sources during the design phase. Preliminary geotechnical analyses can be completed using the fill soil parameters in Table 4. The clay fill soil parameter ranges were selected using engineering judgement after review of available published data of common clay soil properties.

Table 4: Soil parameters of fill materials

Soil Parameter	Low	Nominal	High	Units
% passing US No. 40 Sieve	95	98	100	%
% passing US No. 200 Sieve	95	97	99	%
Plasticity	15	20	23	%
Unit Weight	97	103	110	pcf
Internal Angle of Friction	25	28	30	degrees
Horizontal Permeability	3.28E-09	3.28E-05	6.56E-05	ft/sec
Permeability Ratio (Ky/Kx)	-	0.25	-	-

Note that the data shows how the hydraulic conductivity of clay soils can span several orders of magnitude. The nominal and high "permeability" values are at the high end of the expected range. The values that would be used for analyses would likely fall between the low value and the nominal value. For discussion, the range of hydraulic conductivity would vary from 3.28E-9 to 3.28E-5 ft/sec.

3.5.2. Geotechnical Parameters for Granular Fill

Imported granular materials will be needed in significant quantities for the embankment dams for sand and gravel filters, as well as riprap. Preliminary levee and floodwall designs do not include granular fill material. Limited quantities are anticipated for filters along drainage features that may be included during the design phase, as well as riprap slope protection in high velocity areas as required. These materials are expected to be available from local sources. Analyses for planning can be based on generic design values:

Table 5: Soil parameters of gravel materials

Soil Parameter	Low	Nominal	High	Units
Total Unit Weight	111	120	130	pcf
Drained Internal Angle of Friction	30	33	38	degrees
Horizontal Permeability	9.84E-04	4.92E-02	9.84E-02	ft/sec
Permeability Ratio (Ky/Kx)	-	1.0	-	-

Table 6: Soil parameters of sand materials

Soil Parameter	Low	Nominal	High	Units
Total Unit Weight	98	107	115	pcf
Drained Internal Angle of Friction	29	30	33	degrees
Horizontal Permeability	2.95E-06	8.2E-04	1.64E-03	ft/sec
Permeability Ratio (Ky/Kx)	-	0.5	-	-

It is recommended that source specific material parameters for each material be obtained to provide a better basis for design.

3.5.3. Geotechnical Parameters for Native Soils

Reach and segment specific exploration and testing information should be used to establish parameters for geotechnical analyses when available. Previous studies provide useful information about the expected ranges of various parameters such as unit weight, internal friction angle, and soil permeability within the Papillion Creek Basin. Exploration boring logs and testing data for the existing native materials, which contributed to the selection of the parameters listed in Table 7 below, were found in the Feature Design Memorandum No. MPC-53, Volume 1, Channel Control Structure, Big Papillion Channel Improvements, Papillion Creek and Tributaries Lakes, Nebraska, USACE, dated March 1991.

The parameters listed below can be used for preliminary seepage and slope stability modeling applied over the entire project area, however these analyses were only performed for the levee and floodwall alternatives in the Recommended Plan. Geotechnical analyses were conducted for each of the dam sites during their previous preliminary design efforts and therefore were not re-evaluated for this feasibility report. More information can be found below in Section 5: Geotechnical Analyses.

Local experience and site-specific exploration information should be used to further develop the necessary design parameters for geotechnical analyses during the design phase of selected alternatives.

Table 7: Preliminary soil parameters of native materials

Material	Horizontal Permeability (ft/sec)	Total Unit Weight (pcf)	Cohesion (psf)	Internal Angle of Friction (Total Stress)
Zone A – CL	1.31E -07	123	360	27.9
Zone B – CL	3.28E -08	121	0	29.7
Zone C – CL	1.64E -06	118	0	28.4
Zone D – CL	3.28E -07	121	0	29.7
Zone E – CL	1.64E -05	121	0	29.7
Zone F – CL	3.28E -06	121	0	29.7
Bedrock - Limestone		165	0	45.0

4. ALTERNATIVES AND OPTIMIZATION

4.1. DAM SITE 19

Dam Site (DS) 19 is on South Papillion Creek, in the NE ¼ of Section 19, T 14 N, R 11 E, in Douglas County, Nebraska; immediately west of 192nd Street and 0.3 mi south of Giles Road. In Figures 9 and 10, the optimized footprint for the dam embankment is shown in green and the spillway is shown in red.

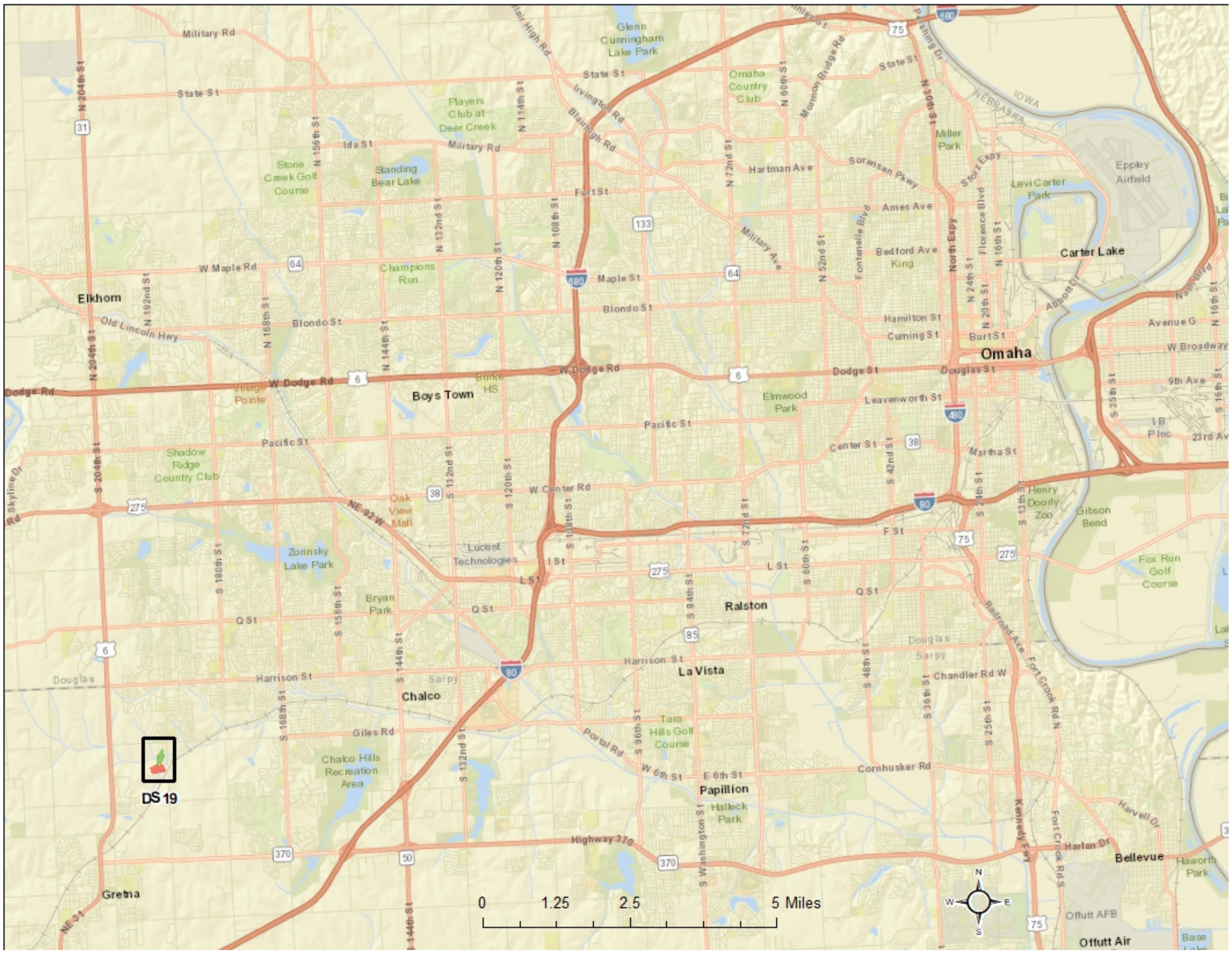

Figure 9: Location map showing Dam Site 19 in relation to the City of Omaha

Figure 10: Vicinity map showing the location of Dam Site 19

4.1.1. Basis of Preliminary Design

The preliminary design of DS19 for this feasibility report is based on an engineering preliminary design that was completed by HDR under contract to the PMR-NRD in 2018. The 2018 design includes construction of a 1,450-foot long earth embankment dam that would impound approximately 74 surface acres of water. The design includes construction of an upstream sediment control basin to manage long-term sedimentation.

The 2018 design effort relied on standards that meet most of the current USACE requirements. Current standards include EM 1110-2-2300, General Design and Construction Considerations for Earth and Rock-fill Dams; EM 1110-1-1804, Geotechnical Investigations; EM 1110-2-1901, Seepage Analysis and Control for Dams; EM 1110-2-1902, Slope Stability; EM 1110-2-1906, Laboratory Soils Testing.

The following updates to the 2018 HDR design are required to meet the current geotechnical USACE requirements:

- The 2018 design crest width must be widened from 20 feet to a minimum of 25 feet in accordance with EM 1110-2-2300.

- ER 1110-2-1156 will require the 2018 design to be updated to address the recommendations and findings from the abbreviated semi-quantitative risk analysis (SQRA) in Appendix L – Life Safety Analysis.

The embankment will be constructed on an impervious foundation; therefore, a cutoff is not required. An inspection trench with a minimum depth of 6 feet will be excavated to inspect for abandoned pipes, soft or pervious foundation zones, or other undesirable features not discovered during previous explorations. To control potential through seepage or concentrated seepage through imperfections in the impervious embankment, a continuous chimney and blanket drain system will be constructed along the entire length of the embankment. The chimney drain will be 6 feet wide with 1-foot horizontal to 1-foot vertical (1H:1V) side slopes. The blanket drain will be 3 feet wide. A representative cross-section of the dam embankment is included below.

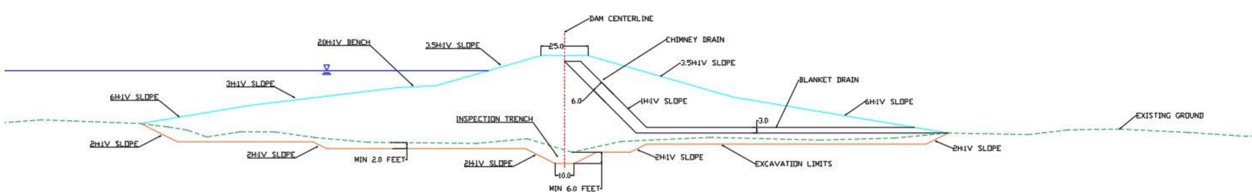

Figure 11: Typical dam embankment cross-section for DS19

The auxiliary spillway will be an unlined earthen and vegetated channel with 3H:1V side slopes. A typical spillway cross-section is shown in the figure below. The preliminary spillway profile has a crest length of 200 feet.

Figure 12: Typical spillway cross-section

4.1.2. Tentatively Selected Plan (TSP)

Since the preliminary design of DS19 by HDR in 2018 met most of the current USACE criteria, it was used to complete preliminary analyses and cost estimates for the preferred alternatives in the TSP. The 2018 design includes construction of a 1,450-foot long earth embankment dam. That design includes construction of an upstream sediment control basin to manage long-term sedimentation as well as other features. The quantities for the features included in the current HDR plan were used as a basis for the cost to determine the benefit cost ratio at the TSP phase to include DS19 for optimization. During optimization, discussed below, the required design changes needed to meet all the current USACE criteria were added and included in the cost estimate to determine the benefit cost ratio and whether to include DS19 in the recommended plan.

4.1.3. DS19 Optimization

Several alternatives were considered for DS19 including both wet and dry dams. The hydrology and hydraulic engineers provided top of dam embankment elevations in combination with a spillway crest elevation and width for each alternative during optimization. Each dam embankment and spillway combination were modeled in the civil model based on Figures 11 and 12 above, to determine the cut and fill quantities and the project footprints to assist in the optimization process. For more detailed information on how the combinations were determined and selected for the optimization process, refer to Appendix A: Hydrology and Appendix B: Hydraulics.

The dam embankment model template did not include the chimney or blanket drain to simplify the modeling process due to its iterative nature and changing top of dam elevations and resultant crest length. The quantities for the chimney and blanket drains were estimated based upon the geometry of those features and dam crest length.

Borings drilled and logged for HDR’s preliminary design of DS19 within the spillway footprint verified that the native material obtained through excavation of the spillway is cohesive and suitable for use as dam embankment fill material. The spillway will be founded on glacial drift and loess material. Since the loess is an unconsolidated, wind-blown silty-clay material and therefore more erodible than the glacial drift foundation material, any loess in the spillway foundation will be over excavated a minimum of 5 feet and replaced with compacted, impervious material.

The primary outlet works will include an uncontrolled intake tower with a low-level intake, and a 72-inch diameter reinforced concrete pipe with an inlet invert elevation of 1164.0. The 72-inch pipe is currently 400 feet in length and will outfall into a stilling basin. Based on recommendations from the SQRA team, a filter will be placed around the primary outlet conduit near the outfall. The drain material will be properly sized to meet filter criteria for permeability, particle retention, and flow. The stilling basin will be protected with riprap revetment.

The primary outlet works will be founded on glacial drift in the left abutment like the existing Papillion Creek Dam Sites. The outlet structures at the existing dam sites have performed well and have experienced minimal settlement on the glacial drift foundation. For more information on the preliminary design of the primary outlet works, refer to Appendix A – Hydrology and Appendix B – Hydraulics.

With the primary outlet works located in the left abutment rather than in the existing stream channel, additional grading will be required on the upstream side of the dam to re-direct flows into the outlet works and on the downstream side of the dam to direct the outflow back into the existing stream channel. This grading was not included in the preliminary design effort.

The results of the optimization process concluded that a combination with the best cut and fill balance was the recommended option moving forward. The selected combination is a wet dam with a top of dam elevation of 1187.7 with an auxiliary spillway crest elevation of 1177.5 and an auxiliary spillway bottom width of 550 feet. The following figures depict the optimized plans cross-sections and profiles for both the dam embankment and the spillway.

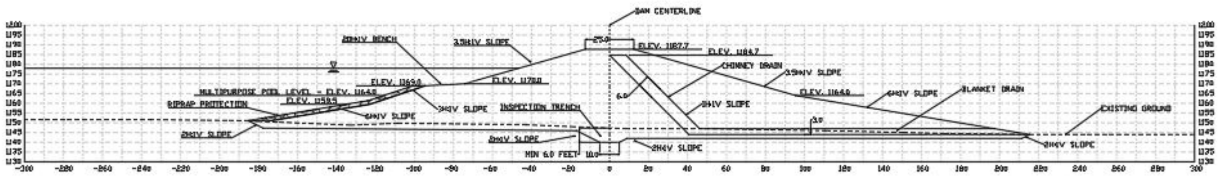

Figure 13: Optimized dam embankment cross-section for DS19

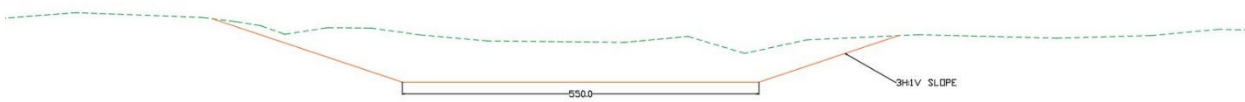

Figure 14: Optimized spillway cross-section for DS19

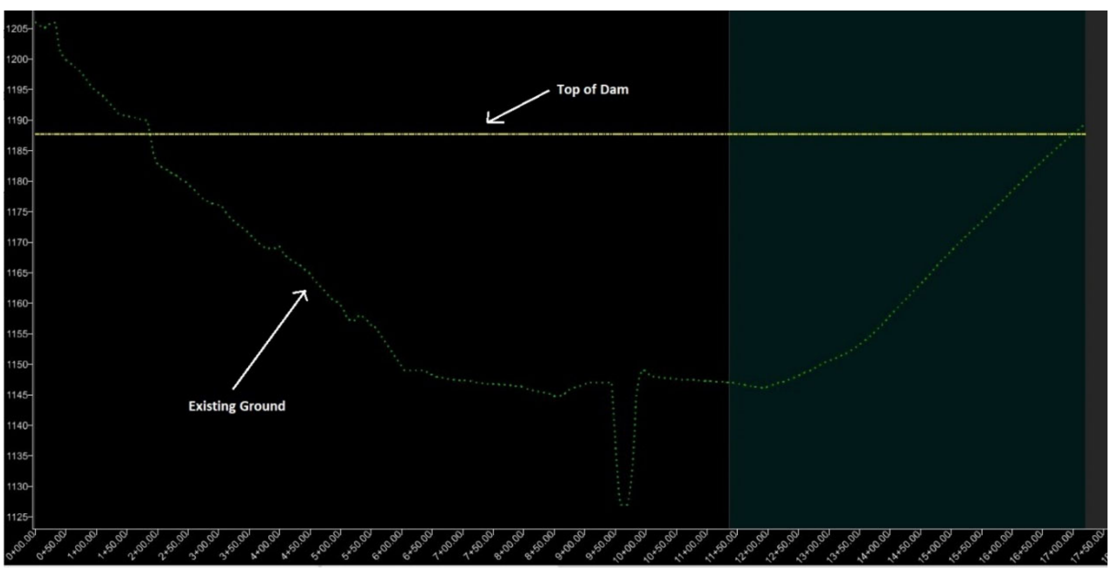

Figure 15: Optimized dam embankment profile for DS19

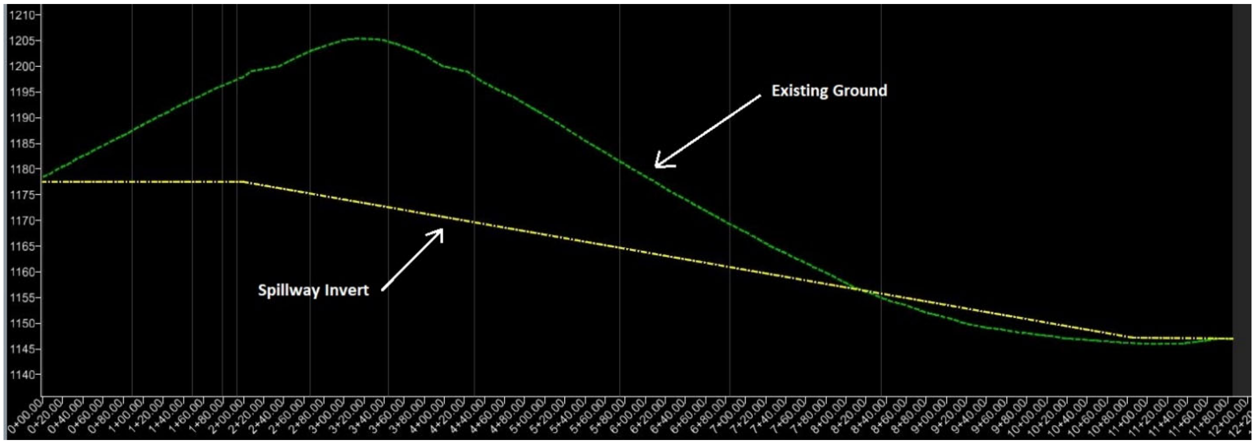

Figure 16: Optimized spillway profile for DS19

4.2. DAM SITE 10

Dam Site 10 is on Thomas Creek in the Little Papillion Creek watershed and is in the SE ¼ of Section 7, T 16 N, R 12 E, in Douglas County, Nebraska. It is 0.4 mi west of Highway 133 and 0.2 mi north of Highway 36. In Figures 17 and 18, the optimized footprint for the dam embankment is shown in green and the spillway is shown in red.

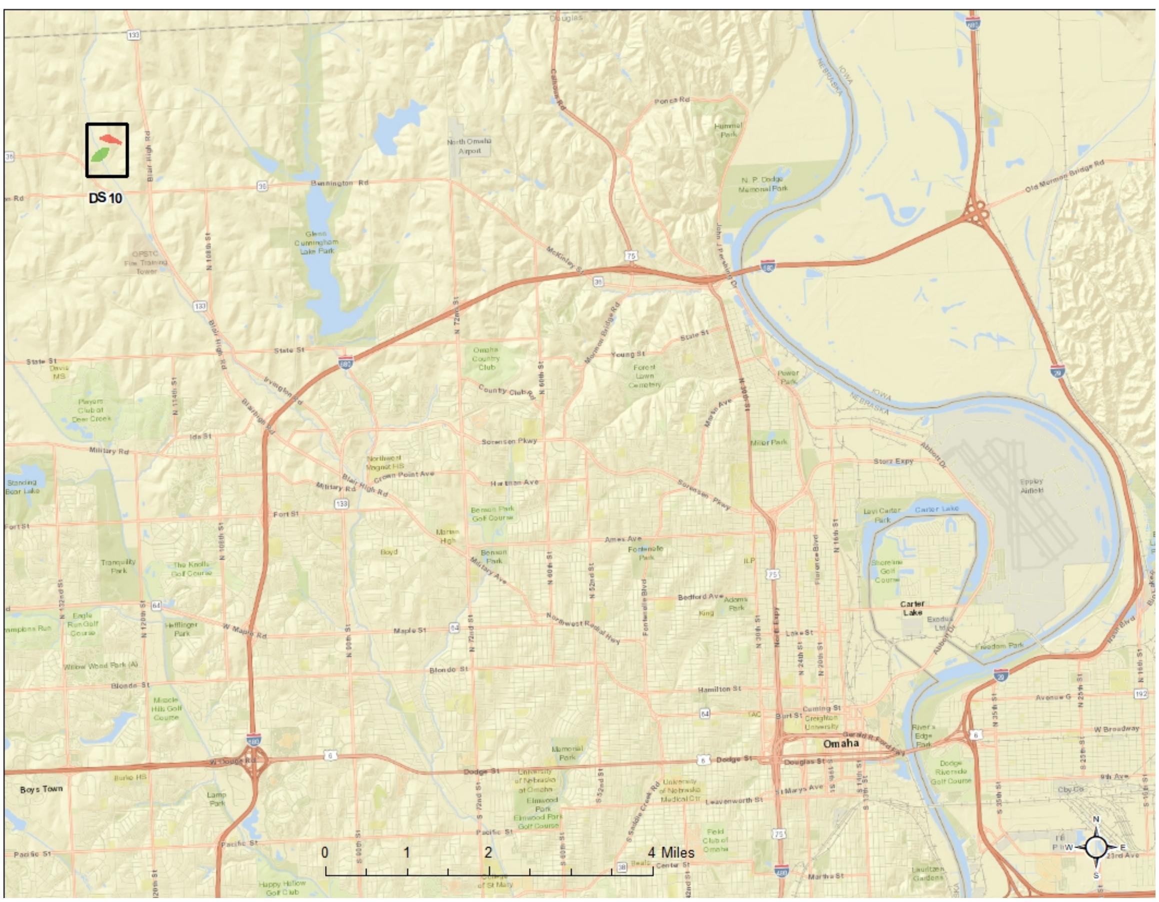

Figure 17: Location map showing Dam Site 10 in relation to the City of Omaha

Figure 18: Vicinity map showing the location of Dam Site 10

4.2.1. Basis of Preliminary Design

Preliminary design of DS10 was completed by USACE in 1975 and includes a homogeneous, impervious fill embankment with an internal, pervious drain consisting of a 6-foot-wide inclined chimney drain and a 3-foot-thick continuous horizontal sand blanket. Additional dam features from the preliminary design include an unlined, earthen auxiliary spillway (shown in Figure 20) and a concrete outlet works. The design includes flood control storage of 1,957 acre-feet to regulate the design standard project flood. The 1,140-acre-foot multipurpose pool would provide a 125-acre lake that could contain all the sediment expected to accumulate during the project's 50-year economic life.

The 1975 design effort relied on standards that do not meet current USACE requirements. Current standards include EM 1110-2-2300, General Design and Construction Considerations for Earth and Rock-fill Dams; EM 1110-1-1804, Geotechnical Investigations; EM 1110-2-1901, Seepage Analysis and Control for Dams; EM 1110-2-1902, Slope Stability; EM 1110-2-1906, Laboratory Soils Testing. The following updates to the 1975 design are required to meet the current geotechnical USACE requirements, but is not a comprehensive list:

- The 1975 design crest width must be widened from 15 feet to a minimum of 25 feet in accordance with EM 1110-2-2300.
- EM 1110-2-2300 requires enough overbuild of the embankment and inclined chimney drain to adequately account for predicted settlement after the dam crest is topped out. Additional borings and consolidation testing will need to be conducted to verify if the 2 feet of overbuild proposed in the 1975 design is enough since nearby Papio Site 11 settled 2 feet after topping out the embankment.
- A filter shall be constructed around the conduit in the downstream zone of the embankment in the updated design to meet the requirements of EM 1110-2-2300.
- ER 1110-2-1156 will require the 1975 design to be updated to address the recommendations and findings from the abbreviated semi-qualitative risk analysis (SQRA) in Appendix L – Life Safety Analysis.

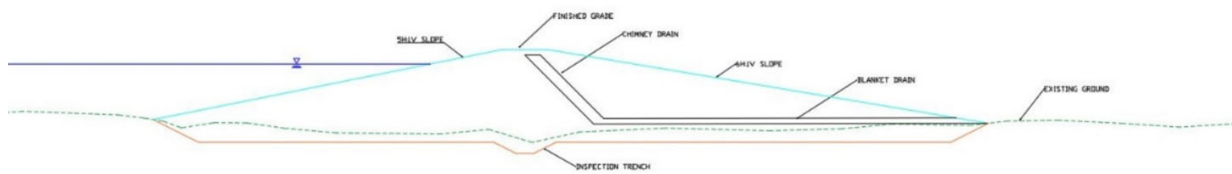

Figure 19: Typical dam embankment cross-section for DS10

Figure 20: Typical spillway cross-section

4.2.1. Tentatively Selected Plan (TSP)

The preliminary design of DS10 by USACE in 1975 was used to complete preliminary analyses and cost estimates for the preferred alternatives in the TSP. The quantities for the features included in the current plan were used as a basis for the cost to determine the benefit cost ratio to include DS10 for optimization.

4.2.2. DS10 Optimization

Several alternatives were considered for DS10 including both wet and dry dams. The hydrology and hydraulic engineers provided top of dam embankment elevations in combination with a spillway crest elevation and width for each alternative during optimization. Each dam embankment and spillway combination were modeled in the civil model based on Figures 19 and 20 above, to determine the cut and fill quantities and the project footprints to assist in the optimization process. For more detailed information on the optimization process refer to Appendix A: Hydrology and Appendix B: Hydraulics.

The dam embankment model template did not include the chimney or blanket drain to simplify the modeling process due to its iterative nature and changing top of dam elevations. The quantities for the chimney and blanket drains were estimated based upon the geometry of those features and dam crest length.

A key component of the design assumptions is that the native material obtained through excavation of the spillway will be suitable for use as dam embankment fill material. This assumption is based upon preliminary geotechnical investigation information provided in the USACE report. The spillway will be founded on glacial drift and loess material. Since the loess is an unconsolidated, wind-blown silty-clay material and therefore more erodible than the glacial drift foundation material, any loess in the spillway foundation will be over excavated a minimum of 5 feet and replaced with compacted, impervious material.

The primary outlet works is a reinforced concrete box culvert with an inlet invert elevation of 1154.0. The box culvert has an 8-foot span and a 7-foot rise, is currently 700 feet in length and will outfall into a stilling basin. Based on recommendations from the SQRA team, a filter will be placed around the primary outlet conduit near the outfall. The drain material will be properly sized to meet filter criteria for permeability, particle retention, and flow. The stilling basin will be protected with riprap revetment.

The primary outlet works will be founded on glacial drift in the left abutment like the existing Papillion Creek Dam Sites. The outlet structures at the existing dam sites have performed well and have experienced minimal settlement on the glacial drift foundation.

For more information on the preliminary design of the primary outlet works, refer to Appendix A – Hydrology and Appendix B – Hydraulics.

With the primary outlet works located in the left abutment rather than in the existing stream channel, additional grading will be required on the upstream side of the dam to re-direct flows into the outlet works and on the downstream side of the dam to direct the outflow back into the existing stream channel. This grading was not included in the preliminary design effort.

The selected combination is a dry dam with a top of dam elevation of 1207.4 with a spillway crest elevation of 1191.6 and a spillway bottom width of 100 feet. The following figures depict the optimized plans cross-sections and profiles for both the dam embankment and the spillway.

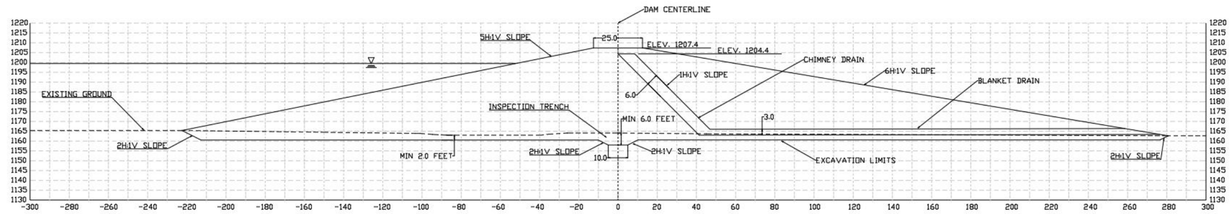

Figure 21: Optimized dam embankment cross-section for DS10

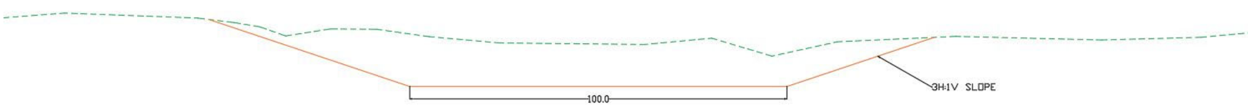

Figure 22: Optimized spillway cross-section for DS10

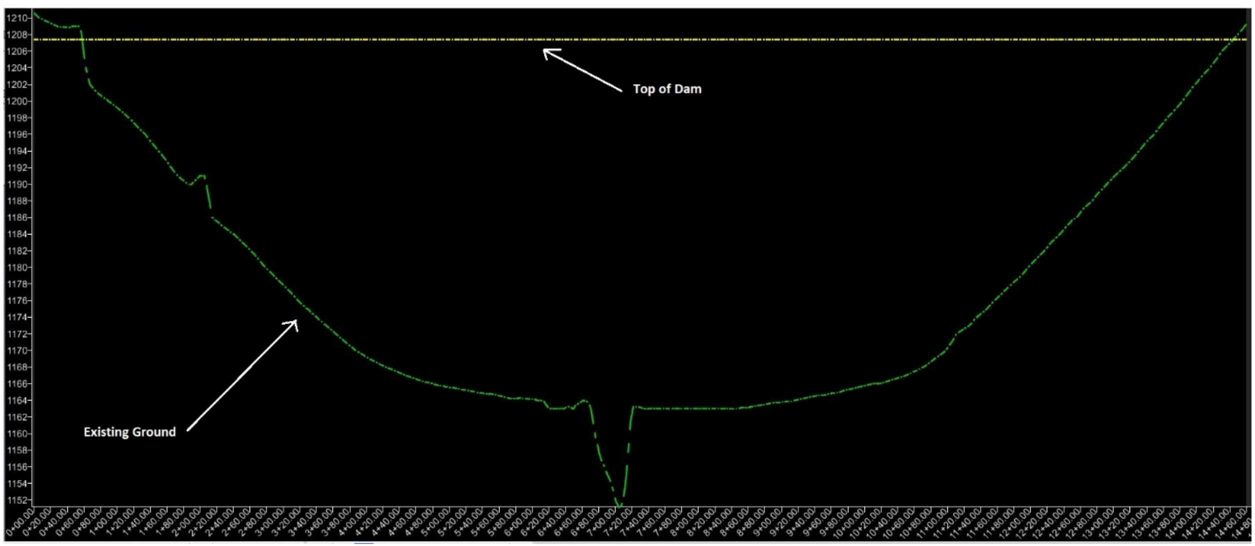

Figure 23: Optimized dam embankment profile for DS10

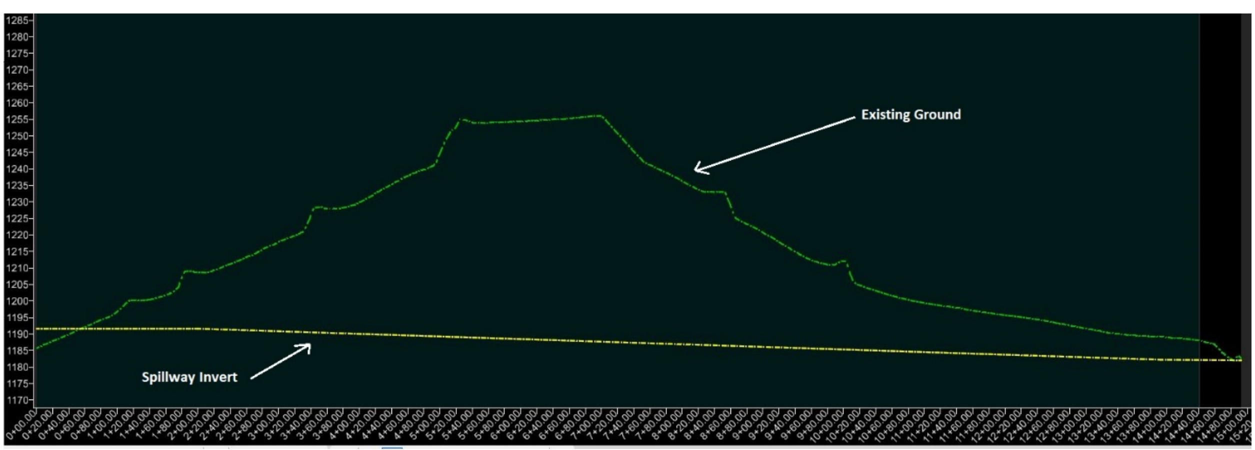

Figure 24: Optimized spillway profile for DS10

4.3. LITTLE PAPILLION CREEK

The TSP alternatives along Little Papillion Creek include:

- New levee and/or floodwall construction on both the left and right banks, in areas identified as reaches LP5, LP6, LP7 and LP8.
- Upstream tie-off areas on both the left and right banks to connect the new levees and/or floodwalls to high ground, identified as reach LP5-Up.
- Levee and/or floodwall construction with and without the construction of DS10.

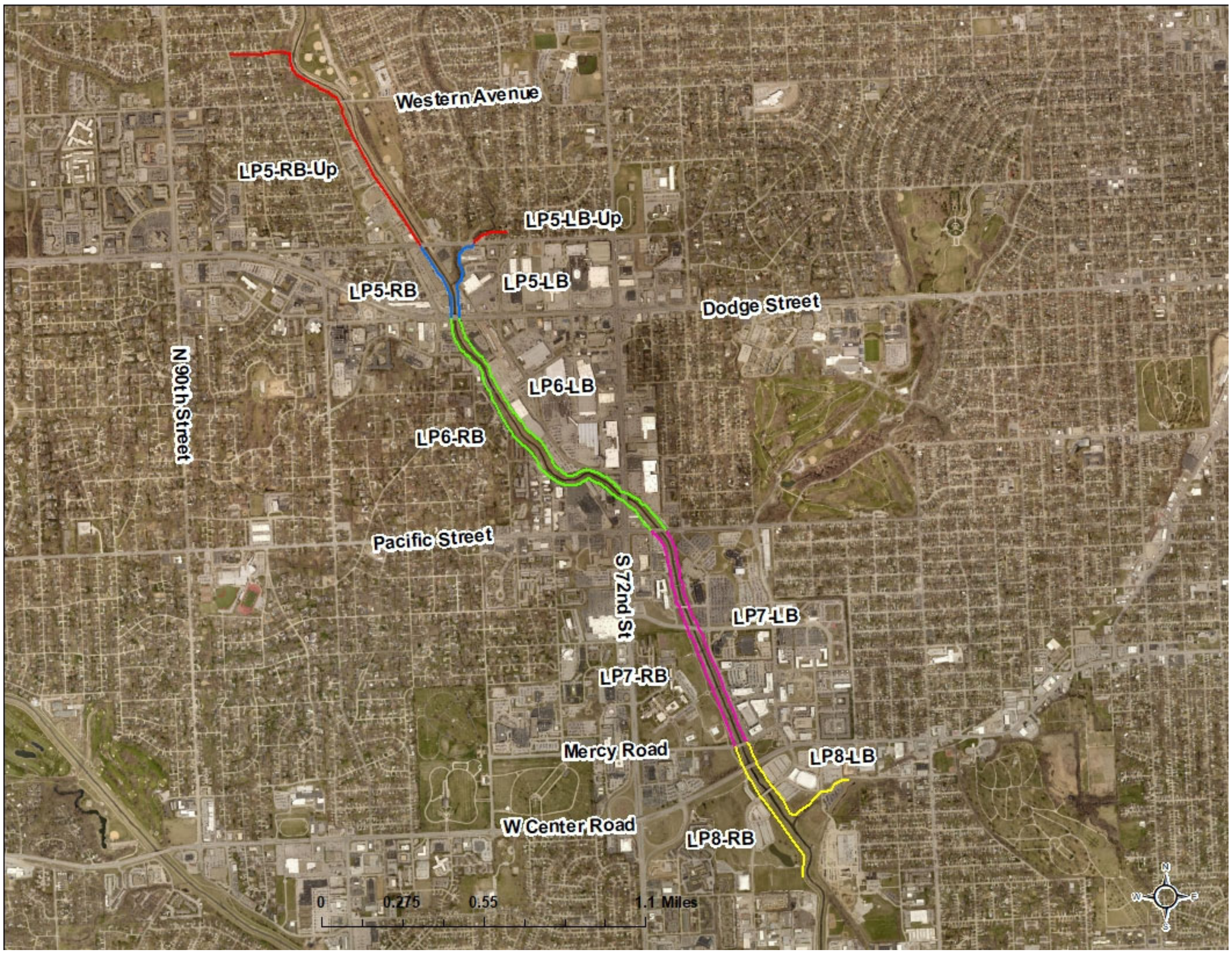

Figure 25: Aerial Photo of Little Papillion TSP Alternatives

4.3.1. Basis of Preliminary Design

The existing topography was developed from 2016 LiDAR data and shows minimal height variations on the landside of the Little Papillion Creek channel bank between Cass Street and Saddle Creek, which would indicate a lack of existing levees in this area. The National Levee Database (NLD) also does not show any existing levees within this area of the Little Papillion Creek.

Per EM 1110-2-1913, Design and Construction of Levees; new levees will be constructed as fill prisms that have a minimum 12-foot-wide crown and minimum 3H:1V landside slopes, as shown in the typical cross-section in Figure 26. Preliminary design of new floodwalls consists of an inverted T wall, a 12-foot-wide levee crown on the landside of the floodwall, and a 3H:1V landside slope, as shown in the typical cross-section in Figure 27. The preliminary T wall design was provided by structural engineers and would be a reinforced concrete section. In both cases, a 15-foot-wide vegetation free zone (VFZ) would be added on the landside, measured from the landside toe, to establish real estate and/or easement boundaries.

Per EM 1110-2-1913 an inspection trench in equal depth to the new levee height has been added, to a maximum depth of 6 feet, for new levee construction. Based upon the preliminary floodwall design, the excavation for construction of the base of the wall and forms would be 7 feet deep and essentially act as an inspection trench. Therefore, additional excavation for an inspection trench was not added to the floodwall areas.

While there are no existing levees within this portion of the Little Papillion Creek, there was a channel widening project in this area in the late 1960's to reduce flood risk. It was assumed that areas within the existing channel that required riprap protection (outfalls and drainage structure outlets, bridge abutments, etc.) would have received protection as a part of the channel widening project and are maintained as routine O&M. Therefore, riprap protection was not included in the levee and floodwall civil modeling. However, Hydraulics has identified some additional areas where velocities are expected to increase to a level that will require the addition of riprap protection and calculated the appropriate quantities for inclusion in the cost estimates. For more information on riprap protection, refer to Appendix B – Hydraulics.

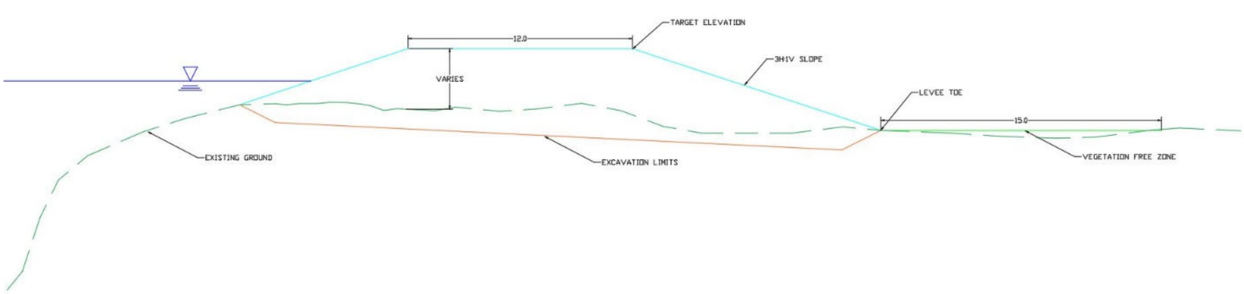

Figure 26: Typical levee cross-section

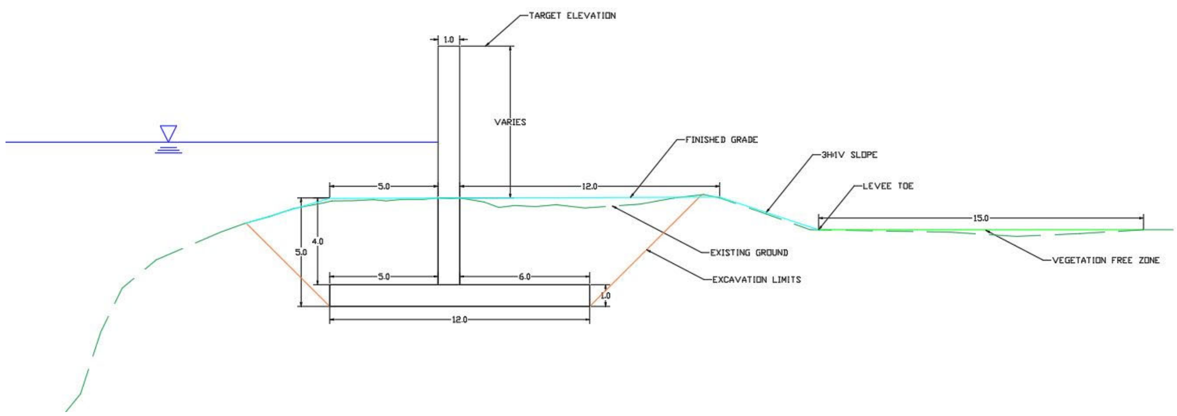

Figure 27: Typical preliminary floodwall cross-section

4.3.1. Initial analysis for TSP alternatives

The initial analysis of alternatives during the screening process used to determine the preferred alternatives in the TSP were based upon limited information and some generalized assumptions. The preferred alternatives were then optimized to determine which alternatives would be carried forward into the recommended plan based upon the benefit to cost ratios. The optimization process is discussed in detail in the next section.

The initial screening assumptions include:

- Generalized existing ground surfaces as LiDAR data had not yet been obtained
- One-dimensional steady flow hydraulic modeling to determine target elevations
- Average height raises determined from the hydraulic models
- Average height raises for each reach applied along the entire length of each reach
- Simplified I wall design for floodwalls

During the initial analysis phase and steady flow modeling, new levees and/or floodwalls along the Little Papillion Creek were hydraulically modeled separately from DS10 (without DS10 construction). After hydraulic modeling of DS10, the water surface produced from the DS10 model was used to estimate a target levee/floodwall height for a with DS10 construction scenario. Due to the changes in the flow dynamics and water surface with DS10 construction the estimated height raises were lower than the without DS10 construction modeled height raises.

4.3.2. Optimization

It was determined prior to the optimization phase that based upon the required elevation raises and number of closure structures at existing bridges that a new levee or floodwall project on Little Papillion Creek would only be feasible in conjunction with the construction of DS10, therefore the elevations provided for optimization were based on unsteady flow hydraulic modeling results with DS10 constructed.

The Hydraulics Section provided three levee elevations to be evaluated in the optimization process. These elevations corresponded to the energy grade line (EGL) of the 1% annual exceedance probability (AEP) event plus zero feet, the 1% AEP plus three feet, and the 1% AEP plus five feet. The EGL elevation was determined from 1D/2D unsteady flow hydraulic modeling. The target elevation decreases in elevation along the length of each reach therefore a specific value is not listed in this appendix.

Hydraulics also provided a shapefile of the existing floodway boundary for the Little Papillion Creek area of interest. More detailed information on the determination of the elevations to be evaluated can be found in Appendix B: Hydraulics.

The purpose of optimization was to determine material quantities for construction costs and to establish project footprints to determine required real estate boundaries and/or easements. Achieving the required height raise with a floodwall would produce the minimum real estate requirement whereas using a levee fill prism would produce the maximum real estate requirement. Achieving the height requirement using a combination of levee fill and floodwall height was not evaluated at this time, however this may be considered during the design phase after real estate has been acquired.

Since there are no existing flood reduction structures along this section of the Little Papillion Creek, a baseline was established to construct a civil model. The baseline was determined by looking at the existing topography and existing floodway boundary. Due to the number of existing structures along this portion of Little Papillion Creek, including many non-residential structures, the baseline was established along the existing break line of the channel bank while remaining outside of the floodway boundary.

Within the civil model, templates for a full height levee prism and a full height floodwall wall were created, based upon the typical sections shown in Figures 26 and 27 above. The templates were created with the riverside toe of the structure as the anchor point of the template, with the remainder of the template building to the landside of this anchor point. The template anchor point followed the path of the baseline and the existing ground elevation.

The templates were set to hit the target elevation provided by hydraulics. At each location where the hydraulic model cross-section intersected the geometric baseline, the elevation for the vertical alignment was set to the target elevation. Between each hydraulic cross-section (vertical anchor points), the target elevation is a consistent slope between the two.

In areas where the existing topography was above the target elevation, no template was applied. If the required elevation raise was approximately one foot or less, a levee prism was used rather than a floodwall, as the real estate boundary at that elevation is essentially the same. For larger elevation changes, aerial imagery assisted in determining whether to use a floodwall or levee to achieve the desired height. In locations with existing structures, typically the floodwall template was applied, to minimize impacts to those structures and real estate costs. In those areas where there appeared to be available space, the levee template was applied.

For all three elevation scenarios the templates were used to determine real estate boundaries and material quantities for each reach. Shapefiles of the project footprints and required real estate boundaries were provided to the real estate section to assist in developing costs, as well as to the environmental specialists to show the areas effected by construction. Material quantities for all three scenarios were provided to the Cost engineer to assist in determining the project construction costs.

The optimization process resulted in the selection of the new levee and floodwall plan that corresponded to the 1% AEP + 3' event as the feasible scenario moving forward based upon the benefit to cost ratios of all three scenarios. More detailed information for each reach, from upstream to downstream, is included below.

4.3.3. Utilities

Shapefiles of existing utilities were provided by Metropolitan Utilities District (MUD) and Omaha Public Power District (OPPD). MUD handles water and natural gas services in the Omaha area and OPPD provides electrical services. Information on sanitary sewer and stormwater sewer systems was not provided. While the available data may not be all inclusive of the existing utilities in and near the project footprint, they were used to determine a best estimate of the utilities that will be encountered during construction. However, due to the uncertainty in the presence of existing utilities, this concern was added as a risk in the CSRA.

Some of the existing utilities may be able to be relocated outside the project footprint, while others may need to be addressed and incorporated during the design phase of the project. Modifications to existing interior drainage culverts was not considered at this stage of the process. During the design phase, the surface runoff and interior drainage will need to be evaluated which may result in the modification of existing culverts or additional of new drainage culverts. Closure structures are planned to provide continuous risk reduction across the roadways. See Appendix D – Structural for more information on closure structures.

In the aerial images below the known existing utilities are symbolized as follow:
- Gas lines are shown in orange
- Water lines are shown in blue
- Overhead electrical lines and transformers are shown in yellow
- Underground electrical lines and transformers are shown in green
- Poles are shown in black as a standard pole symbol of a circle with a line extending on two sides

4.3.4. Reach LP5-Up

This reach spans from Cass Street north to the upstream tie-off locations on both banks. Figure 28 below shows the baseline for the right bank in red, the baseline for the left bank in pink, and the floodway boundary in cyan.

Figure 28: Aerial photo of LP5-Up – Upstream tie-off locations north of Cass Street

4.3.4.1. Utilities

Table 8: Known utilities within the reach LP5-Up footprint

Bank	Approximate Stations	Type of Utility	Description
Left Bank	6+63	Water	Center of Cass Street
	6+77	Gas	Center of Cass Street
Right Bank	48+67 to 52+92	Underground Electrical	North to South within footprint
	49+03	Underground Transformer	Within footprint
	52+77	Underground Transformer	Within footprint
	52+99	Water	Crossing channel North of Cass Street
	53+05	Gas	Crossing channel North of Cass Street
	53+29	Gas	Crossing channel North of Cass Street
	53+40 to 53+65	Underground Electrical	North half of Cass Street

4.3.4.2. Quantities

At the stage of selecting the preferred alternatives for the TSP the upstream tie-off locations had not yet been determined and therefore no initial quantities exist for this reach. Quantities for the optimizations were determined from the civil model and the templates used for both the levee and the floodwall options. Areas where the existing topography showed at a higher elevation than the target elevation did not have a template applied and show lengths of 0 feet in the table below.

The baseline alignment for the left bank is 677.23 feet long and the average height raise is 4.6 feet.
The baseline alignment for the right bank is 5,393.20 feet long and the average height raise is 2.6 feet.
Quantities are shown only for the optimized recommended plan that was selected to move forward with, the 1% AEP plus 3 feet with DS10 construction and was modeled using unsteady flow hydraulic modeling.

Table 9: Optimized quantities for reach LP5-Up for 1% AEP + 3' event

Bank	Stations	Length of Levee (ft)	Length of Floodwall (ft)	Volume of Earthwork Fill (yd^3)	Volume of Floodwall Concrete (yd^3)	Length of Concrete Forms (ft)
Left Bank	0+00.00 – 5+68.00	568.00	0.0	1015.0	83.5	4728.5
	5+68.00 – 6+77.23	0.0	109.23			
Right Bank	0+00.00 – 4+79.65	0.0	0.0	4690.0	1541.5	87706.0
	4+79.65 – 24+36.90	1957.25	0.0			
	24+36.90 – 26+33.60	0.0	0.0			
	26+33.60 – 27+62.90	129.30	0.0			
	27+62.90 – 29+12.35	0.0	0.0			
	29+12.35 – 31+36.00	223.65	0.0			
	31+36.00 – 43+39.25	0.0	1203.25			
	43+39.25 – 43+63.45	0.0	0.0			
	43+63.45 – 53+93.20	0.0	1029.75			

4.3.5. Reach LP5

This reach spans from Cass Street to Dodge Street on both banks. Figure 29 below shows the baseline for the right bank in pink, the baseline for the left bank in red, and the floodway boundary in cyan.

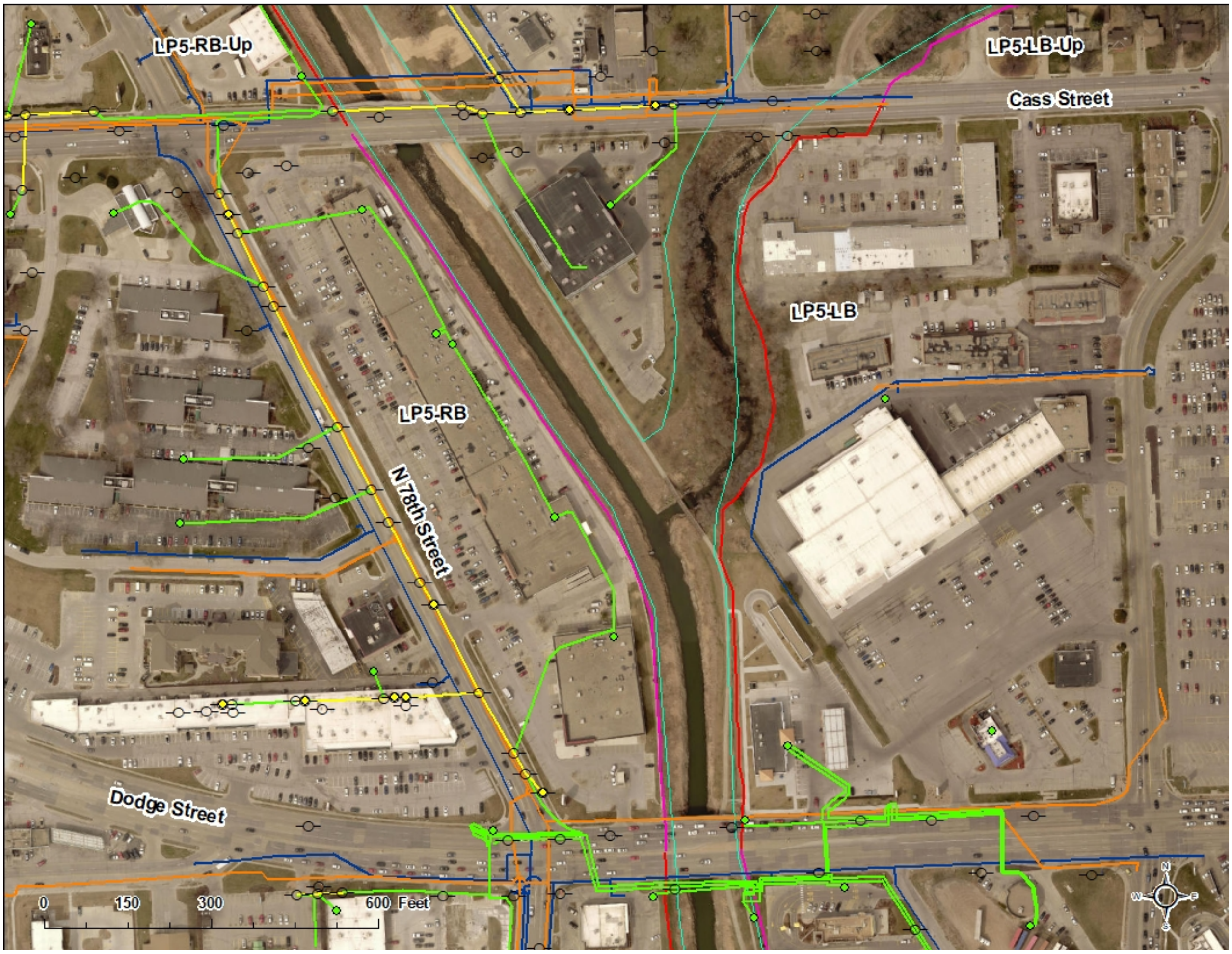

Figure 29: Aerial photo of LP5 – Cass Street to Dodge Street

4.3.5.1. Utilities

Table 10: Known utilities within the reach LP5 footprint

Bank	Approximate Stations	Type of Utility	Description
Left Bank	1+16	Pole	South edge of Cass Street
	1+89	Pole	South edge of Cass Street
	7+70 to 9+70	Water	Within footprint
	14+79	Gas	Crossing channel at North edge of Dodge St
	14+88	Underground Transformer	North edge of Dodge Street
	14+88 to 15+00	Underground Electrical	North half of Dodge Street
Right Bank	13+87	Gas	Crossing channel at North edge of Dodge St

4.3.5.2. Quantities

Quantities for the optimizations were determined from the civil model and the templates used for both the levee and the floodwall options. Areas where the existing topography showed at a higher elevation than the target elevation did not have a template applied and show lengths of 0 feet in the table below. A negative value on the quantity indicates more material is to be excavated than is required for fill.

At the stage of selecting the preferred alternatives, initial analysis using only steady flows, the project on the Little Papillion Creek was still being considered with both DS10 constructed and without DS10 constructed. The without DS10 average height raise was based upon a steady flow hydraulic model of the Little Papillion Creek levee alternative, and the with DS10 average height raise was an estimated height based upon the output water surface from the DS10 constructed steady flow hydraulic model. The alignment was estimated at 1,400 feet long on both banks for both options.

After the preferred alternative was selected, with DS10 constructed, the average height raise was modeled using unsteady flow hydraulic modeling. Quantities are shown only for the optimized recommended plan that was selected to move forward with, the 1% AEP plus 3 feet with DS10 constructed.

Table 11: Analysis results comparison for reach LP5

Scenario	Riverbank	Length (ft)	Average Height Raise (ft)
Initial 1% AEP +3' without DS10	Left	1,400	3.6
Initial 1% AEP +3' with DS10	Left	1,400	0.7
Optimized 1% AEP + 3' with DS10	Left	1,534.87	4.9
Initial 1% AEP +3' without DS10	Right	1,400	5.0
Initial 1% AEP +3' with DS10	Right	1,400	1.8
Optimized 1% AEP + 3' with DS10	Right	1,442.07	3.5

Table 12: Optimized quantities for reach LP5 for 1% AEP + 3' event

<table>
<tr><th>Bank</th><th>Stations</th><th>Length of Levee (ft)</th><th>Length of Floodwall (ft)</th><th>Volume of Earthwork Fill (yd^3)</th><th>Volume of Floodwall Concrete (yd^3)</th><th>Length of Concrete Forms (ft)</th></tr>
<tr><td rowspan="4">Left Bank</td><td>0+00.00 – 3+08.90</td><td>0.0</td><td>308.90</td><td rowspan="4">-1705.0</td><td rowspan="4">1012.2</td><td rowspan="4">57277.5</td></tr>
<tr><td>3+08.90 – 3+89.30</td><td>0.0</td><td>0.0</td></tr>
<tr><td>3+89.30 – 5+35.00</td><td>145.70</td><td>0.0</td></tr>
<tr><td>5+35.00 – 15+34.87</td><td>0.0</td><td>999.87</td></tr>
<tr><td>Right Bank</td><td>0+00.00 – 14+42.07</td><td>0.0</td><td>1442.07</td><td>-825.0</td><td>1041.1</td><td>59104.1</td></tr>
</table>

4.3.6. Reach LP6

This reach spans from Dodge Street to Pacific Street on both banks. This reach has been divided into two sub-reaches, LP6-1 from Dodge Street to 72nd Street and LP6-2 from 72nd Street to Pacific Street. Figure 30 and 31 below the floodway boundary in cyan. Figure 30 shows the LP6-1 right bank baseline in red and the left bank in pink. Figure 31 shows the LP6-2 right bank baseline in pink and the left bank in red.

On the right bank of LP6-1, near the intersection of Harney Street and South 78th Street, there is an area approximately 450 feet long where considerable work has previously been done by USACE to stabilize the creek bottom and banks. During the design phase, this area will require additional consideration and possibly special construction techniques to ensure the existing work is not compromised. As a part of this feasibility level design, no special considerations were given to this area.

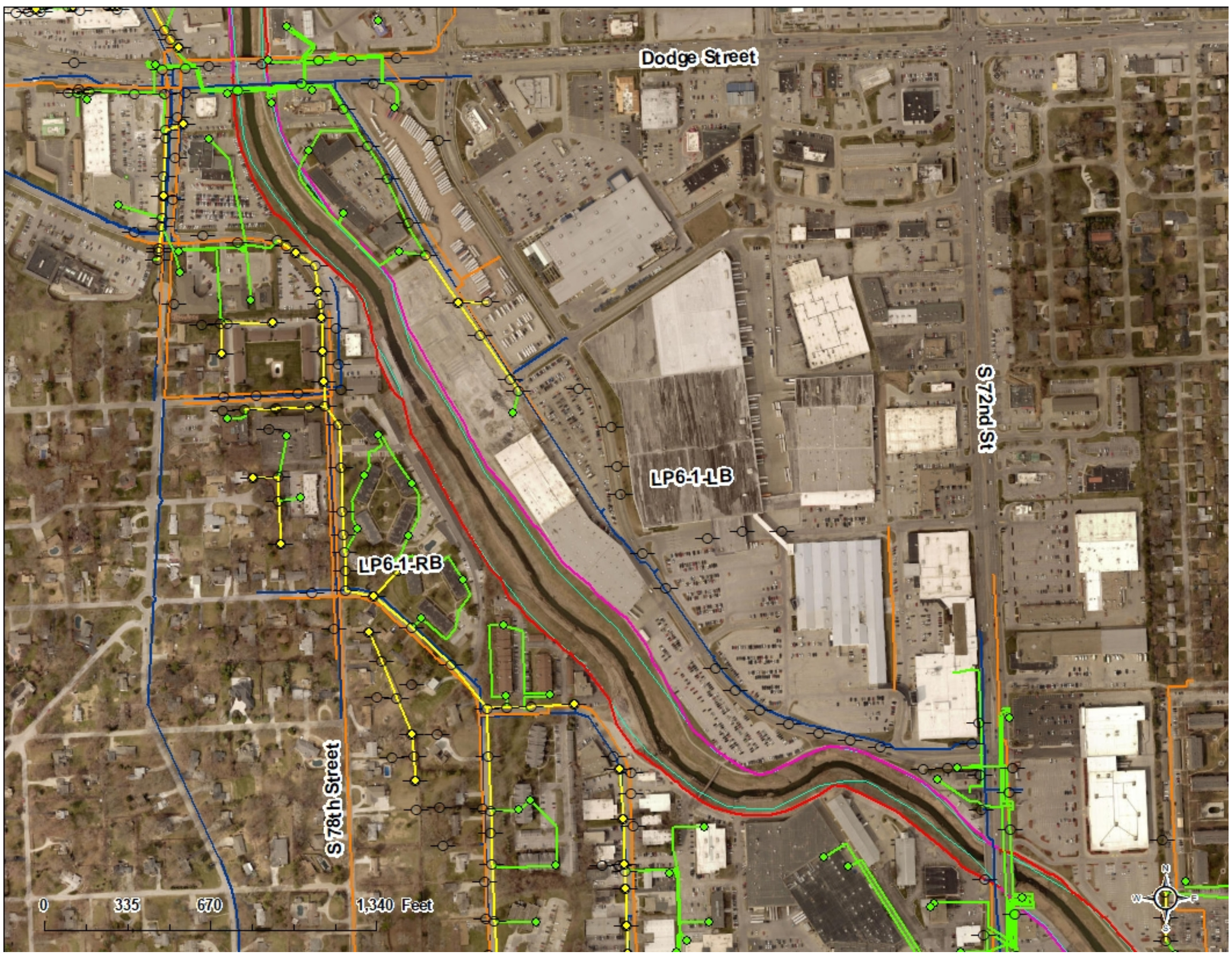

Figure 30: Aerial photo of LP6-1 – Dodge Street to 72nd Street

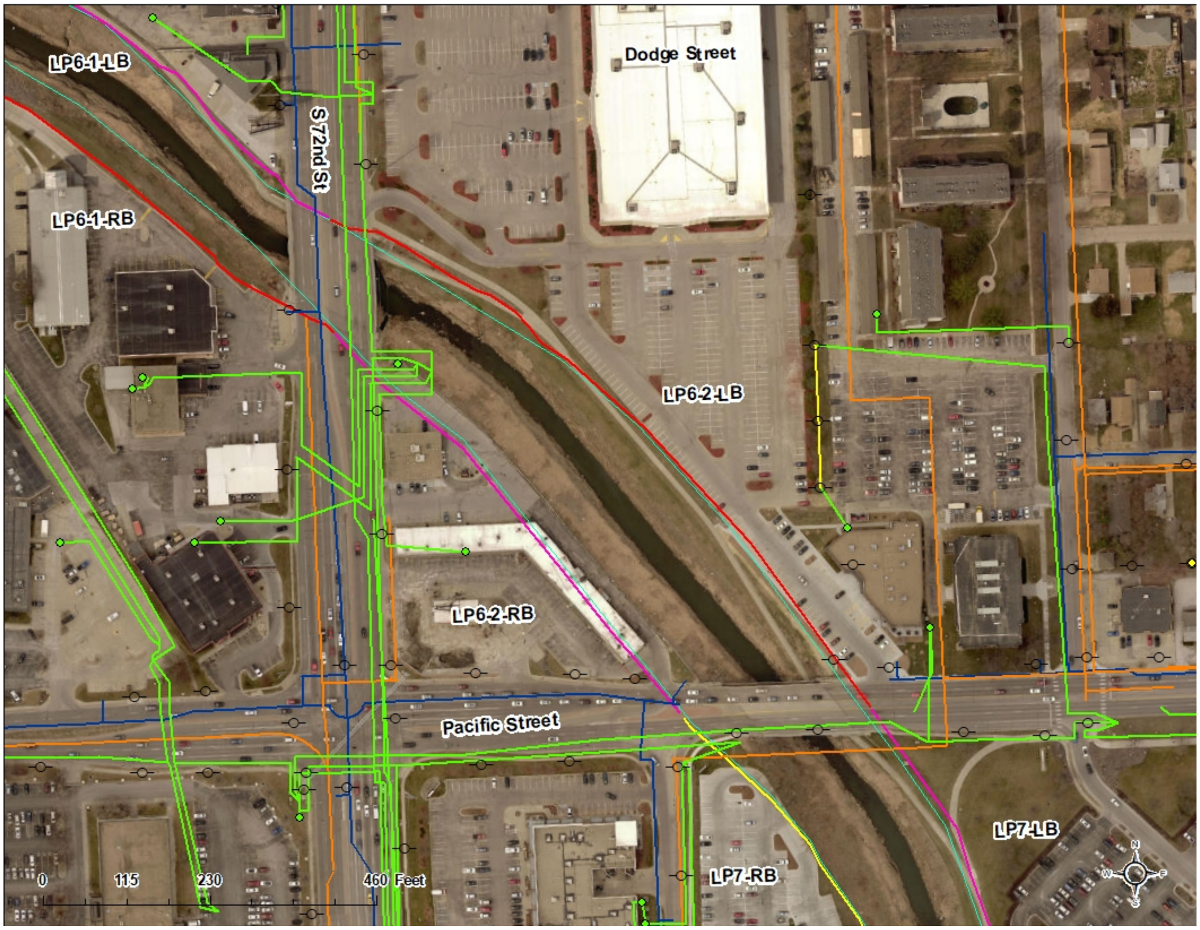

Figure 31: Aerial photo of LP6-2 – 72nd Street to Pacific Street

4.3.6.1. Utilities

Table 13: Known utilities within the reach LP6-1 footprint

Bank	Approximate Stations	Type of Utility	Description
Left Bank	0+48	Water	South edge of Dodge Street
	0+60 to 1+28	Underground Electrical	Crossing channel South of Dodge Street
	1+28	Underground Transformer	South of Dodge Street
	3+74	Underground Transformer	Within footprint
	3+74 to 9+38	Underground Electrical	Within footprint
	6+65	Underground Transformer	Within footprint
	46+51	Water	West half of 72nd Street
Right Bank	0+47	Water	South edge of Dodge Street
	0+52 to 0+82	Underground Electrical	Crossing channel South edge of Dodge St
	0+82	Underground Transformer	South side of Dodge Street
	9+42	Water	Edge of footprint S 78th St & Farnam Dr
	49+34	Water	West half of 72nd Street
	49+43	Gas	West half of 72nd Street
	49+63	Water	Center of 72nd Street

Table 14: Known utilities within the reach LP6-2 footprint

Bank	Approximate Stations	Type of Utility	Description
Left Bank	0+12 to 0+60	Underground Electrical	Crossing Channel East side of 72nd Street
	9+48	Pole	North edge of Pacific Street
Right Bank	0+22 to 1+62	Underground Electrical	East half of 72nd Street
	6+37	Pole	Within footprint
	6+89 to 7+11	Water	North half of Pacific Street

4.3.6.2. Quantities

Quantities for the optimizations were determined from the civil model and the templates used for both the levee and the floodwall options. Areas where the existing topography showed at a higher elevation than the target elevation did not have a template applied and show lengths of 0 feet in the table below. A negative value on the quantity indicates more material is to be excavated than is required for fill.

At the stage of selecting the preferred alternatives, initial analysis using only steady flows, the project on the Little Papillion Creek was still being considered with both DS10 constructed and without DS10 constructed. The without DS10 average height raise was based upon a steady flow hydraulic model of the Little Papillion Creek levee alternative, and the with DS10 average height raise was an estimated height based upon the output water surface from the DS10 constructed steady flow hydraulic model.

After the preferred alternative was selected, with DS10 constructed, the average height raise was modeled using unsteady flow hydraulic modeling. Quantities are shown only for the optimized recommended plan that was selected to move forward with, the 1% AEP plus 3 feet with DS10 constructed.

Table 15: Analysis results comparison for reach LP6-1

Scenario	Riverbank	Length (ft)	Average Height Raise (ft)
Initial 1% AEP +3’ without DS10	Left	4,730	4.5
Initial 1% AEP +3’ with DS10	Left	4,800	1.5
Optimized 1% AEP + 3’ with DS10	Left	4,668.71	3.7
Initial 1% AEP +3’ without DS10	Right	4,966	5.5
Initial 1% AEP +3’ with DS10	Right	4,800	2.6
Optimized 1% AEP + 3’ with DS10	Right	4,984.22	5.1

Table 16: Analysis results comparison for reach LP6-2

Scenario	Riverbank	Length (ft)	Average Height Raise (ft)
Initial 1% AEP +3’ without DS10	Left	1,016	6.3
Initial 1% AEP +3’ with DS10	Left	900	3.0
Optimized 1% AEP + 3’ with DS10	Left	1,026.93	6.1
Initial 1% AEP +3’ without DS10	Right	767	5.4
Initial 1% AEP +3’ with DS10	Right	900	2.1
Optimized 1% AEP + 3’ with DS10	Right	710.94	4.5

Table 17: Optimized quantities for reach LP6 Left Bank for 1% AEP + 3' event

Bank	Stations	Length of Levee (ft)	Length of Floodwall (ft)	Volume of Earthwork Fill (yd^3)	Volume of Floodwall Concrete (yd^3)	Length of Concrete Forms (ft)
LP6-1	0+00.00 – 0+52.00	0.0	52.0	-1170.0	2432.2	138019.4
	0+52.00 – 4+36.00	384.0	0.0			
	4+36.00 – 28+44.00	0.0	2408.0			
	28+44.00 – 30+52.45	208.45	0.0			
	30+52.45 – 33+73.85	0.0	0.0			
	33+73.85 – 37+89.00	415.15	0.0			
	37+89.00 – 46+68.71	0.0	879.71			
LP6-2	0+00.00 – 10+26.93	0.0	1026.93	-630.0	840.6	47443.9

Table 18: Optimized quantities for reach LP6 Right Bank for 1% AEP + 3' event

Bank	Stations	Length of Levee (ft)	Length of Floodwall (ft)	Volume of Earthwork Fill (yd^3)	Volume of Floodwall Concrete (yd^3)	Length of Concrete Forms (ft)
LP6-1	0+00.00 – 25+37.80	0.0	2537.80	-5135.0	3767.8	213121.9
	25+37.80 – 26+91.05	0.0	0.0			
	26+91.05 – 49+84.22	0.0	2293.17			
LP6-2	0+00.00 – 7+10.94	0.0	710.94	-660.0	539.3	30541.9

4.3.7. Reach LP7

This reach spans from Pacific Street to Mercy Road on both banks. Figure 32 below shows the baseline for the right bank in yellow, the baseline for the left bank in pink, and the floodway boundary in cyan.

Figure 32: Aerial photo of LP7 – Pacific Street to Mercy Road

4.3.7.1. Utilities

Table 19: Known utilities within the reach LP7 footprint

Bank	Approximate Stations	Type of Utility	Description
Left Bank	0+21 to 0+60	Underground Electrical	Crosses channel on South side of Pacific St
	0+70	Gas	South side of Pacific Street
	13+55 to 18+60	Underground Electrical	North to South within footprint
	18+34	Water	North of Pine Street
	19+10 to 19+92	Underground Electrical	South side of Pine Street
	19+51	Gas	South edge of Pine Street
	22+45 to 26+20	Water	West side of Ak-Sar-Ben Drive
	21+95 to 41+03	Gas	West side of Ak-Sar-Ben Drive
	33+50 to 41+03	Underground Electrical	West side of Ak-Sar-Ben Drive
	34+05	Gas	South of Frances Street
	38+60	Gas	North of Mercy Road
	39+00	Gas	North of Mercy Road
	39+04	Water	North of Mercy Road
	40+42	Pole	Within footprint
Right Bank	0+00 to 0+80	Underground Electrical	South half of Pacific Street
	0+80	Gas	South side of Pacific Street
	9+00 to 14+32	Gas	East side of South 70th Street
	18+07	Water	North of Pine Street
	19+55	Gas	South edge of Pine Street
	40+20 to 42+09	Gas	North of Mercy Road
	40+20 to 42+09	Water	North of Mercy Road
	41+62	Underground Electrical	Crossing channel North of Mercy Road
	41+95	Pole	Within footprint

4.3.7.2. Quantities

Quantities for the optimizations were determined from the civil model and the templates used for both the levee and the floodwall options. Areas where the existing topography showed at a higher elevation than the target elevation did not have a template applied and show lengths of 0 feet in the table below. A negative value on the quantity indicates more material is to be excavated than is required for fill.

At the stage of selecting the preferred alternatives, initial analysis using only steady flows, the project on the Little Papillion Creek was still being considered with both DS10 constructed and without DS10 constructed. The without DS10 average height raise was based upon a steady flow hydraulic model of the Little Papillion Creek levee alternative, and the with DS10 average height raise was an estimated height based upon the output water surface from the DS10 constructed steady flow hydraulic model.

After the preferred alternative was selected, with DS10 constructed, the average height raise was modeled using unsteady flow hydraulic modeling. Quantities are shown only for the optimized recommended plan that was selected to move forward with, the 1% AEP plus 3 feet with DS10 constructed.

Table 20: Analysis results comparison for reach LP7

Scenario	Riverbank	Length (ft)	Average Height Raise (ft)
Initial 1% AEP +3' without DS10	Left	4,166	4.0
Initial 1% AEP +3' with DS10	Left	4,600	1.9
Optimized 1% AEP + 3' with DS10	Left	4,1033.55	5.7
Initial 1% AEP +3' without DS10	Right	3,181	1.5
Initial 1% AEP +3' with DS10	Right	4,600	0.3
Optimized 1% AEP + 3' with DS10	Right	4,208.38	3.5

Table 21: Optimized quantities for reach LP7 for 1% AEP + 3' event

Bank	Stations	Length of Levee (ft)	Length of Floodwall (ft)	Volume of Earthwork Fill (yd^3)	Volume of Floodwall Concrete (yd^3)	Length of Concrete Forms (ft)
Left Bank	0+00.00 – 0+73.00	0.0	73.0	4415.0	1710.7	96639.1
	0+73.00 – 6+79.00	606.0	0.0			
	6+79.00 – 8+32.00	0.0	153.0			
	8+32.00 – 10+30.55	198.55	0.0			
	10+30.55 – 10+46.25	0.0	0.0			
	10+46.25 – 14+33.25	387.0	0.0			
	14+33.25 – 14+51.35	0.0	0.0			
	14+51.35 – 18+54.50	403.15	0.0			
	18+54.50 – 19+45.25	0.0	0.0			
	19+45.25 – 22+00.00	254.75	0.0			
	22+00.00 – 41+03.55	0.0	1903.55			
Right Bank	0+00.00 – 10+88.90	1088.90	0.0	7810.0	0.0	0.0
	10+88.90 – 11+22.45	0.0	0.0			
	11+22.45 – 14+50.90	328.45	0.0			
	14+50.90 – 15+56.20	0.0	0.0			
	15+56.20 – 18+34.10	277.90	0.0			
	18+34.10 – 19+70.30	0.0	0.0			
	19+70.30 – 37+27.65	1757.35	0.0			
	37+27.65 – 40+60.40	0.0	0.0			
	40+60.40 – 42+08.38	147.98	0.0			

4.3.8. Reach LP8

This reach spans from Mercy Road to Saddle Creek on both banks. Figure 33 below shows the baseline for the right bank in pink, the baseline for the left bank in red, and the floodway boundary in cyan.

Figure 33: Aerial photo of LP8 – Mercy Road to Saddle Creek

4.3.8.1. Utilities

Table 22: Known utilities within the reach LP7 footprint

Bank	Approximate Stations	Type of Utility	Description
Left Bank	9+07	Pole	North side of South 65th Avenue
	22+20	Gas	West side of South 64th Avenue
	22+32	Water	Center of South 64th Avenue
	22+57	Gas	East side of South 64th Avenue
Right Bank	0+00 to 0+28	Gas	Within Mercy Road
	1+32 to 6+44	Water	East side of South 68th Street

4.3.8.2. Quantities

Quantities for the optimizations were determined from the civil model and the templates used for both the levee and the floodwall options. Areas where the existing topography showed at a higher elevation than the target elevation did not have a template applied and show lengths of 0 feet in the table below. A negative value on the quantity indicates more material is to be excavated than is required for fill.

At the stage of selecting the preferred alternatives, initial analysis using only steady flows, the project on the Little Papillion Creek was still being considered with both DS10 constructed and without DS10 constructed. The without DS10 average height raise was based upon a steady flow hydraulic model of the Little Papillion Creek levee alternative, and the with DS10 average height raise was an estimated height based upon the output water surface from the DS10 constructed steady flow hydraulic model.

After the preferred alternative was selected, with DS10 constructed, the average height raise was modeled using unsteady flow hydraulic modeling. Quantities are shown only for the optimized recommended plan that was selected to move forward with, the 1% AEP plus 3 feet with DS10 constructed.

Table 23: Analysis results comparison for reach LP8

Scenario	Riverbank	Length (ft)	Average Height Raise (ft)
Initial 1% AEP +3’ without DS10	Left	1,554	4.2
Initial 1% AEP +3’ with DS10	Left	2,300	4.0
Optimized 1% AEP + 3’ with DS10	Left	2,672.60	9.8
Initial 1% AEP +3’ without DS10	Right	2,376	4.9
Initial 1% AEP +3’ with DS10	Right	2,300	4.5
Optimized 1% AEP + 3’ with DS10	Right	2,701.10	2.8

Table 24: Optimized quantities for reach LP8 for 1% AEP + 3’ event

<table>
<tr><th>Bank</th><th>Stations</th><th>Length of Levee (ft)</th><th>Length of Floodwall (ft)</th><th>Volume of Earthwork Fill (yd³)</th><th>Volume of Floodwall Concrete (yd³)</th><th>Length of Concrete Forms (ft)</th></tr>
<tr><td rowspan="7">Left Bank</td><td>0+00.00 – 0+30.55</td><td>30.55</td><td>0.0</td><td rowspan="7">5250.0</td><td rowspan="7">820.0</td><td rowspan="7">45999.6</td></tr>
<tr><td>0+30.55 – 5+57.55</td><td>0.0</td><td>0.0</td></tr>
<tr><td>5+57.55 – 14+88.00</td><td>930.45</td><td>0.0</td></tr>
<tr><td>14+88.00 – 21+22.85</td><td>0.0</td><td>634.85</td></tr>
<tr><td>21+22.85 – 22+94.15</td><td>0.0</td><td>0.0</td></tr>
<tr><td>22+94.15 – 25+19.10</td><td>0.0</td><td>224.95</td></tr>
<tr><td>25+19.10 – 26+72.60</td><td>0.0</td><td>0.0</td></tr>
<tr><td rowspan="2">Right Bank</td><td>0+00.00 – 0+55.50</td><td>0.0</td><td>0.0</td><td rowspan="2">8765.0</td><td rowspan="2">0.0</td><td rowspan="2">0.0</td></tr>
<tr><td>0+55.50 – 27+01.10</td><td>2645.60</td><td>0.0</td></tr>
</table>

4.4. BIG PAPILLION CREEK

The TSP alternatives along Big Papillion Creek include:

- Channel widening in reaches identified as BP4 and BP5.
- Levee raises and levee construction in reaches identified as BP7 and BP8.
- Floodwall construction, levee construction and levee raises in reach LP8 of the Little Papillion Creek, upstream of the confluence with the Big Papillion Creek.

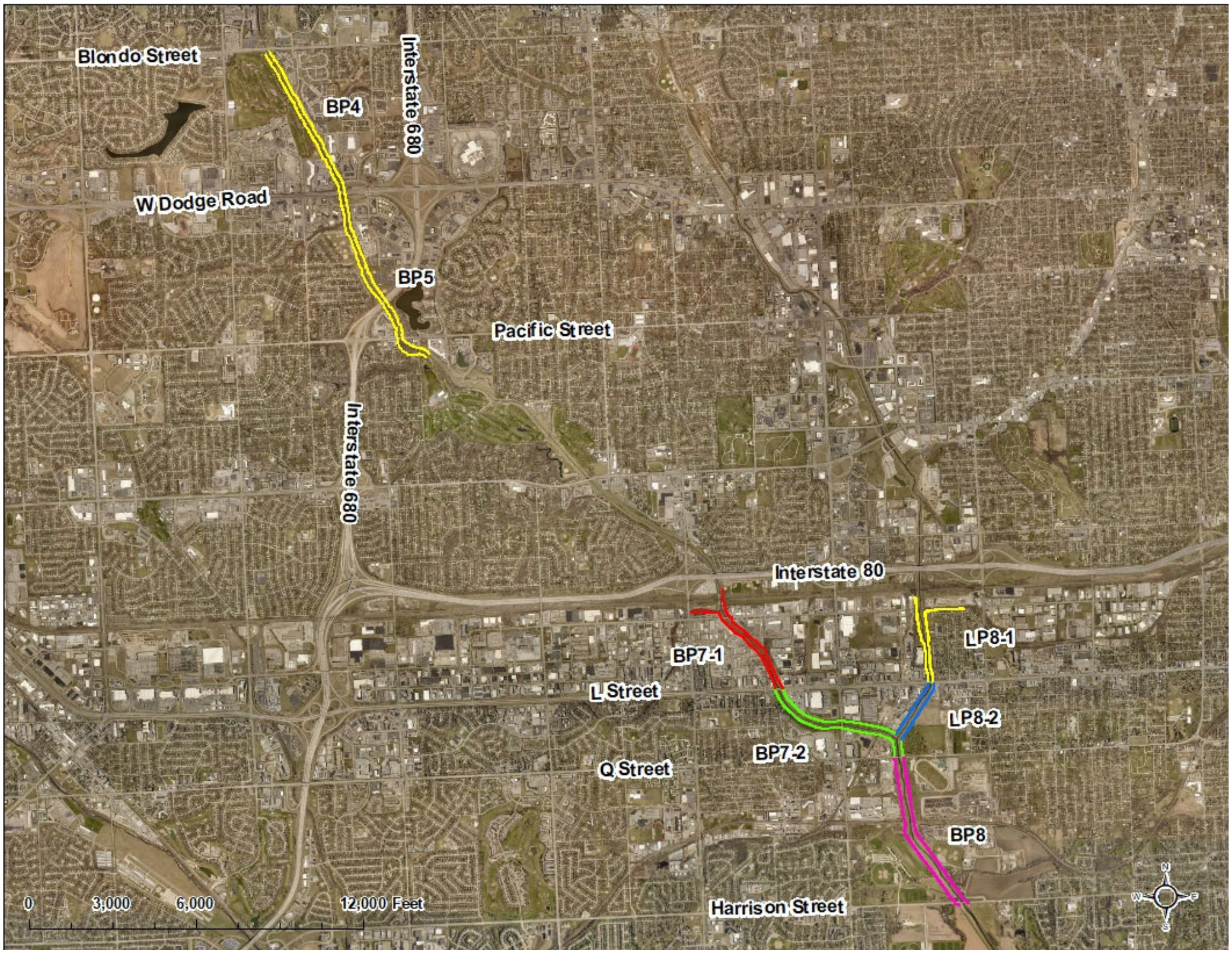

Figure 34: Aerial Photo of Big Papillion TSP Alternatives

4.4.1. Basis of Preliminary Design

The NLD shows several existing levees within the lengths of the Big Papillion Creek where alternatives are being considered. These projects are a mix of federally constructed and non-federally constructed levees. The local sponsor for all these projects is the Papio-Missouri River Natural Resources District, and all the projects are currently active in the USACE PL 84-99 Rehabilitation Program.

Table 25: Existing levee projects within the Big Papillion Creek alternatives area

Bank	Type	Segment Name	Reach
Left	Federal	Big Papio LB – West Center to L St	BP7-1
Right	Federal	Big Papio RB – West Center to L St	BP7-1
Left	Non-Federal	NEDOUG16 – Big Papio LB – L St to Little Papio Segment	BP7-2
Right	Federal	Little Papio Creek RB – Spaulding St to Big Papio Confluence Segment	LP8-2
Left	Federal	Little Papio Creek LB – Spaulding St to Big Papio Confluence Segment	LP8-2
Left	Non-Federal	NEDOUG16 – Big Papio LB – Little Papio to Copper Creek Segment	BP7-2 & BP8
Right	Non-Federal	NEDOUG16 – Big Papio RB – L St to Thompson Cr	BP7-2 & BP8

The existing levees in reach BP7-1 are non-continuous segments. Existing levee alignment centerline data was obtained from the NLD in the form of shapefiles. Existing elevations were derived from the LiDAR topography. The existing topography shows landside slopes are approximately 5H:1V in the areas with existing levees.

New and existing levees will be raised through either the construction of a floodwall or the addition of a landside levee fill prism. Where space is available a landside slope of 5H:1V will be used to match the existing conditions. In areas with limited space, a 3H:1V landside slope will be used as it is the current minimum acceptable slope per USACE design criteria.

Typical sections for each are shown in Figures 26 and 27 above. In both cases, a 15-foot-wide vegetation free zone (VFZ) would be added on the landside, measured from the landside toe, to establish real estate and/or easement boundaries.

There are no existing levees in the reaches identified as BP4 and BP5. In these areas, it was previously determined that an effective flood risk reduction measure would be to widen the channel rather than constructing new structures.

4.4.2. Channel Widening Optimization

Hydraulics provided shapefiles and a bench target elevation for the channel widening areas. A typical channel widening cross-section is shown in Figure 35 below. The purpose of optimization was to determine material quantities for construction costs and to establish project footprints to determine required real estate boundaries and/or easements.

In order to construct a civil model, a baseline along the approximate center of the low flow portion of the channel was established based upon the existing topography. A profile was created at the bench target elevations along this centerline. A template following the typical section was created, with the bench height set to the target profile. Side slopes of 3H:1V were added and set to project to the existing ground surface to create the bank catch points and establish the project footprint. Bench widths of 150 feet, 170 feet and 200 feet were provided by Hydraulics for comparison. In addition, based on aerial imagery, a varied width bench option was created. For the varied width option, the bank catch points were defined, and the side slopes were project down to the target bench elevation resulting in a bench width that varied along the alignment.

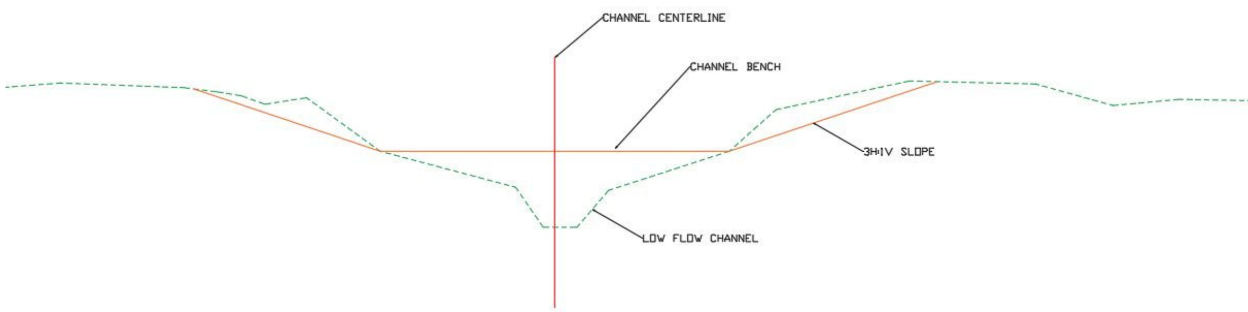

Figure 35: Typical channel widening cross-section

4.4.3. Levee Raises Optimization

The Hydraulics Section provided three levee elevations to be evaluated in the optimization process. These elevations corresponded to the energy grade line (EGL) of the 1% annual exceedance probability (AEP) event plus zero feet, the 1% AEP plus three feet, and the 1% AEP plus five feet. The EGL elevation was determined from 1D/2D unsteady flow hydraulic modeling. The target elevation decreases in elevation along the length of each reach therefore a specific value is not listed in this appendix. More detailed information on the determination of the elevations to be evaluated can be found in Appendix B: Hydraulics.

Achieving the required height raise with a floodwall would produce the minimum real estate requirement whereas using a levee fill prism would produce the maximum real estate requirement. Achieving the height requirement using a combination of levee fill and floodwall height was not evaluated at this time, however this may be considered during the design phase after real estate has been acquired.

Since there are existing levees along most of this section of the Big Papillion Creek, the existing levee centerline alignments were used as a baseline to construct a civil model. Within the civil model, templates for a full height levee prism and a full height floodwall wall were created, based upon the typical sections shown in Figures 26 and 27 above. The templates were created with an additional point for the existing levee centerline as the anchor point of the template, with the remainder of the template building out from this anchor point. The template anchor point followed the path of the baseline and the existing ground elevation.

The templates were set to hit the target elevation for each elevation provided by hydraulics. At each location where the hydraulic model cross-section intersected the geometric baseline, the elevation for the vertical alignment was set to the target elevation. Between each hydraulic cross-section, the target elevation is a consistent slope between the two.

In areas where the existing topography was above the target elevation, no template was applied. If the required elevation raise was approximately one foot or less, a levee prism was used rather than a floodwall, as the real estate boundary at that elevation is essentially the same. For larger elevation changes, aerial imagery assisted in determining whether to use a floodwall or levee to achieve the desired height. In locations with existing structures, typically the floodwall template was applied, to minimize impacts to those structures. In those areas where there appeared to be available space, the levee template was applied.

For all three elevation scenarios the templates were used to determine real estate boundaries and material quantities for each reach. Shapefiles of the project footprints and required real estate boundaries were provided to the real estate section to assist in developing costs, as well as to the environmental specialists to show the areas effected by construction. Material quantities for all three scenarios were provided to the Cost engineer to assist in determining the project construction costs.

The levee raise on Big Papillion Creek was modeled and evaluated independently of the projects on Little Papillion Creek and at Dam Site 10, and therefore could be considered a without DS10 scenario as no impacts from DS10 construction were included. The optimization process resulted in a determination that neither the channel widening, or the levee raises were feasible at this time based upon their benefit to cost ratios. If the project on Big Papillion Creek had been determined to be economically justifiable as a stand-alone project, further optimization would have been performed to include any impacts from the construction of DS10 on the area of study on the Big Papillion Creek. More detailed information for each reach, from upstream to downstream, is included below.

4.4.3.1. Initial analysis for TSP alternatives

The initial analysis of alternatives during the screening process used to determine the preferred alternatives in the TSP were based upon limited information and some generalized assumptions. The preferred alternatives were then optimized to determine which alternatives would be carried forward based upon the benefit to cost (B/C) ratios. The optimization process is discussed in detail in the next section.

The initial screening assumptions include:

- Generalized existing ground surfaces as LiDAR data had not yet been obtained
- One-dimensional steady flow hydraulic modeling to determine target elevations
- Average height raises determined from the hydraulic models
- Average height raises for each reach applied along the entire length of each reach
- Simplified I wall design for floodwalls

4.4.1. Utilities

Shapefiles of existing utilities were provided by Metropolitan Utilities District (MUD) and Omaha Public Power District (OPPD). MUD handles water and natural gas services in the Omaha area and OPPD provides electrical services. This data was provided after optimization was completed and the determinations were made that projects along the Big Papillion Creek were not feasible based upon the B/C ratios and were not included in the recommended plan. Therefore, utility conflicts and crossings were not evaluated and will not be discussed further regarding the Big Papillion Creek.

4.4.2. Reach BP4

This reach spans from Blondo Street to West Dodge Road. The channel widening project footprints are shown in the figures as follows: 150-foot bench in yellow; 170-foot bench in red; 200-foot bench in blue; and the varied width bench in pink. Figure 36 shows the upstream portion of BP4 from Blondo Street to just south of Lafayette Court. Figure 37 shows the downstream portion of BP4 from south of Lafayette Court to West Dodge Road.

Figure 36: Aerial photo of upstream portion of BP4 – Blondo Street to Lafayette Court

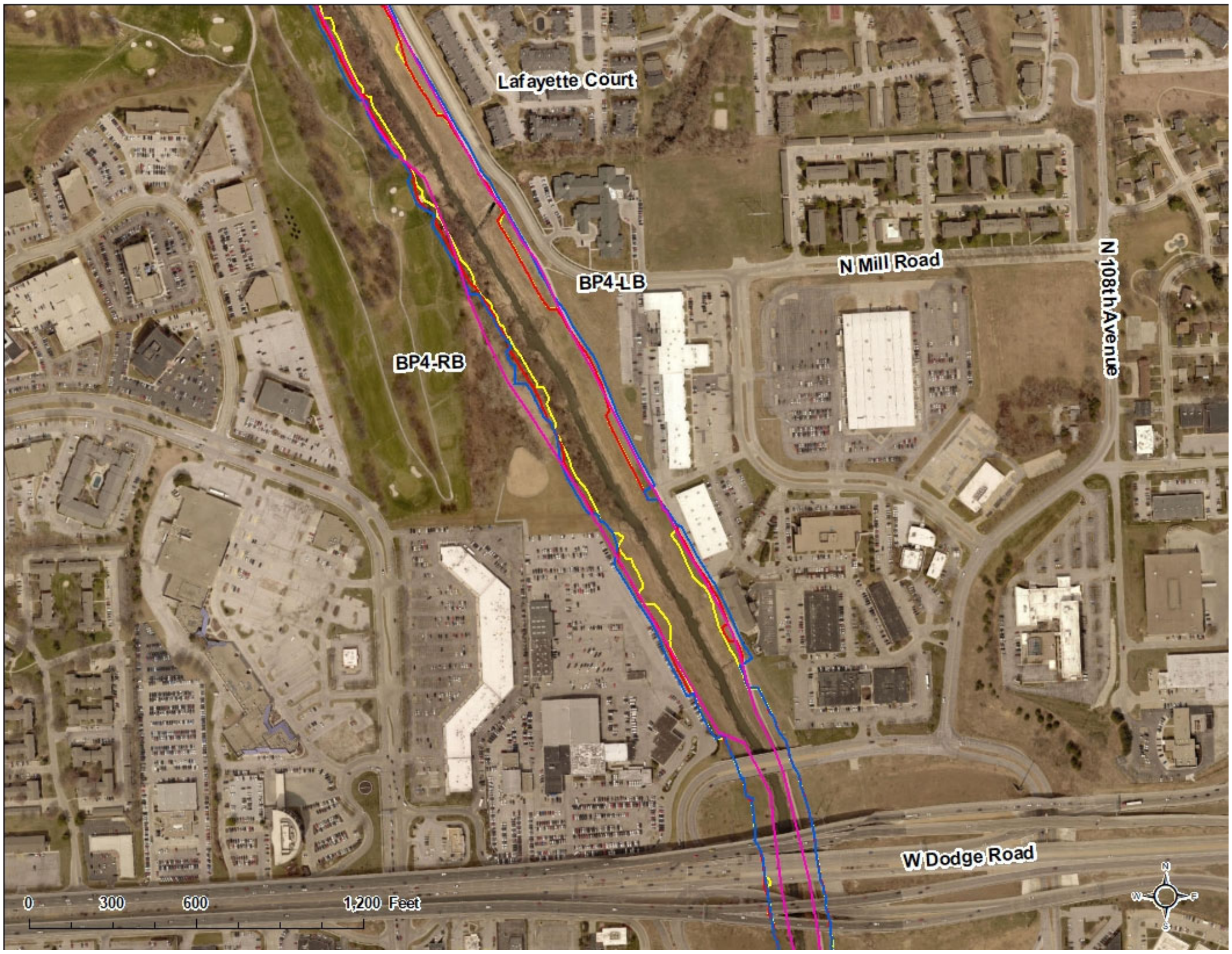

Figure 37: Aerial photo of downstream portion of BP4 – Lafayette Court to West Dodge Road

4.4.3. Reach BP5

This reach spans from West Dodge Road to 105th Street. The channel widening project footprints are shown in the figures as follows: 150-foot bench in yellow; 170-foot bench in red; 200-foot bench in blue; and the varied width bench in pink. Figure 38 shows the upstream portion of BP5 from West Dodge Road to north of Interstate 680. Figure 39 shows the downstream portion of BP5 from north of Interstate 680 to 105th Street.

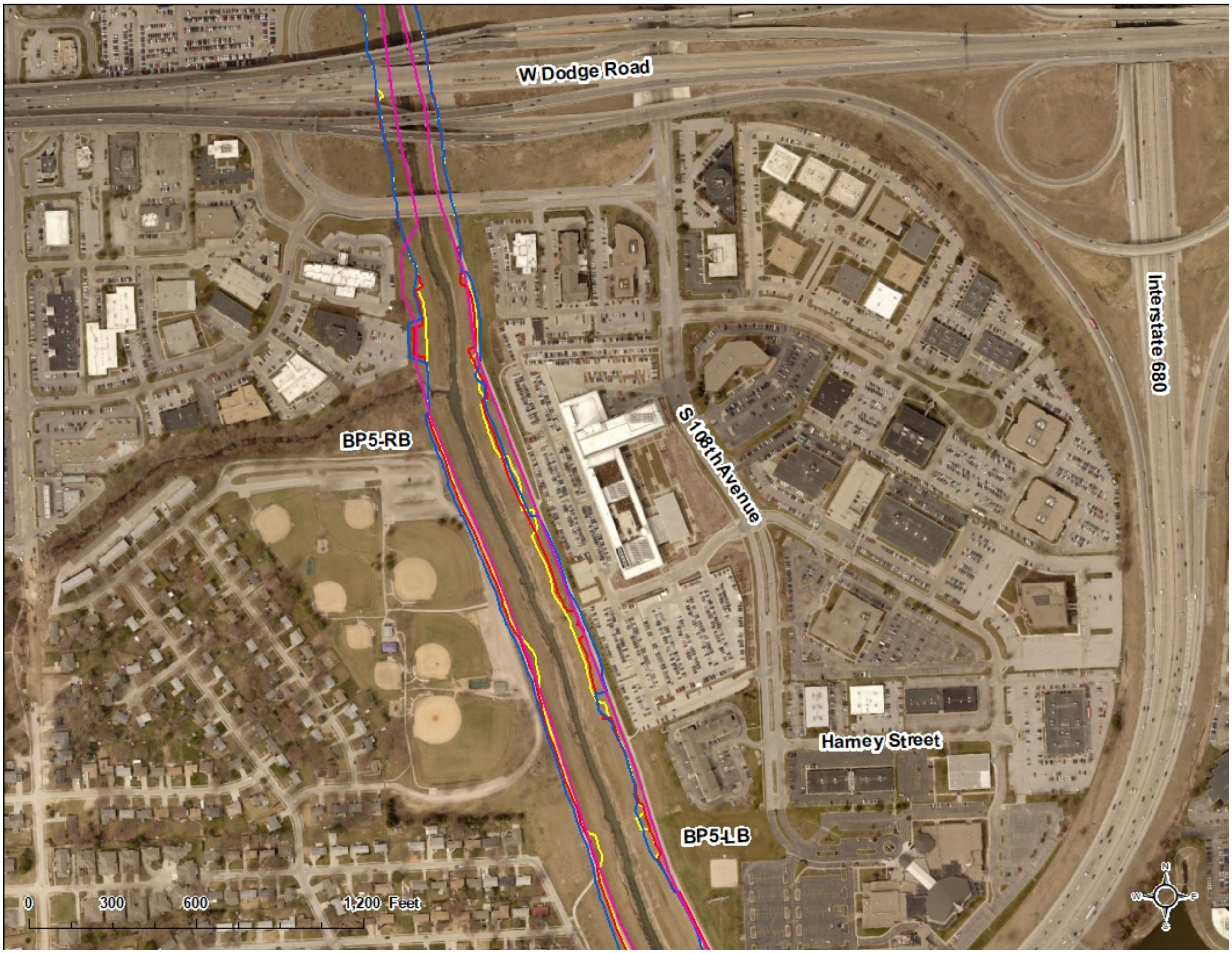

Figure 38: Aerial photo of upstream portion of BP5 – West Dodge Road to north of I-680

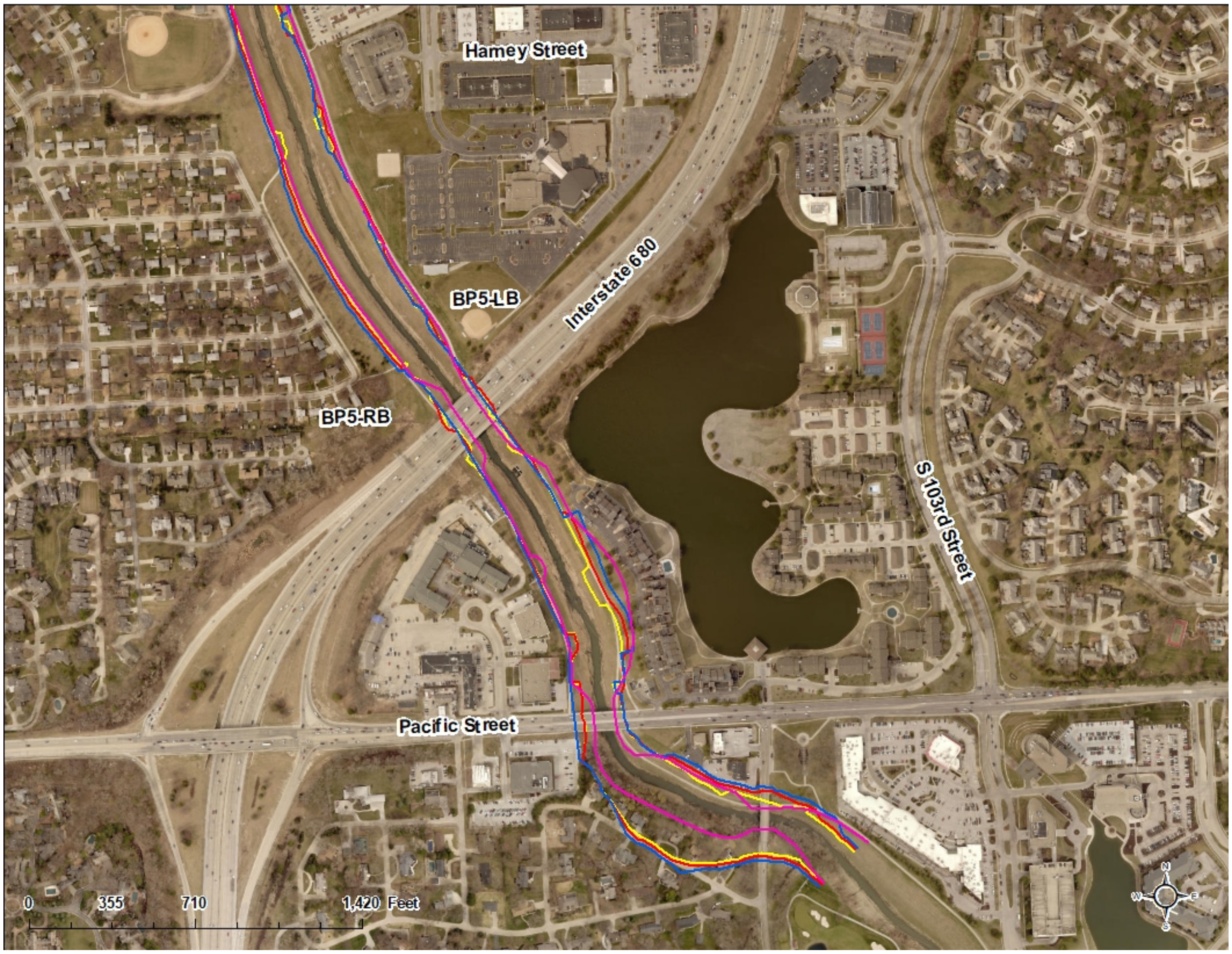

Figure 39: Aerial photo of downstream portion of BP5 – north of I-680 to 105th Street

4.4.3.1. Quantities – Channel Widening

At the stage of selecting the preferred alternatives for the TSP, quantities were calculated based upon the cross-sections from the hydraulic models. Optimized quantities were determined from the civil model and the template for each bench width. The channel alignment is 12,775 feet long and the template was run as a continuous corridor, with no separation between BP4 and BP5. Reported quantities are for the entire length of the channel widening project.

Table 26: Optimized quantities for channel widening in reaches BP4 and BP5

Bench Width	Volume of Earthwork Excavation (yd^3)
150-foot	140,000
170-foot	189,300
200-foot	285,300
Varied width	105,000

4.4.4. Reach BP7

This reach spans from the UPRR Crossing railroad bridge to Q Street on both banks. This reach has been divided into two sub-reaches, BP7-1 from the UPRR Crossing to L Street and BP7-2 from L Street to Q Street. Figure 40 shows the BP7-1 right bank baseline in pink and the left bank in yellow. Figure 41 shows the BP7-2 right bank baseline in green and the left bank in red.

Figure 40: Aerial photo of BP7-1 – UPRR Crossing Railroad Bridge to L Street

Figure 41: Aerial photo of BP7-2 – L Street to Q Street

4.4.4.1. Quantities

Quantities for the optimizations were determined from the civil model and the templates used for both the levee and the floodwall options. Areas where the existing topography showed at a higher elevation than the target elevation did not have a template applied and show lengths of 0 feet in the table below. A negative value on the quantity indicates more material is to be excavated than is required for fill. Although all three elevations scenarios were modeled and material quantities were calculated, since none of the scenarios met the benefit to cost ratio criteria, only quantities for the minimum height raise, the 1% AEP event, are shown below.

At the stage of selecting the preferred alternatives, initial analysis was performed using only 1D steady flows and the assumptions listed above in section 4.4.3.1. During optimization the average height raise was modeled using unsteady flow hydraulic modeling.

Table 27: Analysis results comparison for reach BP7-1

Scenario	Bank	Length (ft)	Average Height Raise (ft)
Initial 1% AEP +3’	Left	3,700	8.8
Initial 1% AEP	Left	3,700	5.8
Optimized 1% AEP	Left	4,407.87	4.8
Initial 1% AEP +3’	Right	3,700	3.6
Initial 1% AEP	Right	3,700	0.6
Optimized 1% AEP	Right	4,527.98	5.4

Table 28: Optimized quantities for reach BP7-1 for 1% AEP event

Bank	Stations	Length of Levee (ft)	Length of Floodwall (ft)	Volume of Earthwork Fill (yd^3)	Volume of Floodwall Concrete (yd^3)	Length of Concrete Forms (ft)
Left Bank	0+00.00 – 2+50.00	250.0	0.0	15430.0	1770.0	99850.0
	2+50.00 – 6+00.00	0.0	0.0			
	6+00.00 – 27+35.00	0.0	2135.0			
	27+35.00 – 38+60.00	1125.0	0.0			
	38+60.00 – 44+07.87	547.87	0.0			
Right Bank	0+00.00 – 13+20.00	0.0	0.0	-2970.0	2700.0	152216.0
	13+20.00 – 45+27.98	0.0	3207.98			

Table 29: Analysis results comparison for reach BP7-2

Scenario	Bank	Length (ft)	Average Height Raise (ft)
Initial 1% AEP +3’	Left	5,600	2.5
Initial 1% AEP	Left	5,600	0.0
Optimized 1% AEP	Left	671.20	4.0
Initial 1% AEP +3’	Right	5,600	1.9
Initial 1% AEP	Right	5,600	0.0
Optimized 1% AEP	Right	5,544.14	4.2

Table 30: Optimized quantities for reach BP7-2 for 1% AEP event

Bank	Stations	Length of Levee (ft)	Length of Floodwall (ft)	Volume of Earthwork Fill (yd^3)	Volume of Floodwall Concrete (yd^3)	Length of Concrete Forms (ft)
Left Bank	0+00.00 – 12+40.00	0.0	1240.0	61185.0	935.0	52970.0
	12+40.00 – 40+00.00	2760.0	0.0			
	40+00.00 – 46+50.27	650.27	0.0			
	0+00.00 – 6+71.20	671.20	0.0			
Right Bank	0+00.00 – 30+25.00	3025.0	0.0	18295.0	2040.0	115198.3
	30+25.00 – 55+44.14	0.0	2519.14			

4.4.5. Reach BP8

This reach spans from Q Street to Harrison Street on both banks. Figure 42 shows the BP8 right bank baseline in pink and the left bank in yellow.

Figure 42: Aerial photo of BP8 – Q Street to Harrison Street

4.4.5.1. Quantities

Quantities for the optimizations were determined from the civil model and the templates used for both the levee and the floodwall options. Areas where the existing topography showed at a higher elevation than the target elevation did not have a template applied and show lengths of 0 feet in the table below. A negative value on the quantity indicates more material is to be excavated than is required for fill. Although all three elevations scenarios were modeled and material quantities were calculated, since none of the scenarios met the benefit to cost ratio criteria, only quantities for the minimum height raise, the 1% AEP event, are shown below.

At the stage of selecting the preferred alternatives, initial analysis was performed using only 1D steady flows and the assumptions listed above in section 4.4.3.1. During optimization the average height raise was modeled using unsteady flow hydraulic modeling.

Table 31: Analysis results comparison for reach BP8

Scenario	Bank	Length (ft)	Average Height Raise (ft)
Initial 1% AEP +3'	Left	5,900	4.1
Initial 1% AEP	Left	5,900	1.1
Optimized 1% AEP	Left	5,897.84	5.7
Initial 1% AEP	Right	5,900	4.7
Initial 1% AEP	Right	5,900	1.7
Optimized 1% AEP	Right	6,106.52	5.8

Table 32: Optimized quantities for reach BP8 for 1% AEP event

<table>
<tr><th>Bank</th><th>Stations</th><th>Length of Levee (ft)</th><th>Length of Floodwall (ft)</th><th>Volume of Earthwork Fill (yd^3)</th><th>Volume of Floodwall Concrete (yd^3)</th><th>Length of Concrete Forms (ft)</th></tr>
<tr><td rowspan="2">Left Bank</td><td>0+00.00 – 24+60.00</td><td>2460.0</td><td>0.0</td><td rowspan="2">87270.0</td><td rowspan="2">0.0</td><td rowspan="2">0.0</td></tr>
<tr><td>24+60.00 – 58+97.84</td><td>3437.84</td><td>0.0</td></tr>
<tr><td>Right Bank</td><td>0+00.00 – 61+06.52</td><td>6106.52</td><td>0.0</td><td>122030.0</td><td>0.0</td><td>0.0</td></tr>
</table>

4.4.6. Reach LP8

This reach spans from the Railroad Bridge to the Little Papillion Creek and Big Papillion Creek confluence on both banks. This reach has been divided into two sub-reaches, LP8-1 from the Railroad Bridge to L Street and LP8-2 from L Street to the confluence. Figure 43 shows the LP8-1 right bank baseline in yellow and the left bank in red. Figure 44 shows the LP8-2 right bank baseline in blue and the left bank in orange.

Figure 43: Aerial photo of LP8-1 – Railroad Bridge to L Street

Figure 44: Aerial photo of LP8-2 – L Street to Confluence

4.4.6.1. Quantities

Quantities for the optimizations were determined from the civil model and the templates used for both the levee and the floodwall options. Areas where the existing topography showed at a higher elevation than the target elevation did not have a template applied and show lengths of 0 feet in the table below. A negative value on the quantity indicates more material is to be excavated than is required for fill. Although all three elevations scenarios were modeled and material quantities were calculated, since none of the scenarios met the benefit to cost ratio criteria, only quantities for the minimum height raise, the 1% AEP event, are shown below.

At the stage of selecting the preferred alternatives, initial analysis was performed using only 1D steady flows and the assumptions listed above in section 4.4.3.1. During optimization the average height raise was modeled using unsteady flow hydraulic modeling.

Table 33: Analysis results comparison for reach LP8-1

Scenario	Bank	Length (ft)	Average Height Raise (ft)
Initial 1% AEP +3’	Left	5,900	7.1
Initial 1% AEP	Left	5,900	4.1
Optimized 1% AEP	Left	3,997.57	7.3
Initial 1% AEP +3’	Right	5,900	5.0
Initial 1% AEP	Right	5,900	2.0
Optimized 1% AEP	Right	3,176.59	7.5

Table 34: Optimized quantities for reach LP8-1 for 1% AEP event

Bank	Stations	Length of Levee (ft)	Length of Floodwall (ft)	Volume of Earthwork Fill (yd^3)	Volume of Floodwall Concrete (yd^3)	Length of Concrete Forms (ft)
Left Bank	0+00.00 – 6+80.00	0.0	0.0	-1630.0	2590.0	145535.1
	6+80.00 – 11+60.00	1234.0	0.0			
	11+60.00 – 39+97.57	0.0	2837.57			
Right Bank	0+00.00 – 11+80.00	1180.0	0.0	3525.0	1300.0	72913.2
	11+80.00 – 18+20.00	640.0	0.0			
	18+20.00 – 31+76.59	0.0	1356.59			

Table 35: Analysis results comparison for reach LP8-2

Scenario	Bank	Length (ft)	Average Height Raise (ft)
Initial 1% AEP +3’	Left	5,900	4.5
Initial 1% AEP	Left	5,900	1.5
Optimized 1% AEP	Left	2,387.97	5.9
Initial 1% AEP +3’	Right	5,900	4.3
Initial 1% AEP	Right	5,900	1.3
Optimized 1% AEP	Right	2,177.33	5.8

Table 36: Optimized quantities for reach LP8-2 for 1% AEP event

Bank	Stations	Length of Levee (ft)	Length of Floodwall (ft)	Volume of Earthwork Fill (yd^3)	Volume of Floodwall Concrete (yd^3)	Length of Concrete Forms (ft)
Left Bank	0+00.00 – 6+00.00	600.0	0.0	53650.0	0.0	0.0
	6+00.00 – 23+87.97	1787.97	0.0			
Right Bank	0+00.00 – 21+77.33	2177.33	0.0	23840.0	0.0	0.0

4.5. WEST PAPILLION CREEK

The TSP alternatives along West Papillion Creek include:

- New levee and/or floodwall construction on both the left and right banks, in areas identified as reaches WP5 and WP6, including upstream tie-off locations.

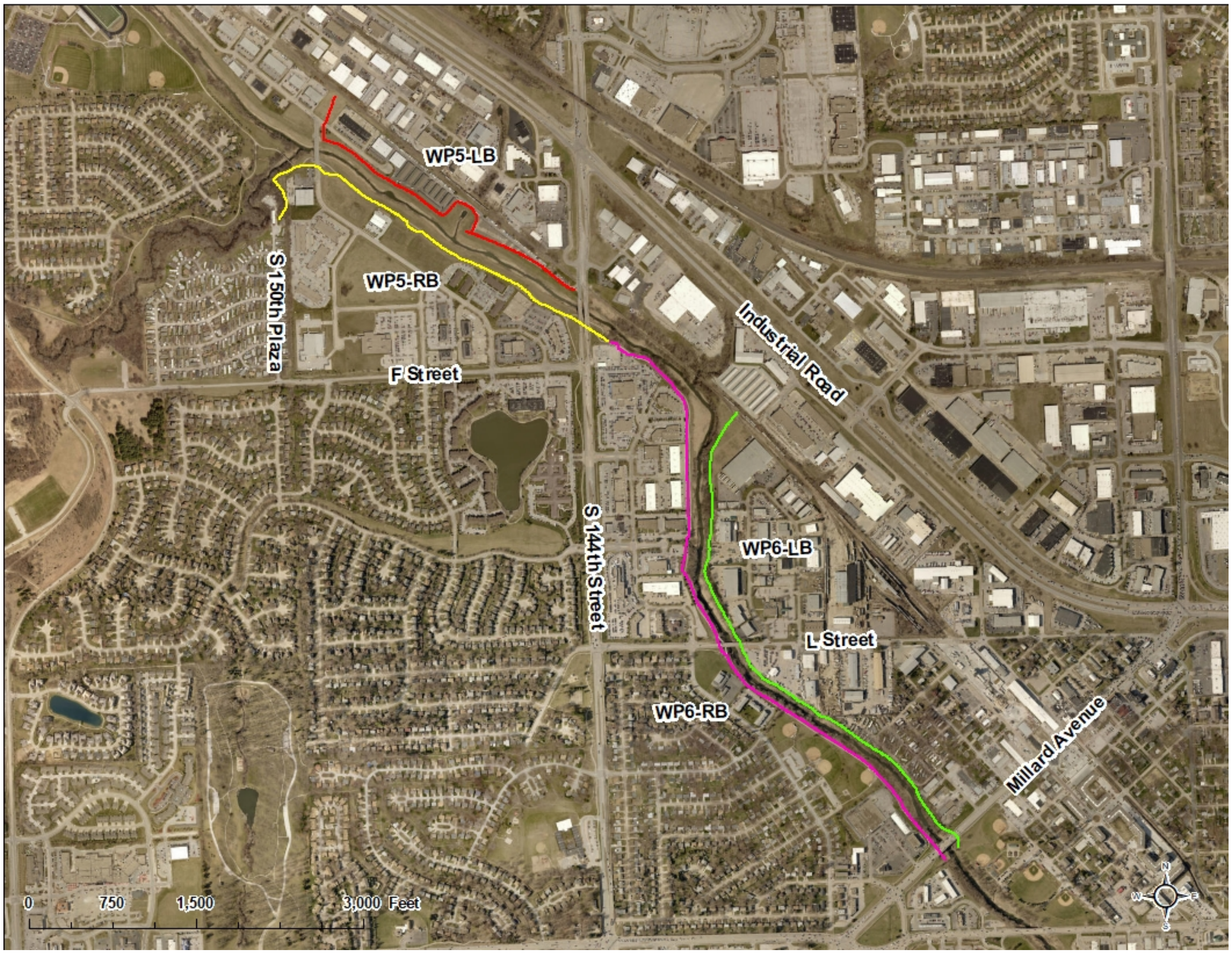

Figure 45: Aerial Photo of West Papillion TSP Alternatives

4.5.1. Basis of Preliminary Design

The existing topography was developed from 2016 LiDAR data and shows minimal height variations on the landside of the channel bank between West Center Road and Millard Avenue, which would indicate a lack of existing levees in this area. The NLD also does not show any existing levees within this area of the West Papillion Creek. The NLD does show existing levees that travel from the confluence with the Big Papillion Creek, upstream along the West Papillion Creek on both banks. However, these existing levees stop between South Washington Street and South 96th Street.

Per EM 1110-2-1913, Design and Construction of Levees; new levees will be constructed as fill prisms that have a minimum 12-foot-wide crown and minimum 3H:1V landside slopes, as shown in the typical cross-section in Figure 26. Preliminary design of new floodwalls consists of an inverted T wall, a 12-foot-wide levee crown on the landside of the floodwall, and a 3H:1V landside slope, as shown in the typical cross-section in Figure 27. The preliminary T wall design was provided by structural engineers and would be a reinforced concrete section. In both cases, a 15-foot-wide vegetation free zone (VFZ) would be added on the landside, measured from the landside toe, to establish real estate and/or easement boundaries.

4.5.1. Initial analysis for TSP alternatives

The initial analysis of alternatives during the screening process used to determine the preferred alternatives in the TSP were based upon limited information and some generalized assumptions. The preferred alternatives were then optimized to determine which alternatives would be carried forward based upon the benefit to cost ratios. The optimization process is discussed in detail in the next section.

The initial screening assumptions include:

- Generalized existing ground surfaces as LiDAR data had not yet been obtained
- One-dimensional steady flow hydraulic modeling to determine target elevations
- Average height raises determined from the hydraulic models
- Average height raises for each reach applied along the entire length of each reach
- Simplified I wall design for floodwalls

4.5.2. Optimization

The Hydraulics Section provided three levee elevations to be evaluated in the optimization process. These elevations corresponded to the energy grade line (EGL) of the 1% annual exceedance probability (AEP) event plus zero feet, the 1% AEP plus three feet, and the 1% AEP plus five feet. The EGL elevation was determined from 1D/2D unsteady flow hydraulic modeling. The target elevation decreases in elevation along the length of each reach therefore a specific value is not listed in this appendix. More detailed information on the determination of the elevations to be evaluated can be found in Appendix B: Hydraulics.

The purpose of optimization was to determine material quantities for construction costs and to establish project footprints to determine required real estate boundaries and/or easements. Achieving the required height raise with a floodwall would produce the minimum real estate requirement whereas using a levee fill prism would produce the maximum real estate requirement. It was determined early in the process that on the West Papillion, floodwalls would be the only option due to the limited space available. The exception to this is on the left bank upstream tie-off where there is a currently undeveloped lot that would allow construction of a tie-back levee.

Since there are no existing flood reduction structures along this section of the West Papillion Creek, a baseline was established to construct a civil model. The baseline was determined by looking at the existing topography and was established along the existing break line of the channel bank.

Within the civil model, templates for a full height levee prism and a full height floodwall wall were created, based upon the typical sections shown in Figures 26 and 27 above. The templates were created with the riverside toe of the structure as the anchor point of the template, with the remainder of the template building to the landside of this anchor point. The template anchor point followed the path of the baseline and the existing ground elevation.

The templates were set to hit the target elevation provided by hydraulics. At each location where the hydraulic model cross-section intersected the geometric baseline, the elevation for the vertical alignment was set to the target elevation. Between each hydraulic cross-section (vertical anchor points), the target elevation is a consistent slope between the two.

The templates were used to determine real estate boundaries and material quantities for each reach for the 1% AEP event. Shapefiles of the project footprints and required real estate boundaries were provided to the real estate section to assist in developing costs, as well as to the environmental specialists to show the areas effected by construction. Material quantities were provided to the Cost engineer to assist in determining the project construction costs.

The optimization process for the 1% AEP elevation resulted in a determination that the creation of new floodwalls was not feasible at this time based upon the benefit to cost ratio. The 1% AEP plus three feet and 1% AEP plus five feet scenarios were not analyzed because of how far below the 1.0 B/C ratio the 1% AEP result was. More detailed information for each reach, from upstream to downstream, is included below.

4.5.3. Utilities

Shapefiles of existing utilities were provided by Metropolitan Utilities District (MUD) and Omaha Public Power District (OPPD). MUD handles water and natural gas services in the Omaha area and OPPD provides electrical services. This data was provided after optimization was completed and the determinations were made that projects along the West Papillion Creek were not feasible going forward. Therefore, utility conflicts and crossings were not evaluated and will not be discussed further regarding the West Papillion Creek.

4.5.4. Reach WP5

This reach spans from West Center Road to South 144th Street on both banks. The upstream tie-off location was selected near South 149th Street for both banks. South 149th Street is downstream from West Center Road. Figure 46 shows the WP5 right bank baseline in yellow and the left bank in red.

Figure 46: Aerial photo of WP5 – South 149th Street to South 144th Street

On the left bank, the alignment was divided into three sub-reaches (WP5-1, WP5-2, WP5-3) in the civil model to allow for the separation of quantities. The middle section on the left bank surrounds a drainage structure and side channel. On the right bank, the alignment was divided into two sub-reaches in the civil model, WP5-1 and WP5-2. The upstream portion (WP5-1) was added as an extension to ensure the new structure was not flanked by flows from the smaller creek or higher flows in the West Papillion Creek. Since none of the scenarios considered in optimization met the benefit to cost ratio criteria, quantities for the 1% AEP event, which is the smallest height raise, are shown below.

At the stage of selecting the preferred alternatives, initial analysis was performed using only steady flows and the assumptions listed above in section 4.5.1. During optimization the average height raise was modeled using unsteady flow hydraulic modeling.

Table 37: Analysis results comparison for reach WP5

Scenario	Bank	Length (ft)	Average Height Raise (ft)
Initial 1% AEP +3’	Left	3,000	6.8
Initial 1% AEP	Left	3,000	3.8
Optimized 1% AEP	Left	1,717.75	3.9
Initial 1% AEP +3’	Right	3,530	5.9
Initial 1% AEP	Right	3,530	2.9
Optimized 1% AEP	Right	3,929.70	4.3

Table 38: Optimized quantities for reach WP5 for 1% AEP event

<table>
<tr><th>Bank</th><th>Stations</th><th>Length of Levee (ft)</th><th>Length of Floodwall (ft)</th><th>Volume of Earthwork Fill (yd^3)</th><th>Volume of Floodwall Concrete (yd^3)</th><th>Length of Concrete Forms (ft)</th></tr>
<tr><td rowspan="3">Left Bank</td><td>0+00.00 – 16+90.20</td><td>275.0</td><td>1404.75</td><td rowspan="3">1434.5</td><td rowspan="3">2294.7</td><td rowspan="3">130143.4</td></tr>
<tr><td>0+00.00 – 5+89.10</td><td>0.0</td><td>589.10</td></tr>
<tr><td>0+00.00 – 11+21.10</td><td>0.0</td><td>1121.10</td></tr>
<tr><td rowspan="2">Right Bank</td><td>0+00.00 – 6+32.90</td><td>0.0</td><td>632.90</td><td rowspan="2">2189.1</td><td rowspan="2">2958.0</td><td rowspan="2">167592.8</td></tr>
<tr><td>0+00.00 – 32+69.80</td><td>0.0</td><td>3269.80</td></tr>
</table>

4.5.5. Reach WP6

This reach spans from South 144th Street to Millard Avenue on both banks. Figure 47 shows the WP6 right bank baseline in pink and the left bank in green.

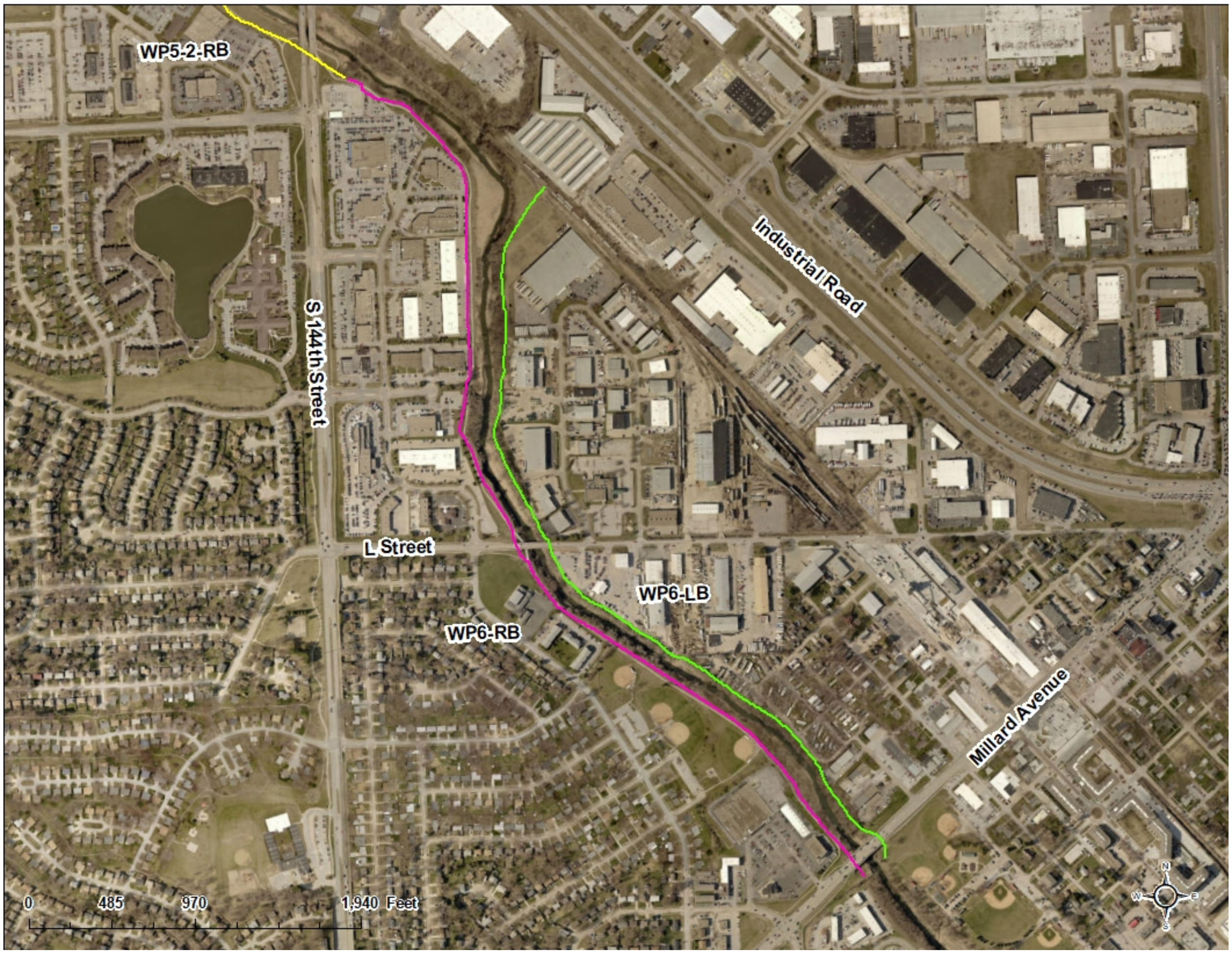

Figure 47: Aerial photo of WP6 – South 144th Street to Millard Avenue

Since none of the scenarios considered in optimization met the benefit to cost ratio criteria, quantities for the 1% AEP event, which is the smallest height raise, are shown below.

At the stage of selecting the preferred alternatives, initial analysis was performed using only steady flows and the assumptions listed above in section 4.5.1. During optimization the average height raise was modeled using unsteady flow hydraulic modeling.

Table 39: Analysis results comparison for reach WP6

Scenario	Bank	Length (ft)	Average Height Raise (ft)
Initial 1% AEP +3’	Left	6,075	6.5
Initial 1% AEP	Left	6,075	3.5
Optimized 1% AEP	Left	5,036	4.9
Initial 1% AEP +3’	Right	6,200	6.0
Initial 1% AEP	Right	6,200	3.0
Optimized 1% AEP	Right	6,052.90	4.5

Table 40: Optimized quantities for reach WP6 for 1% AEP event

Bank	Stations	Length of Levee (ft)	Length of Floodwall (ft)	Volume of Earthwork Fill (yd^3)	Volume of Floodwall Concrete (yd^3)	Length of Concrete Forms (ft)
Left Bank	0+00.00 – 50+36.00	0.0	5036.0	2078.0	3900.6	220704.9
Right Bank	0+00.00 – 60+52.90	0.0	6052.9	3461.8	4589.3	259929.9

5. GEOTECHNICAL ANALYSIS

This section discusses the geotechnical analysis for the alternatives that have been recommended to carry forward. The preferred alternatives in the TSP that are not recommended to be carried forward are not discussed.

5.1.1. Dam Site 19

The preliminary dam and spillway design discussed in this feasibility report is based on a previously completed preliminary design effort for the NRD by HDR Engineering, Inc in 2018. The modifications to the previous HDR preliminary designs, discussed in Section 4.1 above, are to ensure the current USACE design criteria is met.

During the previous HDR design efforts, geotechnical analyses was performed with the preliminary data collected at that time. As a part of this feasibility study, no additional geotechnical analyses were performed for DS19.

During the design phase site specific material parameters for the native soils as well as any borrow materials should be available and used for analysis. Once the dam and spillway designs are near completion, verification geotechnical analysis should be completed.
Additional information on the geotechnical analyses performed, the results, and the design aspects that were included based upon those results can be found in the HDR report. Below is a list of the key points from the previous analyses.

- The stratigraphy and soil properties are similar to the other Papio Dam Sites which have performed well since their construction.
- The foundation materials are suitable to support the construction of a dam embankment and appurtenant structures.
- The dam embankment can be constructed in one season to full height without slope stability issues. Instrumentation should be included to monitor pore pressures during construction.
- The dam embankment will require a chimney drain and horizontal drains to mitigate seepage.

- The dam embankment is expected to settle 32 inches during construction and should be overbuilt by 1 foot to accommodate after construction settlement.

5.1.2. Dam Site 10

The preliminary dam and spillway design discussed in this feasibility report is based on a previously completed design effort by USACE in 1975. The 1975 report, General Design Memorandum MPC-10, Papillion Creek and Tributaries; discusses the design, construction, and development of 20 dams throughout the Papillion Creek Basin. The modifications to the previous design, discussed in Section 4.2 above, are to ensure the current USACE design criteria is met.

During the previous USACE design effort, preliminary geotechnical explorations were completed across the 20 sites in the basin and it was determined that conditions were consistent throughout. Geotechnical analyses were performed for DS10 with the preliminary data. As a part of this feasibility study, no additional geotechnical analyses were performed for DS10.

During the design phase site specific material parameters for the native soils as well as any borrow materials should be available and used for analysis. Once the dam and spillway designs are near completion, verification geotechnical analysis should be completed. Below is a list of the key points from the previous analyses.

- Dam embankment fill material will be available from spillway excavation and from borrow in the pool area.
- Stability analysis meets or exceeds minimum factor of safety requirements.
- Dam embankment crest overbuilt 1 foot to accommodate after construction settlement.
- The dam embankment will require a chimney drain and horizontal drains to mitigate seepage.

5.1.3. Little Papillion Creek

It is recommended that the project along the Little Papillion Creek, reaches LP5-Up to LP8, be carried forward after the optimization process. This portion of the project consists of constructing new levees, new floodwalls, or a combination of the two to reach the desired height raise. Currently, there are no existing levees within this portion of the Little Papillion Creek, however there was a channel widening project in this area in the late 1960’s to reduce flood risk.

After review of the most recent available Levee Periodic Inspection Reports for the existing levees along the Big Papillion Creek and the Little Papillion Creek there does not appear to be a history of seepage or slope stability problems with the existing levees. Based upon this information, with appropriate design, material selection and construction methods, seepage and slope stability should not be a concern with new structures on the Little Papillion Creek.

The Periodic Inspection Report that included inspection of the channel widening project on the Little Papillion Creek did indicate a history of high ground water and some surficial bank sloughing, to varying degrees, on the riverside banks in the channel widening areas.

Site specific explorations and material parameters were not available, however limited borings were performed during the channel widening project and improvements around the UPPR bridge. Geotechnical analyses were conducted on a full-height levee prism and full-height floodwall using generalized material parameters. The generalized parameters were selected based upon available information from areas along the Big and Little Papillion Creeks.

5.1.3.1. Material Parameters

Currently, site specific information on the existing native soils along the Little Papillion Creek and levee fill material were unavailable. However, information on previous explorations along the Big Papillion Creek were available, as discussed above. For the geotechnical analyses, the historical information was reviewed to determine the generalized parameters used for this preliminary analysis.

The parameters, test data, and test methods used in design of the channel control structure should be reviewed carefully. Based on preliminary review, it appears soil strength results are for drained conditions. It should also be noted that some of the testing was completed using remolded samples and this increases uncertainties in the test result data. Special care will be needed where future design analyses rely on existing data.

The historical data and sample soils were classifieds clays, as shown in Table 41 below. Local experience of the project area has shown a common presence of silts in the Loess, however. Additional explorations, sampling, testing, and analyses will be needed to allow for the use of site-specific soil parameters for design refinements during the design phase of the project.

Information and layer elevations for the existing materials in Zones A through F and the Bedrock were found in the testing data included in Feature Design Memorandum No. MPC-53 Volume 1, Channel Control Structure, Big Papillion Channel Improvements, Papillion Creek and Tributaries Lakes, Omaha, Nebraska, USACE, 1991. The improvement project alignment extends along the Big Papillion Creek from Center, downstream to L street. The control structure is beneath the railroad crossing, approximately 700 feet south of Interstate 80.

Table 41: Soil parameters used for preliminary geotechnical analyses

Material	Layer Elevation	Horizontal Permeability (ft/sec)	Total Unit Weight (pcf)	Cohesion (psf)	Internal Angle of Friction (Total Stress)
Zone A – CL	Above 1013	1.31E -07	123	360	27.9
Zone B – CL	1005 - 1013	3.28E -08	121	0	29.7
Zone C – CL	1001 - 1005	1.64E -06	118	0	28.4

Zone D – CL	994 - 1001	3.28E -07	121	0	29.7
Zone E – CL	984 - 994	1.64E -05	121	0	29.7
Zone F – CL	968 - 984	3.28E -06	121	0	29.7
Bedrock - Limestone	Below 968	2.62E -05	165	0	45
Levee Fill – CL	-	3.28E -07	120	150	24
Floodwall – Concrete	-	1.87E -11	150	-	-

5.1.3.2. Cross-Section Selection

The cross-section selected for analysis is at station 5+40 on the left bank of LP7, on the Little Papillion Creek. This location is approximately 500 feet downstream of Pacific Street. The existing ground surface cross-section was pulled from the LiDAR generated civil model. This cross-section was chosen for the following reasons:

- Within the Little Papillion Creek project area, which is the only area of levees and/or floodwalls that have been selected to carry forward after the optimization process
- Largest height increase, approximately 8 feet, between existing ground surface and the target elevation for the 1% AEP plus 3 feet event

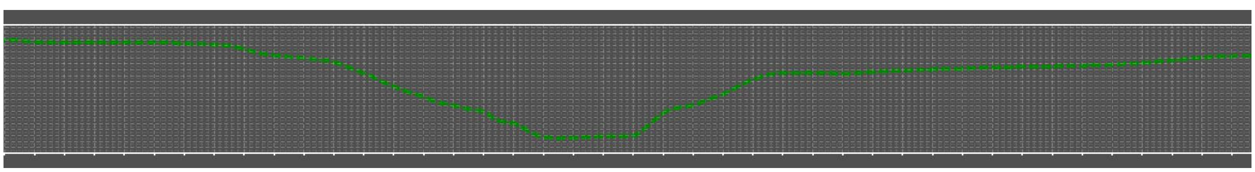

Figure 48: Existing ground cross-section from LiDAR topography in civil model

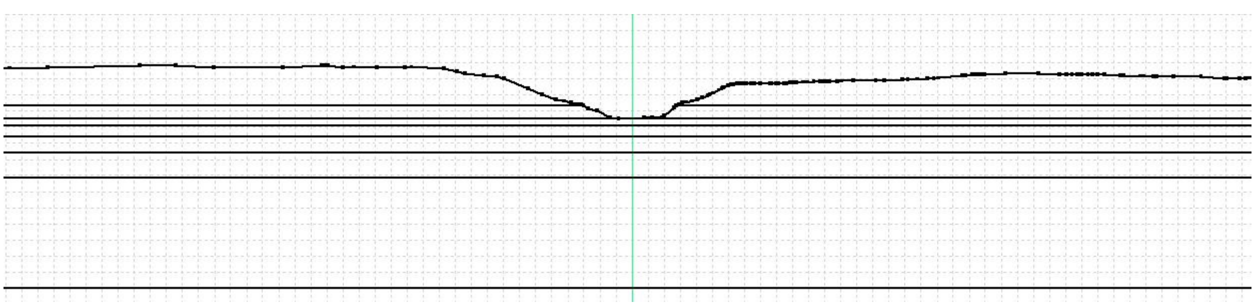

Figure 49: Existing ground cross-section in GeoStudio Models

Within both above cross-sections, and the model images to follow, the views are looking upstream, and therefore the left bank is on the right half of the model and images. For the purpose of the modeling, the levee and floodwall alternatives are only modeled on the left bank, as a stand-alone structure. The corresponding structure on the right bank is not modeled because the ground is higher, which lowers embankment height.

5.1.3.3. New Levees

The new levee geometry is comprised of a 12-foot wide levee crown with 3H:1V side slopes. The target elevation (1% AEP + 3') at the selected cross-section is elevation 1035.25, which results in an approximately 8.25-foot height raise from the existing ground surface. A 1-foot deep swath of the existing ground was removed and incorporated into the levee prism, resulting in 9.25 feet of levee fill at the centerline. This 1-foot depth was intended to cover removal of sod and topsoil, sidewalks and other unsuitable surface material.

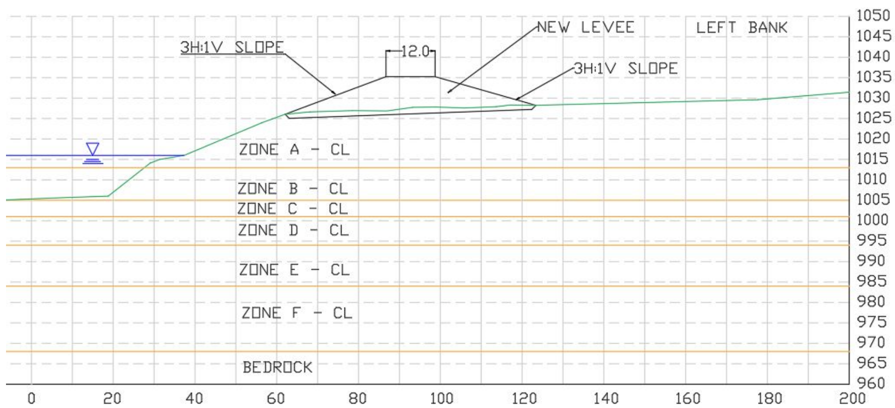

Figure 50: New levee at selected cross-section

5.1.3.3.1. Seepage Analysis

Levee seepage was evaluated using the computer program SEEP/W 2019. SEEP/W is a two-dimensional finite element seepage analysis software program developed by GEO-SLOPE International, Ltd. (2019). For the steady-state seepage analysis, the design water surface boundary condition elevation was set to 1035.0, which is 0.25 feet below the top of the new levee crown. A potential seepage face boundary condition was applied to the landside of the levee. The assumption was made that during a flood event along the Little Papillion Creek that flows would be elevated long enough to allow a steady-state, saturated condition to occur, and therefore only a steady-state seepage analysis was performed.

The landside boundary condition of the model was set to elevation 1025.0, which is approximately the same elevation as the existing ground at the riverbank edge. Based upon reports of high ground water in this area the assumption was made that the groundwater elevation would be near the existing top of slope.

Due to the limited borings and soil data available, the stratigraphy was assumed to be consistent across the area and the material layers were modeled as horizontal layers. During the design phase, when site specific data is available the layering should be modified to reflect any deviations and/or elevation changes in the layering.

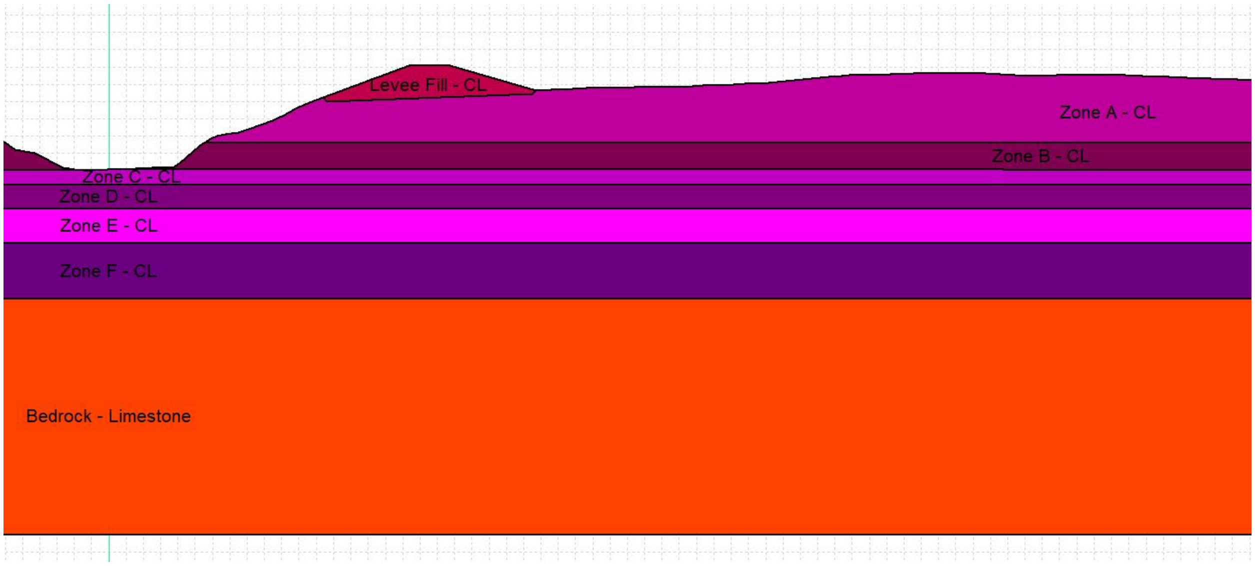

Figure 51: New levee model with material layering

Color	Name	Model	Sat Kx (ft/sec)	Ky'/Kx' Ratio
	Bedrock - Limestone	Saturated Only	2.62e-05	0.5
	Levee Fill - CL	Saturated Only	3.28e-07	0.25
	Zone A - CL	Saturated Only	1.31e-07	0.25
	Zone B - CL	Saturated Only	3.28e-08	0.25
	Zone C - CL	Saturated Only	1.64e-06	0.25
	Zone D - CL	Saturated Only	3.28e-07	0.25
	Zone E - CL	Saturated Only	1.64e-05	0.25
	Zone F - CL	Saturated Only	3.28e-06	0.25

Figure 52: Materials legend and parameters from steady-state seepage model

Per the criteria listed in EM 1110-2-1913, Design and Construction of Levees; the maximum seepage exit gradient at the levee toe is 0.5. With the material parameters selected, the seepage model is showing an exit gradient at the new levee toe of 0.38 which meets the current criteria.

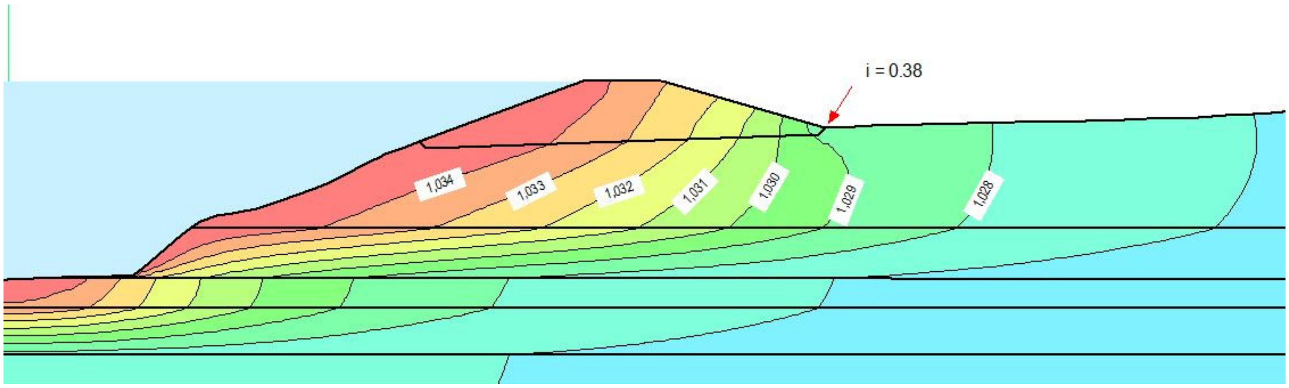

Figure 53: New levee seepage results showing total head contours

5.1.3.3.2. *Long-term Slope Stability Analysis*

The stability of the landside levee slope was evaluated using the computer program SLOPE/W 2019. SLOPE/W is a two-dimensional limit equilibrium stability analysis software program developed by GEO-SLOPE International, Ltd. (2019). For the long-term slope stability analysis, Spencer's method was selected, and a minimum slip surface depth of 1 foot was set. Spencer's method is a two-dimensional limit equilibrium method of analysis that satisfies all conditions of equilibrium (moment and force).

The pore water pressure conditions from the steady state seepage analysis were used in the long-term slope stability analysis. A surcharge load of 250 psf was applied to the crown of the levee. The EM 1110-2-1913 criteria for landside stability under steady-state conditions is a minimum factor of safety of 1.4. The slope stability model result is showing a factor of safety of 2.86 for the critical slip surface.

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock - Limestone	Bedrock (Impenetrable)			
	Levee Fill - CL	Mohr-Coulomb	120	150	24
	Zone A - CL	Mohr-Coulomb	123	360	27.9
	Zone B - CL	Mohr-Coulomb	121	0	29.7
	Zone C - CL	Mohr-Coulomb	118	0	28.4
	Zone D - CL	Mohr-Coulomb	121	0	29.7
	Zone E - CL	Mohr-Coulomb	121	0	29.7
	Zone F - CL	Mohr-Coulomb	121	0	29.7

Figure 54: Materials legend and parameters from long-term slope stability model

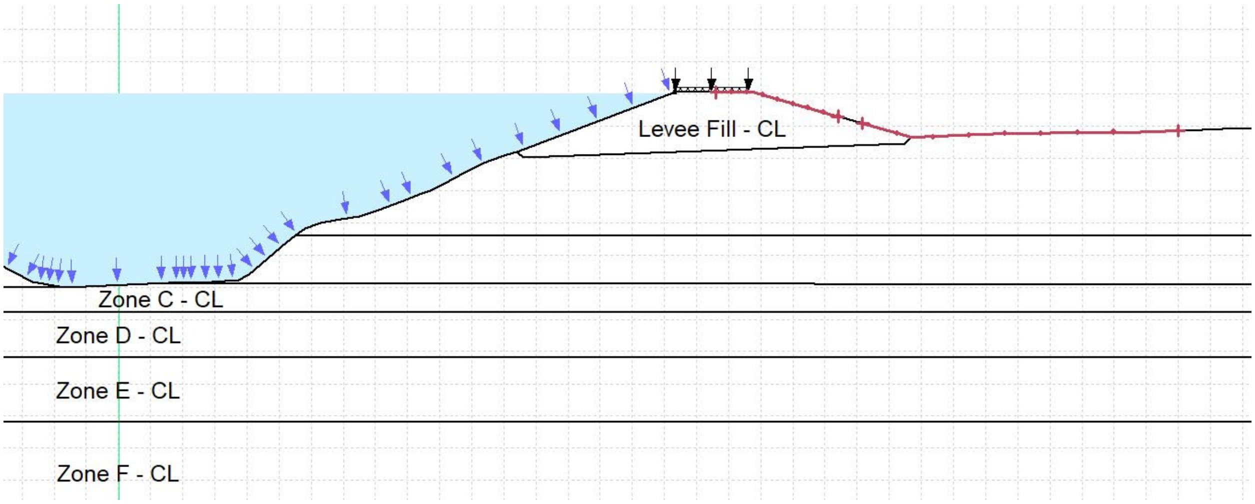

Figure 55: New levee long-term slope stability model setup with slip surface definitions

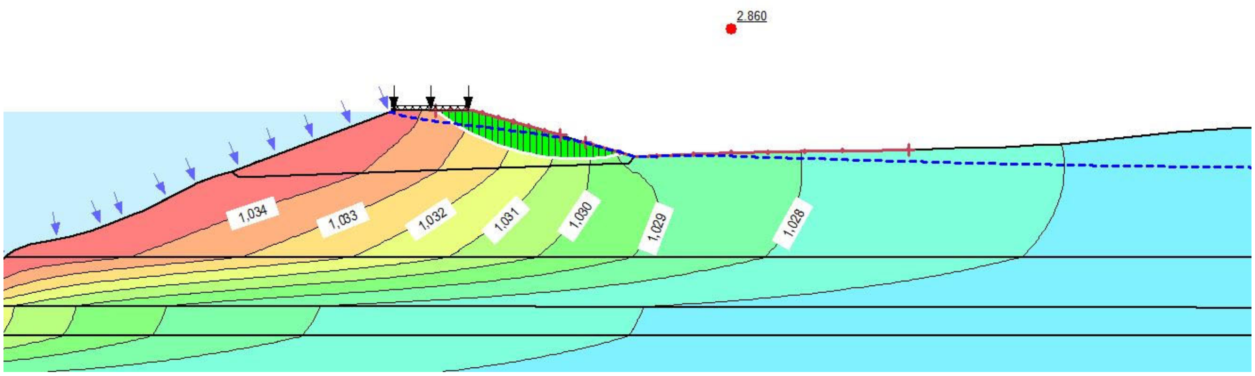

Figure 56: New levee long-term slope stability results showing the critical slip surface

5.1.3.3.3. Rapid Drawdown Slope Stability Analysis

The stability of the riverside levee slope was evaluated using the computer program SLOPE/W 2019. For the rapid drawdown slope stability analysis, Spencer's method was selected, and a minimum slip surface depth of 1 foot was set. Spencer's method is a two-dimensional limit equilibrium method of analysis that satisfies all conditions of equilibrium (moment and force).

A surcharge load of 250 psf was applied to the crown of the levee. The pore water pressures were set by defining two piezometric lines, one at elevation 1035.0 and the second at elevation 1015.0, representing a drawdown of 20 feet.

The EM 1110-2-1913 criteria for rapid drawdown stability for new levees is a minimum factor of safety of 1.2. The rapid drawdown slope stability model result is showing a factor of safety of 1.31 for the critical slip surface.

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)	Cohesion R (psf)	Phi R (°)
	Bedrock - Limestone	Bedrock (Impenetrable)					
	Levee Fill - CL	Mohr-Coulomb	120	150	24	160	14
	Zone A - CL	Mohr-Coulomb	123	360	27.9	365	12
	Zone B - CL	Mohr-Coulomb	121	0	29.7	500	20
	Zone C - CL	Mohr-Coulomb	118	0	28.4	500	20
	Zone D - CL	Mohr-Coulomb	121	0	29.7	300	16
	Zone E - CL	Mohr-Coulomb	121	0	29.7	300	16
	Zone F - CL	Mohr-Coulomb	121	0	29.7	300	16

Figure 57: Materials legend and parameters from rapid drawdown stability model

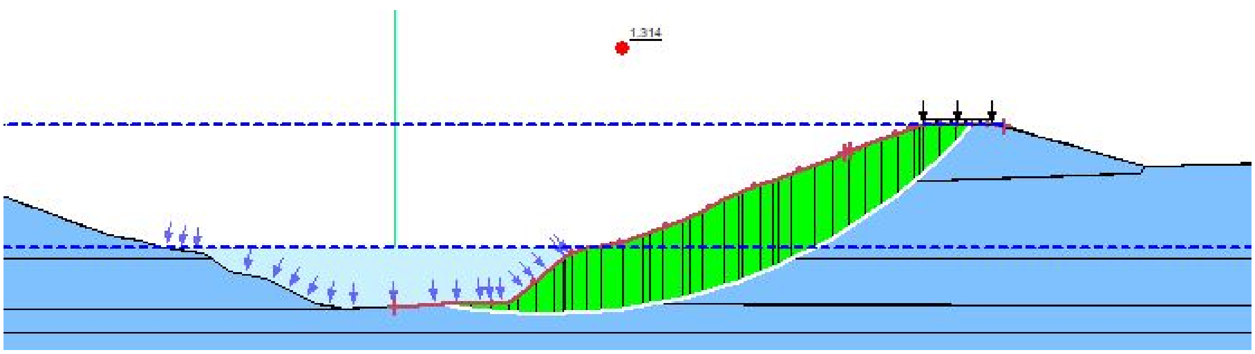

Figure 58: New levee rapid drawdown slope stability results showing the critical slip surface

5.1.3.4. New Floodwalls

The new floodwall geometry which was provided by structural engineers as a preliminary design, is a reinforced concrete inverted T wall. It is comprised of a 1-foot wide stem with a 12-foot wide by 1-foot thick base buried under 4 feet of cover. The stem is offset from center towards the riverside by 1 foot. A 12-foot wide levee crown is applied from the landside face of the wall, with a 3H:1V landside slope down to the existing grade. The excavation for the floodwall is cut back on 1H:1V slopes.

During backfill on the riverside of the floodwall, it is assumed a 3H:1V slope will be used up to an elevation matching the new levee crown elevation. The crown elevation is set to the existing ground elevation prior to construction. The target elevation (1% AEP + 3') at the selected cross-section is elevation 1035.25, which results in an approximately 8.25-foot tall floodwall, above ground, at the centerline.

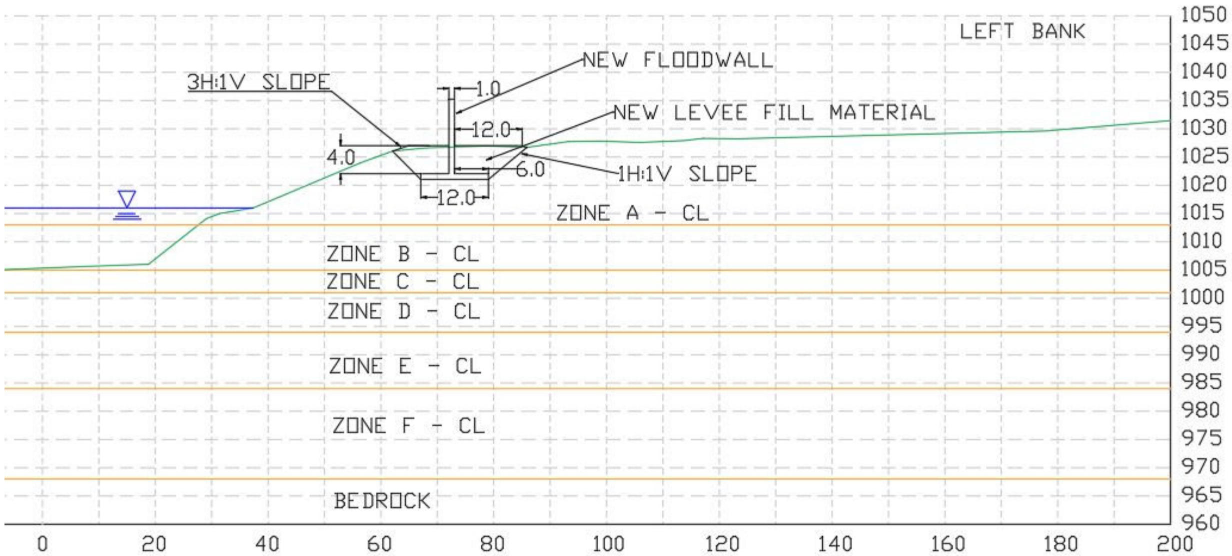

Figure 59: New floodwall at selected cross-section

5.1.3.4.1. Seepage Analysis

Seepage underneath the floodwall was evaluated using the computer program SEEP/W 2019. SEEP/W is a two-dimensional finite element seepage analysis software program developed by GEO-SLOPE International, Ltd. (2019). For the steady-state seepage analysis, the design water surface boundary condition elevation was set to 1035.0, which is 0.25 feet below the top of the new floodwall. The assumption was made that during a flood event along the Little Papillion Creek that flows would be elevated long enough to allow a steady-state, saturated condition to occur, and therefore only a steady-state seepage analysis was performed.

A potential seepage face boundary condition was applied to the ground surface on the landside of the floodwall. A no flow boundary was set around the below ground perimeter of the floodwall to prevent flow from attempting to flow through the floodwall cross-section.

The landside boundary condition of the model was set to elevation 1025.0, which is approximately the same elevation as the existing ground at the riverbank edge. Based upon reports of high ground water in this area the assumption was made that the groundwater elevation would be near the existing top of slope.

Due to the limited borings and soil data available, the stratigraphy was assumed to be consistent across the area and the material layers were modeled as horizontal layers. During the design phase, when site specific data is available the layering should be modified to reflect any deviations and/or elevation changes in the layering.

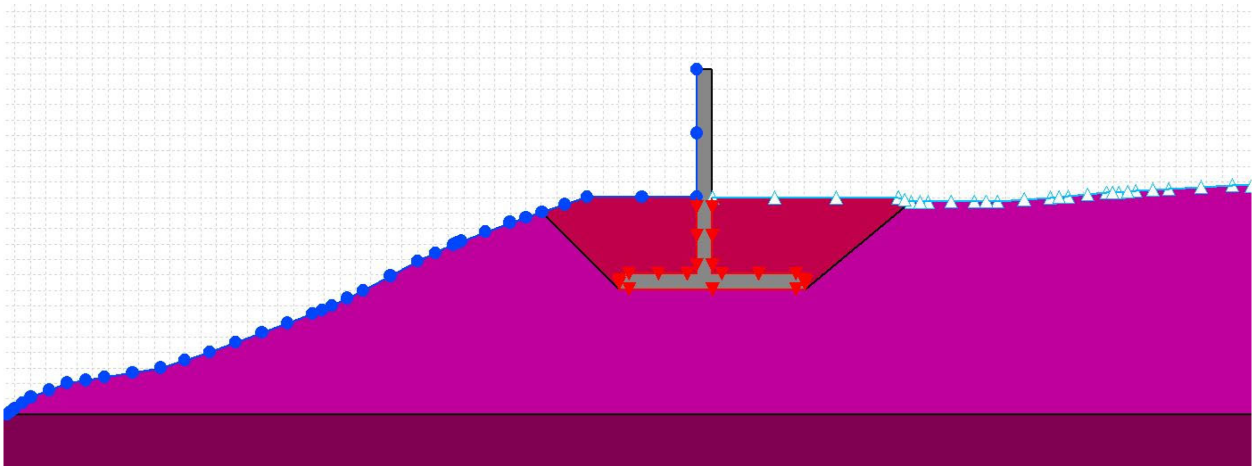

Figure 60: New floodwall steady-state seepage model configuration

Per the criteria listed in EM 1110-2-1913, Design and Construction of Levees; the maximum seepage exit gradient at the levee toe is 0.5. With the material parameters selected, the seepage model is showing an exit gradient at the new levee toe of 0.16 which meets the current criteria.

Color	Name	Model	Sat Kx (ft/sec)	Ky'/Kx' Ratio
	Bedrock - Limestone	Saturated Only	2.62e-05	0.5
	Floodwal - Concrete	Saturated Only	1.87e-11	1
	Levee Fill - CL	Saturated Only	1.64e-06	0.25
	Zone A - CL	Saturated Only	1.31e-07	0.25
	Zone B - CL	Saturated Only	3.28e-08	0.25
	Zone C - CL	Saturated Only	1.64e-06	0.25
	Zone D - CL	Saturated Only	3.28e-07	0.25
	Zone E - CL	Saturated Only	1.64e-05	0.25
	Zone F - CL	Saturated Only	3.28e-06	0.25

Figure 61: Materials legend and parameters from steady-state seepage model

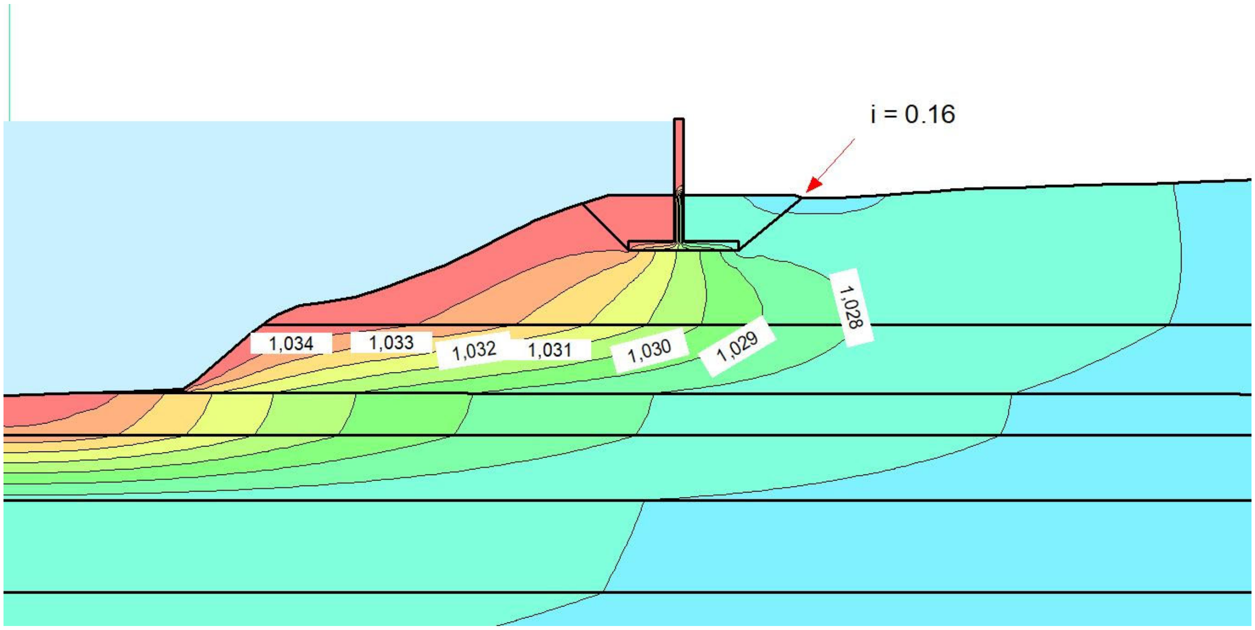

Figure 62: New floodwall seepage results showing total head contours

5.1.3.4.2. Uplift Pressures

Per EM 1110-2-2100, Stability Analysis of Concrete Structures, the minimum Factor of Safety for Flotation is 1.2 for all structures for an unusual loading condition.

$$FS_f = \frac{W_S + W_C + S}{U - W_G} \geq 1.2$$

$W_S = (1 * 12.25 * 150) + (1 * 12 * 150) + (5 * 4 * 60.6) + (6 * 4 * 60.6) = 6303.9\ lb/ft$
$W_C = (5 * 4 * 62.4) + (6 * 4 * 62.4) = 2745.6\ lb/ft$
S = 0; no surcharge load
$W_G = (5 * 8.25 * 62.4) = 2574.0\ lb/ft$
$U = (0.5 * 821.4 * 12) = 4928.4\ lb/ft$

$$FS_f = \frac{6303.9 + 2745.6 + 0}{4928.4 - 2574.0} = \frac{9049.5}{2354.4} = 3.84$$

Uplift pressures along the base of the floodwall were calculated in the seepage model. A graph of the water pressure versus distance along the floodwall base is included below. At the riverside edge of the base, the water pressure is 821.4 psf. At the landside edge of the base, the water pressure is 401.8 psf. The floodwall base is 12 feet wide.

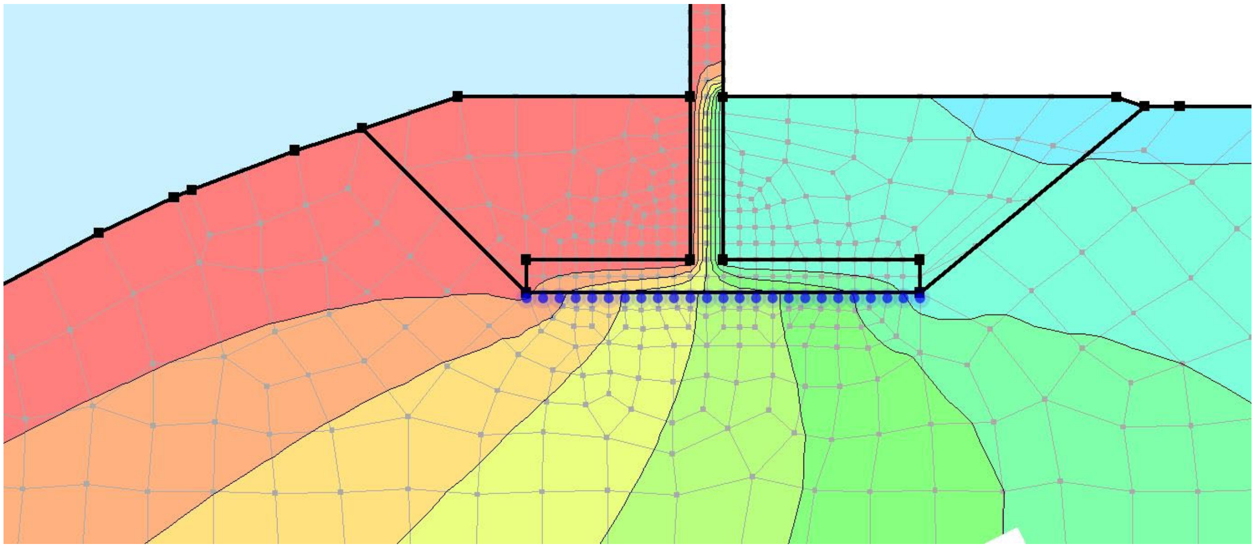

Figure 63: Location of uplift calculations along base of floodwall

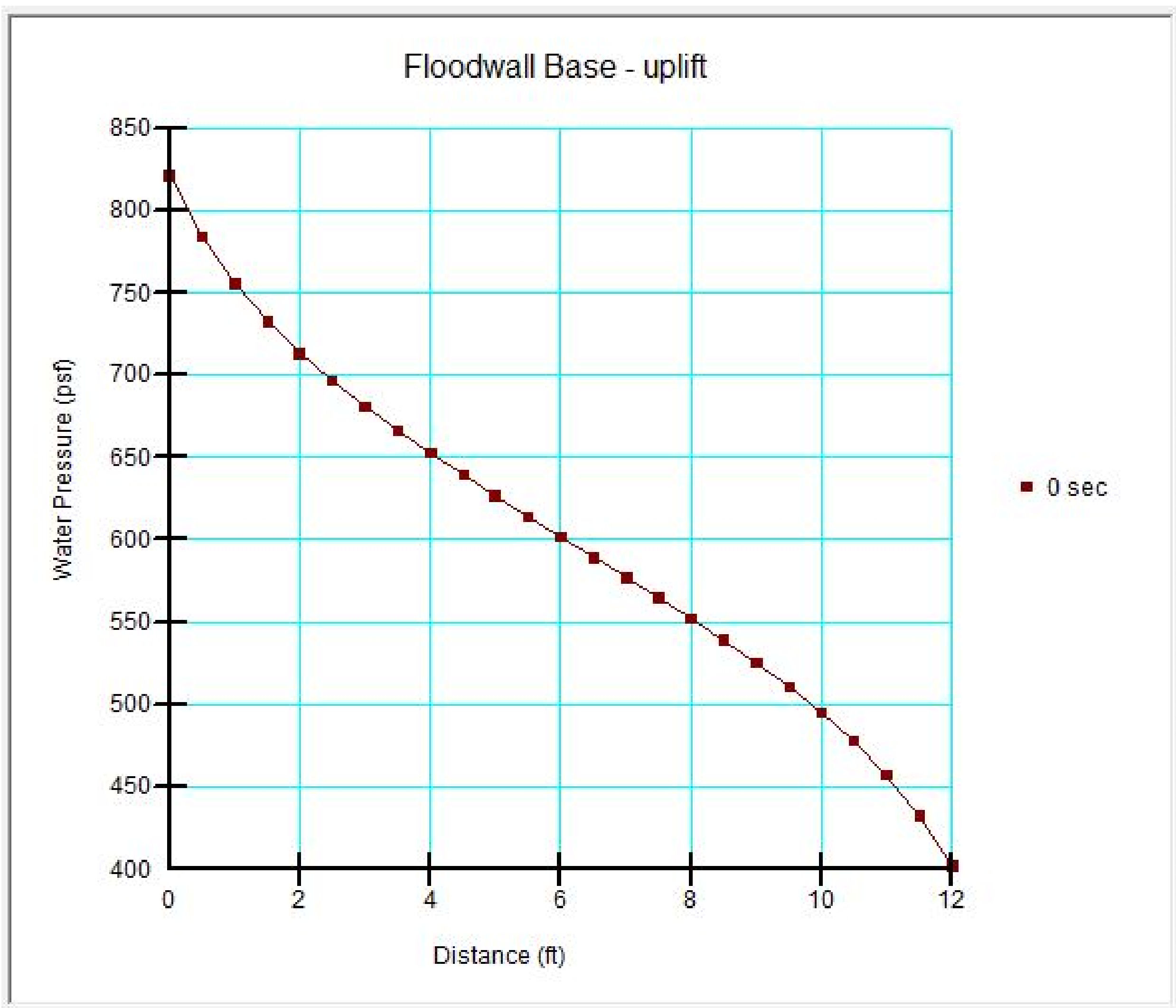

Figure 64: Graph of uplift calculations along floodwall base – Water Pressure vs. Distance

5.1.3.4.3. Long-term Slope Stability Analysis

Per EM 1110-2-2100, Stability Analysis of Concrete Structures, inland floodwalls should be evaluated for Loading Condition I1 – Infrequent Floods, which is considered an unusual loading condition. For a feasibility level analysis, the single-wedge method can be used for estimating the forces in the analysis.

Since generalized parameters are being used, due to a lack of site-specific information, the site information for analysis is categorized as limited site information. According to Table 3-3 (EM 1110-2-2100) for a normal structure with limited site information and an unusual loading condition, the minimum Factor of Safety for Sliding is 2.6.

$$FS_s = \frac{N \tan \varphi + c\,L}{T} \geq 2.6$$

$$W = (1 * 12.25 * 150) + (1 * 12 * 150) + (4 * 5 * 123) + (4 * 6 * 123) = 9049.5\ lb/ft$$
$$U = (from\ above) = 4928.4\ lb/ft$$
$$N = W - U = 0 = 9049.5 - 4928.4 = 4121.1\ lb/ft$$
$$T = P_w = (0.5 * 62.4 * 8.25^2) = 2123.6\ lb/ft$$
$$L = (base\ width\ of\ floodwall) = 12\ ft$$
$$c = (cohesion\ of\ Zone\ A\ soil, from\ above) = 360\ psf$$
$$\varphi = (friction\ angle\ of\ Zone\ A\ soil, from\ above) = 27.9°$$

$$FS_s = \frac{(4121.1 * \tan(27.9)) + (360 * 12)}{2123.6} = \frac{2182.0 + 4320.0}{2123.6} = \frac{6502.0}{2123.6} = 3.06$$

5.1.3.4.4. Bearing Capacity

Per EM 1110-2-2502, Retaining and Floodwalls, Table 4-2, for an inland floodwall with water to the top of the wall, the minimum Factor of Safety for Bearing Capacity is 2.0. For an inland floodwall with a design flood loading condition, the minimum Factor of Safety for Bearing Capacity is 3.0. Regional experience has recommended a minimum Factor of Safety of 3.0 for a wall footing on a clay foundation.

Per EM 1110-1-1905, Bearing Capacity of Soils, the ultimate bearing capacity of soil is determined by Equation 4-1:

$$q_u = cN_c\zeta_c + \frac{1}{2}B\Upsilon'_H N_\Upsilon \zeta_\Upsilon + \sigma'_D N_q \zeta_q$$

$$Shape\ factors = \zeta_c, \zeta_\Upsilon, \zeta_q = 1\ for\ strip\ footings$$
$$Inclination\ factors = 1\ because\ the\ load\ acts\ perpendicular\ to\ the\ base$$
$$Base\ Tilt\ factors = 1\ because\ the\ bottom\ of\ the\ base\ is\ not\ tilted$$
$$Ground\ Slope\ factors = 1\ because\ the\ ground\ at\ the\ landside\ toe\ is\ not\ sloped$$

$c = 360\, psf$
$B = 12\, ft$
$\Upsilon'_H = (123.0 - 62.4) = 60.6\, pcf$
$D = 5\, ft$
$\sigma'_D = \Upsilon_H{}'D = (60.6) * (5) = 303\, psf$
$\varphi = 27.9°$
$N_q = [e^{\pi tan\varphi}]tan^2\left(45° + \frac{\varphi}{2}\right) = [e^{\pi tan27.9}]tan^2\left(45 + \frac{27.9}{2}\right) = 14.56$
$N_c = \left(N_q - 1\right)cot\varphi = (14.56 - 1)cot\, 27.9 = 25.61$
$N_\Upsilon = \left(N_q - 1\right)\tan(1.4\varphi) = 13.56 \tan(39.06) = 11.00$

$$q_u = (360 * 25.61) + \frac{1}{2}(12 * 60.6 * 11.00) + (303 * 14.56) = 17630.88\, psf$$

$$q_a \rightarrow Allowable\ Design\ Load: q_a = \frac{q_u}{FS} = \frac{17630.88}{3.0} = 5876.96\, psf$$
$$\rightarrow 5876.96\, lb/ft\ per\ foot\ of\ footing$$

$Check\ is\ N \leq\ q_a :\ 4121.1\, lb/ft \leq 5876.96\, lb/ft : True$

6. DESIGN CONSIDERATIONS

This section discusses the current elements of the preliminary design for the alternatives that have been recommended to carry forward as well as design considerations and recommendations for future design phases for each of the features. The preferred alternatives in the TSP that are not recommended to be carried forward are not discussed.

6.1. DAM SITE 19

6.1.1. Current Design Elements and Features

- 25-foot wide dam embankment crest
- Continuous chimney drain 6 feet thick with 1H:1V side slopes
- Continuous blanket drain 3 feet thick
- Continuous inspection trench along dam embankment centerline 10 feet wide at base, 2H:1V side slopes and with a minimum depth of 6 feet
- Continuous foundation excavation/scarification within dam embankment footprint with a minimum depth of 2 feet
- Downstream dam embankment slope of 3.5H:1V above elevation 1164.0 and 6H:1V below elevation 1164.0
- Upstream dam embankment slope of 3.5H:1V above elevation 1170.0, a 20H:1V bench between elevations 1169.0 and 1170.0, a slope of 3H:1V between elevations 1159.5 and 1169.0, and a slope of 6H:1V below elevation 1159.5
- Upstream riprap protection from the toe of the dam embankment up to elevation 1169.0 due to wind-wave action at the normal pool elevations

- An uncontrolled intake riser tower with a low-level intake and a primary outlet conduit located in the left abutment and founded on glacial drift material
- A primary outlet conduit that is a 72-inch diameter reinforced concrete conduit 400 feet in length, with an inlet elevation of 1164.0
- A filter around the primary outlet conduit near the outfall and riprap revetment in the stilling basin
- An earthen auxiliary spillway in the right abutment of the dam embankment with 3H:1V side slopes
- Over excavation of the spillway base width to a minimum depth of 5 feet to be replaced with cohesive material

6.1.2. Future Design Elements and Recommendations

- Additional geotechnical investigations within the dam embankment and spillway footprints and characterization of the Red Cloud Formation
- Site specific material parameters for both native materials and required borrow materials
- Additional geotechnical analysis based upon finalized design and site-specific material parameters
- Possible inclusion of an upstream impervious blanket at the abutments to reduce seepage through the native materials (Loess)
- Inclusion of appropriate instrumentation to meet dam safety requirements
- Determination on the need for a dewatering plan during construction
- Further design details for the primary outlet including intake tower and low-level intake
- Determination of access routes for construction as well as operations and maintenance
- Grading plan and possible existing streambank alteration to channelize flow into the primary outlet conduit
- Grading plan and possible existing streambank alteration to channelize from out of the primary conduit back into the existing streambank
- Grading plan within the reservoir and flowage easement footprints
- Grading plan or extended channelizing for auxiliary spillway flows to direct flow back towards existing streambank
- Inclusion of riprap revetment in areas of high velocity flow from the spillway or primary outlet conduit

6.2. DAM SITE 10

6.2.1. Current Design Elements and Features

- 25-foot wide dam embankment crest
- Continuous chimney drain 6 feet thick with 1H:1V side slopes
- Continuous blanket drain 3 feet thick
- Continuous inspection trench along dam embankment centerline 10 feet wide at base, 2H:1V side slopes and with a minimum depth of 6 feet
- Continuous foundation excavation/scarification within dam embankment footprint with a minimum depth of 2 feet
- Downstream dam embankment slope of 6H:1V

- Upstream dam embankment slope of 5H:1V
- A primary outlet conduit that is an 8-foot wide by 7-foot tall reinforced concrete box culvert that is 700 feet in length, placed in the left abutment, with an inlet elevation of 1154.0
- A filter around the primary outlet conduit near the outfall and riprap revetment in the stilling basin
- An earthen auxiliary spillway on the left abutment of the dam embankment with 3H:1V side slopes
- Over excavation of the spillway base width to a minimum depth of 5 feet to be replaced with cohesive material

6.2.2. Future Design Elements and Recommendations

- Additional geotechnical investigations within the dam embankment and spillway footprints and characterization of the Kansan glacial drift foundation
- Site specific material parameters for both native materials and required borrow materials
- Additional geotechnical analysis based upon finalized design and site-specific material parameters
- Possible inclusion of an upstream impervious blanket at the abutments to reduce seepage through the native materials (Loess)
- Inclusion of appropriate instrumentation to meet dam safety requirements
- Determination on the need for a dewatering plan during construction
- Determination of access routes for construction as well as operations and maintenance
- Grading plan and possible existing streambank alteration to channelize flow into the primary outlet conduit
- Grading plan and possible existing streambank alteration to channelize from out of the primary conduit back into the existing streambank
- Grading plan within the reservoir and flowage easement footprints
- Grading plan or extended channelizing for auxiliary spillway flows to direct flow back towards existing streambank
- Inclusion of riprap revetment in areas of high velocity flow from the spillway or primary outlet conduit

6.3. LITTLE PAPILLION NEW LEVEES AND FLOODWALLS

6.3.1. Current Design Elements and Features

- New levee fill prism based upon typical figure in Figure 26 above
- New floodwall design based upon typical figure in Figure 27 above – T wall design provided by structural engineers
- 12-foot wide crown width applied atop new levees and from landside face of floodwall
- 3H:1V landside slopes
- 15-foot wide vegetation free zone applied from toe of landside slope for both options
- Baseline alignment set to riverside toe/hinge point outside existing floodway boundary
- Closure structures across all roadways to provide continuous flood risk reduction

6.3.2. Future Design Elements and Recommendations

- A traditional topographic survey which includes utility locates and identifies the location of existing structures and features (roads, sidewalks, river walks, pedestrian bridges, etc.) will be needed prior to beginning design
- Additional geotechnical analysis based upon finalized design and site-specific material parameters
- Re-evaluation of floodwall design based upon required height raises in each area
- Consideration of a combined levee raise and floodwall solution in some areas
- Possible need to overbuild section to account for settlement
- Adjustments to preliminary baseline alignments towards the floodway boundary to avoid commercial structures in some areas
- Design of levee and/or floodwall tie-ins at existing roadways and to closure structures
- Boundaries of obtained real estate and construction easements
- Determination of access points for both temporary and permanent easements
- Determination of turn-around areas and inclusion in design
- Determination of staging and stockpiling areas
- Evaluation of existing utilities to determine if relocations are possible or if other measures need to be taken in the design to address utilities
- Determination if existing river walks within the project footprint will be re-constructed
- Coordination with the Levee Safety Office on design and requirements for the application of the vegetation free zone and crown widths in floodwall areas

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_____. 1991. Feature Design Memorandum No. MPC-53, Volume 2, Appendices, Channel Control Structure, Big Papillion Channel Improvements, Papillion Creek and Tributaries Lakes, Omaha, Nebraska. March.

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Geo-Slope International, Ltd. 2019. GeoStudio SLOPE/W. Two-Dimensional Slope Stability Analyses Using the Limit Equilibrium Computer Program. Version 10.0.0.17401.

8. ATTACHMENTS

8.1. DS19 BACKGROUND INFORMATION

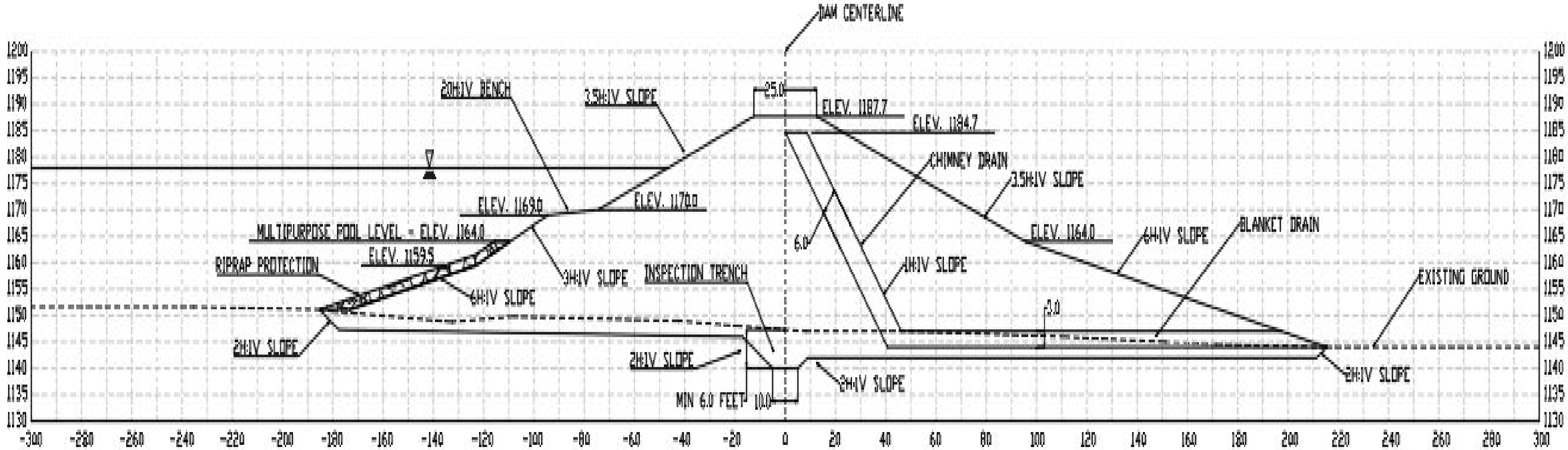


Figure 65: Typical cross-section of DS19 optimized plan

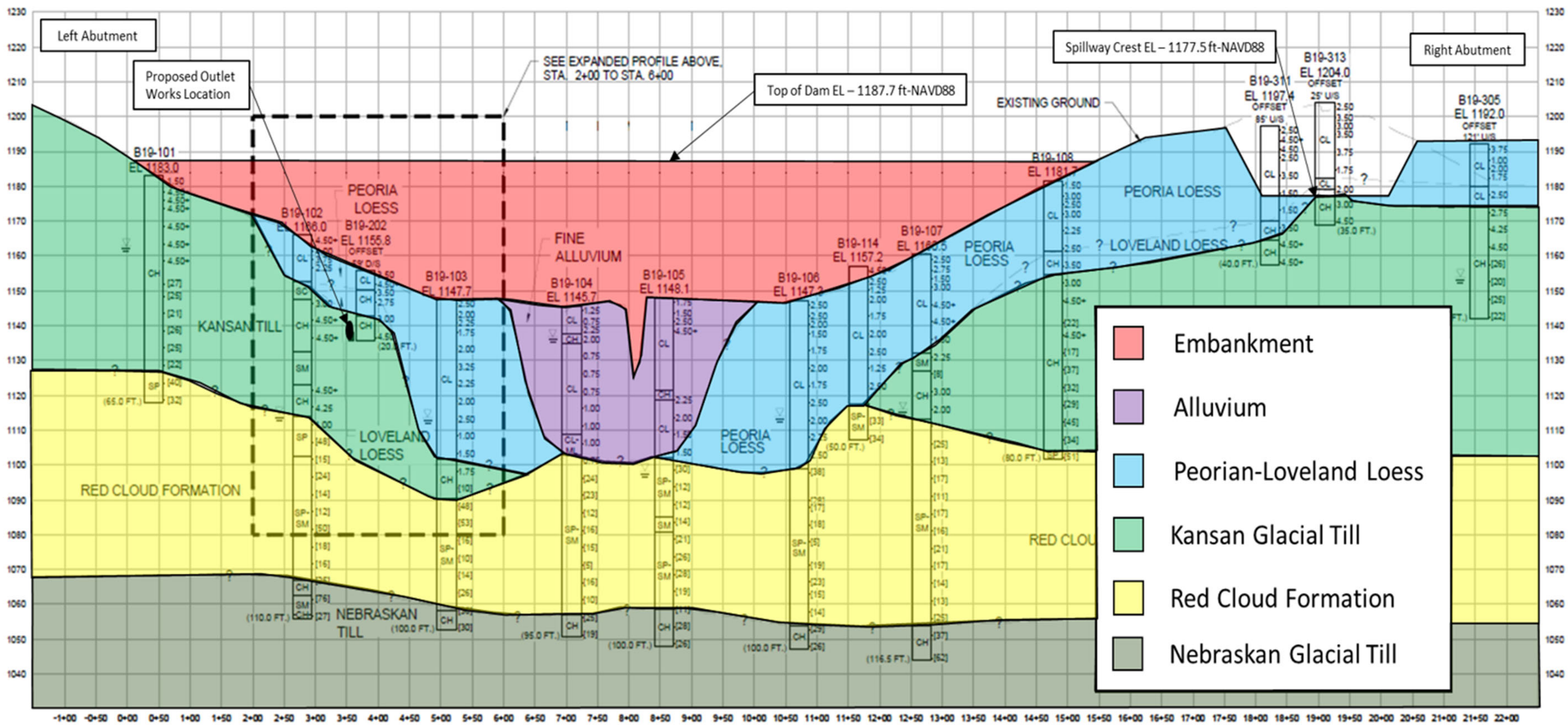


Figure 66: Geological cross-section of DS19 optimized plan

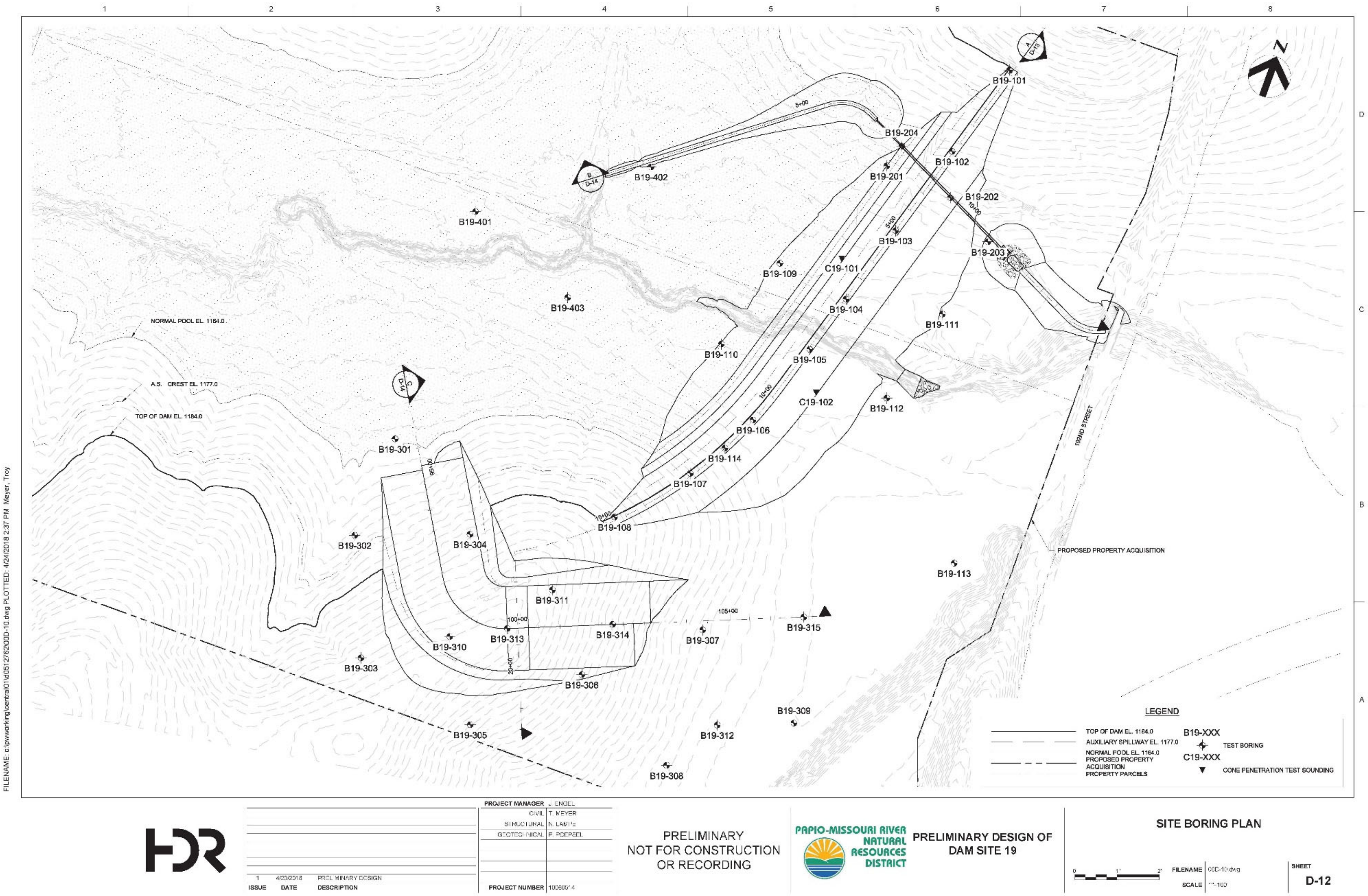


Figure 67: HDR report site boring plan

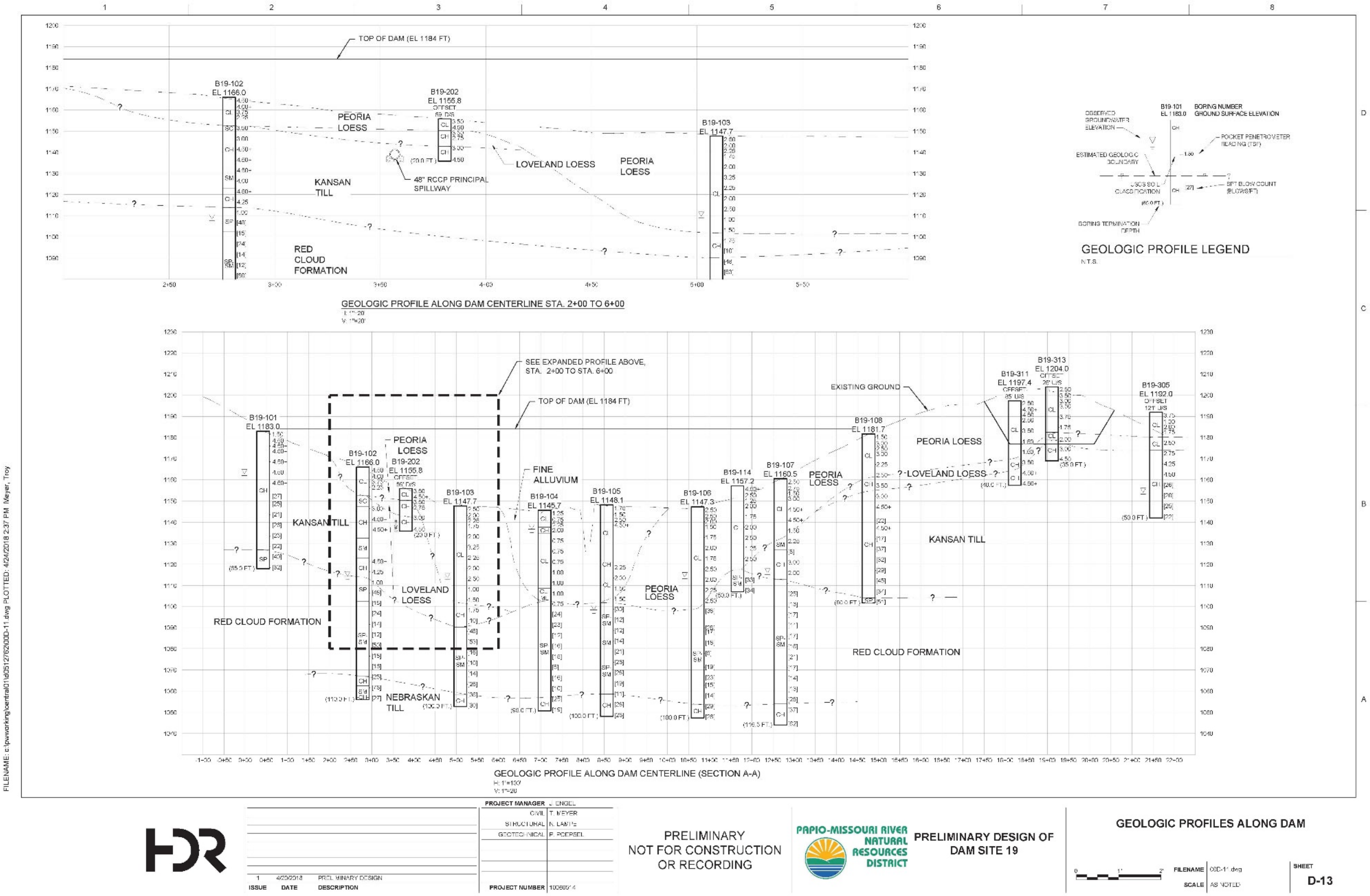


Figure 68: HDR report geological profile along dam embankment

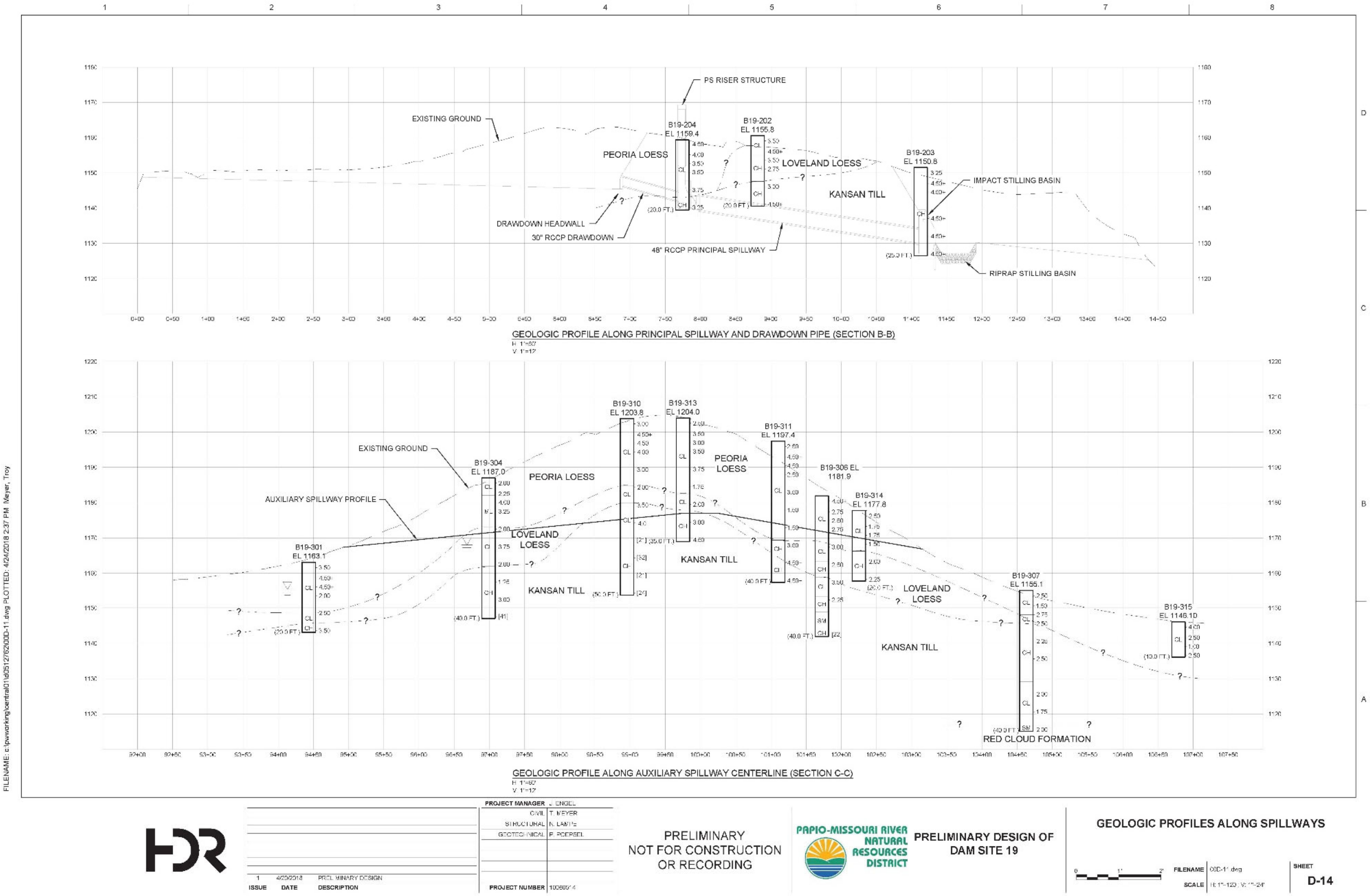


Figure 69: HDR report geological profile along spillway

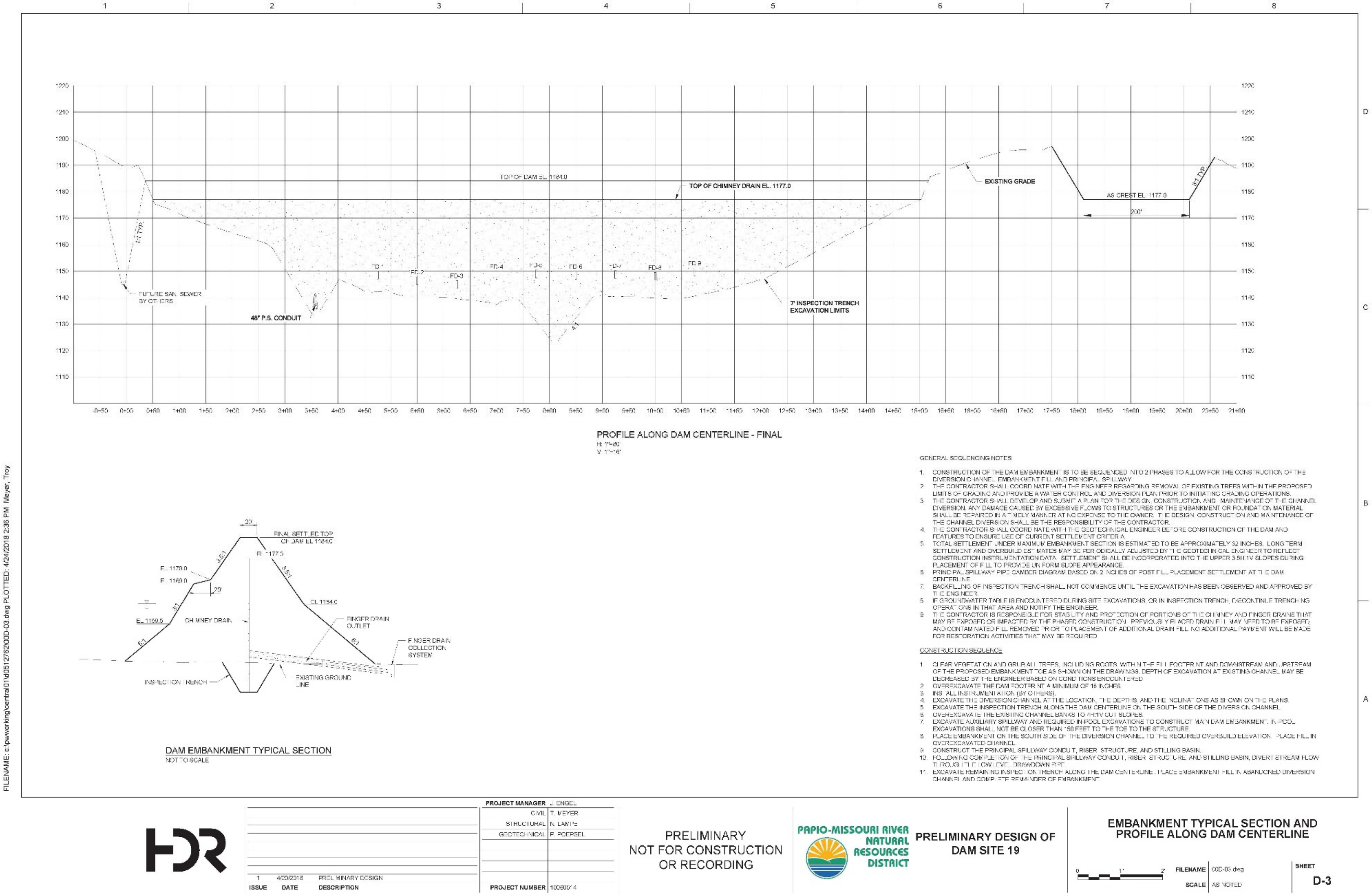


Figure 70: HDR report typical dam embankment section

8.2. DS10 BACKGROUND INFORMATION

Table 1
PERTINENT DATA
SITE 10

Drainage area	4.9 sq. mi.
Lands	
Reservoir construction and operation	478 acres
Exclusive recreation	-
Total project	478 acres
Embankment	
Crest length	1400 feet
Crest elevation	1201 feet, m.s.l.
Height	46 feet
Type of fill	Rolled earth
Volume of fill	300,000 cu. yds.
Freeboard	3 feet
Spillway	
Type	Earth cut
Location	Right abutment
Crest elevation	1193 feet, m.s.l.
Bottom width	200 feet
Length	1200 feet
Side slopes	1 on 3
Outlet works	
Inlet type	Branched Oak
Elev. of multipurpose pool inlet ports	1175 feet, m.s.l.
Elev. of high level inlet	1180 feet, m.s.l.
Conduit type	Reinforced concrete
Conduit diameter	3.0 feet
Conduit length	585 feet
Stilling basin type	SAF

Reservoir

Type of Storage	Storage Volume (acre-feet)	Elevation (feet m.s.l.)	Surface Area (acres)
Valley floor		1145	
Sediment		1175	
Multipurpose	1,200	1175	125
Standard Project Flood	2,250	1188	225
Surcharge	2,750	1198	325
Total Storage	6,200		

PD-10

Figure 71: Pertinent Data for DS10 from General Design Memorandum No. MPC-10

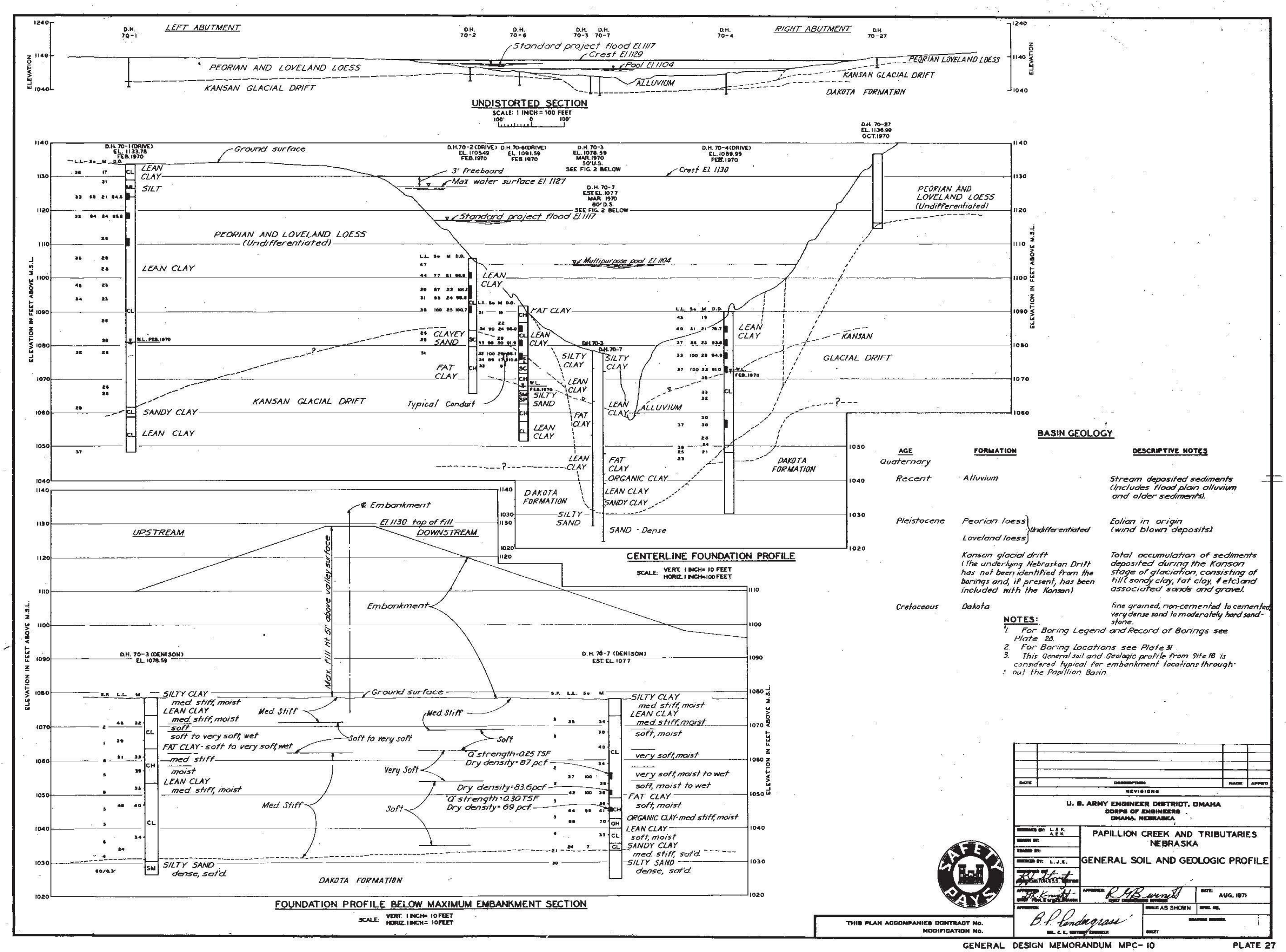


Figure 72: General soil and geological profile from General Design Memorandum No. MPC-10

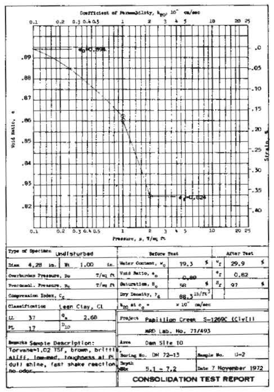


FIGURE 1

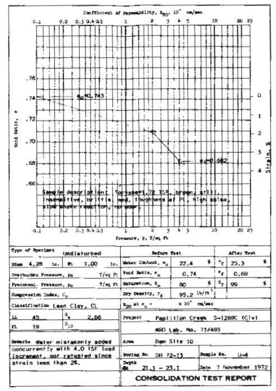


FIGURE 2

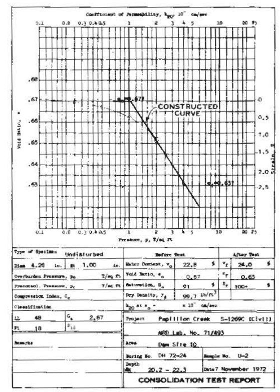


FIGURE 3

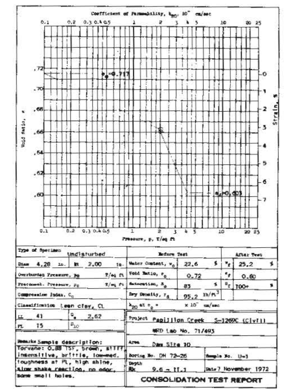
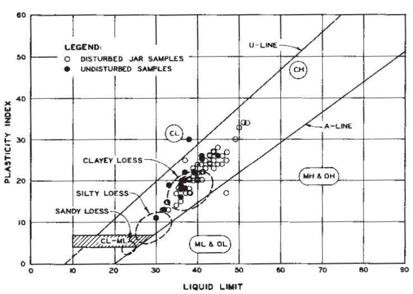


FIGURE 4



ATTERBERG LIMITS SUMMARY

FIGURE 5

Hole No.	Sample No.	Soil Class.	Atterberg Limits LL	PL	PI	Natural Dry Density γ_d PCF	Moist. Cont. Wo %	SP. Grav. Gs	Sat. So %	Void Ratios Init. eo	Final Before Sat. e_1	Final After Sat. e_2	Field Layer Thick. H Ft.	Settlement Before Sat. ΔS_1 Ft.	After Sat. ΔS_2 Ft.	Total S_t Ft.
71-7	U-1	Lean Clay (CL)	37	18	19	101.0	22.7	2.67	93	0.65						
	U-2	"	36	17	19	92.8	26.0	2.67	87	0.80						
	U-3	"	49	16	33	104.6	23.4	2.66	100	0.59						
72-4	U-1	"	37	19	18	99.7	23.4	2.64	94.8	0.65						
	U-2	"	41	15	26	102.7	20.3	2.66	88	0.62						
72-5	U-3	"	36	20	16	97.1	26.8	2.67	100	0.72						
72-9	U-1	"	37	17	20	90.6	21.6	-	69	0.84						
	U-2	"	39	17	22	96.6	22.0	2.71	79.3	0.75	0.73	0.73	24	0.27	0	0.27
72-10	U-1	"	37	18	19	88.7	17.8	-	53	0.91						
	U-3	"	36	17	19	88.4	22.3	2.59	70	0.83						
72-11	U-2	"	45	17	28	97.3	21.0	-	77	0.74						
72-12	U-1	"	37	15	22	89.2	12.5	2.69	38	0.88	0.83	0.78	15	0.4	0.4	0.8
	U-2	"	38	17	21	95.9	17.4	-	61	0.78						
72-13	U-2	"	37	17	20	88.3	19.3	2.68	58	0.89	0.86	0.86	20	0.14	0	0.14
	U-3	"	37	17	20	89.6	21.6	-	68	0.85						
	U-4	"	45	19	26	95.2	22.4	2.66	80	0.74						
72-24	U-1	"	38	16	22	95.1	24.1	-	85	0.76						
	U-2	"	48	18	30	99.7	22.8	2.67	91	0.72	0.67					
72-26	U-1	"	41	15	26	95.2	22.6	2.62	83	0.72	0.66	0.66	22	0.73	0	0.73
74-1	U-1	"	36	18	18	98.3	22.1	2.71	83	0.72						
74-2	U-1	"	37	17	20	88.1	21.0	2.66	63	0.88						

SUMMARY OF LOESS PROPERTIES

FIGURE 6

NOTES:

1. For location of samples within drill holes, see Plates 20 through 25.
2. For location of samples within foundation, see Plate 26.
3. For location of drill holes, see Plate 17.
4. Limits for types of loess in Figure 5 from USBR Engineering Monograph, "Petrographic and Engineering Properties of Loess", by Gibbs and Holland.

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

DATE	DESCRIPTION	MADE	APPR'D

REVISIONS

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

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DRAWN BY:
CHECKED BY: M.A.
SUBMITTED BY: Art Daum — CHIEF ENG. & S.S. SECTION
RECOMMENDED: E.R. Kavanic — CHIEF F. & M. BRANCH

PAPILLION CREEK AND TRIBUTARIES LAKES, NEBRASKA
SITE 10
LABORATORY TEST DATA
CONSOLIDATION TESTS AND SUMMARY
OF SOIL PROPERTIES
LOESS

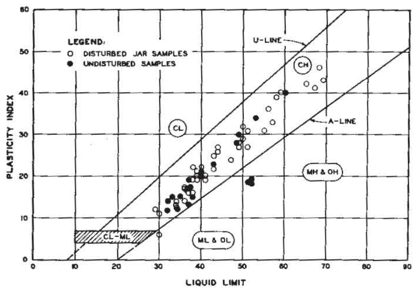
APPROVED: R.M. Burnett — CHIEF ENGINEERING DIVISION
DATE: MAY 1975
APPROVED: Russell A. Glenn — COL. C.E., DISTRICT ENGINEER
SCALE: AS SHOWN
SPEC. NO. DACA 45
DRAWING NUMBER
SHEET

SAFETY PAYS

THIS PLAN ACCOMPANIES CONTRACT NO. DACA 45 7 CO MODIFICATION No.

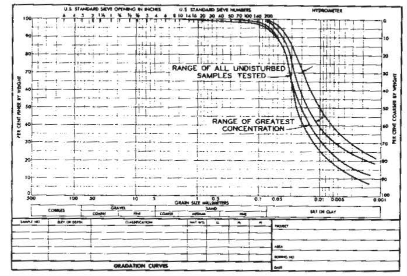
SPECIFIC DESIGN MEMORANDUM NO. MPC-33 PLATE 33

Figure 73: Site 10 Laboratory Test Data of Loess Material from Specific Design Memorandum No. MPC-33



ATTERBERG LIMITS SUMMARY

FIGURE 1



MECHANICAL ANALYSIS SUMMARY

FIGURE 2

DEPTH (FT)	NATURAL WATER CONTENT (%)											
	0-5	5-10	10-15	15-20	20-25	25-30	30-35	35-40	40-45	45-50	50-55	55-60
0-5			1	1	2	2	2	2				
5-10				1	4	4	2	3				
10-15						5	2	2		1		1
15-20					1	5	6			2		
20-25					1	3	3	4	1	1		
25-30						3	3	2				
30-35						2	3	3				
35-40					1	1	1	2				
40-45					3	2	1	3				
45-50					3		4					
50-55					2	2	1					
55-60					3	1	2					

STATISTICAL FREQUENCY SUMMARY
NATURAL WATER CONTENT VS. DEPTH

FIGURE 3

THIS DRAWING HAS BEEN REDUCED TO THREE-EIGHTHS THE ORIGINAL SCALE.

DATE	DESCRIPTION	MADE	APPR'D
	REVISIONS		

U. S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
OMAHA, NEBRASKA

DESIGNED BY: M.A.
DRAWN BY:
CHECKED BY: M.A.
SUBMITTED BY: Art Naum
CHIEF ENG. & S.S. SECTION
RECOMMENDED: E.R. Kovanic
CHIEF F. & M. BRANCH
APPROVED: R. Burnett
CHIEF ENGINEERING DIVISION
APPROVED: Russell A. Glenn
COL. C.E., DISTRICT ENGINEER

PAPILLION CREEK AND TRIBUTARIES LAKES, NEBRASKA
SITE 10
LABORATORY TEST DATA
SUMMARY OF SOIL PROPERTIES
FOUNDATION ALLUVIUM

DATE: MAY 1975
SCALE: AS SHOWN
SPEC. NO. DACA 45
DRAWING NUMBER
SHEET

SAFETY PAYS

THIS PLAN ACCOMPANIES CONTRACT No. DACA 45 7 CO MODIFICATION No.

SPECIFIC DESIGN MEMORANDUM NO. MPC-33 PLATE 41

U. S. GOVERNMENT PRINTING OFFICE

Figure 74: Site 10 Laboratory Test Data of Foundation Alluvium Material from Specific Design Memorandum No. MPC-33

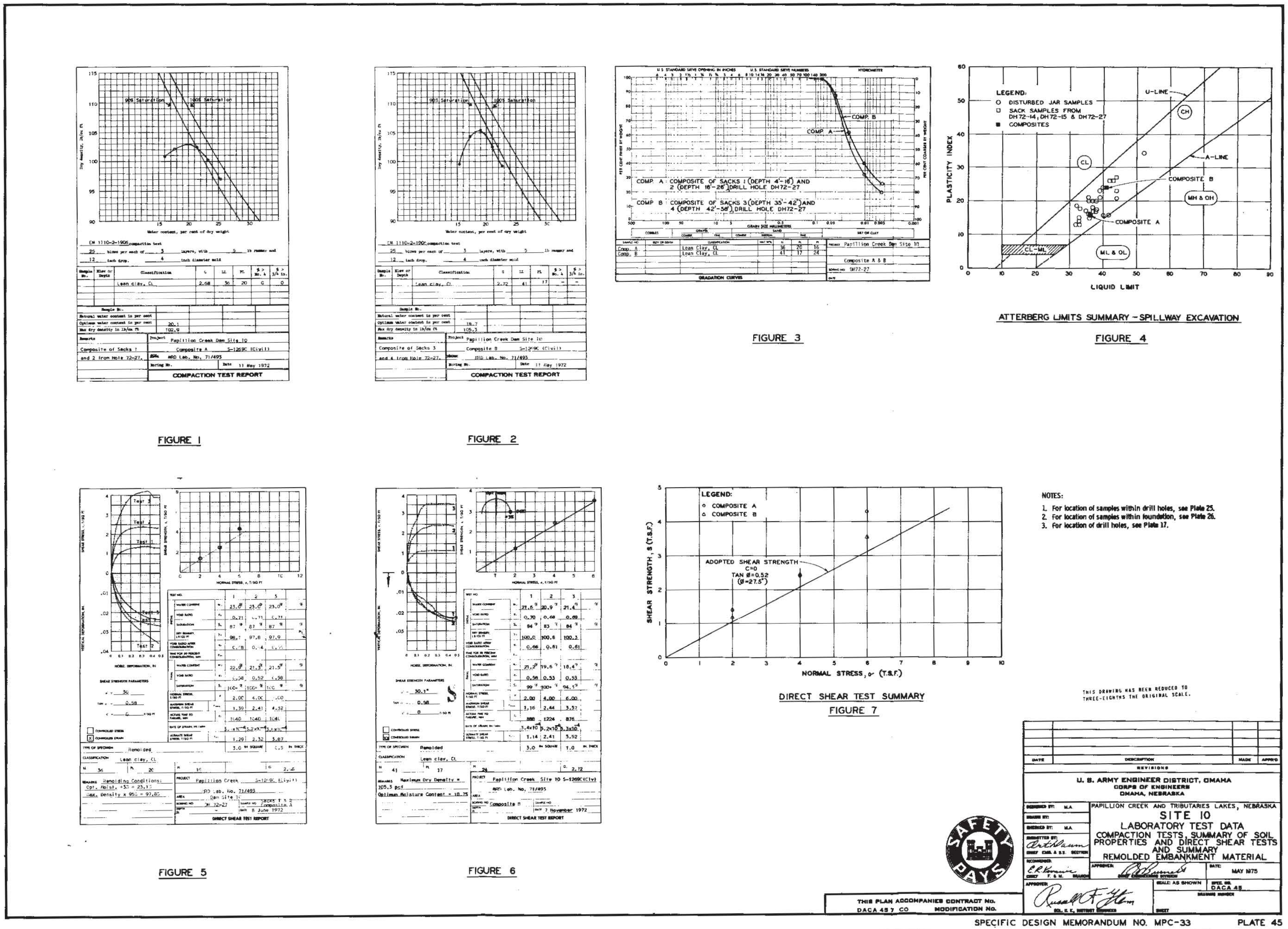


Figure 75: Site 10 Laboratory Test Data of Remolded Embankment Material from Specific Design Memorandum No. MPC-33

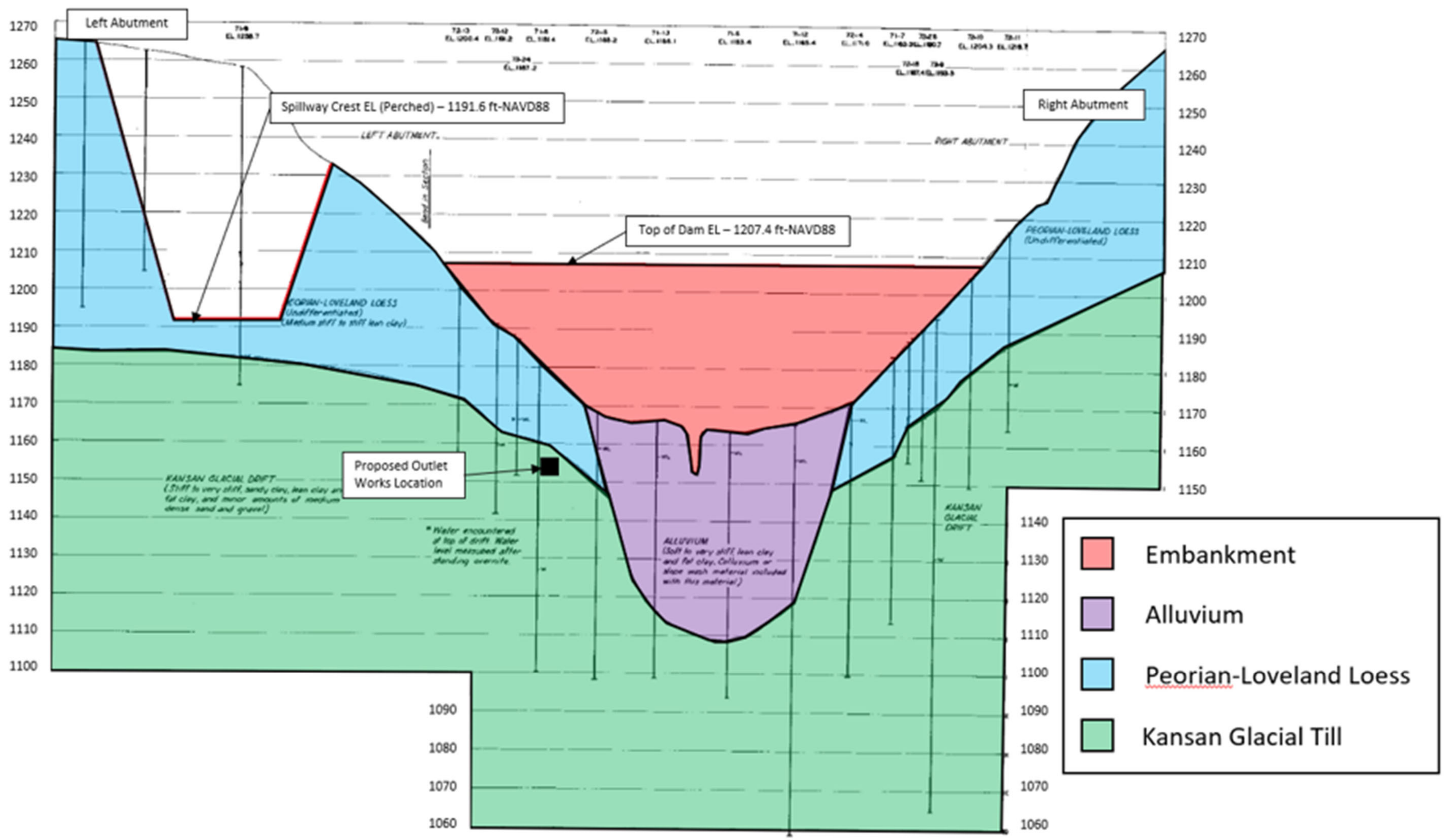


Figure 76: Geological cross-section of DS10 optimized plan

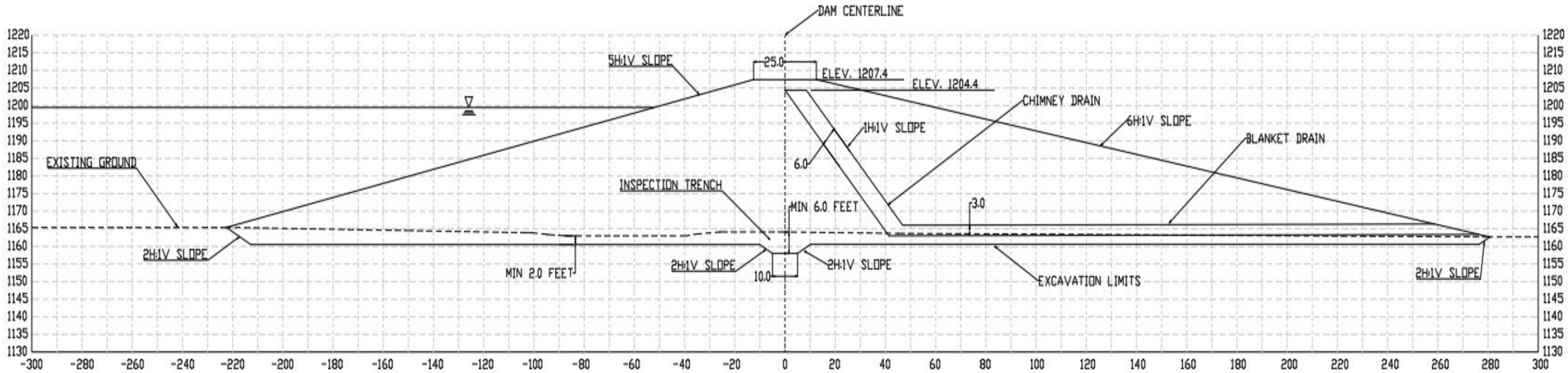


Figure 77: Typical cross-section of DS10 optimized plan

8.3. LITTLE PAPILLION LEVEE BACKGROUND INFORMATION

TABLE 4-1
DESIGN SHEAR STRENGTHS

MATERIAL	ZONE THICKNESS	UNCONSOLIDATED UNDRAINED (Q) STRENGTH TAN 0	COH (TSF)
Embankment			
Zone A (El. 1075.0-1013.0)	62'	0	0.80
Foundation			
Zone B (El. 1013.0-1005.0)	8'	0	0.675
Zone C (El. 1005.0-1001.0)	4'	0	0.4125
Zone D (El. 1001.0- 994.0)	7'	0	0.675
Zone E (El. 994.0- 984.0)	10'	0	0.675
Zone F (El. 984.0- 973.0)	11'	0	0.675

Figure 78: Native soil layering and parameters from MPC-53

5.4.4.1. Cohesive Soil

Saturated Soil Weight	120 pcf
Submerged Soil Weight	57.6 pcf
Effective Phi Angle	28.5°
At-Rest Pressure Coefficient	K = 0.508

5.4.4.2. Rock (Limestone)

Unit Weight	165 pcf
Effective Phi Angle	45°
Passive Pressure Coefficient	K = 5.83
Allowable Bearing Capacity	10 tsf
Modulus of Subgrade Reaction (MSR)	
Elev 965 to 963.5	MSR = 86,344 pci
Elev 963.5 and below	MSR = 138,204 pci

Figure 79: Native material properties from MPC-53

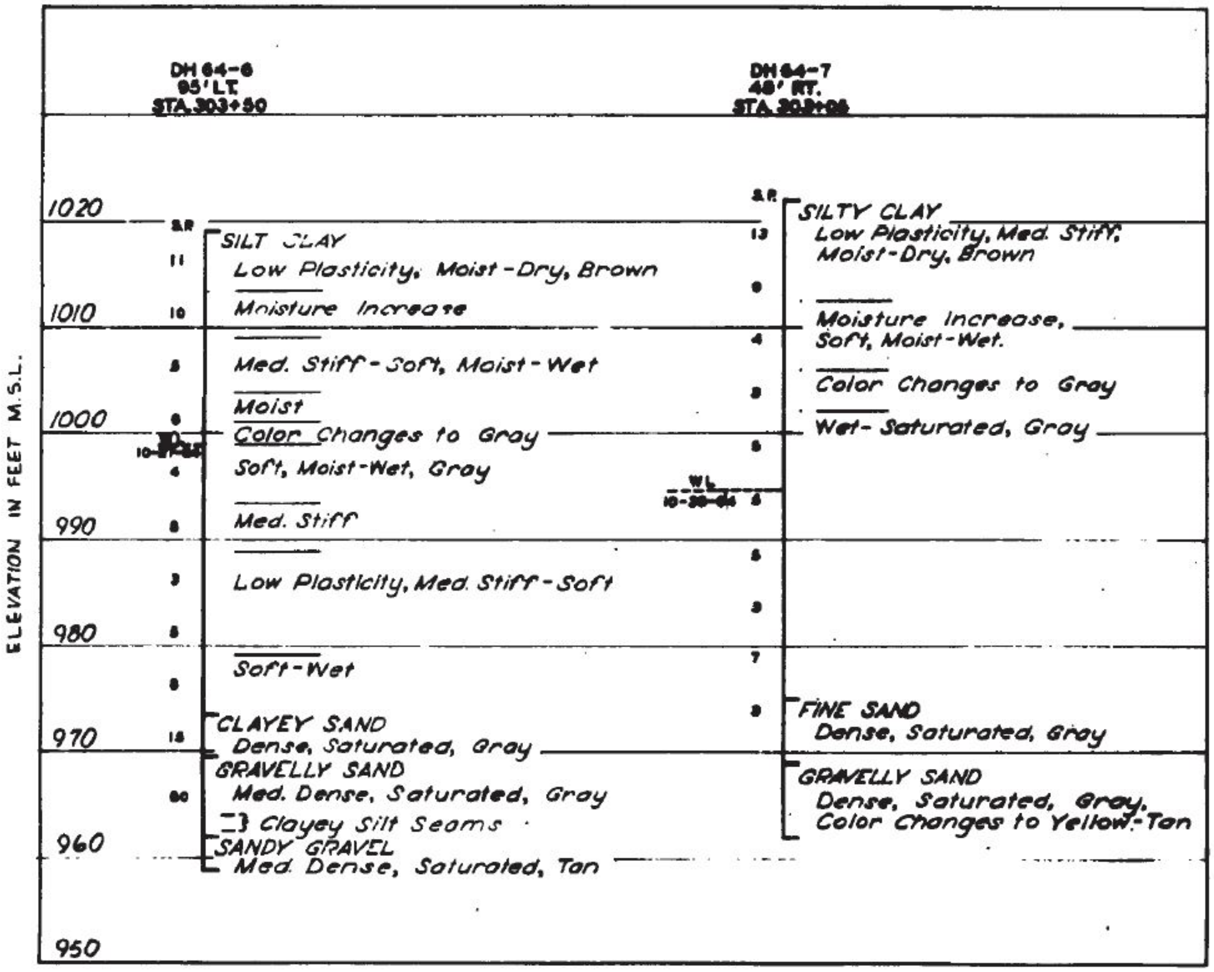


Figure 80: Boring logs from channel widening project – located downstream of Saddle Creek

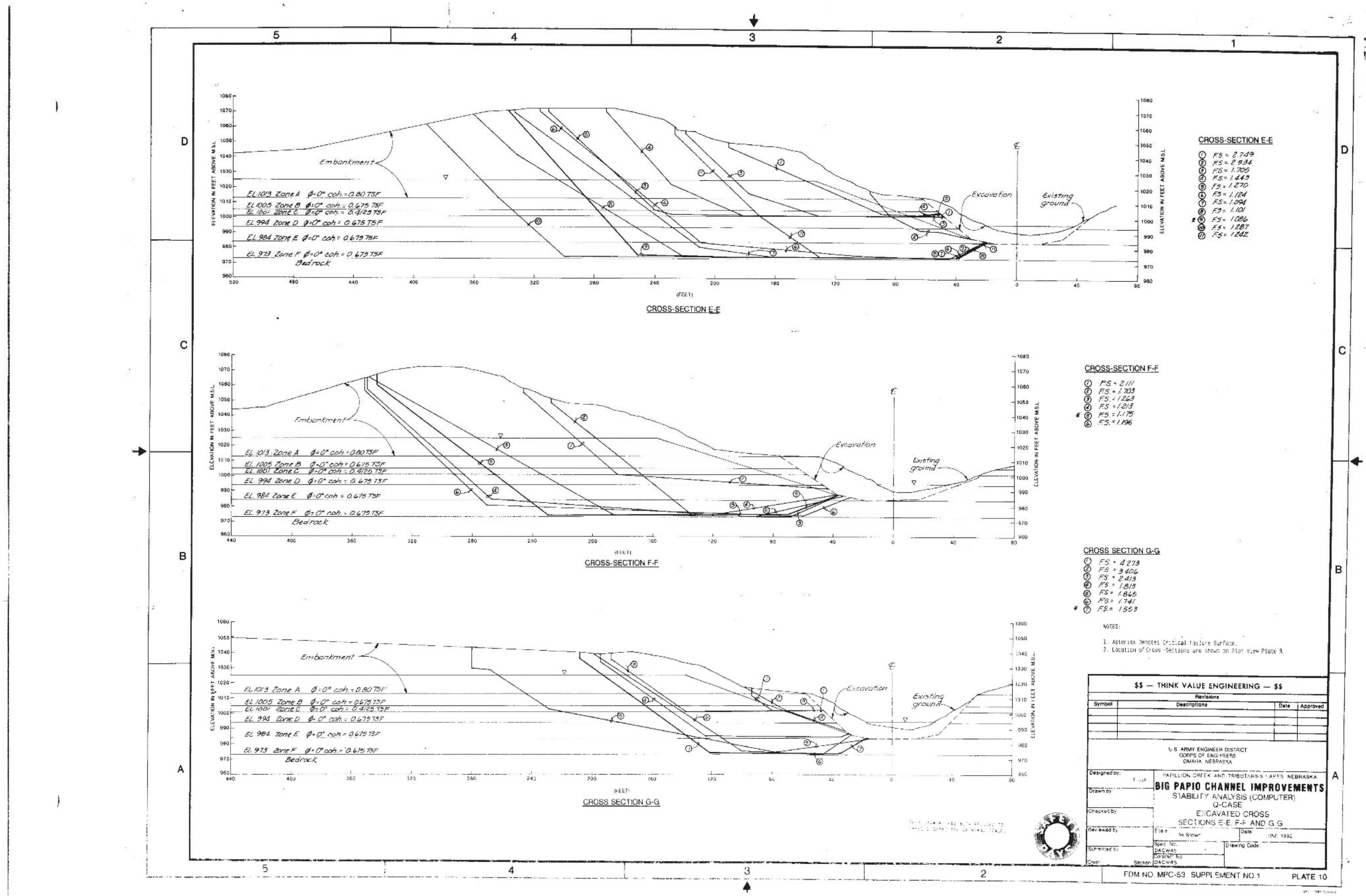


Figure 81: Native material layering from MPC-53

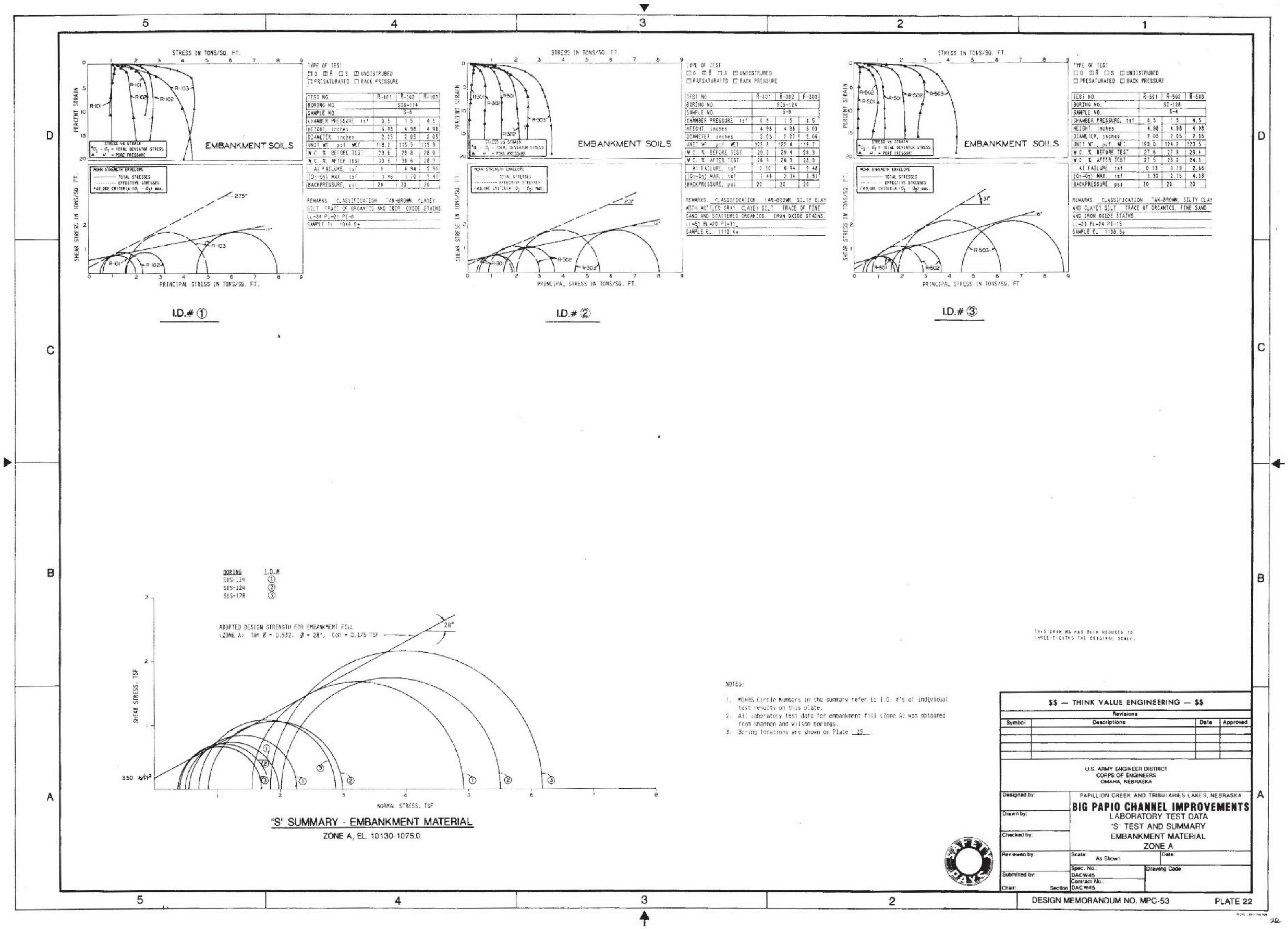


Figure 82: Laboratory test data – "S" tests for zone A from MPC-53

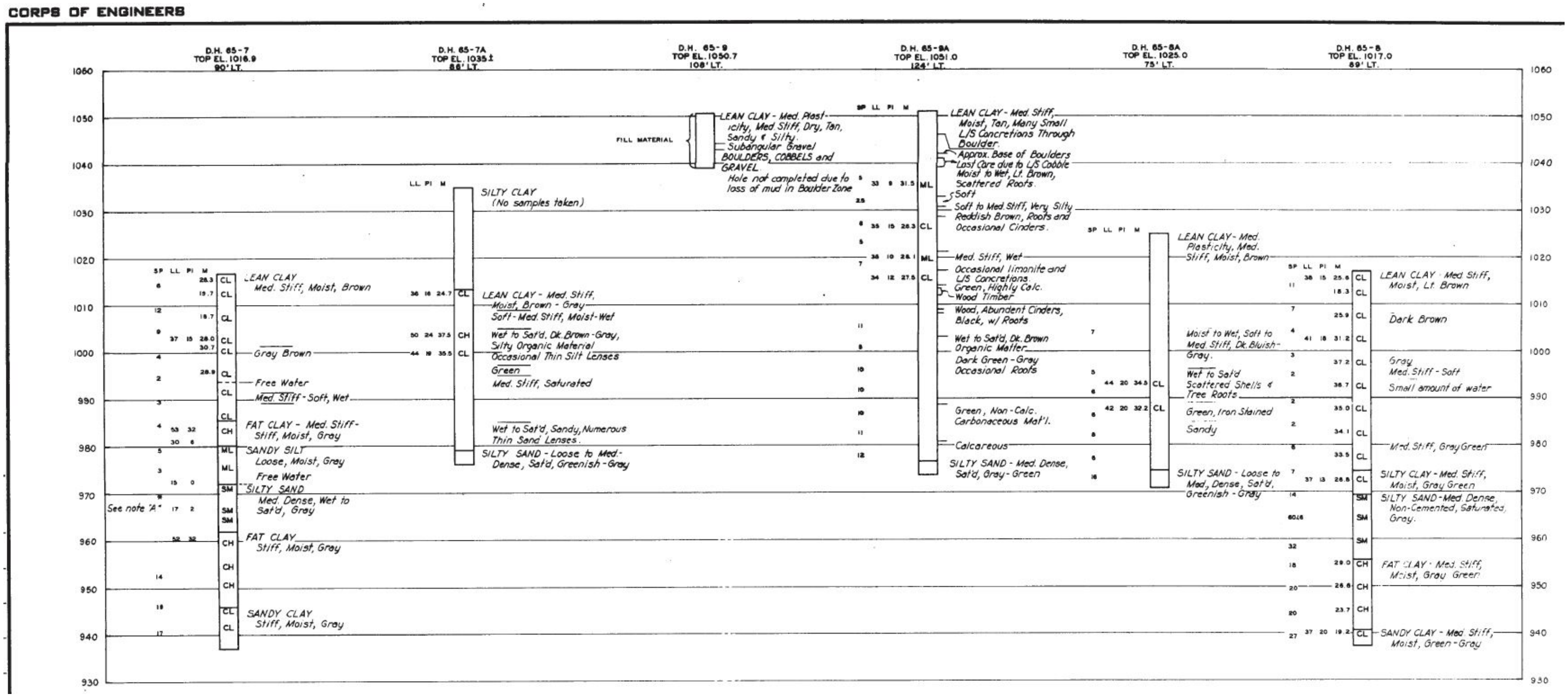


Figure 83: Boring logs from channel improvements at UPPR Bridge No. 828 – from O&M Plate 78 (included below)

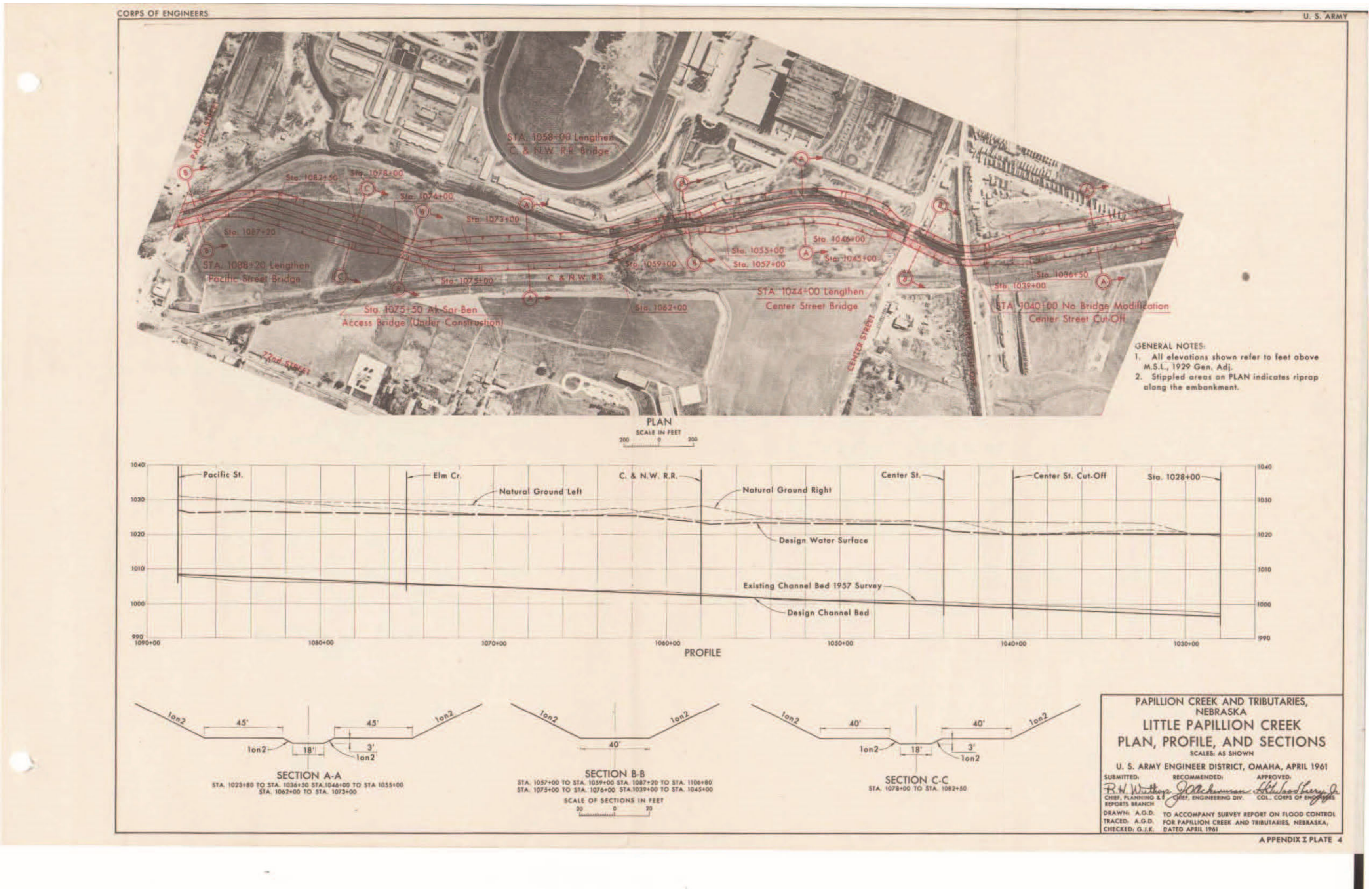


Figure 84: Little Papillion channel widening plan and profile from 1961 Survey Report – Appendix 1 Plate 4

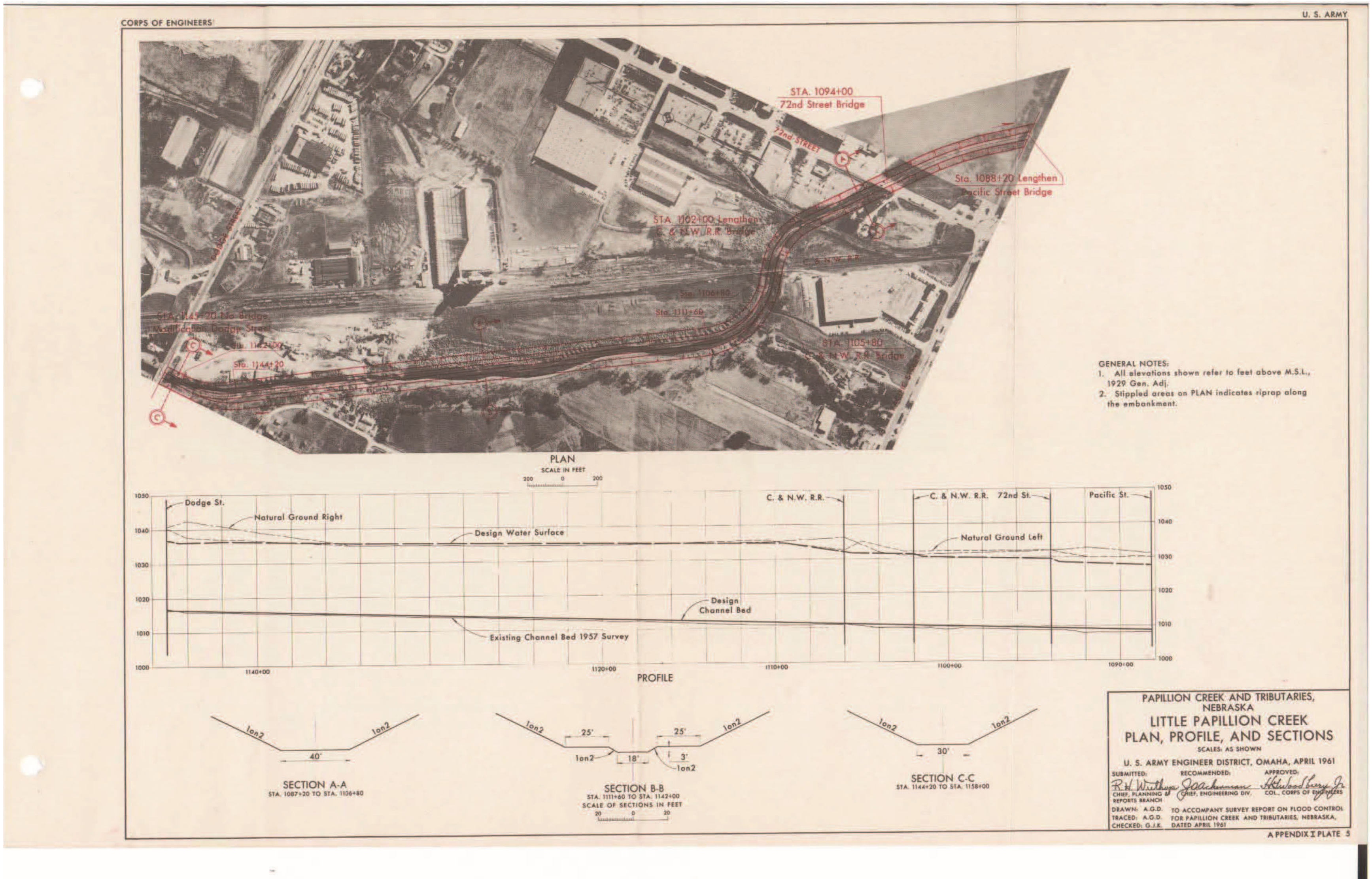


Figure 85: Little Papillion channel widening plan and profile from 1961 Survey Report – Appendix 1 Plate 5

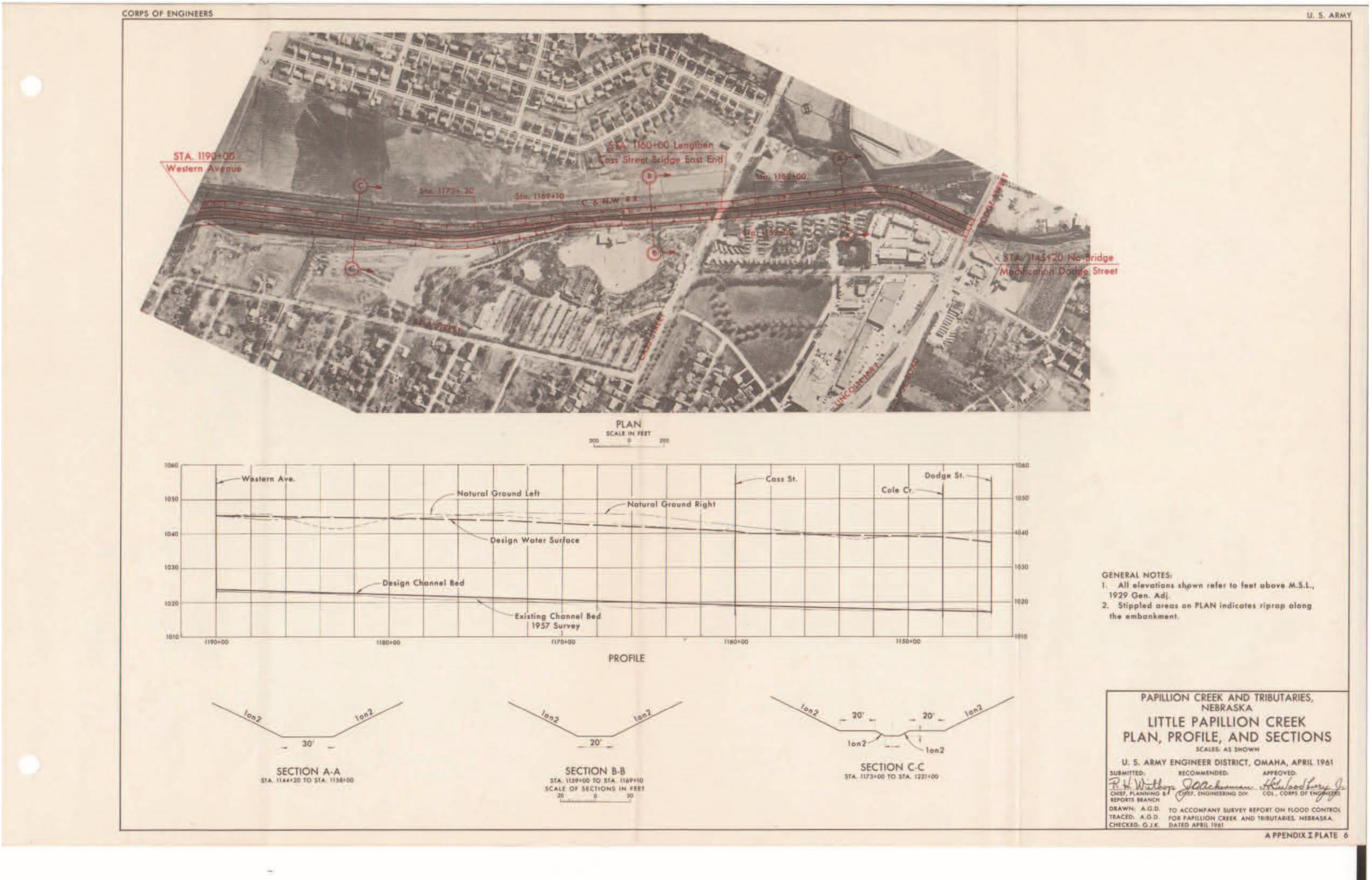


Figure 86: Little Papillion channel widening plan and profile from 1961 Survey Report – Appendix 1 Plate 6

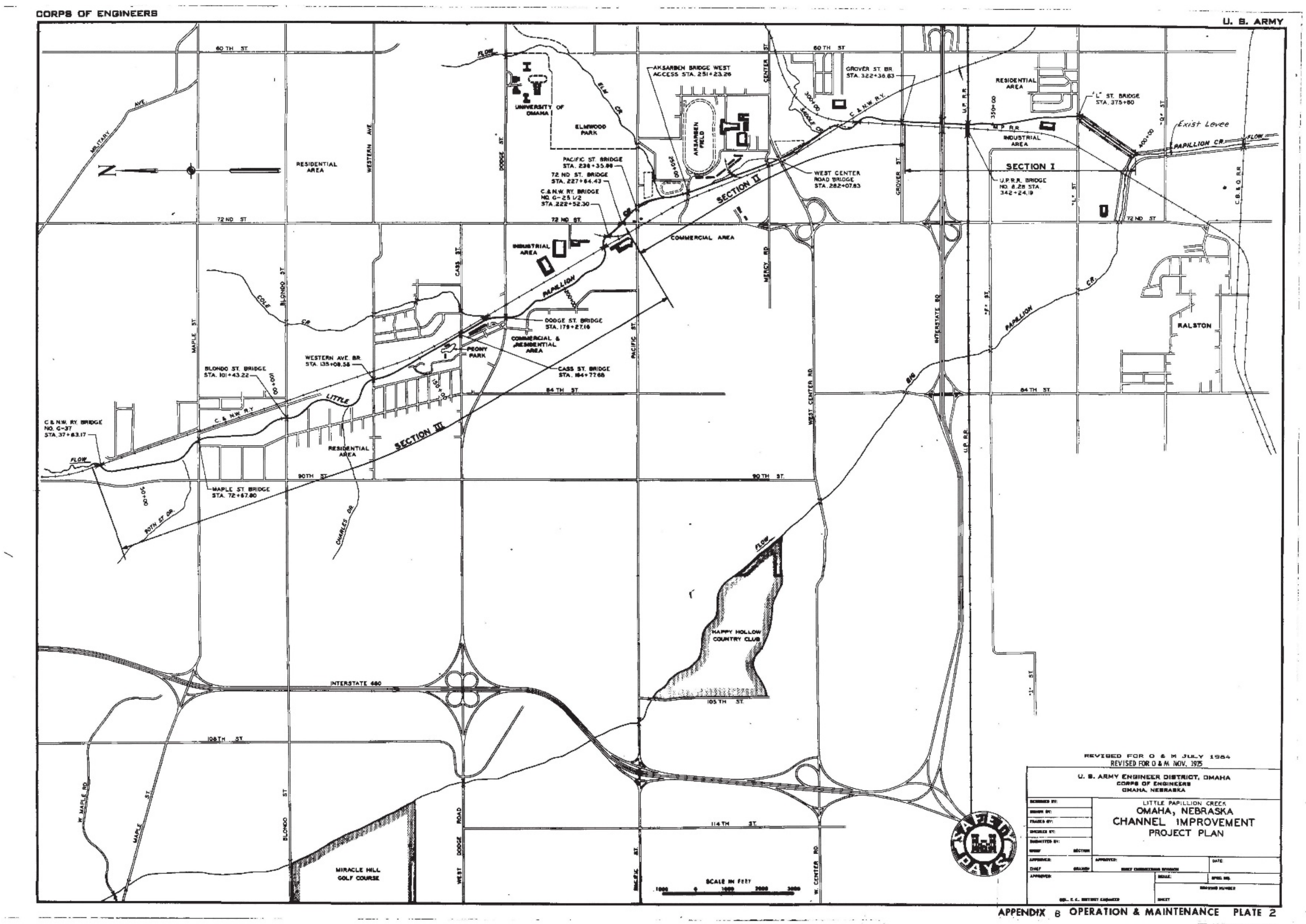


Figure 87: Little Papillion channel widening project areas from 1984 O&M manual – Appendix B Plate 2

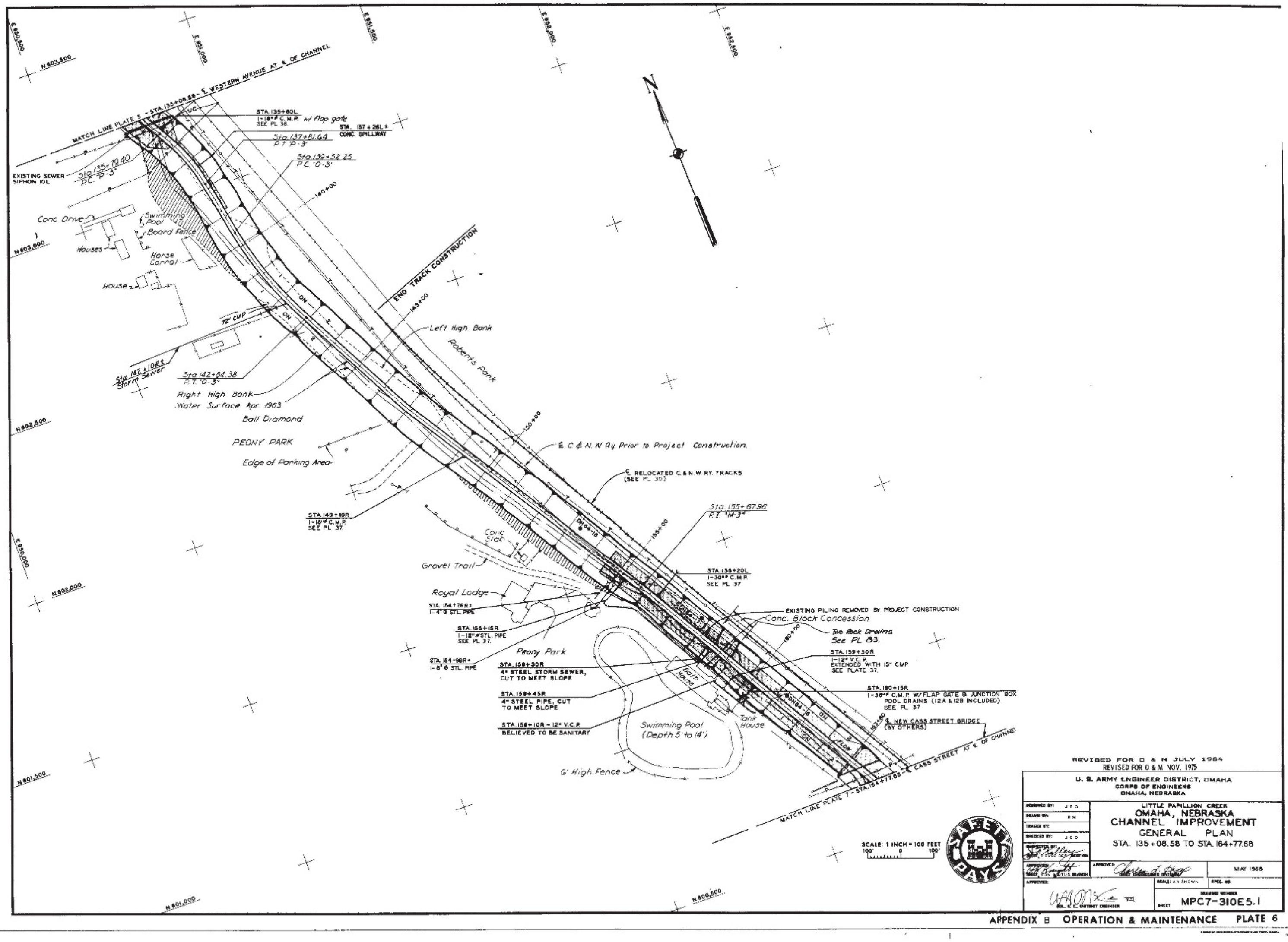


Figure 88: Little Papillion channel widening plan sheet from 1984 O&M manual – Appendix B Plate 6

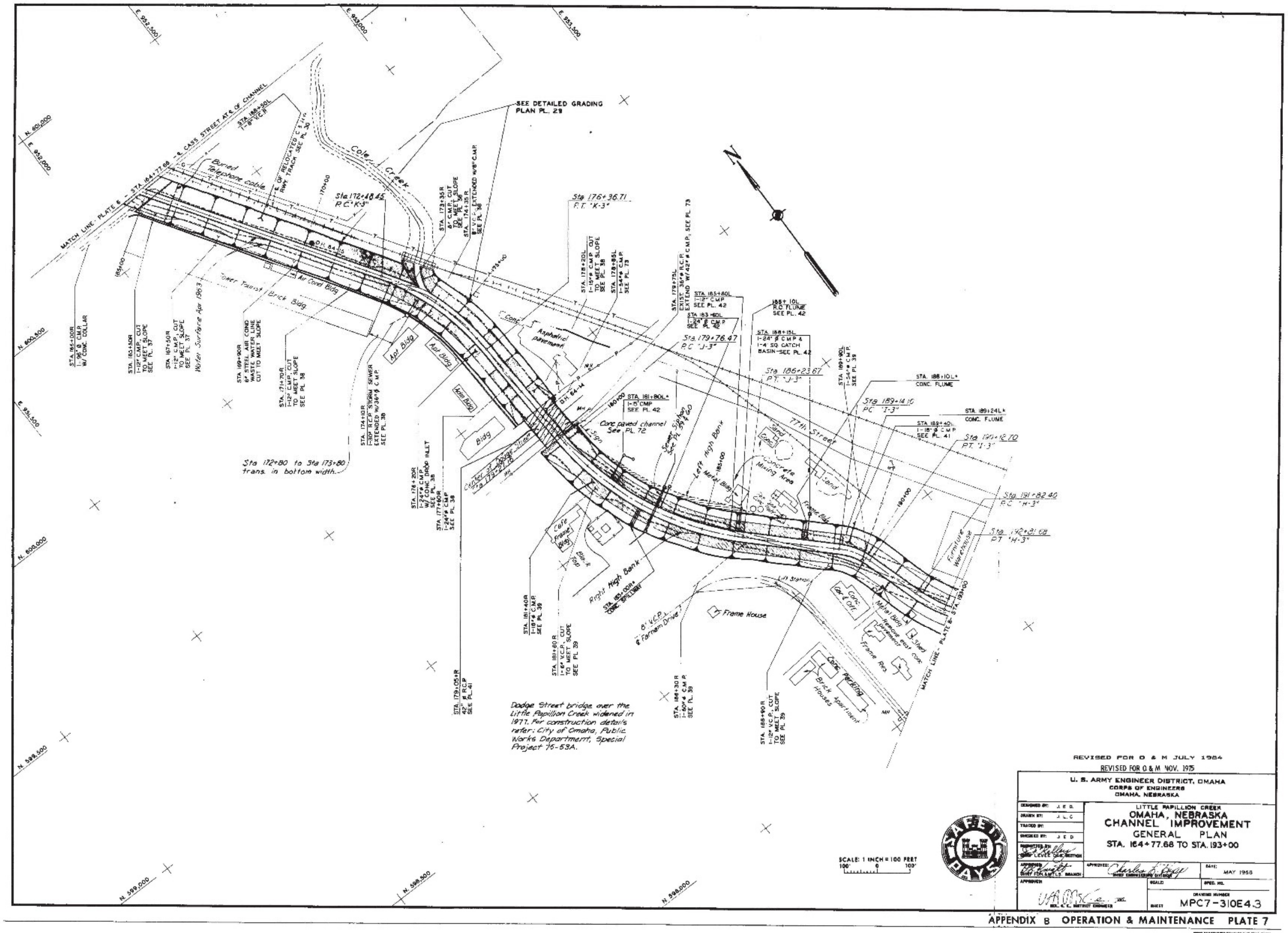


Figure 89: Little Papillion channel widening plan sheet from 1984 O&M manual – Appendix B Plate 7

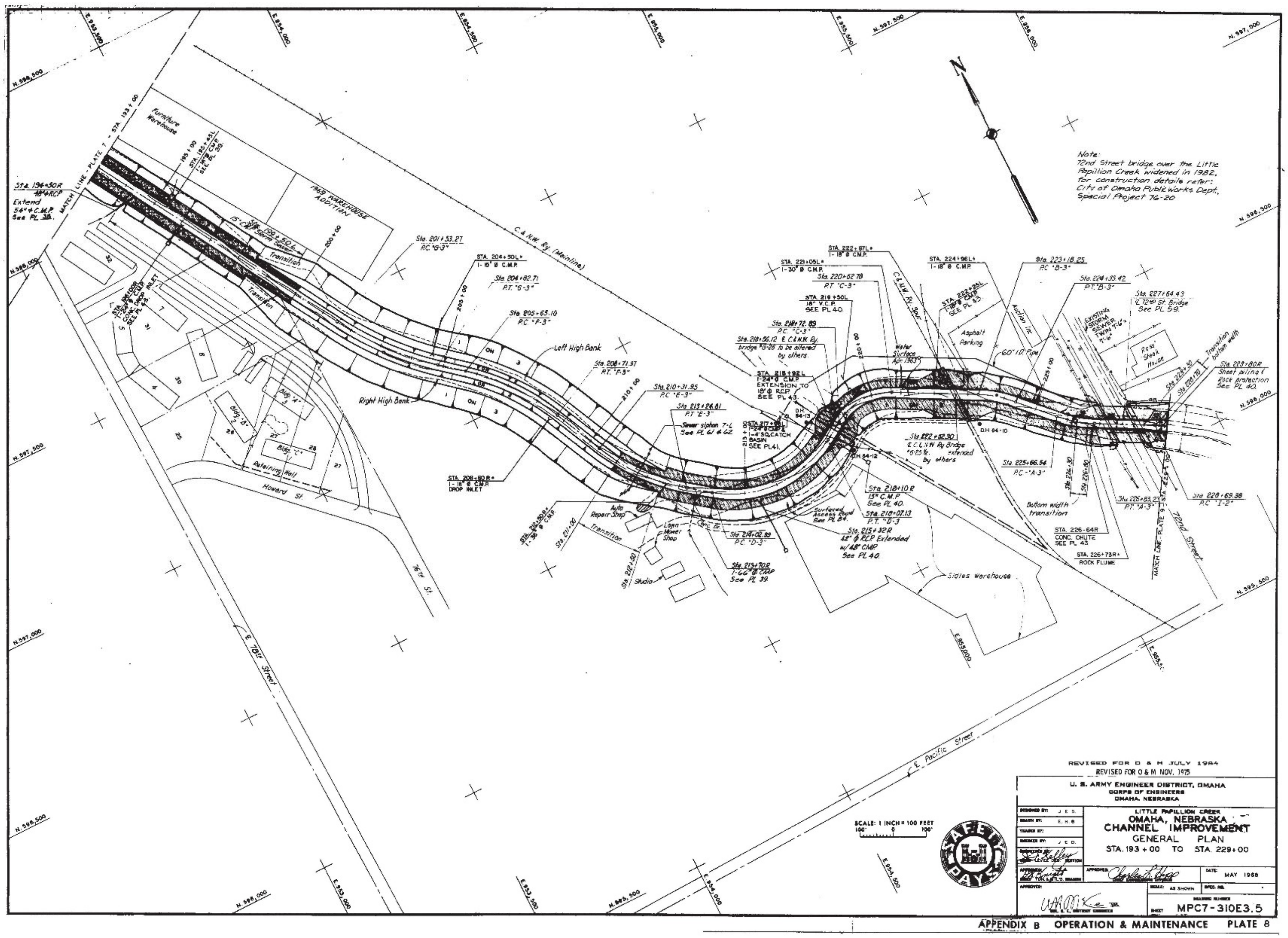


Figure 90: Little Papillion channel widening plan sheet from 1984 O&M manual – Appendix B Plate 8

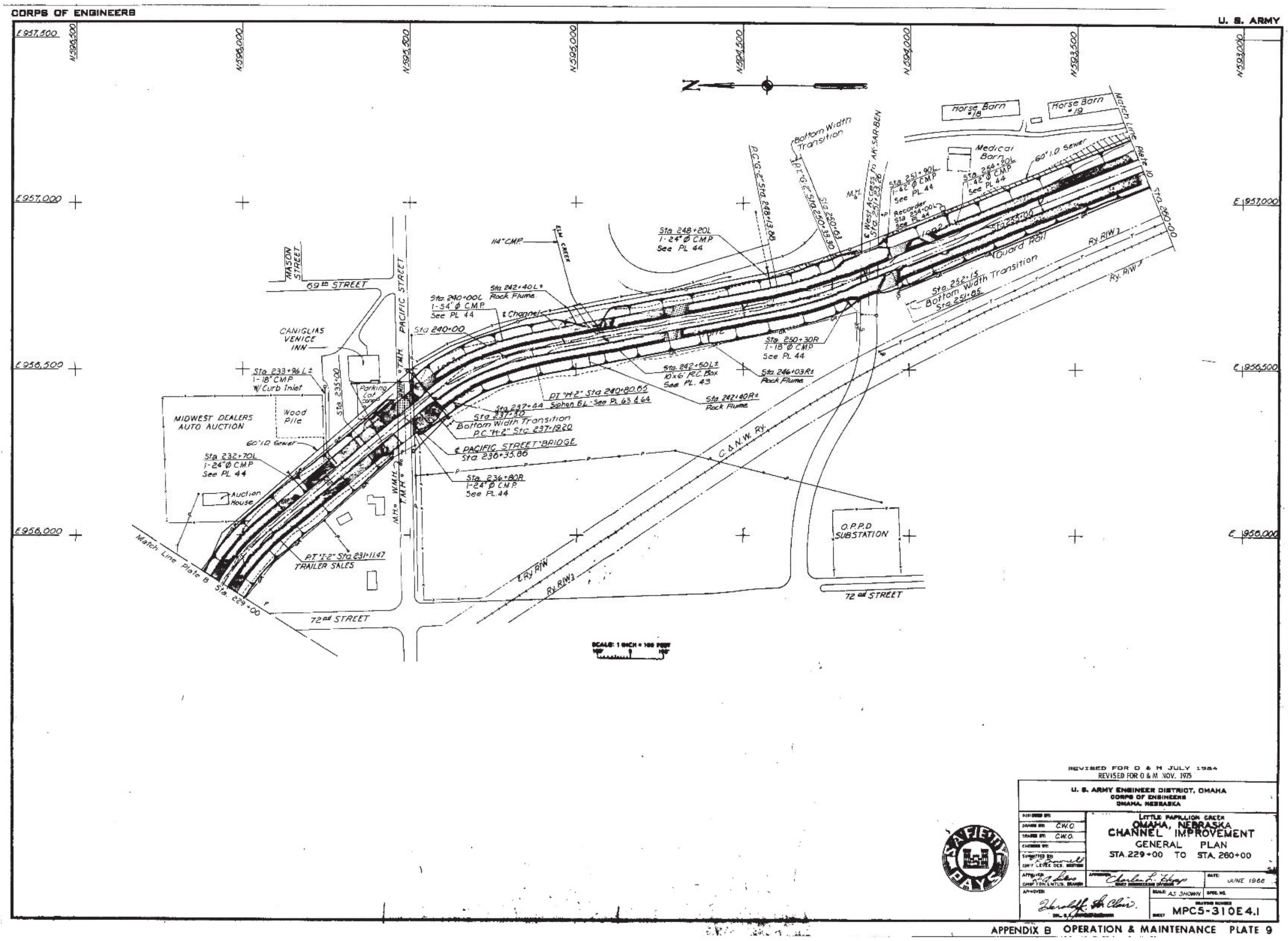


Figure 91: Little Papillion channel widening plan sheet from 1984 O&M manual – Appendix B Plate 9

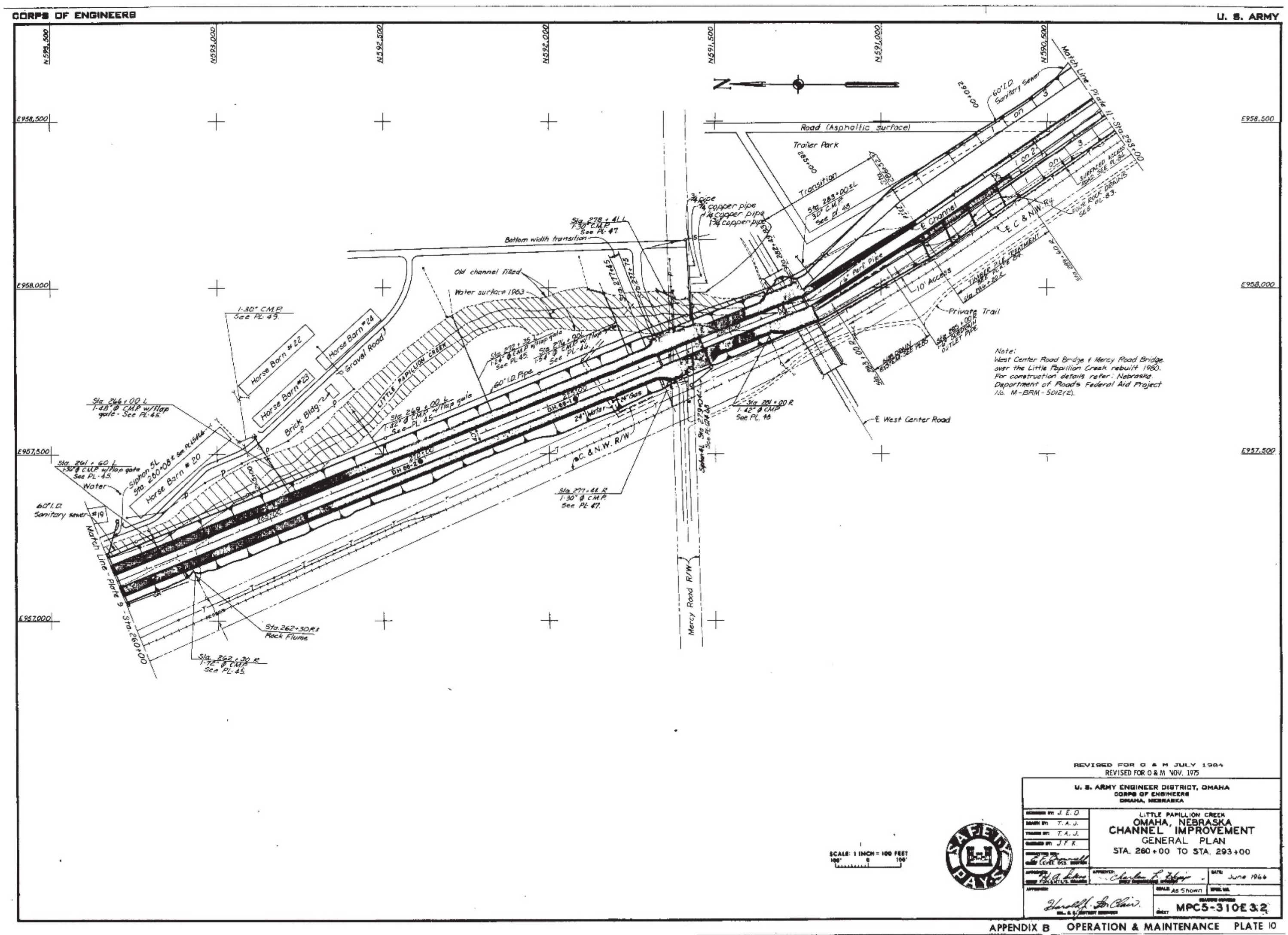


Figure 92: Little Papillion channel widening plan sheet from 1984 O&M manual – Appendix B Plate 10

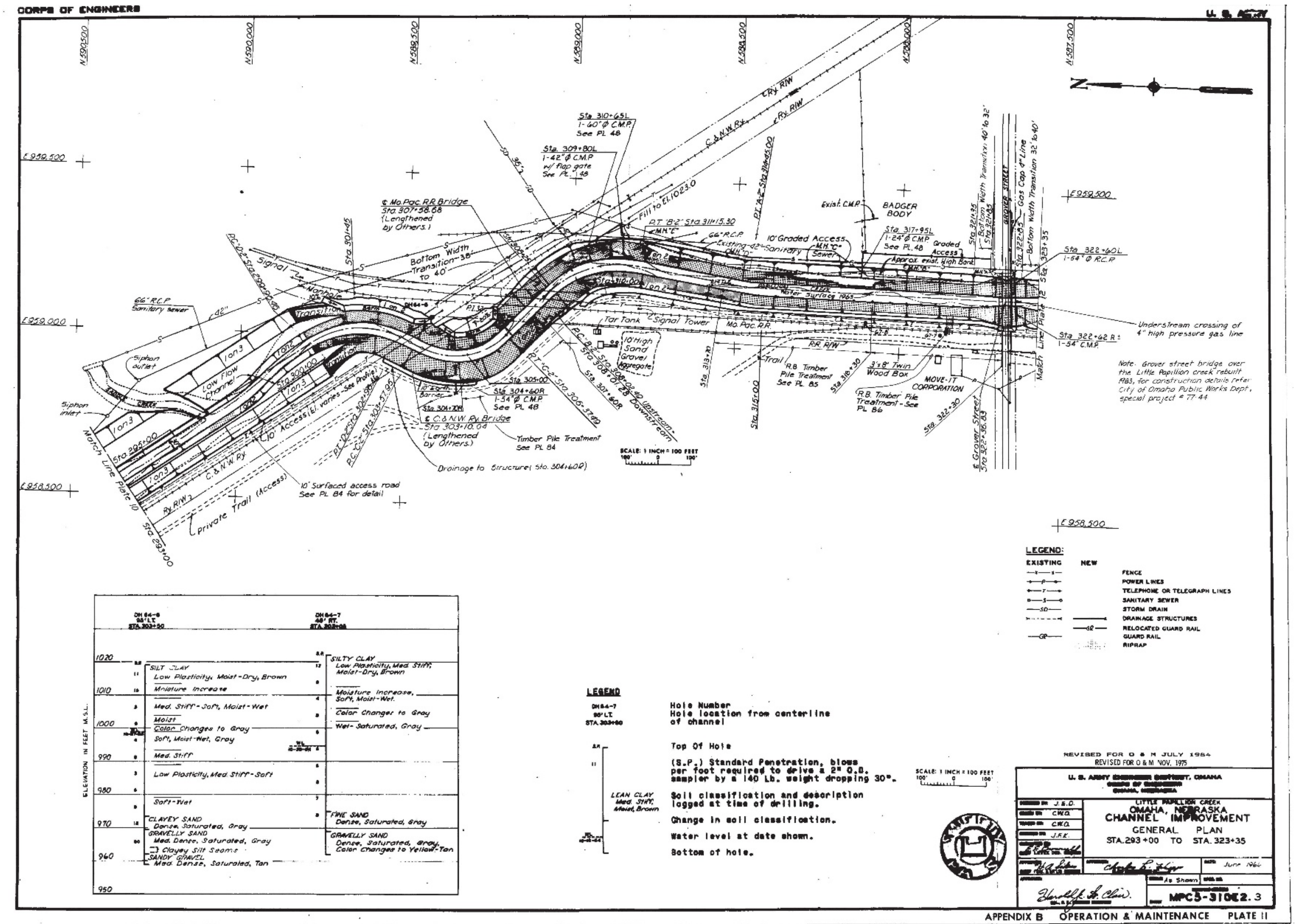


Figure 93: Little Papillion channel widening plan sheet from 1984 O&M manual – Appendix B Plate 11

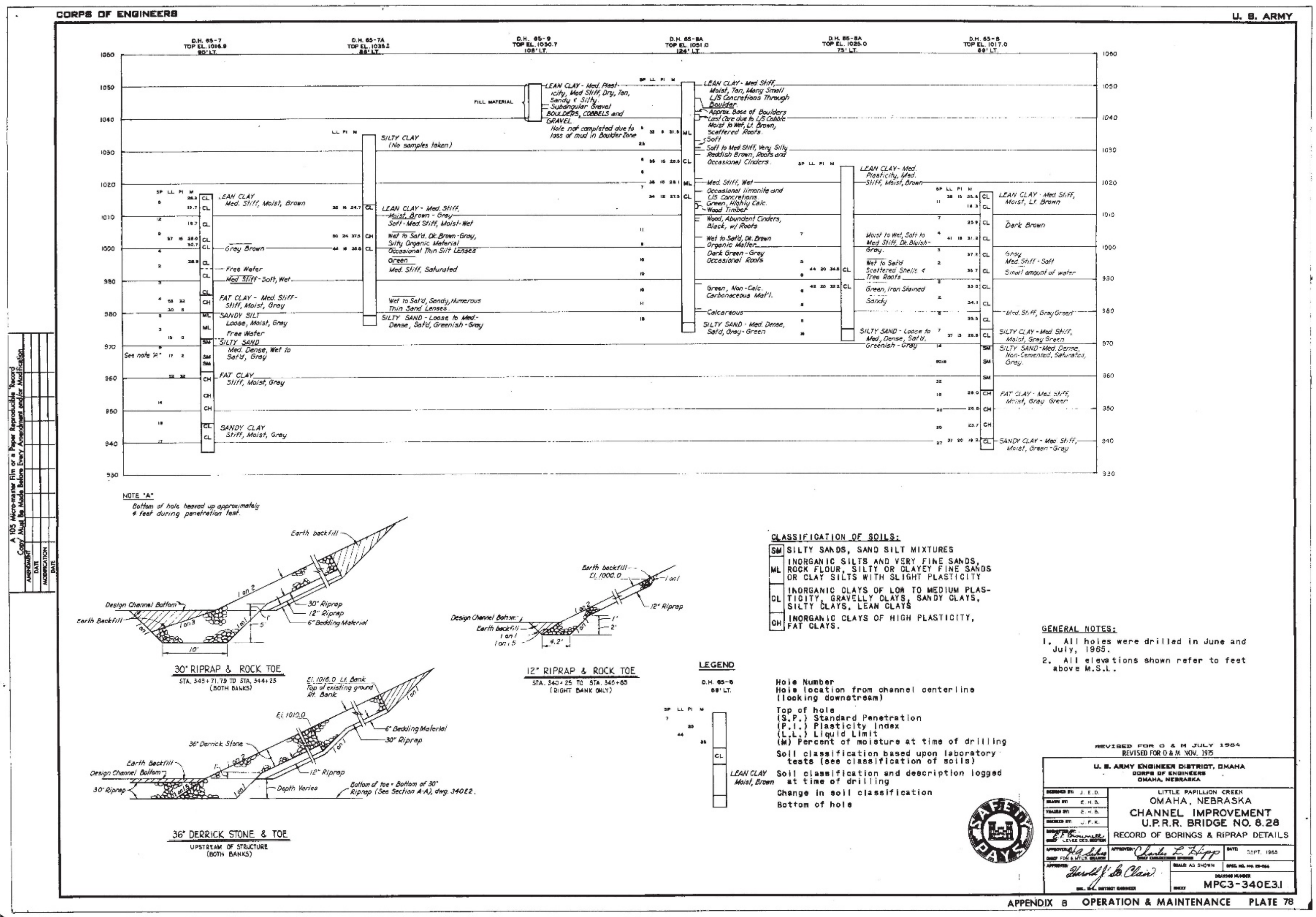


Figure 94: Little Papillion channel widening plan sheet from 1984 O&M manual – Appendix B Plate 78

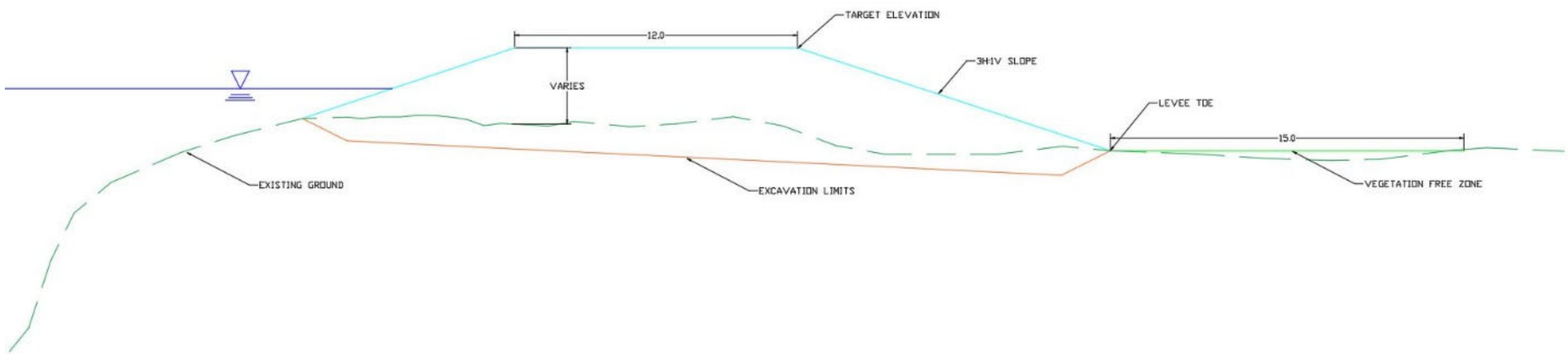


Figure 95: Typical full-height levee cross-section

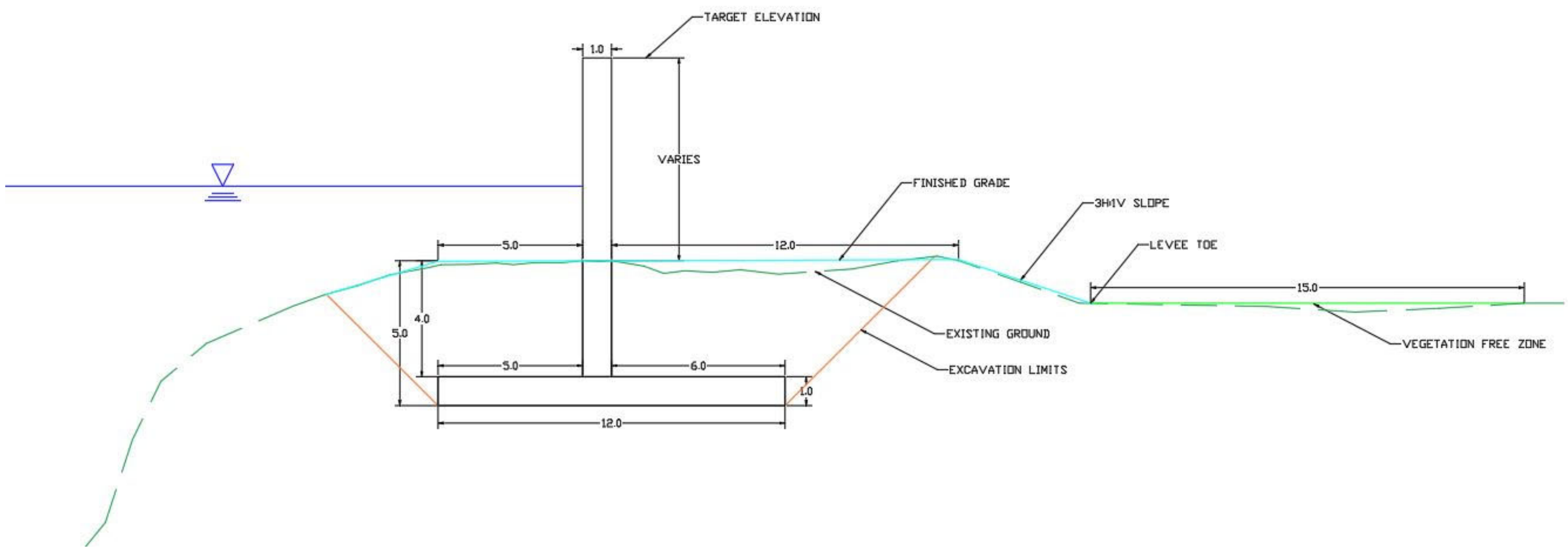


Figure 96: Typical full-height floodwall cross-section

8.4. GEOTECHNICAL ANALYSIS INFORMATION

Color	Name	Model	Sat Kx (ft/sec)	Ky'/Kx' Ratio
	Bedrock - Limestone	Saturated Only	2.62e-05	0.5
	Levee Fill - CL	Saturated Only	3.28e-07	0.25
	Zone A - CL	Saturated Only	1.31e-07	0.25
	Zone B - CL	Saturated Only	3.28e-08	0.25
	Zone C - CL	Saturated Only	1.64e-06	0.25
	Zone D - CL	Saturated Only	3.28e-07	0.25
	Zone E - CL	Saturated Only	1.64e-05	0.25
	Zone F - CL	Saturated Only	3.28e-06	0.25

Figure 97: New levee material properties for steady-state seepage model

Color	Name	Model	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Bedrock - Limestone	Bedrock (Impenetrable)			
	Levee Fill - CL	Mohr-Coulomb	120	150	24
	Zone A - CL	Mohr-Coulomb	123	360	27.9
	Zone B - CL	Mohr-Coulomb	121	0	29.7
	Zone C - CL	Mohr-Coulomb	118	0	28.4
	Zone D - CL	Mohr-Coulomb	121	0	29.7
	Zone E - CL	Mohr-Coulomb	121	0	29.7
	Zone F - CL	Mohr-Coulomb	121	0	29.7

Figure 98: New levee material properties for slope stability model

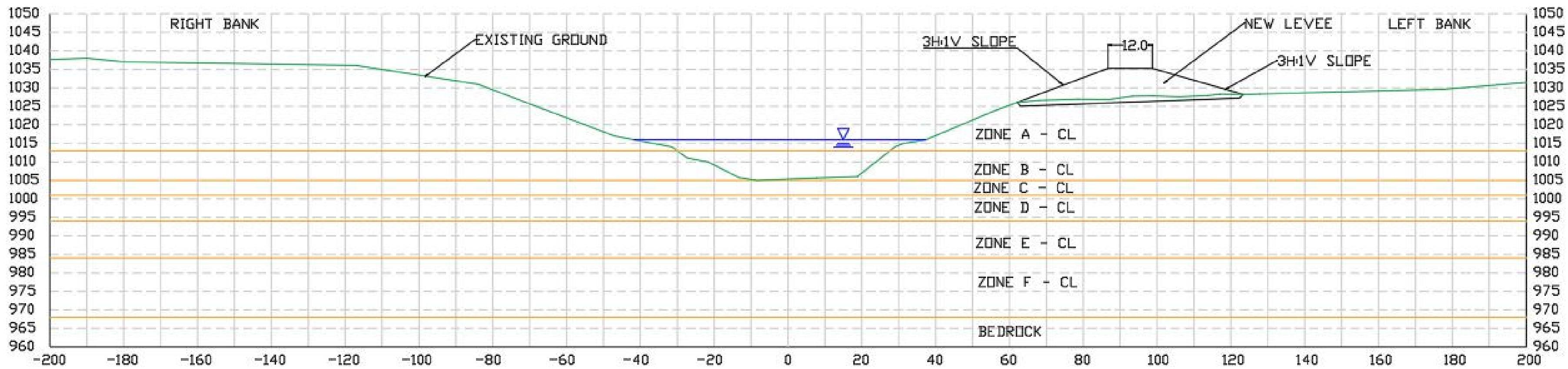


Figure 99: New levee cross-section and existing soil layering

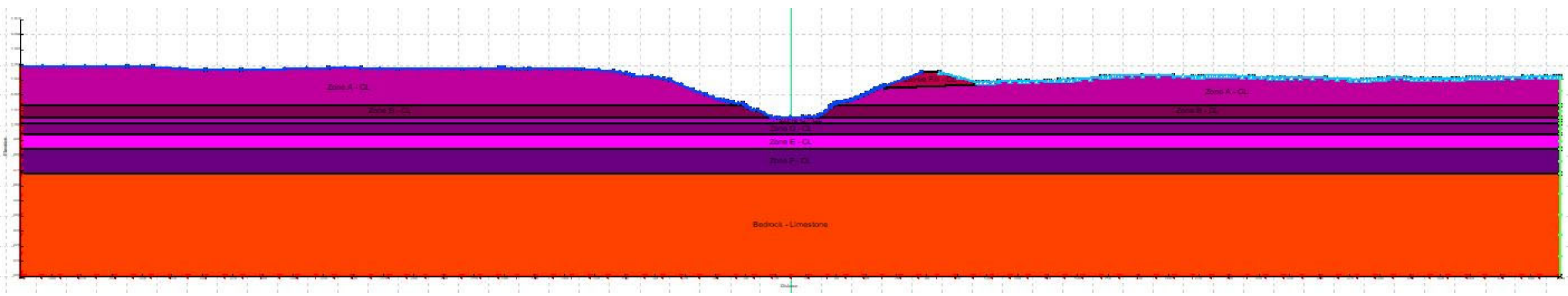


Figure 100: New levee steady-state seepage model setup and material layering

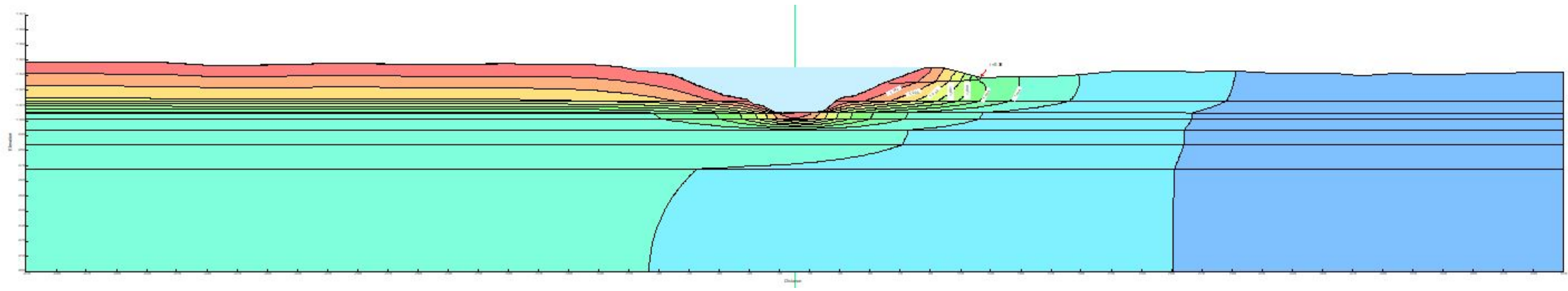


Figure 101: New levee steady-state seepage model results

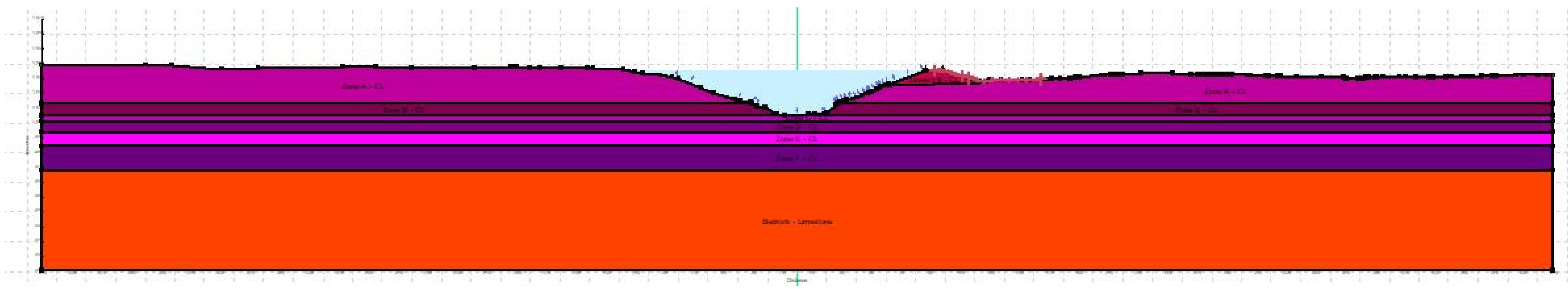


Figure 102: New levee slope stability model setup and material layering

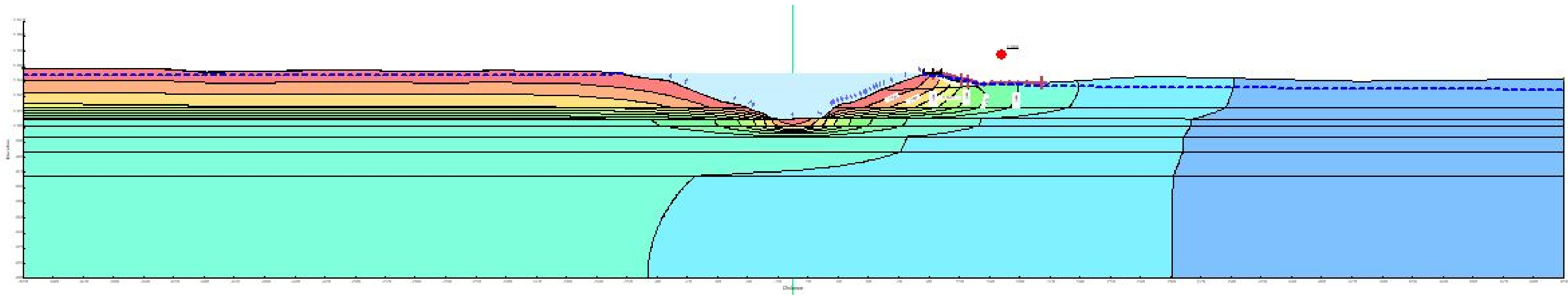


Figure 103: New levee slope stability model results

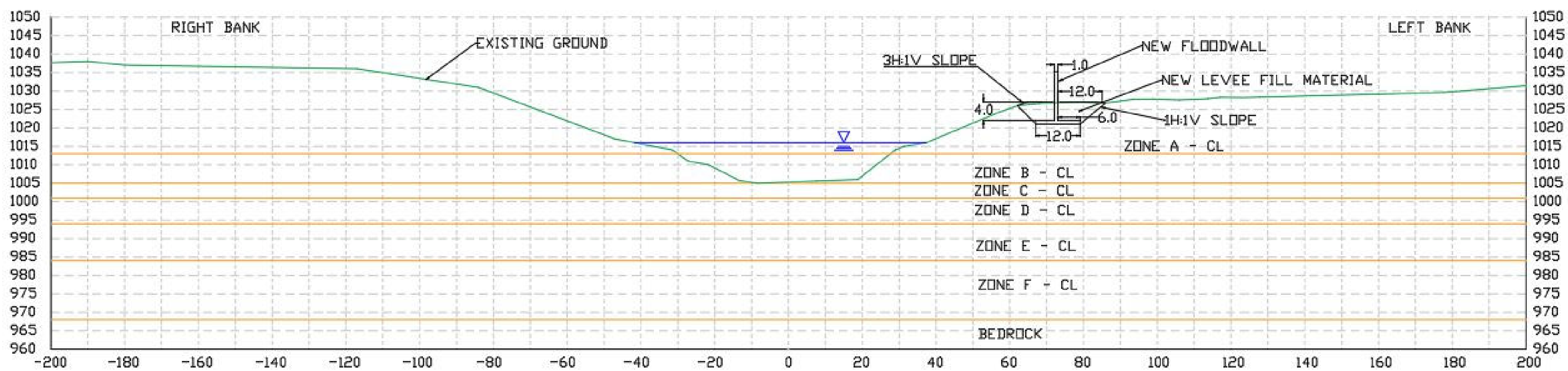


Figure 104: New floodwall cross-section and existing soil layering

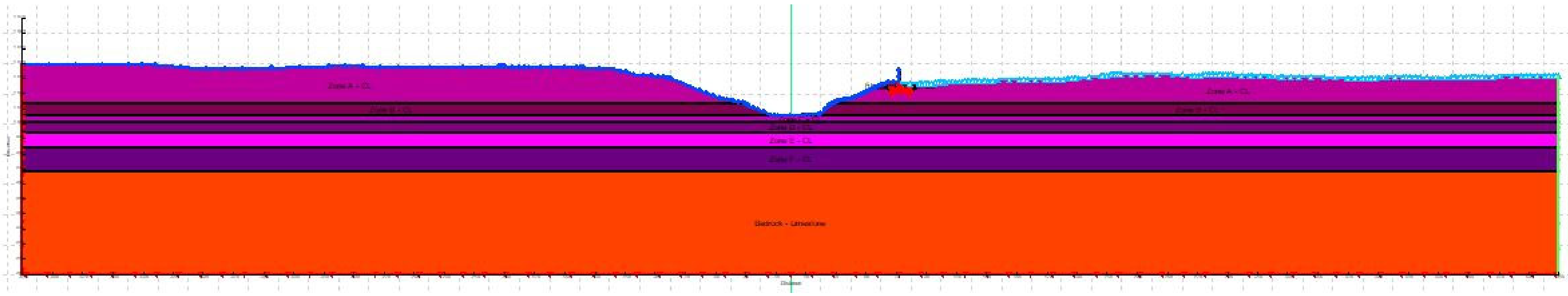


Figure 105: New floodwall steady-state seepage model setup and material layering

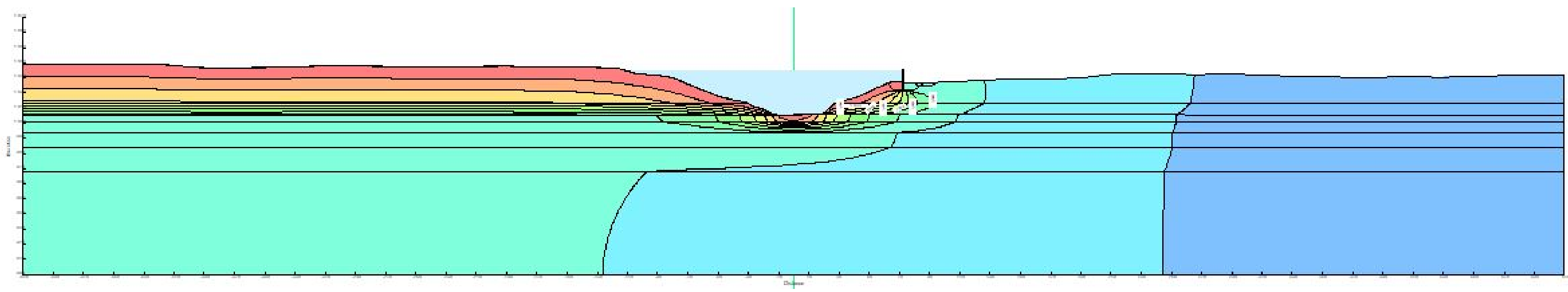


Figure 106: New floodwall steady-state seepage model results

8.5. FRAGILITY CURVES FOR EXISTING LEVEE CONDITIONS

8.5.1. Purpose

Consequence analyses of the existing levees are needed to characterize the existing conditions for planning. The resulting consequence information supports decision making by quantifying "without project" conditions that can be compared to TSP alternatives.

8.5.2. Reaches of Interest

Fragility curves were developed for nine systems in the basin. Two of the systems required analyses of two reaches. The specific locations of the 11 analyses sections were selected to ensure the geometry was representative of the geometry along the reach. The following cross section locations were used for the 11 reaches to be analyzed.

Table 42: Existing Levee Systems and Segments

Levee System			Cross Section
Creek	**Bank**	**Segment**	**Station**
Big Papio	RB	West Center street to L street	20+00
Big Papio	LB	West Center street to L street	18+00
Little Papio	RB	Upstream of confluence *	60+00
Big Papio	LB	Upstream of confluence *	10+00
Big Papio	RB	L street to Thompson Creek	15+00
Little Papio and Big Papio	LB	L street to Copper Creek	40+00
Big Papio	LB	Copper Creek to Big Elk Creek	40+00
Big Papio	LB	Big Elk Creek to Mud Creek	40+00
West Papio	RB	Upstream of confluence *	100+00
Big Papio	RB	Upstream of confluence *	350+00
West Papio	RB	96th street to Big Papio	60+00
* indicates pairs of reaches that are segments of a system.			

These reaches are included in the Fragility Curve Analysis Cross Sections, which are included in this attachment.

8.5.3. Existing Levee System Risks

8.5.3.1. NLD Risk Characterization

The National Levee Database (NLD) presents information to the public about the levees in the Corps inventory. The information includes inspection information and conclusions from levee screening risk assessments. The levee screening information in the NLD for the Papillion Creek Basin is incomplete, and this may be intentional. Previous segment and system delineation was based on the hydrologic analyses available at the time. It is anticipated that the levee inspection system and the NLD segments and systems will be updated to be consistent with the hydrologic analyses completed for this study. While most of the inspection and levee screening information will need to be revised to reflect the new system delineations, the risk characterizations will remain substantially the same.

A few of the existing Papillion Creek systems line up with the systems delineated for this study and will not need to be changed. The NLD includes risk characterization text, as well as useful performance history.

Table 43: Existing Levee Segment Risk Characterization

Levee System			LST
Creek	**Bank**	**Segment**	**Segment ID**
Big Papio	RB	L St. to Thompson Creek	NEDOUG16
Loading history includes "greater than 100% with no observations of negative performance prior to overtopping."			
Risk drivers include embankment stability erosion. Risk Characterization: Low			
Little Papio and Big Papio	LB	L St. to Copper Creek	NEDOUG16
Loading history includes "loaded between 75-100% two times and the levee segment was not damaged. The levee is expected to perform well under a full range of loading conditions."			
Risk Characterization: Low			
Big Papio	LB	Copper Creek to Big Elk Creek	NEDOUG16
Loading history includes "a maximum loading of 100% with no observations of poor performance. The levee is expected to perform well under a full range of loading conditions."			
Risk Characterization: Low			
West Papio	RB	Upstream of Confluence*	NESARP84
Loading history includes "loaded to greater than 100% with no observations of negative performance prior to overtopping."			
Risk Characterization: Low			
Big Papio	RB	Upstream of Confluence*	NEDOUG16
Loading history includes "loaded to greater than 100% with no observations of negative performance prior to overtopping."			
Risk Characterization: Low			
*Indicates pairs of reaches that are segments of a system			

The NLD includes many other systems in the basin, and map information indicates they extend along the full length of the systems being considered for this study. Extracting NLD information based on coincident locations along the Creek was not considered necessary and was not done. The risk characterization information is relatively consistent throughout the basin:

- The identified risk drivers typically include internal erosion associated with corroded CMP drains; bank erosion, and slope stability. The District believes slope stability is the risk driver for the levee systems in this study.
- Past performance includes multiple loadings exceeding 75% of levee height, with 100% levee height events documented for several segments. Only good performance during these events is documented.
- Few of the risk characterizations explain that one of the reasons risk is considered low is the fact that the Sponsor locates damage and completes repairs quickly after flood events. This is taken as a likely indication that the flood events cause little damage.
- Overall, risks are considered Low and the recommended LSAC is 4.

This information and the more detailed loading history information presented in the Hydrology and Hydraulics section of this study support the claim that risks are low and the existing levees should be expected to perform well.

8.5.4. Fragility Curve Methods

8.5.4.1. Guidance

Current guidance, Planning Risk Assessment for Flood Risk Management Studies, ER 1105-2-101 (July 2017) indicates:

- Studies will adopt the risk framework as described;
- The goal of risk assessment is to subject the values of all key variables, parameters, and components of the analyses to probabilistic analyses.
- When a more detailed assessment is required, uncertainty in a set of 8 specific variables, including fragility curves, shall be included.
- Not all variables are critical to project justification in every instance.

Because the levees should be expected to perform well even if they are fully loaded by a flood event, the likelihood of failure resulting from a 100% loading flood event will remain relatively low. This low likelihood condition is an upper bound for a fragility curve regardless of how many failure modes are considered or how much analysis detail is developed. With low failure likelihood values at 100% loading conditions, analyses show that consequences for "without project conditions" are increased, but only slightly.

For the consequence analyses being completed, levee fragility is not critical to project justification. It yields damages prior to overtopping that reduces the value of the existing levee system. This effect is confirmed to be too small to control consequence analyses results.

8.5.4.2. Justification for Simplified Approach

For this planning study, fragility curves were developed considering limited methods presented in the LST Technical Reference Manual with a key likelihood estimate based on a method by Duncan (1999).

An argument in favor of using a simplified method can be based on the good to excellent past performance of the levee systems. Where levee systems have not been tested to 100% loading with flood flows at top-of-levee, it's the result of variable capacity along the drainage. Sections of the stream channels upstream and downstream have experienced bank full flood flows with good performance reported. Many sections have been tested to above 75% of levee height with good performance reported. This good past performance ensures that a reasonable estimate of the likelihood of failure should be low. This limits the range of credible values for P_{TOP}. Considering Figure 97, the combined fragility curve maximum must be restricted to a probability range consistent with a low or very low likelihood.

Other arguments in favor of simplified methods can be based on the economic analyses. Errors or inaccuracies attributable to simplified methods can introduce unwanted bias in the economic analyses procedures. However, for this study, the analyses include fragility for estimation of flood damages prior to overtopping for both the without-project condition and the with-project condition. This is because the alternatives are formulated such that the existing levees remain in place, with flood risk reduction measures upstream and downstream. The effect is that bias that might be introduced by systematic inaccuracies in the fragility curves cannot cause outsize bias in the economic analyses results. The time and effort to develop site specific fragility curves using complex methods is not necessary.

8.5.4.3. Simple LST Fragility Curve Approach

The Levee Screening Tool (LST) is used to characterize levee flood risks, which allows relative ranking of the inventory of levees in Corps programs. It also supports risk communication among stakeholders. One of the objectives is to compute a (combined) total annual probability of inundation (API) as shown in Equation (1). In order to obtain the API, an annual probability of inundation prior to overtopping ($API_{priorOT}$) and annual probability of inundation for overtopping (API_{OT}) are computed across a range of flood water surface elevations for different potential failure modes, for example piping, stability, erosion, etc. The point likelihoods for the different modes are combined to obtain the total. Equations (2) and (3) define $API_{priorOT}$ and API_{OT}, respectively. Figure 97 presents a diagram of the calculation geometry.

$$API = API_{priorOT} + API_{OT} \qquad \text{Eq.(1)}$$
$$API_{priorOT} = \sum i \; [1/2 \; (ACE_{TOE} - ACE_{OT}) * (P_{TOPi} + ACE_{OT}) * P_{TOPi}] \qquad \text{Eq.(2)}$$
$$API_{OT} = ACE_{OT} * (P_{OT} - \sum i \; P_{TOPi} \qquad \text{Eq.(3)}$$

where,

- i = failure mode considered
- ACE_{TOE} = annual chance exceedance for a flood level at the toe of the levee
- ACE_{OT} = annual chance exceedance for a flood level at the onset of overtopping (i.e., top of levee)

- P_{TOPi} = (exceedance) probability of breach for a particular levee performance conditional on flood loading at the top of the levee
- P_{OT} = (exceedance) probability of inundation conditional on an overtopping flood loading (assumed to be 1.0)
- The ACE is based on the water surface elevation (EL) of the levee on the landside estimated from historical, gage and published data as well as rating curves among others. The relationship between the conditional probability of failure (P_{TOPi} and P_{OT}) and EL is referred to as fragility or performance curves which are briefly explained in the following section.

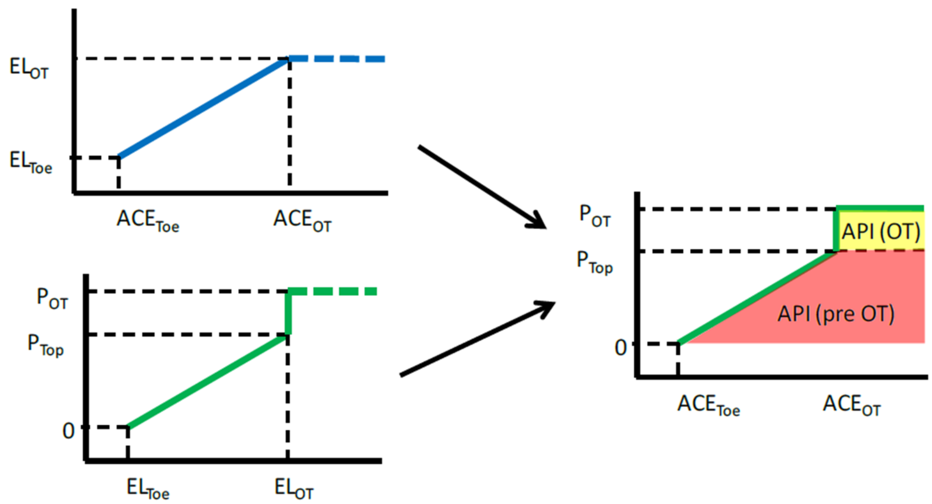


Figure 107. Diagrams showing geometry of $API_{priorOT}$ and API_{OT} (LST 2015). Hydrologic analyses establish peak floodwater elevations corresponding to toe and top of levee.

The plot of likelihood of failure1 via a particular failure mode vs flood water surface elevation is a fragility curve. Conventionally, the flood water surface elevation is converted from elevation to conditional probability of non-exceedance. Fragility curves are developed for relevant failure modes (i) and are assumed to be independent and uncorrelated. The combined conditional probability (for only two modes) is calculated per equation (4). Figure 97 shows an example of particular fragility curves and the combined results.

$P_{TOPi} = 1 - (1 - P_{TOP1})(1 - P_{TOP2})$ Eq.(4)
where,

- P_{TOP} = combined (non-exceedance) likelihood of breach
- P_{TOP1} = (non-exceedance) probability of breach for levee performance 1 conditional on flood loading

[1] Note here that P_{TOP} is a likelihood: the abscissa axis of the fragility curve. The API equations yield the area under the curve and are probabilities. The LST Technical Manual uses, "P" as notation for a likelihood (and the text includes errors where the word "probability" is used incorrectly). This notation is retained in this discussion for consistency.

P_{TOP2} = (non-exceedance) probability of breach for levee performance 2 conditional on flood loading

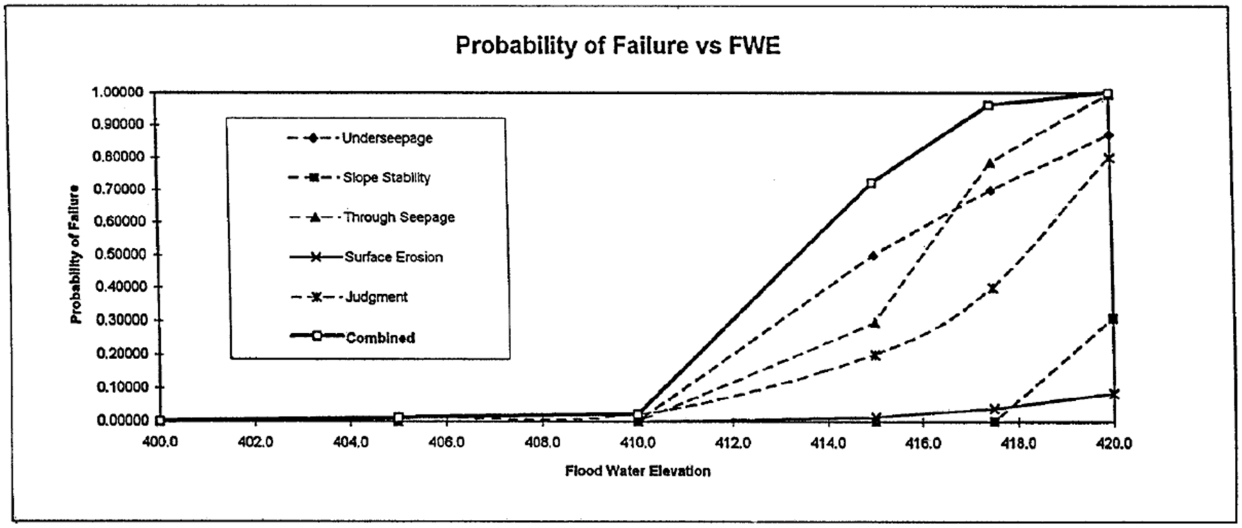


Figure 108. Example of combining multiple fragility curves. (LST 2015)

A simplified fragility curve can be developed by calculating P_{TOPi} per Eq. (2.). It is assumed that the ACE_{TOE} is the same for all the considered failure modes. It should be noted that this works well for piping, stability, and erosion, but not for closure systems, which are operated and loaded at different water surface elevations.

In the LST, a conditional probability of exceedance of 1.0 is used where the water surface elevation is above levee crest. This gets to a 3-point fragility curve with likelihood points at: ACE_{TOE}; ACE_{TOP}, and an ACE corresponding to the maximum flood with overtopping depth.

For the consequence modeling being completed for this study, point estimates of failure likelihood for depths of overtopping are not needed. The fragility curves are used for estimating consequences prior to overtopping and are not extended above top of levee. The result is that an adequate characterization of fragility can be based on a two-point "curve." The likelihood of failure P_{TOE} is 0 so only P_{TOP} must be estimated. The resulting fragility curve could be similar to this:

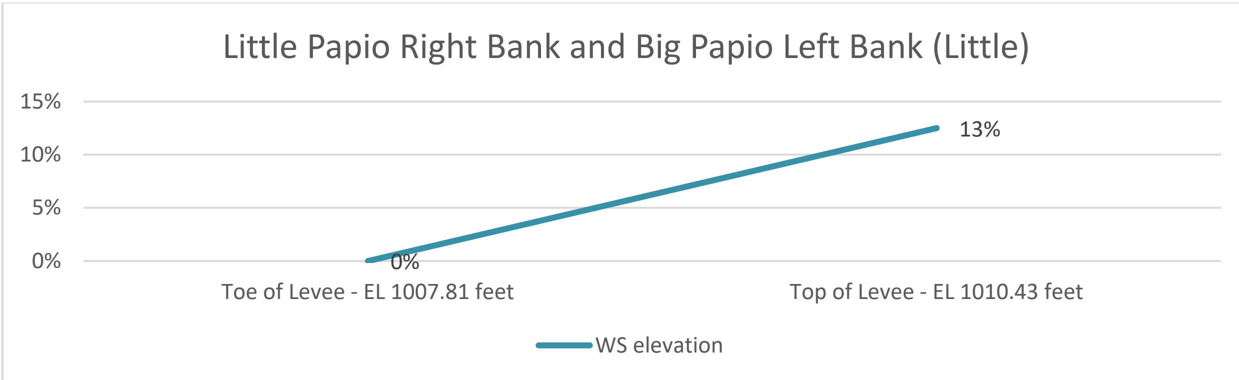


Figure 109. Example two-point fragility curve.

This example uses water surface elevation because the consequence model routinely uses elevation rather than ACE so transforming the abscissa data points is not necessary.

Also note that the likelihood of failure with floodwaters at the top of levee is relatively low. This is a key feature of the fragility functions for this study and it is a strong argument in favor of simplified methods.

8.5.4.4. Estimating P_{TOP} Per the ETL

The LST Technical Manual includes an example with P_{TOP} estimates as a given. It provides no indication of how much effort was invested in developing those estimates. The reference for the procedures, however, is:

- USACE (1999). Risk-Based Analysis in Geotechnical Engineering for Support of Planning Studies. Engineering Technical Letter ETL 1110-2-556, Washington, DC.

The USACE Digital Library indicates the document is for historical reference only. It is no longer active. “Do not distribute this publication.”

(It should be noted that Appendix A of this ETL, “An Overview of Probabilistic Analysis for Geotechnical Engineering Problems” is accessible and is sometimes used in practice.)

The methods presented in the ETL are consistent with Figure 37. They rely on multiple probabilistic models to extract point estimates of likelihood for different water surface elevations and different failure modes. These analyses can be elaborate and time consuming. In practice, it seems engineers rely on a probabilistic model of the risk driver failure mode and avoid the effort of multiple models.

For this planning study, P_{TOP} was estimated consistent with an approach by Duncan (1999).

8.5.4.5. Estimating P_{TOP} Per Duncan

For this study, two points were used to define a straight-line fragility curve approximation. At the toe-of-levee elevation, the likelihood of levee failure is zero. At top-of-levee elevation, it is possible to estimate a likelihood of levee failure (P_{TOP}) by considering extreme values. For the levees being considered, the methods proposed by Duncan (1999) provide an adequate approach for estimating P_{TOP} and this yields a credible fragility curve for use in consequence analyses.

Duncan (1999) explains that traditional Factor of Safety (FS) design can be combined with reliability analyses, "as complementary measures of acceptable design." A key advantage of incorporating reliability into design analyses is the insight it provides into the overall uncertainties. One of the significant needs for incorporating reliability analyses is obtaining an estimate of the standard deviation of a parameter. The paper presents several methods for estimating a standard deviation, including (1) computation, (2) published values, and (3) using the three-sigma rule.

The three-sigma rule falls from the fact that 99.73% of all values of a normally distributed parameter fall within three standard deviations of the average. The utility of the three-sigma rule is where estimates of maximum conceivable values and minimum conceivable values can be estimated or are in some way constrained. Where these max and min conceivable values can be estimated, they can be used to determine a most likely value – the average/midpoint value – as well as the standard deviation, which is 1/6th of the difference between max and min.

This approach can be used to estimate P_{TOP} using a guestimate of the maximum credible likelihood of failure given floodwaters at top-of-levee (P_{TOPMAX}).

8.5.5. Likelihood Estimate for A Baseline Levee System

8.5.5.1. Cross Sections

Fragility curves are used in economic analyses of the existing levee systems. As a practical matter, the consequences are evaluated for sections of levees and this is generally based on cross section geometry along the drainage. The effect is that the fragility curves used for consequence analyses are selected to go with a set of levee cross sections rather than just one per levee system. The cross sections for analyses were selected by the Civil and Hydrologic PDT members based on cross sections extracted from plan information.

Likelihood estimates of maximum and minimum credible values were developed for a typical section or "baseline" levee. In general terms, the baseline levee would have grass-covered 3H:1V slopes, with a 10- to 12-foot crest width, and a height of approximately 5 feet.

The differences between the geometry at the analyses cross sections and the baseline section were used to justify guesstimated deviations from the baseline levee likelihood estimate values. For example, where the crest width was wider than 12 feet, the maximum credible likelihood of failure could be reduced by 10%. Similarly, if the crest was more than 20 feet wide, the minimum credible estimate of failure could be reduced to zero (if it were above zero). This was a simple way to use levee section geometry to establish deviations from the baseline and in this way, create estimates unique to each cross section.

8.5.5.2. Past Performance

Past performance of the levee systems includes multiple loadings greater than 50% with loadings up to 100%, with generally good performance documented.

Available screening information in the National Levee Database (NLD) indicates risks are considered Low. Past performance documented in the NLD for several levee systems in the basin are as follows:

Table 44: Past Performance Information from the NLD

Segment: Little Papio RB & Big Papio LB
The levee was constructed in 1960. Significant flood events in the basin: 1964, 1965, 1993, 1997, and 1999. The largest flood (1999) loaded the levee to approximately 50%. No performance issues have been reported.
Segment: Big Papio LB - West Center to L St
Since construction the project has never been loaded and is expected to perform well under a fully loaded condition.
Segment: Big Papio RB - L St to Thompson Cr
The levee was loaded to greater than 100% with no observations of negative performance prior to overtopping.
Segment: Big Papio LB - Little Papio to Copper Cr
The levee was been loaded between 75-100% two times and the levee segment was not damaged. The levee is expected to perform well under a full range of loading conditions.
Segment: Big Papio LB - Copper Cr to Big Elk Cr
The levee has experienced a maximum loading of 100% with no observations of poor performance. The levee is expected to perform well under a full range of loading conditions.
Segment: West Papio LB & Big Papio RB
The Levee has been loaded to greater than 100% with no observations of negative performance prior to overtopping.
Segment: Big Papio RB - 36th St to Willow Lakes Golf Course
The segment has been loaded to more than 75% four times with no noted issues and has overtopped without breach.

In general, the foundation and levee fill materials are clayey, low permeability soils that have low to moderate erodibility and generally good grass cover. Flood durations are brief and the system is well maintained. In future floods, the levee systems are expected to perform well. The risk characterization for the levees in the Papillion Creek basin is "Low." The NLD describes this characterization as follows:

"Likelihood of inundation due to breach and/or system component malfunction in combination with loss of life, economic, or environmental consequences results in low risk."
This characterization is not linked by definition or custom to a particular likelihood of levee failure.

8.5.5.3. Selecting P_{TOPMAX} for the Baseline Levee

As previously stated, a baseline levee is in the Papillion Creek basin with 3H:1V grass-covered slopes, a 10- to 12-ft crest width, and a height of approximately 5 feet. For these analyses, the maximum credible likelihood of failure for the baseline levee during a 100% load condition event (P_{TOPMAX}) was guesstimated to be 15%. Per Duncan (1999), this yields a most likely likelihood of levee failure for a fully loaded baseline levee of 8%, which is the average of the minimum (0) and P_{TOPMAX}.

The maximum credible estimate of failure given a 100% load condition flood event should be consistent with the descriptive scale for risk characterization. The scale has 5 increments: Very High, High, Moderate, Low, and Very Low. This invites a 5 increment, 20% step scale that explicitly incorporates both likelihood and consequences.

The maximum credible likelihood of failure for the baseline levee was estimated to be 15%. This estimate was developed in two steps:

- Step 1: The risk characterization combines consequences and the likelihood of failure. Considering site soil conditions and erodibility, grass cover, and short flood durations invites attributing more of the risk characterization to consequences rather than likelihood. This justifies incrementing the maximum credible likelihood estimate downward, to the range of "Very Low," which would correspond to a maximum value of 20%.
- Step 2: Considering flood history, past performance, and good maintenance and repair practices, further reducing the maximum credible likelihood estimate is justified. A reduction of 25% was selected as significant, but not dramatic. This yields the 15% value that was selected.

This approach is contrived and incorporates subjective presumptions and uncertainties that are not consistent with the numerical precision of the numbers. However, the reasoning captures a number in the context of the overall risk for the baseline levee. Conventionally, this kind of estimation would be done by an expert elicitation effort that would rely on opinions of at least several technical people who discussed the issues as a team. This was not done for selecting this value.

It should be noted that it is not "conservative" to use higher likelihood values for fragility analyses. Higher values decrease the damages prevented by the existing levees, which correlates to lowering the economic value of the existing levees. The economic analyses confirm the fragility curves that were used do not control the planning decision.

8.5.5.4. Adjusting P_{TOPMAX} for Different Cross Sections

Several of the selected cross sections were very different from the baseline levee. Generally, these were locations where the crest width was substantially wider than 12 feet or levee height was less than 2 feet. The "adjustment" amounted to reducing the maximum credible likelihood of failure during a 100% loading flood event by either 5% or 10%.

Attached cross section data sheets include a vicinity map, cross section, elevation data, and crown width data for the selected cross sections. The data sheets also present the calculation table, which shows extreme values, adjustments for section variation from the baseline, and the most likely value, as well as the fragility curve.

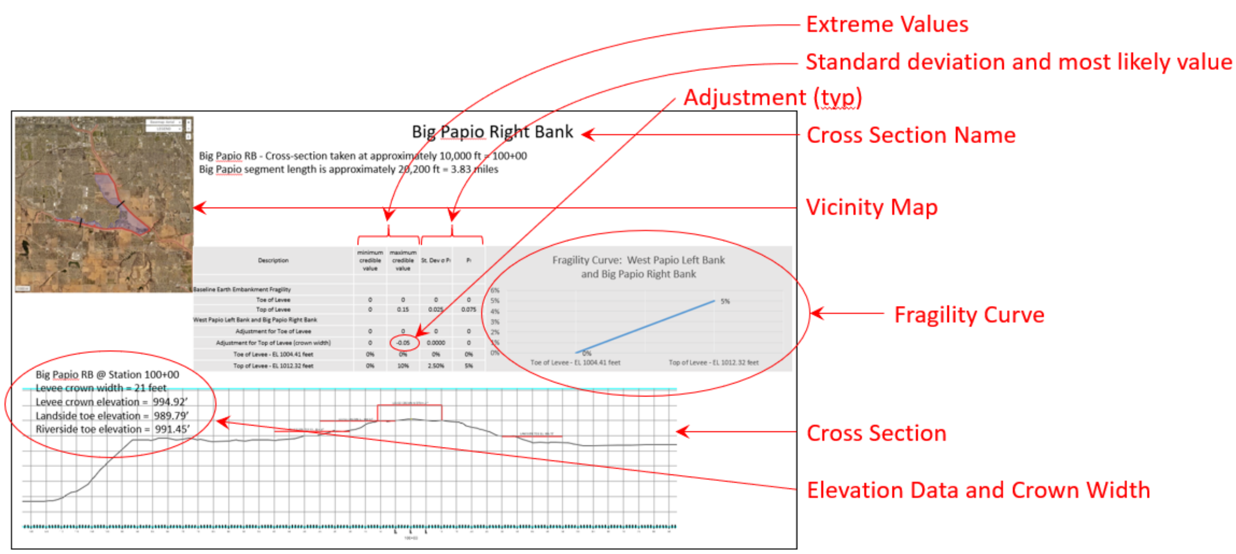
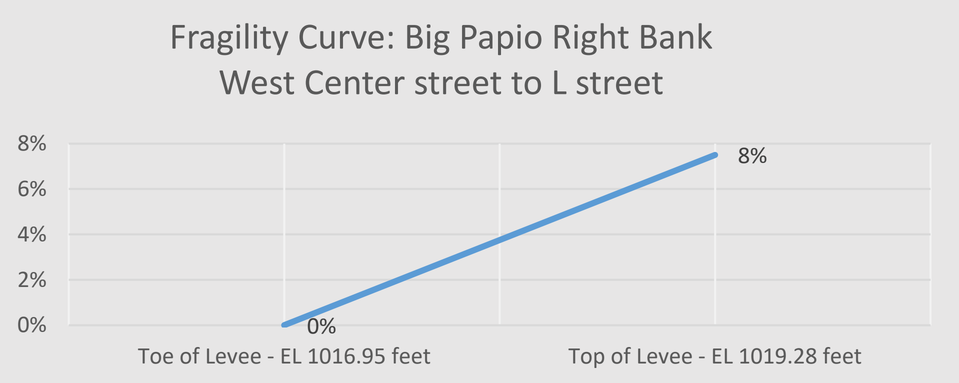


Figure 110. Example data sheet showing calculation table and fragility curve for a cross section. See attached.

The adjustment values are arbitrary, intended only to capture a difference in likelihood estimates from the baseline to better correspond to the cross section. The standard deviations are calculated per Duncan as 1/6th of the difference of the max and min estimates. The most likely values are the average of the max and min estimates. The fragility curve data needed for the consequence analyses can be read from the fragility curve plots. All of the systems and cross sections with tables and the straight-line fragility curves shown in the figure are attached.

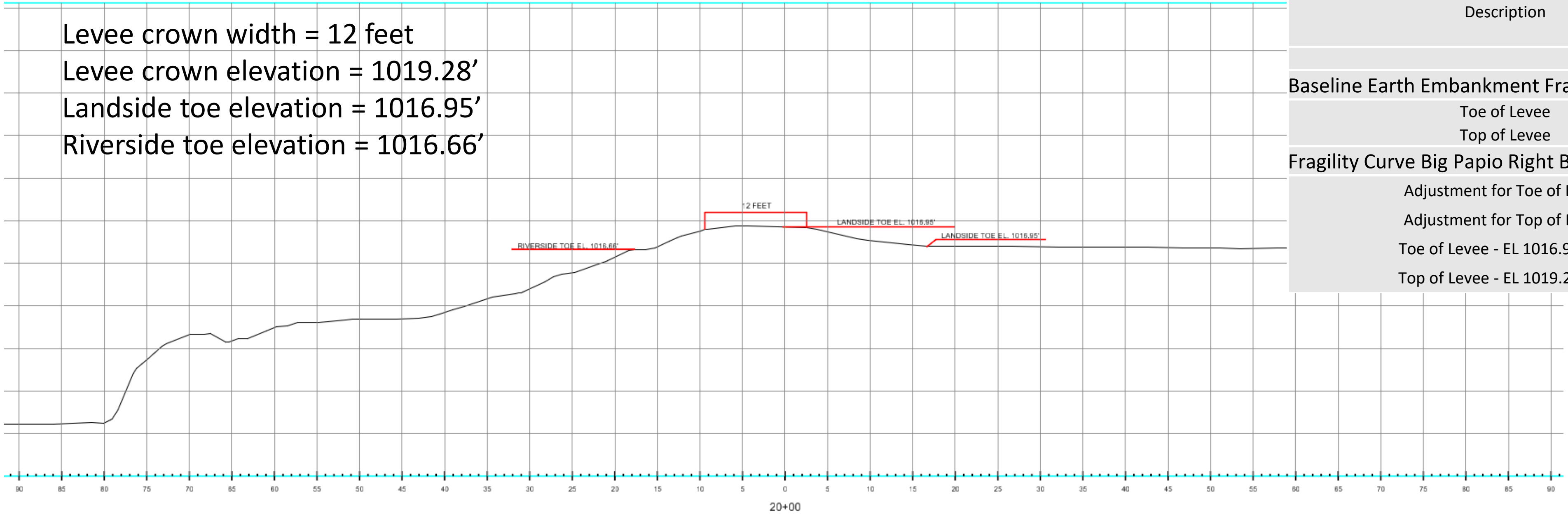
Big Papio Right Bank – West Center Street to L street

Cross-section taken at approximately 2000 ft = 20+00
Total length of levee is approximately 2,400 ft = 0.46 miles
Total length along channel is approximately 8,500 ft = 1.6 miles



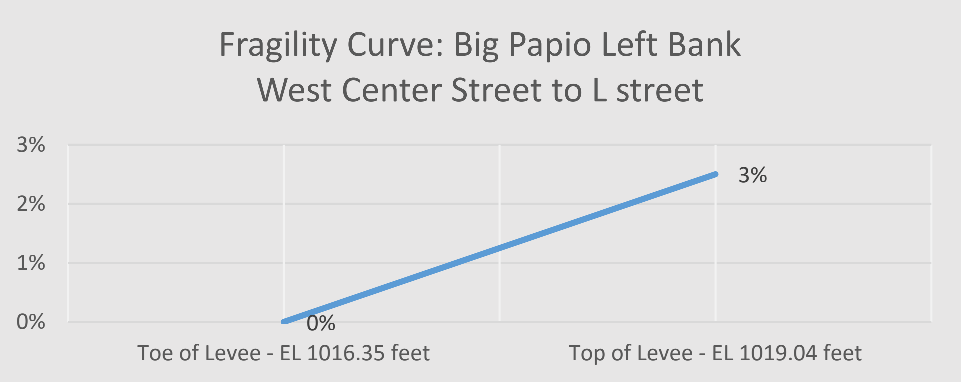
Description	minimum credible value	maximum credible value	St. Dev σ P_f	P_f
Baseline Earth Embankment Fragility				
Toe of Levee	0	0	0	0
Top of Levee	0	0.15	0.025	0.075
Fragility Curve Big Papio Right Bank – West Center street to L street				
Adjustment for Toe of Levee	0	0	0	0
Adjustment for Top of Levee	0	0	0	0
Toe of Levee - EL 1016.95 feet	0%	0%	0%	0%
Top of Levee - EL 1019.28 feet	0%	15%	2.50%	8%

Levee crown width = 12 feet
Levee crown elevation = 1019.28’
Landside toe elevation = 1016.95’
Riverside toe elevation = 1016.66’

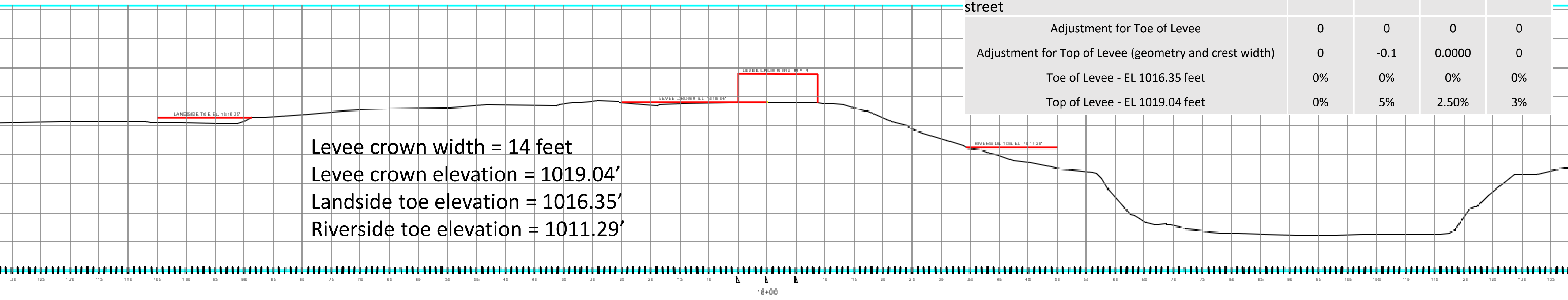


Big Papio Left Bank – West Center Street to L Street

Cross-section taken at approximately 1800 ft = 18+00
Total length of levee is approximately 3,300 ft = 0.62 miles
Total length along channel is approximately 5,800 ft = 1.1 miles



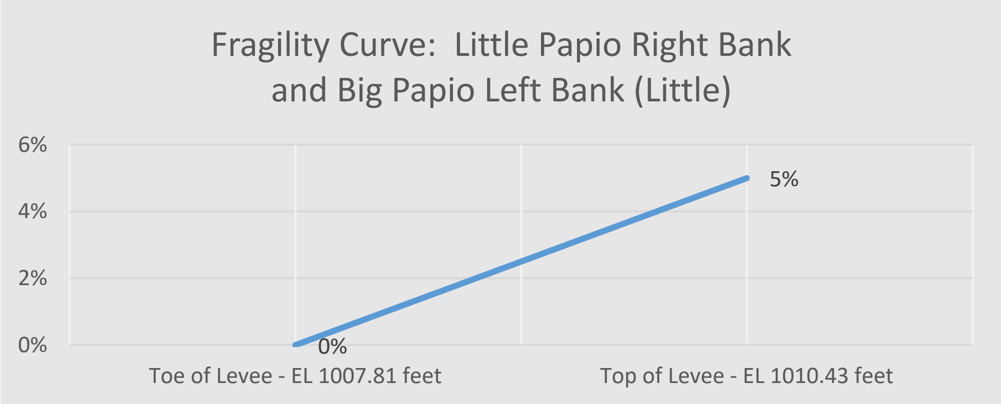
Description	minimum credible value	maximum credible value	St. Dev σ P_f	P_f
Baseline Earth Embankment Fragility				
Toe of Levee	0	0	0	0
Top of Levee	0	0.15	0.025	0.075
Big Papio Left Bank – West Center Street to L street				
Adjustment for Toe of Levee	0	0	0	0
Adjustment for Top of Levee (geometry and crest width)	0	-0.1	0.0000	0
Toe of Levee - EL 1016.35 feet	0%	0%	0%	0%
Top of Levee - EL 1019.04 feet	0%	5%	2.50%	3%



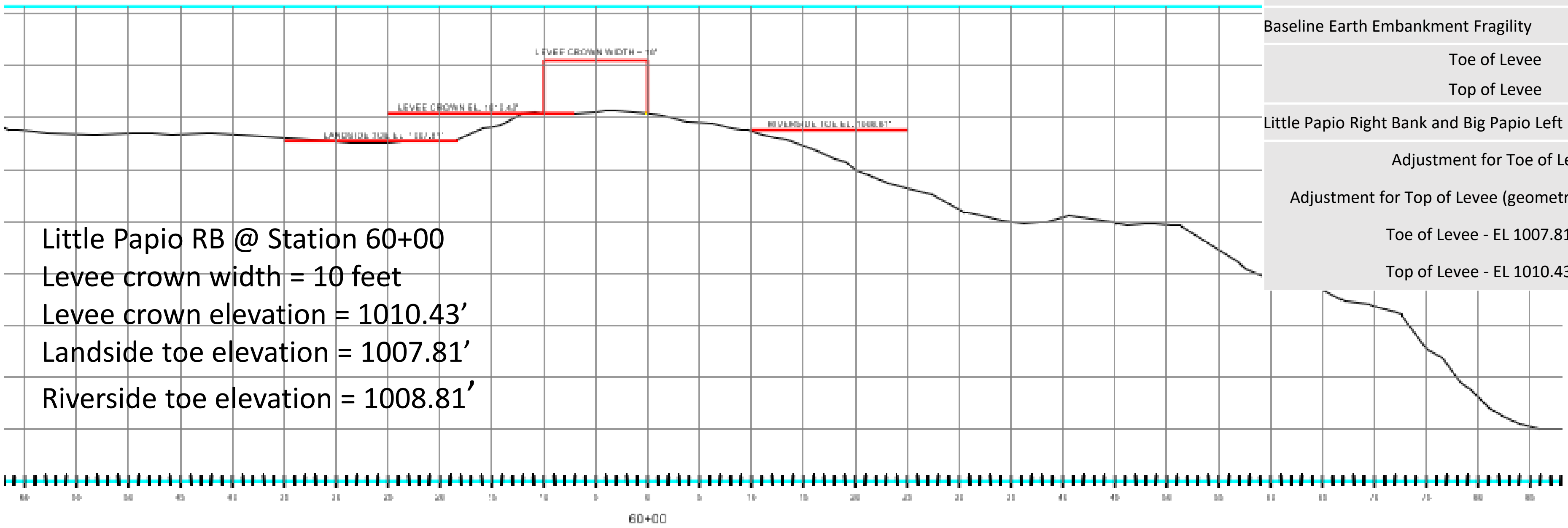
Levee crown width = 14 feet
Levee crown elevation = 1019.04’
Landside toe elevation = 1016.35’
Riverside toe elevation = 1011.29’

Little Papio Right Bank & Big Papio Left Bank (Little Papio)

Little Papio RB - Cross-section taken at approximately 6000 ft = 60+00
Little Papio segment length is approximately 2,000 ft = 0.39 miles



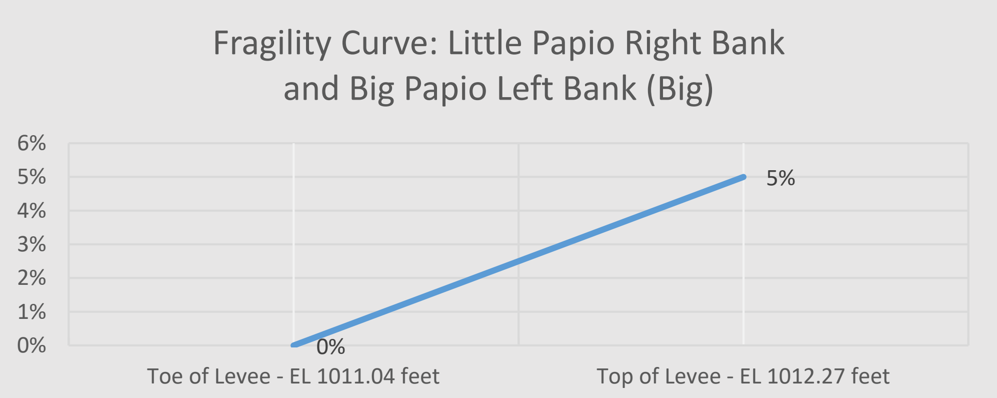
Description	minimum credible value	maximum credible value	St. Dev σ P_f	P_f
Baseline Earth Embankment Fragility				
Toe of Levee	0	0	0	0
Top of Levee	0	0.15	0.025	0.075
Little Papio Right Bank and Big Papio Left Bank (Little)				
Adjustment for Toe of Levee	0	0	0	0
Adjustment for Top of Levee (geometry and crest width)	0	-0.05	-0.0083	0
Toe of Levee - EL 1007.81 feet	0%	0%	0%	0%
Top of Levee - EL 1010.43 feet	0%	10%	1.67%	5%



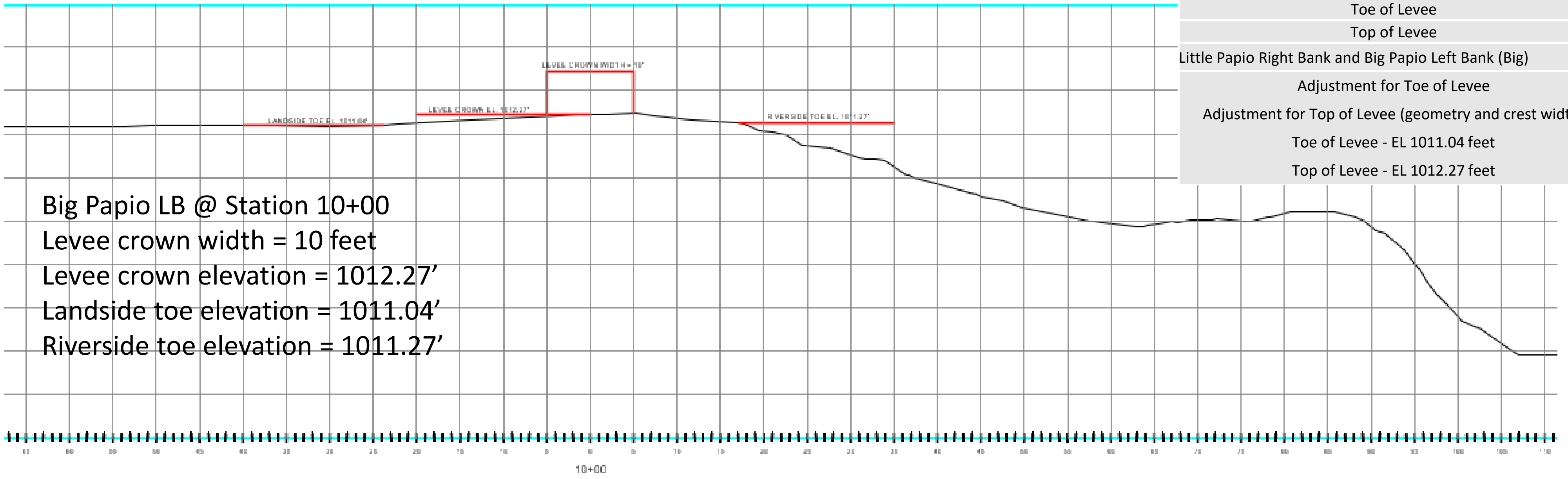
Little Papio RB @ Station 60+00
Levee crown width = 10 feet
Levee crown elevation = 1010.43’
Landside toe elevation = 1007.81’
Riverside toe elevation = 1008.81’

Little Papio Right Bank & Big Papio Left Bank (Big Papio)

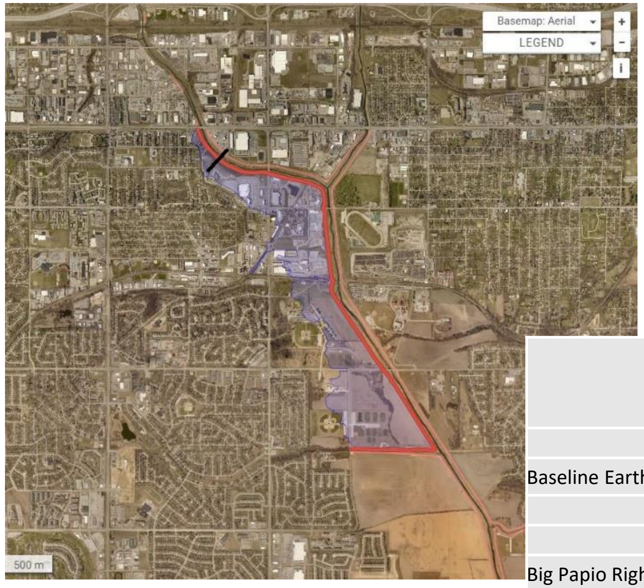
Big Papio LB – Cross-section taken at approximately 1000 ft = 10+00
Big Papio segment length is approximately 4,500 ft = 0.86 miles



Description	minimum credible value	maximum credible value	St. Dev σ P_f	P_f
Baseline Earth Embankment Fragility				
Toe of Levee	0	0	0	0
Top of Levee	0	0.15	0.025	0.075
Little Papio Right Bank and Big Papio Left Bank (Big)				
Adjustment for Toe of Levee	0	0	0	0
Adjustment for Top of Levee (geometry and crest width)	0	-0.05	0.0000	0
Toe of Levee - EL 1011.04 feet	0%	0%	0%	0%
Top of Levee - EL 1012.27 feet	0%	10%	2.50%	5%



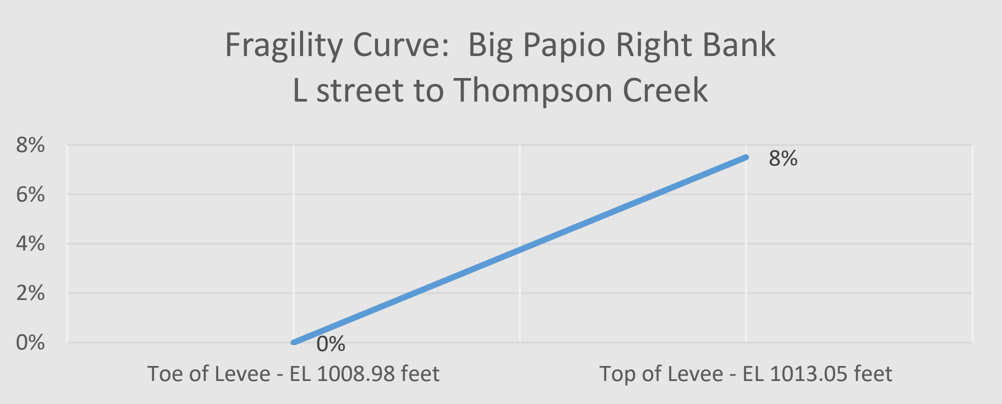
Big Papio LB @ Station 10+00
Levee crown width = 10 feet
Levee crown elevation = 1012.27’
Landside toe elevation = 1011.04’
Riverside toe elevation = 1011.27’



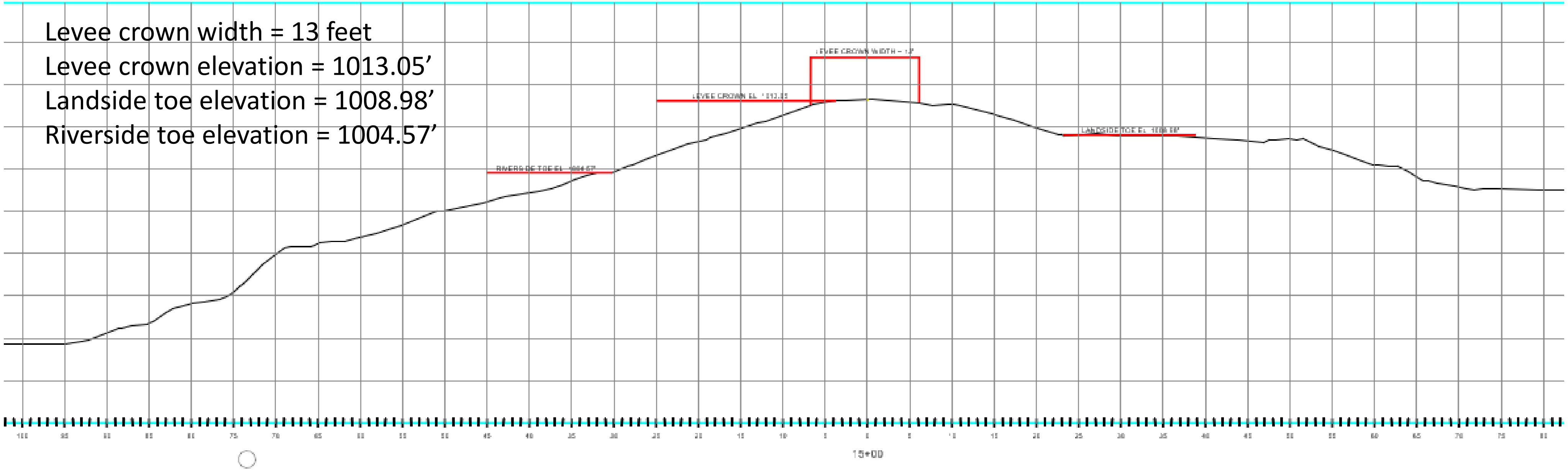
Big Papio Right Bank – L street to Thompson Creek

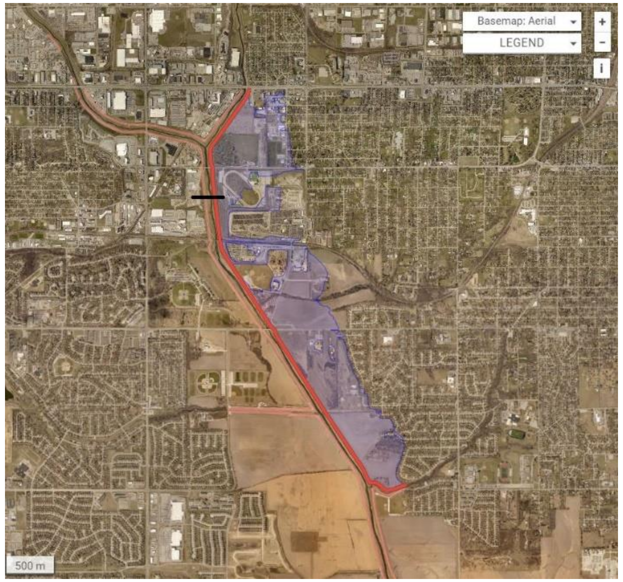
Cross-section taken at approximately 1500 ft = 15+00
Total length is approximately 17,300 ft = 3.28 miles

Description	minimum credible value	maximum credible value	St. Dev σ P_f	P_f
Baseline Earth Embankment Fragility				
Toe of Levee	0	0	0	0
Top of Levee	0	0.15	0.025	0.075
Big Papio Right Bank – L street to Thompson Creek				
Adjustment for Toe of Levee	0	0	0	0
Adjustment for Top of Levee	0	0	0.0000	0
Toe of Levee - EL 1008.98 feet	0%	0%	0%	0%
Top of Levee - EL 1013.05 feet	0%	15%	2.50%	8%



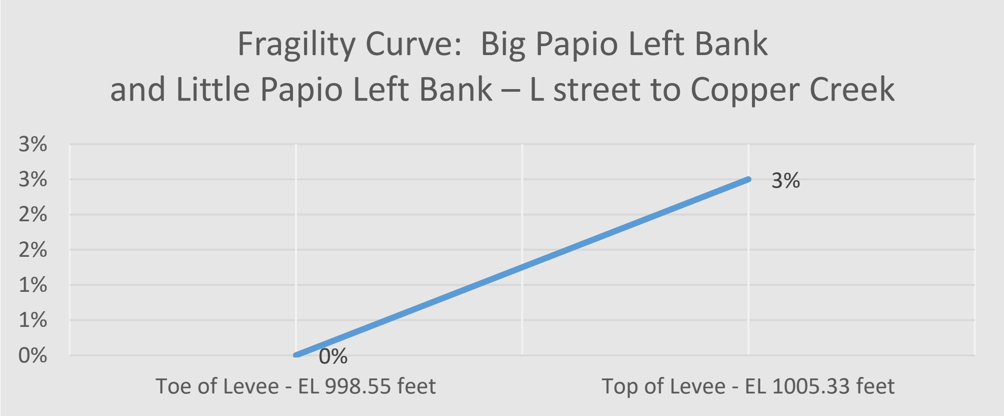
Levee crown width = 13 feet
Levee crown elevation = 1013.05’
Landside toe elevation = 1008.98’
Riverside toe elevation = 1004.57’



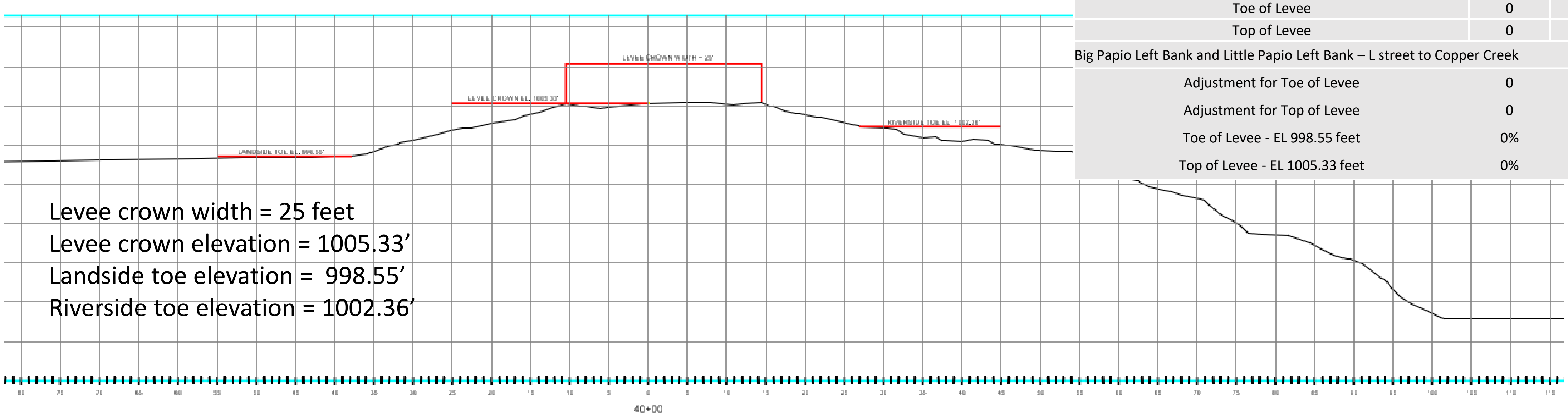


Big Papio Left Bank and Little Papio Left Bank – L street to Copper Creek

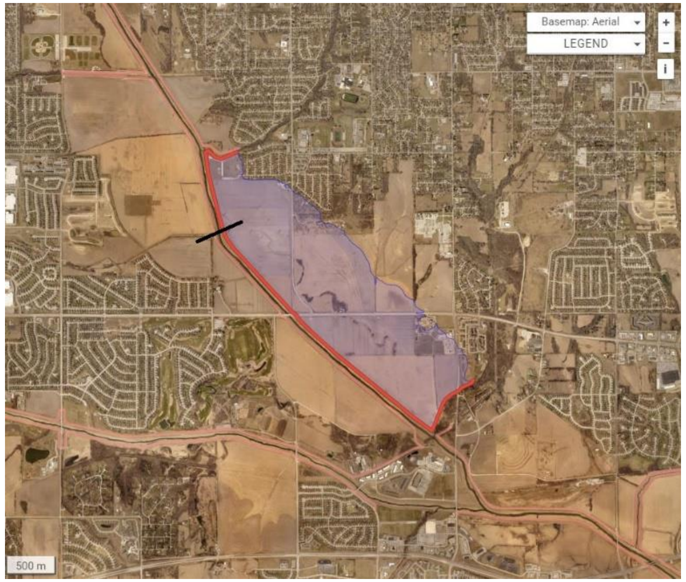
Big Papio Left Bank - Cross-section taken at approximately 4000 ft = 40+00
Little Papio segment length is approximately 2,300 ft = 0.44 miles
Big Papio segment length is approximately 13,700 ft = 2.59 miles



Description	minimum credible value	maximum credible value	St. Dev σ P_f	P_f
Baseline Earth Embankment Fragility				
Toe of Levee	0	0	0	0
Top of Levee	0	0.15	0.025	0.075
Big Papio Left Bank and Little Papio Left Bank – L street to Copper Creek				
Adjustment for Toe of Levee	0	0	0	0
Adjustment for Top of Levee	0	-0.1	0.0000	0
Toe of Levee - EL 998.55 feet	0%	0%	0%	0%
Top of Levee - EL 1005.33 feet	0%	5%	2.50%	3%

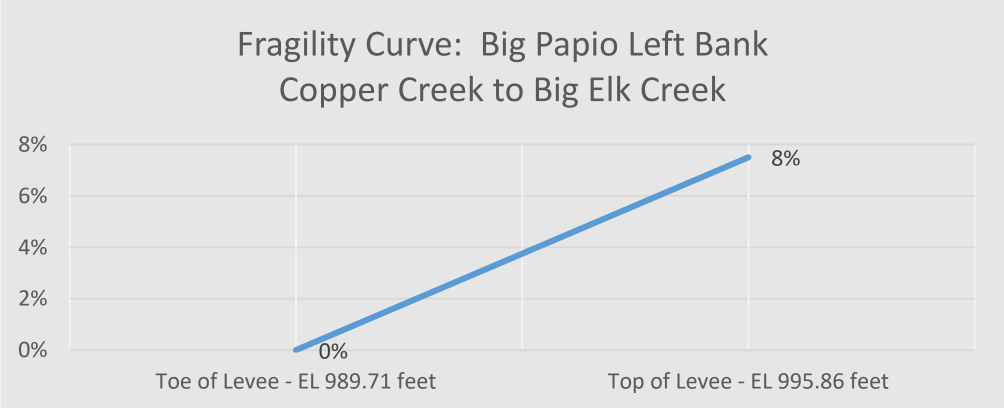


Levee crown width = 25 feet
Levee crown elevation = 1005.33’
Landside toe elevation = 998.55’
Riverside toe elevation = 1002.36’

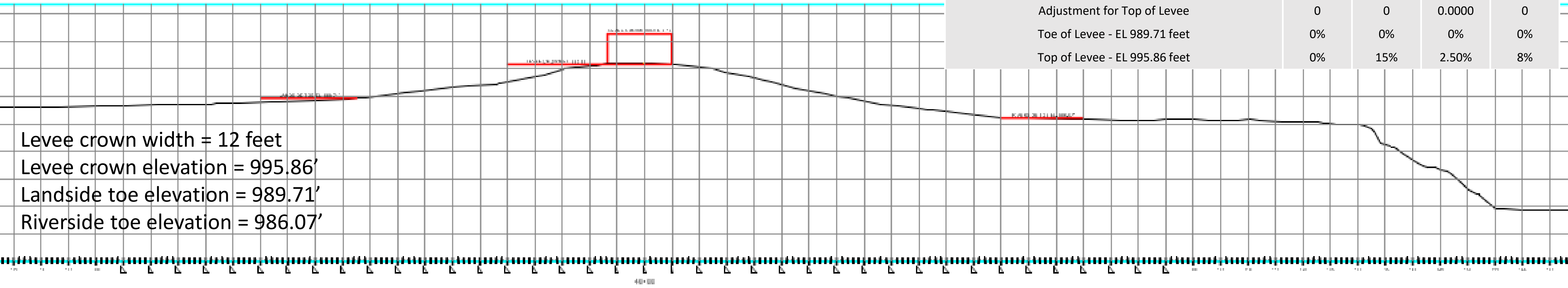


Big Papio Left Bank – Copper Creek to Big Elk Creek

Cross-section taken at approximately 4000 ft = 40+00
Total length is approximately 15,600 ft = 2.96 miles

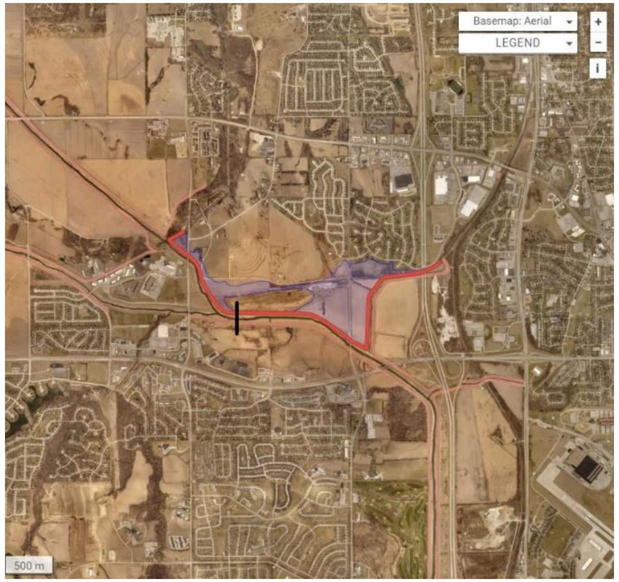


Description	minimum credible value	maximum credible value	St. Dev σ P_f	P_f
Baseline Earth Embankment Fragility				
Toe of Levee	0	0	0	0
Top of Levee	0	0.15	0.025	0.075
Big Papio Left Bank – Copper Creek to Big Elk Creek				
Adjustment for Toe of Levee	0	0	0	0
Adjustment for Top of Levee	0	0	0.0000	0
Toe of Levee - EL 989.71 feet	0%	0%	0%	0%
Top of Levee - EL 995.86 feet	0%	15%	2.50%	8%

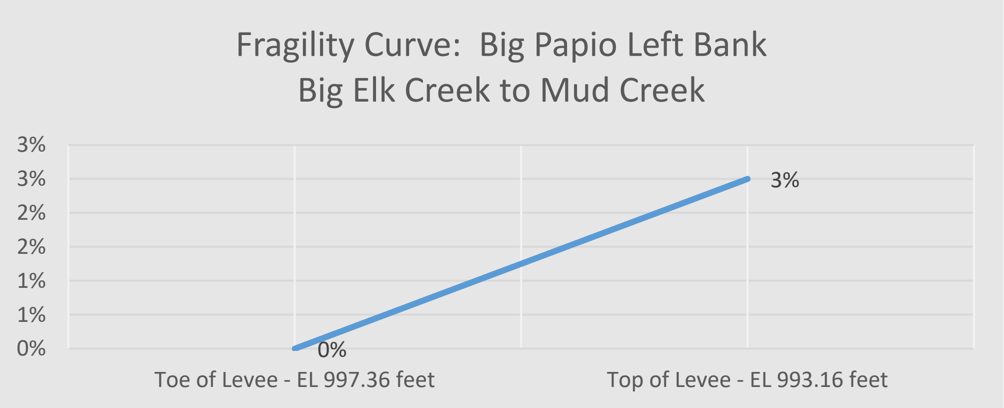


Levee crown width = 12 feet
Levee crown elevation = 995.86’
Landside toe elevation = 989.71’
Riverside toe elevation = 986.07’

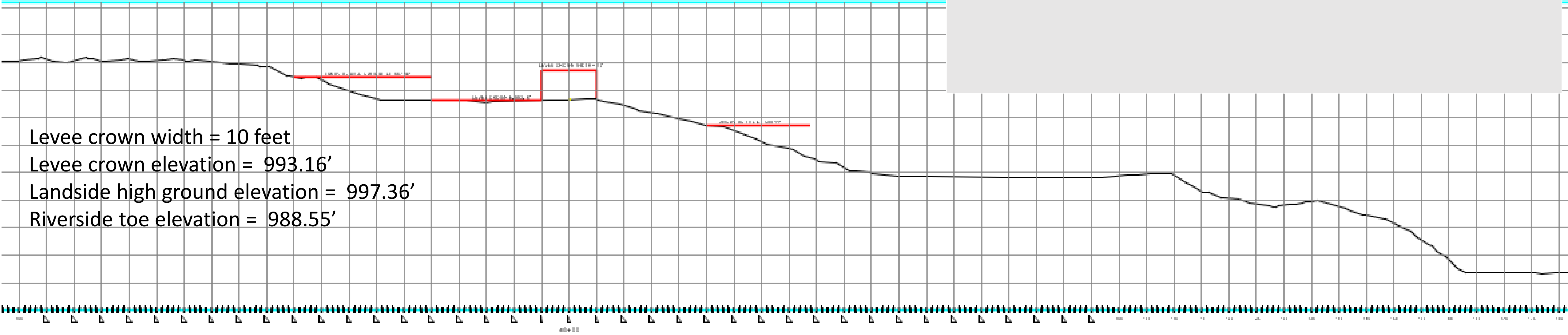
Big Papio Left Bank – Big Elk Creek to Mud Creek



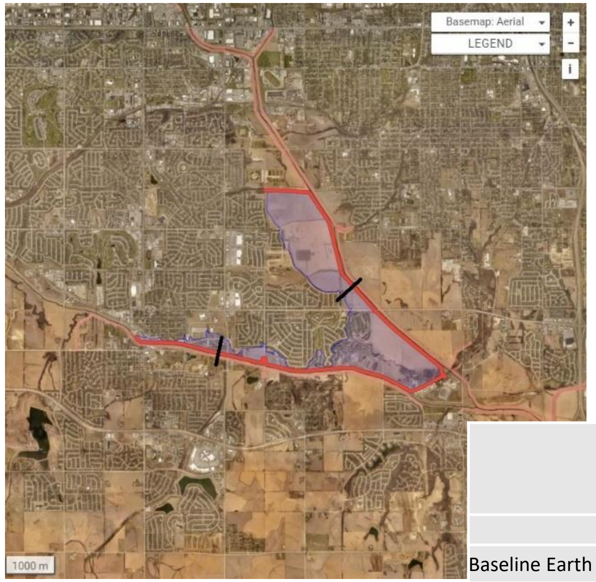
Cross-section taken at approximately 4000 ft = 40+00
Total length is approximately 12,500 ft = 2.38 miles



Baseline Earth Embankment Fragility				
Toe of Levee	0	0	0	0
Top of Levee	0	0.15	0.025	0.075
Big Papio Left Bank – Big Elk Creek to Mud Creek				



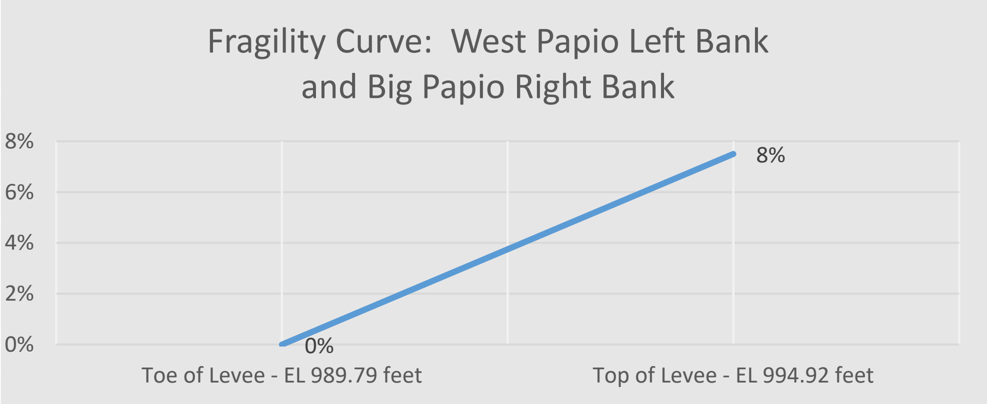
Levee crown width = 10 feet
Levee crown elevation = 993.16’
Landside high ground elevation = 997.36’
Riverside toe elevation = 988.55’



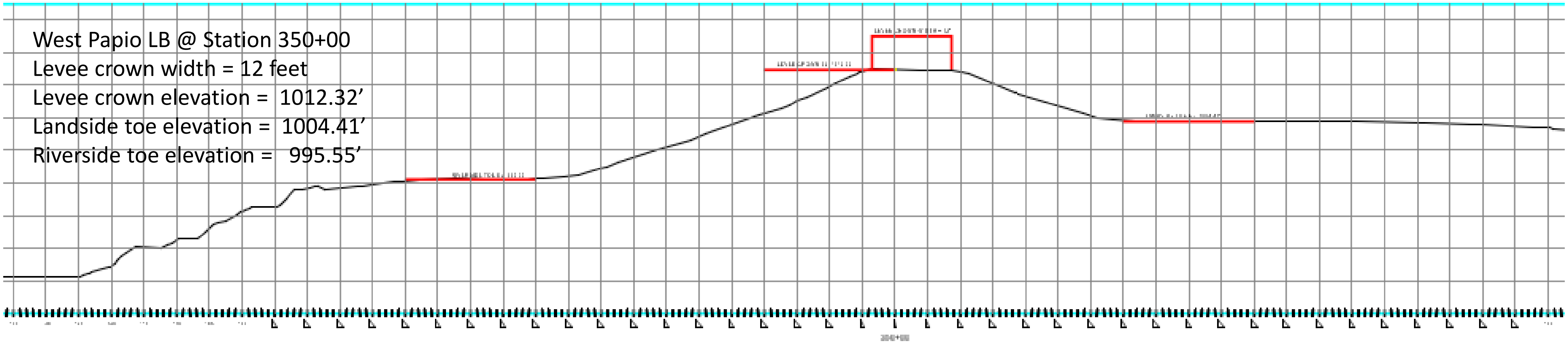
West Papio Left Bank

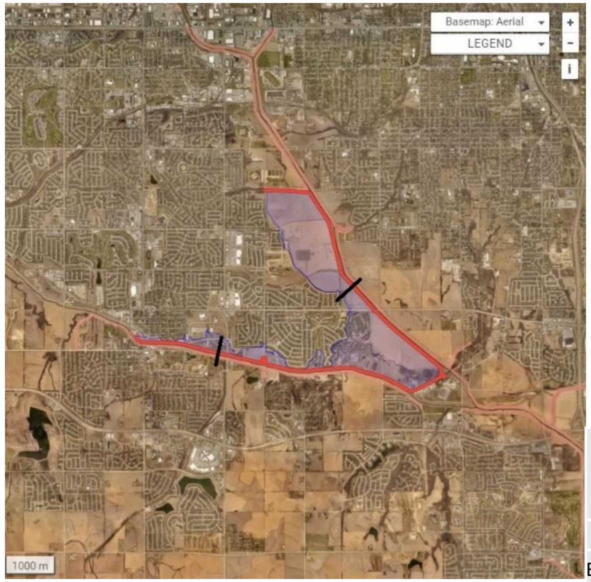
West Papio LB – Cross-section taken at approximately 35,000 ft = 350+00
West Papio segment length is approximately 20,275 ft = 3.84 miles

Description	minimum credible value	maximum credible value	St. Dev σ P_f	P_f
Baseline Earth Embankment Fragility				
Toe of Levee	0	0	0	0
Top of Levee	0	0.15	0.025	0.075
West Papio Left Bank and Big Papio Right Bank				
Adjustment for Toe of Levee	0	0	0	0
Adjustment for Top of Levee	0	0	0.0000	0
Toe of Levee - EL 989.79 feet	0%	0%	0%	0%
Top of Levee - EL 994.92 feet	0%	15%	2.50%	8%



West Papio LB @ Station 350+00
Levee crown width = 12 feet
Levee crown elevation = 1012.32’
Landside toe elevation = 1004.41’
Riverside toe elevation = 995.55’

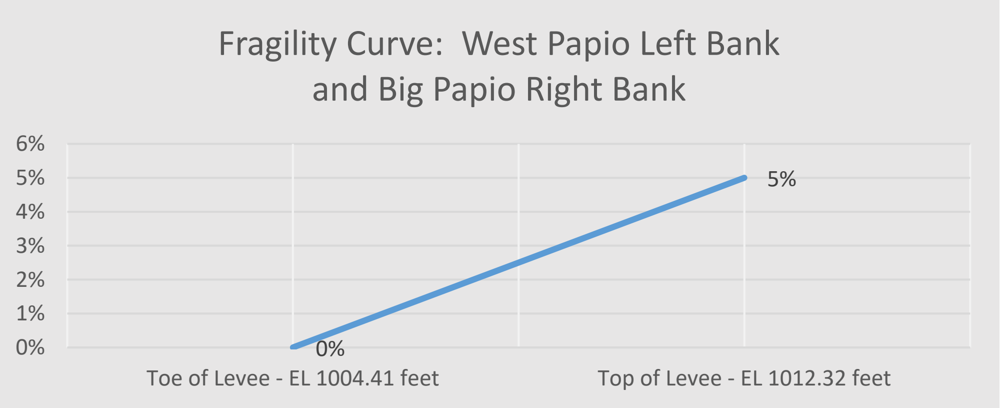




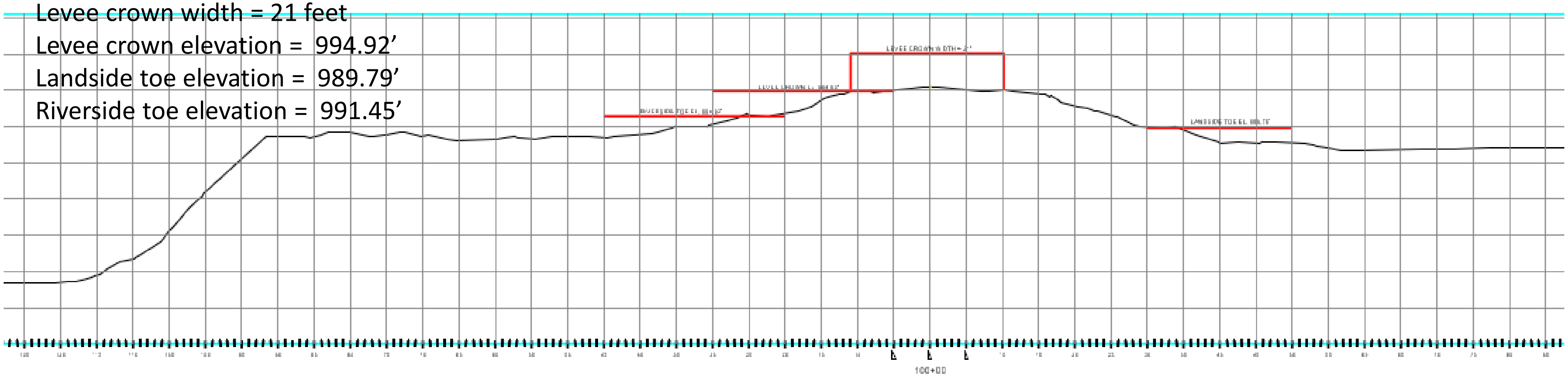
Big Papio Right Bank

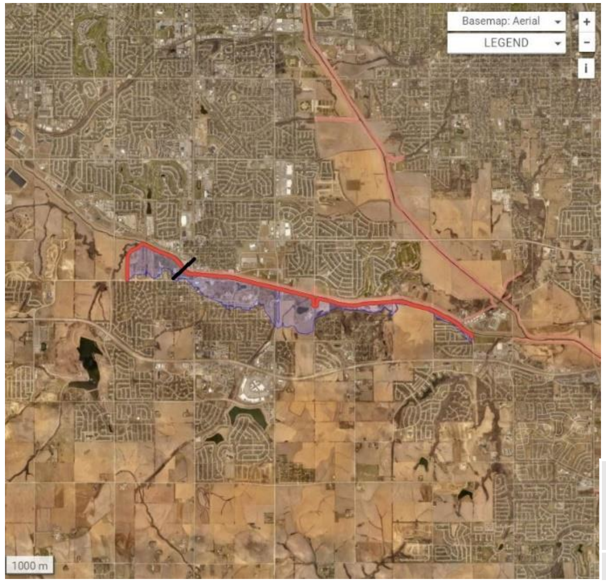
Big Papio RB - Cross-section taken at approximately 10,000 ft = 100+00
Big Papio segment length is approximately 20,200 ft = 3.83 miles

Description	minimum credible value	maximum credible value	St. Dev σ P_f	P_f
Baseline Earth Embankment Fragility				
Toe of Levee	0	0	0	0
Top of Levee	0	0.15	0.025	0.075
West Papio Left Bank and Big Papio Right Bank				
Adjustment for Toe of Levee	0	0	0	0
Adjustment for Top of Levee (crown width)	0	-0.05	0.0000	0
Toe of Levee - EL 1004.41 feet	0%	0%	0%	0%
Top of Levee - EL 1012.32 feet	0%	10%	2.50%	5%



Big Papio RB @ Station 100+00
Levee crown width = 21 feet
Levee crown elevation = 994.92’
Landside toe elevation = 989.79’
Riverside toe elevation = 991.45’

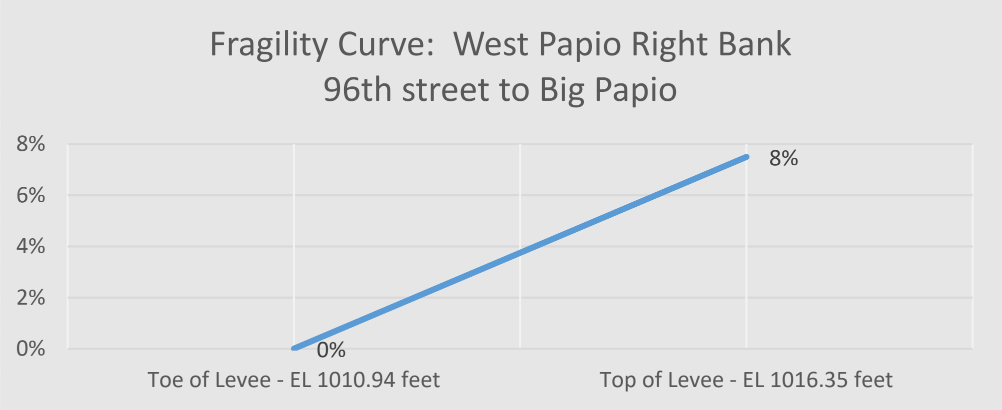


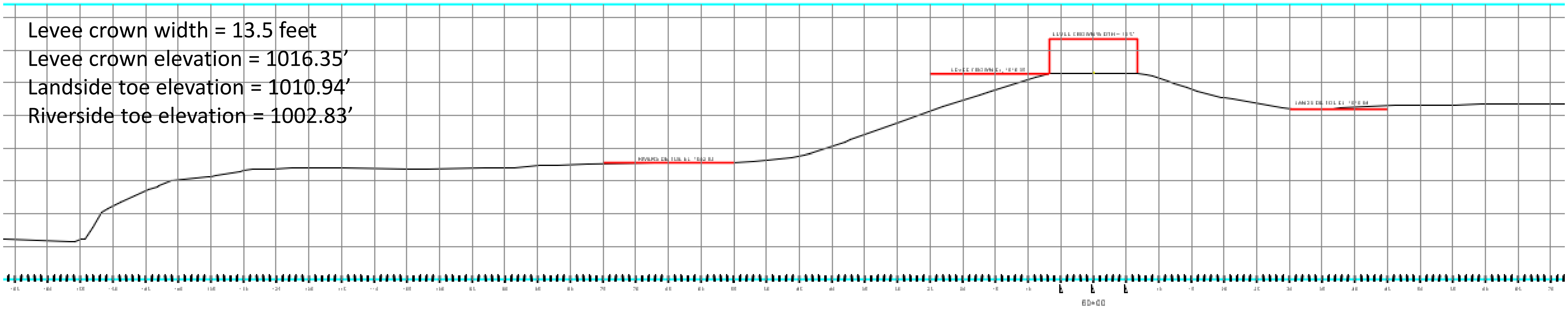


West Papio Right Bank – 96th street to Big Papio

Cross-section taken at approximately 6000 ft = 60+00
Total length is approximately 27,200 ft = 5.15 miles

Description	minimum credible value	maximum credible value	St. Dev σ P_f	P_f
Baseline Earth Embankment Fragility				
Toe of Levee	0	0	0	0
Top of Levee	0	0.15	0.025	0.075
West Papio Right Bank – 96th street to Big Papio				
Adjustment for Toe of Levee	0	0	0	0
Adjustment for Top of Levee	0	0	0.0000	0
Toe of Levee - EL 1010.94 feet	0%	0%	0%	0%
Top of Levee - EL 1016.35 feet	0%	15%	2.50%	8%





Extreme Values

Standard deviation and most likely value

Adjustment (typ)

Cross Section Name

Vicinity Map

Fragility Curve

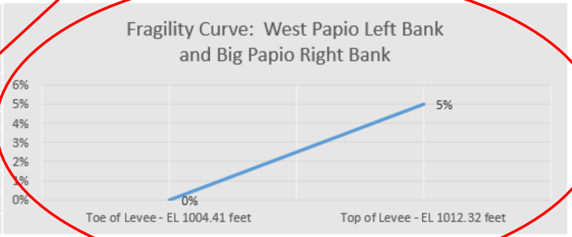
Cross Section

Elevation Data and Crown Width

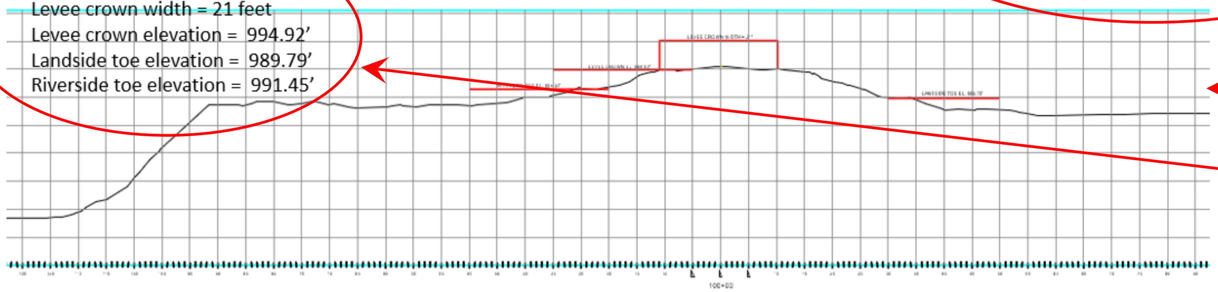
Big Papio Right Bank

Big Papio RB - Cross-section taken at approximately 10,000 ft = 100+00
Big Papio segment length is approximately 20,200 ft = 3.83 miles

Description	minimum credible value	maximum credible value	St. Dev σ Pr	Pr
Baseline Earth Embankment Fragility				
Toe of Levee	0	0	0	0
Top of Levee	0	0.15	0.025	0.075
West Papio Left Bank and Big Papio Right Bank				
Adjustment for Toe of Levee	0	0	0	0
Adjustment for Top of Levee (crown width)	0	-0.05	0.0000	0
Toe of Levee - EL 1004.41 feet	0%	0%	0%	0%
Top of Levee - EL 1012.32 feet	0%	10%	2.50%	5%



Big Papio RB @ Station 100+00
Levee crown width = 21 feet
Levee crown elevation = 994.92’
Landside toe elevation = 989.79’
Riverside toe elevation = 991.45’



Little Papio Right Bank and Big Papio Left Bank

Little Papio RB - Cross-section taken at approximately 6000 ft = 60+00

Big Papio LB – Cross-section taken at approximately 1000 ft = 10+00

Little Papio segment length is approximately 2,000 ft = 0.39 miles

Big Papio segment length is approximately 4,500 ft = 0.86 miles

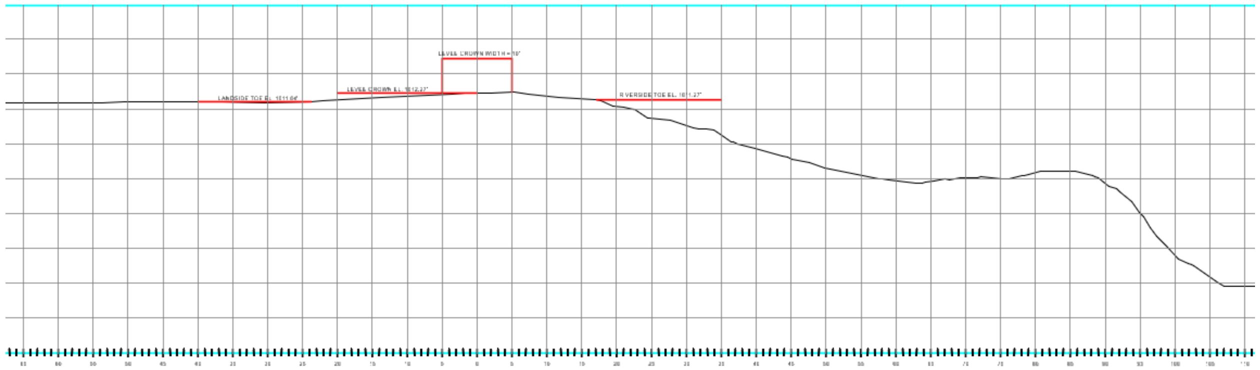
Big Papio LB @ Station 10+00

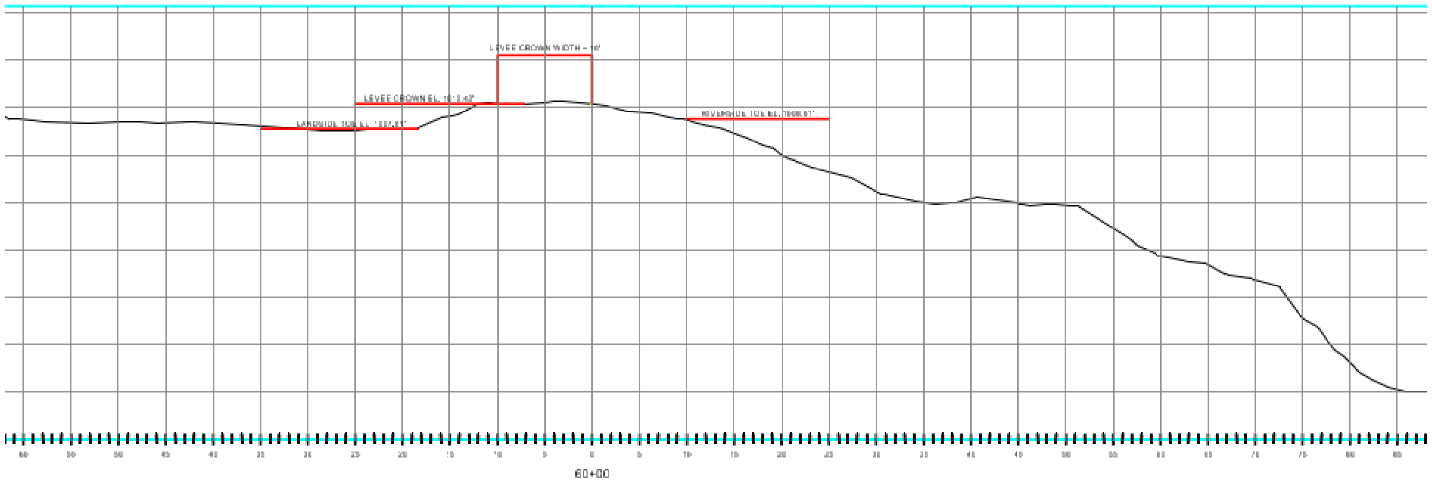
Levee crown width = 10 feet

Levee crown elevation = 1012.27’

Landside toe elevation = 1011.04’

Riverside toe elevation = 1011.27’





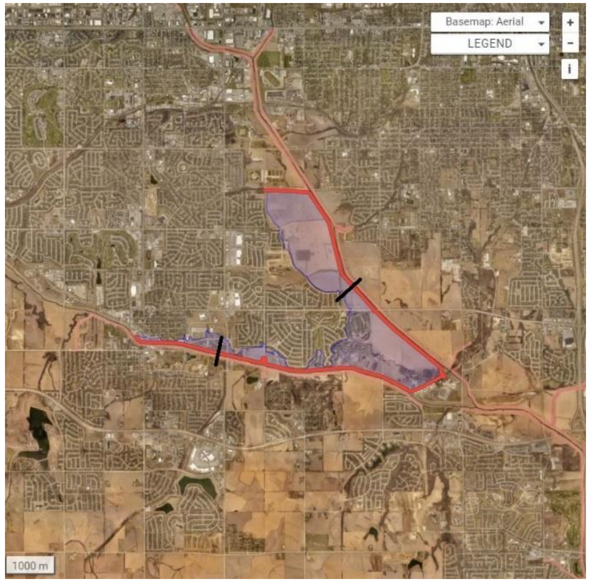
Little Papio RB @ Station 60+00

Levee crown width = 10 feet

Levee crown elevation = 1010.43’

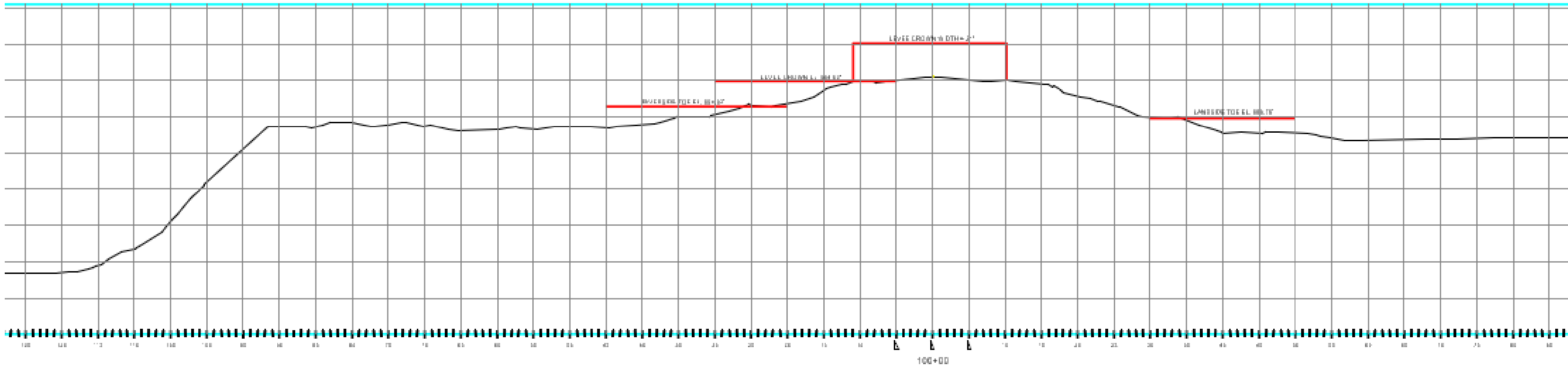
Landside toe elevation = 1007.81’

Riverside toe elevation = 1008.81’



West Papio Left Bank and Big Papio Right Bank
Big Papio RB - Cross-section taken at approximately 10,000 ft = 100+00
West Papio LB – Cross-section taken at approximately 35,000 ft = 350+00
Big Papio segment length is approximately 20,200 ft = 3.83 miles
West Papio segment length is approximately 20,275 ft = 3.84 miles

Big Papio RB @ Station 100+00
Levee crown width = 21 feet
Levee crown elevation = 994.92’
Landside toe elevation = 989.79’
Riverside toe elevation = 991.45’



West Papio LB @ Station 350+00
Levee crown width = 12 feet
Levee crown elevation = 1012.32’
Landside toe elevation = 1004.41’
Riverside toe elevation = 995.55’